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COMITÉ FRANÇAIS DE MÉCANIQUE DES SOLS ET DE GÉOTECHNIQUE

Workgroup « Foundations of Offshore Wind Turbines »

RECOMMENDATIONS FOR PLANNING AND DESIGNING FOUNDATIONS OF OFFSHORE WIND TURBINES

VERSION 2.1

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FOREWORD

In 2012, under the auspices of the French Committee for Soils Mechanics and Geotechnics, the « Recommendations on the design, calculation, implementation and control of wind turbines foundations » have been published. They are dedicated to onshore wind turbines.

In 2013, the CFMS took the decision to create a new Working Group, to address the geotechnical aspects of the foundations of offshore wind turbines. The present document, finalised in 2018 in French language, is the outcome of this project.

These « Recommendations for planning and designing foundations of offshore wind turbines » aim at assisting the various designers by mitigating the absence of normative documents or national regulatory texts regarding the design and installation of the foundations of offshore structures in the French territorial waters.

Patrick Berthelot
President of the Working Group

1 INTRODUCTION

1.1 OBJECTIVES AND SCOPE OF APPLICATION

1.2 DEFINITION AND ROLE OF THE GEOTECHNICAL ENGINEER

1.3 TYPES OF SUPPORT STRUCTURE

1. INTRODUCTION

1.1. OBJECTIVES AND SCOPE OF APPLICATION

The present recommendations address the planning and designing of the foundations of fixed offshore wind turbines founded on :

- monopiles;
- piles;
- gravity bases.

They cover the site investigations of the seabed and of the marine soils required for planning and designing the foundations of wind turbines, the foundations of sub-stations and meteorological masts as well as the design of cable routes interconnecting the wind turbines and connecting the wind farm to the shore.

The design of the foundations of sub-stations and meteorological masts, as well as the anchoring of floating wind turbines, are not dealt with in the present issue.

These recommendations do not address the mechanical or structural components of a wind turbine, such as the nacelle, the rotor, the generator, the gearbox, the rotor blades, the mast (or tower) or the sub-structure. The definition of the structure and sub-structure is illustrated in Figure 1.1 for the various types of foundations. However, it should be noted that, within the framework of the ground-structure interaction, the mast (or tower) and the sub-structures elements should be taken into account at the very first steps of the design process.

The document aims at being used as a technical reference during the processes of planning, designing, building and installing the foundations, for all aspects pertaining to geotechnics and ground-structure interactions.

It may be subject to updates, notably resulting from feedback or evolutions of knowledge in the field of offshore engineering.

The present recommendations are compatible with, and complementary to, the international standards relative to offshore wind turbines IEC 61400 and DNVGL, listed in paragraph 2.1.

These recommendations also allow:

- covering specific soils met on the French continental shelf (chalks, marls, calcarenites...);
- addressing the case of drilled piles adequate for these types of soils with a greater degree of detail, and bridging a gap between the offshore design practice and the French know-how of the onshore sector;
- introducing outputs from the SOLCYP project about the dimensioning of piles under cyclic loadings.

Offshore wind turbines represent building projects having a high degree of geotechnical issues. Ground conditions play a major role. But geotechnics alone cannot govern on its own the choice of the foundation system, which highly depends on other technical factors, such as the conditions of installation, manufacturing, transportation..., as well as considerations of economy or land-use planning.

1.2. DEFINITION AND ROLE OF THE GEOTECHNICAL ENGINEER

The geotechnical engineer is a natural or legal person that carries out services of geotechnical investigations and/or geotechnical engineering.

At each successive step of study and implementation of the project, a qualified geotechnical engineer shall be appointed. It is notably the case for the scheduling and carrying out of geotechnical investigations (chapter 5) and for the planning and dimensioning of foundations (chapters 8, 9 and 10).

Geotechnical engineering has to be associated to other types of engineering relative to the design, construction and installation of the structure. It contributes to the efficient management of geological risks, in order to maximise the reliability of the scheduling, costs and quality of the geotechnical structures comprised in the project.

In particular, the geotechnical engineer :

- schedules and pilots geotechnical investigations, assesses the results, drafts the geotechnical profiles including data on soils types and soil layering (simplified ground model), determines the characteristics of soils and their variability, and draws the necessary conclusions relative to the design of the structure and its foundations;
- advises the developer and the design engineer throughout the elaboration of appropriate ground-structure interaction models;
- carries out geotechnical calculations on behalf of the design engineer. Unless there are particular specifications, the design engineer endorses the overall project responsability;
- provides advices to the developer and design engineer for the instrumentation and follow-up during building stages, for periodic inspections, and, within the framework of the application of the observational method, for the definition of the geotechnical observations programme, its assessment and the drafting of the measures to be taken.

1.3. TYPES OF SUPPORT STRUCTURE

The support structures of an offshore wind turbine comprise, from top to bottom (Figure 1.1):

- the tower, or mast, bearing the rotor-nacelle assembly, ending on the flange level (transition piece with the sub-structure);
- the sub-structure, located between the flange and the seabed;
- the foundation laid on or anchored in the seabed.

The tower is an element that is shared by all types of wind turbines and is not addressed in the present recommendations. There may be various types of sub-structures and foundations. Choosing the foundations and sub-structures is a multifactor process that takes into account technical (sites conditions, installations methods, type of turbine), environmental (water height, sea conditions, ground conditions) and socioeconomic issues.

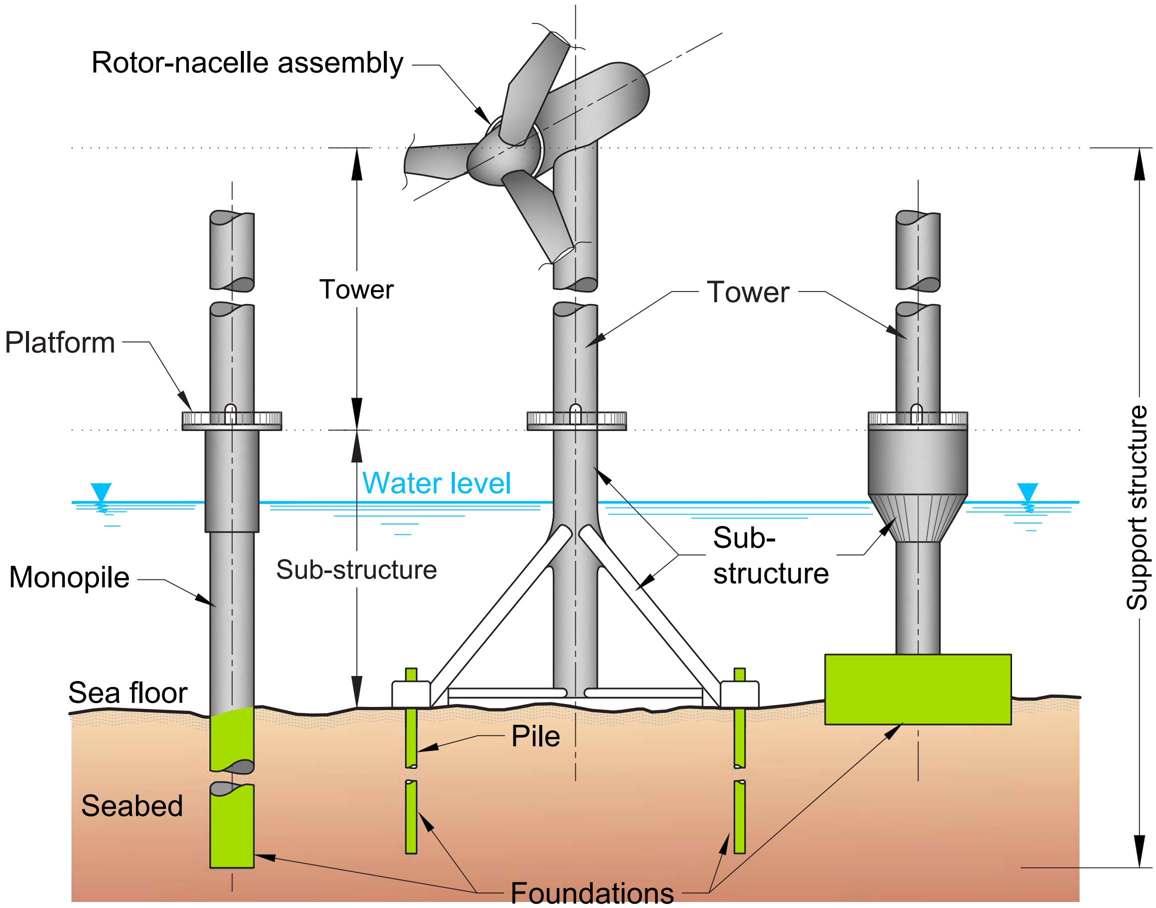

Figure 1.1 : Components of an offshore wind turbine

Note : In some sources, definitions may differ (for instance, the term « foundation » may become a misnomer and describe the elements set under the wind turbine mast, therefore including the sub-structure and the foundation).

1.3.1. TYPES OF SUB-STRUCTURES

Sub-structures can be classified into several generic types, which are mainly :

- the monopod;
- the tripod;
- the jacket;
- the gravity base;
- the floating structure (not addressed in this document).

Sub-structures classified as hybrid ones may be used by combining the various configurations above.

1.3.2. TYPES OF FOUNDATIONS

Foundations of offshore wind turbines can be classified into several generic types, which are mainly:

- shallow or gravity base foundation, with or without skirts;
- semi-shallow foundation, or caisson foundation;
- single pile foundation (monopile);
- multi-piled foundation;
- anchoring to the seabed.

The present recommendations only address fixed wind turbines foundations, with the most commonly used types currently being:

- monopiles: they represent the type of foundation with the most widespread use in offshore wind farms. As of today, monopiles diameter are usually larger than 5m and may exceed 8m. Their slenderness ratio is low (D/B < 5). They are addressed in the chapter 8.
- piles: they are usually metallic tubes with diameters ranging from 1.5m to 3m with a high slenderness ratio (D/B>10). They are most often driven into the ground. They are also used as foundations for structures bearing ancillary equipments (sub-stations, meteorological mast...). They are addressed in the chapter 9.
- gravity base foundations: they are shallow foundations, usually made of reinforced or pre-stressed concrete, for which stability is maintained by their own weight. Their diameter lies typically between 20m to 35m. They are laid on a base course or slightly buried. They may be equipped with relatively shallow skirts. They are addressed in the chapter 10.

2 REFERENCES

2.1 STANDARDS

2.2 PROFESSIONAL RECOMMENDATIONS

2.3 OTHER TECHNICAL DOCUMENTS

2. REFERENCES

2.1. STANDARDS

The main standards pertaining to the use of the present document are listed below:

- BSH (2007) Standard : *Design of Offshore Wind Turbines*, Bundesamt für Seeschifffahrt und Hydrographie, Hamburg
- BSH (2008) Standard 7004 - *Ground investigations for offshore windfarms*, Bundesamt für Seeschifffahrt und Hydrographie, Hamburg
- BSH (2011) *Guidance for use of the BSH standard "Design of offshore wind turbines*" Bundesamt für Seeschifffahrt und Hydrographie, Hamburg
- DNVGL-RP-C212 (2017) *Offshore soil mechanics and geotechnical engineering*
- DNVGL-ST-0126 (2016) *Support structures for wind turbines*
- DNVGL-ST-0145 (2016) *Offshore substations*
- DNVGL-ST- 0437 (2016) *Loads and site conditions for wind turbines*
- IEC 61400-1 (2005), Wind turbine generator systems – Part 1: *Safety requirements*
- IEC 61400-3 (2009), Wind turbine generator systems – Part 3: *Design requirements for offshore wind turbines*
- IEC 61400-6 (draft 2016, official document to be published soon) Wind turbines – Part 6 : *Tower and foundation design requirements*
- ISO 19901-1 (2015) Petroleum and natural gas industries – Specific requirement for offshore structures – Part 1 : *Metocean design and operating considerations*
- ISO 19901-4 (2016) Petroleum and natural gas industries – Specific requirement for offshore structures – Part 4 : *Geotechnical and foundations design considerations*
- ISO 19901-8 (2014) Petroleum and natural gas industries – Specific requirement for offshore structures – Part 8 : *Marine Soil Investigations*
- NF P 94-262 (2012) National application standard of the Eurocode 7 – *Fondations profondes.*

The « Marine Soil Investigations » standard (ISO 19901-8, 2014), even though initially drafted for the offshore oil industry, remains to a large extent, and notably for its technical considerations, applicable to the offshore wind farms sector. In particular, one may find a description of the best practices relative to:

- planning and carrying out campaigns of geotechnical reconnaissances;
- implementing drilling and coring systems, as well as in-situ tests;
- acquiring, transporting and storing soil samples;
- carrying out common or advanced laboratory tests on samples, including cyclic or dynamic tests;
- presenting the results and drafting reports.

2.2. PROFESSIONAL RECOMMENDATIONS

- ARGEMA (1988) *Pieux dans les formations carbonatées.* Practical guide for offshore structures. Editions Technip, Paris
- ARGEMA - CLAROM (1994) *Foundations in carbonate soils.* Design guides for offshore structures. Editions Technip, Paris
- CIRIA (2002) *Engineering in chalk*, C574
- CIRIA (2004) *Piles foundations in weak rock*, Report 181
- SOLCYP (2017) *Recommandations pour le dimensionnement des pieux sous chargements cycliques*. Projet National SOLCYP, ISTE éditions
- SOLCYP (2017) *Design of piles under cyclic loading - SOLCYP recommendations*, Ed. ISTE & WILEY
- SUT (2014) *Guidance notes for the planning and execution of geophysical and geotechnical ground investigations for offshore renewable developments*. Offshore Site Investigation and Geotechnics Committee (OSIG), London

2.3. OTHER TECHNICAL DOCUMENTS

- ISSMGE Technical Committee TC1 (2005), *Geotechnical and geophysical investigations for offshore and nearshore developments*, September 2005
- ISSMGE Technical Committee TC209 (2013) *Design for cyclic loading: piles and other foundations*. Proceedings of TC209 workshop, 18th ICSMGE Paris 2013. Edited by A. Puech/IREX

The "*Geotechnical and geophysical investigations for offshore and nearshore developments*" manual, issued by the Technical Committee TC1 of the International Society for Soils Mechanics and Geotechnical Engineering (ISSMGE) contains:

- recommendations on how to plan offshore campaigns of geophysical and geotechnical reconnaissances;
- a description of required naval means: drilling vessels, jack-up rigs...;
- a description of deployment modes of equipments;
- a description of applicable geophysical methods, and notably: high resolution seismic reflection (multibeam echo-sounder), LiDAR, side scan sonar (SSS), penetrators (pingers, boomers, sparkers), seismic refraction dragged on, or close to, the seabed, electrical resistivity;
- a description of methods for drilling, coring and in-situ measurements: from a floating support or with a system laid on the bottom;
- a description of systems and methods of sampling: gravity corer, vibrocorer, push or rotary sampling;
- a description of in-situ testing methods (CPT, CPTU, seismic cone (SCPT), vane shear test (VST), T-bar and Ball probe);
- a review of the various parameters required for the dimensioning of the foundations of offshore structures, notably platforms set on piles, gravity platforms, floating structures and pipelines;

- recommendations relative to how applicable are the various geophysical methods, the in-situ measurements and laboratory measurements on samples to obtain the geotechnical parameters required for the dimensioning;
- recommendations to carry out geophysical and geotechnical campaigns dedicated to specific structures, notably piled platforms, gravity base platforms, floating structures and pipelines.

3 SITE CONDITIONS

3.1 INTRODUCTION

3.2 ENVIRONMENTAL CONDITIONS

3.3 GROUND CONDITIONS

3.4 REFERENCES

3. SITE CONDITIONS

3.1. INTRODUCTION

Offshore wind turbines in open sea are subject to environmental conditions that may affect their load, durability and operability. In order to maintain an appropriate level of safety and reliability, parameters related to the environment and the ground should be taken into account during the design process and should be explicitly stated in the design documents.

Data characterising these elements are provided by the developer.

Environmental conditions are essential when determining the loads that can be borne by the wind turbine. They are subdivided into:

- wind conditions;
- marine conditions: waves, swell, ocean currents, sea levels, sea ice, biofouling, seabed slides and scour;
- other environmental conditions: air temperature and humidity, water temperature, salinity, seismicity...

The effects of the environmental phenomena and their interactions are specific to the site.

Acquiring knowledge of the metocean parameters usually requires, in addition to desk studies, collecting enough in situ data to obtain distribution probabilities that can be exploited to analyse these phenomena, whether they are separated or combined.

The various values of the metocean parameters to be examined during the process of dimensioning structures (normal, severe, extreme, accidental conditions) are defined in the IEC 61400-3 (2009) standard.

The measurements and studies required to assess the environmental conditions, to the exception of ground conditions, are defined in the ISO 19901-1 (2015) standard.

Ground conditions are critical for the design process of foundations and the determination of the structure stability. The ground characteristics of the site should be subject to specific investigations. This aspect is addressed in chapter 5 of the present document.

Ground conditions influence the design process of the wind turbine through the interaction between ground and structure.

Ground and environmental conditions may interfere, for instance in the case of seabed slides or instabilities, and of scour.

3.2. ENVIRONMENTAL CONDITIONS

3.2.1. WIND CONDITIONS

An offshore wind turbine in open sea shall be designed so it can withstand the wind conditions determined during design.

Wind regimes, in terms of loads and safety, are subdivided into normal wind conditions, which occur more frequently than once per year during normal operation of an offshore wind turbine, and into extreme wind conditions, which are defined as having a 1-year or 50-year recurrence period.

The design of a support structure of an offshore wind turbine in open sea shall be based on wind conditions that are representative of the wind farm site. They shall be assessed in compliance with IEC 61400-3 (2009), clauses 6 and 12 and/or with DNVGL-ST-0437 (2016), sections 2 and 3.

In the case of a rotor-nacelle assembly, the wind conditions assessed for the design may be specific to the site, or be defined by models and parameters values, as specified by IEC 61400-1(2005), clause 6 and/or DNVGL-ST-0437 (2016), sections 2 and 3.

3.2.2. MARINE CONDITIONS

An offshore wind turbine in open sea shall be designed so it can withstand the marine conditions selected as a design basis. Marine conditions described in the present paragraph include waves, swell, ocean currents, sea levels, sea ice, marine growth, seabed movements and scour. Some other external conditions relative to the marine environment are defined in the paragraph 3.2.3 of the present document.

Marine conditions, in terms of loads and safety, are subdivided into normal marine conditions, which occur more frequently than once per year during normal operation of an offshore wind turbine, and into extreme wind conditions, which are defined as having a 1-year or 50-year recurrence period.

The design of a support structure of an offshore wind turbine in open sea shall be based on the environmental conditions, including marine ones, that are representative of the wind farm site. They shall be assessed in compliance with IEC 61400-3 (2009), clauses 6 and 12 and/or with DNVGL-ST-0437 (2016), sections 2 and 3.

The designer shall also consider how marine conditions will affect the rotor-nacelle assembly.

3.2.3. OTHER CONDITIONS

Environmental conditions (climate) other than wind and marine conditions may affect the integrity and safety of an offshore wind turbine in open sea, because they may have thermal, photochemical, corrosive, mechanical, electrical effects, or because of some other physical effect. Furthermore, weather parameters may combine with each other and increase the impact of the previous effects.

The following environmental conditions should also be taken into account, and the consequent action to be taken should be stated in the design documents:

- air temperature, air density, humidity;
- solar radiation;
- rain, hail, snow, ice and frost;
- active chemical substances;
- mechanically active particles;
- salinity;

- seismic activity;
- water density and temperature;
- maritime traffic.

Weather conditions that have been taken into account should be defined in terms of representative values, or as limits of variable conditions. The simultaneous occurrence probability of weather conditions should be defined during the design process.

Variations of weather conditions, within the range of normal limits, i.e. having a recurrence period of one year or shorter, shall not interfere with the normal operating mode for which the open sea wind turbine was designed.

Taking into account these other environmental conditions and how they may combine with marine and wind conditions shall be assessed in compliance with IEC 61400-3 (2009), clauses 6 and 12 and/or DNVGL-ST-0437 (2016), sections 2 and 3.

3.3. GROUND CONDITIONS

3.3.1. CHARACTERISATION

Assessing ground conditions is required to plan and design foundations. These ground conditions include:

- the seabed surface: slope, unevenness, escarpments, rocks, coral, channels, sandbanks, dunes, former dredging and discharge areas...;
- the deep soils: lithological nature, mechanical properties, variability caused by depositional modes (channels, reefs, discontinuities...), by heterogeneities (indurated crusts, blocks, anisotropy...) or by alteration phenomena.

One should note that surface conditions may slowly evolve or be brutally modified by various natural phenomena.

If needed, modifications of the ground conditions caused by field preparations or by the installation of temporary structures for the main works should be considered.

Reconnaissances required to acquire knowledge about the ground conditions for each stage of the project are described in the chapter 5 of the present document. Procedures for the definition of the ground parameters required for the design of the foundation, and of the ground-structure interactions, are described in the chapter 6 of the present document.

3.3.2. SEABED INSTABILITY

Seabed instability may affect soils locally or over large areas.

Instabilities (slides) are triggered by various factors (swell, current, quake, liquefaction, gaseous soils, faults) and may have consequences over large distances, either downstream (turbidity) or upstream (retrogression). Even a slope of a few degrees may be critical in zones of rapid sedimentation and with poorly consolidated soils.

Since seabed instability, either downstream or upstream in regard to the site, may affect a structure, it is essential that reconnaissances and studies cover a sufficiently large area around the structures.

3.3.3. SCOUR AND SEDIMENTS MOBILITY

The presence of structures laid over the seabed leads to a disruption of currents and may result in scour around the foundations, down to significant depths. Anti-scour protection systems may, if needed, be used so that the ground around the foundation can be safeguarded.

Chapter 11 in the present document describes scour phenomena, their consequences on the stability of structures and practicable protection systems.

Under the effect of currents, swell and tides, some zones are subject to the erosion of the existing soils or to the deposit of new sediments. These deposits (sandbanks, dunes or seabed ripples) may be temporary or long-lasting.

The knowledge of the nature of the soil, of marine currents, of swell and of tides is critical when determining the movements of sediments and the scour, as well as when designing protective measures.

3.3.4. OTHER GROUND RELATED HAZARDS

Various hazards may be met depending on the geological context: gaseous soils, karsts, faults, excess pore pressures, old slides...

These hazards should be identified in the early stages of the design process since they may prove to be critical when defining the master plan (location of turbines) of the wind farm and when determining the foundation concept.

3.4. REFERENCES

DNVGL-ST- 0437 (2016) *Loads and site conditions for wind turbines*

ISO 19901-1 (2015) Petroleum and natural gas industries – Specific requirement for offshore structures – Part 1 : *Metocean design and operating considerations*

IEC 61400-1 (2005) Wind turbine generator systems – Part 1: *Safety requirements*

IEC 61400-3 (2009) Wind turbine generator systems – Part 3: *Design requirements for offshore wind turbines*

4 LOADS AND LOAD CASES

4.1 DEFINITION OF LOADS

4.2 PROCESSING OF CYCLIC LOADINGS

4.3 TAKING ACCOUNT OF THE EFFECT OF LOADS : SOIL-STRUCTURE INTERACTION

4.4 REFERENCES

4. LOADS AND LOAD CASES

The following paragraphs describe the types of loads resulting from the environmental conditions, in combination with the operating behaviour of the turbine, and provide explanations regarding the methodology to be followed when determining load cases relative to various conceptual situations under IEC 61400-3 (2009). Only the case of fixed wind turbines equipped with 3 blades and a horizontal axis is considered.

4.1. DEFINITION OF LOADS

4.1.1. PERMANENT LOADS G

Permanent loads are loads that do not vary in terms of intensity, position or direction during the considered period.

As an example of permanent load, one may mention the self-weight of the structure, the weight of equipments (including the tower, nacelle and rotor), the permanent ballasts, the weight of the possible anti-scour systems, marine concretions, internal and external permanent hydrostatic pressures, and the vertical hydrostatic force between the sea floor and the mean sea level.

4.1.2. OPERATING LOADS Q

Operating loads are loads that may vary in terms of intensity, position or direction during the considered period and that are relative to the normal operation of the structure.

As an example of operating loads, one may mention berthing and mooring loads, as well as temporary maintenance loads (staff, equipment).

4.1.3. VARIABLE AND ENVIRONMENTAL LOADS E

Environmental loads are loads that may vary in terms of intensity, position or direction during the considered period and that are relative to environmental changes, including variable loads directly related to the functioning of the turbine.

As an example of operating loads, one may mention:

- wind (including wind blowing on the blades, nacelle and mast);
- ice and snow;
- current;
- swell and waves;
- hydrostatic force due to the tide (trough to crest variations);
- forces due to sedimentary deposits;
- forces due to sea ice.

4.1.4. ACCIDENTAL LOADS A

Accidental loads are loads that are not related to the normal functioning of the turbine, and that originate from abnormal operations or from technical failures.

As an example of accidental loads, one may mention collisions with the structure, whether they are caused by internal or external elements, as well as the effects from explosions or from fire.

The effects of an accidental modification of the stabilising load (e.g. ballast) belong to this category of loads.

4.1.5. SEISMIC LOADS S

Seismic or earthquake loads include the effects applied directly by the ground to the structure and the hydrodynamic forces resulting from the oscillations of the structure in the water.

Seismic loads are not addressed in this document.

4.1.6. DEFORMATION LOADS D

Deformation loads are loads generated by inflicted deformations.

Deformation loads may, for instance, be generated by temperature variations or foundation settlements.

4.2. PROCESSING OF CYCLIC LOADINGS

4.2.1. CHARACTERISATION OF CYCLIC LOADINGS

4.2.1.1. REGULAR LOADINGS - DEFINITIONS

In the ideal case of cyclic loadings with a constant amplitude and a constant period (called regular loading), it is easy to characterise the loading by means of the following values (Figure 4.1):

- Q_m : mean load or mean component of the cyclic load;
- Q_c : cyclic component or half-amplitude of the cyclic load;
- Q_{min} : minimum cyclic load ($Q_{min} = Q_m – Q_c$)
- Q_{max} : maximum cyclic load ($Q_{max}= Q_m + Q_c$)
- T : period of cycles (T = 1/f with f = frequency of cycles);
- N : number of cycles.

One may distinguish:

- one-way loadings, for which $Q_c < Q_m$;
- two-way loadings, for which $Q_c > Q_m$.

The script Q is used for undifferentiated loads. For lateral (horizontal) loads, it will be replaced by the script H, and for vertical (axial) loads by the script V.

4.2.1.2. CYCLIC LOADING OF SOIL SAMPLES IN THE LABORATORY

One can immediately see the analogy with the definition of the parameters of cyclic loadings of samples subject to series of cycles in the laboratory (the load Q and the applied shear stress τ having identical roles). Cyclic tests in the laboratory show that the response of a soil sample can be characterised by:

- the mean shear stress τ_m and the half-amplitude of the cyclic shear stress τ_{cy} : these two parameters affect differently the evolution of permanent and cyclic strains;

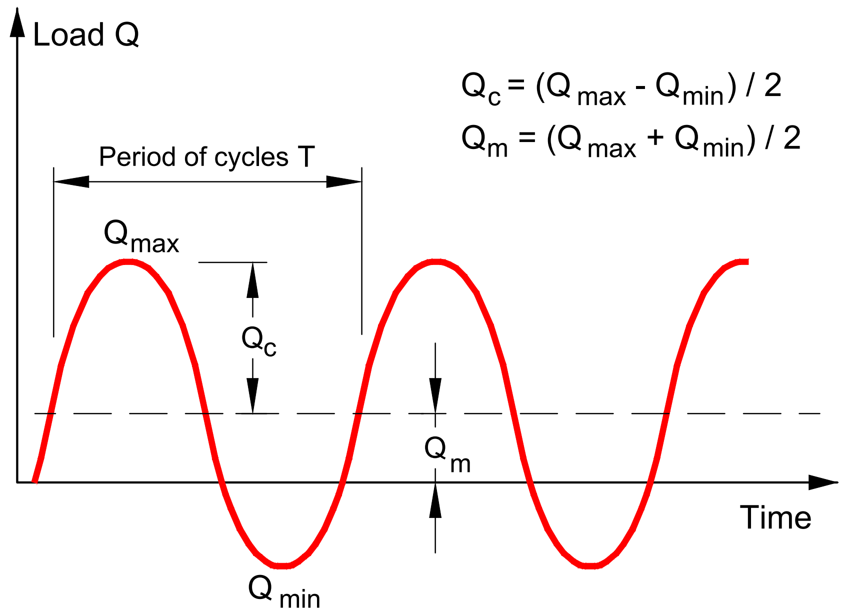

Figure 4.1 : Definitions for a regular cyclic loading

- the drainage conditions imposed to the sample: fully drained or fully undrained;
- the loading frequency f (or the period T);
- the loading rate: this parameter affects directly the undrained shear strength of clays;
- the number of cycles N: the number of cycles characterising a cyclic event may vary from a few cycles to several thousand or even millions of cycles.

4.2.1.3. REAL CYCLIC LOADING

Figure 4.2 and Figure 4.3 display examples of calculated cyclic loadings transmitted by offshore wind turbines to their foundations. The example of Figure 4.2 pertains to an operational loading dominated by wind conditions. Figure 4.3 relates to an extreme loading dominated by swell. The sequences simulated in both examples are of short durations (200s and 100s, respectively).

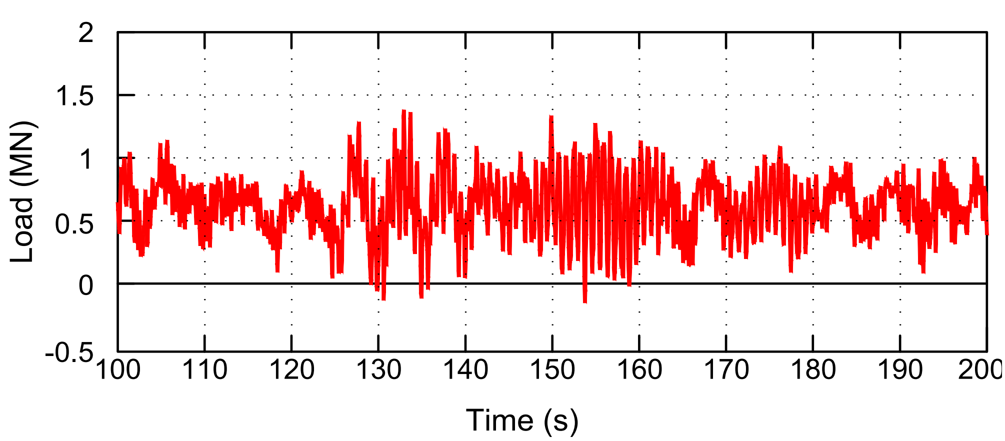

Figure 4.2 : Example of loading induced at the top of a monopile by a wind turbine in operation (calculated)

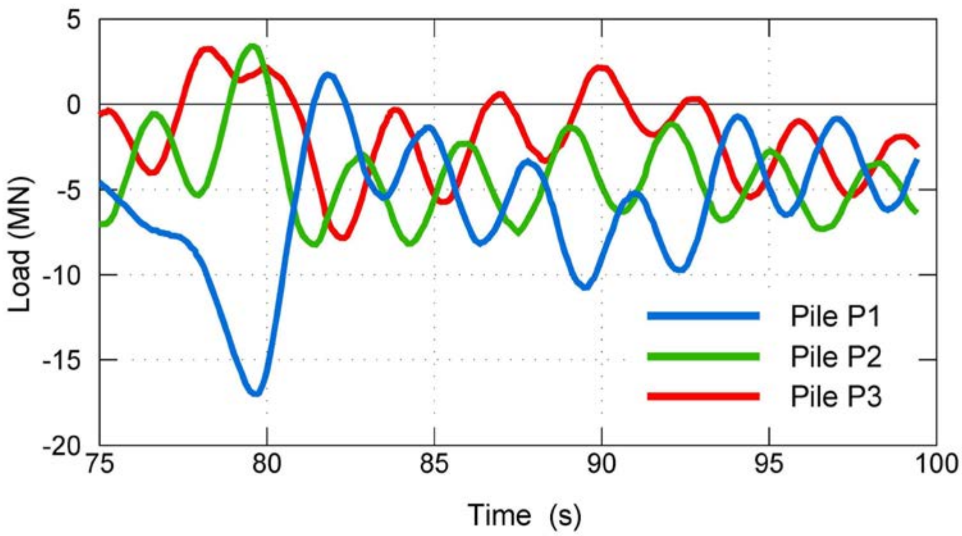

Figure 4.3 : Extreme axial loads transmitted to the head of the three foundation piles of an offshore wind turbine (tripod structure) - Waves with a return period of 50 years

4.2.2. TAKING ACCOUNT OF REAL CYCLIC LOADING IN THE DESIGN PROCESS

The histograms of the loads transferred to the foundation are most often composed of series of loads with irregular amplitudes and showing a relatively random distribution over time (Figure 4.2 and Figure 4.3). In contrast, laboratory tests aiming at capturing the phenomenology of soils under cyclic loadings are usually carried out on series of regular cycles (amplitude and frequency maintained constant throughout the series).

During the design stage, an essential step consists then in converting the true random load into a regular, idealised, one. Cycle counting methods, which are derived from so-called "rainflow" analyses, are widely used, notably in the field of metal fatigue analysis, to turn histograms of real loads into idealised series of cycles having constant amplitude and frequency (*e.g.* ASTM E 1049-85 and NF A03-406). More thorough information regarding this aspect may be found in the SOLCYP recommendations (2017).

Then, Miner's cumulative damage concept is applied (Downing and Socie, 1982) to obtain equivalent cyclic loadings from the fatigue curves (such as Wölher curves, often called S-N curves), determined by representing the number of cycles to failure of samples subject to series of constant amplitude stress cycles. The SOLCYP (2017) recommendations feature a discussion as to whether Miner's hypothesis is valid in the case of soils (independency of the order of application of cycles series and of frequency).

In order to avoid any confusion regarding the wording, an « idealised » loading will be used to define loading in series of constant amplitude cycles that are determined by applying a counting method on the real loading, and an « equivalent » loading will be used to define a loading that generates the same damage on the material as the real loading. In the process of foundation design, taking into account cyclic loadings combines both steps, as outlined in Figure 4.4:

- step 1: turning the random loading into an idealised loading sequence using a counting method;
- step 2: determining an equivalent loading from the idealised one by using a damage law according to Miner's concept.

The methods to obtain equivalent cyclic loadings (step 2 in Figure 4.4) will be addressed in the chapters relative to the various types of foundations.

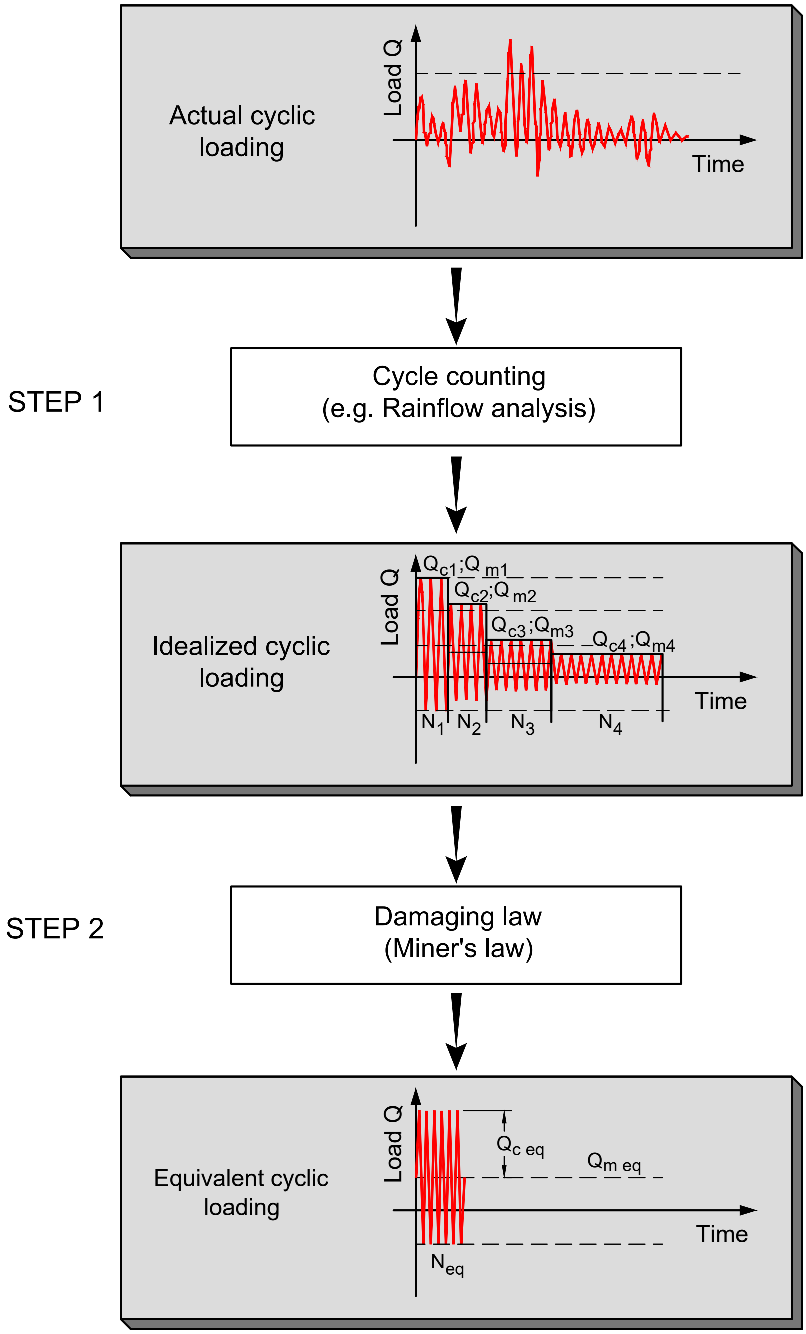

Figure 4.4 : Taking account of cyclic loading in the design process of foundations
(from SOLCYP recommendations, 2017)

4.3. TAKING ACCOUNT OF THE EFFECT OF LOADS : SOIL-STRUCTURE INTERACTION

4.3.1. INTRODUCTION

Calculating the loads and load cases for an offshore wind turbine depends on the global dynamics of the structure, and of the interactions between the ground and the structure. Taking into account this coupling is essential during all stages of the project, including during preliminary steps.

In the following paragraphs, the calculation methods used for designing the main elements of the offshore wind turbines will be addressed: rotor + nacelle, tower (or mast), sub-structure and foundation. The whole set of elements form the structure (= wind turbine). These definitions are illustrated in Figure 1.1 of paragraph 1.3.

Ground modelling is one of the key tasks that influences the design of wind turbine structures. Foundations stiffnesses and damping are parameters that will greatly affect the global dynamics of the system, and therefore the results of loads calculations.

The states of sea and winds used for these calculations are introduced in chapter 3 and addressed in paragraph 4.3.3.2.

4.3.2. MODELLING

The very first step of the dimensioning of an offshore wind turbine is an analysis of the dynamics and an assessment of the loads acting on the various components of the structure. This step requires the modelling (Figure 4.5) of the whole set of these components, of the ground and of environmental actions (winds, waves, currents...). This whole set forms a tridimensional model that is used in both the frequency and time domains (for, respectively, the analysis of frequencies – paragraph 4.3.3.1 – and the calculations of loads – paragraph 4.3.3.2).

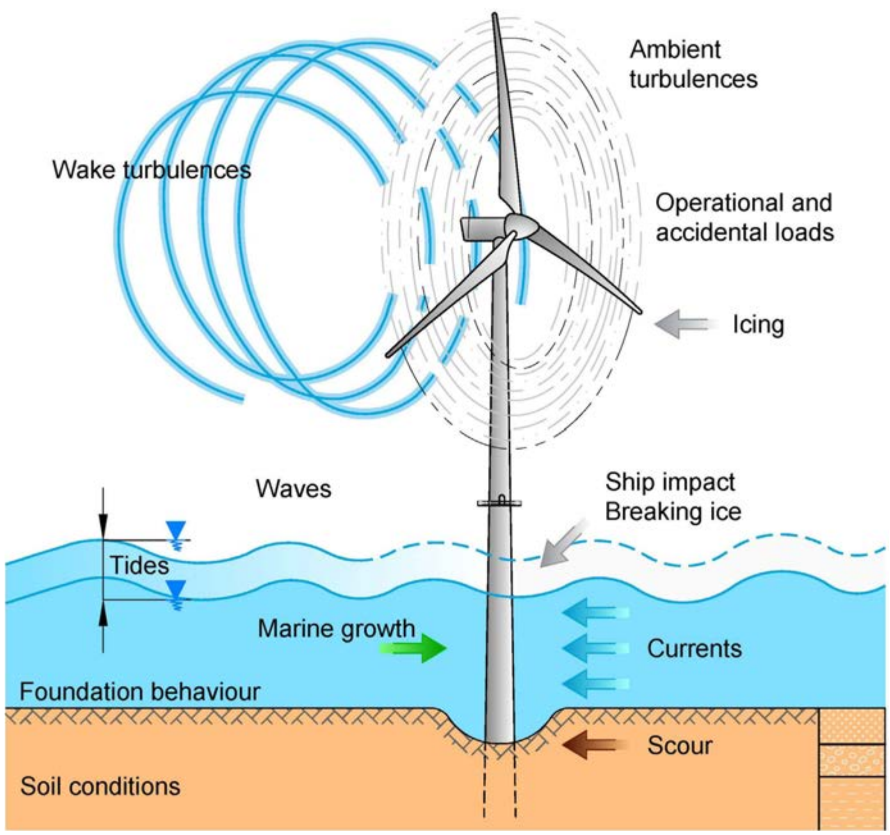

Figure 4.5 : Modelling of an offshore wind turbine

4.3.2.1. SOILS

Soils, and more precisely the interactions between the ground and the structure, are usually modelled using interactions curves of the p-y and t-z types for monopiles and piles, or using global stiffness and mass matrixes for gravity base foundations.

Integrated modelling of the structure and the ground is also possible.

Damping phenomena due to the ground (of the hysteretic and radiative types) may also be addressed by taking into account a modal damping or a global damping matrix.

Modal decomposition may only be carried out using a linear and fully elastic model, which implies that ground models be linearised. Linearisation should be compatible with ground and structure deformations for the type of loading considered. For the analysis of frequencies, soils should be modelled as realistically as possible. Upper and lower bound parameters should be defined in order to cover variability and uncertainties of soil properties as well as uncertainties about the strain range. These aspects are addressed in chapter 6.

When computing loadings in the time domain, non-linearities of the global model, and notably non-linearities of soils models, may be taken into account. For each time step, forces/displacements of the elements of foundations can be calculated by taking into account the curves of soil-structure interaction, or with elasto-plastic modellings of the foundation response. These aspects are further developed in chapters 8, 9 and 10.

The time and frequency analyses are carried out without applying partial coefficients on loads and materials. The application of such coefficients would introduce a bias within the analysis, and lead to a skewed assessment of the loads applied to the structure.

4.3.2.2. STRUCTURES

The structure of an offshore wind turbine is modelled with its whole set of components, including the nacelle-rotor assembly, the tower, the sub-structure and the foundation. A model of the rotor controller is also integrated in the model, since it can manage, among other things, blades pitch and nacelle yaw in order to optimise the production of electricity and to reduce loads.

For the structure analysis, a modal decomposition of the model is carried out to obtain the eigenmodes and the natural frequencies. The definition of an eigenmode may be outlined as such:

« For a system having several degrees of freedom, an eigenmode is a deformation under which a system may oscillate after having been disturbed near its equilibrium state; a natural frequency is then associated to this deformation. »

In the case of an offshore wind turbine structure, the flexural modes of the first and second order in the horizontal directions X and Y are considered (respectively, along the rotor axis and perpendicularly to the rotor axis) as well as the first torsion mode around the vertical axis (Figure 4.6).

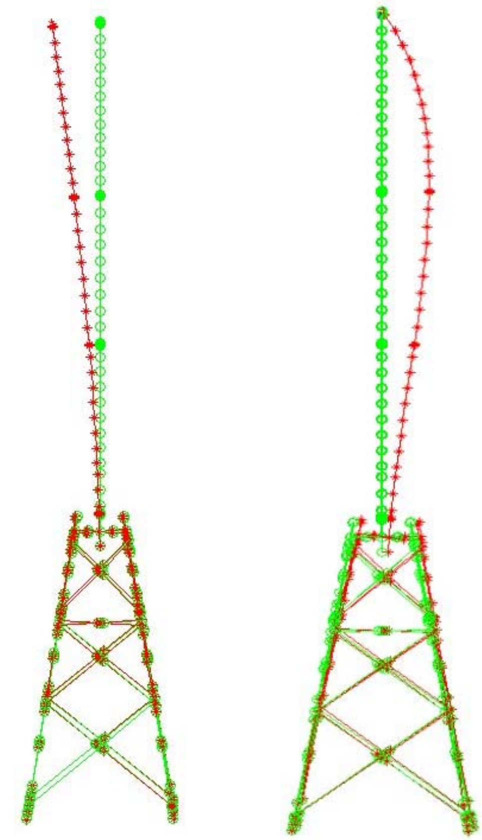

Figure 4.6 : Illustration of 1st and 2nd flexural mode shapes for a wind turbine on tripod

In order to avoid any resonance issue, during permanent operations the natural frequencies of the structure shall be distant from the excitation frequencies to which the structure is subject, due essentially to the rotation of blades and the action of wind and waves.

The excitation frequencies of the turbine, as defined in the paragraph 4.3.3.1 shall be avoided. The specifications of the turbine manufacturer shall be strictly enforced. If needed, the effect of such excitations should be assessed within the loading calculations.

4.3.2.3. SEA STATE

Within the framework of the loading calculations, hydrodynamic forces are accurately modelled by defining several sea states.

The various sea states to be considered are defined in IEC 61400-3 (2009). The calculations models of hydrodynamic forces are defined, amongst others, in IEC 61400-3 (2009) and in DNVGL - ST - 0437 (2016).

4.3.2.4. WINDS

Within the framework of the loading calculations, wind forces are modelled accurately in order to take into account various phenomena, of which are:

- different winds velocities and distributions;
- aerodynamic turbulences;
- wind gusts;
- wake effect.

Wind conditions to be considered for the calculations are defined in IEC 61400-3 (2009). Calculations models of aerodynamic forces are defined, amongst other, in IEC 61400-3 (2009) and in DNVGL- ST - 0437 (2016).

4.3.3. CALCULATIONS AND SIMULATIONS

Simulations aim at:

- firstly, ensuring that the structure, as far as possible, does not lock-in with the cyclic loading due to the turbine harmonics;
- secondly, determining loads.

4.3.3.1. ANALYSIS OF FREQUENCIES

Excitation frequencies (natural frequencies and harmonics) are mentioned in function of the rotation speed of the rotor. Therefore, 1P corresponds to the rotation frequency of the rotor (in rotations per second or hertz), 3P corresponds to 3 times the rotation frequency, i.e. the frequency at which one blade goes past the tower for a turbine with 3 blades…

The turbine, as any other rotating device, generates periodic « vibrations » that are directly linked to the following phenomena:

- **Rotor unbalance:**

The weight and geometry of each blade vary slightly. Thus, despite precautionary and corrective measures, the rotor cannot be perfectly balanced. Therefore, the centre of gravity is not set on the rotation axis, which leads to an eccentric force occuring on the rotor at each rotation, i.e. at a base frequency 1P.

- **Blades passing the tower:**

When a blade passes the tower, an aerodynamic phenomenon occurs (called « Tower Shadow ») which induces a variation of pressure on that blade. This impulse occurs on each blade every time they go past the tower, i.e. one impulse on the rotor 3 times per rotation, i.e. at a base frequency 3P.

- **Wind gradient:**

On the surface swept by the rotor, the wind is not evenly distributed: the velocity of wind increases with height. Thus, the resultant force on each blade (thrust centre) is never centred on the rotor axis, but moves periodically at each third of a rotation. This periodic excitation occurs on the rotor 3 times per rotation, i.e. at a base frequency 3P.

In addition to the fundamental frequencies described above, the various associated harmonics shall be taken into account. The first harmonic for the phenomena of wind gradient and blade rotation (i.e., at a 6P frequency) may raise issues of fatigue of the structure.
One should note that the amplitude of excitations is usually low for high frequencies (above 2 Hz), and therefore they are seldom detrimental to the tower and sub-structure.

Since the excitations due to the rotor depend on its rotation speed, the analyses of frequencies shall be carried out for two levels of operation phases: transient and permanent.
The figure below (Figure 4.7) shows a typical, and simplified, functioning sketch of a wind turbine.

Therefore, in order to check resonances frequencies, two situations should be considered:

1. the turbine operates below the nominal speed (step 2). In that case, the natural variability of the wind speed will lead to a constantly variable rotor speed, and the structure will never be continuously excited at a constant frequency. It is then possible to meet an unauthorised frequency range at a given rotation speed. To avoid imposing too much fatigue on the whole structure, the controller is preset to have the turbine operate as little as possible in this unauthorised frequency range, by managing the rotation speed and the torque. It should be noted that the design of the structure does not allow avoiding this resonance issue in a transient phase.

2. the turbine operates at its nominal speed (steps 3 and 4). In that case, the turbine controller will manage the rotor speed so it remains constant, and the structure will consequently be continuously excited at a constant frequency.

The diagram in Figure 4.8 shows the various frequency spectra to be preferably avoided (1P, 3P, 6P and 9P), as well as the frequencies of the first and second modes. In this example, one may note that several undesired frequency ranges are crossed at certain rotation speeds. In a transient phase, these occurrences of resonance are managed by the turbine controller.
In contrast, it is necessary that the rotor does not excite the structure when it rotates at its nominal speed (permanent functioning), which is the case in the example (the 1P and 3P frequencies do not excite the first mode. However, the 6P frequency excites the second mode).

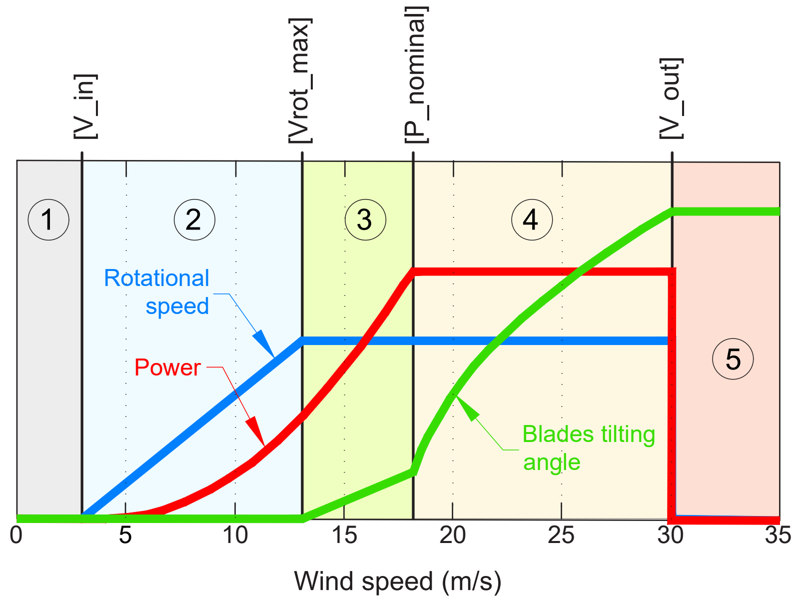

Figure 4.7 : Functioning sketch of a wind turbine in function of wind speed

1 Wind speed is too low, the wind turbine is in idling position

2 From [V_in], the wind turbine can produce power. The turbine rotational speed varies with the wind speed.

3 From [Vrot_max], the turbine rotational speed remains constant and the blades start to tilt. The produced power keeps increasing.

4 From [P_nominal], the tilting of the blades follows a different scheme to keep the nominal power constant.

5 From [V_out], the wind speed is too high for the turbine to safely produce power. The wind turbine stops and stays in "safety" condition.

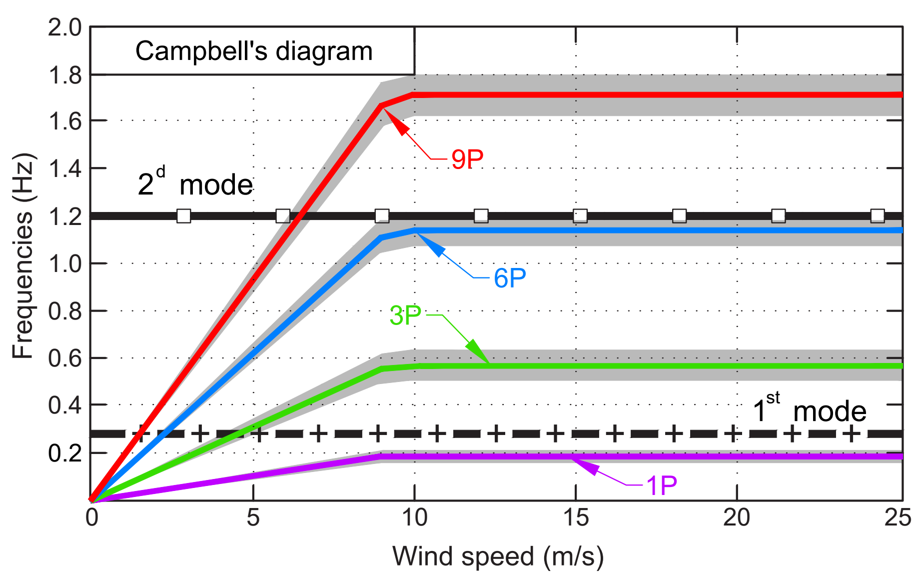

Figure 4.8 : Campbell's diagram

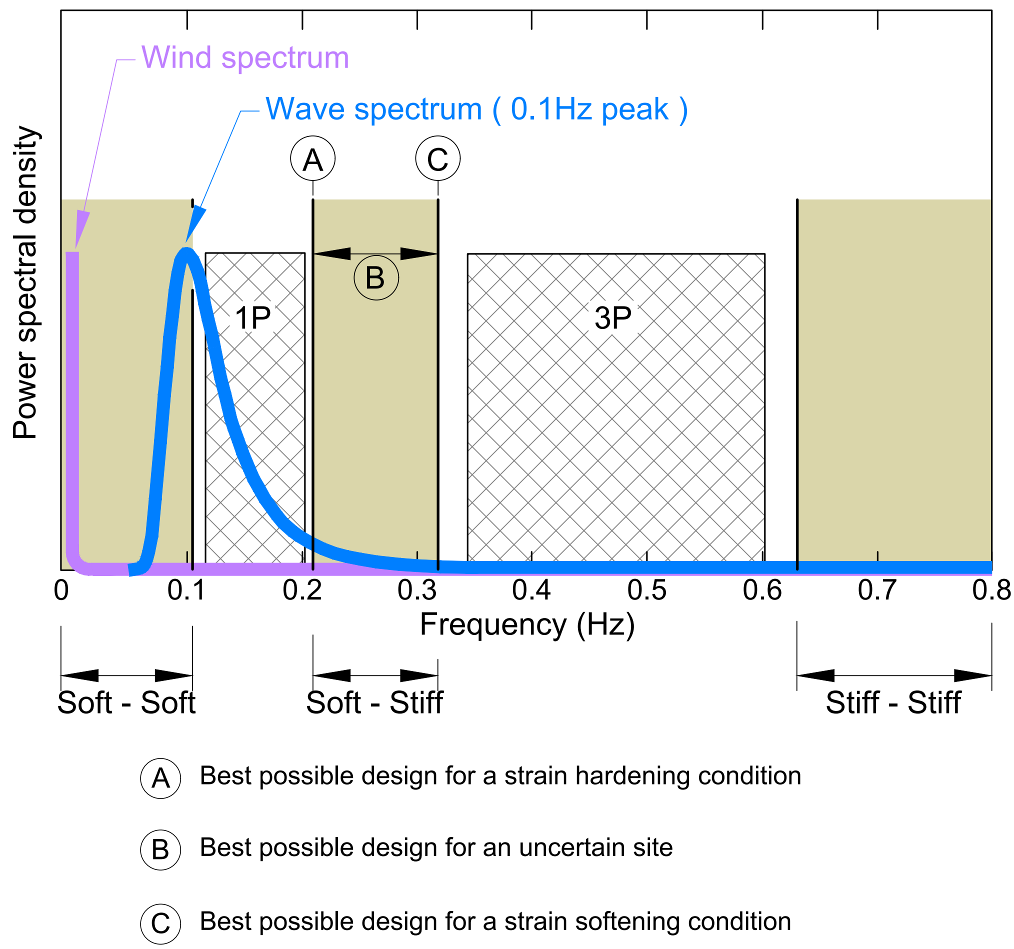

Figure 4.9 : Functioning spectrum

pan because of a possible degradation or improvement of the deformation moduli of the ground. This evolution should be planned for during the earliest design stages.

On the example of Figure 4.9, a « stiff » design, with a first mode having a natural frequency close to the undesired frequency range, may appear to be the best solution, should the mechanical features of the existing soils be prone to deteriorate over time (strain softening condition). Inversely, a « soft » design, with the natural frequency of the first mode being close to the frequency 1P to be avoided, may be the right solution if the soils are likely to consolidate (strain hardening condition). In both cases, the evolution of the ground stiffness will progressively move away the natural frequency from the most critical frequency range. In the case where an uncertainty remains about the value of the initial stiffness and its possible evolution, an intermediate position should be sought as a compromise.

4.3.3.2. DESIGN LOAD CASES

Design Load Cases, or DLC, describe the whole set of load configurations that may be encountered by the wind turbine during its whole lifespan. Eight main situations are considered, as defined in IEC 61400-3 (2009) and further developed in DNVGL-ST-0437 (2016):

1) production;
2) production with failure;
3) starting;
4) normal shutdown;
5) emergency shutdown;
6) stopped rotor (motionless or slow motion);
7) stopped rotor with failure;
8) transportation / assembly / maintenance / repair.

For each of these situations, several load cases are defined with highly precise conditions, as detailed in IEC 61400-3 (2009) and, with a few variants, in DNVGL-ST-0437 (2016).

As an example, the design load case DLC 2.1 is briefly detailed in the table below:

Design Situation	DLC	Wind condition	Waves	Wind and waves directionality	Sea currents	Water level	Other conditions	Type of analysis	Partial safety factor
2) Power production plus occurence of fault		NTM $V_{in} < V_{hub} < V_{out}$	NSS $H_S=E[H_S \mid V_{hub}]$	COD, UNI	NCM	MSL	Control system fault or loss of electrical power	U	N

Design situation	Power production
Load case	2.1
Wind condition	Normal Turbulence Model – NTM; wind velocities comprised between the starting speed and the shutdown speed.
Waves condition	Normal Sea State – NSS
Wind and waves directionality	Co-directional – COD – and Uni-directional – UNI
Sea currents	Normal Current Model – NCM
Water level	Mean Sea Level – MSL
Other conditions	Control system fault or loss of electric network
Type of analysis	Ultimate strength
Partial safety factor	N (Normal – in this case, equal to 1.35)

Models of loads calculations to be considered (winds, waves...) are more accurately described in IEC 61400-3 (2009).

A summary table of loads cases under IEC 61400-3 (2009) is provided in Appendix A.

Each load case should be calculated by having several parameters varying, such as the mean directions and velocities of winds. It is therefore required to calculate several simulations for the same load case.

It should be noted that the level of alignment of waves and wind will significantly influence the loading levels, which will lead to an additional multiplication of the number of simulations.

4.3.4. CALCULATION LOOPS

The calculation of the loads applied to the whole set of the components of an offshore wind turbine is an iterative process that aims at ensuring that the loads are compatible with the dynamics and design of the structure in its entirety.

The loads calculations are usually carried out by both the turbine manufacturer (for the nacelle and tower parts) and the foundation designer (for the sub-structure and foundation parts), and since each party needs to model the structure in its entirety, iterations are required.

The iterative process may be described as follows:

- **Step 0 :**

production of the generic loads at the bottom of the mast by the turbine manufacturer. At this step, loads are calculated by considering the wind data of the site, a regular tower and a rigid foundation. Therefore, at this step, neither the dynamics generated by marine conditions nor the sub-structure deformations or the ground-structure interaction are taken into account.

- **Step 1 :**

dimensioning of the tower, of the sub-structure and of the foundation, on the basis of generic loads, of metocean conditions and of the available geotechnical data of the site (V_1 concept). The soil-structure interaction is considered for the determination of natural frequencies and for the dimensioning of the tower, sub-structure and foundation, in order to avoid excitation ranges by the turbine.

- **Steps 2 to N :**
  a. generation of the loads applied to the structure on the basis of the following data:
     - wind conditions and marine conditions of the site;
     - geotechnical conditions of the site, updated and refined;
     - dimensioning V_{N-1};
  b. if needed, the re-designing of the structure, to guarantee its resistance to the calculated loads (V_N concept)

The iterative calculation process ends as soon as the design of the structure that was considered at the start of the loading calculations is able to resist that type of load, and that its optimisation is considered as being achieved.

Obviously, there are several ways to proceed, depending on the type of sub-structure and of foundation, and on the various actors (developer, sub-structure and foundation design engineer, turbine manufacturer).

What remains constant is that ground modelling is a key factor for both the sub-structure and foundation design engineer and for the turbine manufacturer, and that is it is essential to hold soil data as representative as possible at the very start of the project.

4.3.5. CLUSTERS

Generally, an offshore wind turbine farm is composed of tens of turbines and structures to be installed.

Two extreme approaches can then be considered: to proceed to a detailed and specific design of each structure (turbine, sub-structure and foundation), or to carry out a design adequate for the whole site. The first solution implies exceedingly long engineering and manufacturing times, whereas the second one implies to install an over-dimensioned structure on most of the site locations. In both cases, the project is not economically optimised.

In practice, both approaches will usually be combined in order to standardise the production of the foundations. The methodology consists in determining clusters of wind turbines on the basis of two essential criteria: water depth and geotechnical data. For each cluster, a specific envelope of loads may be calculated, and structures may be individually checked on the basis of these sets of loads and on the environmental conditions of each of the locations.

Determining the number of clusters and the clustering strategy depends on several economical, technical and contractual conditions that are discussed between the wind farm developer, the sub-structure design engineer and the turbine and tower designers.

4.4. REFERENCES

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5 FIELD STUDIES

5.1 INTRODUCTION

5.2 TERMS AND DEFINITIONS

5.3 ELEMENTS TO BE PROVIDED TO THE GEOTECHNICAL ENGINEER

5.4 FIELD STUDIES OBJECTIVES

5.5 SITE GEOLOGICAL MODEL

5.6 RECOMMENDED RECONNAISSANCE

5.7 REFERENCES

5. FIELD STUDIES

5.1. INTRODUCTION

The properties of a soil below the installation site of wind turbines shall be assessed using field studies complying with the applicable standards and regulations and according to the state of the art. As of to-day, no French official document does regulate the building of structures in offshore high seas. This document, and more particularly the present chapter, intends to specify the 'good practices' that should be applied during field studies carried out to build offshore wind turbines.

Field studies shall eventually provide all data required for a detailed dimensioning. They are usually divided into geological, geophysical and geotechnical studies. These studies will be carried out over various stages depending on the project needs and progress.

The scope of field reconnaissance and the choice of the methods to be implemented shall take into account the type and size of the wind turbine structure, and shall also be adapted to the anticipated geological conditions within the site (soil complexity, seabed conditions...). The surface to be covered by field investigations shall cover the whole area of the wind farm and take into account tolerances regarding the positioning and installation of the structures.

Offshore wind farms involve a large number of machines (tens to hundreds of units) as well as a wide surface area (tens to hundreds of km^2). The ground stratigraphy, the mechanical properties of materials and their lateral and vertical variability shall be accurately determined at each foundation location. Furthermore, a solid knowledge of the mechanical properties of shallow sediments is required over the cable routes, between wind turbines and to the coast. The reconnaissance of landfall areas (areas where a submarine cable comes onshore) is not covered by the present document.

5.2. TERMS AND DEFINITIONS

The following terms and definitions are required for a better understanding of the present chapter.

Crushability:

Sensitivity of soil grains to break under stress. This phenomenon is particularly significant in carbonate sands.

Field Studies:

Field studies comprise all geological, geophysical and geotechnical studies. They include all operations carried out on the field or at the office that allow establishing geological and geotechnical models of the study area.

Geological Hazard:

Geological event whose possible occurrence could generate unfavourable effects on the project objectives.

Geological Province:

A part of the site characterised by the same sequence of geological units. The notion of geological province may evolve during the project, notably in function of the seismostratigraphic data.

Geological Unit:

A soil or rock formation defined by its lithology and geological history.

Geotechnical Profile:

A sequence of geotechnical units with defined thicknesses.

Geotechnical Province:

A part of the site characterised by the same geotechnical profile, or several geotechnical profiles featuring the same sequence of geotechnical units.

Geotechnical Unit:

A soil or rock formation defined by homogeneous geotechnical parameters: classification parameters, state parameters and mechanical parameters.

Influence Height:

The influence height of a foundation is characterised by the depth under the surface beyond which the properties of encountered materials are no longer able to affect the behaviour of the foundation, in terms of both the bearing capacity and the displacements under cyclic or long-term loads (settlements due to consolidation and creep).

Investigations:

Investigations include all operations made to collect and process data.

Major Geotechnical Risk:

Risk that can jeopardise the whole project.

Minor Geotechnical Risk:

Risk that can justify adaptations during the construction stage

Reconnaissance:

All operations carried out on the site to collect geological, geophysical and geotechnical data sets on the rocks and soils, such as nature, composition, structure, spatial distribution as well as physical, chemical, geo-mechanical and hydro-geological features. These operations can be intrusive (use of drilling and surveying equipment, geotechnical measurements and testing carried out both in-situ and in laboratory) or indirect (geophysical measurements)

Representative Borehole:

A borehole can be considered representative with respect to a specific geotechnical issue if it can bring elements that meet the requirements in terms of depth and data content.

Note 1. The borehole must be deep enough to provide data on a height equalling at least the planned burial depth of a cable, the penetration of a skirt or the influence height of a foundation.

Note 2. Geotechnical parameters that have been collected must allow bringing relevant elements with respect to the raised geotechnical problem. For instance, a simple drilling with parameters recording can be deemed representative for a cavity search, or similarly, a cone penetration test to assess the penetrability of a skirt. However, to be deemed as representative for a foundation study, a borehole must provide information with sufficient quantity and quality to allow establishing a profile of geotechnical parameters.

Risk:

Unfavorable consequence of an uncertainty or hazard on the project objectives.

Routing:

All studies allowing optimising the route of a submarine cable by taking into account the nature and topography of the seafloor, as well as obstructions or constraints both natural or man-made.

Seismo-stratigraphic Unit:

A soil or rock formation defined from seismic reflection data, characterised by a seismic facies and delimited by reflectors.

Significant Geotechnical Risk:

Risk that can justify significant changes during the design stage.

Stratigraphic Profile:

A sequence of stratigraphic units defined by their lithology and thickness.

Substratum:

In this document, and taking into account the dimensions of the foundation and the loads applied to it, substratum means a formation whose mechanical characteristics do not allow failure lines to develop, and with a compressibility that is sufficiently low to be ignored during the settlement calculations of the foundation.

5.3. ELEMENTS TO BE PROVIDED TO THE GEOTECHNICAL ENGINEER

The data that the owner or the "maître d'oeuvre" shall make available to the geotechnical engineer are to a large extent depending on both the nature of the services required and the project stage. The elements to be provided should be defined for each operational stage.

The indications given below in this paragraph must be deemed as informative, and thus, as minimum.

Regardless of the nature of the contract (studies, reconnaissance services), the geotechnical engineer should be made aware of:

- the precise location of the project;
- the development state of the project (conceptual studies, pre-project, detailed project);
- the decisions previously taken and how they may evolve in terms of foundations type and installation;
- the history and results of the investigations already completed;
- the precise objectives of the geotechnical mission.

The geotechnical engineer assigned to the field reconnaissance operations should in addition have at disposal a complete overview of the site conditions, notably: bathymetry, seafloor morphology, expected geology, operational and extreme weather and sea conditions (waves, wind, current, tide).

The owner or "maître d'oeuvre" should share without any restriction its knowledge of geohazards and hazards caused by man (shipwrecks, cables, unexploded ordnances). In the event where unexploded ordnances would be present or suspected, the owner has the responsibility, and prior to any operations on the site, to take all necessary measures to establish the nature and level of the associated risks, as well as the appropriate preventive measures.

5.4. FIELD STUDIES OBJECTIVES

Field studies shall provide the appropriate information regarding soils and rocks, up to a depth that will allow detecting the presence of weak formations, able to:

- affect the stability of the structure;
- generate excessive deformations (settlements).

Field studies will usually comprise:

- studies of the geological context, at the scale of the site;
- geophysical studies;
- geotechnical studies.

Geological studies shall allow identifying major hazards and subsequent risks.

Geophysical studies essentially include surveys from echosounder, from side-scan sonar and from seismic reflection. The objective is to establish the bathymetry and the sea floor morphology, to define lithological units and tectonic structures, and to provide the data required to establish stratigraphic profiles. They will allow a spatial correlation with the one-off data from sampling and in-situ tests.

Geotechnical studies comprise geotechnical investigations and data interpretation. The geotechnical investigations include:

- reconnaissance from in-situ tests [for instance: cone penetration tests (CPT/CPTU), pressuremeter tests (PMT), dilatometer tests (HPDT)] as well as sampling followed by laboratory tests;
- data processing.

The objective of the geotechnical investigations is to obtain for each geotechnical formation the following data:

- classification and description of the soils and rocks;
- geotechnical parameters: shear strength and deformation properties, in-situ stress state (e.g. overconsolidation) relevant for the type of analysis planned.

The interpretation of the geotechnical parameters provided shall allow a detailed and complete design of the foundations. Lateral extent and variation of geotechnical units and geotechnical parameters are issues that should be answered.

It is of utmost importance that ground samples collected during the geotechnical reconnaissance and dedicated to laboratory testing be of sufficient quality to allow producing the geotechnical parameters used for dimensioning.

The laboratory testing programme designed to determine the strength and deformation properties of the soil should entail tests that are adapted, and carried out in a sufficient number.

The effects of the cyclic loads generated by swell and wind on the geotechnical parameters should be taken into account throughout the dimensioning process of the foundations of offshore wind turbines.

There are several effects, and they notably concern:

- how shear strength and shear moduli may evolve due to the accumulation of loading cycles;
- how strengths and moduli may be modified in relation with loading rate.

These evolutions notably depend on how pore pressures may vary.

When combined, these effects may significantly affect the long-term response of foundations (cyclic movements, settlements, horizontal displacements). The evolution of the stiffness of the soil-foundation system may affect the structure natural period and resistance to fatigue. Specific tests are required to determine the cyclic behaviour of the soils and how the shear moduli vary with the distortion level.

Several steps are required to reach sufficient level of knowledge regarding the geological and geotechnical conditions of the site (see paragraph 6.2). Each step must be concluded with the proposal of a ground model (see paragraph 6. 2). While initially temporary and incomplete, this model will eventually help defining the content of the following stages and will be gradually supplemented until it becomes the final model. This final model is characterised by an accurate description of the geology over the whole site and provides geotechnical parameters profiles for the dimensioning of the foundations below each structure (wind turbine, sub-station, meteorological mast, cables.

5.4.1. ISSUES SPECIFIC TO THE FOUNDATIONS OF OFFSHORE WIND TURBINES

The main issues regarding the design of the foundations of offshore wind turbines are discussed in this paragraph.

Scour: waves and currents may generate scour around the foundations (piles or gravity bases). Scouring is particularly hazardous if fine to medium sands are present, but assessing their magnitude is always difficult. Anti-scouring solutions may be necessary. Alternating tilt movements due to the waves may also generate "flushes" under the edges of the gravity bases, as well as generate erosion.

Ultimate capacity: no matter what type of foundation is anticipated, the surrounding soil should be capable of bearing both the static and cyclic loads transmitted by the structure, with a sufficient safety margin regarding failure, and without any excessive displacement. For monopile or gravity base foundations, loads are compressive ones. In the case of multipods or anchor foundations, loads may be tensile ones. The capacity under cyclic loading may differ from the capacity under monotonic loading. The capacity under cyclic loading should be considered carefully.

Cyclic degradation: some types of soils (for instance: soft clays, sensitive clays, carbonate soils) may undergo a significant degradation of their mechanical properties due to cyclic loading. This phenomenon has consequences on the ultimate capacity and the displacement of foundations.

Permanent displacements: static (permanent) loads result in initial displacements of the structure, that may be followed by further displacements over time generated by soil consolidation and creep effects. Cyclic loads due to wind and waves may also cause additional permanent displacements resulting from shear deformations and pore pressure dissipation generated by the repetition of loads in soils of low permeability. Vertical displacements or settlements should be anticipated if gravity base foundations are considered. Permanent horizontal displacements are particularly critical in the case of monopile foundations as they result in permanent rotations.

Cyclic displacements: cyclic loads generate cyclic and post-cyclic displacements of the foundation and of the structure. Some soils (soft or sensitive clays, loose granular materials, carbonate soils) may be particularly sensitive to these phenomena and consequently generate excessive settlements. The sum of these displacements (both permanent and cyclic) at the rotor level should remain lower than the tolerance limits associated to wear and fatigue risks.

Cables burial: in the zones currently considered for the installation of offshore wind farms, protecting the cables will preferably require burial. There are several burial methods, such as ploughing, jetting, trenching. How efficient is each method entirely depends on the type of soils encountered over the burial depth required to protect the cable, or imposed by regulations. Within rocky materials, alternative protective methods may be considered, such as rock dumping or prefabricated elements. The type and features of the means to be used should be established for each application.

Piles installation: the driving of metallic elements (essentially tubes) remains the most common solution used to install offshore piles. Driving large diameter monopiles (typically from 5 to 8 m) is achievable. In stiff soils, it may be necessary to remove the pile plug or to drill pilot holes to assist driving. In rocky soils, drilling and grouting is a possible solution.

Specific studies are necessary to ensure that:

- it is possible to drive the piles to the depth required to mobilise the design resistance;
- hammers will be appropriately selected;
- stresses generated by pile driving will not damage the pile elements.

In stiff soils and soft rocks, particular attention should be given to risks of premature refusal, to potential damages on the pile tip in hard levels, and to potential pile collapse due to structural instability and steel fatigue generated by large numbers of blows. Preliminary tests for ensuring pile driving feasibility may be required. These tests should be anticipated sufficiently early on the project schedule, either in the offshore site, or onshore on a site with proven similar geotechnical features.

Sediments mobility: how the sea floor level may possibly evolve and affect the wind farm life cycle shall be determined by a hydro-sedimentary study, which will include potential dunes displacement, seabed erosion, accretion...

Skirts penetration: it may be necessary to equip gravity bases with skirts either to ensure that foundations remain stable or to avoid phenomena such as peripheral scouring or erosion due to wash out under the base. The penetration of these elements should be achievable down to the required level.

Liquefaction potential: the risk of liquefaction (loss of mechanical resistance) of sands or silty sands, which results from cyclic loads, should be assessed in seismic and/or strong swell zones.

Sea floor preparation work: sea floor preparation work may be required prior to installing the foundation. For instance, the sea floor may require rocks removal or ground levelling prior to installing the piles or burying the cables. In the case of gravity base foundations, it will be generally necessary to build an artificial flat platform by adding materials. In some cases, removing surficial sediments that are heterogeneous or of poor material properties should be considered. The stability of the added materials should be subject to a specific study.

Foundations stiffness: foundations stiffness is an essential element when assessing the structure natural frequency. Offshore wind turbines are particularly sensitive to resonance and fatigue issues. The structure natural frequency and how it evolves over time due to cyclic loading (stiffness degradation) should be accurately assessed.

Soil reactions: ground reactions under the base, due to monotonic and cyclic loads, should be accounted for in gravity base design. In the case of stiff soils, or soils with highly heterometric grain size, these reactions may be very strong.

Overall stability: the overall stability of the soil units bearing the foundations should be ensured, notably in the case of submarine slopes and when foundations generate significant stress on large surfaces (e.g. gravity foundations). Specific slope stability studies may be required. They should consider the various possible triggering issues (gravity, seismic acceleration, gas within sediments, etc.) and the effect of stresses induced by the structure.

5.4.2. ACQUISITION OF THE PARAMETERS REQUIRED FOR THE DESIGN OF THE FOUNDATIONS OF OFFSHORE WIND TURBINES

5.4.2.1. PARAMETERS REQUIRED FOR THE DESIGN OF THE FOUNDATIONS OF OFFSHORE WIND TURBINES

In order to answer the issues raised above, a number of information, both geological and geotechnical, should be gathered. The basic parameters required to identify and classify soils and rocks encountered in the stratigraphic profile are listed in Table 5.1. The classification shall be made in compliance with a recognized standard (ISO, BS, ASTM, AFNOR). Tables 5.2 and 5.3 specify the additional parameters required for specific issues, or for soil types that do not behave in a standard way, such as carbonate sands, soils of volcanic origin or chalk. These materials, sometimes called unconventional soils (ISO 19901-8, 2013), are present in both metropolitan and overseas French waters.

Table 5.1: Parameters required for a standard characterisation of soils and rocks

CLAY, SILT	SAND, GRAVEL	ROCK
General description **Lithography**	General description Lithography	General description Lithography
Grain size distribution	Grain size distribution Angularity	Presence of heterogeneous elements (blocks, flint, gypsum …) Fracturation (RQD, opening and state of fractures, spacing, orientation) Weathering
Water content **Total unit weight** **Atterberg limits (w_L and w_P)**	Minimum and maximum densities Relative density	Total unit weight Porosity, saturation Weight of solid blocks
Organic matter content **Carbonate content**	Organic matter content Carbonate content	Carbonate content
Undrained shear strength **Drained shear strength** **Residual and/or remoulded shear strength**	Effective angle of friction (φ') Undrained shear strength	Unconfined compressive strength (UCS = σ_c)
Mineralogy	Mineralogy	Mineralogy
Stress history	Stress history	

Table 5.2: Additional parameters that might be required for specific issues

ISSUE	PARAMETERS
Ultimate strength	Monotonic shear strength under various stress paths (strength anisotropy) Cyclic shear strength under various combinations of average stress and cyclic amplitude for triaxial or simple shear stress paths Sand: Effective angle of friction (φ'), critical angle or phase transition angle, constant volume friction angle (φ'_{cv})
Permanent displacements	Compressibility Permeability Permanent strains and pore pressures generated under various combinations of average stress and cyclic amplitude for triaxial stress paths or simple shear Compressibility after cycles
Cyclic displacements	Cyclic shear strain versus cyclic shear stress for triaxial or simple shear stress paths Initial cyclic shear modulus
Foundation stiffness	Cyclic shear strain versus cyclic shear stress for triaxial or simple shear stress paths Shear modulus at very small distortion (G_o or G_{max}) and evolution with distortion level Damping
Soil reactions	Monotonic and cyclic shear strength Compressibility under virgin loading and reloading Permanent and cyclic strains and permanent pore pressures under various combinations of average stress and cyclic amplitude for triaxial or simple shear stress paths Sea floor topography and morphology, presence of anomalies on the sea floor
Skirt penetration	Undrained shear strength Remoulded shear strength (or sensitivity) Drained angle of friction (φ') - Sand Residual sand-steel or sand-concrete interface angle (δ_r) Cone resistance (q_c) Sea floor topography and morphology, presence of anomalies on the sea floor Presence of blocks in the soil
Pile installation	Shear strength Young's modulus (E_{50}) or shear modulus (G_{50}), or strain at 50% of ultimate strength (ε_{50}) - Clays Cone resistance (q_c) Unconfined compressive strength (UCS = σ_c) - Rocks Abrasivity Clay sensitivity
Liquefaction potential	CPTU data (q_c or q_t, F_R, B_q) Grain size and fines content Atterberg limits (w_L and w_P) and water content Shear waves velocity (V_s)
Scouring and erosion	Grain size Permeability
Cable burial	Cone resistance (q_c) - Sands and clays Grain size and permeability – Sands Rock abrasivity Thermal conductivity Electrical resistivity Velocity of compression (V_p) and shear (V_s) waves

Table 5.3: Additional parameters required to characterise some non-standard soils and rocks

TYPE OF SOIL	PARAMETERS	NOTES
Carbonate sands, with or without cementation	Compressibility (limit compressibility index) Crushability Degree of cementation Unconfined compressive strength if cementation	Classification according to Clark and Walker (1977) based on three criteria: carbonate content, grain size, unconfined compressive strength See Argema (1988, 1994).
Soils of volcanic origin	Compressibility Others: case-by-case study	High nature and behaviour variability Case-by-case study
Clays	Accurate description of weathering levels Compressibility Creep Matrix permeability Soil mass permeability Water absorption	Classification according to CIRIA (2002) based on density, weathering, fracturation state
Organic soils	Organic matter content Compressibility Creep Presence of gas	

5.4.2.2. RELEVANCE OF THE IN-SITU AND LABORATORY METHODS USED TO ACQUIRE THE PARAMETERS

The relevance of the various in-situ or laboratory methods used to determine soils parameters is assessed in the following tables. There is a distinction between tests that are commonly used during usual investigations (Table 5.4) and specific tests that should be performed for particular applications (Table 5.5). A table is dedicated to tests on rocks (Table 5.6). The applicability level of each method is assessed over a 1 to 5 scale, with:

1 = weak or inappropriate
2 = acceptable for non-critical analyses
3 = moderately good
4 = good
5 = very good

Table 5.4 : Methods for usual investigations

Soil parameters	In-situ tests			Laboratory tests		
	Type of test	Applicability		Type of Test	Applicability	
		Sand	Clay		Sand	Clay
Stratigraphy	Seismic reflection [(a)]	2 to 3	2 to 3			
Surface soils classification (sea floor)	Multibeam ba-thymetry Side Scan Sonar (SSS)	1 1	1 1	Grain size Grain size and fines content Water content Atterberg limits	5 2 	2 4 3 5
Sub-surface soils classification	CPT CPTU	2 4 to 5	2 4 to 5	Grain size Grain size and fines content Water content Atterberg limits	5 2 	2 4 3 5
In situ density	CPT, CPTU	2	2	Unit weight measurement		4
Undrained shear strength	CPT, CPTU VST PMT Tbar, Ball probe		3 to 4 4 to 5 2 to 3 4 to 5	Triaxial UU Triaxial CIU DSS Fall cone, torvane Pocket penetrometer	 4 	2 to 3 4 4 2 2
Effective angle of friction	CPT, CPTU	2 to 3	1	Triaxial CIU, CID DSS	5[(b)] 4	5 1
Sensitivity	CPT, CPTU VST Tbar, ball probe		2 3 to 4 4 to 5[(c)]	Fall cone, lab vane Triaxial UU on intact and remoulded materials		3 to 4 3 to 4
Deformability (G_{50}, E_{50})	PMT	3 to 4	4 to 5	Triaxial CIU, CID DSS	3 to 4 3 to 4	4 4
Consolidation properties	CPTU	1	3	Oedometer	3[(b)]	5
Permeability	CPTU		3	Oedometer Permeameter	 4	3 4

(a) multichannel technics to be used when water height is lower or equal to the target penetration (need to erase the multiple)
(b) provided in situ density is known
(c) only if cyclic tests are made

Table 5.5: Methods for specific applications

Soil parameters	In-situ tests			Laboratory tests		
	Type of test	Applicability		Type of Test	Applicability	
		Sand	Clay		Sand	Clay
Soil units interpolation	Seismic refraction Electrical resistivity	3 to 4[a] 1 to 3[b]	3 to 4[a] 1 to 3[b]			
Identification of carbonate soils	CPT, CPTU	4[c]	3	Carbonate content	5	5
Compressibility of carbonate sands				Oedometer Crushability	4 3	
Strength anisotropy of clays				Triaxial CAU_C, CAU_E and DSS		5
Cyclic response and loading rate effect				Triaxial CIU/CAU (static/cyclic) DSS/CSS	5	5
Thixotropy				Thixotropy test		4
Interface behaviour (piles, caissons)				Ring shear (soil/soil and soil/steel) Shear box (soil/soil and soil/steel)	3 to 4 3 to 4	3 to 4
Initial shear modulus G_{max}	Seismic cone MASW	4 to 5 3 to 4	4 to 5 3 to 4	Resonant column Bender elements on triaxial, DSS or oedometer	4 to 5 4 to 5	4 to 5 4 to 5
Corrosion potential	Electrical resistivity cone	4	4	Electrical resistivity	4	4
Liquefaction potential	CPT, CPTU	3 to 4		Cyclic triaxial	3 to 4[d]	

(a) subject to a good calibration on in-situ tests (CPT) or on samples
(b) poor definition of interfaces; an extensive calibration on in-situ tests (CPT) or on samples is required
(c) CPT data is highly sensitive to the level of cementation
(d) provided in situ density is well known

Table 5.6: Specific methods for investigations in rocks

Soil parameters	In-situ tests		Laboratory tests	
	Type of test	Applicability	Type of test	Applicability
Stratigraphy	Videologging Neutron Gamma-ray	3 to 5 3 to 4 3 to 4		
Fracturing (frequency and orientation)	Videologging Eastman Camera	3 to 4 4 to 5		
In situ density	Gammagraphy (gamma-gamma)	3 to 4	Density measurement	4 to 5
Strength			Unconfined compressive strength test Brazilian test Point load test	4 4 1 or 3[a]
Deformability (G_{50}, E_{50})	HPDT	3 to 5	Unconfined compressive strength test with strain gauges	3 to 5
Initial shear modulus G_{max}	Seismic logging (V_p ; V_s) MASW	4 to 5 3 to 5	V_p and V_s measurements on cores	3 to 4

(a) subject to a correlation with the unconfined compressive strength

5.5. SITE GEOLOGICAL MODEL

The large size of an offshore wind farm development (tens to hundreds of km²) coupled with the low density of foundations (on average: one structure per km²) requires designing an adequate reconnaissance strategy. It is both about obtaining a complete assessment of the stratigraphic and tectonic structures on the whole site, and determining the geotechnical parameters required for the dimensioning of the foundations of each of the wind turbines.

It is therefore necessary to develop the knowledge of geological and geotechnical conditions at both the scale of the site and the scale of the foundation. One way to reconcile both scales of knowledge is to build a geotechnical and geological model that can evolve over time. As the project progresses, the model will integrate and synthesise all available information about the site.

The main objective is to eventually define geotechnical design profiles. For that matter, and during the various stages of studies, one should define:

- firstly, geological provinces whose features (lithology, stratigraphy) may be considered as homogeneous;
- secondly, geotechnical provinces characterised by similar features regarding the nature of soils, layer thicknesses and geotechnical hazards.

Each step of the model helps improving the schedule, nature and content of the reconnaissance campaigns by integrating the knowledge previously acquired.

The model embeds the various geological hazards that may affect the choice of the type of foundations, their design or their construction. Geological hazards to be taken into account are listed in paragraph 5.6.2.

The hazards that may affect the project shall be subject to specific studies. Some studies may require fields of expertise that are outside the scope of the present document.

Geological information systems (GIS) may represent efficient tools to manage data and build the geological and geotechnical model.

The main stages of the building process are described below. Each of them constitutes an improved version with respect to the previous one.

Stage 1: Initial geological model

The first version of the model is achieved by studying documents (bibliography, bathymetric maps, local or regional geological maps, geotechnical studies of the same area or of an adjacent one...), as detailed in paragraph 5.6.2. The quality and accuracy of this first model may highly vary depending on whether the area of interest has been previously studied or researched scientifically. It should usually allow establishing the following elements:

- general stratigraphy and lithology of the main geological formations;
- tectonic elements;
- main geological hazards and constraints.

Stage 2: Stratigraphic (or seismo-stratigraphic) model

The second stage of the model is elaborated from preliminary reconnaissance (geophysical in particular) carried out over the whole field (paragraph 5.6.3, Table 5.8). Bathymetric data is used to establish a digital ground model, while data from seismic reflection is used to define the geometry of the main stratigraphic units. At this point, the transformation of seismic waves propagation is most often based on hypotheses on propagation velocities within the various layers. Thus, defining the geometry of stratigraphic units remains a rather inaccurate process.

If, at this stage, boreholes data can be obtained, the additional information acquired should be taken into account to enhance and calibrate geophysical data.

The stratigraphic model allows defining areas of similar nature and seismic features, which will provide directions for the preliminary geotechnical reconnaissance.

Stage 3: Site geological model

The site geological model is formed by integrating the results from the preliminary geotechnical reconnaissance (paragraph 5.6.3, Table 5.9) into the previous seismo-stratigraphic model. Preliminary geotechnical data should allow:

- improving the velocity model and therefore enhancing layers geometry as well as their lateral thickness variability;
- improving the lithological characterisation of layers;
- assigning preliminary geotechnical parameters to these layers;
- putting forward a draft of the geotechnical provinces.

Stage 4: Geotechnical model

At this point, data collected from the detailed reconnaissance (paragraph 5.6.4, Tables 5.10 and 5.11) are integrated within the model. Integrating the geotechnical data within the geological model of the site should allow defining with more accuracy the geotechnical provinces proposed previously.

This task may lead to defining geotechnical units separate from the seismo-stratigraphic units that were previously defined, for the following reasons:

- several seismo-stratigraphic units may disclose similar geotechnical parameters;
- conversely, some seismo-stratigraphic units may show internal variations that require defining several geotechnical subunits;
- at last, some phenomena that cannot be detected by using indirect geophysical methods (weathering in particular) may affect entirely or partially some seismo-stratigraphic units.

Geotechnical provinces allow proposing one or several geotechnical design profiles, featuring similar layer thicknesses and homogeneous mechanical properties.

Each geotechnical design profile shall define:

- soils classification and description;
- shear strength and deformation properties that will be required for the planned type of analysis;
- the in-situ state of stress (OCR and K_0, anisotropy …);
- geotechnical parameters offering a response to offshore wind turbines specificities (cyclic loading, fatigue …).

The geotechnical parameters provided shall meet the needs for a complete and detailed design of the foundations. Assessing their variability is an essential issue.

5.6. RECOMMENDED RECONNAISSANCE

5.6.1. PLANNING OF SITE INVESTIGATIONS

The objective of soils investigations (geological studies, geotechnical and geophysical reconnaissance) is to reach a level of knowledge as thorough as possible, in order to:

- build a geological and geotechnical model of the site (paragraph 5.5);
- define the type(s) of foundations that will be best suited to the established geotechnical profiles and to the project needs.

In reference to international practices, developing an offshore wind farm entails three main stages:

- a **preliminary stage** aiming at establishing the technical and financial feasibility of the project;
- a **project stage**, strictly speaking, which covers all steps related to designing and building the structures;
- an **operation stage** in which the owner shall inspect and maintain the structures.

Table 5.7 shows a synthesis of the various stages required to develop an offshore wind farm.

5.6.1.1. PRELIMINARY STAGE

The French current context of public procurement implies a two steps sequence within the preliminary stage (conceptual design):

- the first step, called **pre-project**, follows a public tender: the tenderer is led to preselect a type of structure and its associated foundation, to perform a pre-dimensioning and to estimate a cost. The relevance of the choices made during this step and the representativeness of the estimated costs are to a large extent dependent on the representativeness of the initial geological model available at this point. It is the duty of the tenderer to define the risks associated with its bid and to decide if investing in reconnaissance operations could reduce these risks. The present document does not recommend whether performing site investigations or not is needed during this step.
- the second step, called project draft or **tender confirmation**, lasts for a duration of 1 to 2 years after the concession is awarded. In this step, the validity of the selected technical options should be proven and financial assessments should be precised. In particular, all major geological hazards shall be identified. This stage should include a significant volume of geotechnical and geophysical reconnaissance. Results from these investigations will feed the geological and seismo-stratigraphic models of the site. The nature and scope of the preliminary reconnaissance to be achieved during this step are both developed below, and summarised in paragraph 5.6.3 (Tables 5.8 and 5.9).

5.6.1.2. PROJECT STAGE

The project stage entails two aspects (design and construction) and three steps: basic design, detailed design and installation.

At the end of the project stage, the geotechnical parameters required for the final design and installation of the foundation system of each wind turbine shall be available. The nature and scope of the reconnaissance will notably depend on the type of foundations selected and on the heterogeneity of each site. This last criterion may be critical in the geological context of the French continental shelf.

The final investment decision is taken (or not) based on conclusions of the design phase. During this design stage, significant geotechnical hazards shall be identified and the geotechnical model shall be finalised. The model should eventually include geotechnical profiles applicable at each wind turbine location, or by group of turbines.

During the project construction stage, additional reconnaissance may still be required to avoid minor or localised risks.

The programmes of the detailed reconnaissance to be carried out during the project stage are developed below and summarised in paragraph 5.6.4 (Tables 5.10 and 5.11).

5.6.1.3. OPERATION STAGE

During the operation stage, inspection and maintenance works should be performed by the owner to ensure the long-term stability and safety of the installations. For instance, campaigns to inspect the sea floor or additional reconnaissances may be considered to address specific issues, such as risks related to scouring.

It is also recommended to set up an information feedback regarding the structure behaviour. This feedback should entail an operational monitoring of the structures and foundations, as well as an analysis of the data gathered.

Table 5.7 : Planning of a development project for an offshore windfarm, and organisation of the geotechnical and geophysical investigations

Project stages			Objective of the project stage Assessment of geotechnical risks	Geological studies, geophysical and geotechnical reconnaissance to be carried out
Preliminary	*Conceptual design*	**Pre-project**	Pre-selection of the structures and foundations types Financial and technical assessment of the project	Compulsory geological (bibliographical) study (DTS) Constitution of the initial geological model Optional geophysical and/or geotechnical reconnaissance
		Project draft	Major risks assessment Confirmation of tenders in the French context Validation of the technical options Validation of the financial assessment Drawing up of the general building principles Choice of the structures and foundations type Structures settlement Pre-design of foundations Installation feasibility of foundations and cables	Compulsory preliminary geophysical and geotechnical reconnaissance. Objectives: • Identification of the major geotechnical hazards • Definition of the stratigraphy and lithology • Constitution of the stratigraphic and geological site models • Definition of the geotechnical parameters for the pre-design of foundations for each geological province • Preliminary characterisation of cable routes and installation conditions
Project	*Basic design FEED*	**Design**	Significant risks assessment Validation of construction means, costs and planning Design for each group of wind turbines Investment decision and switch to construction stage	Compulsory detailed geophysical and geotechnical reconnaissance Objectives: • Identification of the significant hazards • Definition of the stratigraphic profiles and of the geotechnical parameter profiles for the design of the foundations • Constitution of the geotechnical model • Definition of the cable laying and burial conditions • If necessary, feasibility tests regarding installation or burial
	Detailed design	**Detailed design and construction studies**	Minor or localised risks assessment Detailed study of each wind turbine. Design of each foundation. Burial predictions. Detailed installation procedures for foundations and cables. Remediation procedures	Additional specific reconnaissance(s) if required Objectives: • Identification of minor or localised hazards
	Installation	**Installation**	Installation follow-up	Implementation of monitoring
Operation	*Inspection Maintenance*	**Inspection Maintenance**	Ensuring the long-term stability and safety of the structures Organising the feedback regarding the behaviour of structures	Scour monitoring (bathymetry) Instrumentation set up and data analysis

5.6.2. STUDIES ON EXISTING DOCUMENTS

This initial study, called Desk Top Study (or DTS) entails gathering and processing all existing and accessible "bibliographical" data.

The full study should allow identifying the major hazards and the associated risks. It requests various skill sets and usually consists in assembling information dealing with all environmental site conditions:

- bathymetric conditions (water depths);
- metocean conditions;
- geotechnical and geophysical conditions;
- presence of man-made elements on the site: operational or disused cables or pipes, ship-wrecks, unexploded ordnances (UXO) or other obstacles, either on the sea floor or buried;
- fishing activities;
- navigation traffic;
- leisure boating activities;
- existence of wildlife/protected reserves;
- prohibited areas (military…).

The objective of the geological and geotechnical bibliographic study is to gather as much accessible information as possible, which could highlight major hazards due to soils or define a realistic choice for a foundation solution. Data sets may directly concern the site, or its proximity.

The conclusions of the study may become critical when assessing the technical feasibility of some types of foundations and the economic consequences on the overall project. In any case, they are essential to give directions to further stages.

Particular attention should be taken to characterise more specifically the risks arising from the following issues:

- faulting networks and their activity;
- fractured zones;
- paleo- thalwegs;
- complex hydrogeological conditions, artesian groundwater;
- seismic hazards;
- liquefaction and cyclic mobility of sands;
- shallow gas;
- slopes stability;
- karsts, cavities;
- erosion;
- mobility of surface sediments, either of natural origins, or caused by the influence of structures, substructures and foundations;
- soils with specific behaviour (carbonate, volcanic, polluted...);
- presence of large size elements (boulders…) or of indurated zones that could prevent building the foundations;
- presence of soils that could evolve over the scale of the structures life span.

These information sets will be researched using documents and technical publications from specialised organisations: Ifremer, SHOM, BRGM, etc. as well as academic and private scientific archives. Experience shows that geophysical records can be accessed and somehow re-processed. In some cases, geotechnical data acquired on the site or close to it or from formations of a similar nature may prove useful and relevant.

This bibliographic stage will lead to the design of the initial geological model and will allow defining:

- the level of knowledge regarding the geological and geotechnical features on the whole site;
- the missing critical parameters (morphological, stratigraphic or geotechnical) needed to achieve the following stages;
- the objectives and specifications of the preliminary geotechnical and geophysical reconnaissance to be carried out.

5.6.3. PRELIMINARY RECONNAISSANCE

After the preliminary development stage, major ground hazards should have been identified, the structures and foundations types should have been defined and the pre-dimensioning of foundations should allow realistic cost estimates.

These objectives require:

- a good understanding of the geological and geophysical features of the site;
- an assessment of the geotechnical characteristics of the materials, as well as their spatial variability.

Data gathered from the preliminary geotechnical campaign should be made available as soon as possible in order to take into account the potential geological heterogeneities and to accurately specify the objectives of the geotechnical campaign that will ensue.

Preliminary reconnaissance should allow a clear definition of both the geophysical and geotechnical means that are best suited to the characterisation of the soils encountered on the site and should be later utilised for further reconnaissance.

5.6.3.1. GEOPHYSICAL PRELIMINARY RECONNAISSANCE

The geophysical preliminary reconnaissance on the whole development site of the offshore wind farm should allow:

- establishing the bathymetry and seafloor morphology;
- defining lithological units and tectonic structures;
- understanding the site geology;
- putting forward a seismo-stratigraphic model, down to at least the influence depth of the foundations;
- providing directions for the geotechnical reconnaissance, and more particularly so that data can be acquired on all geological provinces.

Usually, the means to be used are:

- multibeam echosounder;
- side scan sonar;
- seismic reflection;
- magnetometry.

The means to be used should also allow detecting man-made obstructions (unknown shipwrecks, cables, unexploded ordnances). In the case of UXOs, it must be precised that, if the detection of magnetic anomalies related to their presence may be performed during the geophysical campaign, specifying which data should be gathered and how it should be processed falls outside the competence of the geophysicist. Magnetometer surveys are indeed usually performed during campaigns that are separate from the geophysical campaign, so they may meet specific objectives requiring a dense grid map.

Equipments specifications and implementation are described in the document « Geotechnical and geophysical investigations for offshore and nearshore developments », published by the ISSMGE (2005).

The **recommended programme of preliminary geophysical reconnaissance** is described in Table 8.

Indicated quantities, which comply with the state of the art, are deemed necessary. However, they may be adjusted depending on:

- available information, such as information gathered during the bibliographic study (DTS);
- proven site complexity.

Particular care should be given to the seismic reflection methods to be implemented when carrying out geophysical reconnaissance. The two following issues should be addressed in particular:

- the choice of the type of source: there are several types of seismic sources, such as electrical (Sparker), electro-mechanical (Boomer), electro-acoustic (Pinger) sources, Chirp (Compressed High Intensity Radar Pulse). Each of them offers different signal frequencies and power. These parameters will affect the accuracy of results, as well as the penetration depth. The choice of the method should therefore be adapted to the penetration and accuracy objectives of the campaign. Furthermore, several sources are often tested at the beginning of the geophysical campaign in order to determine which one will provide the best results. Implementing jointly two systems during the same campaign may prove necessary to meet different penetration and resolution objectives (e.g. Pinger system with an objective of 5 m or 10 m of penetration and 0.2 m of resolution; and Sparker or Boomer system with an objective of 50 m of penetration and 0.5 m of resolution).
- single-trace or multi-trace seismic reflection (UHR). Single-trace seismic reflection is severely limited in penetration because of the phenomenon of multiple reflections between water surface and sea floor. Sea floor multiples appear at a penetration equal to the water height, and it becomes very difficult to identify reflectors beyond this limit. Penetration objectives to be reached when developing offshore wind farms being usually between 50 m and 100 m, with water depths between 15 m and 40 m, single-trace seismic reflection cannot allow reaching the required penetration with enough accuracy. A multi-trace seismic reflection method should usually be implemented for this type of project, since multiples can be removed digitally.

The quality of geophysical recordings also depends on the implementation conditions of equipments and on the characteristics of the naval support. It is commonly admitted that boat speed should remain below 4 knots and that operations should not be carried out on seas where wind exceeds a force of 4 on the Beaufort scale.

Table 5.8 : Recommended programme of preliminary geophysical reconnaissance

Objective	Method	Grid	Penetration	Notes
Sea floor topography	Multibeam bathymetry (MBES)	Full field coverage with a 50% to 100% overlap (T: 20% overlap)	NA	Processing of MBES data by backscattering is recommended Single-trace echo-sounder to calibrate the MBES
Sea floor morphology **Nature of surface sediments**	Side Scan Sonar (dual frequency)	Full field coverage with a 50% to 100% overlap	NA	R: collect samples to calibrate sediments nature: grab sampler (or gravity corer)
Stratigraphy	Single- or multi-trace seismic reflection Source: boomer or sparker for significant penetrations; R: to be complemented with pinger/ chirp for shallow penetrations	250 m x 1000m (cross lines) grid	Typically 50 m-100 m depending on soil/rocks conditions Resolution: < 1 m in depth Pinger/chirp: Resolution: < 0.3 m	Full field coverage Surface seismic reflection required on all cables routes (see paragraph. 5.6.5)

R: specific recommendation T: tolerated

5.6.3.2. PRELIMINARY GEOTECHNICAL RECONNAISSANCE

Preliminary geotechnical reconnaissance should allow establishing on the whole wind farm site a typical geotechnical profile for each geological province that was highlighted during the interpretation of geophysical data:

- stratigraphy;
- nature of soils and identification;
- basic geotechnical features: mechanical strength, deformability, stress history.

These objectives can be met by performing:

- boreholes with the acquisition of intact samples followed by laboratory tests;
- in-situ tests;
- a mix of both.

The recommended programme of preliminary geotechnical reconnaissance is described in Table 5.9.

Naval and reconnaissance means shall meet the proposed objectives.

Equipment specifications, implementation and quality requirements are described in the documents « Geotechnical and geophysical reconnaissance for offshore and nearshore developments » from the ISSMGE (2005), which was previously mentioned, and ISO/DIS 19901-8 (2014)

The campaign should be designed so that it can provide the capital elements for:

- feeding the site geological model. For that matter, boreholes penetration should be sufficient to cross all main formations and understand their configuration at the scale of the site. Depths should be defined by the geotechnical engineer according to the local context. Typically, penetrations from 30 to 50 meters, or even deeper for some boreholes and with specific configurations, are to be considered in connection with results from geophysical data;
- providing the geotechnical parameters required for a pre-dimensioning of the foundations considered for each geological province. It is highly recommended to mix in-situ tests and sampling. The geotechnical parameters profiles are to be established over the influence height of the foundations;
- assessing the variability of geotechnical data on the whole site.

When the number of geological provinces is low, performing twin boreholes may prove worthwhile at this step if soil conditions are favourable. Twin boreholes mean boreholes with continuous coring and boreholes with continuous CPTs performed at a few metres from each other. This method, introduced for offshore oil and gas works (for instance, see Borel and Puech, 2010) allows a good correlation of geotechnical data, and later the extrapolation of data based on CPTs alone, faster and cheaper to perform.

In the case of highly heterogeneous sites, it may prove more relevant during the preliminary reconnaissance to multiply boreholes so that the main geological provinces are covered, by alternating sampling and in-situ tests within a same borehole.

Current international practices show that performing a number of boreholes of about 10% of the number of wind turbines to be installed allows meeting the objectives set on most of the sites. However, this percentage shall not be deemed as restrictive, but rather as indicative, since the volume of investigations to be performed may vary in function of how heterogeneous the site is. In the case of a high number of geological provinces, the required number of boreholes may be significantly higher. It is recommended to allow for sufficient flexibility in the reconnaissance contract to adapt the final programme to the site complexity, as revealed by the first boreholes.

In any case, the expertise of the geotechnical engineer should be called for and considered to optimise the reconnaissance programme.

Table 5.9: Recommended programme of preliminary geotechnical reconnaissance

Objective	Method	Grid	Penetration	Notes
Stratigraphy **Nature of soils and identification** **Basic geotechnical properties** **Typical geotechnical profile for each geological province** **Assessment of the geotechnical properties of materials and their spatial variability**	Coring + Boreholes with in-situ tests, such as CPTU, PMT or HPDT and/or with well logging (natural radioactivity, V_p, V_s, imaging)	Achievement of twin boreholes*: • 1 borehole with continuous coring/sampling • 1 borehole with in-situ tests • At least a couple of boreholes for each geological province and/or Single boreholes such as: • alternated borehole** CPTU/coring/sampling • borehole with CPTU as continuous as possible if relevant • borehole with continuous coring/sampling and well logging To be distributed on the whole field to establish the spatial variability of the site	Sufficient to: 1. cross the main formations and understand their configuration at the scale of the site 2. establish profiles of geotechnical parameters over the height of the influence of foundations	* Prioritise twin boreholes if relevant and low number of provinces ** Alternated boreholes may prove financially attractive in the preliminary stage

5.6.4. DETAILED RECONNAISSANCE

The project stage entails two steps:

- the design step, which shall allow characterising the major hazards, and after which the geotechnical parameters shall be known with enough accuracy to proceed to the dimensioning, individually or by group, of the wind turbines. Validating the construction means, costs and schedule should be made possible;
- the construction step during which detailed construction studies will be carried out.

Detailed reconnaissance aims at meeting all the needs of the project stage. A single detailed geophysical reconnaissance and a single detailed geotechnical reconnaissance will most often meet the objectives. However, additional reconnaissance may prove necessary during the achievement stage to remove uncertainties raised from minor or localised risks.

5.6.4.1. DETAILED GEOPHYSICAL RECONNAISSANCE

The detailed geophysical reconnaissance aims at completing the geophysical reconnaissance previously achieved during the project draft stage. The campaign objectives are the following:

- providing more accurate data (bathymetry, sea floor morphology, obstructions) about the structures locations;
- complementing the existing seismic reflection data below the structures, with specific objectives of penetration and resolution;
- providing additional data using « geophysical engineering » methods (seismic refraction, surface waves, electrical resistivity). These methods will only be used if objectives demand it.

Table 5.10 indicates the type of recommended programme of detailed geophysical reconnaissance.

Table 5.10: Recommended programme of detailed geophysical reconnaissance

Objective	Method	Grid	Penetration	Notes
Sea floor topography	Multibeam bathymetry (MBES)	Coverage of each structure location with overlap of 100%	NA	Size depends on the type of structure (wind turbines, meteorological mast, transformation substations and cables)
Sea floor morphology Surface obstructions	Side Scan Sonar (dual frequency)	Coverage of each structure location with overlap of 100%	NA	Size depends on the type of structure (wind turbines, meteorological mast, transformation substations and cables)
Stratigraphy	Single or multi-trace seismic reflection Source: • boomer or sparker for significant penetrations • chirp for small shallow penetrations	Two perpendicular lines for each structure	Depending on the type of foundation and on specific objectives	
Measurement of the velocity of compression waves V_P by seismic refraction	Refraction (dragged on the sea floor or static)	On structures locations: to be defined according to objectives Cable route: continuous profile	5 m to 20 m depending on objectives 5 m	
Measurement of shear wave velocity V_S by surface waves	MASW	On structures locations: to be defined according to objectives	5 m to 15 m depending on objectives	

5.6.4.2. DETAILED GEOTECHNICAL RECONNAISSANCE

The final dimensioning of the foundations and the installation studies assume the definition of a profile of geotechnical parameters applicable below each wind turbine.

By principle, it is required during the **detailed geotechnical campaign** to carry out at least one representative borehole for each wind turbine location regardless of the considered type of foundation.

The number of representative boreholes may be exceptionally reduced if it can be demonstrated that the site, in whole or in part, is homogeneous enough to interpolate geotechnical data at some locations. This demonstration should be founded on a high-quality geological model, a detailed risk assessment, and a thorough integration process of the geotechnical and geophysical data. Methods from geostatistics may prove useful.

On sites characterised by a strong geophysical and geotechnical heterogeneity, and in the event where gravity foundations are considered, it shall be necessary to carry out at least three peripheral boreholes in addition to the deep « central » borehole, to ensure that subsurface soil conditions are homogeneous over a depth of at least 10 m or until refusal (CPT). If foundations equipped with skirts are considered, it will be necessary to ensure that the investigation depth equals at least the penetration of the skirts, plus 2 metres.

For piled foundations, the influence height of the foundations is at least equal to the pile penetration (length of the shaft) increased by the influence zone of the tip. The latter is usually estimated at 3 diameters for piles of common diameters (< 2 m). For monopods with piles of very wide diameter, where capacity is essentially ensured by friction, the influence zone under the pile may be limited to half of the pile diameter.

For gravity base foundations, the influence zone related to the bearing capacity may be limited to the depth of the deepest failure line matching the characteristics (inclination) of the maximum applied load. The influence zone with respect to settlements may be significant in compressible soils and reach up to 1.5 times the foundation diameter. In any case, in the presence of a substratum, the influence zone may be limited to the depth of this substratum.

Table 5.11 indicates the type of recommended programme of detailed geotechnical reconnaissance.

When pile foundations are considered in soils where no proven dimensioning solution is available (for instance: carbonate or volcanic soils, chalks, soft rocks), it may be relevant to carry out one or several loading tests on one or several test piles beforehand. The test pile(s) should be installed using the method considered for the wind turbines foundations. Ideally, tests should be carried out on the same site as the one of the wind turbines. However, given the high cost of offshore tests, it may be relevant to carry out the tests on a land site showing similar features and at a reduced scale.

When pile driving in rocky or hard formations is considered, it may be relevant to carry out one or several feasibility tests beforehand, to make sure that driving the piles will be possible and to guarantee their structural integrity. Ideally, tests should be carried out on the same site as the one of the wind turbines. However, given the high cost of offshore tests, it may be appropriate to carry out the tests on a land site showing similar features and a reduced scale.

In the case where tests are considered (onshore or offshore) at a reduced scale, scale effects should be taken into account. In geotechnics, scale effects arise from not respecting the stress conditions between the scale model and the actual foundation, and/or not respecting the relative size of soil elements with respect to the model dimensions. The consequences are biases on stresses and/or on strains measured on the model that simply cannot be extrapolated to the actual foundation. In the case of field tests at reduced scale (onshore or offshore), the soil material for the scale model and the soil material for the foundation are deemed identical. One should ensure that the model dimensions are close enough to the ones of the foundation, in order that stress levels and the relative dimensions of the model are not too distorted so that a direct extrapolation of the observed phenomena and measured quantities is possible. For most of the issues considered, a scale reduction from 1/2 to 1/3 may be deemed acceptable. The pile should have geometrical (ratio of driven length/diameter) and structural (ratio of pile diameter/tube thickness) properties that are compatible with the driving induced phenomena (plug formation, risks of structural instability).

Table 5.11: Recommended programme of detailed geotechnical reconnaissance

Objective	Method	Type of foundation	Programme	Penetration
Final design of foundations **Installation studies**	Coring / sampling boreholes Boreholes with in-situ tests such as CPT/CPTU Boreholes with in-situ deformation tests (PMT, HPDT) Mixed boreholes with alternating coring/sampling and in-situ testing	PILED	1 borehole at the centre of each wind turbine location	Anticipated piles lengths + 3D minimum
		MONOPILE	1 borehole at the centre of each location	Anticipated monopiles lengths + 0.5D minimum
		GRAVITY BASE	1 borehole at the centre of each location + 3 boreholes on the periphery*	1.5 x foundation width or penetration of at least 2 m in the substratum At least 10 m penetration or until refusal (CPT)
		SHALLOW WITH SKIRTS	1 borehole at the centre of each location + 3 CPT boreholes on the periphery	1.5 x foundation width or penetration of at least 2 m in the substratum Skirts height + 2 m; min. 5 m
		ANCHORING	1 borehole at each anchor location	Depending on the anchor type and nature of soils

* in case of a strong geological or geotechnical heterogeneity

5.6.5. CABLES ROUTES

Cables routes are spread out between wind turbines on the wind farm itself, and between the wind farm and the coast. Cables are most often buried (within the limits of technical/financial constraints) so that their protection is ensured, their stability is guaranteed and/or the sea floor remains free of obstructions. Burial depths will usually not exceed 2 metres, except, for instance, on vessel anchorage areas or in the influence zone of maintained channels.

Reconnaissance for cables routes should be carried out in two steps.

The **first step** aims at:

- providing directions for the orientation of cables corridors;
- assessing the risks incurred by the cables and define their protection level;
- defining the target depth of burial;
- determining the feasibility of the laying and burying means.

This first step usually occurs during the Project Draft stage (Table 5.7). It is composed of a geophysical reconnaissance complemented by a light geotechnical reconnaissance.

5.6.5.1. FIRST STEP RECONNAISSANCE

In principle, the **geophysical first step reconnaissance** should entail:

- bathymetric recordings and side scan sonar surveys on the entirety of the wind farm and planned cables routes areas;
- sub-surface seismic surveys on a few standard lines, selected because of their particular interest (wind turbines alignment, planned cables route between the site and the coast, etc.).

The means to be implemented are, in nature, similar to the ones used for the preliminary geophysical reconnaissance on the site. A single preliminary geophysical campaign is usually carried out that should allow meeting the objectives set for the cables routes. However, seismic reflection means should be selected so that accuracy, rather than penetration, is prioritised for the first metres below sea floor.

Geophysical reconnaissance should be completed by a direct characterisation of the materials present on the first metres of seabed. Depth will range between 1 and 5 metres, with a common target depth of 3 metres. In addition to the information gathered from deep boreholes, the preliminary determination of the physical and mechanical properties of both surface and subsurface soils may be obtained by using light geotechnical tools, i.e. those that do not require significant naval means (or that can be achieved onboard the ship used for geophysical reconnaissance):

- grab-sampler (limited to identifying surface soils);
- gravity corer;
- vibrocorer;
- CPT operated from a seabed frame;
- rotary corer operated from a seabed frame (in the case of a rocky seafloor).

Strictly speaking, the sampling frequency should depend on the lateral variability of sediments. At this point, the latter remains a priori unknown. It may be assumed that a statistical assessment of the properties of the soils concerned by cables burial can be approached by obtaining a few tens of boreholes. These boreholes should be adequately distributed, either on the whole windfarm site and the planned site-to-shore route if precise cables routes are not defined at this point, or more directly on the routes themselves if they have been pre-established. Determining the number of boreholes and their locations should be done on the basis of the geophysical recordings. Information gathered from the deep boreholes locations may be used, but will not necessarily provide relevant data over the first few metres.

Thermal conductivity measurements, which are usually required for the dimensioning of power cables, may complement the geotechnical reconnaissance. They may be carried out either in-situ or in the laboratory.

Table 5.12 : Recommended programme of preliminary reconnaissance for cables routes

Objective	Method	Grid	Penetration	Notes
Sea floor topography	Depending on Table 5.8			
Sea floor morphology Nature of surface sediments	Depending on Table 5.8			
Stratigraphy	Depending on Table 5.8 Prioritise accuracy over penetration on the first 5 to 10 metres			
Characterisation of the nature and strength of soils and rocks over the anticipated cables burial depth	Depending on context: • Gravity coring, vibrocoring, • CPT/CPTU rotary coring carried out from a seabed frame	Typically: 20 to 30 borehole locations for a 100km² site	Most often: 2 to 3 metres depending on the planned burial depth; exceptionally: up to 5 metres	Often carried out during the geotechnical preliminary reconnaissance
Thermal Insulation	Thermal conductivity measurement: made in-situ by using a probe set by push penetration or on sampled cores	A few measurements for each geological province	Most often: 2 to 3 metres depending on the planned burial depth	

5.6.5.2. SECOND STEP RECONNAISSANCE

The **second step** aims at:

- enabling cables routing within corridors previously defined;
- confirming / specifying the burial target depths in function of the desired protection, as well as their variations along the route;
- determining the burial tools that are best suited to soils conditions (adequate method, type of tools and machines, required power);
- forecasting operational conditions (notably, rate of progress) and their variations along the cables route;
- identifying areas requiring specific processes (rock outcrop, obstruction to avoid, etc...).

This second step will occur during the **design stage**.

It should be composed of:

- a geophysical reconnaissance using high resolution seismic reflection on the cables corridors previously defined;
- a specific geotechnical reconnaissance along the defined cable route.

This step may possibly be followed by burial tests aiming at demonstrating if a particular method can be implemented, or at comparing the efficiency of several methods.

Routing should preferably be carried out prior to geotechnical reconnaissance, so that borehole locations are precisely localised on the planned route.

If an UXO constraint exists on the site, UXO reconnaissance may be (with reserves related to how valid it is over time) carried out prior to the routing, so that the number of magnetic anomalies to be identified can be optimised during routing.

Geophysical reconnaissance includes performing bathymetric and high resolution side scan sonar surveying. If required, this may be complemented by shallow seismic reflection and by the acquisition of "engineering geophysic" data (surface seismic refraction).

Geotechnical reconnaissance includes boreholes along the axis of the cables routes, using CPTs and/or coring boreholes (or vibrocoring), adequately alternated or twinned, so that a geotechnical profile can be obtained for each location on the first three metres of penetration. Boreholes frequency should be adapted to the conditions of the site. A spacing of 500 m to 1,000 m may be acceptable on sites deemed homogeneous. On sites with complex subsurface geology, gathering information every 300 m may be relevant. Data collected, whether it is of a geotechnical or geophysical nature, should then be correlated in order to produce a ground model as continuous as possible along the route and over the burial depth.

Geophysical seismic refraction systems dragged on the sea floor allow characterising soils in terms of compression waves velocities (V_p). Obtaining a continuous profile of velocities along the cables routes greatly facilitates the integration of data, and the constitution of the ground model. Implementing these methods is particularly recommended when soils conditions are deemed difficult regarding cables burial, notably when rocks or shallow hard layers are expected.

Particular attention should be given to the following:

- if hard soils conditions are present at surface or close to the surface, geophysical methods based on seismic reflection will not allow defining soils conditions with sufficient accuracy for the needs of a burial study;
- an irrelevant or insufficient reconnaissance will most often result in operational difficulties, loss of time and significant costs overruns during the burial works.

5.6.6. SUB-STATION

A network of submarine cables allows interconnecting turbines and carrying the whole production towards one (or several) substations located within, or next to, the wind farm. The role of a sub-station is to centralise power production and recondition it for cable exportation to shore.

Sub-stations are relatively heavy structures (transformers) that are usually constituted of jackets founded on piles (driven or drilled and grouted).

Geophysical and geotechnical reconnaissance of soils for the installation of sub-stations may be combined with the various other campaigns (preliminary and detailed ones) achieved for the wind turbines. Tables 5.8 to 5.11 provide indications on that matter. The methodology and means to be implemented are identical.

Depending on the soils complexity at the sub-stations locations, defining the profile of soil parameters for the design of their foundations should be based at least on data sets acquired from an alternated borehole (in-situ tests and sampling with laboratory testing) or from two twinned boreholes: one with in-situ tests and the other with sampling and laboratory testing.

5.6.7. METEOROLOGICAL MAST

Installing a meteorological mast on a wind farm site is common, but not mandatory. A meteorological mast is usually constituted of a light latticed structure.

The foundations of the meteorological mast are most often constituted of a monopile or of a latticed structure secured by piles. In the event where the meteorological mast would be installed very early in the development process of the wind farm, it may be used as a test bench for the future turbines foundations.

Planning reconnaissance for a meteorological mast installation and for future turbines foundations is usually not compatible.

Most often, it will be necessary to schedule a specific geotechnical campaign targeting the area selected for the mast installation. This campaign will be similar to the preliminary geophysical campaign designed for wind turbines (see paragraph 5.6.3 and Table 5.8) and will include bathymetric recordings, surveys made using a side-scan sonar on an area of about 1 km², and seismic surveys of subsurface on a few lines crossing at the planned location of the support.

Furthermore, it should be ensured that at least one alternated geotechnical borehole is made at the mast location (in-situ tests and sampling with laboratory tests). The penetration of this borehole will depend on the planned type of foundation (see paragraph 5.6.4 and Table 5.11).

In the event where the meteorological mast would be installed during later stages, the corresponding reconnaissance could be integrated within the preliminary reconnaissance stage of wind turbines.

Table 5.13: Recommended programme of additional standard reconnaissance for the second step of cables routes

Objective	Method	Grid	Penetration	Notes
Sea floor topography	Multibeam bathymetry (MBES)	200 m corridor* centred on the cable axis, with a 50% to 100% overlap	NA	*Corridor width to be defined in function of the hetero-geneity of the subsurface geology and of density of obstructions
Sea floor morphology **Nature of surface sediments subject to appropriate signal processing (backscattering)**	Side Scan Sonar	200 m corridor* centred on the cable axis, with a 100% overlap	NA	* Corridor width to be defined in function of the hetero-geneity of the subsurface geology and of density of obstructions
Stratigraphy	HR seismic reflection Source: to be defined depending on geology (pinger /chirp)	One run on the cable axis and two runs at a 100 m distance from each other. Even transversal cross-checks (about 300 m to 500 m)	Prioritise accuracy on the first 3 to 5 metres	
Characterising continuously the soils conditions over the burial depth by using sound velocities (V_P, V_S)	VHR seismic refraction implemented very close to the seafloor (system dragged on the seafloor or towed just above seabed) Optional: mix seismic refraction and MASW measurements	One run on the cable axis	3 to 5 m	Seismic streamers will be of the short type (typically: 24 m) with a minimum of 24 geophones spread so that they will collect as many information as possible on the first 2 to 3 meters
Characterising punctually the nature and strength of soils and rocks over the foreseeable burial depth	CPT/CPTU carried out from a seabed frame Gravity coring, vibrocoring, rotary coring from underwater boreholes	One borehole every 300 to 1000 m depending on the complexity of the sub-surface geology	Most often: 2 to 3 metres, depending on the planned burial depth Exceptionally: up to 5 metres	
Thermal insulation*	Thermal conductivity measurements: in-situ with a probe installed by push penetration or on samples	A few measurements for each geological province	Most often: 2 to 3 metres, depending on the planned burial depth	

* if needed and not obtained during the preliminary stage

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6. SOIL PARAMETERS AND DESIGN PROFILES

6.1 INTRODUCTION

6.2 PRINCIPLES OF DETERMINATION OF DESIGN PROFILES

6.3 USUAL PARAMETERS

6.4 DEFORMABILITY PARAMETERS

6.5 CYCLIC PARAMETERS

6.6 REFERENCES

6. SOIL PARAMETERS AND DESIGN PROFILES

6.1. INTRODUCTION

The chapter 5 « Field Studies » outlines the objectives of seabed and soils reconnaissance, defines the means to be implemented and the scheduling of operations, and describes how to handle the flow of data that is progressively collected. The final result is a geotechnical model that offers, at each borehole location, a partition with depth in geological units, each of them being divided into layers and sub-layers that are identified by their geotechnical features.

The objective of this chapter is to specify the steps that lead from this purely factual information to the design of foundations.

6.2. PRINCIPLES OF DETERMINATION OF DESIGN PROFILES

6.2.1. OVERVIEW

The dimensioning of foundations requires to obtain at each structure location:

- a partition of the ground into geotechnical units all over the influence zone of the foundation;
- the definition for each geotechnical unit of a set of geotechnical parameters that is representative of the ground behaviour in relation to the applied loading.

There is a very high interdependency between the refinement of partition into units and the representativeness of the geotechnical parameters assigned to each unit.

The process of establishing the design profiles may be divided into four main steps:

1. synthesising the available geotechnical data;
2. discretising the ground profile into geotechnical units;
3. highlighting for each unit the characteristic values representative of the ground behaviour and relative to the limit states to be considered;
4. choosing geotechnical design parameters directly applicable to designing the considered structure.

The remainder of the chapter describes these different steps.

6.2.2. SYNTHESIS OF THE FIELD DATA AND CRITICAL ANALYSIS

This step focuses on gathering, critically analysing and updating all the geotechnical data previously collected during the successive stages of reconnaissance.

No matter which data management system is chosen – an available geographical information system (GIS) or one's own organisation format – it is advised to maintain an organisation that is open (where data may be read without using a specific software), geo-referenced (with an access being possible by localisation) and scalable (that can be improved over time while keeping previous versions readable).

The gathering of data is applicable to the whole site and to the various campaigns carried out by different actors. The characterisation of soils and rocks should not be limited to the available information at each structure location, which are usually limited because of the relatively small number of tests that can effectively be done in each borehole, but to the whole site, by including information from other wind turbines locations and information from reconnaissances of cable routes. This critical analysis should apply to the whole sets of in situ and laboratory data.

The critical analysis of in situ tests should take into consideration:

- how relevant the test is in relation with the parameter that is looked for and the type of soil;
- the correct execution of the test (compliance with standards and operating procedures);
- the certification of the device (calibration sheets…);
- the quality of interpretation;
- the consistency of results between several types of tests.

The critical analysis of laboratory data should take into consideration:

- the quality of the sample: notably by relying on the analysis of recovery rate, photographic observation, geotechnical descriptions and comments made by operators during sea or laboratory operations. The quality of sampling in clays may be favourably assessed using methods based on the variation of the void ratio (as developed by Lunne et al., 1998). In clays, chalks and in some instances rocks, imaging methods (X rays, tomography) may produce useful additional data that can help when collecting samples for mechanical testing;
- the quality of transportation, storing and unpacking, as well as their potential impact on the disturbance of the sample (see: ISO 19901-8, 2014);
- the pertinence of the specifications of the testing programme: for instance, level of the applied confining and cyclic stresses, value of K_0 in anisotropic tests;
- the quality of performance of the test: laboratory accreditation, calibration of the measuring devices, quality control from the laboratory, documentation of the method used to prepare samples, description of results;
- the quality of interpretation: applicability of the interpretation method being used, representation of results and possibility of further representation;
- representativeness of the test: localisation in the unit, harder or softer zone.

This final (and challenging) evaluation requires obtaining the opinion of the person in charge of the laboratory test programme, the assessment of the consequences of a possible insufficient sampling, the comparison of the test result with other tests on the same unit, the comparison of the result with other types of characterisation tests (in-situ tests, laboratory test of another type, or possibly geophysical measurement). The objective is to establish whether, and how, the test (or the series of tests) improves our understanding of the ground behaviour. If the test is inconsistent with the other tests within the unit, the cause of discrepancy should be investigated: quality bias (see above), scale effect, attachment to another ground unit, sub-unit having a different geotechnical behaviour.

In the case of different reconnaissance campaigns, or of different laboratories that have been working on the same site, an additional step to homogenise the representation (graphs, units) or to interpret the tests results may be required.

During that step, results from tests deemed as having a low representativeness or a suspect reliability shall be clearly identified, and they shall be attributed a lesser weight when establishing the soil parameters.

6.2.3. ESTABLISHING THE SOIL PROFILE

The purpose here is to establish the model of spatial sequencing of the tests results. This dual sequencing, vertical and lateral, aims at defining the units and, if necessary, the sub-units, that show mechanical characteristics that are as homogeneous as possible.

The geological model inferred from the seismo-stratigraphic model, described in paragraph 5.5, represents a first degree of sequencing. In sedimentary formations, the mechanical characteristics of soils and rocks are tightly bound to their deposition and consolidation processes. However, a weathering process or a chemical combination may occur within a unit, and override the sedimentary factor. In that case, the geological discretisation is insufficient and requires additional discrimination with respect to weathering. Depth below sea bottom, distance to faults or content of some mineral components are worth investigating. The knowledge of the site geological history as well as the understanding of the phenomena conditioning the mechanical characteristics and their in-situ variability should then help grouping the tests results. It should be ensured that the final site model remains consistent with the geological data.

The stratigraphic partition resulting from the geological considerations is usually insufficient to be used as a geotechnical model. The geotechnical model should be sufficiently detailed to attribute to each unit physical and mechanical characteristics showing a high degree of homogeneity. Degree of homogeneity here means that the whole population of parameters used to characterise the material of the unit possesses consistent and comparable properties. When the population reaches a sufficient size, statistical processing should produce a reasonable standard deviation and the mean value make sense with respect to the foundation design process. Deviations that are too great between the extreme values of the classification parameters such as void ratio or unit weight, or of mechanical parameters such as shear strength, should prompt to redefine or refine the units partition.

The number and type of tests to be spatially represented remain at the discretion of the geotechnical engineer, but at the very least the main classification (moisture content, unit weight, carbonate content) and resistance tests should be considered. In particular, it should be useful at this step to know the parameters that are the most sensitive for the type of foundation considered.

Creating a unit (or sub-unit) should be determined in relation to:

- its capacity to be physically and mechanically qualified by a sufficiently representative number of tests;
- the engineering issues that should be addressed: typically, a layer of low thickness (in an order of decimetres) and of poor mechanical properties will have a greater effect on a gravity foundation, for which sliding issues may occur, than for the design of piles, for which friction and lateral resistance are little sensitive to local variations.

In each of the geotechnical units that are defined, the parameters attributed to the unit should possess a dual consistency:

- firstly, the parameters of a same category should be compatible with each other;
- secondly, parameters of different categories should be correlated.

Parameters categories consist of classification parameters, state parameters, resistance parameters and deformability parameters.

State or classification parameters may be measured independently or in relation with each other. The consistency of the data chain should be preserved by applying the elementary relations of soils mechanics (dependent parameters) or by searching correlations of the same rank (i.e., between the parameters of a same category).

Resistance or deformability parameters originate from different sources (various in-situ tests, more or less complex laboratory tests). It is essential to establish the consistency of the parameters deriving from these different sources. Works will deal with:

- the role of the resistance anisotropy on the values of shear strength: one should distinguish shear strengths measured in direct simple shear test (DSS), triaxial tests in compression (TX_C), and triaxial tests in extension (TX_E). If possible, the corresponding coefficients of resistance anisotropy are established;
- using the correlations of same rank to compare resistances from laboratory tests and from in-situ tests (VST, CPT, PMT). The correspondence coefficients should be established (for instance the cone factor N_k);
- using correlations of same rank to compare deformability parameters from laboratory tests and in-situ tests (CPT, PMT, HPDT). A particular attention should be given to the distortion level assigned to each measurement, and to the level of stress associated to this measurement, notably in sands;
- using correlations of different ranks to check the consistency between the state parameters and the resistance or deformability parameters.

The final soil model should be comprehensive, i.e. it should contain the entirety of the available information, and should maintain consistency throughout the different sources.

6.2.4. CHARACTERISTIC VALUES

The notion of characteristic value of a geotechnical parameter is thoroughly developed in documents such as ISO 19900 (2014), Eurocodes (EC0 and EC7) or DNVGL-RP-C207 (2017).

The selection of the characteristic values of the ground properties shall meet several criteria:

- the selection of the characteristic values of the ground properties shall be based on values that are directly obtained or on values derived from the entire relevant sets of laboratory or in-situ tests;
- the characteristic value of a geotechnical parameter shall be chosen as a cautious estimate of the value affecting the occurence of the limit states under consideration;
- depending on the limit state under consideration, the chosen characteristic value may be a lower one, smaller than the most probable value, or an upper one, larger than the most probable value. For each calculation, the most unfavourable combination of lower or upper values of independent parameters shall be chosen;
- the ground zone governing the behaviour of a foundation under a particular limit state is usually much larger than the dimensions of a soil sample being tested in a laboratory, or than the influence zone of an in-situ test. The value of the parameter governing the limit state is therefore the average of a set of values obtained in a significant volume of ground. The characteristic value to be chosen shall therefore be a cautious estimate of this average;
- when a layer exhibits parameters that obviously vary with depth (e.g. shear strength in a normally consolidated clay), a trend should be initially defined, and the concept of characteristic value should be applied over this trend (e.g. by defining a value at the top of the layer and a gradient).

When selecting the characteristic values of the ground properties, one should take into account:

- all the available information from the geological, hydrogeological and geotechnical fields, including the information obtained prior to the current project;
- the variability of the property being measured, as deduced from current measurements or resulting from local experience;
- the reliability and representativeness of the test method and of the results that were obtained;
- the scope of in-situ and laboratory investigations, and the sampling frequency;
- the type of tests that are carried out and their representativeness with respect to the parameter considered;
- the amount of data that can be assigned to the zone governing the limit state under consideration.

The characteristic value, under the ISO 19900 (2014) standard, is defined in relation to the statistical distribution of values. Three characteristic values are considered:

- low estimate (LE): it is applied when the most unfavourable design condition is generated by low values. It is chosen so that only 5% of tests results are lower than the characteristic value (5% quantile);
- best estimate (BE): it is applied when one wishes to obtain the most probable value of the parameter. It represents a mean value if the distribution is normal, and a median one in all other cases;
- high estimate (HE): it is applied when the most unfavourable design condition is generated by high values. It is chosen so that 95% of tests results are lower than the characteristic value (95% quantile).

Offshore wind turbines design practice has led to adapt these notions and introduce two additional characteristic values: the conservative estimate (CE) and the optimistic estimate (OE).

Both values are justified on the basis of the interpretation of paragraphs 7.4.2.1 to 7.4.2.9 of DNVGL-ST-0126 (2016) and the clause 2.4.5.2 of EC7 suggesting that, when small volumes of ground are being loaded, calculations should be based on the local soil properties with their full variability (it is typically the case for the end bearing capacity of piles) whereas when large volumes of ground are taken into account, averaged fluctuations of the soil properties from one point to another may be applied (it is typically the case for the bearing capacity of a gravity base).

Table 6.1 summarises the characteristic values that should be considered when dimensioning offshore wind turbines.

The concept of characteristic value, as outlined above, is based on the study of statistical distributions. The validity of statistical studies assumes that the population of samples is large (more than 20 samples either way).

A recurring issue in geotechnical studies is that populations of samples (the number of measurements to be associated to each layer) are often limited. In that case, it is recommended to rely on the expertise of the geotechnical engineer, who can assess the representativeness of data, the natural variability of the unit, and the sensitivity of the soil property to the occurence of the limit state under consideration. When defining the characteristic values, the geotechnical engineer should use the above general concepts as guidelines.

6.2.5. SELECTING THE DESIGN PARAMETERS

Each design value is associated to the combination of a limit state, a deformation or failure state, a representative ground volume and a characteristic value.

The recommended design parameters for the different dimensioning calculations of the foundations of offshore wind turbines are outlined in:

- Table 6.2 for foundations on piles and monopiles;
- Table 6.3 for foundations on gravity bases.

The design value of the unit weight of soils and rocks (γ_h, γ', γ_d, γ_s) is a best estimate (BE) in all types of calculations.

For the analysis of the behaviour under cyclic loads, degradation laws (shear strength, friction, displacements) should be applied on the conservative estimate (CE) of the corresponding static parameter.

Table 6.1 : Definition of the characteristic values

Characteristic value	Designation	Quantile	Comments
Low estimate	LE	0-10	The interval is centred on the quantile at 5% of the distribution (see ISO 19900,2014, EC7 or DNVGL-RP-C207, 2017) When the distribution is highly uneven, it is required to call on the geotechnical engineer's expertise to remove values deemed as abnormal, while still producing a reasonable minimum value.
Conservative estimate	CE	25-45	This value may be interpreted as the one corresponding to the recommendations of § 7.4.2.1 to 7.4.2.9 of DNVGL-ST-26 (2016) and to the clause 2.4.5.2 of EC7. For layers that are well characterised, and for limit states related to global behaviours controlled by a large volume of ground, this value may be estimated as being in the order of BE – 0,5 to 1,0 σ (σ = standard deviation) in function of the number of data and how they are scattered.
Best estimate	BE	50	It is the mean value for a normal distribution, and a median value for other distributions.
Optimistic estimate	OE	55-75	Same considerations than for the Conservative Estimate, but on the side of values higher than the average.
High estimate	HE	90-100	The interval is centred on the quantile at 95% of the distribution (see ISO 19900, 2014, EC7 or DNV-RP-C207, 2017) When the distribution is highly uneven, it is required to call on the geotechnical engineer's expertise to remove values deemed as abnormal, while still producing a reasonable maximum value.

Table 6.2: Design values suggested for the design of piles or monopiles

	Limit state	Shear strength: s_u, tgφ', q_t (1), p_l*(2)	Unconfined compressive strength of rocks: σ_c (with or without a mass factor)	Soil-pile friction: tg δ_{cv}	Deformation moduli: G_0, G(γ) ou E_0, E(ε), E_M (with or without a mass factor in rocks)	ε_{50}	ν K_0
Axial capacity							
Skin friction	ULS	CE	CE	CE			BE
End bearing	ULS	LE	LE				BE
Axial performance	SLS	CE	CE	CE	CE		BE
Lateral reaction							
Design of piles	ULS	CE	CE		CE	BE	BE
Displacements in service	SLS	CE	CE		CE	BE	BE
Verification of natural frequencies; foundations stiffness							
Upper bounds (3)		OE	OE	OE	OE	OE	BE
Lower bounds (3)		CE	CE	CE	CE	CE	BE
Load calculations; foundations stiffness							
Ultimate loads (4)	ULS	BE	BE	BE	BE	BE	BE
Fatigue loads	FLS	BE	BE	BE	BE	BE	BE
Installation							
Most probable conditions		BE	BE	BE			BE
Most unfavourable conditions (5)		HE	HE	HE			HE

(1) specific case of CPT methods
(2) specific case of PMT methods
(3) in function of the project stage, and of the suspected heterogeneities, a sensitivity analysis may be carried out on the parameters with the highest effect, using the LE and HE conditions
(4) ultimate loads are calculated by practitioners with the BE conditions, but a parametric study with the CE and OE conditions, at least at the preliminary stage, may be cautiously required
(5) some SRD assessment methods include a conventional increase of the parameters in the process of the calculation of the maximal SRD. In that case, OE parameters may be selected in order not to cumulate an excess safety.

Note : For Accidental Limit States (ALS), essentially related to ships collisions (see paragraph 7.1.1, it is suggested to adopt conservative estimates (CE) for the calculations of capacities and displacements.

Table 6.3 : Design values suggested for the design of gravity bases

	Limit state	Shear strength: s_u, tgφ', q_t (1)	Unconfined compressive strength of rocks: σ_c (with or without a mass factor)	Soil-base friction	Deformation moduli: G_0, G(γ) or E_0, E(ε) (with or without a mass factor in rocks) M, C_c, C_g	Consolidation and creep parameters k_h, k_v, c_v C_α	ν K_0
Bearing capacity	ULS	CE	CE				BE
Sliding	ULS	CE	CE	CE			BE
Primary and secondary settlements	SLS				CE	CE	
Displacements in service	SLS	CE	CE	CE	CE		BE
Verification of natural frequencies; foundations stiffness Upper bounds (2) Lower bounds (2)		OE CE	OE CE	OE CE	OE CE		BE BE
Load calculations; foundations stiffness Ultimate loads (3) Fatigue loads	ULS FLS	BE BE	BE BE	BE BE	BE BE		BE BE
Installation (skirts) Most probable conditions Most unfavourable conditions		BE HE	BE HE	BE HE	BE HE		BE HE

(1) for some approaches (for instance: skirts penetration)
(2) in function of the project stage, and of the suspected heterogeneities, a sensitivity analysis may be carried out on the parameters with the highest effect, using the LE and HE conditions
(3) ultimate loads are calculated by practitioners with the BE conditions, but a parametric study with the CE and OE conditions, at least at the preliminary stage, may be cautiously required

Note : For Accidental Limit States (ALS), essentially related to ships collisions (see paragraph 7.1.1), it is suggested to adopt conservative estimates (CE) for the calculations of capacities and displacements

In rocks, the rigidity of the rock mass is not equal to the rigidity of the intact rock matrix and should be corrected with a mass factor as outlined in the paragraph 6.4.5. The same type of correction may be applied to the unconfined compressive strength.

For the calculation of natural frequencies, the notion of low estimate (LE) should integrate the degradation of the deformation moduli induced by the accumulation of cyclic loads applied over time.

6.3. USUAL PARAMETERS

6.3.1. CLASSIFICATION PARAMETERS

The main classification parameters of soils or rocks are listed below:

- particle size from sieving analyses and hydrometer tests: grain size distribution, uniformity coefficient C_U;
- grain characteristics: angularity, abrasivity;
- specific weight of particles: γ_s;
- Atterberg limits: liquid limit w_L (%), plastic limit w_P (%), plasticity index $I_P = w_L - w_P$ (%);
- carbonate content: $CaCO_3$ (%);
- organic matter content: MO (%).

These parameters are obtained from normalised laboratory tests (ASTM, BS, NF), whose measurements are independent from each other.

6.3.2. PARAMÈTRES D'ÉTAT

The basic and derived state parameters are listed below:

- void ratio: e, or porosity: n;
- moisture content: w (%);
- unit weight: saturated: γ_{sat}; dry: γ_d; submerged: γ';
- density index I_D for granular materials;

- liquidity index I_L, or consistency index I_c for consistent materials
- degree of overconsolidation: OCR;
- coefficient of earth pressure at rest K_0;
- coefficients of vertical and horizontal permeability: k_v, k_h.

State parameters are obtained from normalised laboratory tests (ASTM, BS, NF), whose measurements are either independent or interdependent. For instance, there are constitutive links between porosity, saturated moisture content and saturated unit weight, or also degree of overconsolidation and coefficient of earth pressure at rest. Some parameters (K_0, coefficients of permeability) may be obtained from in-situ tests (e.g. PCPT).

6.3.3. RESISTANCE PARAMETERS

The resistance of geomaterials to monotonous loading (also called static resistance) may be measured with a wide range of laboratory tests or normalised in-situ tests.

As a reminder, the resistance of a material is not an intrinsic value, but may depend on several parameters such as:

- the nature of the loading: drained or undrained;
- the rate of loading;
- the mode of loading: direct simple shear, compression, extension;
- the initial state of the sample (state of consolidation, state of the initial stress).

The usual laboratory tests used to measure the resistance of rock samples are:

- the unconfined compressive strength test (or UCS): σ_c;
- the Brazilian test;
- the point load strength test.

The usual laboratory tests used to measure the resistance of soils are:

- the direct shear box test;
- the direct simple shear test (DSS);
- the compression triaxial test (TX_C), which may be unconsolidated and undrained (UU), isotropically consolidated and sheared in undrained conditions (CIU) or drained conditions (CID), anisotropically consolidated and sheared in undrained conditions (CAU or CK_0U) or drained conditions (CAD or CK_0D). The sample may also be sheared in extension (TX_E), in drained or undrained conditions. It is critical that the testing conditions be perfectly specified and the obtained resistance be clearly identified.

The static shear resistance of soils may be measured or derived from in-situ tests, notably:

- the vane shear test (VST) in soft to firm clays. This test provides a direct measurement of the undrained cohesion c_u;
- the cone penetration test (CPT) in soft to hard clays and loose to very dense sands. Shear strength is characterised by the measurement of the corrected cone resistance q_t;
- the Ménard pressuremeter test (PMT) in soils and weak rocks. Shear strength is characterised by the measurement of the net limit pressure pl*.

There are several correlations between in-situ and laboratory tests (for instance, see: Lunne et al., 1997). One should ensure that the quantities that are compared are consistent with each other. For instance:

- undrained cohesion measured by a vane shear test (VST) is correlated to an undrained shear strength of the DSS type;
- in clays, cone resistances have often been correlated to undrained shear strength in compression (s_u^C), but it may be pertinent to correlate them to the direct simple shear strength (s_u^{DSS}) or also to the average of all three strengths [s_{uav} = 1/3 (s_u^C+ s_u^{DSS} + s_u^E)], with s_u^E being the shear strength in extension.

6.3.4. COMPRESSIBILITY AND PERMEABILITY PARAMETERS

This paragraph addresses the compressibility parameters that govern the plastic deformation of the foundation ground, and that are likely to generate irreversible displacements of the foundation: settlements and permanent rotations.

The deformability parameters (deformation moduli) that govern the behaviour of the foundation under cyclic loads (determination of the natural frequencies, stiffness of foundations and extreme displacements, cumulated displacements under cycles) are addressed in paragraph 6.5.

The compressibility parameters are measured in a laboratory using an oedometer test (more rarely, a triaxial test). The parameters that are usually measured are:

- the constrained modulus M (or oedometric modulus E_{oed});
- the vertical permeability of the sample k_v;
- the consolidation coefficient c_v.

These parameters may be obtained from tests with incremental loading, or from tests with a controlled rate of loading (CRL tests). They are used to estimate the consolidation settlements, (or primary settlements).

If necessary, oedometer tests may be carried out over large periods of time to estimate the creep coefficient C_α for clays, which allows assessing secondary settlements.

Permeability parameters govern the calculation hypothesis: calculations in drained, partially drained, or undrained conditions; they mainly concern gravity bases or monopiles foundations.

Usually, stability calculations are carried out in undrained conditions (conservative assessment). Taking into account a partial draining during the loading may prove beneficial in the case of more or less clean sandy soils.

Determining the permeability parameters may be carried out in a laboratory with an oedometer (for some load increments), with a permeameter or with a triaxial apparatus. It may also be carried out from in-situ tests (piezocone dissipation test): the coefficient of horizontal permeability k_h is then obtained. The relationship between k_v and k_h may prove to be difficult to establish for anisotropic soils. Carrying out oedometer tests on specimens cut perpendicularly to the core axis may prove useful.

6.4. DEFORMABILITY PARAMETERS

Analysing the natural frequencies of the wind turbine and determining the design loads require acquiring knowledge on two essential parameters: the deformation modulus of the soil (shear modulus G or Young's modulus E) and its damping β. These parameters should be acquired sufficiently early and with a high accuracy: not only do they govern the choice of foundations, but they also govern the geometry of the chosen foundations, so the fundamental frequencies 1P, 3P, 6P, or even 9P are avoided (see: chapter 4).

Because of the non-linear behaviour of soils, these parameters shall be estimated in function of the strain level (distortion or shear strain γ, or axial strain ε), associated to a given loading (FLS, SLS, ULS).

6.4.1. NON-LINEAR SOIL RESPONSE

Experimental observations made in a laboratory, for instance the stress-strain curves obtained with a triaxial device or with a direct simple shear box, reveal a non-linear behaviour of soils. Whether it is under a monotonous quasi-static loading or under a cyclic loading, the deformation characteristics of the material depend on the stress path that is followed.

Figure 6.1 displays schematically the stress – strain curves obtained during a classic triaxial test in compression, with a constant radial stress.

Experimental results show that there is a range of small strains, close to the origin, for which the relationship between the applied stress and the associated strain is linear. Beyond a certain stress threshold (which can be very low), and no matter what the stress path is, the ground behaviour ceases being linear: irreversible plastic deformations occur, which can lead to a state of failure.

During an unloading of the sample beyond this stress threshold, the path followed in the unloading differs from the one of the first loading (Figure 6.1). In the case of a re-loading, the re-loading path is close to the unloading one, which reveals a strain hardening of the material.

The previous statements can be generalised to more complex loadings, including cyclic loadings.

Figure 6.2 displays the typical recording of stress-strain curves $\tau = f(\gamma)$ obtained from a direct simple shear box for closed stress cycles, centred on the origin. This figure shows that, for a closed cycle, the soil behaviour is characterised by a hysteretic loop, whose area and inclination depend on the strain amplitude throughout the cycle.

For a small number of cycles and a moderate amplitude, the ends of the loops, which correspond to different amplitude cycles, are located on the curve of the first loading, passing through the origin.

Classically, the two fundamental parameters that characterise the stiffness response of the soil are defined from hysteresis loops:

- the secant shear modulus G, equal to the slope of the line that connects both ends of the loop;
- the damping coefficient β, associated to the area of the loop and that characterises the energy dissipating from the material during a cycle.

The dependency of these two parameters to the cyclic strain is highlighted in Figure 6.3: the modulus G decreases with the level of distortion, while damping β increases.

The maximum value G_0 (or G_{max}) of the modulus is the slope at the origin of the first loading (Figure 6.2 and Figure 6.3). It is obtained for very small strains ($\gamma = 10^{-6}$ to 10^{-5}) and is associated to the domain of elastic behaviour of the material.

Variations of G are very often presented under the normalised form G/G_0.

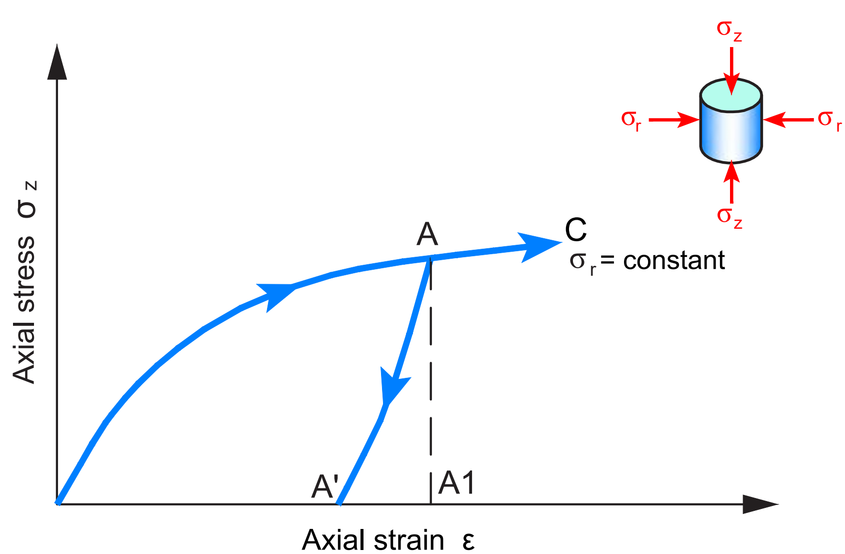

Figure 6.1 : Stress-strain curves under a quasi-static monotonous loading

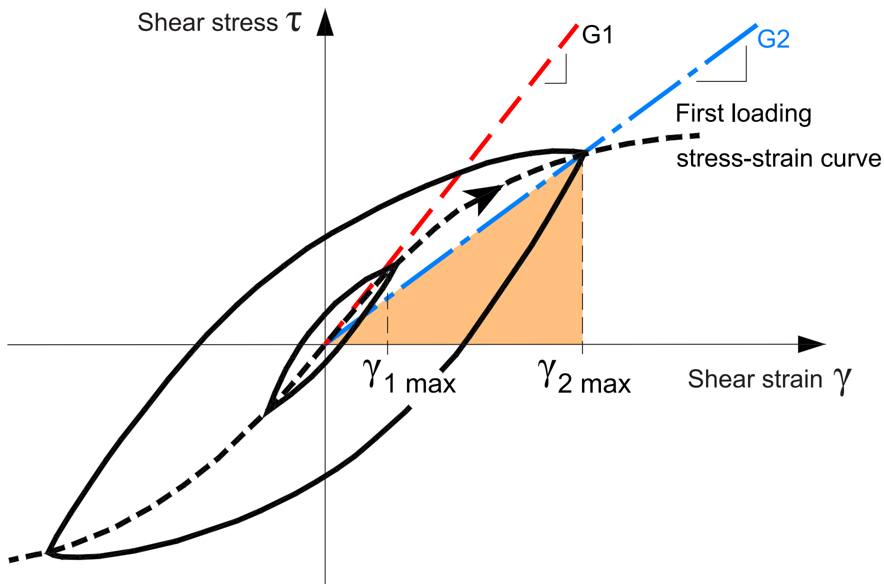

Figure 6.2 : Centred cyclic loading – Direct simple shear test

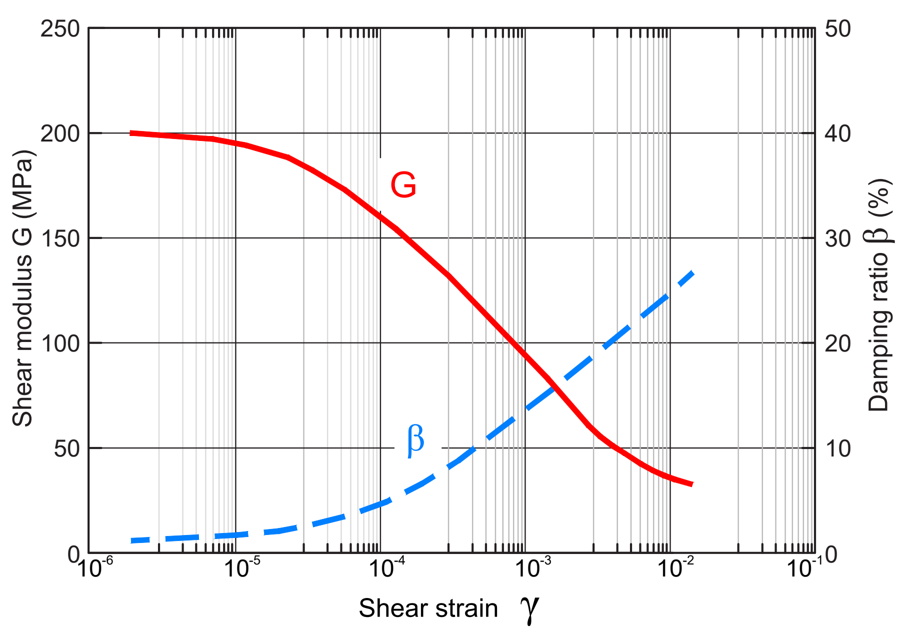

Figure 6.3 : Variations of the shear modulus G and of the damping β with distortion γ

6.4.2. G AT VERY SMALL DISTORTIONS (G_0 or G_{max})

G_0 moduli (or G_{max}) may be measured in a laboratory on representative intact samples, after being reconsolidated under the existing stress state, using various types of tests: e.g. resonant column, bender elements, cyclic triaxial test with local strain measurements, hollow cylinder of torsion, measurements of wave velocities on cores.

Experimental results show that these moduli, and regardless of what the material is, are function of several parameters, which highlight the nature and history of the material:

- void ratio;
- plasticity;
- overconsolidation;
- effective mean stress of consolidation.

The generalised laws of variation of G_0 (or G_{max}) are of the form (Hardin & Black, 1969):

$$G_0 = A_m \cdot p_a \cdot F(e) \cdot (OCR)^k \cdot \left(\frac{p'}{p_a}\right)^{0,5}$$

with

A_m: constant depending on the material

e: void ratio

OCR: overconsolidation ratio

k: factor depending on the plasticity index I_P

p': effective mean stress

p_a: atmospheric pressure

Table 6.4 synthesises how these various parameters influence the shear modulus G_0.

Table 6.4 : Factors influencing the variations of the shear modulus at very small strains

Increasing parameter :	G_0 variations
Effective mean stress, p'	Increases
Void ratio, e	Decreases
Overconsolidation ratio, OCR	Increases
Plasticity index, I_p	Increases

Various methodologies, essentially based on the measurement of the propagation velocities of shear waves V_s, have been developed to determine the modulus G_0 through in-situ measurements. In an isotropic linear elastic continuum, G_0 and V_s are bound by the relationship:

$$G_0 = \rho.V_s^2$$

with ρ = unit mass of the material.

Geophysical tests that allow achieving the measurement of G_0 or G_{max} in-situ are listed in chapter 5. The most accurate test is the cross-hole test, but it proves difficult to perform on offshore locations because it requires drilling two or three parallel boreholes. The most used offshore tests are: the down-hole test, the up-hole test or the PS-logging test, carried out in a single borehole, or also the seismo-cone test. Measuring the propagation of surface waves may also be achieved by using the MASW method.

The values of shear waves velocities inferred from these various tests are interpreted as being mean values on the variable thicknesses associated to each type of test. This leads to a certain ranking of results.

Discrepancies between in-situ tests and laboratory tests may be observed. They may be due to a representativeness flaw of the sample being tested, a scale effect due to the sample size, or also to a possible disturbance of the sample (the stiffness characteristics of soils being the parameters most affected by disturbance). Besides the effect of the disturbance caused by the making of specimens, the results of tests where very small strains are measured (resonant column, cyclic triaxial...) are likely to be affected by the limitations/deformations of the device itself. Overall, it will frequently be observed that:

- in soils, the velocities measured in a laboratory are smaller than the ones measured in-situ (overriding effect of the sample disturbance, notably in sands).
- in rocks, the velocities measured in a laboratory are greater than the ones measured in-situ (overriding effect of fracturing and heterogeneity of the rock mass).

In any case, a detailed analysis of the database is required prior to determining the design parameters. Overall, results from in-situ tests will be favoured over the ones from laboratory tests.

6.4.3. VARIATION OF G WITH DISTORTION

The variations of G or G/G_0 (or G/G_{max}) in function of the distortion γ are most often determined from laboratory tests: resonant column tests supplemented with cyclic triaxial tests or cyclic direct simple shear tests (more rarely tests with a hollow cylinder of torsion).

The high-pressure dilatometer test (HPDT) allows measuring moduli at small strains (10^{-4} to 10^{-2} range) in soft rocks (σ_c < 20 MPa). Pressuremeter devices (probes of the mono-chamber type), which are being currently developed, seem to be able to achieve similar performances. In both cases, moduli are obtained by carrying out 2 or 3 cycles similar to the ones shown in Figure 6.4.

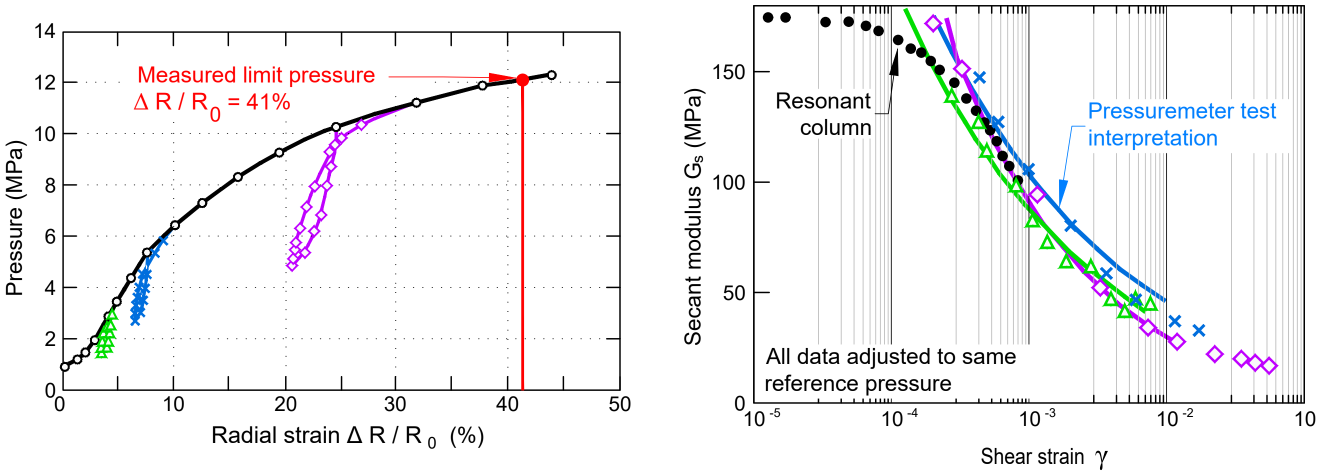

Figure 6.4 : Example of measurement of the shear modulus with a high pressure dilatometer

Table 6.5 synthesises the ranges of distortion levels investigated by the laboratory tests, and the main in-situ strain tests.

Table 6.5: Application ranges of in-situ and laboratory tests

Distortion [-]	10^{-6}	10^{-5}	10^{-4}	10^{-3}	10^{-2}	10^{-1}
Bender elements	▬▬▬					
Resonant column	▬▬▬	▬▬▬	▬			
Cyclic triaxial	- - -	- - -	▬▬▬	▬▬▬	▬▬▬	▬▬
Cyclic simple shear	- - -	- - -	▬▬▬	▬▬▬	▬▬▬	▬▬
Hollow cylinder of torsion	▬▬▬	▬▬▬	▬▬▬	▬▬▬	▬▬▬	▬▬
Geophysical tests (cross-hole, down-hole, up-hole, PS logging, MASW)	▬▬▬					
HPDT, flexible dilatometer			▬▬▬	▬▬▬		
Ménard pressuremeter					▬▬	▬▬

The shape of the curve representing the variations of G or of G/G_0 is steered by two main parameters:

- the plasticity of the material (plasticity index I_p);
- the mean effective consolidation stress, notably in the case of sands.

The overconsolidation ratio OCR has a very small influence, almost negligible in comparison with the other parameters.

The G/G_0 curve shifts towards the right (for a same G/G_0 ratio, the distortion value γ is greater) when one of these parameters increases, as shown in Figure 6.5 and Figure 6.6. This curve proves to be rather insensitive to the disturbance of the sample.

In Figure 6.5, the curve I_p = 0 % characterises non-plastic materials: silts, sands and gravels.

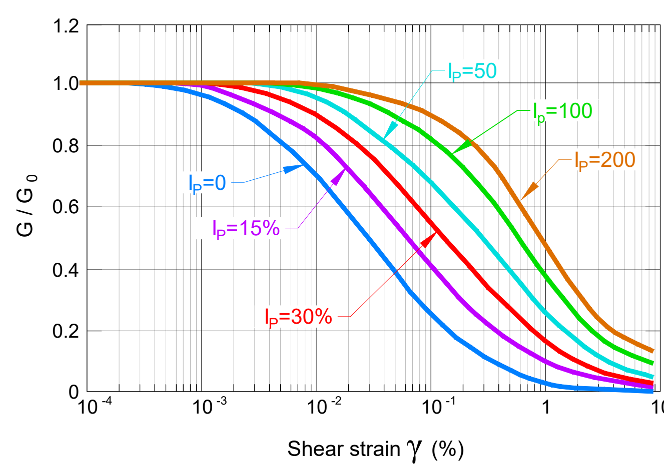

Figure 6.5 : Influence of plasticity on the variations of the shear modulus

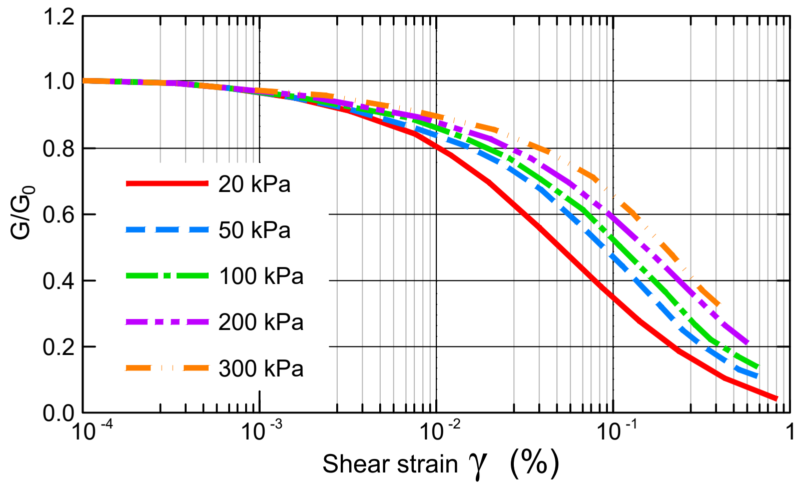

Figure 6.6 : Influence of the consolidation stress on the variations of the shear modulus

Table 6.6 synthesises the influence of these different parameters on the variations of the G/G_0 ratio and of the damping β.

Table 6.6 : Factors influencing the variations of the G/G_0 ratio and of the damping factor β in function of distortion (Kramer, 1996)

Increasing parameter :	G/G_0 variations	β variations
Effective confining stress, σ'_0	Increases	Decreases
Void ratio, e	Increases (clays) No effect (sands)	Decreases (clays) No effect (sands)
Overconsolidation ratio, OCR	No or little effect	No or little effect
Plasticity index, I_p	Increases	Decreases
Cyclic distortion, γ_c	Decreases	Increases
Number of loading cycles, N	Decreases (clays) Increases (sands)	Decreases

Analytical formulations providing the shape of the curve G/G_0 in function of these various parameters can be found in the available literature (Darendeli, 2001; Zhang, 2005). These empirical formulations may prove useful during preliminary studies, but should not be used when assessing G_0.

6.4.4. DAMPING

The studies of the soil-structure interaction under cyclic loading (vibrational, dynamic…) require the use of the damping notion, which is a fundamental value when studying vibrational phenomena in particular when being close to resonance. Damping is the factor that allows limiting forces and displacements in a structure vibrating at a frequency close to its resonance frequency.

Global damping is the sum of several components:

- the internal damping of the materials that constitute the structure (steel and/or concrete, which are usually well known);
- the internal damping of the materials that constitute the ground;
- the radiative damping (also called geometric damping), due to the dissipation of the energy of waves that propagate ad infinitum in the ground and in the water.

Radiative damping is function of the foundation geometry and of the excitation frequency. For loading frequencies smaller than 0.5 Hz, typical of loading under swell, it is very small (even for gravity base foundations with large dimensions) and often lower than the internal damping of the ground. Its contribution may then be ignored (excluding seismic loading, which is not addressed in the present document). In the case of frequencies greater than 1 Hz, it may be favourably taken into account.

For most soils, experience shows that the shape of the hysteresis loop, and hence the dissipated energy, does not depend on the excitation frequency of the system, and therefore on the strain rate. Damping is then not of a viscous origin, but rather of an hysteretic one. It is commonly called hysteretic damping and expressed under a non-dimensional form.

As for the G/G_0 ratio, the shape of the variation curve of β depends essentially on the plasticity and on the effective consolidation stress (but very little on the degree of overconsolidation), as shown in the following figures (Figure 6.7 and Figure 6.8) and in Table 6.6.

Analytical formulations providing the shape of the variation curve β in function of these various parameters can be found in the available literature (Darendeli, 2001; Zhang, 2005). These empirical formulations may prove useful during preliminary studies.

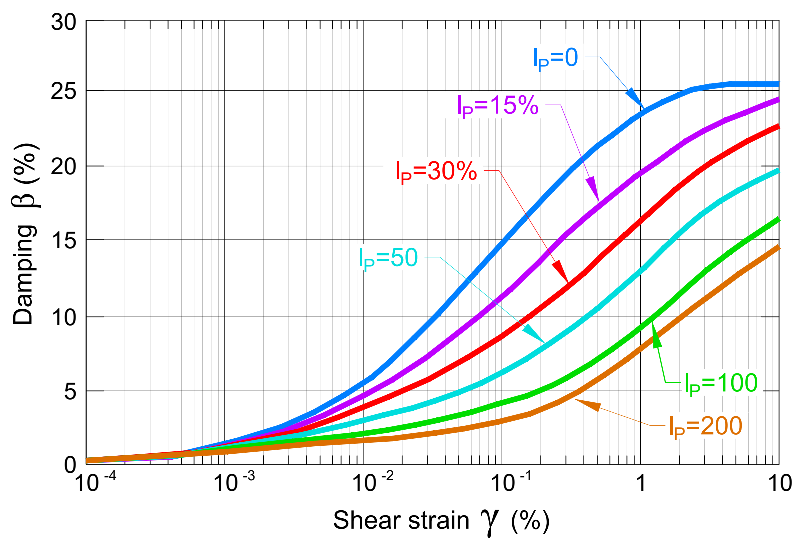

Figure 6.7 : Influence of plasticity on damping variations

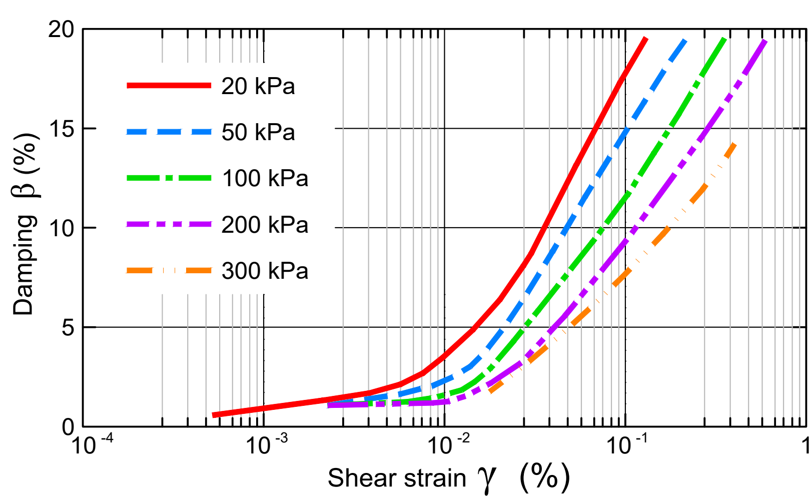

Figure 6.8 : Influence of the consolidation stress on damping variations

6.4.5. STIFFNESS OF ROCK MASSES

The stiffness of a rock mass is usually not equal to the proper stiffness of the intact rock. The stiffness of the rock mass depends on its true state of weathering and fracturing.

The mass stiffness can be deduced from the stiffness of the intact rock by applying a mass factor j_m ($G_{mass} = j_m . G_{intact}$), which may be expressed in function of the rock quality designation (RQD), of the frequency of fractures or of the ratio between the velocity of compression waves measured in-situ (V_F) and in a laboratory (V_L) on an intact sample (Table 6.7).

Mass stiffness may also be assessed using the RMR (Rock Mass Rating) of Bieniawski (1989) and its declinations that have been since published. This index takes into account the resistance of an intact rock (σ_c = UCS or $I_{S\ 50}$), the quality of the rock on cores (RQD), the spacing of discontinuities, the state of discontinuities and the hydrogeological conditions. It gives a great importance to fracturing and it seems its use should be limited to masses of resistant rocks, whose behaviour is governed by discontinuities.

The GSI (Geological Strength Index) was first introduced by Hoek in 1995. It is a variant of the RMR that tends to generalise its use.

For further details on the methods used to characterise rock masses in terms of resistance and deformability, one may look up the works of Hoek and Brown (Hoek and Brown, 1997; Hoek, Kaiser and Bawden, 1995) as well as the AFTES (2003) recommendations on the « characterisation of rock masses that are pertinent when building underground structures ».

Guidance for characterising rock masses for designing drilled and grouted offshore pile foundations can be found in Puech and Quiterio-Mendoza (2019).

Table 6.7 : Correlation between mass factor j_m, RQD, frequency of fractures and velocity index (Deere and Miller, 1966; Coon and Merritt, 1970)

Rock Quality Classification	R.Q.D. %	Fracture frequency per metre	Velocity index $(V_F / V_L)^2$	Mass factor j_m
Very poor	0 - 25	15	0 – 0.2	0.2
Poor	25 - 50	15 - 8	0.2 – 0.4	0.2
Fair	50 - 75	8 - 5	0.4 – 0.6	0.2 – 0.5
Good	75 - 90	5 - 1	0.6 – 0.8	0.5 – 0.8
Excellent	90 - 100	1	0.8 – 1.0	0.8 – 1.0

V_F velocity of waves in situ, V_L velocity of waves in a laboratory

6.5. CYCLIC PARAMETERS

6.5.1. NOTION OF CYCLIC DEGRADATION

The notion of the cyclic degradation of soils is introduced in the SOLCYP (2017) recommendations, which can be perused for further details.

Generally speaking, carrying out shear tests in undrained conditions on soils samples generates:

- an increase of the pore pressure u within the sample;
- a decrease of the cyclic stiffness, characterised by the modulus G_{cy};
- an accumulation of the displacements γ under a constant loading rate;
- a degradation of the strength τ_f of the material.

These phenomena evolve with the number of cycles N that are applied, and become more severe when the amplitude of cycles increases. Each of the parameters u, γ, τ_f is the sum of a mean component (respectively u_m, γ_m, τ_m) and a cyclic component (u_{cy}, γ_{cy}, τ_{cy}).

An example of response of a soil under cyclic loading is shown in Figure 6.9, with the most usual case of a non-symmetrical cyclic shear test.

The level of shear resulting from the state of the ground at rest is noted as τ_0, while the level of shear characterising the start of the application of cycles of amplitude τ_{cy} is noted as τ_m.

The transition from the τ_0 state to the τ_m state may be carried out in undrained or drained conditions depending on the nature of the simulated loading (e.g. application of the self-weight of the structure in the short term or after consolidation).

In undrained conditions, when the shear stress increases by $\Delta\tau_m$ to reach τ_m, the ground undergoes a mean strain $\Delta\tau_m$ and a mean increase of pore pressure Δum.

The cyclic shear stress τ_{cy} leads to a mean strain τ_m and a cyclic strain γ_{cy}, that both increase with the number of cycles. Similarly, the cyclic shear stress leads to an increase of the mean pore pressure u_m and an increase of the cyclic component u_{cy}.

Excess pore pressure generated by the cyclic load shifts the effective stress path towards the failure envelope. After a certain number of cycles ($N = N_f$), the cyclic failure envelope may be reached and major strains occur. The cyclic shear strength, written $\tau_{f,cy}$ is defined by a pair of values τ_m and τ_{cy}, which have led to failure under a number of cycles Nf :

$$\tau_{f,cy} = (\tau_m + \tau_{cy})_f$$

Cyclic shear strength is not a constant of the material. It depends on:

- the value of the mean shear stress τ_m;
- the amplitude of the cyclic shear stress τ_{cy};
- the loading mode (simple shear, compression, extension);
- the loading history, notably the number of cycles;
- the frequency of cycles in the case of clays.

The cyclic shear modulus G_{cy} is defined in Figure 6.10 in the case of a symmetrical cyclic load.

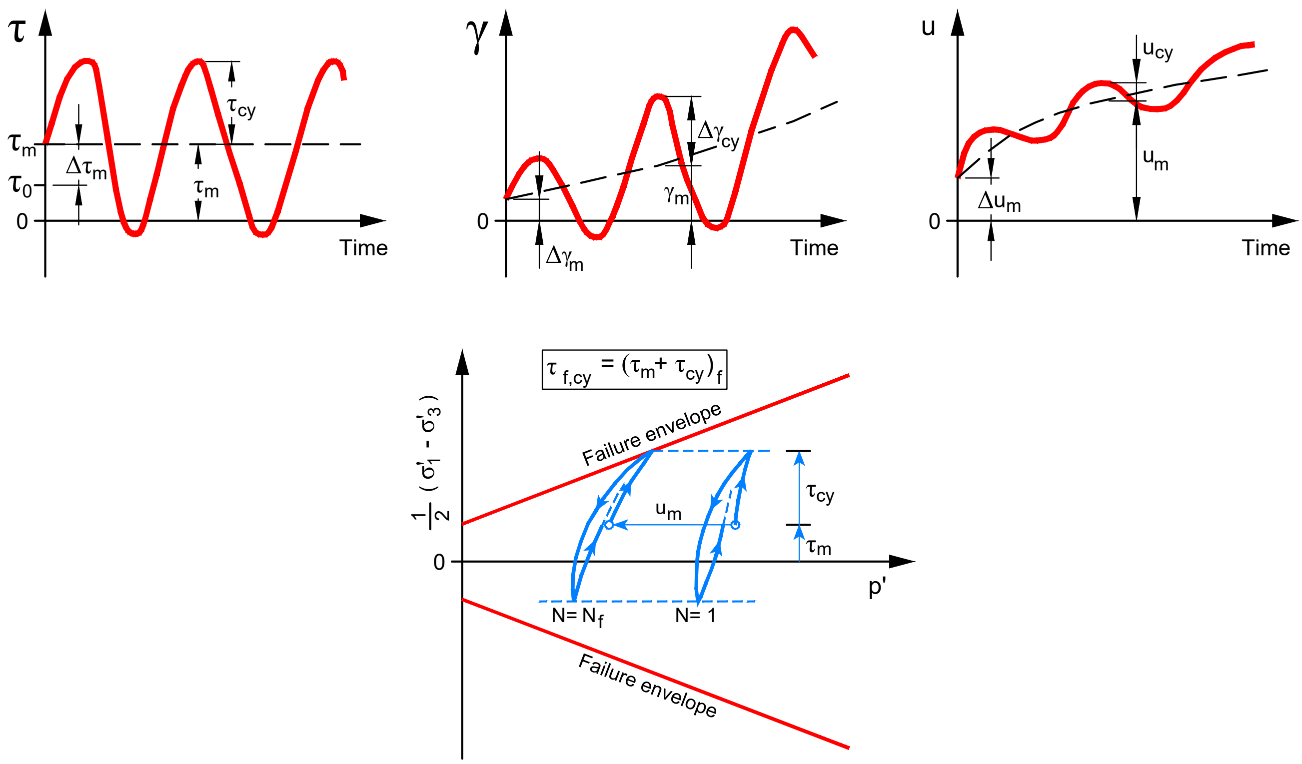

Figure 6.9 : Evolution with time of strains and excess pore pressures in a non-symmetrical cyclic shear test

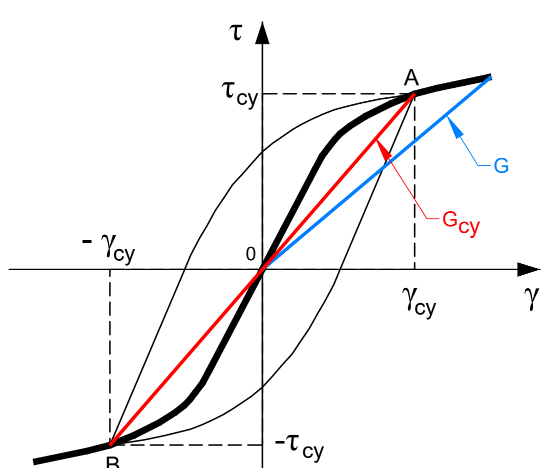

Figure 6.10 : Definition of the shear moduli

To the extent where laboratory tests show that the cyclic strength of the material depends on the stress path followed during the cyclic event that is being studied, it is critical that the determination of the cyclic response of a soil in contact with, or under, a foundation be compatible with the stress path likely to be followed on the potential failure line.

In the case of an axially loaded pile, the shear on, or close to, the interface will control the failure (Figure 6.11). Assessing the degradation of friction along the shaft is the critical phenomenon.

In the case of a gravity base, the failure line goes through zones involving failure conditions of the extension, simple shear or compression types (Figure 6.12). The decrease of capacity, the accumulation of displacements and the decrease of stiffness should be assessed by taking into account the complexity of the loading modes.

For further details, Jardine et al. (2012) et Andersen et al. (2013) may be perused.

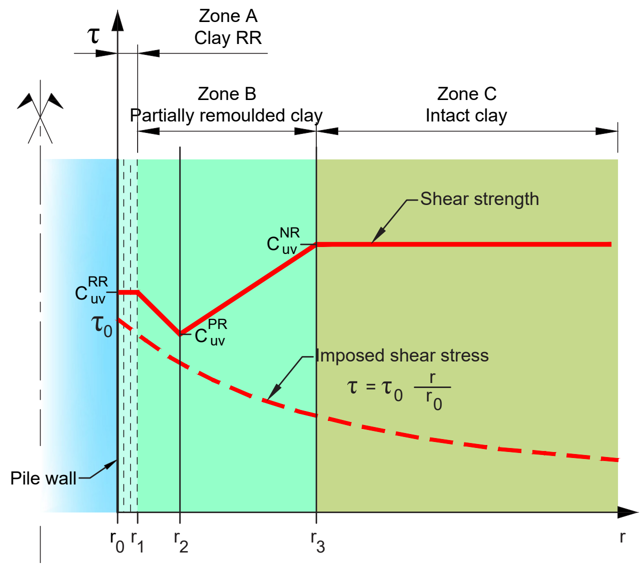

Figure 6.11 : Schematic representation of the variations of undrained shear strength and shear stress around the test piles in HAGA (from Karlsrud et al., 1992)

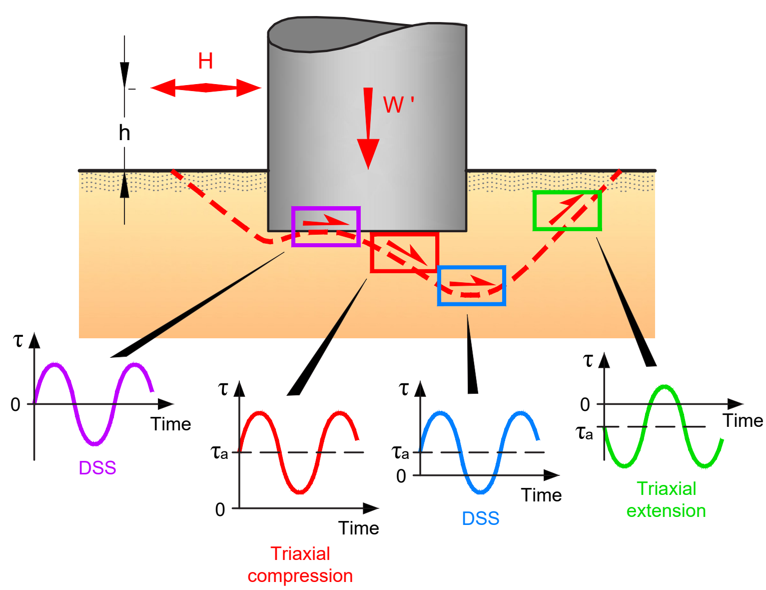

Figure 6.12 : Idealised stress paths for typical soil elements along the potential failure line (from K. Andersen, 2009)

6.5.2. ACQUISITION OF PARAMETERS REQUIRED TO DETERMINE CYCLIC DEGRADATION LAWS

Determining the cyclic parameters of soils is usually achieved using cyclic shear tests in a laboratory on undisturbed samples.

Tests of the undrained consolidated type shall be reconsolidated under an effective stress state representative of the various stress states prevailing in the influence zone below and close to the foundations. Loading is applied in undrained conditions.

To determine the degradation of friction along the piles shaft, direct shear tests of the DSS type (Direct Simple Shear) can be used, in which the vertical effective stress applied is equal to the horizontal stress imposed to the wall by the surrounding soil. The consequences of the installation mode (partial or full displacement driving, decompression due to drilling and pressure of the concrete/grout before setting) should be considered. In sands, interface tests of the CNS type (Constant Normal Stiffness) may be carried out as an alternative. Here again, the installation simulation is essential. The SOLCYP (2017) recommendations provide all the required information for the realisation of such tests.

To determine the cyclic parameters required for the design of gravity bases, it is necessary to consider the shear strength anisotropy. In that case, the full characterisation of the cyclic response of a material will require establishing contour diagrams for three types of loadings: direct simple shear (DSS), anisotropic triaxial compression (CAUC test) and anisotropic triaxial extension (CAU_E). This task is addressed in paragraph 10.3.3 of chapter 10. Further details can be found, for instance, in N.G.I's publications (e.g. Andersen, 2015).

Results from tests are conveniently displayed under the form of contour diagrams that allow representing synthetically the behaviour of the material. As an example, the principles of construction of a contour diagram based on simple shear tests (DSS) are shown for:

- a contour diagram of distortions (Figure 6.13). This type of diagram is privileged in the case of clays;
- a contour diagram of pore pressures (Figure 6.14). This type of diagram is privileged in the case of sands.

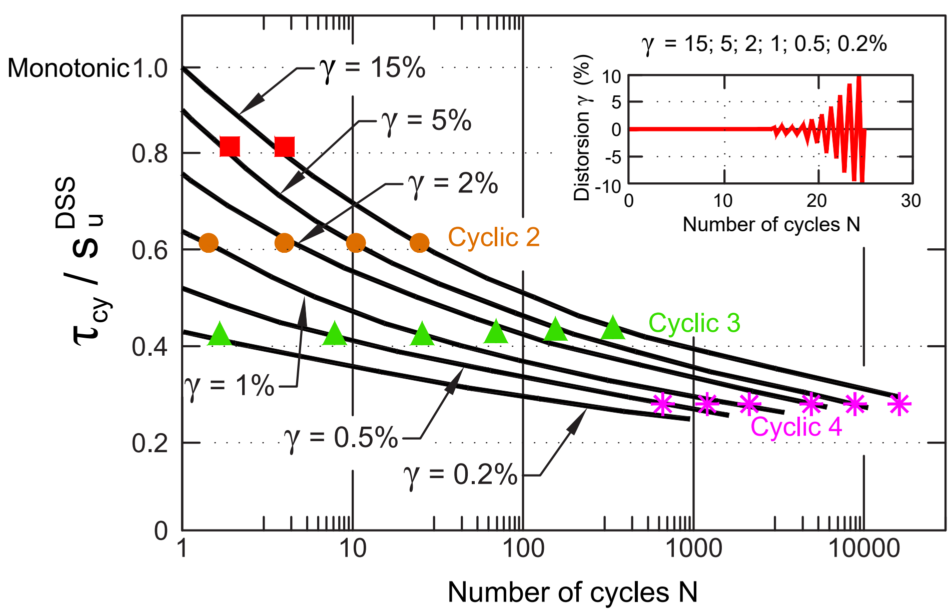

Figure 6.13 : Construction principle of a contour diagram of distortions from alternated direct simple shear tests

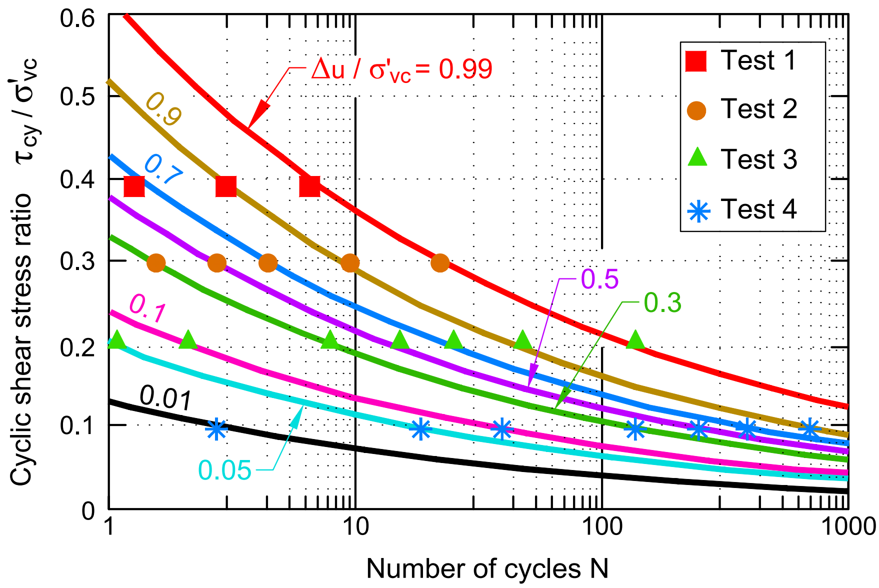

Figure 6.14 : Construction principle of a contour diagram of pore pressure

An example of construction of a contour diagram of distortions is shown in Figure 6.15.

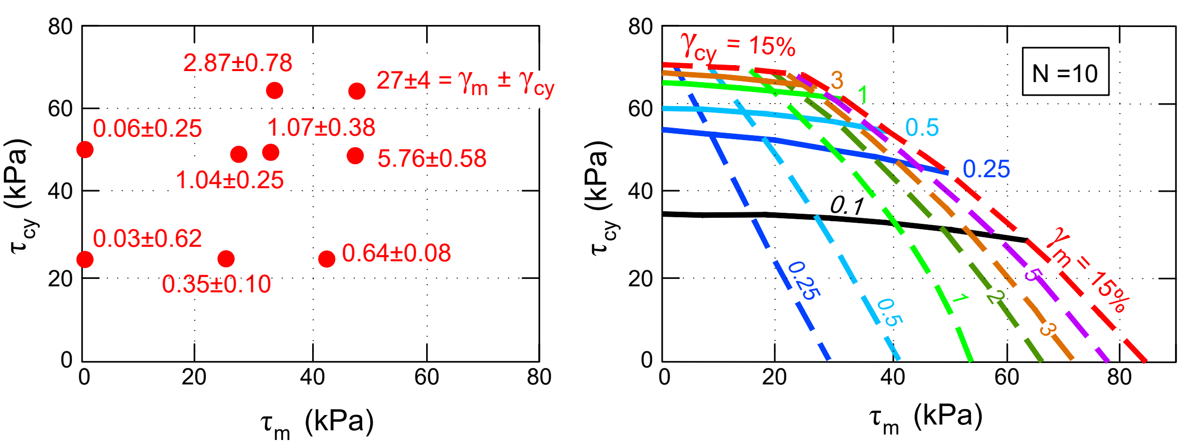

Figure 6.15 : Construction of contour diagrams of distortions (from NGI)

6.5.3. CYCLIC STRENGTH DIAGRAMS

Contour diagrams of strength allow identifying the number of cycles leading to failure, and the mode of failure. When mean shear stresses (respectively cyclic) start overriding the cyclic shear stress (respectively mean), ground failure occurs mainly through the increase of the mean shear strain γ_m, i.e. through creep, (respectively through the increase of the cyclic distortion γ_{cy}).

Such a diagram is presented in Figure 6.16 for DSS tests on Drammen clay. Normalising is done in relation to undrained shear strength under monotonous loading S_u^{DSS}. The red lines separate the domains of failure mode by cyclic strain (blue area, on the left, close to the ordinate axis) from the failure mode by mean strain (yellow area, on the right).

On this type of diagram, the value of the cyclic strength $\tau_{f,cy}$ is equal to the sum of the values of τ_m and τ_{cy} for the number of cycles that led to failure N_f, i.e. $\tau_{f,cy} = (\tau_m + \tau_{cy})_f$.

It should be noted that $\tau_{f,cy}$ decreases with the number of cycles and is notably lower for two-way loading than for one-way loading. For a purely alternated loading ($\tau_m = 0$), cyclic strength for 1000 cycles is only 55% of static strength.

It should also be noted that, despite degradation values being different from one clay to another, the shapes of the contour diagrams are similar (Figure 6.17). This observation allows building the contour diagram of a specific clay from a limited number of cleverly positioned tests.

In sands, contour diagrams are most often presented under the form of liquefaction diagrams (Figure 6.18). These diagrams are highly dependent on the particle size of the material, and it is recommended to build a specific diagram for each material. Obtaining a diagram is relatively easy with a triaxial device, or with the direct simple shear box (DSS).

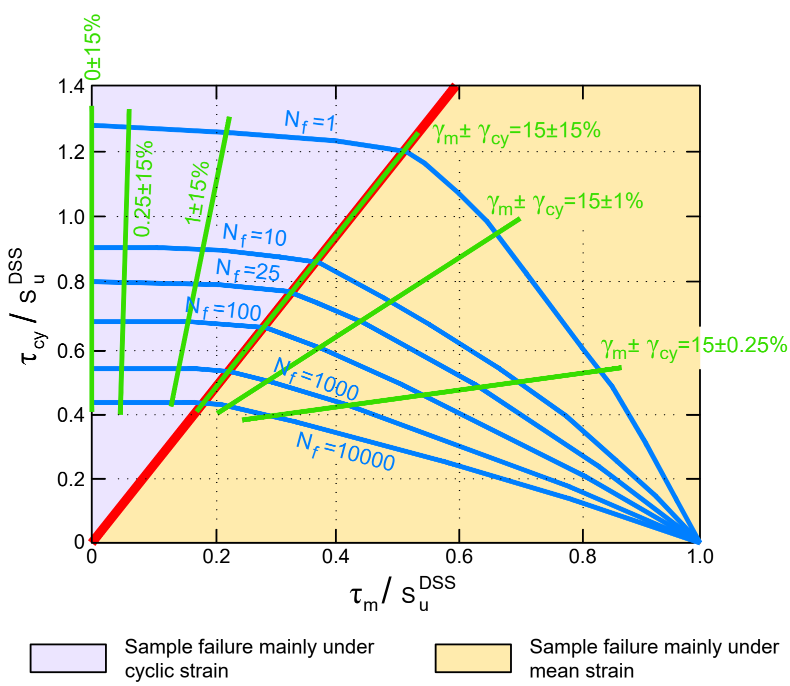

Figure 6.16 Contour diagram of the Drammen clay (OCR=1) obtained from the DSS test (from Andersen, 2009)

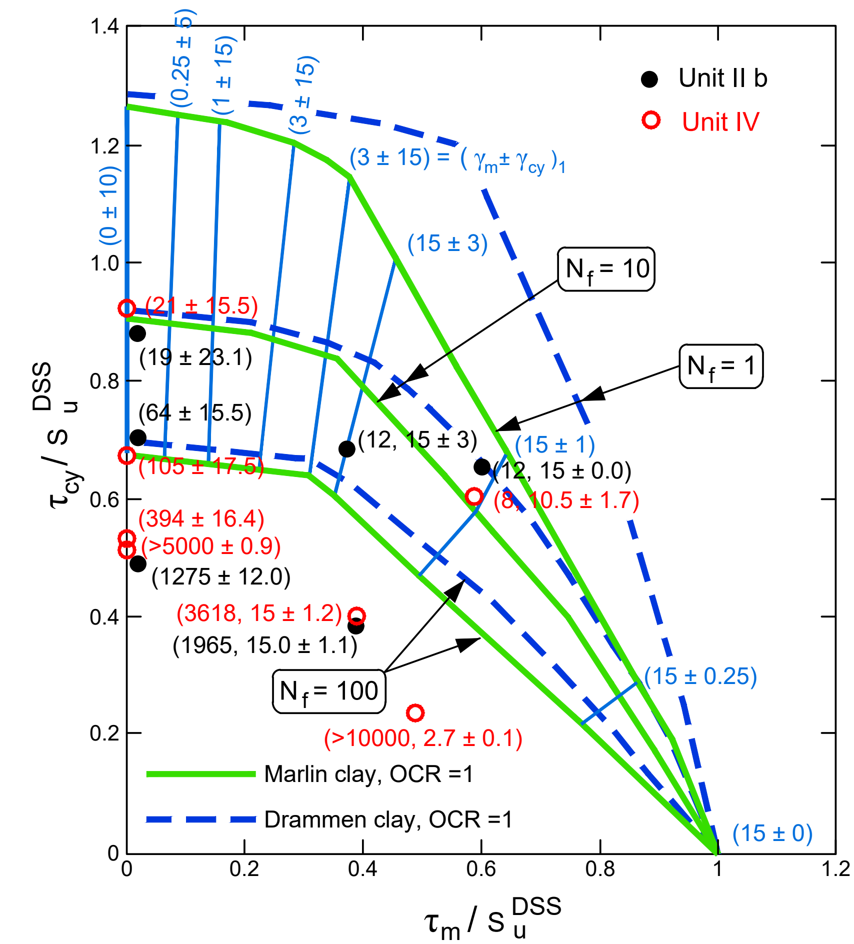

Figure 6.17 : Comparison of contour diagrams for two clays (direct simple shear) (from Jeanjean et al., 1998)

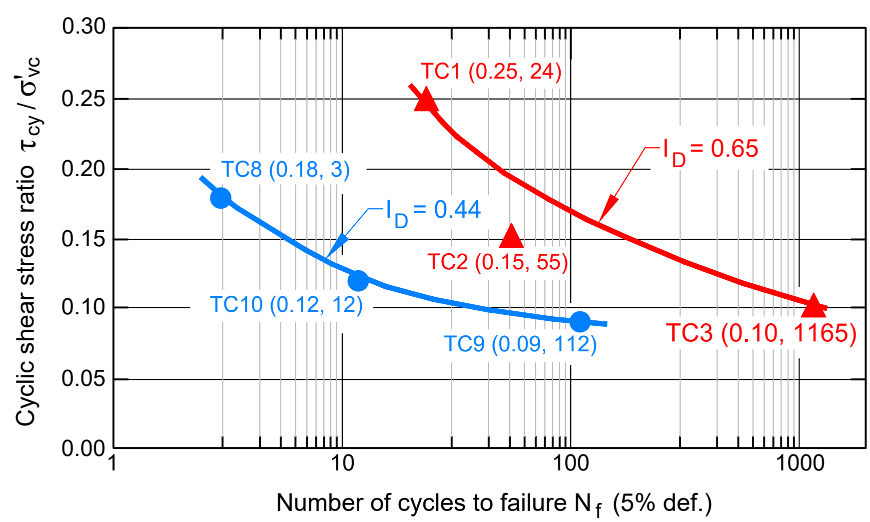

Figure 6.18 : Curves of cyclic shear strength of the N34 Fontainebleau sand for two density indices, with N_f = number of cycles leading to liquefaction, defined for 5% of axial strain of the sample (from SOLCYP, 2017)

6.5.4. GENERALISED CURVES $G(\gamma, Ip, \sigma, N)$

The cyclic shear modulus G_{cy} decreases with the normalised cyclic stress ratio and the number of cycles. An example of variation of G_{cy} in a clay and under various overconsolidation ratios is presented in Figure 6.19. It should be noted that, very logically, for very low shear stress ratios, G_{cy} becomes close to the value of G_0 (or G_{max}).

Vucetic et Dobry (1991) introduce the concept of a generalised curve $G/G_0 = f(\gamma)$, in which the parameter N operates additionally to the other factors that are the plasticity index I_p and the level of stress σ (Figure 6.20).

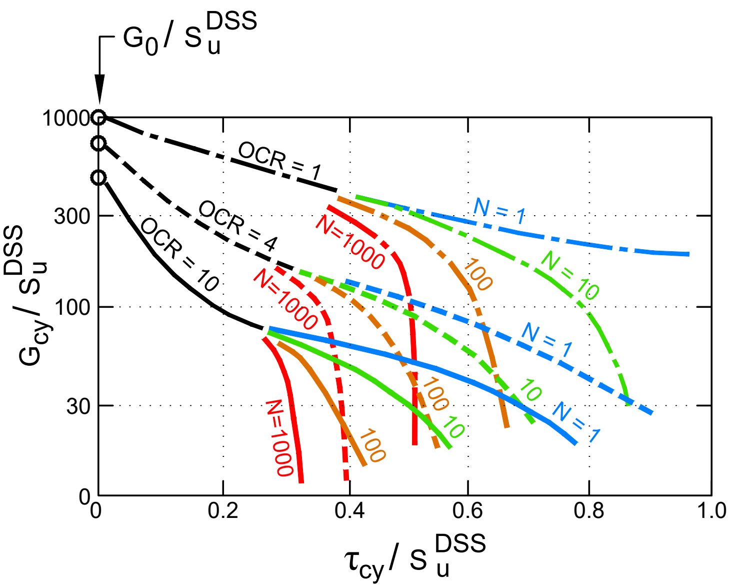

Figure 6.19 : Evolution of the cyclic shear modulus G_{cy} (from O'Reilly & Brown, 1991)

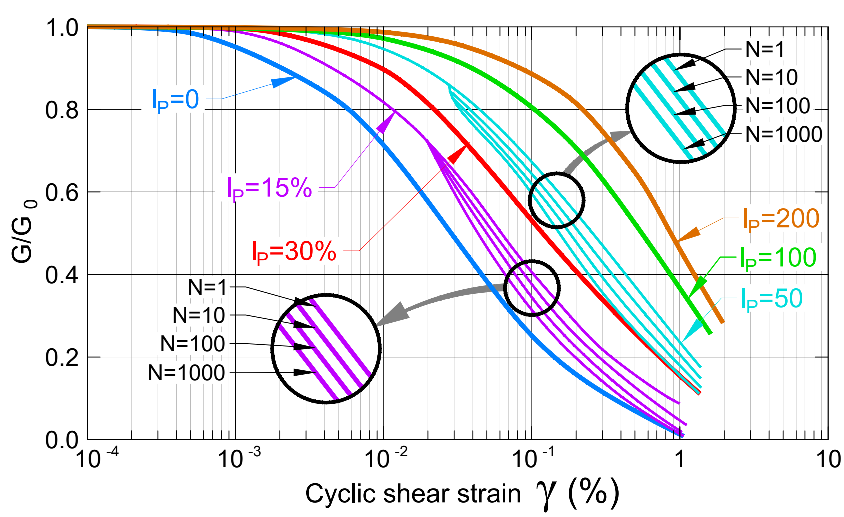

Figure 6.20 : Variation of the normalised shear modulus G/G_0 in function of the level of cyclic strain γ_{cy} for a normally consolidated soil (with a consolidation stress below 150kPa) showing the effect of the plasticity I_p and of the number of cycles N (from Vucetic et Dobry, 1991)

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7 DESIGN LOADS AND VERIFICATIONS

7.1 PRINCIPLE OF VERIFICATIONS

7.2 DESIGN LOADS

7.3 VERIFICATIONS

7.4 REFERENCES

7. DESIGN LOADS AND VERIFICATIONS

7.1. *PRINCIPLE OF VERIFICATIONS*

Within the framework of current recommendations, verifications deal essentially with the response of the wind turbine in operation. Loads occurring before the structure commissioning and which may affect its further behaviour should also be considered.

7.1.1. DEFINITIONS OF LIMIT STATES

A limit state is a condition beyond which the structure (including its foundation), or an element of the structure, no longer meets the performance requirements for which it has been designed.

The following limit states are addressed in this document:

- ULS – Ultimate Limit States:

  Ultimate limit states correspond to the maximum resistance of the structure, or of a part of the structure, to the loads it must carry.

  Examples of ultimate limit states are:

  - a loss of structural resistance (by plastification, buckling...);
  - the brittle failure of an element;
  - the loss of the static equilibrium of the structure, or of one of its parts, considered as a rigid body (for instance, overturning);
  - the failure of critical constitutive elements of the structure when their ultimate resistance is exceeded or when an excessive deformation occurs. The repetitiveness of loads may cause a decrease of ultimate resistance.

- SLS – Service Limit States:

  Service limit states correspond to states beyond which the specified criteria for normal operations (tolerances) are no longer met.

  Criteria of service limit states may be formulated, for instance, in relation to:

  - deformations or deflections that may modify the effect of the action taken into account;
  - displacements of the structure that may alter or limit its operating conditions or affect the functioning of certain devices. In the case of wind turbines, manufacturers set very strict criteria for the inclination of rotors over the total operational life of the structure. The differential settlement of foundations is a particularly sensitive factor. The repetitiveness of loads may cause an excessive accumulation of strains or displacements;
  - excessive vibrations that may cause a malfunctioning of the equipments;
  - the percentage of gapping (loss of contact) under a foundation of the gravity type.

- FLS – Fatigue Limit States:

  Fatigue limit states correspond to the possibility of failure of elements of the structure and of structural elements of the foundation under the repetitive effect of cyclic loads (including installation loads). No fatigue limit states concern the foundation ground, since the levels of cyclic loading taken into account to assess the fatigue of structural elements are relatively moderate. The effect of cycles on the behaviour of soils is addressed in the paragraphs concerning the ULS and SLS.

  However, it should be noted that the response of the ground is a significant element of the study of the dynamic response of the structure. The choice of the ground parameters used to assess foundations stiffnesses associated to fatigue analyses is a major factor.

- ALS – Accidental Limit States:

  Accidental limit states correspond to damages caused to the structure, or to part of it, by accidental events or by operating accidents.

  Ships collisions are an example of accidental events.

7.1.2. PRINCIPLE OF VERIFICATIONS BY THE PARTIAL FACTORS METHOD

The partial factors method is a design method under which the objective, in terms of safety, is obtained by applying factors on the characteristic values of loads and resistances that govern the response of the structure, and then by enforcing a design criterion defined on the basis of these factors and characteristic values.

Variables governing the design are:

- the loads acting on the structure, or the effect of the structure internal loads;
- the resistance of the structure, or of the constitutive materials of the structure.

Foundations analyses will generally consider:

- the impact of loads applied to the turbine, to the tower and to the sub-structure, reduced to a force torsor at the seabed level;
- the resistance of the soil, or the resistance to the effect of the soil on the soil-structure interface, depending on the type of foundation.

The safety of the studied limit state is deemed as being satisfied when the design loads S_d do not exceed the design resistance of the system R_d :

$$S_d \leq R_d$$

This inequality defines the design criterion.

Design loads S_d are obtained by multiplying the characteristic loads S_k by a partial factor γ_F:

$$S_d = \gamma_F \cdot S_k$$

Design resistances can be obtained from the characteristic resistances using two procedures, detailed in paragraph 7.3, which depend on the type of calculation that is carried out.

The recommendations used to establish characteristic resistances values for soil and interfaces are provided in chapter 6.

The values of the partial factors γ_F on the loads are set out in paragraph 7.2.

7.2. DESIGN LOADS

Design loads stem from the combination of different load cases (G, Q, E, A, S, D, as defined in paragraph 4.1), with their appropriate partial factors γ_F. Only the loads combinations that are relevant for the design of foundations are considered in this document.

Note : the explicit calculation of loads of a seismic origin is not addressed in this document.

7.2.1. ENVIRONMENTAL LOADS

Load cases relative to variable and environmental conditions of the E type (including the loads brought by the wind turbine) are defined in chapter 4 and set out in the table of Appendix A. These load cases are associated to three limit states:

- an ultimate limit state called normal: ULS;
- an ultimate limit state called abnormal: ULSa;

  The abnormal limit state corresponds to abnormal operating conditions, involving serious failures of the system or the simultaneous impact of several environmental conditions of a rare occurrence. Examples of abnormal operating conditions are the accumulation of faults in the safety system or the combination of an extreme wind with a network failure;
- a fatigue limit state: FLS.

Table 7.1 provides the correspondence between the DLC load cases of the E type, the limit states and the associated partial factors.

For normal ULS, partial factors on loads are usually γ_F =1.35, except for the 1.1 and 2.5 DLC, for which DNVGL-ST-0126 (2016) recommends values of 1.25 and 1.20 respectively.

For abnormal ULS (ULSa), partial load factors are γ_F = 1.10.

For FLS, partial load factors are γ_F = 1.00.

Foundation studies require additionally to verify the Service Limit States (SLS).

SLS can be defined from IEC 61400-6 (draft, to be published) and from DNVGL-ST-0437, Annex C (2016):

- all DLC associated to ULS called normal can be used to characterise rare SLS;
- abnormal ULS cases (ULSa) shall not be considered as SLS;
- more specifically, three SLS cases should be considered for foundations studies:
  - the occurence of the characteristic extreme load;
  - the load case called LDD 10^{-4}, which corresponds to a load level that can only be exceeded during 0.01% of the structure life (i.e. 17.5 hours over 20 years);
  - the load case called LDD 10^{-2}, which corresponds to a load level that can only be exceeded during 1% of the structure life (i.e. 1,750 hours over 20 years);
  - these last two cases shall be assessed under DNVGL-ST-0437 (2016) from DLC 1.2 and 6.4

All SLS cases are associated to partial load factors γ_F=1.00.

7.2.2. NON-ENVIRONMENTAL LOADS

For all other types of loads (non-environmental), partial factors on loads are stated below:

- permanent loads G and potential overloads Q: under ultimate limit states (ULS or ULSa) partial factors γ_F are equal to 0.90 if the gravity effect is favourable to the stability case under consideration, and to 1.10 if it is unfavourable. When appropriate measures are implemented, partial load factors γ_F may be brought back to 1.00;
- accidental loads A: the partial load factor γ_F is always equal to 1.00;
- deformation loads D: loads generated by deformations, and notably by the differential settlements of the foundation supports, are assigned with a partial factor γ_F equal to 1.00.

Table 7.2 below summarises the partial load factors γ_F that shall be applied for defining design loads.

Table 7.1 : Classification (IEC 61400-3, 2009, and DNVGL-ST-0437, 2016) of load cases of the E type, and partial factors on the associated loads

Design situation	DLC load case	Limit state	Partial factor γ_F
Power production	1.1	ULS	1.25
	1.2	FLS	1.00
	1.3	ULS / Rare SLS	1.35 / 1.00
	1.4	ULS / Rare SLS	1.35 / 1.00
	1.5	ULS / Rare SLS	1.35 / 1.00
	1.6a	ULS / Rare SLS	1.35 / 1.00
	1.6b	ULS / Rare SLS	1.35 / 1.00
Power production plus occurrence of faults	2.1	ULS / Rare SLS	1.35 / 1.00
	2.2	Abnormal ULS	1.10
	2.3a	Abnormal ULS	1.10
	2.3b	ULS / Rare SLS	1.35 / 1.00
	2.4	FLS	1.00
	2.5	ULS	1.20
Start up	3.1	FLS	1.00
	3.2	ULS / Rare SLS	1.35 / 1.00
	3.3	ULS / Rare SLS	1.35 / 1.00
Normal shutdown	4.1	FLS	1.00
	4.2	ULS / Rare SLS	1.35 / 1.00
Emergency shutdown	5.1	ULS / Rare SLS	1.35 / 1.00
Parked (standing still or idling)	6.1a	ULS / Rare SLS	1.35 / 1.00
	6.1b	ULS / Rare SLS	1.35 / 1.00
	6.1c	ULS / Rare SLS	1.35 / 1.00
	6.2a	Abnormal ULS	1.10
	6.2b	Abnormal ULS	1.10
	6.3a	ULS / Rare SLS	1.35 / 1.00
	6.3b	ULS / Rare SLS	1.35 / 1.00
	6.4	FLS	1.00
Parked and fault conditions	7.1a	Abnormal ULS	1.10
	7.1b	Abnormal ULS	1.10
	7.1c	Abnormal ULS	1.10
	7.2	FLS	1.00
Transport, assembly, maintenance and repair	8.2a	Abnormal ULS	1.10
	8.2b	Abnormal ULS	1.10
	8.2c	Abnormal ULS	1.10
	8.3	FLS	1.00

Table 7.2: Partial factors γ_F to be applied on loads (IEC 61400-3, 2009, and DNVGL-ST-0437, 2016)

Limit states	Loads of environmental nature: E	Permanent loads: G, Q	Other loads: A, D
ULS	1.35*	**Favourable: 0.90** **Unfavourable: 1.10****	1.00
ULSa	1.10		1.00
SLS	1.00	1.00	1.00
FLS	1.00	1.00	1.00
ALS	1.00	1.00	1.00

* except for DLC 1.1 and DLC 2.5

** 1.00 under certain conditions

7.3. VERIFICATIONS

7.3.1. USUAL VERIFICATIONS UNDER QUASI-STATIC LOADS

The verification of the limit state consists in ensuring that the design load effect S_d does not exceed the design resistance R_d:

$$S_d \leq R_d$$

The design resistance can be obtained from the static characteristic resistance R_{ks} using two approaches, depending on the calculation method used to establish R_{ks}.

The first approach consists in dividing the characteristic resistance R_{ks} by a partial factor γ_R:

$$R_d = R_{ks} / \gamma_R$$

This approach is applicable when the characteristic resistance R_{ks} is calculated from the static characteristic resistance of the ground material σ_{ks}:

$$R_{ks} = R(\sigma_{ks})$$

The second approach consists in establishing the design resistance R_d directly from the design resistance of the material σ_d:

$$R_d = R\ (\sigma_d)$$

with:

$$\sigma_d = \sigma_{ks} / \gamma_M$$

γ_M being the partial factor applied on the static characteristic resistance of the material σ_{ks}.

The applicable approach should be specified on a case-by-case basis depending on the type of foundation under consideration.

7.3.2. VERIFICATIONS UNDER CYCLIC LOADS

The principle of verification is similar to the one recommended for quasi-static loads, but the static characteristic load R_{ks} or the static characteristic resistance of the material X_{ks} are replaced by, respectively, the cyclic characteristic resistance R_{kc} or the cyclic characteristic resistance of the material X_{kc}.

Cyclic characteristic resistances (R_{kc} or X_{kc}) are the degraded resistances corresponding to the cyclic event that is considered. The recommended methodology to assess the degradation of resistances is outlined in chapter 6.

The principle consists in assessing the degradation due to cycles from unfactored loads characterising the design cyclic event. Then, the partial coefficient of resistance γ_R or of material γ_M is applied on the value of the cyclic characteristic resistance.

$$R_d = R_{kc} / \gamma_R$$

or

$$R_d = R\ (X_d)$$

with:

$$X_d = X_{kc} / \gamma_M$$

7.3.3. PARTIAL FACTORS OF RESISTANCE AND MATERIAL

Partial factors of resistance γ_R and of material γ_M are identical for verifications under quasi-static and cyclic loads.

For foundations on piles (monopiles or jacket piles), the factors of resistance or materials proposed by DNVGL-ST-0126 (2016) are provided in Table 7.3. Given the reliability studies on foundations on piles carried out for offshore structures, the factor 1.25 applicable on the axial resistance is fully justified for the verification of the axial capacity of steel tubular piles driven into conventional soils (siliceous sands and clays).

For other types of piles (for instance, drilled and grouted) and/or other types of soils (notably carbonate soils, chalks and other soft rocks), the recommendations of chapter 9 (piles for multipods) shall be perused. Partial factors for ULS are provided in Table 9.5 for piles driven in non-conventional soils, and in Table 9.7 for bored piles.

For gravity base foundations, material factors proposed by DNVGL-ST-0126 (2016) are provided in Table 7.4.

The recommended methods of analysis are set out in the chapters corresponding to each type of foundations.

Table 7.3: Partial resistance and material factors for foundations of offshore wind turbines on piles under DNVGL-ST-0126 (2016) applicable to steel tubular piles driven into conventional soils (siliceous sands and clays)

Loading mode	Method of analysis	Limit state	
		ULS	SLS / ALS
Axial	Calculation of the limit skin friction and of the limit end bearing from X_k.	$\gamma_R = 1.25$	$\gamma_R = 1.00$
Lateral	Effective stress	$\gamma_M = 1.15$	$\gamma_M = 1.00$
	Total stress	$\gamma_M = 1.25$	$\gamma_M = 1.00$

Table 7.4: Partial material factors for foundations of offshore wind turbines on gravity bases under DNVGL-ST-0126 (2016)

Method of analysis	Limit state		
	ULS (stability)	ALS (stability)	SLS (settlements)
Effective stress	γ_M = 1.15	γ_M = 1.00	γ_M = 1.00
Total stress	γ_M = 1.25	γ_M = 1.00	γ_M = 1.00

7.4. REFERENCES

DNVGL-ST- 0126 (2016) *Support structures for wind turbines*

DNVGL-ST- 0437 (2016) *Loads and site conditions for wind turbines*

IEC 61400-3 (2009) *Wind turbine generator systems* – Part 3: *Design requirements for offshore wind turbines*

IEC 61400-6 (2016 draft, to be officially published) Wind turbines – Part 6 : *Tower and foundation design requirements*

8 FOUNDATIONS ON MONOPILES

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8. FOUNDATIONS ON MONOPILES

8.1. OVERVIEW

This chapter addresses the foundations of support structures of wind turbines of the monopod type. A monopod is a cylindrical structure (essentially made of steel) of a large diameter (B) with a lower part extending in the ground (penetration length: D). The buried part constitutes the monopod foundation and is designated under the term monopile (see: Figure 8.1).

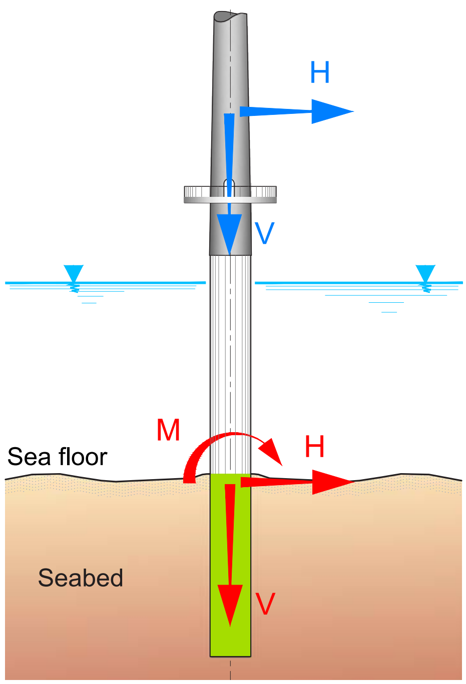


Figure 8.1: Schematic representation of a structure on a monopile

Monopiles represent the most widespread type of foundations for wind turbines installed so far at sea, due to the relative simplicity of their design and to their competitive cost in relation with the range of water depth of wind farms being developed until now (usually < 40 m). This type of foundations seemed at first limited to water depths ranging from 30 m to 35 m. Monopiles diameters, initially between 3 m and 5 m, are now commonly found between 6 m to 8 m for projects being currently developed. The increase of water depths for future wind farms, the increase of the turbines power and the improvement of the processes and capacity of production allow envisioning the manufacturing of monopiles with even larger diameters, up to 10 m. Monopiles are characterised by a low slenderness (D/B typically between 2 and 4).

The most commonly used installation methods for monopiles are: driving, and in the case of rocky soils: drilling (or a combination of drilling and driving). However, depending on the ground conditions that are met, other installation methods may be considered (vibrodriving for instance).

The different loads applied on an offshore wind turbine are thoroughly described in chapter 4. They can be represented by equivalent torsors:

- (H, V, T) at a depth above the bottom corresponding to the point of null moment;
- (M, H, V, T) at the sea floor level. This last torsor characterises loads applied to the foundation.

The terminology used is according to offshore practice: H horizontal force, V vertical force, M moment around a horizontal axis, T torsion moment around a vertical axis.

The horizontal load H and the overturning moment M often prove to be the main dimensioning components for the verification of the stability of monopiles.

8.2. DESIGN CRITERIA

Three criteria predominate the design of wind turbines at sea:

- the verification of the resistance against ULS (ultimate capacity) under combined loadings;
- the compliance with the criteria in displacement under SLS loadings;
- the response to fatigue under FLS loadings.

A specificity of monopiles is that the soil-structure interaction plays a major role, resulting in interdependent criteria. Iterations are required throughout the design process. The monopile is very often integrated in the structural model: either of the full wind turbine structure (coupled approach), or of the monopod (in other words: the sub-structure under the definitions of paragraph 1.3) in the framework of a semi-coupled approach. Ground response is most often included in the structural model under the form of non-linear p-y curves. In the case of a frequency analysis, a linearisation of the ground response will usually be carried out (as detailed in chapter 4).

Verification of the resistance against ULS: The verification of the ultimate capacity should be carried out under combined loadings. The effect of the soil degradation under cyclic loadings should be taken into account. The verification of the resistance against ULS is usually a determining factor for assessing the minimum penetration required for the monopile. The lateral resistance often proves to be the dimensioning component. Since the ultimate lateral resistance is often mobilised under significant lateral displacements, it is recommended to limit the displacements at pile toe and/or the degree of ground plastification (see, for instance, DNVGL-ST-0126, 2016 or the German guidelines EA Pfähle, DGGT, 2013). One should notably ensure that a slight decrease of the selected penetration length does not result in a significant increase of the displacement at the head. A practical way of ensuring this is to stay on the near-linear variation part of the « head displacement - penetration length » curve of the monopile.

Since the values of the ultimate loads depend on stiffnesses and dampings of the foundation under these same ultimate loads, several iterations are required between the geotechnical engineer (dimensions and stiffnesses of the foundation) and the structural engineer (loads calculations), to finalise the ULS loads.

Compliance with the SLS displacement criteria: Displacements accumulated throughout the life of the structure include settlements and rotations at the turbine level. Those shall comply with the rotation criterion set by the operator and/or the turbine manufacturer. This criterion is most often highly stringent (for instance: 0.5° with 0.25° reserved for installation tolerances). Complying with this criterion is one of the most compelling factor for the design.

Response to fatigue in FLS: The natural frequencies of structures of the monopile type are close to the frequencies of the sources of excitation (swell, wind, blades rotation). Foundation stiffnesses play a major role in the global response. In order to limit fatigue phenomena, natural frequencies should be set in relatively narrow ranges (see Figure 4.9). Therefore, foundation stiffnesses, and how they evolve with time, are parameters that should be controlled.

The analysis of natural frequencies is often a determining factor for the selection of the monopile diameter.

It should be noted that, because of the increase of the monopiles diameter and their installation in very stiff soils (for instance, rocks), the load generated by the rotation of blades 6P, or even 9P, could prove significant.

8.3. GLOBAL BEHAVIOUR OF A MONOPILE

A monopile is subject to vertical (axial) and horizontal (lateral) loads, and to torsion moments (torques). Because of the small rotations authorised by the operator and/or the turbine manufacturer, horizontal and vertical components can be respectively assimilated to axial and lateral components.

Under axial and torsion loadings, the response of a monopile is similar to the response of a pile.

The response of a laterally loaded monopile is highly conditioned by the relative stiffness of pile and soil, which results into a flexible or rigid behaviour, as shown in Figure 8.2.

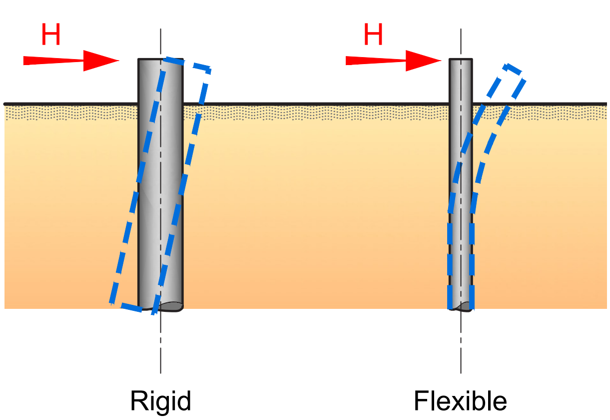


Figure 8.2: Comparison between the rigid and flexible behaviours of laterally loaded piles

In the case of a rigid behaviour, the lateral displacement of the pile does not depend, or very little, of the stiffness of the pile. In that case, only the soil or rock characteristics (stiffness and resistance) will determine the foundation response.

In the case of a flexible behaviour, for a given load and given ground characteristics, the displacement at the head of the pile is greater and all the more important with the deformability of the pile.

The relative rigidity of a pile anchored by a length D into an elastic and homogeneous ground can be assessed by using the notion of transfer length l_0, developed in the works of Timoshenko (1970) for a beam on continuous elastic supports (Winkler beam). This transfer length is expressed by:

$$l_0 = \left(4\,\frac{E_p \cdot I_p}{K_s}\right)^{0.25}$$

where the product (E_p . I_p) represents the bending stiffness of the pile (E_p Young's modulus of the pile and I_p area moment of inertia), and K_s is the modulus of horizontal subgrade reaction, expressed in kN/ml/m (K_s = B . k_s, with k_s being the coefficient of horizontal subgrade reaction, expressed in kPa/m).

The behaviour of the pile is infinitely rigid if the ratio l_0 / D is greater than 1, and infinitely flexible if l_0 / D is lower than 0.3.

Various criteria have been published following these works, based on this notion and involving the soil Young's modulus E_s (which is of the same order of magnitude than the modulus of horizontal subgrade reaction K_s). Poulos and Davis (1980) notably introduce a rigidity index I_R defined by the following expression, for a constant value of E_s:

$$I_R = \left(\frac{E_p \cdot I_p}{E_s \cdot D^4}\right)^{0.25}$$

In that case, the behaviour will be considered as being flexible if I_R is lower than 0.2 and infinitely rigid if I_R is greater than 0.7.

Table 8.1 provides a few examples of values of I_R for tubular metallic piles with usual dimensions for monopiles and typical soils and rocks.

Monopiles of conventional dimensions (i.e., outer diameter > 5 m, steel thickness > 50 mm and length 20 m – 40 m) have most often a rigid behaviour.

Note : Using this simplified criterion could lead to inferring a flexible behaviour in the case of very stiff rock masses. But in that case, displacements will remain very small.

A more rigorous approach, which takes into account the non-linearity of the ground response, is proposed in the recommendations of the SOLCYP research project (Solcyp, 2017, chapter 9). It confirms the prominently rigid behaviour of monopiles.

In the case of a rigid behaviour, the different components of the ground resistance are as outlined in Figure 8.3. They will be set out in paragraph 8.5. Generally speaking, reactions in the lower part of the monopiles contribute significantly to the global resistance.

Table 8.1: Rigidity indexes for tubular metallic piles

Monopile geometry	Young's modulus of the ground	$I_R = \left(\frac{E_p \cdot I_p}{E_s \cdot D^4}\right)^{0.25}$
Diameter B = 5 m Thickness 50 mm Length 40 m	E_s = 40 MPa (stiff clay) E_s = 100 MPa (dense sand)	0.26 0.21
Diameter B = 5 m Thickness 50 mm Length 40 m	E_s = 40 MPa (stiff clay) E_s = 100 MPa (dense sand)	0.36 0.28
B = 7 m Thickness 90 mm Length 40 m (soil) Length 25 m (rock)	E_s = 40 MPa (stiff clay) E_s = 100 MPa (dense sand) E_s = 200 – 2 000 MPa (very soft to moderately soft rock, rock mass)	0.39 0.31 0.42 / 0.24
B = 9 m Thickness 60 mm Length 35 m	E_s = 40 MPa (stiff clay) E_s = 100 MPa (dense sand)	0.49 0.39
B = 9 m Thickness 100 mm Length 35 m (soil) Length 25 m (rock)	E_s = 40 MPa (stiff clay) E_s = 100 MPa (dense sand) E_s = 200 – 2 000 MPa (very soft to moderately soft rock, rock mass)	0.56 0.44 0.52 / 0.29

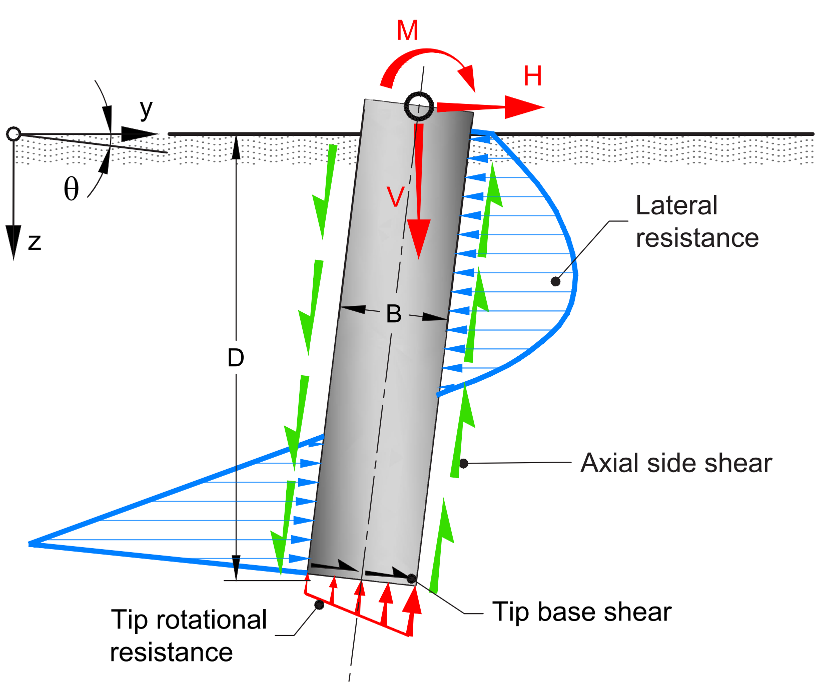


Figure 8.3: Representation of the resistance components of the ground under the lateral loading of a rigid monopile

8.4. BEHAVIOUR UNDER AXIAL LOADING

The approach used for the behaviour of piles under axial loading and in torsion (chapter 9) can be directly transposed to the case of monopiles.

8.4.1. AXIAL CAPACITY

The axial capacity (in compression or in tension) depends on the nature of the soil and on the mode of installation. Only the capacity in compression is to be considered for monopiles.

8.4.1.1. NON-ROCKY MATERIALS

In the case of monopiles driven into usual soils such as sands or clays, the calculation methods of the axial capacity of offshore piles (as described in API RP 2GEO (2011) or ISO 19901-4:2016 (E), 2016) should be used. Alternative calculation methods may however be considered if their use is justified on the basis of results of tests made on instrumented piles in similar ground conditions (representative tests).

In the case of carbonate sands, the recommendations of the ARGEMA/CLAROM (1994) guidebook should be taken into consideration. The main elements are recalled in chapter 9.

If other installation methods are considered (for instance, vibro-driven piles), the influence of the mode of installation on the axial capacity should be taken into account.

8.4.1.2. ROCKY MATERIALS

Off the French coasts, the most commonly encountered sedimentary rocks include chalks, marls and limestone rocks. Calcarenites, volcanic rocks (basalt) or metamorphic rocks (granite, gneisses) can also be encountered.

In the case of rocky materials, the common installation methods in use include driving (in the case of a sufficiently soft rock), drilling and grouting, and possibly a combination of both.

Experience feedbacks that have been published about the case of piles driven into rocks remain relatively scarce, and the uncertainty about skin friction resistance remains high:

- specifically, the particular case of piles driven into chalk can be mentioned, for which dimensioning elements are provided in the documents published by CIRIA (2002). Improvements on the basis of experience feedbacks have been presented by Carrington et al. (2011), as well as by Barbosa et al. (2015).
- in any case, the state (fracturing, degree of weathering, characteristics of joints/cracks) and stiffness of the rock mass will be key elements when designing piles to be driven into rocky materials.

A more thorough literature is available (Table 8.2) about the designing of drilled and grouted piles.

Table 8.2: Existing references for the calculation of the axial resistance of drilled and grouted piles in rocky materials

References	Type of rocks	Geotechnical parameters
CIRIA (2004)	Sandstones, marls, limestones, schists	Unconfined compressive strength (σ_c = UCS) Possibly: state of fracturing and elastic modulus
CIRIA (2002) **CIRIA (1979)**	Chalk	Nature of the chalk (CIRIA grade) For the calculation of shaft friction: angle of interface friction, possibly UCS (chalk of the CIRIA grade 'A') For the calculation of end bearing toe resistance: UCS / cone resistance of the penetrometer / number of SPT blows
NF P 94-262 (2012)	Chalk, marl and marl-limestone, weathered or fragmented rock	Data from the pressuremeter test or penetrometer test with limiting values (empirical calculations)
ARGEMA/CLAROM (1994)	Cemented carbonate sands and calcarenites	Angle of interface friction

8.4.2. AXIAL PERFORMANCE

The approach commonly used for pile design consists in simulating the axial response under the form of:

- t-z curves for the mobilisation of shaft friction along the monopile;
- q-z curves to represent the mobilisation of the end bearing resistance.

Types of t-z and q-z transfer curves are proposed in literature for types of soils that are commonly encountered.

8.5. BEHAVIOUR UNDER LATERAL LOADING

8.5.1. COMPONENTS OF THE GROUND REACTION UNDER LATERAL LOADING

According to paragraph 8.3 (Figure 8.3), the response of a laterally loaded monopile combines the following elements:

- a lateral resistance, often represented under the form of p-y curves, including the three following components:

- a frontal reaction (passive pressure of the ground);
- a reaction at the back of the pile, which can have the same direction than the frontal reaction (case of a clay without gapping), an opposite direction (active pressure in the case of a sand), or be null (case of a clay with gapping);
- a tangential shear strength along the shaft;

- shear strength at the base of the pile (possibly represented under the form of a T_B-y curve);
- axial shear stress along the shaft, generated by the rotation of the pile. These stresses lead to a resistant moment that can be modelled by M-θ curves distributed along the pile;
- a rotational resistance M_B at the base of the pile (usually not subject to modelling).

The four elements listed above are usually represented under the form of non-linear local transfer curves of soil response applied to an elastic beam.

The correspondence between the soil resistance components of a laterally loaded rigid monopile (such as the components introduced in Figure 8.3) and the four elements of the ground local response applied on an elastic beam is illustrated in Figure 8.4.

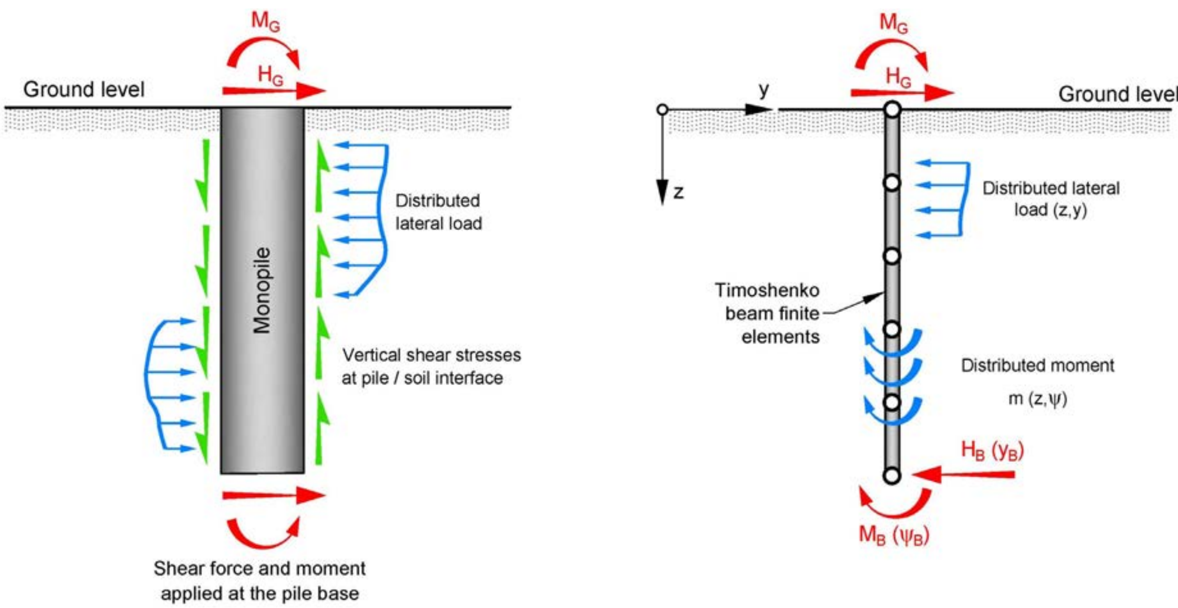


Figure 8.4: Correspondence between the components of the soil resistance of a laterally loaded monopile and the elements of the local ground response applicable to an elastic beam (from Byrne et al., 2017)

Within the framework of the PISA project, these four elements of the ground response have been quantified in dense sands and stiff clays. The outlines and philosophy of the approach advanced within this project, as well as a few elements useful for the dimensioning process, have been published in the articles of Byrne et al. (2015) and Byrne et al. (2017).

At time of the elaboration of the present document details were only available to the members of the PISA project. However, the elements published in the mentioned articles, as well as some partial project results that were publicly presented in May 2017, confirmed that, within the framework of the use of the PISA method:

- the rotational resistance at the base of the pile M_B has a negligible impact on the modelling of the behaviour of a laterally loaded monopile, including in the case of very short monopiles (see Figure 8.5).
- moments distributed along the pile provide a contribution to the lateral response that can be measured only for D/B ratios smaller than 3.
- when the D/B ratio exceeds 3, the contribution of p-y curves to the lateral resistance dominates (at least 70%, or even above 90% when D/B > 4).

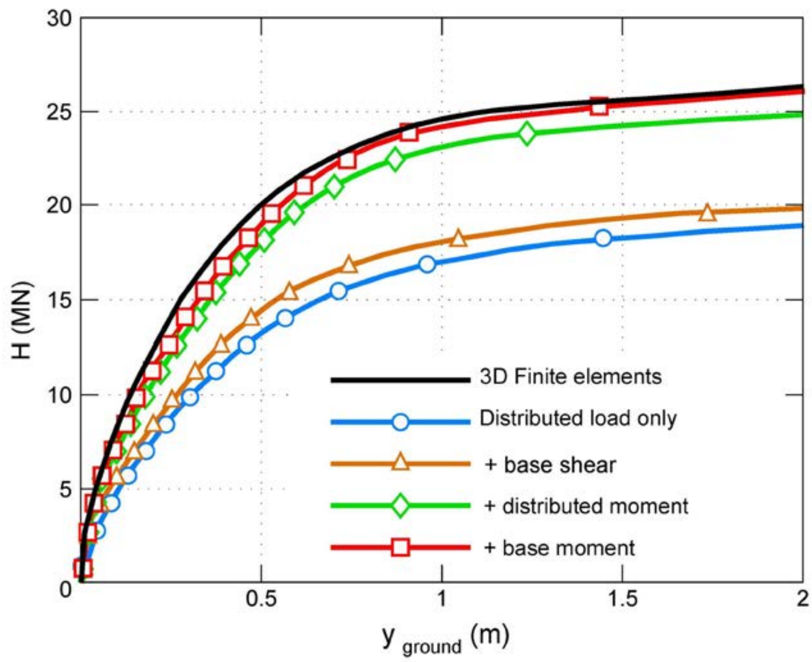


Figure 8.5: Impact of the 4 elements of the ground response on the lateral load curve H in function of the lateral displacement y of a very short monopile (penetration length = 2 diameters) in a sand (from Byrne et al., 2015)

Given the previous results, only the recommendations that are specific to the components of horizontal resistance (often represented under the form of p-y curves) and of shear strength at the base of the pile are detailed in the following paragraphs, since both these components should be integrated in the process of monopiles design.

Indeed, even if the M-θ curves distributed along the pile seem to contribute somehow significantly to the lateral response of short monopiles (D/B = 2), these results are specific to the PISA integrated method. In fact, even if the M-θ curves are not explicitly included in a design process, their influence on the behaviour of a monopile may be integrated into the p-y curves, provided they have been calibrated by FEM calculations where an appropriate ground model has been used (see paragraph 8.5.3).

8.5.2. USUAL P-Y CURVES (FOR PILES)

The behaviour of a laterally loaded pile is usually modelled through an approach of the p-y type, in which (Figure 8.6):

- the pile is assimilated to a beam on elasto-plastic supports;
- the ground response is expressed by the non-linear relations between the mobilised pressure p (or the mobilised resistance P= p.B) and the local displacement y (p-y curves or transfer curves). The slope from which the curves originate is the coefficient of horizontal subgrade reaction ki (or modulus of horizontal subgrade reaction $K_i = k_i$. B).

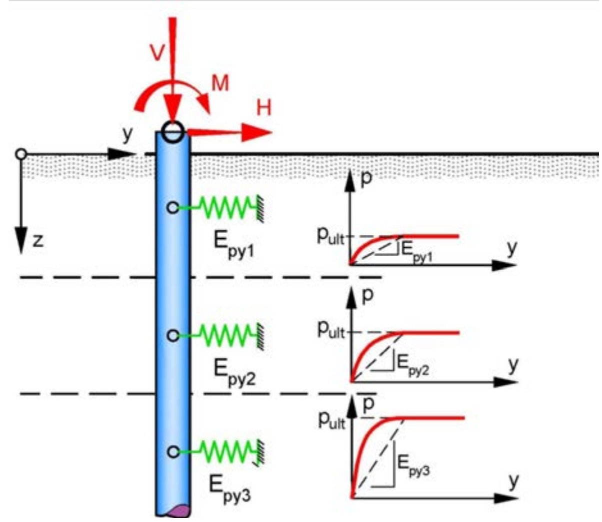


Figure 8.6: Lateral response curves p-y

Different p-y curves are available for a certain number of materials.

The standards applicable to offshore structures (API RP 2GEO, 2011, and ISO 19901-4:2016(E), 2016) provide p-y curves for materials of the following types: siliceous sands, normally consolidated clays and fissured overconsolidated clays. In sands and in normally consolidated clays, the p-y curves typically have a hyperbolic shape (Figure 8.7), which expresses the progressive plastification of the material.

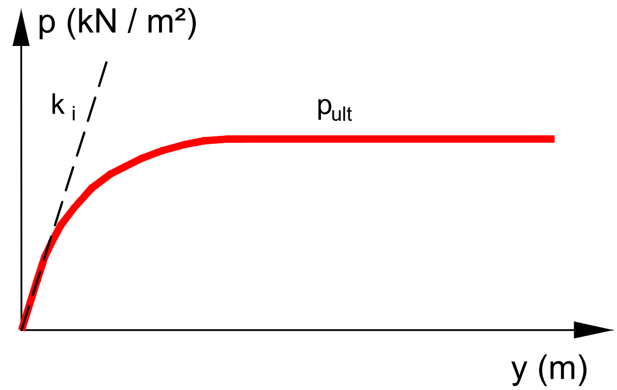


Figure 8.7: p-y curve in a sand

In the case of non-conventional soils, the lateral behaviour of piles is significantly less documented:

- p-y curves have been proposed by Wesselink et al. (1988) and Novello (1999) to simulate the behaviour of carbonate sands with low cementation;
- in the case of soft rocks (marls/claystones, limestone rocks), the p-y curves proposed by Fragio et al. (1985), Abbs (1983) and Reese (1997) are commonly applied in the offshore industry. These curves combine a first part of curve similar to the one of a stiff clay, with, at shallow depth, a second part where a residual resistance is mobilised (after a large displacement). The example taken from Fragio et al. (1985) is represented in Figure 8.8.

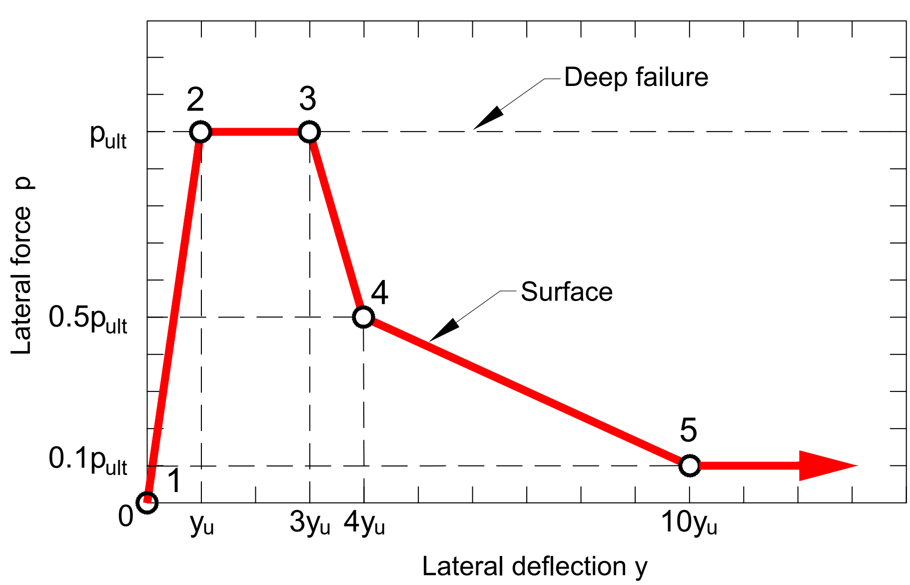


Figure 8.8: Example of a p-y curve in a soft rock (after Fragio et al., 1985)

More recent models (Erbrich, 2004) allow simulating with a higher realism the damage on rocks at shallow depth, which is a key parameter to study the lateral behaviour of a monopile.

French standards (NF P 94-262 – Annex I, 2012) propose lateral response curves built on the basis of results of pressuremeter tests. They may be taken into consideration in relation to the conventional field categories from the NF P 94-262 standard – Annex B (2012).

Some particular soils will require a specific approach. Amongst those, one may notably note the glauconitic sands and the volcanic rocks (including tuff).

8.5.3. P-Y CURVES ADAPTED TO MONOPILES (RIGID BEHAVIOUR)

Even though p-y curves have been established on the basis of tests conducted on piles with a relatively low diameter (essentially < 1 m), their application to the oil industry has been satisfactorily extended to the very long piles of jacket platforms, up to diameters of about 2 m. However, this application remained restrained to piles of greater slenderness than monopiles, having most often a flexible behaviour, as shown in Figure 8.2.

Initially, the concept of p-y curves has been directly transposed to the design of monopiles of greater diameters (4 m to 5 m). Experience feedbacks from offshore operators (Kallehave et al., 2012) have quickly pointed out that the response of the foundations designed that way was more flexible than in reality, which led to significant biases on the natural frequencies of the structures and on the amount of steel. Today, it has been established that a more appropriate approach should be carried out to calculate the response of large diameters monopiles.

The generic wording « diameter effect », which is often used to justify the usually stiffer behaviour of monopiles, is actually the consequence of various elements:

- firstly, the essentially rigid behaviour of a laterally loaded monopile involves several mechanisms, as explained in paragraph 8.5.1. The combination of these elements contributes to increase the stiffness and the global resistance of the system.
- secondly, the compliance with the operating criteria of the offshore wind turbines (limited rotation, frequency) is conditioned by the response of the foundation in the domain of small strains. This domain is not correctly described by the usual p-y curves derived for flexible piles, which have a behaviour that is rather governed by medium strain ranges (entry in the plastic domain).

More or less empirical approaches have been proposed by various authors to modify the initial stiffness of the traditional p-y curves, notably:

- in the case of sands, the works of Kallehave et al. (2012), of Sorensen et al. (2010), both summarised in Table 8.3, and of Kirsch et al. (2014);
- in the case of clays, the works of Stevens and Audibert (1979), who have introduced a corrective coefficient validated only for diameter ranges smaller than the ones of monopiles (< 2 m). In their publication, Kirsch et al. (2014) have also proposed an approach that is applicable in clay.

These approaches may be taken into consideration when carrying out preliminary studies.

Table 8.3: Comparison of the different methods of assessment of the initial stiffnesses of the p-y curves in sands (preliminary study step)

Reference	K_i = initial modulus of subgrade reaction (kN/ml/m)	Comments
API, ISO **Conventional approach**	$K_i = k_i.B = k.z$ k_i = coefficient of horizontal subgrade reaction (kN/m^2/m) k = increase gradient of the modulus of horizontal subgrade reaction with depth (in kN/m^3); function of relative density z = depth, in m	Tests on small diameters piles May be used as a reference curve, for comparison purposes only
Sorensen et al. (2010)	$K_i = k_i \,.\, B = C \,.\, \varphi^{3.6} \,.\left(\frac{z}{z_0}\right)^{0.6} .\left(\frac{B}{B_0}\right)^{0.5}$ φ : friction angle in radians C = 50 MPa z_0 = 1 m B_0 = 1 m z and B in m	Calibration on numerical analyses
Kallehave et al. (2012)	$K_i = k_i \,.\, B = k \,.\, z_0 \,.\left(\frac{z}{z_0}\right)^{n} .\left(\frac{B}{B_0}\right)^{0.5}$ (n between 0.4 and 0.7) z_0 = 2.5 m B_0 = 0.61 m z and B in m	Semi-empirical approach, attempt to reproduce the measurements of the natural frequencies of the Walney (UK) wind farm Density not specified

A more rigorous approach consists in linking directly the stiffness of the p-y curves to the value of the shear modulus G of the soil. For a given soil, this modulus depends on the distortion level, on the in- situ stress and on the number of cycles. Examples of variations of the shear modulus are given in chapter 6.

An alternative way consists in calibrating the p-y curves on calculations by finite elements, in which the ground behaviour under small strains is correctly taken into account, particularly if the axial shear along the shaft has not been separately modelled. A similar approach has been followed within the PISA project during the validation of this method. It may produce a multiplying coefficient that can be applicable to the parameter K_i or k_i. This multiplying coefficient depends on the level of loading, therefore on the displacements, and it decreases with them (see: illustration in Figure 8.9). In the example provided in this figure, the initial slope (stiffness) of the loading curve is equal to the stiffness of the ground at very small strains (proportional to E_0 or E_{max}), as shown by the upper curve. As lateral displacements increase, this slope decreases progressively and reaches about 1/10th of the initial stiffness when becoming close to the maximum resistance threshold (lower curve). The intermediate curve represents the curve of « true » response (progressive decrease of stiffness).

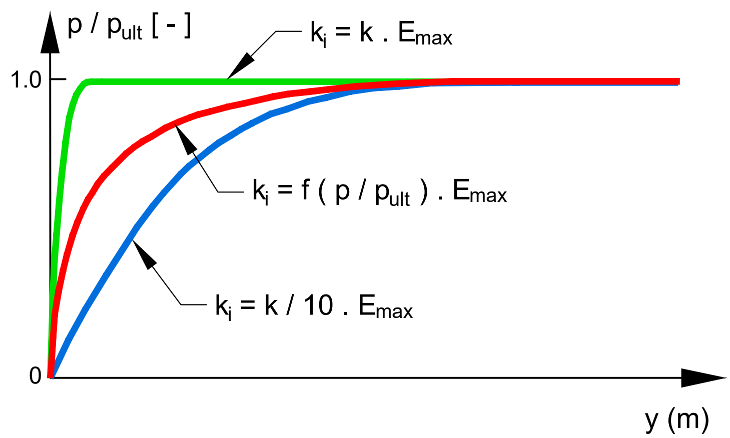


Figure 8.9: Variation of stiffness in the case of a p-y curve with a hyperbolic shape

Such an approach was implemented by Schroeder et al. (2015) for designing the monopiles of the Godewind (Germany) wind farm.

In non-conventional soils, implementing the approaches described above may require carrying out lateral loading tests. These tests may be carried out on the very site of the structure or on a land site having representative mechanical characteristics. They may be carried out at a reduced scale. The installation method should be duplicated.

In any case, it is highly recommended to equip a few wind turbines with monitoring systems, so that the validity of the selected hypotheses can be verified. Measurements will essentially deal with the natural frequencies of the foundation and of the structure, and with the displacements of the foundation.

8.5.4. T_B-Y CURVES

The mobilisation of shear strength at the base of the monopile can be represented by a curve having a bi-linear or hyperbolic shape, as shown in Figure 8.10. The shear strength that can be mobilised at the base of the monopile is assumed to be reached for local displacements of the order of 10 mm to 20 mm. These values are of an order of magnitude similar to the displacements calculated for a circular foundation having a diameter close to a monopile and being horizontally loaded in undrained conditions.

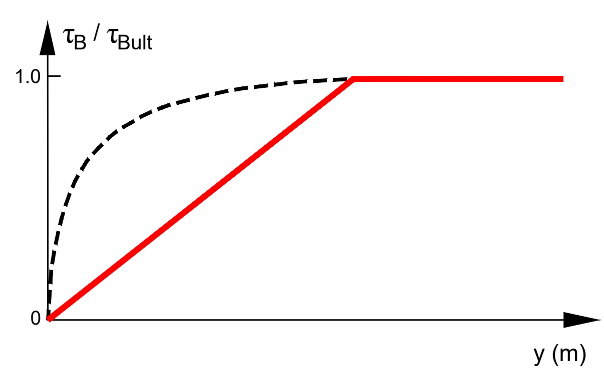


Figure 8.10: Example of a curve of mobilisation of shear strength at the base of the monopile

8.5.5. ULTIMATE LATERAL RESISTANCE

Assessing the ultimate lateral resistance can be achieved through one of the four following methods:

- the method of limit equilibrium, or plasticity analysis: the calculation is quick and allows visualising the mode of failure. Lateral resistance is often overestimated, but the method remains acceptable for a preliminary verification of the influence of the design parameters (diameter, length, ground resistance);
- the method of p-y curves: the p-y method is aimed at determining the full lateral response curve, including the ultimate resistance. It may be appropriate, provided that the modifications described in paragraphs 8.5.1 and 8.5.3 are integrated, so that all the mechanisms involved are taken into account. It may be combined to the use of T_B-y curves and possibly M-θ curves;
- the integrated method that is specifically developed for rigid monopiles (for instance, the PISA method);
- the finite elements method (FEM): it allows determining the full lateral response curve, including the ultimate resistance. The method is lengthy and hardly adapted to designing a large number of monopiles. However, it is well adapted to the calibration of simplified models (notably the p-y method).

In the event where pile tests would be available, their results should be used to calibrate the calculation methods of the ultimate resistance. This approach will prove particularly useful with non-conventional soils.

8.6. BEHAVIOUR UNDER COMBINED LOADINGS

8.6.1. AXIAL CAPACITY / TORSIONAL RESISTANCE INTERACTION

The shear strength or friction resistance profiles that will be selected to assess the bearing capacity can also be used to assess the ultimate resistance in torsion of the monopile.

In the case of a high torsion moment, the axial capacity may be reduced by mobilising a portion of the available friction. The interaction between the axial and torsional capacities can be represented under the form of failure envelopes. An example for caissons in clays (Taiebat and Carter, 2005) is represented in Figure 8.11.

In the case where the maximum vertical load, or where the maximum torsion moment transferred to the foundation, is smaller than 40% of the ultimate resistance (which will very often be the case for wind turbines on monopiles), the interaction between the torsion moment and the vertical loading will be very low, as shown by the failure envelope. In the event where the torsion moment is equal to 40% of the ultimate torsional resistance, the vertical capacity of the foundation is only reduced by about 5%.

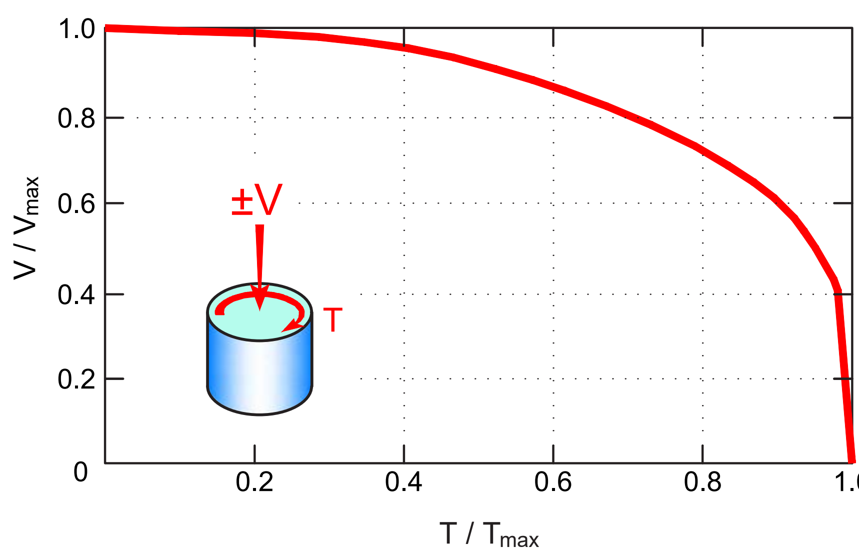


Figure 8.11: Example of an interaction envelope between the vertical capacity and the torsion moment in a clay in undrained conditions (from Taiebat and Carter, 2005)

8.6.2. AXIAL CAPACITY / LATERAL RESISTANCE INTERACTION

Generally speaking, failure envelopes of the V, H, M, T type are obtained using numerical modelling by finite elements.

In the case of monopiles, the moment at the head (M) is characterised by the height z such as M = H.z (with H being the horizontal load). Depending on the water depth, the order of magnitude of z, in the case of ultimate loads, ranges typically from 40 m to 60 m, i.e. once to twice the penetration length of the monopiles.

Examples of failure envelopes of caissons subject to a combination of vertical and lateral loadings (including moment) in clays have been developed. The works of Taiebat and Carter (2005) can notably be mentioned. Even though the authors are focusing on the behaviour of anchor caissons, the results can be slightly extrapolated, and lead to the conclusion that in the case where the vertical load is smaller than 40% of the bearing capacity (which will usually be the case for monopiles) there will not be a significant reduction in the lateral capacity.

In practice, the interaction between the vertical and lateral loadings will most often be negligible when designing monopiles, in both undrained and drained conditions.

8.7. BEHAVIOUR UNDER CYCLIC LOADS

The general principles of the integration of cyclic loadings into the design of offshore foundations are set out in chapter 6.

The degradation of the shear strength of clays and the accumulation of pore pressure in sands should be quantified on the basis of the estimated levels of shear stress (for instance, on the basis of a finite elements analysis) and of the results of cyclic laboratory tests (triaxial test or DSS) in undrained conditions.

In sands, the development of excess pore pressures resulting from the cyclic loading, as well as the partial concomitant dissipation of these pressures (in partially drained conditions) may be modelled (see the example in Figure 8.12).

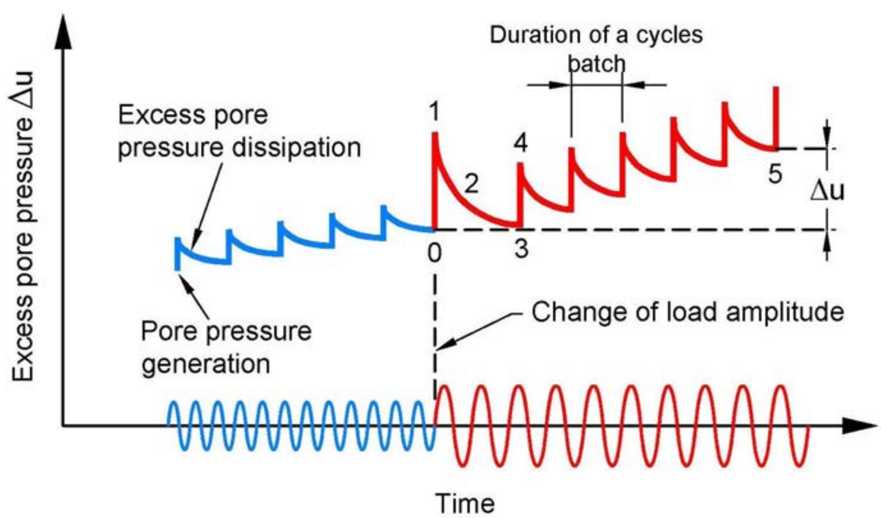
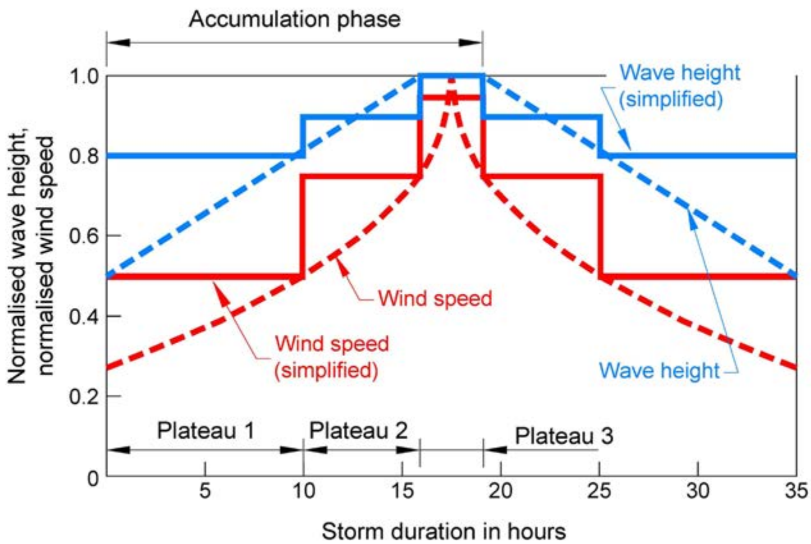


Figure 8.12: Example of generation and dissipation of pore pressure in a soil element (from Taiebat, 1999, quoted by Kirsch, 2014)

As an example, the results from a numerical simulation studying the effect of cyclic loading and the resulting potential accumulation of excess pore pressure on the lateral resistance of a monopile of the MEG I (Germany) wind farm are presented by Kirsch (2014) and Kirsch et al. (2014), see Figure 8.13. The example that is examined corresponds to a mixed stratigraphy (mainly loose to very loose sand, with a multi-metric layer of stiff clay):

- the upper diagram represents schematically the evolutions of wind speed (in blue) and of wave height (in green) during a 35 hrs project storm. The third plateau represents the peak of the storm. During this 3 hours period, the wave height under consideration is the significant wave height of this very storm.
- the mesh used for the numerical analysis of the monopile is represented in the intermediate figure (over the first 18 metres). The purple and pink layers correspond to silt and clay formations. The excess pore pressures at the end of the storm peak (3rd plateau) are represented on the bottom figure. These excess pore pressures are limited to the silt and clay layers.



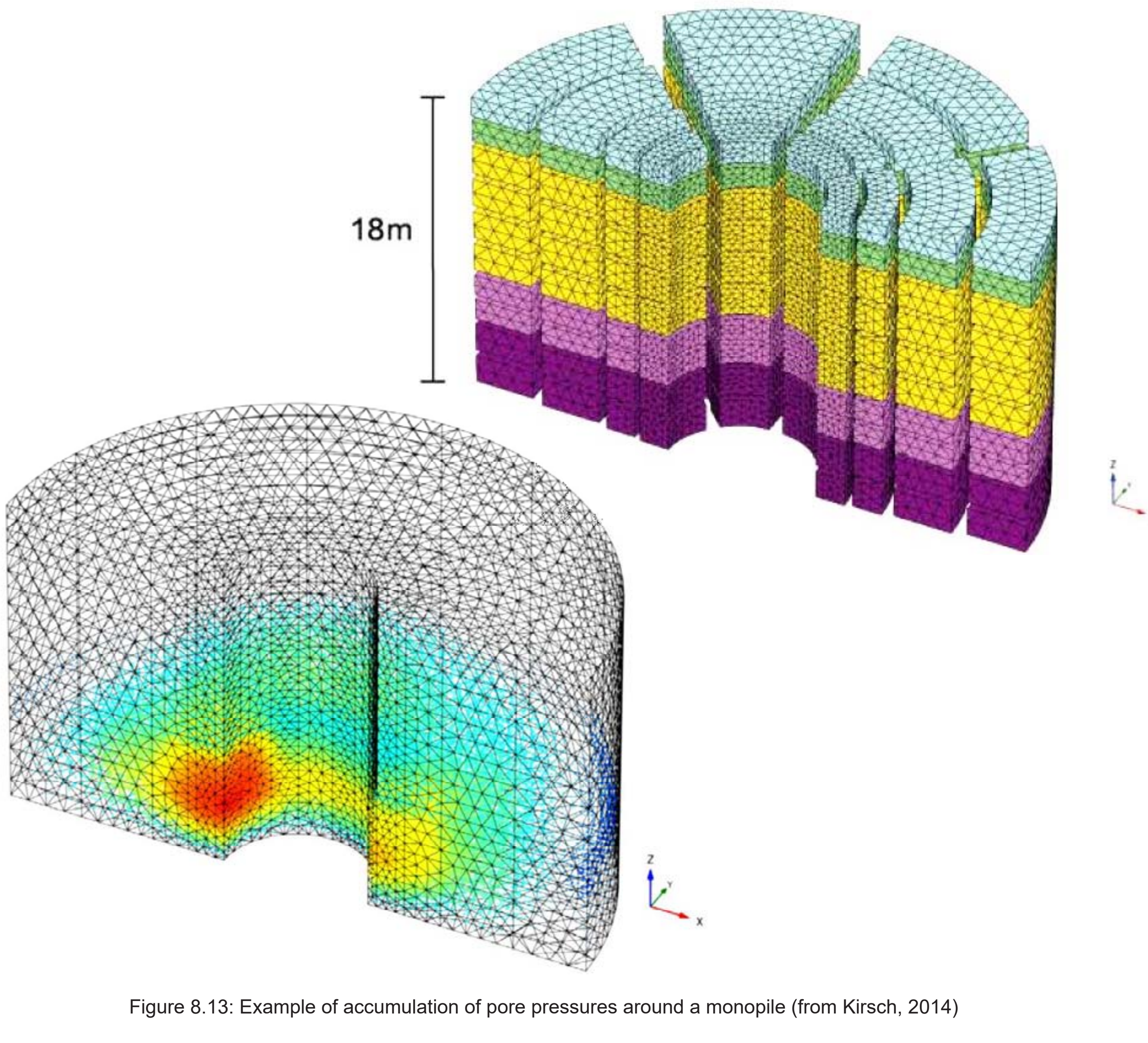


Figure 8.13: Example of accumulation of pore pressures around a monopile (from Kirsch, 2014)

In the case of sands, various methods in use to calculate the displacements/rotations under cyclic loading have been documented. One of the most widespread approach consists in assessing the permanent rotation due to cyclic loading using the rotation under static loading (calculated for the same maximum load), in function of the number of cycles:

- Hettler (1981) quoted by the German directives EA Pfähle (DGGT, 2013) proposes a law of logarithmic degradation in function of the number of cycles for rigid piles.
- LeBlanc et al. (2010) have developed, for rigid piles, a method in which the degradation law in function of the number of cycles also depends on the relative density of the sand, on the ratio between the amplitude of the applied cyclic load and of the lateral capacity, and on the asymmetry of the cyclic loading.
- very recently, Abadie et al. (2017) have presented a constitutive model that allows analysing a monopile response under a cyclic lateral loading in sands.

The SOLCYP approach is currently the most advanced one (SOLCYP, 2017a et b), since:

- the limit load H_{lim}, used as a reference to characterise the severity of cyclic loadings (H_{max}/H_{lim} ; H_m/H_{lim} ; $\Delta H/H_{lim}$ ratios), is clearly identified;
- the rigidity of piles is taken into account through a coefficient of rigidity CR (set out in chapter 9 of SOLCYP, 2017a et b), which allows reconciling the experimental data of displacements under cyclic loading obtained on flexible and rigid piles;
- the formulations that describe the accumulation of cyclic strains (y_N/y_1) proposed for sands integrate the near-totality of the experimental data that is currently available (tests on models and in-situ tests);
- formulations are also proposed for clays, even though they are exclusively based on tests results from models in a centrifuge.

8.8. VERIFICATIONS

8.8.1. PRINCIPLES OF THE VERIFICATIONS

The essential purpose of the **preliminary study** is to determine the geometry of monopiles, and to subsequently ensure that:

- firstly, the natural frequencies of the structure are correctly positioned in relation to the excitation frequencies. Here, the diameter is the major parameter;
- secondly, the vertical (axial) and horizontal (lateral) capacities are sufficient.

The global response of the foundation is mainly governed by its behaviour under lateral loading. The latter is usually modelled using reaction curves of the p-y type. This type of approach may be acceptable at that stage provided that the following elements are being considered:

- p-y curves should be defined following a mesh that allows ensuring that the calculations are accurate enough for the stratigraphy of the site. A sensitivity study on the refinement of the mesh may be considered to determine the adequate mesh, notably if thin layers are met;
- p-y curves should be adapted to the types of analysis being carried out (respectively: ULS, SLS and FLS). Several sets of curves should be produced, in function of the type of loading under consideration. The choice of parameters is addressed in chapter 6;
- p-y curves should take into account an increase of the lateral stiffness with depth z, consistent with the stiffness profile of the ground (therefore, usually a non-linear one). For that matter, correlated or measured profiles of the shear modulus at small strains G_{max} may be exploited;
- the mobilisation of shear strength at the base of the pile should be taken into account under the form of a T_B-y curve.

The vertical (axial) capacity of monopiles of large diameters is usually not a dimensioning element. It may be established with the methods conventionally used for piles (see paragraph 9.3).

At this stage, taking into account cyclic loadings:

- is not required for the verification of capacities (ULS);
- may be considered as a preliminary step to assess lateral displacements (SLS), under the condition that the available data is representative (see paragraph 6.3).

Within a **detailed study** (validation of proposals and final design), and beyond the elements that have been previously examined, one should ensure that the modellings under lateral loadings:

- integrate all components, notably the vertical resistances (M-θ effect), either by explicitly modelling them (with, for instance, the PISA integrated method), or by including them implicitly using calibrations from adequate tridimensional numerical analyses;
- appropriately take into account the effect of the strain level on the soil stiffness. The laws being used should have been validated/calibrated through tridimensional numerical analyses and/or piles tests (see paragraph 8.5.3).

Furthermore, the effect of cyclic loadings should be taken into account:

- for the verification of vertical and lateral capacities (ULS);
- for the assessment of permanent displacements (SLS);
- for the assessment of stiffnesses and their evolution with time (FLS and dynamic analyses).

8.8.2. PARTIAL FACTORS

The partial material and resistance factors applicable to soils are at least equal to the factors recommended in DNVGL-ST-0126 (2016) as set out in paragraph 7.3.

For monopiles driven into conventional soils (siliceous sands and clays), the applicable values in ULS conditions are provided in Table 7.3.

For other types of monopiles (for instance, drilled and grouted) and/or other types of soils (notably soft rocks), the recommendations of chapter 9 shall be perused, with Tables 9.5 and 9.7 for ULS conditions.

The material factors applicable to the characteristic parameters of the soil for SLS analyses (cumulated displacements) and FLS (fatigue) analysis are equal to 1.00.

To analyse the stresses in the monopile structure, the lateral resistance of the ground should be modelled with a resistance or material factor equal to 1.00 ($\gamma_M = \gamma_R = 1.00$).

8.8.3. DESIGN VALUES

The design values are documented in Table 6.2 of paragraph 6.2.5 of chapter 6.

8.8.4. FLS VERIFICATIONS (OR DYNAMIC ANALYSIS)

The geotechnical data necessary to the dynamic studies for each wind turbine includes:

- foundation stiffnesses estimated on the basis of the strain ranges that are adequate for these analyses, typically ranging from 10^{-6} for the lower bound to 10^{-4} for the upper bound (see: chapter 6);
- soil damping for the assessment of loads, including (see: chapter 6):
  - the hysteretic damping in function of distortion,
  - the radiative damping.

Experience feedbacks tend to prove that natural frequencies (therefore, stiffnesses) are frequently underestimated. It is also the case for soil damping, which leads to an overestimation of the loads applied to the foundation. In both cases, it very often results in overdimensioning the piles.

As a reminder, foundation stiffnesses are by far the parameter that most influences the analyses of natural frequencies and the analyses of structure fatigue, as well as calculations of forces. Damping essentially influences the calculations of forces, and has little impact on the calculation of frequencies, because of the levels of damping at stake.

Two main sets of soil-structure interaction data are commonly applied:

- interaction curves of the type: p-y, t-z, shear at the base of the pile..., which describe the non-linear behaviour of the soil;
- global matrixes of stiffness, including the coupling terms, linearised in a strain domain that is representative of the level of loading being studied.

8.8.5. ULS VERIFICATIONS

The stability of the structure under maximum loading should be verified, even though this criterion does not prove to be the most dimensioning one.

The ultimate lateral resistance will be often mobilised for major lateral displacements. Consequently, displacements at the head of the pile and/or the degree of plastification of the ground under ultimate loading should be verified as stated in paragraph 8.2.

The vertical (under the action of the vertical load combined to a torsion moment) and lateral capacities of the monopile should first be verified independently from each other. The potential interaction between both is addressed in paragraph 8.6.2.

Finally, since the lateral loading is mainly constituted of cyclic loads, the effect of the cyclic loading should be analysed:

- the recommendations presented in paragraph 7.4.4.4 of DNVGL-ST-0126 (2016) may be taken into consideration to select the loading conditions for which the cyclic degradation will be analysed. More specifically, one should pay particular attention to the cases:
  - of a single design storm (as contractually defined),
  - of an emergency shutdown or a storm following normal operating conditions,
  - of any other scenario that may be defined as covering the most critical ULS actions for the ground;
- the degradation of shear strength for clays and the accumulation of pore pressure in sands should be quantified on the basis of the results from cyclic tests in a laboratory (triaxial test or DSS) in undrained conditions, of the levels of the estimated shear stress (for instance, on the basis of finite elements analyses) and of the drainage conditions of the foundation;
- the degradation of resistance due to cyclic loadings is estimated from unfactored cyclic loads. The result is a cyclic resistance that corresponds to the loading being studied;
- the partial resistance factor γ_R of table 7.3 is applied on this cyclic resistance.

8.8.6. SLS VERIFICATIONS

As stated in paragraph 8.2, the accumulation of lateral displacements generated during the structure life should be assessed and compared to the maximum value set by the turbine manufacturer.

During preliminary studies, several simplified scenarii may be analysed in function of an equivalent number of cycles. For instance (further details are provided in paragraph 7.4.4.4 of DNVGL-ST-0126, 2016):

- single design storm: extreme maximum loading for an equivalent number of cycles (usually comprised between 5 and 20);
- emergency shutdown or storm following operational conditions;
- a series of storms (an example is presented in LeBlanc et al., 2010).

The methods of analysis are set out in paragraph 8.7.

During the detailed study, the real histories of loading should be used. The directionality of the cyclic loading may be taken into account to avoid overestimating the calculated displacements.

This was the case for the dimensioning of the monopiles of the Godewind wind farm off the German coast (Schroeder et al., 2015).

The settlements of monopiles supporting wind turbines will most often be small. A particular attention should be given to materials that are sensitive to creep (for instance, chalk).

8.9. INSTALLATION

Three methods of installation are commonly used (possibly combined to each other): driving, drilling and grouting, and vibrodriving. However, any other method of installation may be proposed, notably depending on the conditions of soils that are encountered. Only the three methods mentioned above are described in this paragraph.

At all events, and regardless of the considered solution, the impact of the installation method on the resistance and stiffness of the ground should be taken into account during the design process.

In some cases, drilling has been used as an additional method in cases of premature refusals with another mean of installation (notably during driving). This case is not addressed in this document.

A short summary for each method is presented in Table 8.4, including the available reference documents, their main advantages and drawbacks.

Table 8.4: Comparison of the methods of installation of monopiles

Methods	Reference documents	Advantages	Drawbacks
Driving with hydraulic hammer	• API RP 2A (2014), § 9.10 • ISO 19901-4:2016(E), 2016, § 9.2-9.7 • DNVGL-RP-C203 (2016): fatigue during driving	• Most widespread installation method for offshore structures (mature technology) • Fit for the majority of soils (including very soft rocks such as chalk...)	• Risk of damaging the monopile in rocky or glacial soils (erratic blocks/boulders) • Underwater acoustic pollution
Drilling & grouting	• NF EN 1536 (2010) • ISO 19901-4:2016 (E), 2016, § 9.8	• The only feasible method for some rocks	• High cost • Risk of cement loss in some formations • Drilling speed forecast is less usual than driving forecast
Vibrodriving	No standards	• Silent installation method • Fast installation in appropriate ground conditions (sand for instance) • The pile does not require support after the penetration stage under its own weight • On the basis of preliminary results from ongoing research projects, vibrodriving would not negatively affect the lateral capacity of monopiles	• High risk of premature refusal in glacial soils (to the exception of sands) • Low efficiency in cohesive soils

Driving

A comprehensive driving study should be carried out with the objective of selecting the appropriate hammer, of identifying the necessity of mobilising additional installation means in the event of a premature refusal, and of planning driving criteria adapted to the site geotechnical conditions.

At the very least, the driving study should include the following elements:

- assessing the profiles of soil resistance to driving (SRD) adapted to the conditions of the site. For that matter, back-analyses of driving carried out in similar geotechnical conditions may be used;
- driving analysis using a qualified software (based on the wave equation theory) in order to assess the number of blows and the maximum driving stresses, which will be compared to the driving criteria;
- an analysis of the fatigue of the pile steel, on the basis of the profiles of stress in compression and in tension inferred from the driving analysis;
- when crossing hard layers, the risk of overstress at the toe should be studied. The recommendations associated to this study should be presented (for instance, use of a driving shoe, limiting the energy delivered by the hammer).

In the case of monopiles driving, some points require a particular attention:

- use of a submerged conical structure overhanging the monopile. Gjersøe et al. (2015) have presented the results from numerical analyses that were carried out in order to assess the additional energy losses due to the conical structure;

- vents should be planned for in the higher part of the monopile to ensure the water inside the tube is drained away, and possibly injecting an air cushion under the hammer anvil. Any contact between the water retained inside the tube and the hammer anvil should be avoided, or otherwise driving efficiency would be considerably reduced;
- conventional models of wave equations are based on dampings of the Smith type, whereas radiative damping can play a significant trole in the case of large diameter piles;
- there is a risk of damage spreading from the pile toe during driving, notably because of the large ratio between diameter and thickness. The base of the monopile can be easily damaged locally, or globally ovalised, during installation operations, whether during the loading/unloading, the handling, the lifting or the driving (Holeyman, 2015). The risk of an initial damage occuring during driving and then spreading and growing should be analysed. Simplified equations such as the ones presented by Aldridge et al. (2005) may be taken into consideration at an initial design stage for a qualitative approach. They can allow highlighting the effect of an increase of the thickness of the monopile base over the risk incurred during driving. A more detailed approach may prove necessary for detailed studies, for instance with an extrusion analysis (Erbrich et al., 2011);
- if the risk of damage spreading from the toe is very high, a study on the risk of damage on the pile toe during the various installation steps should be carried out. During driving itself, the heterogeneity of the soil/rock resistance under the toe should be taken into account. The objective of this study should be to define the appropriate measures to be taken to best circumvent this risk (for instance, limiting the hammer energy during driving);
- in the case of difficulties met when crossing a hard layer, drilling the plug can be considered, with drilling possibly extending beyond the pile base, and then proceeding back to driving. During such operations, the integrity of the pile structure should be justified.

The installation of test piles equipped with monitoring systems (using strain gauges and accelerometers, see the specifications of ASTM D4945, 2012) may be considered at an early stage to validate the feasibility of the installation and the model of analysis, possibly by benefitting from the prior installation of the meteorological mast.

Monitoring the driving of a certain number of monopiles is particularly recommended to verify the energy transferred to the pile and the driving stresses, as well as the potential identification of a compression wave reflected from the toe (an indication of a very hard layer).

The installation method can affect the lateral behaviour of monopiles. It may be required to consider it during the dimensioning. Notably:

- the alterations of the mass properties of a soft rock during driving: rock fracturing and decrease of the radial rigidity;
- the modification of the ground-pile interface during driving in chalks and calcarenites: the formation of an annulus of a few centimetres of pulverised rock around the pile may influence lateral stiffness.

Drilling and grouting

In the case of the installation of a drilled and grouted pile, the following points should be considered with a particular attention:

- verification of how soils and rocks can accommodate drilling (degradation);
- verification of the excavation stability during and after drilling (prior to grouting);
- cement integrity;
- thickness of the grout annulus.

Vibrodriving

This method has most often been used while being combined with hammer driving, since it is suspected that vibrodriving affects negatively the axial capacity of vibrodriven piles. This is less critical for monopiles, since their axial behaviour is not dimensioning. As previously stated, there are ongoing research projects having the objective of determining if vibrodriving monopiles as the sole method of installation is acceptable.

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9 FOUNDATIONS ON PILES

9.1 OVERVIEW

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9.3 DRIVEN METALLIC PILES

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9. FOUNDATIONS ON PILES

9.1. OVERVIEW

This chapter addresses the foundations of metallic structures such as jackets (latticed structures) or multipods.

Jackets are commonly used as supports for sub-stations. In that case, the structures are heavily loaded, which generates high compression gravity loads in the piles. This case is not addressed within the present document.

Jackets can be used as supports of the turbine. In that case, the structure is lightly loaded and therefore subject to great variations of cyclic loading that are prone to generate significant pull-out forces. As of today, few jacket supports of wind turbines have been installed, but this number may quickly rise with the increase of water depths. The principle is illustrated in Figure 9.1.

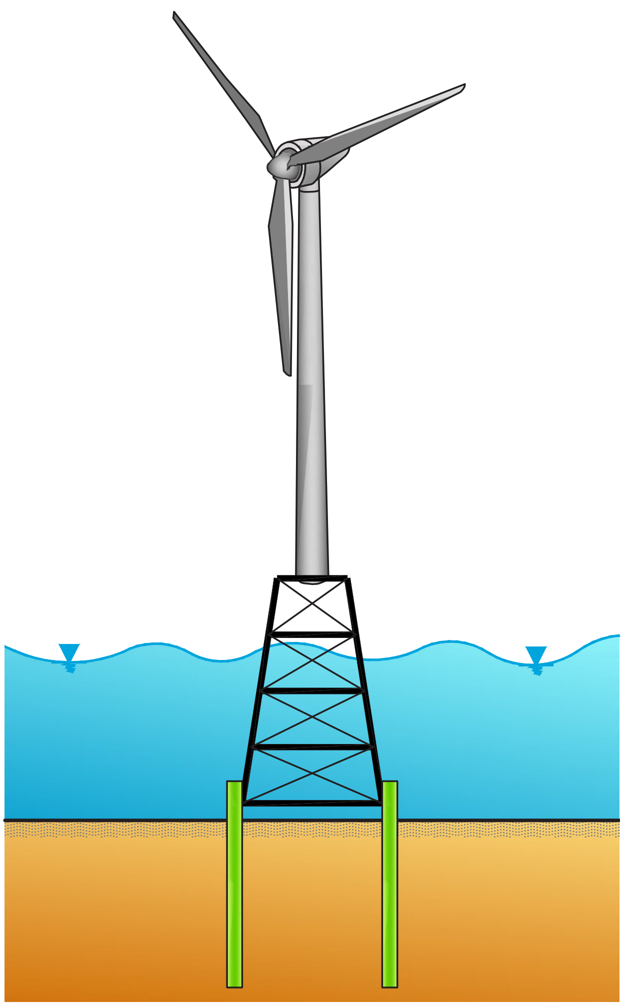


Figure 9.1: Jacket support of a wind turbine – Principle

Multipods, essentially from the tripod type, have sometimes been used as wind turbine supports. The principle is illustrated in Figure 9.2.

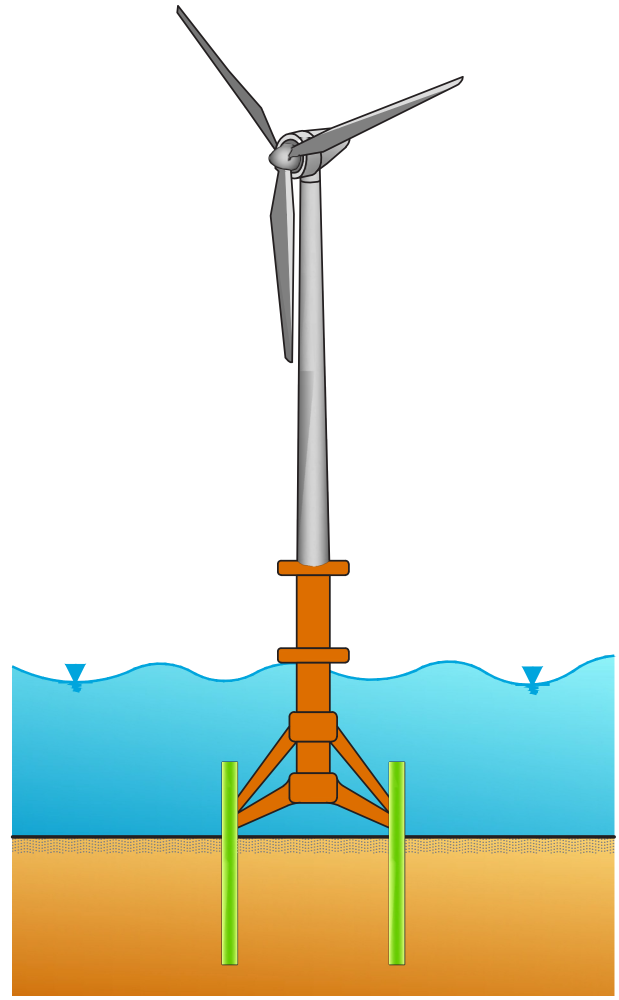


Figure 9.2: Wind turbine support of the tripod type

In the vast majority of cases, the piles of the metallic structures considered in this document are metallic (steel) tubes driven into soils or soft rocks, with a slenderness ratio greater than 10. The engineering for metallic piles driven into conventional soils (siliceous sands, clays, soft rocks) is thoroughly documented in the codes and standards applicable in the offshore oil and gas industry.

However, resorting to bored piles may be required when encountering harder rocks. Most often, those will be drilled and grouted piles, using the method usually implemented in offshore works: a metallic tube (« insert » pile) is placed in the drill-hole and cemented with a grout that fills the ground-pile annulus. One can also consider making the pile with the method carried out in land works: bored cast-in-situ reinforced concrete piles. The rock socket can have a small slenderness ratio (lower than 5).

As opposed to the case of driven metallic piles, the engineering of bored piles is only marginally addressed in offshore codes and requires references from land works.

This dichotomy is reflected in the organisation of the present chapter, which makes a distinction between the case of driven metallic piles (paragraph 9.3) and the case of bored piles (paragraph 9.4).

The metallic structures under consideration are subject to compressive vertical loads, due to the weight of the structure and of the various equipment, as well as to lateral environmental loads, which generate overturning forces. As a result, the force torsor at the head of the pile is always composed of:

- a vertical (axial) force of compression V_c or of tension V_t
- an horizontal (lateral) force H;
- a bending moment M;
- a torsion moment T (often neglected).

The principle is illustrated in Figure 9.3.

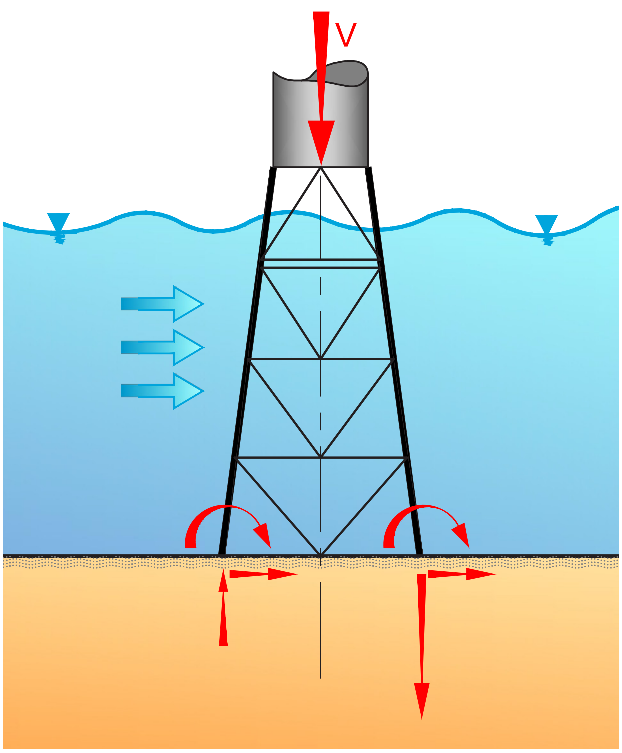


Figure 9.3: Schematic representation of loads acting on a latticed platform

The loads associated to this force torsor, factored or not, are used as a basis for designing the foundations. When researching the extreme loadings that condition the dimensioning of foundations, the developer shall consider the combinations of operating and environmental loads that produce the most unfavourable forces for the foundation.

During their installation, jackets are most often temporarily supported on shallow foundations, called mudmats. The verification of the temporary bearing capacity of mudmats foundations is addressed in paragraph 9.7.1.

9.2. DESIGN PRINCIPLES AND CRITERIA

The design of a structure of the lattice or multipod types is to a large extent bound by the site conditions (water depth and sea states). In practice, the diameter of the piles to be considered is more or less dictated by the structural concept.

The geotechnical design of the piles shall focus on determining:

- their axial capacity as a function of their length;
- their axial response;
- their lateral response;
- their structural integrity;
- the analyses required for their installation.

Assessing the capacities and responses of the piles should take into account the monotonous or cyclic nature of loadings. If necessary, group effects should be taken into consideration.

The axial and lateral responses of the piles should be used to assess the displacements (axial, horizontal an in rotation), and to determine the stiffnesses of the foundations that contribute to the response of the structure. In addition to the iterations made with the turbine manufacturer, iterations between the structural and the geotechnical engineerings may be required to optimise the design (dynamic response).

Other considerations should also be taken into account depending on the project, such as:

- erosion and scour;
- the overall stability of the site;
- seismicity (not addressed in the present document).

The design of piles shall satisfy the following limit states conditions:

- Ultimate Limit State (ULS): axial capacities in tension V_{ut} and in compression V_{uc} shall be verified, as well as the structural integrity of the pile under combined loadings (V, H, M);
- Service Limit State (SLS): it shall be verified that the axial and lateral displacements of the pile head remain compatible with the criterion of serviceability of the turbine. It shall also be verified that the stresses in the constitutive elements of the pile (steel, grout, concrete, frames) remain below limiting values related to the long-term deformation characteristics of these materials;
- Fatigue Limit State (FLS): it shall be verified that the natural frequency of the structure (lattice or multipod) is sufficiently far from the excitation frequencies of the swell and of the turbine, in order to limit the phenomena of fatigue, which shall be assessed in any case. Foundations stiffnesses play a significant role in the global response: their values and how they evolve with time are parameters that shall be addressed.

9.3. DRIVEN METALLIC PILES

The piles used in the offshore industry are most often metallic tubes that are open at their base. The diameters of these piles usually vary from 0.76 m (30'') to 2.14 m (84''). In the wind turbine industry, the trend is to use piles of large diameters, in an order of magnitude of 2 to 3 meters. The ratio B/e between the diameter of the pile B and the thickness of the tube e is comprised between 20 and 60, 20 being the limit curvature that can be reached by roll-bending equipments, and 60 the limit beyond which ovalisation issues may become critical.

Pile installation procedures depend on the installation mode of the latticed platform.

In the case of the simultaneous installation of the platform and the piles, two options are prevalently used (Figures 9.4a et b):

- either driving the piles through the platform legs, and fastening them at the head by welding or by grouting the pile-leg annulus;
- or driving the piles through sleeves used as guides. The piles are then grouted or swaged into the sleeves. The most massive platforms may require several sleeves at each of their angles to form a group of piles.

Installing piles on tripod platforms is assimilated to an installation by sleeve.

An alternative solution consists in pre-installing the piles through a guide frame or a template that can be reused, with the metallic lattice being connected to the piles afterwards (Figure 9.4c).

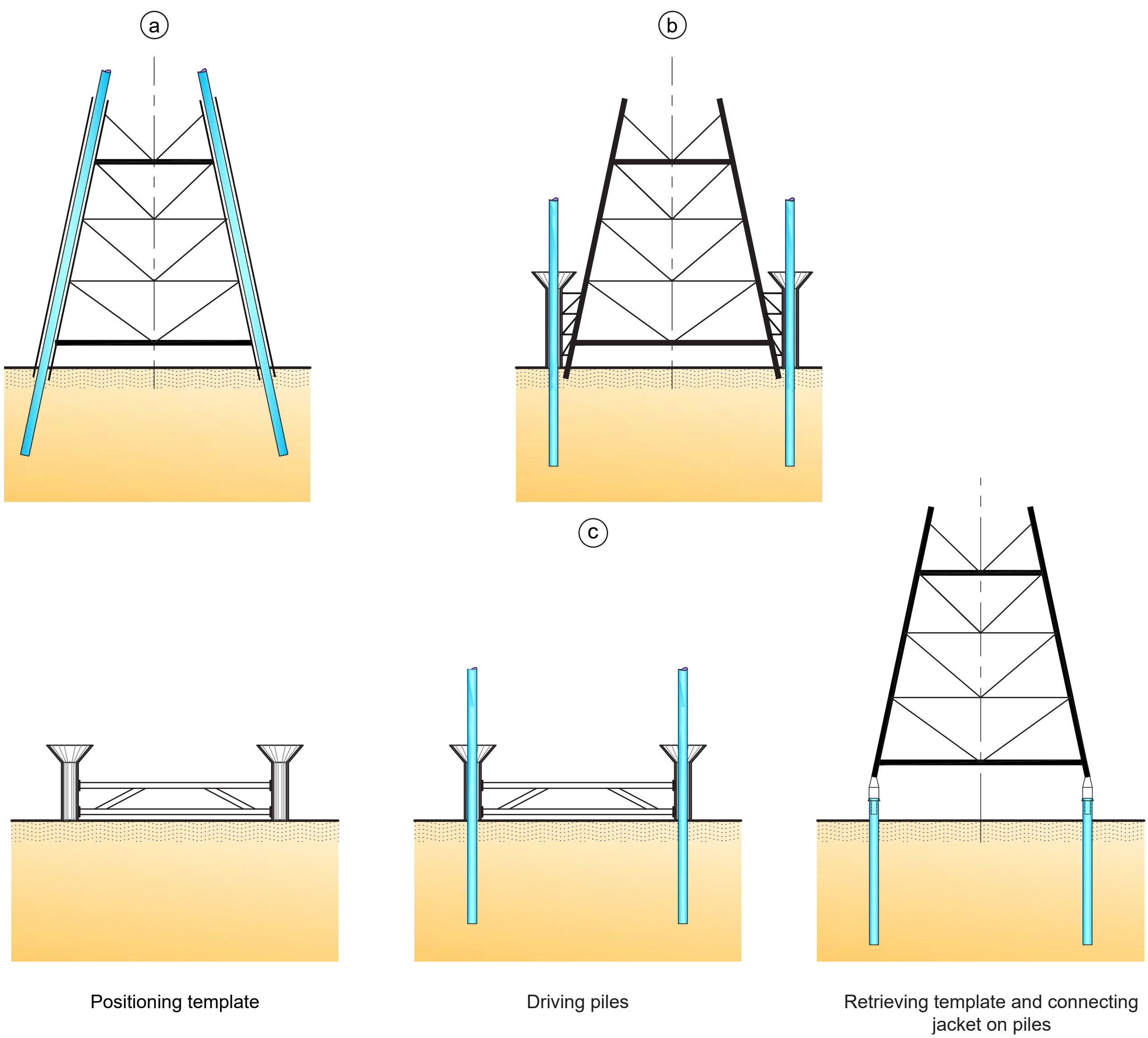


Figure 9.4: Illustration of piles installed in legs (a), in sleeves (b), and through a guide frame (c)

The use of driven piles is particularly adequate for soft soils (sands, clays, silts) or for soft rocks (marls, calcarenites, chalks...).

When encountering a hard layer of soil that can be an obstacle to driving, a combined pile can be carried out as an alternative. In that case, the plug is removed down to the base of the driven pile, a hole is drilled under the base of the driven pile, and then a tube (or « insert » pile) is placed and grouted in the drilled section (Figure 9.5).

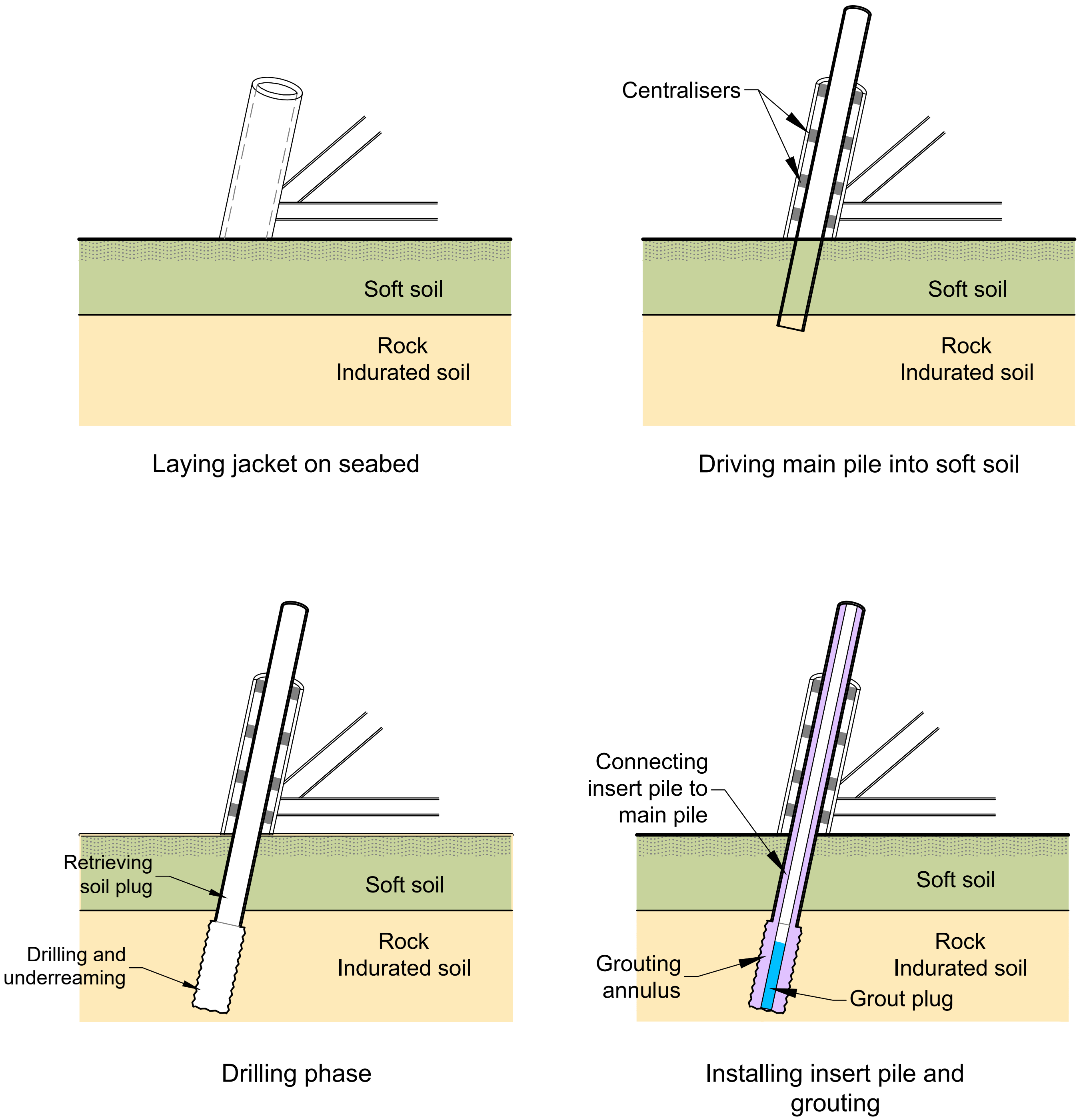


Figure 9.5: Diagram of implementation of an « insert » pile

9.3.1. BEHAVIOUR UNDER AXIAL LOADING

9.3.1.1. AXIAL CAPACITY

In the case of piles driven in non-rocky soils, the conventional methods to calculate the axial capacity of offshore piles may be used. References are, for instance, DNVGL-RP-C212 (2017) and ISO DIS 19901- 4 (2015) which include the commonly used methods.

The case of uncemented or weakly cemented carbonate sands requires a particular attention because of the high brittleness of the grains and the high compressibility of these materials. Driving piles into these materials leads to a crushing of the grains and a sharp drop of skin friction along the shaft. Typically, the limit skin friction is in a range of values between 5 kPa and 20 kPa. A method to assess the limit skin frictions in carbonate sands is proposed in the ARGEMA guidelines: *Foundations in Carbonate Soils* (1994). Limit skin frictions are expressed in function of the limit compressibility index of the material C_{pl} obtained from an oedometric test. The value of C_{pl} is by convention the tangential compressibility value C_{pi} for a pressure of 800 kPa (Figure 9.6). Typical C_{pl} values are indicated in Table 9.1.

Table 9.2 provides the values of limit skin frictions and limit end bearing capacities in function of the C_{pl} index.

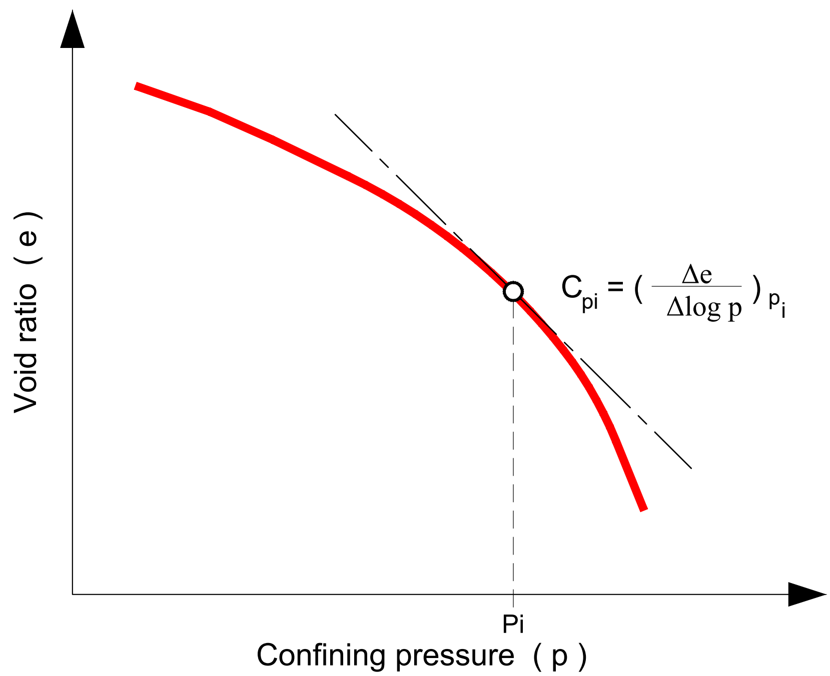


Figure 9.6: Definition of the tangential compressibility index

Table 9.1: Typical values of indices of limit compressibility in carbonate sands (according to ARGEMA-CLAROM, 1994)

Type of soil	C_{pl}
Siliceous sands	0.01 à 0.04
Detritic carbonate sands	0.01 à 0.04
Coral and shell carbonate sands	0.10 à 1.00
Algae carbonate sands	> 1.00

Table 9.2: Values of limit skin frictions and limit end bearing capacities for driven piles in carbonate sands (according to ARGEMA-CLAROM, 1994)

Limit compressibility index C_{pl}	Limit skin friction $f_{s\ lim}$ (kPa)		Limit end bearing capacity $q_{p\ lim}$ (MPa)
	Driven piles – open-ended (unplugged)	Driven piles – closed-ended (or open-ended with soil plug)	
< 0.02	100	120	≥ 12
0.02 à 0.03	50	100	12
0.03 à 0.04	20	50	10
0.04 à 0.05	10	50	8
0.05 à 0.10	5	20	4
0.10 à 0.20	0	10	2
0.20 à 0.30	0	5	1
0.30 à 0.50	0	2	< 0.55
> 0.50	0	0	

Calcarenites are assimilated to carbonate sands with a high to very high degree of cementation. Here, limit skin frictions can be very low (< 30 kPa) but can also reach several hundred kPa in the event of a very strong cementation (Beake and Sutcliffe, 1980; Settgast, 1980; Hagenaar and Van Seters, 1985; Gilchrist, 1985). Preliminary piles tests are highly recommended.

Another specific case is the one of chalk, which, under some conditions, may offer a very low resistance to driving, but allow significant axial friction over time. In fact, there are very different materials placed under the word *chalk*, from highly weathered puttified chalks, to high density intact chalks, going through all possible degrees of fracturing and weathering. The presence of nodules or beds of flint can further increase the complexity of the structure and of the behaviour of these materials.

The document « *Engineering in chalk* » from CIRIA (2002) proposes a detailed description of the chalk, and a classsification for engineering purposes that is based on dry density, on the opening and filling of discontinuities, and on the spacing of the latter. Overall, chalks are classified into puttified chalks of grade D and structured chalks. The structured chalks are characterised by their densities (low, moderate, high, very high) and their grade (A, B, C) in function of their state of fracturing.

The CIRIA (2002) document recommends, for open driven metallic piles, to select (in the absence of dedicated pile tests) skin friction values as indicated below:

- 20 kPa in chalks of low to moderate density;
- 120 kPa in chalks of high density and of grade A (non-fractured).

The French practice, set out in the NF P 94-262 (2012) standard, introduces three categories of chalks (soft, weathered, unweathered) on the basis of in-situ results (pressuremeter and CPT). For open-ended driven metallic piles, the authorised limit skin friction values for each category are summarised in Table 9.3.

Table 9.3: Open-ended driven metallic piles - Limit skin friction values in chalks according to NF P 94-262 (2012)

Category of chalk	Net limit pressure p_l*(MPa)	Cone resistance q_c (MPa)	Limit skin friction $f_{s\ lim}$* (kPa)
Soft	< 0.7	< 5.0	5 - 20
Weathered	0.7 - 3.0	5.0 – 15.0	20 - 40
Unweathered	> 3.0	> 15.0	40 - 50

* $f_{s\ lim} = q_{s\ max}$ under NF P 94-262 (2012)

Since the databases used either by CIRIA (2002) or NF P 94-262 (2012) are relatively limited, the proposed values should be selected with caution. Within the framework of a particular project, it is recommended to carry out preliminary pile tests. Progress has been made through experience feedbacks by Carrington et al. (2011) as well as Barbosa et al. (2015). One should note that recent, and unpublished, experiments of piles driven into moderate to high density chalks clearly show the formation of an annulus of pulverised chalk, with a relatively low thickness (a few centimetres) around the pile. The characteristics of this material most likely condition the ground-pile interaction (axially, and to a lesser extent laterally). But as of today, there is no documented study about this aspect.

In marls and marly limestones, the experience acquired in regions such as the Arabian Gulf seem to show that the ground-pile interface shear strengths of driven metallic piles are not affected by the carbonate content. The calculation methods mentioned in DNVGL-ST-0126 (2016) and ISO DIS 19901-4 (2015) applicable to clays may therefore be used. Furthermore, NF P 94-262 (2012) standard provides design parameters for piles driven into marls and marly limestones. These materials are classified into three categories (soft, stiff and very stiff) on the basis of in-situ tests (pressuremeter and CPT). For open-ended driven metallic piles, the limit skin friction values for each category are summarised in Table 9.4.

Table 9.4: Open-ended driven metallic piles– Skin friction limit values in marls and marly limestones according to NF P 94-262 (2012)

Category of marls or marly limestone	Net limit pressure p_l*(MPa)	Cone resistance q_c (MPa)	Limit skin friction $f_{s\ lim}$* (kPa)
Soft	< 1.0	< 5.0	70
Stiff	1.0 - 4.0	5.0 – 15.0	90
Very stiff	> 4.0	> 15.0	90

* $f_{s\ lim} = q_{s\ max}$ under NF P 94-262 (2012)

For metallic piles driven in chalks, in marls or in marly limestones, with formation of a plug (or possibly closed-ended piles), it is proposed according to the current state of knowledge, that the values of end bearing stresses, applicable on the total base section, be limited to:

- $0.30.m_c.q_c$ in the case where the assessment is based on cone penetrometer data;
- $1.45.m_p.p_l$* in the case where the assessment is based on pressuremeter data;
- $2.5.m_l.\sigma_c$ in the case where the assessment is based on laboratory data.

For metallic piles driven open-ended and unplugged in chalks, in marls or in marly limestones, it is proposed according to the current state of knowledge, that the values of end bearing stresses, applicable on the annular surface, be limited to:

- $0.75.m_c.q_c$ in the case where the assessment is based on cone penetrometer data;
- $2.30.m_p.p_l$* in the case where the assessment is based on pressuremeter data;
- $3.50.m_l.\sigma_c$ in the case where the assessment is based on laboratory data.

The factors m_c, m_p and m_l are lower than, or equal to unity. They should reflect the loss of resistance of the rock mass due to its degree of fracturing or weathering. These factors decrease when the degree of fracturing or weathering increases, or when the dimensions of the pile base are large with respect to the volume of ground mobilised for the type of in-situ (m_c or m_p) or laboratory (m_l) tests carried out.

These values of end bearing stresses are the maximum values that should be weighted in function of the expected axial performance of the pile (see paragraph 9.3.1.2). They assume an embedment in the soft rock being at least 5 times the diameter of the pile.

9.3.1.2. AXIAL PERFORMANCE

Assessing the response in deformation of an axially loaded pile is required to fully grasp:

- the effect of the pile flexibility, notably in the hypothesis of materials softening in function of the local displacement;
- the effect of cyclic degradation;
- the axial stiffness at the head of the pile, which is a component of the foundation rigidity matrix.

The response in axial deformation of offshore metallic piles is most often computed using local curves of load transfer, called t-z curves for the transfer of local skin friction, and Q-z curves for the transfer of end bearing resistances. In this simplified method, the pile is segmented into uniform sections. The axial stiffness of the pile is taken into account by a set of linear springs between each segment, and the ground response at the level of each segment is schematised by a non-linear spring modelled by a t-z curve for the response in friction along the shaft, and by a Q-z spring for the end bearing response. The advantage of this simplified method is that the structural numerical modelling is then facilitated using a finite differences or finite elements calculation software.

Response under static load

ISO 19901-4 (2015) proposes curves of axial load transfer under static loadings for usual soils: siliceous sands and clays.

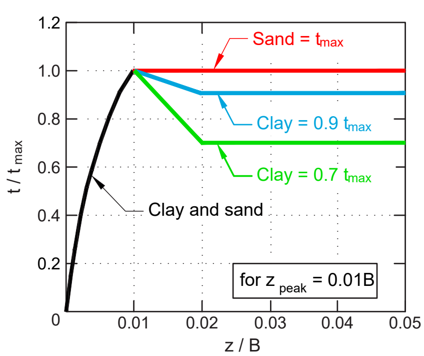
Figure 9.7 schematises the axial transfer curves t-z (for friction) and Q-z (for the toe) that are most commonly accepted. A summary on the origin of load transfer curves that were initially developed for API RP2A, then further recognized by ISO 19901-4, as well as other forms of curves, can be found in Reese et al. (2006).

No matter what the type of soil is (sand or clay), the initial part of the t-z curve is parabolic, defined in function of t/t_{max} values for a relative displacement z/B (Figure 9.7a). The maximum value t_{max} is reached for a displacement equivalent to 1% of the diameter. This value is indicated by default, and it is specified that it can be comprised in the 0.25% to 2% range, on the basis of the pile tests analysed in the database.

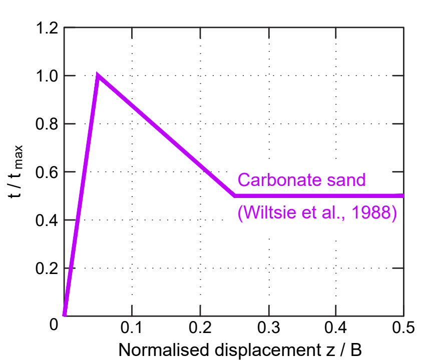
For clays, shear strength then decreases to reach a residual value to an additional displacement of 1% of the diameter. The residual value varies between 70% and 90% of the peak value, in an inversely proportional way to the degree of overconsolidation of clays according to Vijayvergiya (1977).

For siliceous sands, the curve continues on a plateau of constant friction $t = t_{max}$.

The response of carbonate sands may be described by the transfer curves under static loading proposed by Wiltsie et al. (1988). It is characterised by having soil-pile relative displacements higher than in siliceous sands, and by having an important softening of the post-peak friction (Figure 9.7b).



(a)



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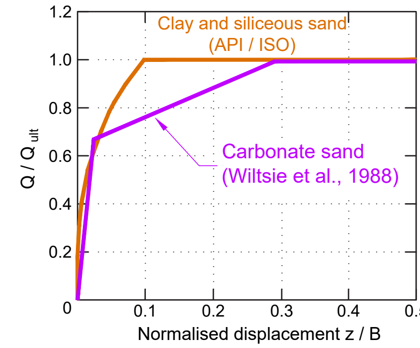


(c)

Figure 9.7: Examples of t-z and Q-z curves:
(a) t-z curves for siliceous sands and clays (ISO 19901-4, 2016); (b) t-z curves for carbonate sands (from Wiltsie et al., 1988); (c) Q-z curves

Effect of axial pile flexibility

During the calculation in deformation of a pile in a homogeneous ground, local skin frictions are progressively mobilised. If the pile is perfectly rigid and loaded in tension, the peaks of the transfer curves are reached simultaneously, and the integration of peak frictions t_{max} on the entirety of the lateral surface of the pile is equal to the ultimate resistance of the pile, as calculated by the methods set out in paragraph 9.3.1.1.

However, in the general case of a flexible pile, and when having a mechanism of mobilisation of friction with softening, some sections of the pile may end up in a post-peak situation while other may not have reached the peak (Figure 9.8). The maximum resistance of the pile obtained with a calculation in displacement will then be lower than the ultimate resistance of the pile as calculated with the methods set out in 9.3.1.1. The result can be generalised to the case of a loading in compression and to the case of multilayered grounds.

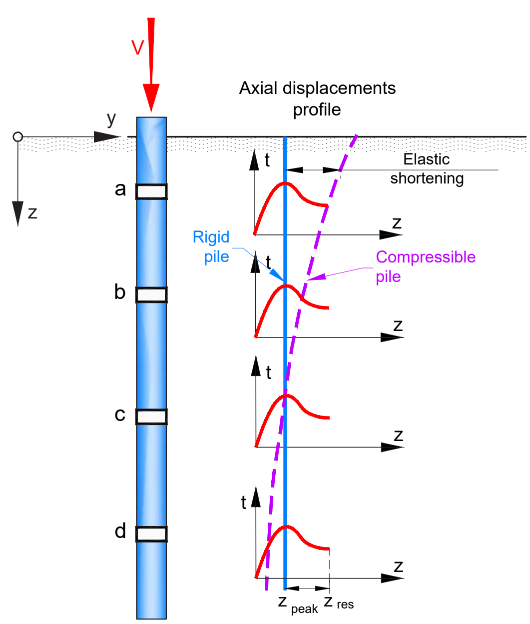


Figure 9.8: Softening of the axial friction and resulting axial flexibility of the pile

Cyclic loading

Current standards and regulations, in the offshore sector as well as in the land sector, stigmatise the potentially detrimental effect of cyclic loadings on the capacity of piles (degradation of friction) and on their performance (loss of stiffness). However, they do not currently provide any clear procedure to take into account this effect in a practical manner. One should notably note the absence of proposition of cyclic t-z curves, which would allow assessing the rate of cyclic degradation of friction in function of the number and severity of cycles.

The SOLCYP (2017) recommendations present a full synthesis of all the currently available data on the behaviour of piles under cyclic loadings. They address both experimental and theoretical aspects.

The foundation of piles of offshore wind turbines are identified in this document as being highly sensitive to cyclic loadings because of their low gravity component with respect to the environmental cyclic loadings, which have strong amplitudes and have an essentially alternated nature (« two-way »: tension/compression alternation). The verification of the safety of offshore wind turbines piles under cyclic loadings is essential.

For this verification, it is recommended to follow the SOLCYP strategy as described in chapter 5 of the SOLCYP (2017) recommendations.

The laws of cyclic degradation in conventional materials (sands and clays) can be obtained from laboratory tests of the CSS type (Cyclic Simple Shear) or the CNS type (Constant Normal Stiffness Shear).

Numerical tools with a sufficient reliability allow rendering the response of the pile under cyclic loading (degradation of capacity and modification of stiffnesses): SCARP (Poulos, 1989); RATZ (Randolph, 1986, 1994); TZC (Burlon, 2013).

In the case of sands, the SOLCYP (2017) recommendations propose an original method based on the determination of degradation laws from CNS tests. The degradation laws inferred from the tests can be used to:

- build cyclic t-z curves, of the envelope curve type, that present a reduction of the maximum friction in function of the number and severity of the cycles;
- calibrate the algorithms of the TZC software, which simulates the behaviour of the pile cycle by cycle;
- carry out a calculation by finite elements in which the degradation laws are explicitly used to manage the interface conditions within the contact elements.

A particular attention should be given to the role played by the end bearing capacity when it is significant. Instrumented pile tests in compression clearly show that cyclic loadings firstly affect the axial friction, this friction being mobilised for displacements of an order of magnitude ranging from 1 mm to a few mm.

The degradation of friction leads indeed to axial loads being progressively transferred to the toe, but to the price of large displacements of the pile, with end bearing resistances being mobilised for displacements of an order of a few centimetres to one decimetre. The axial stiffness of the pile is therefore detrimentally affected. During the process of assessing the effect of cyclic loads, it is recommended:

- either to, conservatively, take only into account the capacity in axial friction;
- or to limit the pile toe contribution to a fraction of the full end bearing capacity. This fraction should be compatible with the displacements of the pile that are acceptable in terms of stiffness. This fraction shall be justified and not exceed 20% of the ultimate end bearing capacity.

Some non-conventional, very compressible, soils, such as cemented carbonate sands, calcarenites or chalks are highly sensitive to cyclic degradation:

- in carbonate sands and calcarenites, it seems reasonable to accept by default that the very low values of limit skin friction admitted for driven piles integrate the impact of cyclic degradation;
- in chalks, available databases are very limited. Piles tests were recently carried out on behalf of the offshore wind energy industry (Barbosa et al., 2015; Buckley et al., 2017), and there is an ongoing collaborative research project in the United-Kingdom (ALPACA project, 2018). New approaches to take into account the effect of cyclic loadings on piles driven in chalk are expected within the next few years.

9.3.1.3. PILES TESTS

Generally speaking, the prior execution of piles tests is highly recommended. To be considered as being representative, these tests should be carried out:

- on the site itself, or on another site having similar geological and geotechnical conditions;
- on piles having sufficiently large dimensions, so that a scaling distortion does not occur compared to the real piles;
- with an installation technique (driving conditions, behaviour of the ground plug...) that is preferably similar to the one selected for the real piles;
- with conditions of static and/or cyclic loadings that are adapted to the functioning mode of the pile (tension/compression, static/cyclic…). In any case, the loading test programme shall include a static loading carried out until failure.

For further details, one may peruse the annex S of the NF P 94-262 (2012) standard, and the NF P 94-150-1 (1999) and NF P 94-150-2 (1999) standards for piles tests under static loadings, as well as the chapter 11 of SOLCYP (2017) for piles tests under cyclic loadings.

9.3.2. VERIFICATIONS UNDER AXIAL LOADING

Given:

S_d : the design load

R_{ks1} : the characteristic static resistance obtained from the calculation of axial capacity

R_{ks2} : the characteristic maximum resistance obtained from the calculation in displacement ($R_{ks2} \leq R_{ks1}$)

R_{kc} : the cyclic resistance for the design event. The cyclic resistance is obtained from the static resistance by taking into account the degradation due to cycles ($R_{kc} \leq R_{ks1}$)

It shall be verified that:

Static condition: $S_d < R_{ds2}$

Cyclic condition: $S_d < R_{dc}$

with: $R_{ds2} = R_{ks2} / \gamma_R$ $\quad$ $R_{dc} = R_{kc} / \gamma_R$

The partial resistance factor γ_R is expressed by:

$$\gamma_R = \gamma_{R0} \cdot \gamma_{R1} \cdot \gamma_{R2}.$$

with:

γ_{R0} : partial resistance factor as defined and recommended in DNVGL-ST-0126 (2016) for the considered conditions :

γ_{R1} : partial model factor, function of the type of material;

γ_{R2} : partial model factor, function of the loading direction.

The partial resistance factor γ_R shall not be less than γ_{R0}.

The calculation of cyclic degradation for the design event is carried out with unfactored loads (γ_F = 1.00) and unfactored soils properties (γ_M = 1,00).

9.3.2.1. ULS CONDITIONS

Given the reliability studies of foundations on piles that have been carried out for offshore structures, the partial factor that is applicable on the axial resistance $\gamma_R = \gamma_{R0}$ = 1.25, and recommended by DNVGL-ST-0126 (2016), is fully justified for the verification of the axial capacity of tubular metallic piles driven into conventional soils (siliceous sands and clays).

For chalks and carbonate materials (sands and calcarenites), it is recommended to apply an additional partial factor γ_{R1} because of the uncertainty of the models used to determine the capacity in these materials. This factor may be reduced, without being lower than 1.00, provided there are well-documented and representative piles tests that improve significantly the degree of reliability in the evaluation of capacity. These tests should be associated to tests of interface behaviour under static and cyclic loadings that allow fully capturing the phenomena involved.

The partial factor γ_{R2} takes into account the loading direction. For driven piles, γ_{R2} is equal to 1.00 when the pile is loaded in compression. For piles loaded in tension (non-permanent), a value of γ_{R2} = 1.10 is recommended in the case of conventional soils (siliceous sands, clays) and γ_{R2} = 1.25 in chalks and carbonate materials (sands and calcarenites). If representative tests piles carried out in tension are available, the value of the coefficient γ_{R2} may be reduced to 1.00.

The partial factors to be applied to cases of driven metallic piles are compiled in Table 9.5.

Table 9.5: Values of partial resistance factors in ULS for driven metallic piles

OPEN-ENDED DRIVEN METALLIC PILES				
Type of soil	γ_{R0}	γ_{R1}	**Compression** γ_{R2}	**Tension** γ_{R2}
Siliceous sands	1.25	1.00	1.00	1.10*
Clays	1.25	1.00	1.00	1.10*
Carbonate sands and calcarenites	1.25	1.20*	1.00	1.25*
Chalks	1.25	1.20*	1.00	1.25*

*these factors may be reduced following specific justifications (see text)

9.3.2.2. SLS/FLS CONDITIONS

Analyses in the SLS (displacements, stiffnesses) and in the FLS (fatigue analysis) shall be carried out without factoring the ground properties: γ_M = 1,00.

9.3.3. BEHAVIOUR UNDER LATERAL LOADING

9.3.3.1 OVERVIEW

Under the effect of a lateral load applied at the head (i.e. at the seabed level), a long (flexible) pile displaces itself laterally. Displacements are large on the upper part of the pile (z/B < 5) and may lead to local failure of the ground (plastification). Displacements then quickly reduce with depth, the consequence being that the lateral behaviour of the pile-ground system is widely governed by the response of the first few meters under the sea floor (z/B < 10). A diagram of the reaction of a laterally loaded long pile is presented in Figure 9.9.

In these conditions, the notion of lateral capacity of the pile is inadequate. The behaviour of the pile should be assessed in terms of:

- lateral displacement (under the effect of monotonous and cyclic loadings);
- lateral stiffness at the head of the pile, which is a component of the foundation rigidity matrix;
- pile integrity (bending resistance of the pile structure).

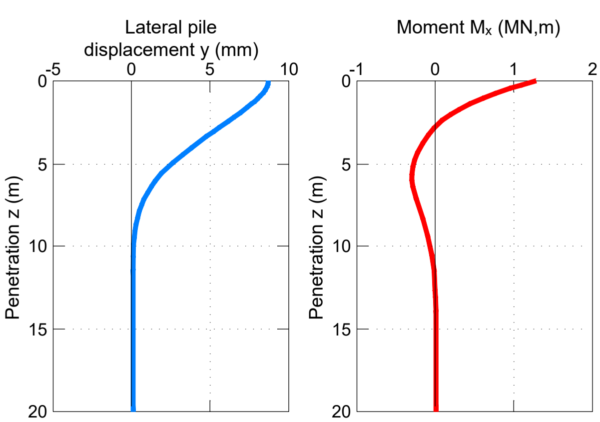
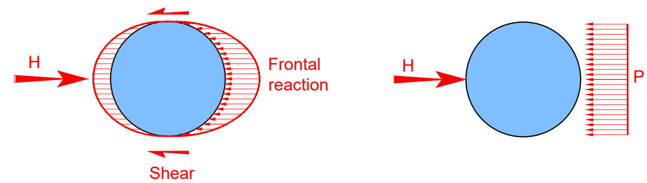


Figure 9.9: Horizontal displacement and bending moment distribution in a long pile (B = 1 m) laterally loaded at the head

9.3.3.2 P-Y CURVES

The response in lateral deformation of offshore metallic piles is most often carried out using local curves of load transfer, called p-y curves for the transfer of horizontal forces (lateral). In this simplified method, the pile is segmented into uniform sections. The flexural pile stiffness is taken into account by a set of linear springs between each segment, and the ground response at the level of each segment is schematised by a non-linear spring modelled by p-y curves. The advantage of this simplified method is that the structural numerical modelling is then facilitated using a finite differences or finite elements calculation software.

As indicated in Figure 9.10, p-y curves integrate all the effects of active/passive pressures, of shearing or suction/gapping that are likely to occur in a uniform section.



Note : the reaction on the rear of the pile may be:

- in the same direction as the frontal reaction (clay, no gapping)
- in the opposite direction (sand)
- nil (clay, with gapping)

Figure 9.10: Distribution of the ground reactions around a laterally loaded pile - Simplified model

Response under static loading

ISO 19901-4 (2015) proposes curves of lateral load transfer under static loadings for conventional soils: siliceous sands and clays.

For siliceous sands, the static P-y curves originate from the works of Murchison and O'Neill (1984), which are essentially based on the piles tests of Reese et al. (1974). They are characterised by an ultimate value P_{ult} and a hyperbolic shape of growth where the initial slope is linked to the gradient k_i of the modulus of horizontal subgrade reaction K_i (Figure 9.11). The values of P_{ult} and k_i are expressed in function of the internal friction angle of the sand.

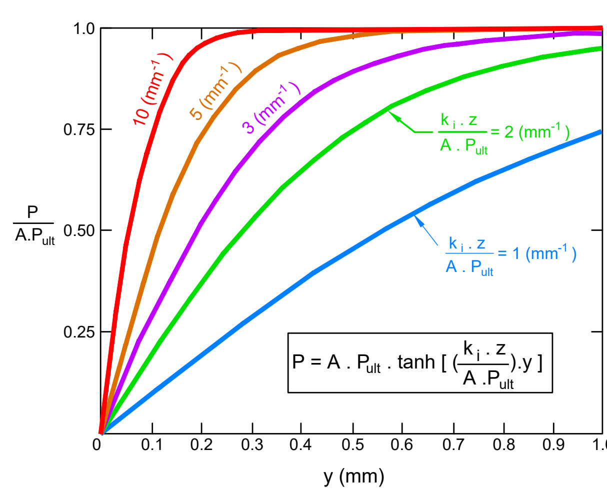


Figure 9.11 : Static P-y curves (sands)

P-y curves inferred from the works of Matlock (1970) are based on the interpretation of piles tests in normally consolidated lacustrine clays. The offshore tradition allows applying them to « soft » clays, but it is more appropriate to consider that their scope of application is for normally consolidated clays that do not have a pronounced tendency to softening. They are characterised by an ultimate value p_{ult} beyond a displacement value y_c, and a hyperbolic shape before reaching y_c (Figure 9.12). The value of y_c is expressed in function of the parameter ε_{50}, which is the axial strain of a clay sample measured at 50% of the limit deviator in an undrained triaxial test consolidated under the in-situ pressure.

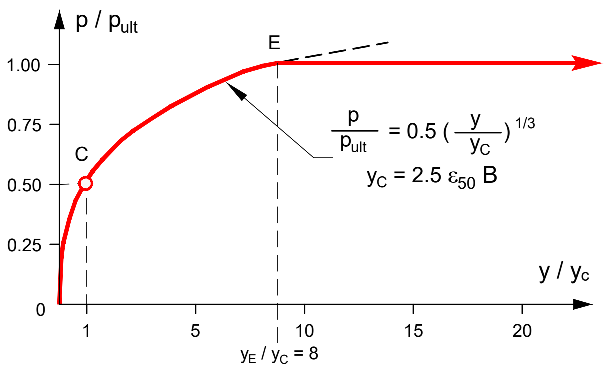


Figure 9.12: Static p-y curves (normally consolidated clays)

The p-y curves inferred from the works of Reese et al. (1975) are based on the interpretation of piles tests in fissured stiff clays showing a fragile response. Offshore tradition allows applying them to « stiff » clays, but it is more appropriate to consider that their scope of application is for overconsolidated clays having a pronounced fragile behaviour (which can be observed in undrained triaxial tests consolidated under the in-situ pressure). They are characterised by a peak followed by a sharp drop of the ultimate resistance, down to a residual value (Figure 9.13).

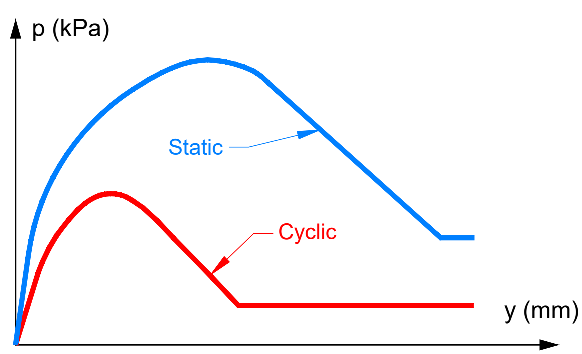


Figure 9.13: Static p-y curves (overconsolidated stiff clays)

The response of carbonate sands and soft calcarenites may be described by the transfer curves under static loading proposed by Novello (1999), on the basis of centrifuge tests carried out by Wesselink et al. (1988). P-y curves are characterised from the cone resistance values q_c.

Response under cyclic loading

ISO 19901-4 (2015) proposes lateral transfer curves under cyclic loadings for conventional soils: siliceous sands and clays.

For siliceous sands, the effect of cyclic loadings is taken into account by applying a reduction by default to the ultimate pressure p_{ult} over a height not exceeding 3 B. In surface, the reduction reaches 70% and decreases with penetration. The principle is schematised in Figure 9.14.

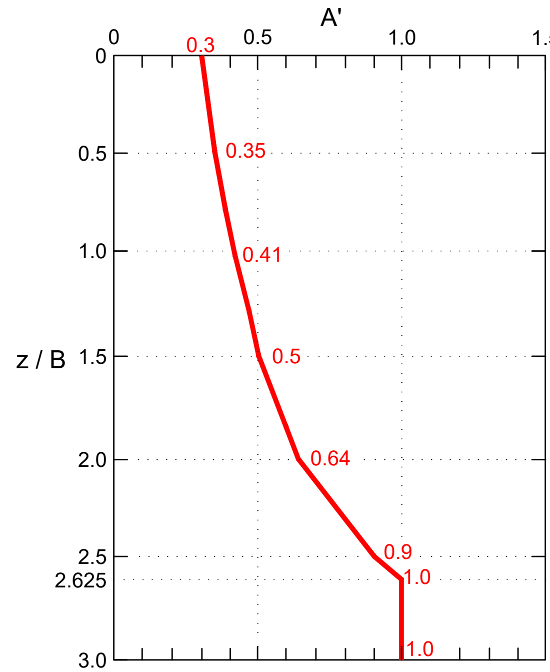


Figure 9.14: Reduction coefficient of the ground reaction P caused by a cyclic loading

In normally consolidated clays, the effect of cycles is reflected by a reduction by default of the ultimate pressure p_{ult} of 28% beyond a so-called "critical" depth z_R. Before reaching the critical depth ($z < z_R$), the reduction is more severe, notably at large relative displacements of the pile (Figure 9.15).

For stiff clays having a brittle behaviour, the overall shape of the static curve is similar, but cycles degrade the main characteristics: peak of resistance, displacement value to the peak, residual resistance (Figure 9.13).

These curves, often named with the wording "cyclic p-y curves", have been developed in the USA during the 70s on the basis of tests on piles having relatively small diameters (B < 1 m) subject to the cyclic loadings representative of Gulf of Mexico storms. They are envelope curves and aim at reproducing the displacement that the pile would incur under the quasi-static design load if that load was applied after the storm swept through (as opposed to the displacement obtained with the corresponding monotonous curve producing the displacement for the design load before the storm).

Understandably, the method is limited because it:

- is based on highly specific experimental conditions;
- does not account for the actual severity of the cyclic loading or the number of cycles that are effectively applied.

However, the method proved satisfactory when modelling the response of long piles of offshore metallic platforms, including for diameters in the 2m range.

The SOLCYP (2017) recommendations propose a « global » method, which in certain conditions allows determining the displacement at the head of piles under cyclic loading, in function of its displacement under static loading, of the severity of the cyclic loading and of the number of cycles. The displacement under cyclic loading is expressed under the following form:

- sands: $y_N/y_1 = 1+k_s.\log(N)$
- clays: $y_N/y_1 = k_a.\ N^{(m)}$

with:

y_N = displacement at cycle N

y_1 = displacement at 1st cycle (= displacement under monotonous loading)

N = number of cycles

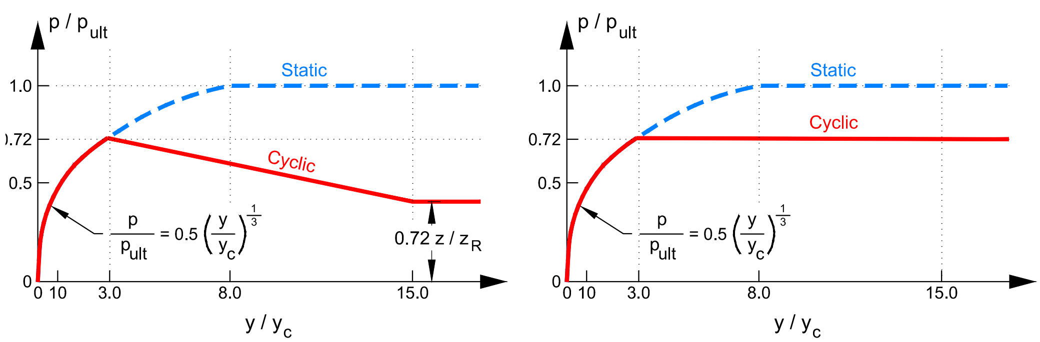
k_s = function of the pile-ground stiffness coefficient and of the characteristics of cycles (maximum cyclic loads H_{max} and cyclic amplitude H_c)

k_a = empirical coefficient calibrated on centrifuge tests

m = function of the characteristics of cycles (maximum cyclic load H_{max} and cyclic amplitude H_c)

The proposed formulations are derived from series of tests in macrogravity.

Resorting to the SOLCYP method may be useful when the "envelopes curves" method is deemed inadequate to capture the phenomena over the longer term.



a - Depth $z < z_R$ b - Depth $z > z_R$

Figure 9.15: Building of a cyclic p-y curve (soft clay)

9.3.4. VERIFICATIONS UNDER LATERAL LOADING

For analyses in SLS (displacements, stiffnesses) and FLS (fatigue analysis), partial materials or resistance coefficients (depending on the considered methods to build the p-y curves) shall be equal to unity:

$$\gamma_M = \gamma_R = 1.00$$

Calculations of the lateral displacements should take into account realistic conditions of liaison between the pile and the superstructure: most often, it is assumed that piles are interdependent with the platforms legs or the sleeves (fixed-head conditions).

Strictly speaking, there is no regulatory criterion to be satisfied on what constitutes an acceptable displacement of the pile. However, it is usually accepted that displacements at the level of the sea bottom should not exceed 10% of the pile diameter. This value may be acceptable in soft clays but in certain cases more severe criteria (< 5% of the pile diameter) may be applicable, for instance to ensure the ground will not ovalise, and to consequently avoid gapping and ground erosion when flushing occurs at the ground-pile interface under the effect of cyclic loadings. It will notably be the case in stiff clays. In rocks, very stringent criteria may be required to protect against rock fracturing or formation of wedges of chipped material near the surface.

Specific contingencies on pile displacements may be required to satisfy adequate conditions of foundations stiffness with regard to natural frequencies.

For the verifications of the integrity of the pile structure, piles deformations are calculated by applying partial material or resistance factors (depending on the methods used to build the considered p-y curves) equal to 1.00 ($\gamma_M = \gamma_R = 1.00$) in accordance with DNVGL-ST-0126 (2016). Moments in the pile and stresses in materials are deduced following the rules of resistance of materials, and it shall be verified that the combined stress for bending and axial compression/tension satisfies the criteria of section 13 of ISO 19902 (2011).

9.4. BORED PILES

9.4.1. OVERVIEW

The vast majority of bored piles that could be used for the foundations of offshore wind turbines off the French coast will most likely be installed with the method that is usually carried out in offshore works (Figure 9.16). The various steps required to install a pile of a nominal diameter B are as follows:

- drilling of a hole of a diameter « B + 2a » with a = annulus thickness (API RP2A, 2014, recommends a minimum value of a ~ 7.5 cm (3'') to ensure a proper grouting);
- descent into the pre-drilled hole of an open metallic tube of an outer diameter B, held in place using centralisers;
- grouting of the annulus with direct gravity injection of a cement grout;
- grouting continues in the annulus between the pile and the jacket leg, or between the pile and the sleeve, to ensure the transfer of forces to the structure.

However, the implementation methods of bored piles that are commonly used in land works may be considered, provided they are adapted to offshore works (Figure 9.17):

- installation of a guide tube of a diameter > B, which is either the jacket leg or the sleeve, to ensure forces are transferred to the structure;
- drilling of a hole of a diameter B, with the potential use of a drilling fluid;
- construction of a composite pile made in concrete with reinforcing cages.

Both methods present common features that impact on the design process of the piles:

- drilling decompresses the surrounding soil, as opposed to driving which leads to a partial or total soil displacement;
- drilling decompresses the ground at the base of the future pile. Furthermore, dredging the bottom part of the hole after drilling, and recycling the drilling fluid are awkward operations that do not allow ensuring the full removal of debris, and therefore cannot guarantee a proper contact on the entire surface between the ground and the grout or concrete. The mobilisation of end bearing resistance of drilled piles is therefore affected. The contribution of the end bearing resistance when calculating the axial capacity of the pile can therefore be « purposefully » reduced (see: paragraph 9.4.2.1);
- friction at the soil-pile or rock-pile interface is of the ground-grout or ground-concrete type;
- friction can be affected by the formation of a cake on the drilling walls when using drilling mud.

Recommendations in this chapter are applicable to both types of piles mentioned above.

Unless specified, recommendations are applicable to long piles (D/B > 10). The specific case of short piles anchored in rock (« socket piles ») is addressed in paragraph 9.4.2.

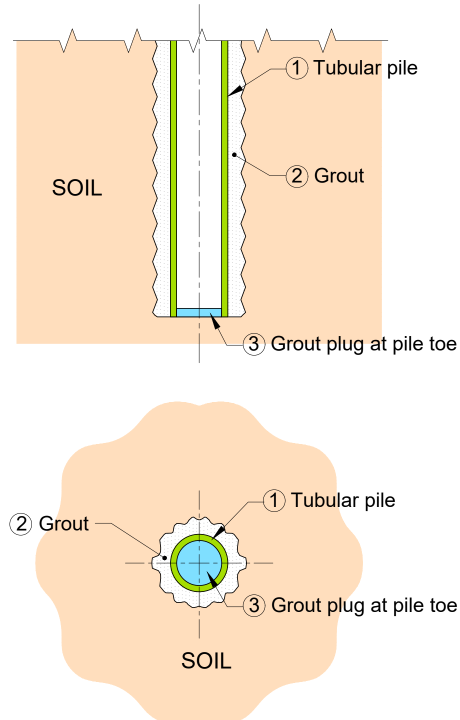


Figure 9.16: Drilled and grouted tubular pile

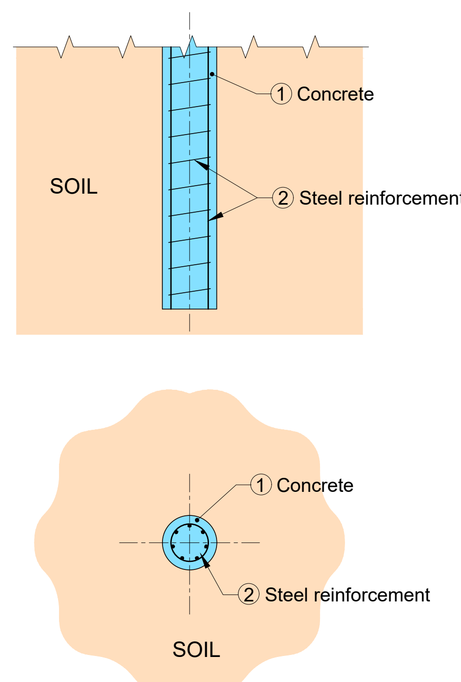


Figure 9.17: Bored cast-in-situ pile with reinforcement

In an offshore context, drilling is only considered when encountering very hard soils or rock. When soft soils overlay a hard level, a hybrid pile is usually implemented (Figure 9.5):

- driving of a tube in soft soils down to the top of the rock (main pile);
- excavation of the plug and drilling of a pre-hole in the rock;
- insertion of a tube of lower diameter ("insert" pile) and grouting of the annuli between the soil and the rock and between the insert-pile and the main pile, or construction of a reinforced concrete pile.

The types of soils/rocks considered in this chapter are:

- clays and stiff to hard marls;
- chalks;
- limestone and marly limestone;
- sandstones;
- schists.

The case of sands is considered only as a layer of low thickness interbedded in a ground profile.

9.4.2. BEHAVIOUR UNDER AXIAL LOADING

9.4.2.1. AXIAL CAPACITY

Piles installed following offshore techniques

For drilled and grouted piles, as executed in offshore works, it is required to pay attention to the behaviour of the pile-grout interface. In hard rocks, the shear strength at the pile-grout interface may be lower than the shear strength at the grout-rock interface.

ISO 19902 (2011) provides all the necessary elements regarding:

- the determination of steel-grout shear strengths;
- the criteria for designing « shear keys », that allow improving the shearing resistance at the steel-grout interface, if required.

In the absence of pile tests, CIRIA (2002) proposes to assess the shaft resistance of bored piles in chalks of low to moderate density, on the basis of a mean axial friction ζ_{sf} that would be expressed in function of the effective mean vertical stress σ_v' applied along the shaft using the empirical formula: $\zeta_{sf} = 0.8\ \sigma_v'$. For long piles (D > 30 m – 40 m) the friction values would exceed the limit values authorised by the French standard. For chalks of high density and grade A (intact), the mean axial friction ζ_{sf} would be expressed in function of the unconfined compressive strength of chalk σ_c with:

$$\zeta_{sf} = 0.1.\sigma_c.$$

In sands, it is commonly accepted to calculate the ultimate friction ζ_{sult} at the soil-grout interface with:

$$\zeta_{sult} = p_g'.\mathrm{tg}\ \delta$$

with:

ζ_{sult}: ultimate friction at the grout-sand interface at the considered level

p_g': effective pressure exerted by the grout column before curing at the considered level

δ: friction angle of the interface between grout and sand. It is recommended to consider the friction angle of sand at constant volume (φ'_{cv}).

One should note that ARGEMA (1994) recommends limiting shaft friction at 50 kPa in carbonate sands.

In poorly to moderately cemented carbonate formations (typically: σ_c < 5 MPa), ARGEMA (1994) recommends referring to the method proposed by Abbs and Needham (1985) to determine the ultimate shaft friction. The latter is given in function of the value of unconfined compressive strength σ_c (Figure 9.18).

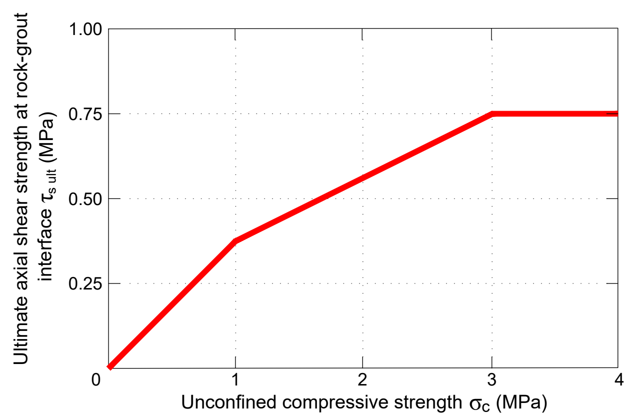


Figure 9.18: Ultimate shaft friction at the carbonate rock - grout interface (from Abbs and Needham, 1985)

In soft to moderately strong rocks (non-carbonated), high friction values are susceptible to be mobilised. The works of Rosenberg and Journaux (1976), Horvath (1978), Horvath and Kenney (1979), Meigh and Wolski (1979), Rowe and Armitage (1987) suggest that the ultimate friction at the grout-rock interface can be expressed with a relation in the form:

$$\zeta_{sult} = \alpha.\ (\sigma_c)^{\ m}$$

with:

- σ_c: unconfined compressive strength of the intact rock
- m = 0.5
- α comprised between 0.2 and 0.45.

The relations proposed by various authors are documented in Table 9.6 and illustrated in Figure 9.19. The uncertainty is high and highlights the limitations of the method, which is based on the sole value of σ_c, measured on intact samples, and which therefore does not take explicitely into account the actual state of fracturing of the rock mass.

Table 9.6: Values of ultimate shaft friction at rock-grout interface based on the unconfined compressive strength σ_c of the rock

Pile calculation method	Ultimate shaft friction $\tau_{s\ ult}$ (MPa) based on the unconfined compressive strength σ_c (MPa)
1.Rosenberg and Journeaux (1976)	$0.375\ (\sigma_c)^{0.515}$
2. Horvath (1978)	$0.33\ (\sigma_c)^{0.5}$
3. Horvath and Kenney (1979)	$0.2 - 0.25\ (\sigma_c)^{0.5}$
4. Meigh and Wolski (1979)	$0.22\ (\sigma_c)^{0.5}$
5. Williams and Pells (1981)	$\alpha.\ \beta.\ \sigma_c$
6. Rowe and Armitage (1987)	$0.45\ (\sigma_c)^{0.5}$

The works of Williams and Pells (1981) propose to express the ultimate shaft friction with the expression:

$$\tau_{sult} = \alpha.\ \beta.\ \sigma_c$$

with:

- α: decreasing function of σ_c, as shown in Figure 9.20;
- β: a reduction factor, function of the mass factor j_m under Figure 9.21.

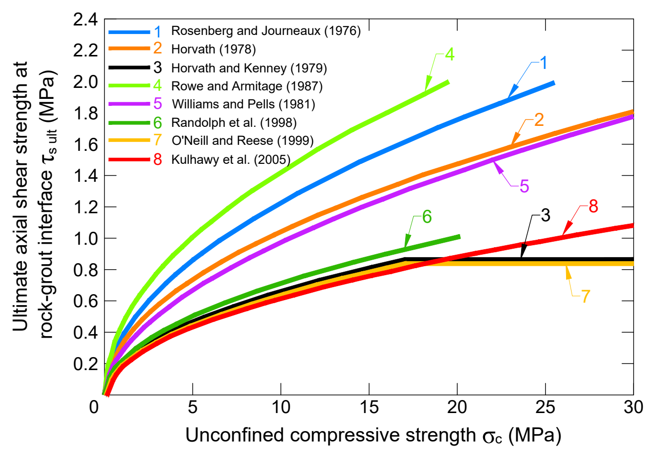


Figure 9.19: Ultimate shaft friction at rock-grout interface (non-carbonate rocks)

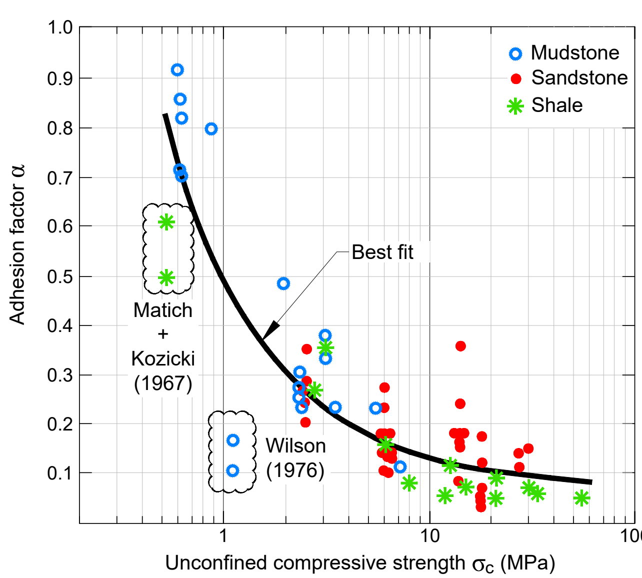


Figure 9.20: Ultimate shaft friction at rock-grout interface – Adhesion factor α (from Williams and Pells, 1981)

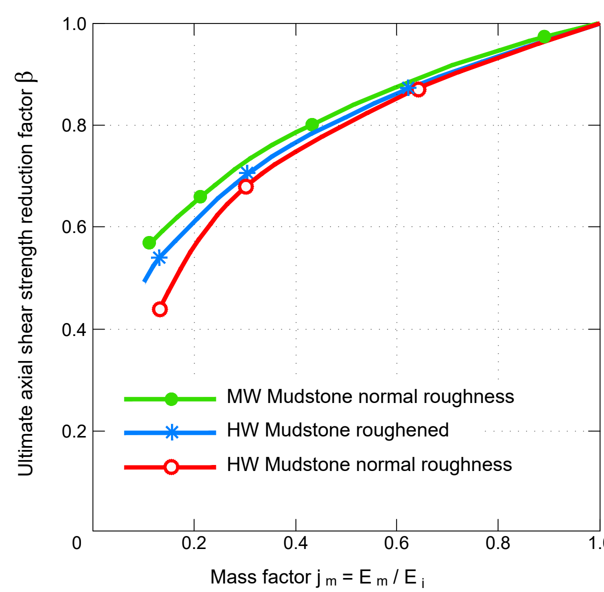


Figure 9.21: Ultimate shaft friction at rock-grout interface – Reduction factor β (from Williams and Pells, 1981)

More recently, methods (e.g. Kuhlawi and Carter, 1992) have been developed for the design of « socket piles », which are characterised by short penetration lengths and a high pile-rock relative axial rigidity.

The working principle is illustrated in Figure 9.22. The axial resistance of the pile is mainly governed by:

- firstly, the geometry of the interface, resulting from the drilling process;
- secondly, the lateral rigidity of the rock mass that controls the normal stress applied to the pile; this normal stress is generated by the prevented dilatancy resulting from the axial displacement of the pile.

It is increasingly admitted that these phenomena can be simulated using rock-grout interface tests carried out under CNS conditions (shear box test under constant normal stiffness) on samples whose interface simulates the state of the borehole wall after drilling. The imposed siffness should be representative of the actual rigidity of the rock mass. It should also take into account its real state of fracturing, as discussed in paragraph 6.4.5.

Guidance for characterising rock masses for designing drilled and grouted offshore pile foundations can be found in Puech and Quiterio-Mendoza (2019)

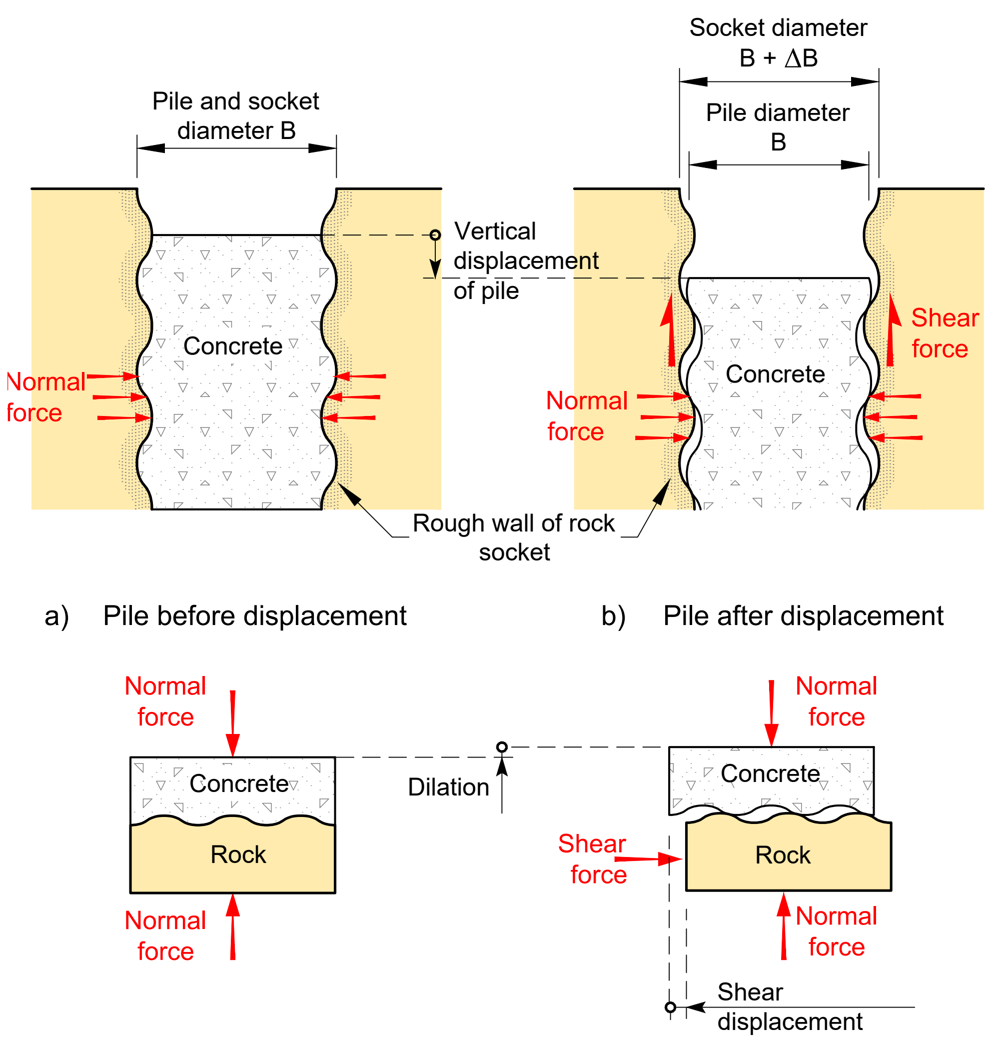


Figure 9.22: Idealised displacement of a pile socketed in rock

The CIRIA document (2004): « Piled foundations in weak rock » proposes a full review of these approaches and a framework for the design of drilled piles in weak to moderately strong rocks (characterised by values of unconfined compressive strength typically ranging from 1 MPa to 50 MPa).

Given the current knowledge, for drilled piles in chalks, marls and marly limestones as well as in weathered or fractured rocks, it is proposed that the values of end bearing stress applicable on the total sectional area be limited to:

- $0.30.m_c.q_c$ in the case where the assessment is based on cone penetrometer data;
- $1.45.m_p.p_l^*$ in the case where the assessment is based on pressuremeter data;
- $2.50.m_l.\sigma_c$ in the case where the assessment is based on laboratory data.

The factors m_c, m_p and m_l are lower than, or equal to, unity. They should reflect the loss of resistance of the rock mass due to its degree of weathering or fracturing. These factors decrease when the degree of weathering or fracturing increases and when the dimensions of the pile base are large with respect to the volume of ground mobilised for the type of in situ (m_c or m_p) or laboratory (m_l) tests carried out.

These values of end bearing stresses are maximum values, that should be weighted in function of the quality of grouting under the toe, and of the expected axial performance of the pile (see paragraph 9.4.2.2). They assume an embedment in the rock being at least 5 times the diameter of the pile.

Piles installed following onshore techniques

The French standard NF P 94-262 (2012) provides design parameters for bored piles (with or without drilling mud) in chalks, marls and marly limestones, weathered or fractured rocks of a limestone, schist or granite origin. These parameters are inferred from Ménard pressuremeter tests (PMT), or from cone penetrometer tests (CPT). They are expressed in function of the net pressuremeter limit pressures p_l^*, or from the penetrometer cone resistances q_c. One should note that the database refers to pressuremeter limit pressures values that do not exceed 5 MPa (exceptionally 8 MPa) and cone resistances values that do not exceed 40 MPa.

In this context, the maximum friction values do not exceed 90 kPa in usual soils (clays, silts, sands and gravels), 170 kPa in marls and marly limestones, and 200 kPa in chalks and weathered or fractured rocks. The values of end bearing resistance are given in function of the equivalent pressuremeter limit pressure p_{le}, or equivalent cone resistance q_{ce}, measured over a depth of around 1.5.B under the pile toe, and multiplied by a term called end bearing factor, k_p or k_c respectively. For rocky materials, these values should not exceed 1.45 for k_p and 0.3 for k_c. However, depending on the degree of weathering, on the degree of fracturing, on dipping or on the degree of overconsolidation (marls), the values of the bearing factors may be significantly lower. Furthermore, depending on the drilling tool being used (auger, bucket…), a proper contact at the bottom of the pile may be difficult to obtain and to maintain until the curing of the grout or concrete. In some cases, and for safety reasons, the contribution of the end bearing resistance may be neglected (notably, see: paragraph 9.4.2.2).

One should note that the currently available equipments for in situ measurements allow measuring limit pressures up to 20 MPa and cone resistances up to 100 MPa. Using this data during the design process should be done with great caution, given the limitations of the current database. A validation by pile testing is recommended on a case-by-case basis.

Note: The French standard is solely based on in-situ tests carried out with a pressuremeter or a cone penetrometer associated to a database comprising 174 static loading tests carried out over more than 40 years on 75 distinct sites, and covering a wide range of piles and soils types, including loadings in compression and in tension.

Approaches to assess shaft friction and end bearing resistance of drilled piles, other than the ones described in the NF P 94-262 (2012) standard, are worth considering, notably those proposed in paragraph 9.4.2.1 for piles installed following offshore techniques. However, they shall be sufficiently documented so they can guarantee a reliability comparable to the reliability associated to the definition of the partial resistance factors of table 9.7, paired with the comments of paragraph 9.4.3. This documentation should notably be based on representative piles tests (see: paragraph 9.4.2.3), associated to interface shear tests under static and cyclic loading devised to fully capture the phenomena involved.

9.4.2.2. AXIAL PERFORMANCE

As of today, there is no acknowledged offshore engineering method to build t-z transfer curves for drilled piles in rocks, whether for monotonous or cyclic loadings. Axial response assessments, if necessary, may be carried out from numerical simulations by finite elements. In the latter, a particular attention should be given to the state of the rock mass (weathering, fracturing...). When instrumented piles tests are available, appropriate t-z curves allowing to describe the local transfer of the axial loads may be developed as an alternative

The NF P 94-262 (2012) standard proposes formulations based on the pressuremeter modulus E_M, that allow building mobilisation laws of the axial friction and end bearing stress (the rock being assimilated to a hard soil). These laws, known as Frank and Zhao (1982) laws, are illustrated in Figure 9.23,

with:

- τ : axial friction mobilised along the shaft, in kPa
- q : end bearing stress mobilised under the toe, in kPa
- s : vertical displacement.

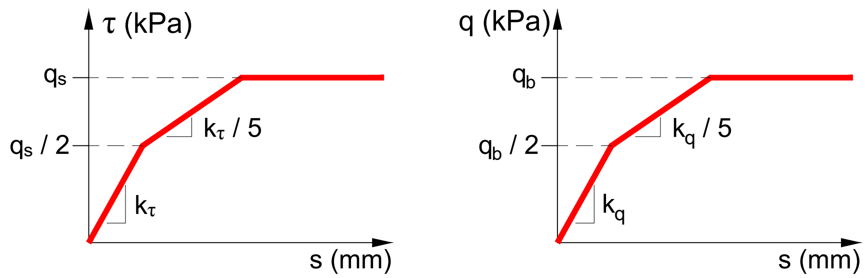


Figure 9.23: Assessment of the axial pile performance based on mobilisation laws of axial friction and end bearing stress according to NF P 94-262 (2012) standard

It is proposed:

- for granular soils and weathered chalks:

$$k_\tau = \frac{0.8\ E_M}{B} \qquad k_q = \frac{4.8\ E_M}{B}$$

- for fine soils, marly limestones and rocky chalks:

$$k_\tau = \frac{2.0\ E_M}{B} \qquad k_q = \frac{11.0\ E_M}{B}$$

This approach may be used during preliminary studies, but is only fully valid for piles with a diameter that does not exceed 1.20 m.

Effect of axial pile flexibility

For long drilled piles, the effect of axial flexibility should be considered, as stated in paragraph 9.3.1.2. Assessing the grout-pile residual friction, when applicable, may be done on the basis of instrumented piles tests and/or interface tests under constant stiffness (CNS).

Cyclic loading

Assessing cyclic degradation may be achieved using simulations by finite elements, following the SOLCYP procedure, to determine the interface laws. As an alternative, and when t-z curves have been developed, the degradation laws may be applied on the static t-z curves.

A particular attention should be paid to the role played by the end bearing capacity when it is significant. Instrumented piles tests in compression clearly show that cyclic loadings firstly affect axial friction, the latter being mobilised for displacements of an order of magnitude ranging from one millimetre to a few millimetres.

Admittedly, friction degradation leads to a transfer of loads on the toe, but this is detrimentally accompanied by significant displacements of the pile, with end bearing stresses being mobilised for displacements in an order of magnitude ranging from centimetres to one decimetre. This affects greatly the axial stiffness of the pile.

In the process of assessing the effect of cyclic loads, it is therefore recommended:

- either to, conservatively, take into account only the frictional capacity;
- or to limit the pile toe contribution to a fraction of the full end bearing capacity. This fraction should be compatible with the pile displacements that are acceptable in terms of stiffness. This fraction should be justified and should not exceed 20% of the ultimate end bearing capacity.

Experiments made throughout the SOLCYP project have shown that, in conventional soils, bored piles are generally more sensitive to cyclic loadings than driven piles. The high sensitivity of bored piles to cyclic loadings in carbonate sands and calcarenites is documented in the SOLCYP (§ 6.4.2) recommendations. Even though the available data on the behaviour of bored piles under cyclic loading in chalk is scarce, there are indications, on the basis of cyclic shear tests on these materials, that the cyclic degradation of the interface may be severe. A great attention should be given to shaft friction cyclic degradation for bored piles in chalk. Representative piles tests (see: paragraph 9.4.2.3) under static and cyclic loadings, associated to interfaces shear tests, will be required to validate the design process.

9.4.2.3. PILES TESTS

Generally speaking, the prior execution of piles tests is highly recommended. To be considered as being representative, these tests should be carried out:

- on the site itself, or on another site having similar geological and geotechnical conditions;
- on piles having sufficiently large dimensions, so that a scaling distortion does not occur compared to the real piles;
- with an installation technique (drilling fluid, drilling tools...) that is preferably similar to the one selected for the real piles;
- with conditions of static and/or cyclic loadings that are adapted to the functioning mode of the pile (tension/compression, static/cyclic…). In any case, the loading test programme shall include a static loading carried out until failure.

For further precisions, one may peruse the annex F of the NF P 94-262 (2012) standard, and the NF P 94-150-1 (1999) and NF P 94-150-2 (1999) standards for piles tests under static loadings, as well as the chapter 11 of SOLCYP (2017) for piles tests under cyclic loadings.

9.4.3. VERIFICATIONS UNDER AXIAL LOADING

Let us note:

S_d: the design load

R_{ks1}: the characteristic static resistance obtained from the calculation of axial capacity

R_{ks2}: the characteristic maximum resistance obtained from the displacement calculation ($R_{ks2} \leq R_{ks1}$)

R_{kc}: the cyclic resistance for the design event. The cyclic resistance is obtained from the static resistance by taking into account the degradation due to cycles ($R_{kc} \leq R_{ks1}$)

It shall be verified that:

Static condition: $S_d < R_{ds2}$

Cyclic condition: $S_d < R_{dc}$

with: $R_{ds2} = R_{ks2} / \gamma_R$ $R_{dc} = R_{kc} / \gamma_R$

The partial resistance factor γ_R is expressed by:

$$\gamma_R = \gamma_{R0} \cdot \gamma_{R1} \cdot \gamma_{R2}$$

with:

γ_{R0} : partial resistance factor as defined and recommended in DNVGL-ST-0126 (2016) for the considered conditions;

γ_{R1} : partial model factor, function of the type of material;

γ_{R2} : partial model factor, function of the loading direction.

The partial resistance factor γ_R shall not be less than γ_{R0}.

The calculation of cyclic degradation for the design event is carried out with unfactored load ($\gamma_F = 1.00$) and without factoring the soil properties ($\gamma_M = 1.00$).

9.4.3.1. ULS CONDITIONS

The partial resistance factor γ_R shall be at least equal to the value proposed in DNVGL-ST-0126 (2016):

$$\gamma_{R0} = 1.25.$$

The French experience of land foundations on bored piles (NF P 94-262, 2012), backed by reliability analyses achieved using the database mentioned in paragraph 9.4.2.1, show that this factor may prove insufficient to cover all the inherent uncertainties of this type of foundation. In order to guarantee a level of reliability for offshore bored piles that is similar to the one of onshore bored piles, it is recommended to introduce two additional partial model factors: γ_{R1} and γ_{R2} . The values proposed for these two factors guarantee the compatibility, in terms of reliability, between the offshore and onshore approaches.

In siliceous sands, in clays and in non-carbonate rocks, it is recommended to use: $\gamma_{R1} = 1.10$.

In chalks, given the uncertainty associated to the models presently available to assess the capacity in these materials, it is recommended to adopt: $\gamma_{R1} = 1.30$. This factor may be lowered if representative and well-documented piles tests (as per paragraph 9.4.2.3) allow an in depth understanding of the phenomena involved and improve significantly the degree of confidence in the assessment of capacity. However, the partial factor γ_{R1} shall never be smaller than 1.10.

The partial factor γ_{R2} takes into account the direction of loading. For bored piles, $\gamma_{R2} = 1.00$ when the pile is loaded in compression and $\gamma_{R2} = 1.25$ when the pile is loaded in tension. If representative piles tests carried out in tension (as per paragraph 9.4.2.3) are available, the value of the coefficient γ_{R2} may be brought back to 1.00.

Partial factors to be applied in the case of bored piles are compiled in Table 9.7.

Table 9.7: Values of partial factors of resistance in the ULS for bored piles

BORED PILES				
Type of soil	γ_{R0}	γ_{R1}	Compression γ_{R2}	Tension γ_{R2}
Siliceous sands	1.25	1.10	1.00	1.25*
Clays	1.25	1.10	1.00	1.25*
Carbonate sands and calcarenites	1.25	1.30*	1.00	1.25*
Chalks	1.25	1.30*	1.00	1.25*
Other rocks	1.25	1.10	1.00	1.25*

*these factors can be lowered under particular justifications (see text)

9.4.3.2. SLS CONDITIONS

The analyses against the SLS (displacements, stiffnesses) and against the FLS (fatigue) shall be carried out with unfactored soil properties:

$$\gamma_M = 1.00$$

As a reminder, verifications should deal with the constitutive materials of the pile (depending on the case: steel, grout, reinforced concrete). In the case of piles sealed by grout, the potential degradation of grout under cyclic loadings should be considered.

Piles made with reinforced concrete shall be reinforced over their entire height. A particular attention should be notably given to the acceptable compressive stress in concrete and to the cracking conditions under tension loads. These aspects, developed in the NF P 94-262 (2012) standard, are an integral part of the verification of bored piles made of reinforced concrete, and may lead to a sharp reduction of the acceptable stress in concrete and in steel.

9.4.4. BEHAVIOUR UNDER LATERAL LOADING

9.4.4.1. OVERVIEW

The overview of paragraph 9.3.3.1 relative to driven piles is transposable to long (flexible) bored piles.

A critical difference concerns short piles (« socketed piles »), which behave like rigid bodies because of their high flexural rigidity. These piles are likely to lead to a ground failure in rotation, and should therefore be verified against the ULS under the effect of lateral loads and of moments applied at the pile head.

9.4.4.2. P-Y CURVES

Response under static loading

The considerations of paragraph 9.3.3.2 relative to driven piles in sands and stiff clays (normally consolidated or overconsolidated) are pertinent for bored piles.

For cemented carbonate sands (notably calcarenites) that remain below 5 MPa of unconfined compressive strength, the static p-y curves proposed by Abbs and Needham (1985) may be used. They combine a first response of the material before failure based on Reese's model (1975) for stiff clays with a residual response of the material after failure based on Murchison and O'Neill's model (1984) for sands (Figure 9.24).

Reese (1997) has proposed a method to build p-y curves in rocks, considering that at very small strains the p-y relation is determined by the elastic properties of the intact material, but that very quickly rock cracking occurs at the surface (Figure 9.25). A reduction factor of resistance is then introduced. The method attempts to reflect the influence of the rock secondary structure (joints, fractures, inclusions...) on the modulus.

Erbrich (2004) has developed an original model that takes into account the breaking of the rock into successive chips on the upper part of the pile, and its migration in depth (Figure 9.26).

Response under cyclic loading

A few attempts made to develop lateral transfer curves under cyclic loadings can be mentioned: Abbs (1983) for soft carbonate rocks; Fragio et al. (1995) for hard marls; Novello (1989) for calcarenites; Erbrich (2004) for calcarenites.

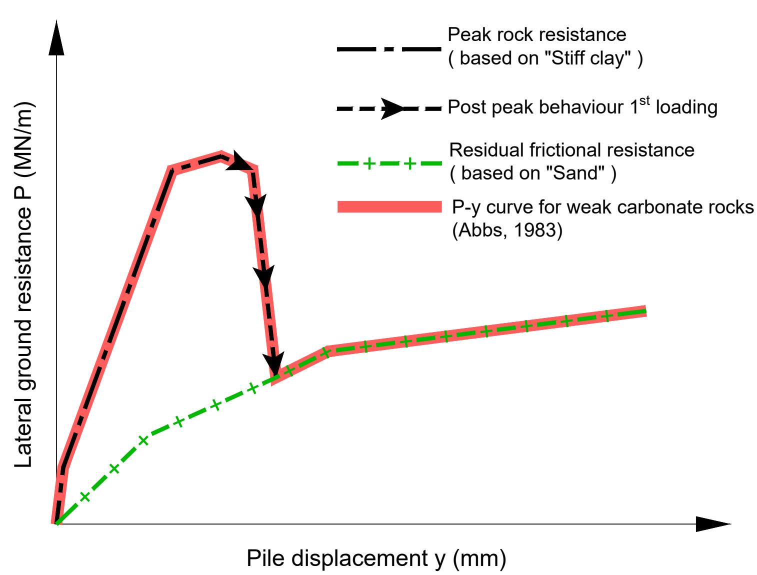


Figure 9.24: Hybrid P-y curve for a brittle carbonate rock (from Abbs and Needham, 1985)

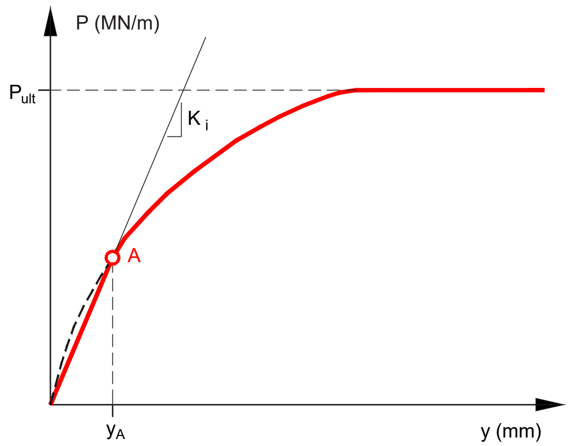


Figure 9.25: P-y curve for a non-carbonate rock
(from Reese, 1997)

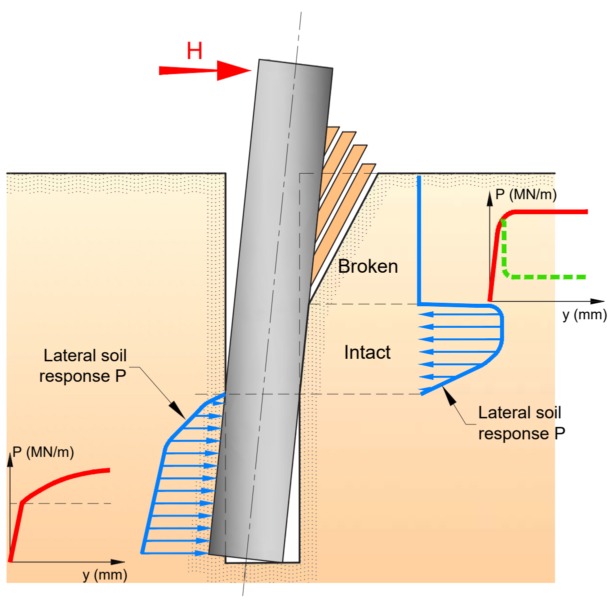


Figure 9.26: Model of P-y curve for a rock breaking into chips
(from Erbrich, 2004)

9.4.4.3. CAPACITY IN ROTATION OF SHORT (RIGID) PILES

The capacity in rotation of short piles exhibiting a high pile-soil/rock relative rigidity may be assessed:

- by building an H-M envelope curve. The degrees of weathering and fracturing of the rock should be taken into account, as well as the possibility of failure of the rock wedges near the surface;
- by applying a simplified method, which considers the pile as being a rigid body pivoting around a rotation point. Frontal and lateral reactions may be approached by building p-y curves, or by considering elementary mechanisms (frontal passive pressure, wedges sliding…). In any case, the mechanism of progressive failure of rock wedges should be taken into consideration (Figure 9.27).

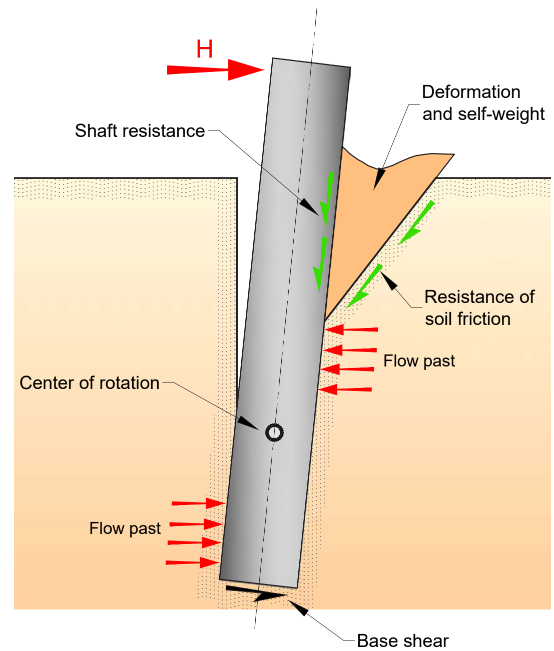


Figure 9.27: Short pile in rock – Failure of a rock wedge

9.4.5. VERIFICATIONS UNDER LATERAL LOADING

For the SLS (displacements, stiffnesses) and FLS (fatigue) analyses, the partial material (soil) or resistance factors (depending on the method used to build the p-y curves) shall be equal to unity:

$$\gamma_M = \gamma_R = 1.00.$$

The calculations of lateral displacements should take into account realistic conditions of liaison between the pile and the superstructure: most often, it is considered that the platform legs or the sleeves impose fixed head conditions to the piles.

Strictly speaking, there is no regulatory criterion to be satisfied on what constitutes an acceptable displacement of the pile in soils (see 9.3.4). However, in rocks, very stringent criteria may be required to protect against rock fracturing or formation of wedges of chipped material near the surface.

Determining the natural frequencies of the structure may lead to specific contingencies regarding the stiffness of foundations.

For the verifications of the integrity of the pile structure, piles deformations shall be calculated by applying materials or resistance factors (depending on the method used to build p-y curves) equal to 1.00 ($\gamma_M = \gamma_R = 1.00$). Bending moments in the pile and stresses in the constitutive materials of the pile are inferred according to the rules of strength of materials.

In the case of piles composed of tubes sealed with a grout, it shall be verified that the combined flexural and axial compressive/tensile stresses in the steel meet the criteria of section 13 of ISO 19902 (2011).

Piles made of reinforced concrete shall be reinforced over their entire length. A particular attention should be notably given to the allowable compressive stress in concrete, and to cracking conditions under the effect of bending moments and cyclic loadings. These aspects, developed in the NF P 94-262 (2012) standard, are an integral part of the verification of bored piles made of reinforced concrete, and may lead to severely limit the allowable stresses in concrete and steel.

For short piles, the verification of the capacity in rotation shall be carried out in ULS conditions, by applying partial load and material factors recommended by DNVGL-ST-0126 (2016) for the case of monopiles, with:

- partial material factor - calculations in total stress:

$$\gamma_M = 1.25$$

- partial material factor - calculations in effective stress:

$$\gamma_M = 1.15$$

9.5. GROUPS OF PILES

When a foundation is composed of several adjacent piles, the response of each pile may be affected by the loadings applied by the other piles. This interaction is called group effect.

It is usually admitted that the group effect is negligible when the centre-to-centre spacing between the surrounding piles is greater than 8.B (B: piles diameter).

The main expression of the group effect is a modification of stiffness, i.e. the stiffness of a pile from the group that bears a load Q is lower than the stiffness of the same pile being isolated and bearing the same load Q. This phenomenon concerns axial loadings as well as lateral loadings.

The group effect (increase of displacements and drop of stiffnesses) may be assessed from Mindlin's equations, with the ground being modelled as an elastic half-space. One may notably refer to Poulos, 1980 (DEFPIG software) or Randolph, 1987 (PIGLET software).

In the case of piles being very close to each other, it should be verified that the capacity of the group of N piles, considered as a single foundation (hypothesis of the Terzaghi's block), is not lower than the capacity of N piles isolated from each other. This case is uncommon in offshore works. It should be noted that for driven piles in cohesionless soils, the capacity of the group is always higher than the capacity of N piles isolated from each other, because of the densification of sands during driving.

9.6. DYNAMIC ANALYSIS

The dynamic analysis of the support structure is essential to:

- determine the natural frequencies of the structure;
- assess its sensitivity to fatigue;
- ensure that the turbine displacements remain compatible with the criteria set by the manufacturer.

The response of the foundation itself is a significant component of the global response of the structure.

The dynamic soil-structure interaction is characterised by:

- foundation stiffnesses: axial stiffness (1 component: K_V); lateral stiffnesses (2 components: K_{Hx}, K_{Hy}); stiffnesses in rotation (2 components: K_{Mx}, K_{My}); stiffness in torsion (1 component: K_T); terms of coupling;
- the associated dampings β (damping due to the soil called hysteretic damping and geometric or radiative damping).

In the case of a decoupled study, it is the duty of the geotechnical engineer to determine these parameters. They should be compatible with the strain levels resulting from the type of analysis under consideration (intensity of loads). Iterations will usually be required.

It is increasingly frequent that the dynamic analysis is carried out from an integrated structural model, in which the soil-structure interactions are directly modelled from the t-z, Q-z and p-y transfer curves. Linearisations on the forecasted strain ranges may be required. It is the duty of the geotechnical engineer to ensure that the parameters injected when building the transfer curves, and their potential linearisation, be consistent enough to produce results that are compatible with the strain levels resulting from the type of analysis under consideration (intensity of loads). Iterations will usually be required.

9.7. INSTALLATION

9.7.1. STABILITY FLOORS

During the first stage of installation, jacket type structures are usually stabilised temporarily (before driving the piles) using stability floors or "mudmats". Most often, those are shallow foundations, built at the platform angles, which aim at taking over forces due to the self weight of the jacket and to the environmental forces considered at time of installation.

The design of shallow foundations, such as mudmats, may be processed as stated in the Annex G of DNVGL-ST-0126 (2016).

It is emphasised that the design of these foundations is submitted to high geometric constraints. As a result, mudmats are subject to not only inclined forces, but also to highly eccentric forces. These eccentricities should be taken into consideration from the preliminary stage because they greatly reduce the bearing capacity and condition the sizing.

9.7.2. DRIVING PREDICTIONS

More or less accurate driving predictions should be carried out as the project progresses. These forecasts are essential to:

- ensure that the pile can be installed by driving at the required target depth to guarantee its nominal capacity;
- verify that the installation will not impair the integrity of the metallic tube (allowable compressive and tensile stresses in steel, steel fatigue under the cumulated number of hammer blows);
- dimension the hammer(s) required for the installation;

- plan adequate remedial procedures for the event where a driving-only installation cannot be guaranteed: plug removal, execution of a pre-drilled hole and a grouted insert pile, moving from the driven pile solution to a drilled and grouted solution.

A driving prediction is composed of two main steps:

- determining the static component of the soil resistance to driving;
- simulating the hammer-pile-soil complex.

The procedures used to assess the response of piles to driving are not detailed by standards. They essentially belong to the know-how field and are mastered by a small number of experts. Even though they are now backed up by a theoretical formalisation, a significant part of these procedures remains semi-empirical. The experience acquired in a specific geographical area, or from a given type of soil, can improve the reliability of a forecast.

9.7.2.1. OVERVIEW

When a pile is subject to hammer blows on its head, the resistance to penetration comprises a static component and a dynamic component.

The static component (i.e., non-affected by the loading rate) is conventionally called SRD (Soil Resistance to Driving). The SRD may, notably in clays, greatly differ from the ultimate capacity.

In sensitive clays, the SRD will greatly differ from the ultimate capacity and be essentially governed by the remoulded properties of the material. In strongly overconsolidated clays, the SRD may be higher than the capacity, due to the generation of negative pore pressures. When the driving stops, resistances will evolve over time towards the capacity reached on the longer term. The phenomena involved during this stage, called « set-up », are the recovery of soil resistance through thixotropy and the dissipation of pore pressures associated with radial soil re-consolidation around the pile.

In sands, the SRD is relatively close to the ultimate capacity (excluding the consideration of ageing phenomena). A critical phenomenon related to driving is the « friction fatigue », which means that at a given level in the ground, the local shaft friction decreases in function of the pile penetration (or, in other words, of the number of hammer blows). This phenomenon should be taken into consideration in the SRD assessment as well as in the capacity assessment.

The impact of the hammer on the head of the pile generates a compression wave that propagates along the metallic tube at a velocity close to 5,400 m/s. The ground at the soil-pile interface is therefore dynamically loaded.

The local dynamic resistance is expressed simply, by using Smith's equation (1962):

$$R_{dy} = R_{st} \cdot j \cdot \dot{v}$$

with:

R_{dy} = dynamic resistance developed on the considered element

R_{st} = static resistance of the soil on the interface (obtained from the SRD calculation)

j = Smith's dynamic amplification factor, or damping factor (in s/m). Values of j for shaft friction, written j_s, and for the toe, written j_p, are distinguished.

$\dot{v}$ = particle velocity of the element.

The various components of the dynamic resistance are illustrated in Figure 9.28.

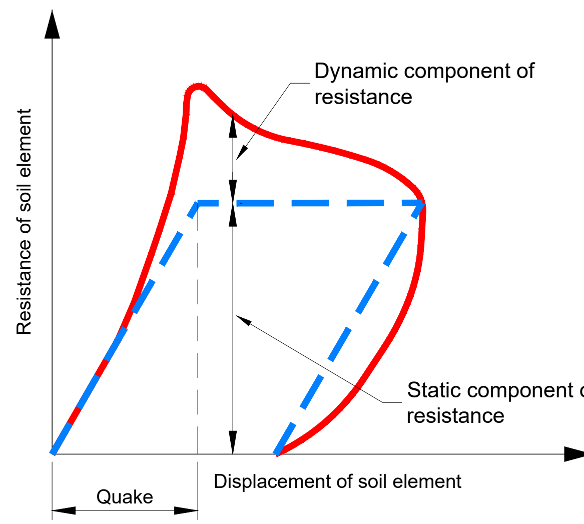


Figure 9.28: Dynamic response at the soil-pile interface during driving (toe resistance and shaft friction)

9.7.2.2. STATIC COMPONENT OF THE SOIL RESISTANCE TO DRIVING

The SRD represents the static component of the soil resistance to driving. The calculation principles of the SRD are close to the principles of determination of the ultimate capacity. The fundamental difference lies in the choice of the resistance parameters of soils, which must be adapted to the hypotheses: degree of remoulding of the clay, set up duration, sands fatigue...

Amongst the most commonly used methods, there are:

- Stevens et al. (1982), which preferably applies to siliceous sands, to clays with low sensitivity, to sands and gravels or to soft rocks destroyed by driving;
- Puech et al. (1990), which deals more specifically with sensitive clays, with stiff clays and with the crossing of rock strata of relatively low thicknesses;
- Colliat et al. (1996), which extends the Puech et al. (1990) method to the case of silts of low plasticity;
- Alm and Hamre (2001), which proposes to take into account friction fatigue in sands based on CPT data.

A forecast of the driving resistance should, at least, provide the foreseeable maximum and minimum SRD values in continuous driving (without interruption) and compare those values to the static capacity of the pile. The static capacity is usually assimilated to the SRD value at full set up. Depending on the types of soils, the SRD can be:

- lower than the static capacity (case of normally consolidated clays), or even much lower if the clays are sensitive or structured;
- of the same order than the static capacity (case of sands);
- higher than the static capacity (case of heavily overconsolidated clays).

A diagram of principle is provided in Figure 9.29.

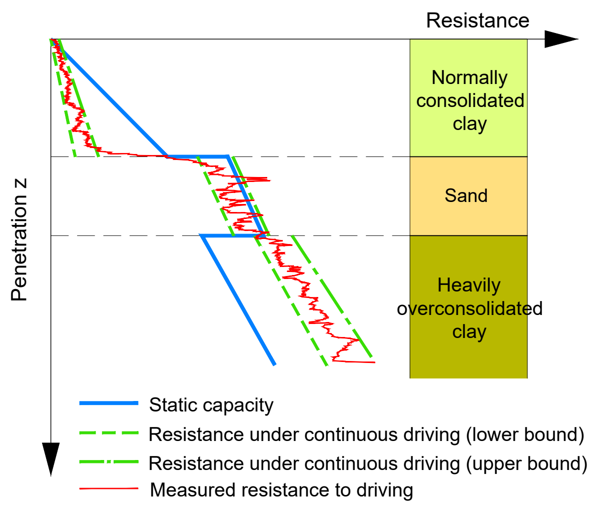


Figure 9.29: Components of the SRD during continuous driving

The SRD in continuous driving should be distinguished from the SRD at driving resumption, i.e. after an interruption of a few minutes or a few hours (e.g., to change hammer, to weld a pile section...).

The SRD at driving resumption can be much higher than the SRD in continuous driving due to the effect of partial set up, as shown in Figure 9.30.

The reliability of a driving prediction may be considerably improved by carrying out instrumented driving tests (piles equipped with stress gauges and accelerometers at the head) on the site itself or on another site with similar ground conditions. The interpretation of results allows calibrating the parameters of the prediction method.

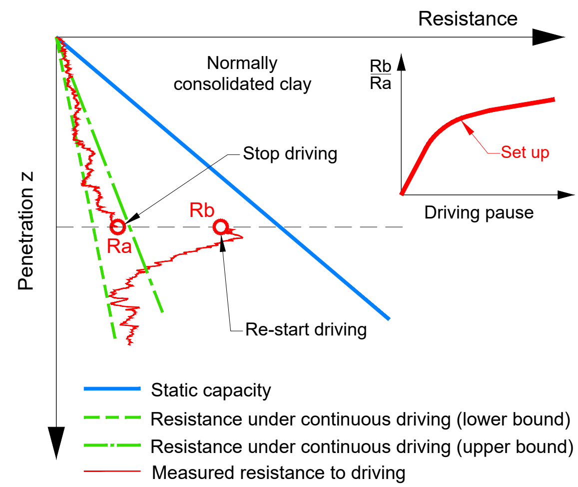


Figure 9.30: Components of the SRD after re-start of driving

9.7.2.3. DRIVING SIMULATION

The impact of the hammer ram generates a compression wave at the head of the pile. The enthru signal (ram signature) depends on the hammer characteristics and on the pile impedance. This wave then propagates in the pile steel at a velocity close to 5,400 m/s without deformation if no obstacles are met. Any variation of impedance will lead to an alteration of the signal. For instance, a change in steel section will generate reflected waves. On its entry into the ground, the wave will induce a ground resistance by friction. When the signal energy is higher than the generated dynamic resistance, an irreversible (plastic) local displacement occurs. The same applies to the pile toe. However, if the wave carries insufficient energy to displace the pile toe when it reaches it, no irreversible sinking of the whole pile can occur. Only a reversible displacement is possible; the limit value of the reversible displacement is called « quake ».

Driving simulations aim at ensuring that the energy transferred to the pile by the ram is capable of inducing a global pile movement sufficiently large to achieve final penetration under an acceptable number of blows. Besides, stresses in the pile wall should be limited to avoid any damage to the steel.

Simulations are carried out using dedicated software based on the integration of the wave equation. One of the most commonly used software is GRL WEAP (Goble Rausche Likins and Associates, 2015). Its principle is relatively simple: it consists in discretising the hammer and the pile into a series of linear elements characterised by their mass, their elasticity modulus, their coefficient of energy restitution and their internal damping.

The input data is:

- the type of hammer and its characteristics: software currently has a database that facilitates its modelling;
- the discretisation of the pile into elements that are sufficiently small to best render the variations of section and of ground profiles;

- the dynamic resistance to apply on these elements at the soil-pile interface, which is characterised by the static component of driving resistance (SRD), Smith's dynamic amplification coefficient (j) and the values of the reversible limit displacement at pile shaft and at pile toe (quakes Q_S and Q_p). The values relative to the dynamic response of the soil-pile system (quake and damping) are relatively poorly known and hardly measurable. It is recommended to ensure that the damping values chosen to achieve prediction are compatible with the ones used to calibrate the selected SRD calculation method. By default, one may use the parameters proposed by Roussel (1979), which are documented in Table 9.8.

Table 9.8: Values of the dynamic response parameters of the soil-pile system during driving

Parameters	Unit	Clay					Sand/silt	Rock
		Soft	Firm	Stiff	Very stiff	Hard		
Shaft displacement Q_S	mm	5.00	3.80	2.50	2.50	2.50	2.50	2.50
Toe displacement Q_p	mm	5.00	3.80	2.50	2.50	2.50	2.50	2.50
Shaft damping j_S	s/m	0.35	0.25	0.20	0.15	0.10	0.25	0.25
Toe damping j_P	s/m	0.65	0.50	0.50	0.50	0.50	0.50	0.50

Originally, rock was not mentioned in Roussel's article (1979). According to Stevens et al (1982), the driving into rocks would lead to a fracturing of rocky layers, which, in certain conditions of resistance and compressibility, would reduce them into granular materials.

The model of wave propagation simulates for each blow the impact generated by the hammer by taking into account the global efficiency of the system, and allows calculating the following data:

- the curve of soil resistance in function of the number of hammer blows: this curve immediately gives to the geotechnical engineer the maximum SRD value that a given hammer can overcome in normal operating conditions;
- the energy transferred at the head of the pile (enthru energy) by taking into account all of the system efficiencies;
- the maximum compressive and tensile stresses in the pile.

The results of the driving simulations can be integrated in the pile stress history for the verification of steel fatigue.

9.7.3. DRILLING

Recommendations for bored piles construction are provided in the NF EN 1536 (2015) standard.

A few essential points are reminded below:

- during the execution of drilled piles, all measures should be taken to prevent soil/rock instabilities in the borehole. The risks are high in poorly consistent cohesive soils, in granular soils, in heterogeneous soils and in fractured rocks.
- when the risks of borehole wall instabilities are high, the use of a drilling mud with an appropriate density should be considered to ensure the walls of the drilling hole are stabilised.
- once the borehole is achieved, it should remain open only for the duration of the cleaning and the installation of the metallic tube or of the reinforcing cages.
- debris deposited at the bottom of the borehole during drilling should be carefully cleaned before grouting/concreting. Their presence is very likely to alter the performance of the bored pile in relation to end bearing capacity.
- the drilling system and the type of tool should be adapted to the soil or rock conditions, so that the excavation is fast and roughness conditions of the wall are compatible with the assumptions made during the design process (choice of the limit friction values between the grout or the concrete on one hand, and the soil or the rock on the other hand).
- in the case where bentonite mud is used for drilling, particular precautions should be planned for with respect to sea water. Specific care should also be given to recycling of mud before concreting.

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10 GRAVITY BASE FOUNDATIONS

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10.3 ULS VERIFICATIONS

10.4 SLS VERIFICATIONS

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10. GRAVITY BASE FOUNDATIONS

10.1. OVERVIEW

10.1.1. DEFINITION OF A GRAVITY BASE FOUNDATION

Gravity base foundations are foundations that maintain their stability essentially through their self-weight and the weight of the supported elements. Their embedment into the ground is small or even inexistent.

The slab, base of the foundation, has usually a circular, full or annular shape. It may also have an octagonal, square or rectangular shape. Typically, for offshore wind turbines, the diameter (or width) of the slab is in an order of magnitude of 20 m to 35 m.

The gravity base foundation may be equipped with relatively short skirts (< 3 m) that penetrate the ground. However, in most instances, skirts will not contribute significantly to the resistance of the foundation to sliding (mobilisation of the passive resistance). Placed on the periphery, these skirts may offer a protection against scour or prevent the erosion effect due to pumping under the slab associated to the movements of the foundation. Skirts grids adequately distributed under the foundation (internal skirts) may allow transferring forces in a ground layer that is more favourable, in the case of soils that are poorly competent in surface. Their spacing and structural capacity shall then be dimensioned accordingly.

Designing the gravity base foundation is a process that should take into account the morphological features of the seabed, and in particular its slope and potential unevenness. In order to ensure a full contact on the entire surface of the foundation, it is appropriate to consider if it is needed or not to create a flat artificial platform by bringing materials, or to proceed to underbase grouting. In some cases, the removal and/or substitution of surface sediments that are heterogeneous or that have poor properties can be taken into consideration.

10.1.2. DESIGN CRITERIA

The criteria to be verified for the geotechnical design of foundations are classified into three main categories: the Ultimate Limit States or ULS, the Service Limit States or SLS and the Fatigue Limit States or FLS. For the case of the gravity base foundations, the verifications to be done for each of the Limit States are listed below.

ULS are mostly concerned with the elements leading to the geotechnical failure of the foundation, or to its loss of stability. The design criteria that shall be analysed against the ULS include at the very least: the resistance to sliding of the foundation, its bearing capacity (under the combined effect of horizontal and vertical loads, as well as of overturning moments), its stability in regard to overturning as well as its hydraulic stability, under the cases of extreme loads. The methodology used to verify the ULS criteria is detailed in paragraph 10.3.

SLS are concerned with temporary or permanent deformations relative to the operating criteria of the wind turbine. The design criteria that shall be analysed against the SLS include at the very least: the average total settlement, differential settlements and permanent rotations. The acceptable rotation criterion at the turbine level, set by the manufacturer, is extremely stringent (typically: 0.5°, with 0.25° reserved to installation tolerances), and proves to be one of the most constraining factors for the design. The verification methodology of the SLS criteria is detailed in paragraph 10.4. An additional design criterion is applied on the minimum ratio of contact area. This criterion is addressed in paragraph 10.6.

The Limit States of Fatigue, FLS, are criteria that do not prove as being dimensioning for the geotechnical analysis of the foundations. However, the geotechnical analysis shall provide the input data required for the study of the structural elements against the FLS, and allow modelling the dynamic behaviour of the foundation, as developed in paragraph 10.6.

The effect of cyclic loads shall be taken into account:

- in the ULS: the accumulation of pore pressures and/or deformations under the foundation may lead to a decrease of the soil resistance;
- in the SLS: cycles may lead to an accumulation of permanent deformations and to a modification of deformation moduli;
- in the FLS: cycles affect the value of deformation moduli.

It is also necessary to verify some criteria that are specific to the installation (paragraph 10.8), and to guarantee the stability in regard to erosion phenomena (paragraph 10.11).

10.1.3. LOAD CASES TO BE ANALYSED FOR THE GEOTECHNICAL DESIGN

Load cases (or Design Load Cases, DLC) describe the whole sets of configurations in which the wind turbine is likely to operate during its lifespan. Some DLCs apply to verifications against the ULS, while other apply to verifications against the SLS or FLS. These aspects are addressed in chapter 7. The whole set of situations that take into account the different required combinations represent several thousand load cases. It is therefore appropriate to select amongst all these load cases the ones that pertain to the geotechnical analysis of the foundation. This selection will be made jointly between the engineering company that produces the set of load cases and the geotechnical engineer.

Generally, the following load cases will be critical for the geotechnical calculation of gravity bases:

- in ULS: extreme loads with their concomitant forces, and the cases of maximum eccentricity. The cyclic amplitude corresponding to these cases will also be essential;
- in SLS: the extreme loads with a high cyclic amplitude and a small number of cycles, as well as lower loads applied with a larger number of cycles, and possibly the succession of these conditions and the repetition of extreme events.

The cyclic content, i.e. the loading history derived from a modelling of the storm, or of any relevant cyclic event, shall be analysed for the critical cases.

The values of loads depend on the stiffness and damping of the foundation under these loads. Several iterations are usually required between the geotechnical engineering and the structural engineering to converge towards the final loads values that are applicable to the ULS and SLS verifications.

10.1.4. NOTION OF EFFECTIVE AREA

It is usual, notably during the preliminary stage, to base the geotechnical analyses of gravity base foundations on idealised cases of geometry and load, so that simple analytical methods of calculation can be used, in particular for stability calculations.

Thus, a rectangular foundation, subject to a vertical force and to an overturning moment, can be represented by an equivalent rectangular foundation with smaller dimensions, eccentric in regard to the real foundation, and only subject to the vertical force distributed uniformly, while maintaining the compatibility to the isostatic equilibrium of applied external forces. For a real foundation of width B and length L, an effective reduced cross-sectional area is defined with a width B' and a length L', according to the Meyerhof's method, as illustrated in Figure 10.1 below.

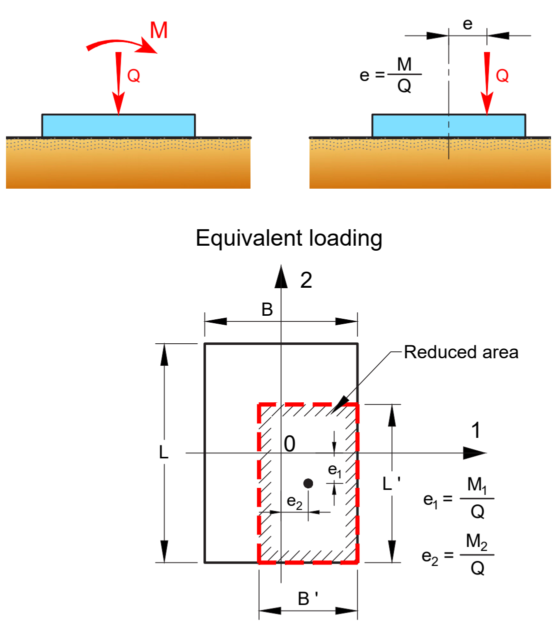

Figure 10.1: Effective cross-sectional area of a rectangular foundation

A circular foundation subject to an overturning moment may be represented by an effective rectangular cross-sectional area determined under the lune model described in RFG N° 138-139 (2012) and DNVGL-ST-0126 (2016) recommendations, as illustrated in Figure 10.2 below.

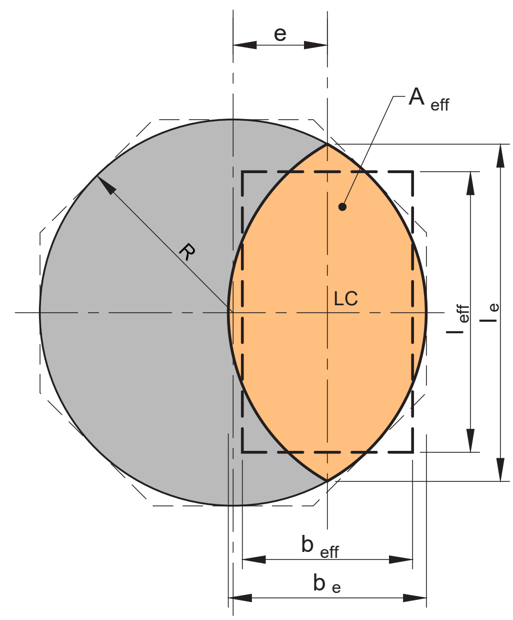

Figure 10.2: Effective cross-sectional area of a circular foundation

For the case of a circular full foundation with a diameter B (B = 2.R), the effective area A_{eff} thus determined, and normalised by the total area ($A_{tot} = \pi.B^2/4$), is represented in function of the normalised eccentricity e/B in Figure 10.3.

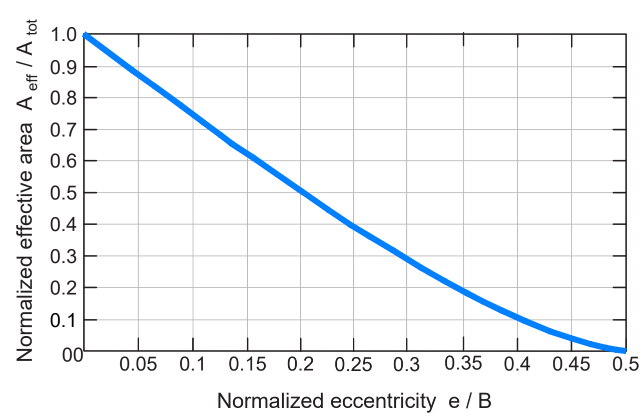

Figure 10.3: Relation between effective area and eccentricity for a circular full foundation

These principles of determination of the effective cross-sectional area of a circular full foundation can be extended to the case of an annular foundation, as outlined in Appendix B.

However, it should be reminded that a foundation analysis made using the effective area method remains an acceptable approximation in the preliminary study stage. During the detailed study stage, the capacity of the foundation under combined loadings shall be checked using more thorough methods.

10.1.5. IDEALISATION OF THE FOUNDATION GEOMETRY

In the case of a foundation with a complex geometric shape, i.e. non-rectangular or non-circular, it may be required to select an idealised shape for the geotechnical analysis of the foundation, so that simple analytical methods of calculation can be applied, notably during the preliminary stage.

It is then adequate to verify that the area and the moments of inertia of the foundation are identical, or close enough, to those of the real foundation. This verification may be carried out using the approach detailed in ISO 19901-4 (2016), Annex A.7.2.4.1.

However, during the project study stage, the idealised shape shall be validated using more thorough methods.

10.2. PARTIAL MATERIAL FACTORS

The partial material factors applicable to the design of gravity bases are detailed in paragraph 7.3.3.

10.3. ULS VERIFICATIONS

The criteria to be verified against the ULS include: the resistance of the foundation to sliding, its bearing capacity (under the combined effect of vertical and horizontal forces, and of overturning moments), its stability to overturning as well as its hydraulic stability.

The recommendations presented in paragraph 7.4.4.4 of DNVGL-ST-0126 (2016) may be considered in order to select the loading conditions for which the verifications against the ULS shall be carried out. More particularly, a special interest should be given to the cases of:

- a single design storm (as contractually defined);
- an emergency shutdown or a storm preceded by normal operating conditions;
- any other scenario that covers the most critical ULS action on the soil surrounding the substructure.

Verifications are highly dependent on the drainage conditions of the soil, and on its behaviour under cyclic loading.

10.3.1. DRAINAGE CONDITIONS OF THE SOIL

In order to evaluate if the response of the ground under a given loading will occur in drained, undrained or partially drained conditions, and therefore to justify the application of the calculation methods and the adapted soil parameters, it is adequate to consider the failure mechanism involved, the associated drainage path, the consolidation properties of the soil (permeability and compressibility), as well as the rate and duration of the loading.

Given the size of the considered foundations, the bearing capacity mechanism mobilises a large volume of ground. Consequently, the bearing capacity analyses should be carried out in undrained conditions by default. This will notably be the case during preliminary studies. To justify the analyses of bearing capacity in drained or partially drained conditions, specific studies of pore pressures dissipation in the mobilised soil mass are required, taking into account the permeabilities, the boundary conditions, and the loading rates. This type of analysis can only be considered during the detailed study stage.

In dense sands, high undrained shear strengths can be observed on samples tested in a laboratory, resulting from the development of high negative pore pressures. These dilatancy effects should be considered with caution when analysing the foundation capacity, because of the risk of dissipation of these pore pressures, as stated in DNVGL-RP-C212 (2017), § 5.2.2.4.

The resistance to sliding of the gravity base foundation is governed by the nature of the interface. For the vast majority of materials (clays, sands) it is pertinent to carry out the analyses in undrained conditions. In the case of a foundation laid on a coarse material (typically, an existing top gravel layer, an engineered top layer or an artificial platform) the works of Pederstad et al. (2015) show that the hypothesis of full draining under cyclic loadings due to the effect of waves (i.e. for a loading duration of about 10 seconds) is not necessarily verified. The effective stress used to check the non-sliding criterion should take into account pore pressures that are potentially generated in the base course material under the effect of cyclic loading, as stated in DNVGL-RP-C212 (2017), § 5.2.3. In the case where the design of the base course can guarantee that draining properties are sufficient to justify the absence of generation of pore pressure, a calculation in drained conditions may be carried out.

The selection of soil parameters adapted to the various verifications mentioned in this paragraph is addressed in chapter 6, paragraph 6.2.5 (Table 6.3).

10.3.2. ULS STABILITY: SLIDING

The resistance of the foundation to sliding is traditionally calculated using the conventional methods described in DNVGL-ST-0126 (2016) and ISO 19901-4 (2016).

In a simplified approach, the calculation of the resistance to sliding under the method set out in DNVGL-ST-0126– Annex G6 (2016) is commonly achieved by considering the mobilisation of the soil ultimate resistance on the effective area of the foundation (as outlined in paragraph 10.1.4). However, in the presence of skirts, a transfer of the horizontal forces over the whole contact area of the foundation, or even over the total area, may be justified. This approach may have significant benefits in the case of cohesive soils, but will have no effect in the case of drained calculations on a purely frictional soil (because, in this case, the horizontal resistance is bound to the vertical forces and not to the contact surface).

For foundations that are not equipped with skirts, the interface properties between the foundation base and the underlying material have a major importance when assessing the resistance of the foundation to sliding. An interface coefficient (also called roughness coefficient) is applied on the soil shear strength, whether the soil response occurs in drained or undrained conditions. This interface coefficient r (with $r \le 1$) is specific to the existing soil and to the constitutive material of the foundation (in particular, the relative roughness of the contact surface in regard to the size of the soil constitutive elements is a major factor). It is required to justify the interface properties selected for the design on an experimental basis. It may prove beneficial to improve the interface properties on the underbase of the foundation with adapted constructive solutions.

The preferential underbase sliding surfaces to be analysed are located at the interface between the foundation and the existing ground, or at the interface between the artificial platform (if there is one) and the existing ground, as well as inside the soil mass by considering a failure surface at shallow depth. Beyond a certain depth, the failure mechanism is associated to the bearing capacity analyses.

For foundations equipped with a network of skirts, the preferential mechanism of failure will be determined, and if necessary, the layout of the 'internal' skirts will be adapted so it forces the path of the failure plane at the base of the skirts. Short skirts barely contribute to the foundation resistance to sliding, and will usually be neglected. However, in some cases, a fraction of the passive resistance can be taken into account, without exceeding 30%. In addition, the displacements necessary to mobilise this resistance should remain compatible with the displacement required to mobilise friction at the base and the allowable displacements of the structure (provided no erosion occurs during the structure life). The friction on the skirts walls arising from the earth pressure barely contributes to the lateral capacity of the foundation.

The interaction between horizontal forces H and the torsion moment T (i.e., the moment around the vertical axis) should be taken into account either through the direct calculation of an equivalent horizontal force, as proposed in DNVGL-ST-0126 – Annex G2 (2016), or by using H-T interaction envelopes curves, such as the ones proposed by Finnie and Morgan (2004), Yun et al. (2009). Alternatively, other specific analytical models may be applied; they should account for the combined and cumulative effects of shear stresses arising from the horizontal force H and torsion moment T.

10.3.3. ULS STABILITY: BEARING CAPACITY

The bearing capacity of the gravity base foundation should be analysed under the combined effect of vertical loads (V), horizontal loads (H) and overturning moments (M), in an approach called V-H-M. It is reminded that forces applied to the foundations of offshore wind turbines are characterised by significant overturning moments and eccentricity (M/V).

During the preliminary approach, the V-H-M bearing capacity calculation is commonly based on a simple V-H calculation, since the force V is distributed uniformly on the effective area described in paragraph 10.1.4. This simplified method, as described in DNVGL-ST-0126 – Annexes G4 and G5 (2016) and ISO 19901-4 (2016), applies to full foundations and remains approximate, notably in the case of high overturning moments. This approach is potentially detrimental (i.e., highly conservative) for a homogeneous cohesive soil (see: Taiebat and Carter, 2010). One should note that in the case of an undrained calculation, the DNVGL-ST-0126 (2016) formulation requires knowing the undrained shear strength on the failure surface within the soil mass. In a non-homogeneous soil, it therefore requires determining the geometry (and notably the depth) of the failure surface. For the particular case of a profile of cohesion that increases linearly with depth, the analytical solution proposed in ISO 19901-4 (2016) may be used.

Numerical methods that allow producing envelope curves in the V-H-M space for a given soil-foundation system are currently available. In the case of homogeneous soils and foundations with simple geometric shapes (rectangular or circular), generic solutions of V-H-M envelopes are available in the literature, with a few examples published in Randolph and Gourvenec (2011), Gourvenec (2007), Taiebat and Carter (2010), Feng et al. (2014). For profiles of non-homogeneous soils and for more complex geometries, it will be adequate to develop V-H-M envelope curves that are specific to the studied case. The V-H-M global envelopes based on published results or on numerical analyses that are specific to the project should be favoured during project studies.

The resistance anisotropy associated to the failure mechanism involved (direct simple shear, compression, extension), should be taken into consideration during these analyses. An illustration of the various mechanisms involved under a gravity base foundation is presented in Figure 6.12.

10.3.4. ANNULAR FOUNDATIONS AND OTHER COMPLEX GEOMETRIES

Generic V-H-M envelopes curves of capacity are only available for cases of rectangular or circular full foundations and of idealised soil profiles. For other geometries of foundations and more complex soil conditions, assessing the stability of the foundation requires the application of numerical methods accounting for the specificities of the project.

During the preliminary stage, simplified methods may be applied. To verify the bearing capacity, and regardless of the shape of the foundation, one may apply the method based on comparing a reference stress q_{ref} to the allowable stress, with q_{ref} being determined from a trapezoidal distribution of the vertical stress on the surface of the foundation. The interaction effect of the horizontal forces on the vertical capacity may be taken into account by introducing an inclination coefficient of the loads.

In the case of an annular foundation, the effective surface method may also be used. The Appendix B presents a method that allows calculating the effective surface of an annular foundation.

For an idealised foundation geometry, the recommendations of paragraph 10.1.5 are applicable.

10.3.5. ULS STABILITY : TAKING INTO ACCOUNT THE EFFECTS OF CYCLIC LOADING

When cyclic loadings are applied, soils are susceptible to develop and accumulate pore pressures and strains, which may cause degradation of their mechanical resistance and stiffness, and to the generation of permanent deformations. Some materials such as carbonate soils may undergo a particularly severe degradation under cyclic loading.

Besides the accidental case of loads due to earthquakes, the cyclic loads transferred to the ground by the foundation are essentially associated to the action of waves, swell and wind. The effects of these cyclic loads should be considered in the ULS stability calculations. In certain cases, cyclic loading may not induce significant detrimental effects on the soil resistance (ULS).

The true history of cyclic loadings applied to the foundations of offshore wind turbines is highly complex due to the large number of cycles and their irregular and asymmetrical nature. Following individual cycles throughout a loading history that comprises hundreds or thousands of cycles is not deemed feasible in practice.

The most advanced methods of cyclic design are based on an idealisation of loadings and on the implementation of semi-empirical models of soil behaviour.

How cyclic loads should be processed is described in paragraph 4.2.

The principles of selection of the soil parameters adapted to analyses under cyclic loadings are addressed in chapter 6.

Here, it will be assumed that a model of the soil response under cyclic loading is available and can be used for the design of the foundation. This model should include:

- a representation of the accumulation of damages in the soil, based on the accumulation of strains (usually more reliable in clays) or of pore pressures (in sands), in function of the cyclic stress ratio and of the number of applied cycles. An example is shown in Figure 10.4;
- a representation of cyclic shear strength in function of the cyclic stress ratio, for a number of cycles that is applicable to the design, as shown as an example in Figure 10.5 (a, b, c, d).

Note : the representations proposed in Figure 10.4 and Figure 10.5 are the most commonly used ones. However, different representations are possible, using other normalisation parameters.

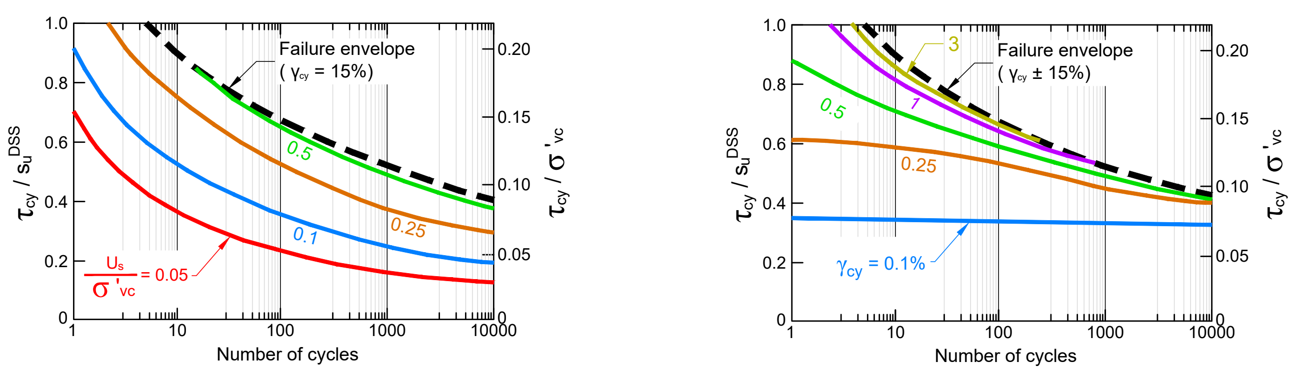

Figure 10.4: Examples of representation of permanent pore pressure (left) and cyclic shear strain (right) in function of the cyclic stress ratio and the number of cycles. Two-way DSS tests (from Andersen et al., 2013)

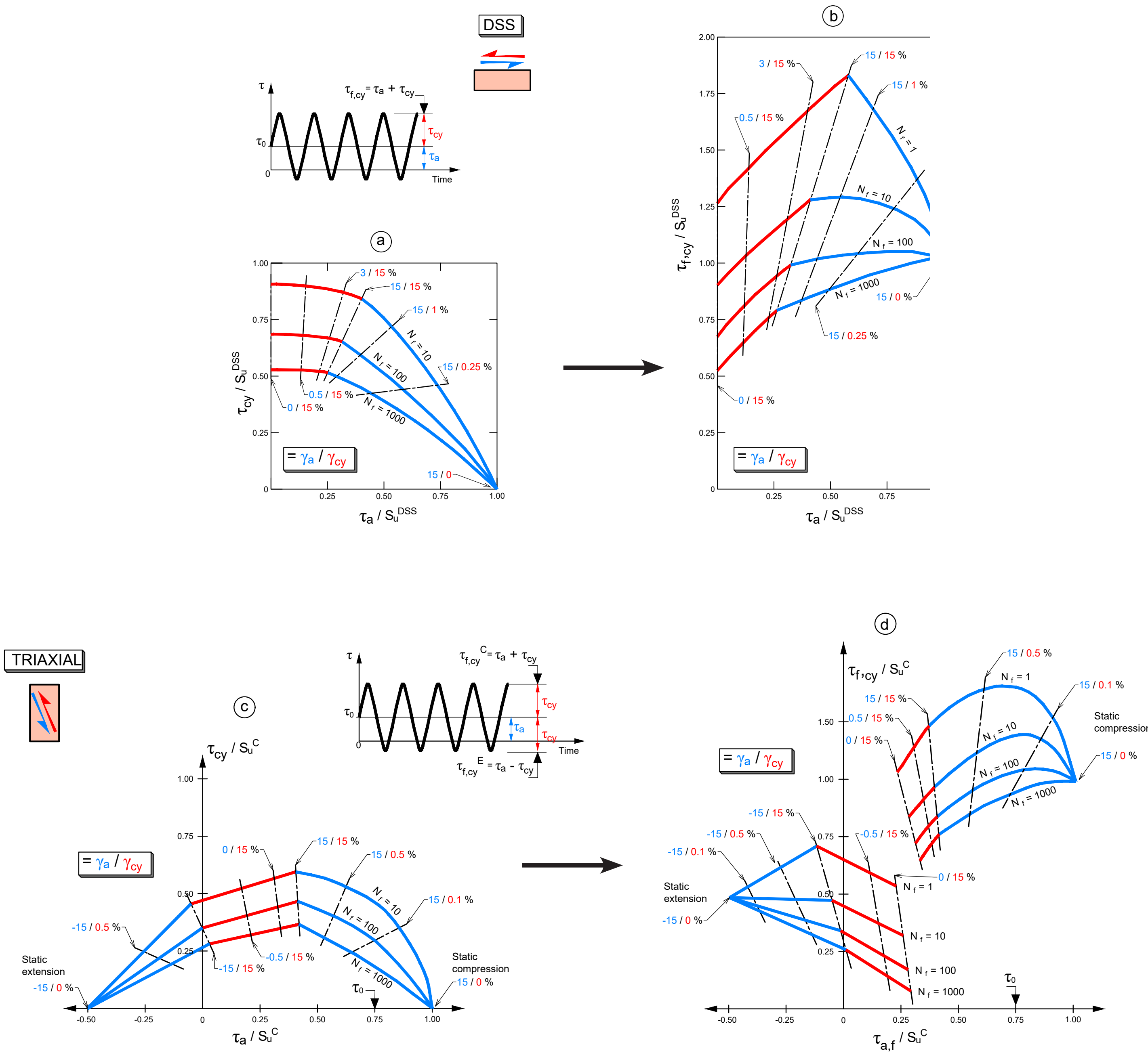

Figure 10.5: Examples of representation of the degradation of shear strength for the whole spectrum of cyclic loading and for different number of cycles, under DSS (a and b) and Triaxial (c and d) loadings, (from Andersen et al., 2013)

In practice, analyses are achieved through several steps.

As a first step, an « idealised » loading is determined from the design event (the storm), which includes a series of parcels of cycles with constant amplitudes, as described in paragraph 4.2.

Then, the average and cyclic stresses corresponding to each amplitude level obtained in the previous step are analysed at each point of the soil mass being loaded by the foundation. By successively applying these parcels of cyclic stress following an increasing order of amplitude, and with a principle of accumulation of damage (from strains or from pore pressures), the final damage is obtained as well as the « equivalent » number of cycles N_{eq} that should have been applied to the last loading « parcel » (corresponding to the maximum stress) to obtain a similar damage. The procedure is illustrated in Figure 10.6.

This makes clear that the equivalent number of cycles N_{eq}, specifically reflects the considered event, i.e. the design storm, and the soil response during this event for the foundation under study.

At each point of the soil mass, for the average and cyclic stress ratios corresponding to the maximum forces and for the N_{eq} previously obtained, the cyclic resistance of the soil can be obtained from the representation proposed in Figure 10.5 (b and d).

This approach assumes a soil response in undrained conditions at the time scale of the storm and of the frequency of cyclic loadings. During the optimisation stage, it is possible to take into account the effect of a partial drainage during the storm, which, if relevant, will lead to a reduction of the equivalent number of cycles N_{eq}. The application of such a method is illustrated in Jostad et al. (2015).

In practice, the methodology proposed above is strictly applicable only during the project study stage when the entire set of required data is available. The final design should take into account the specific loading history of the site being studied for the reference storm. On French coasts and in the future, the results of measurements campaigns will allow improving the forecasting models of loads, and consequently the applicable range of N_{eq}.

During preliminary studies, depending on the level of available data, the following simplified procedure may be applied:

- in the absence of a specific loads history for the verifications against the ULS, a value by default of N_{eq} may be adopted. The choice of this value of N_{eq} may be based on the established histories of storms in other regions of the world. Values of N_{eq} in an order of 5 to 20 equivalent cycles have been obtained for oil and gas projects in the North Sea or in other regions of the world;
- a nominal degradation of cyclic shear strength may be estimated by applying pertinent and conservative reduction factors (associated to a given value of N_{eq} and to a cyclic stress ratio); a unique factor may be applied to the whole mobilised soil mass, or different factors may be applied layer by layer;
- while awaiting a complete soil model applicable to the considered site, a reference soil model may be used by default, provided it is well documented and deemed representative of the existing ground.

The value of cyclic shear strength depends on the studied failure mechanism and may therefore be different for analysing the resistance to sliding or the bearing capacity.

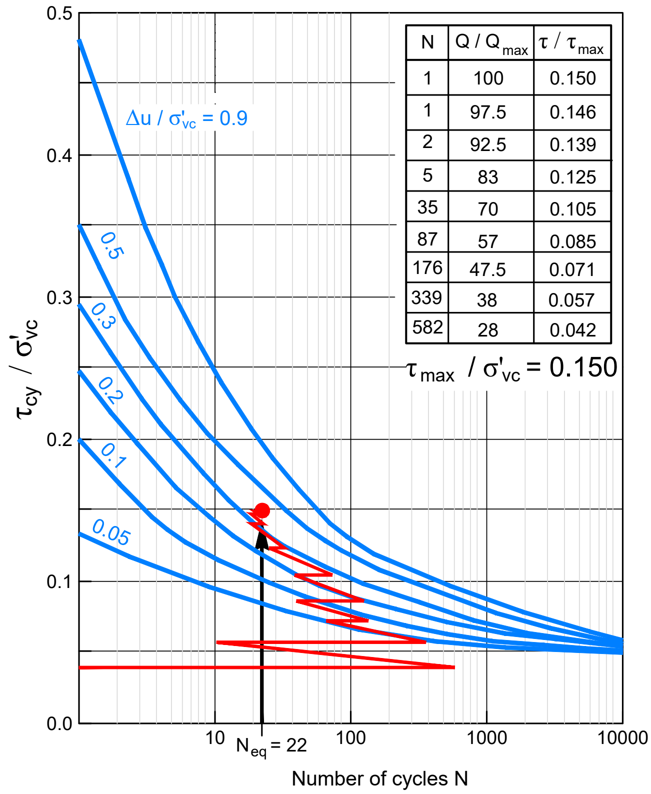

Figure 10.6: Example of implementation of the procedure of accumulation of pore pressure (from Andersen et al., 1988)

10.3.6. OVERTURNING VERIFICATION

The application of other design criteria, notably the verifications of bearing capacity, the criteria of contact area, and the allowable deformations in rotation, implicitly allow satisfying the stability to overturning.

The correspondence between eccentricity, contact area and safety coefficient to overturning for circular and annular foundations is presented in Appendix B.

10.3.7. CONSEQUENCES OF THE WORKS PHASING ON THE SOIL CHARACTERISTICS

In some cases, phasing works to install the foundation implies having clearly identified temporary configurations prior to applying the design forces in the ULS. The improvement of the soil characteristics, notably the increase of monotonous shear strength, resulting from the consolidation under the self-weight associated to these temporary situations, can be taken into account. These effects may be potentially significant depending on the soil properties and the foundation characteristics. Taking into account these effects should be justified given the consolidation properties of the concerned soil layers and the duration of application of these temporary states before applying the design forces.

The increase of the monotonous shear strength relative to consolidation should be applied before the reduction due to cyclic loading. Some models allow integrating both effects into the same calculation.

In theory, the improvement of the soil characteristics associated to a natural cyclic pre-loading (relative to nominal environmental conditions) before the occurrence of the design storm may also be considered. However, quantifying these effects and defining the specific criteria to be taken into account (such as the duration before the application of the design storm and the nature of these nominal environmental conditions) seems difficult to achieve.

10.4. SLS VERIFICATIONS

The purpose of these verifications is to assess the accumulation of expected deformations during the structure life (settlements, rotations and other permanent deformations) in order to ensure that the foundations meet the criteria set by the manufacturer and the operator (paragraph 10.1.2), and consequently to guarantee its operability.

The recommendations presented in paragraph 7.4.4.4 of DNVGL-ST-0126 (2016) may be considered to select the loading conditions for which the verifications against the SLS should be carried out. More specifically, the following cases will require a particular attention:

- single design storm: extreme maximum load for an equivalent number of cycles (usually between 5 and 20);
- emergency or storm shutdown following operational conditions;
- series of storms (an example is presented in LeBlanc et al., 2010);
- LDD 10^{-2} and LDD 10^{-4} load levels as defined in paragraph 7.2.

Under the effect of service loads, the foundation is subject to long-term static displacements and to displacements induced by cyclic loadings. The static displacements, of a gravity origin, are the sum of shear strains, of consolidation settlements (primary consolidation) and of creep settlements (secondary consolidation). The permanent displacements induced by the environmental loads include cyclic shear displacements, as well as post-cyclic settlements due to the dissipation of pore pressures generated during cyclic loadings.

In most cases, static settlements (vertical displacements related to consolidation) may be calculated through an uni-dimensional approach in primary consolidation and in secondary consolidation. The effect of the decrease of overburden with depth should be integrated. In the case of a spatial variability of the ground under the foundation surface and/or predominant eccentric loads (notably related to the preferential directionality of the environmental conditions), the differential settlements and the associated permanent rotations should be determined using an advanced model, as described below.

Assessing the displacements due to cyclic and static shear requires using an advanced model by finite elements that allows extracting, at all levels of loadings, the displacements and stiffnesses of the foundations from the fatigue loads (FLS) to the service loads (SLS) and the ultimate loads (ULS). This model uses a representation of the ground response that allows determining the cyclic and mean strains associated to the considered cyclic loading. As an example, some applicable processes to build such a model are proposed in Appendix C.

The post-cyclic re-consolidation settlement may be calculated by an uni-dimensional model, from the pore pressures generated during a major cyclic event. The distribution of pore pressures under the foundation is determined on the basis of the cyclic and static shear stresses using the same model. The re-consolidation settlement under the foundation is obtained by applying a relation that links local strains to the value of excess pore pressure for the considered soil. As a reference, the works of Yasuhara and Andersen (1991) can be perused.

In the case of a drained, or partially drained, soil response under the considered cyclic loading, the densification may lead to permanent deformations. These additional deformations, and the resulting displacements and rotations at the level of the foundation, should also be assessed within the framework of the verification of the foundation against the SLS.

10.5. MINIMUM CONTACT AREA

The verifications below are associated to the SLS defined in DNVGL-ST-0437-C.3.7 (2016). They are carried out for unfactored loads (partial load coefficient $\gamma_F = 1.00$).

For the LDD 10^{-2} load level that is exceeded during 1% of the structure lifespan (i.e., 1,750 hours over 20 years), the bearing area of the foundation shall be in full contact with the soil, so erosion from a « pumping » effect or from a « flushing » effect can be avoided, an event that could occur following the repeated gapping of the foundation edge over a large number of cycles. The erosion phenomenon due to pumping was observed on a gravity base foundation without skirts and directly laid on sand, as reported by Burland et al. (1978) and by Bishop et al. (1980). In that case, the erosion observed under the foundation had led to an excessive inclination of the structure, under environmental conditions far below the acceptable extreme conditions with respect to the ultimate capacity of the foundation. The condition of full contact area, or condition of non-gapping, is imposed by DNVGL-ST-0126 (2016), § 7.5.5.3.

For the characteristic extreme load, gapping under the footing can be acceptable, but it shall not develop beyond the geometric centre of the foundation (which corresponds to a contact area equal to 50% of the slab area). This condition is also imposed by DNVGL-ST-0126 (2016), §7.5.5.4.

Depending on these criteria, a foundation gapping might be observed for intermediate loadings, between the LDD 10^{-2} reference load and the extreme characteristic load (applied only once) during the design storm. The repetition of gapping during a storm may lead to geotechnical disorders related to the erosion of underlaying materials through pumping effects. Therefore, when the gapping of the footing is expected under certain environmental conditions, it is required to justify that this gapping is not prone to cause irreversible erosion phenomena that could harm the functionality of the foundation. The critical factors to be taken into account during this process of justification are the number of applied cycles leading to the gapping, the amplitude of gapping as well as the resulting discharge flows and flow velocities. These factors should be considered in relation with the sensitivity of surface materials to erosion and with the risk of internal erosion.

The requirement of guaranteeing the absence of any significant erosion related to gapping under the foundation is reminded in chapter 11, which addresses scour.

10.6. MODELLINGS FOR DYNAMIC ANALYSES

10.6.1. OVERVIEW AND USUAL ANALYTICAL SOLUTIONS

As mentioned in paragraph 4.3, in the framework of the studies of soil-structure interaction, the parameters to be used for the dynamic analyses of the structure should be defined. The stiffness coefficients of the foundation represent an essential input data when analysing the dynamic response of the structure and the verification against the fatigue limit states (FLS), as well as when determining the loads applied to the foundation. The natural frequencies, and consequently the FLS, are usually massively influenced by the dynamic response of the foundation, notably by its response in rotation.

The stiffness coefficients corresponding to the different degrees of freedom (vertical, horizontal, torsion, moment) can be represented under the form of 6x6 rigidity matrixes.

The stiffness coefficients of the foundations may be determined in a first step from simplified methods based on the hypothesis of elastic deformations for rigid circular bases. The solutions of Poulos and Davis (1974) may notably be mentioned in the case of a homogeneous soil mass, as well as the ones of Doherty and Deeks (2003) for circular foundations embedded in non-homogeneous soil masses. Gazetas' formulas (1991) extend these solutions to different geometries of foundations, embedded or not, for both homogeneous and non-homogeneous soil masses, as well as for the case of a homogeneous soil mass of low thickness over a bedrock of infinite stiffness. Some of these solutions are recalled in DNVGL-ST-0126 (2016).

These formulations make use of a single value of the shear modulus and do not reflect the non-linearity of the shear modulus with respect to the applied strain level and its dependency to the cyclic characteristics of the considered load case. To apply these simplified methods, it is therefore necessary to select a value of the shear modulus G that is compatible with the strains induced by the loads being analysed. Different values of G are generally used depending on the considered displacement mode, since these modes mobilise different volumes of soil. Specific recommendations are detailed in Gazetas (1991) for idealised profiles of shear modulus.

10.6.2. EFFECT OF GAPPING UNDER THE FOOTING

In the formulations of stiffness coefficients, it is critical to consider, if needed, the effect of gapping under the footing resulting from heavy loads. The evolution of the stiffness coefficient in rotation in the event of gapping is illustrated in RFG 138-139 (2012) and in IEC 61400-6 Annex L3 (draft 2016, to be officially published). In this representation, only the geometric effects (i.e., the ones directly related to the reduction of the contact area) are taken into account.

As an example, Figure 10.7 presents the moment-rotation curve relative to a rigid circular base that supposedly exerts a pressure of trapezoidal distribution (or triangular in the gapping stage), with:

- M : overturning moment applied on the foundation;
- M_u : moment that causes the overturning of the foundation (M_u = R.V, with R the radius of the base and V the resultant of the vertical load);
- θ : rotation of the foundation under the effect of M;
- θ_0 : selected reference rotation, equal to the foundation rotation at the initiation of gapping, i.e. the rotation generated under $M_0 = 0.25.M_u$. If $K_{M,0}$ is the foundation stiffness in rotation in the absence of any gapping (the stiffness resulting, for instance, from the application of the analytical models previously mentioned), then: $\theta_0 = 0.25.\ M_u / K_{M,0}$

This curve may also be obtained numerically for configurations requiring a treatment beyond the usual analytical solutions.

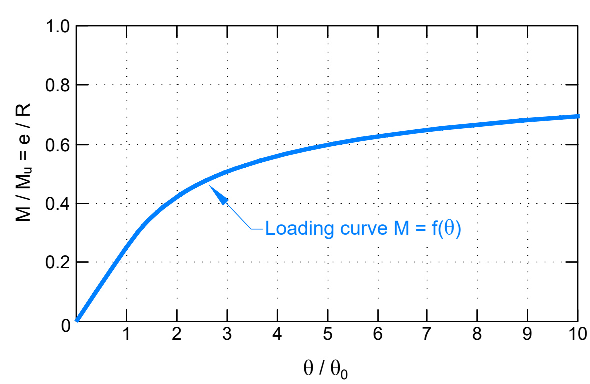

Figure 10.7: Normalised moment-rotation curve – Circular foundation

This curve may constitute a basis to define an equivalent secant stiffness value. The evolution of the stiffness coefficient in rotation in the event of gapping under the footing is illustrated in RFG N° 138-139 (2012). In this representation, only the geometric effects (i.e., the effects directly related to the reduction of the contact area) are taken into account. The result of this approach is illustrated in Figure 10.8 for the case of a rectangular or circular foundation.

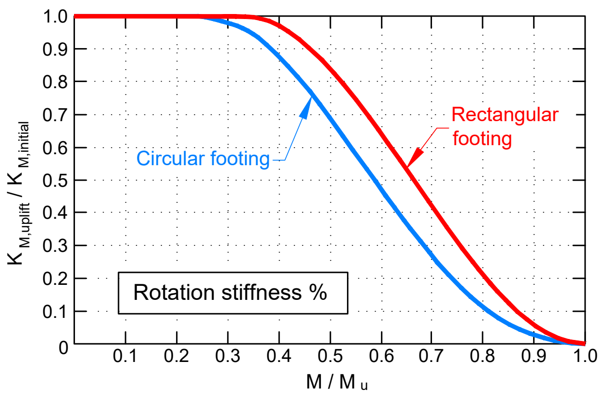

Figure 10.8 : Normalised stiffness - moment curve for two types of foundations

One should pay attention to the fact that such an approach, which consists in defining the secant stiffness value as being the ratio between the applied moment and the resulting rotation, may, in the case of a cyclic or dynamic loading, lead to underestimating the elastic energy and therefore overestimating the apparent flexibility, as shown in Figure 10.9. In reality, the energy equivalence requires to take into account an « apparent » term of damping, which is representative of the geometric non-linearity (ratio between the area of the « missing term » and the one of the triangle of Figure 10.9).

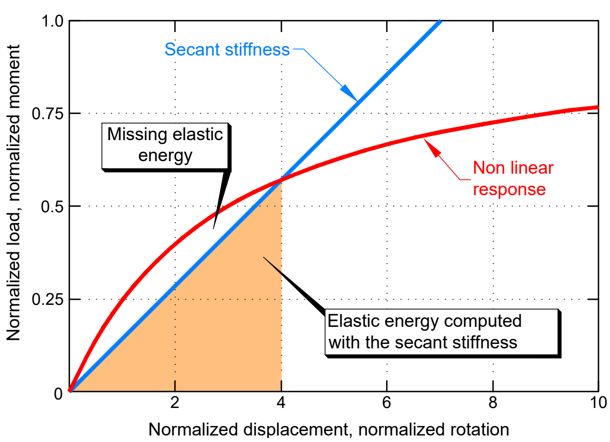

Figure 10.9: Illustration of the underestimation of the elastic energy by taking into account a secant value

In some cases, generic studies may allow justifying the selected values.

However, in most cases, a subsequent verification, and, if necessary, an adjustment of these stiffness coefficients should be carried out on the basis of an advanced model by finite elements, as described in paragraph 10.4 - SLS verifications.

10.6.3. EFFECTS OF THE NON-LINEARITY OF THE SOIL RESPONSE AND HYSTERETIC DAMPING

The analyses of soil-structure interaction usually result in an iterative process between the geotechnical calculation (leading to the determination of the rigidity matrixes for the different load cases being considered) and the calculation of the dynamic response of the structure (leading to the determination of the loads applied to the foundation and to the analyses against the FLS).

Because the ground behaves non-linearly, deformation moduli should take into account the strain level induced in the ground by the considered loading. The variations of moduli with strain level are discussed in chapter 6, paragraph 6.4.3.

If the soil shear modulus is likely to evolve over the structure lifespan (for instance, through the effects of initial consolidation and/or re-consolidation following a storm), these evolutions should be taken into account in the analyses of soil-structure interaction, since they may have a significant impact, notably in relation to the FLS. The solution is then to bound the ground response range, through calculation, and therefore bound the potential stiffness range. Technical exchanges should take place between the teams in charge of the geotechnical calculation, and the teams carrying out the dynamic analysis of the structure, in order to grasp the factors that allow determining the upper and lower bounds relevant for the analyses of the foundation.

The analysis of the dynamic response of the wind turbine may be enhanced by taking into account the material (or hysteretic) and radiative (or geometric) damping effects specific to the « soil + foundation » complex. The hysteretic damping results from the non-linear behaviour of the soil and is function of the distortion. These aspects are discussed in chapter 6, paragraph 6.4.4.

10.6.4. EFFECT OF FREQUENCY ON RADIATIVE DAMPING

The radiative damping term depends on the geometry of the foundation and of the frequency content of the loading. Figure 10.10 presents a result obtained from applying Gazetas' analytical solutions (1991), established for a circular base with a diameter B laid on a semi-infinite homogeneous soil mass characterised by a shear modulus G and a unit mass ρ. The « radiative » damping ratio for the « rotational » mode (or rocking mode) is then exclusively expressed in function of the non-dimensional frequency a_0, defined below:

$$a_0 = \pi \cdot B \cdot f \cdot \sqrt{(\rho/G)}$$

where f is the considered vibration frequency. For values of a_0 lower than 0.4, radiative damping is negligible.

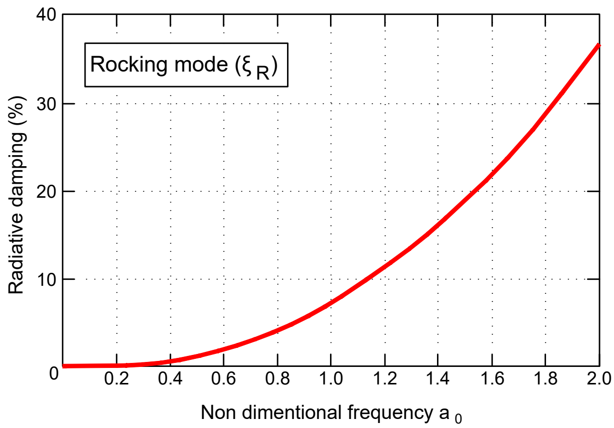

Figure 10.10: Radiative damping in function of the non-dimensional frequency a_0

10.7. REACTIONS OF THE GROUND ACTING UPON THE STRUCTURE

10.7.1. DISTRIBUTION OF VERTICAL STRESS

In a first approach, the distribution of vertical stress from the ground reaction acting upon the foundation is calculated by assuming a plane linear variation that allows balancing the overturning moments applied to the foundation. This may be completed through a more detailed analysis that takes into account the rigidities of the ground and of the structure, and, if possible the non-linearity of the ground response. This approach presupposes a uniform reaction of the footing on the ground under the effect of vertical loads.

The distribution of vertical stress resulting from this analysis is notably used to check the criterion of minimum contact area as detailed in paragraph 10.5.

10.7.2. OTHER CONSIDERATIONS

The design of the structure elements that constitute the foundations should take into account the stress applied by the ground on these structure elements. This stress should be assessed conservatively on the basis of the in-place soil properties, and of the properties of the artificial platform if any, as well as for the load cases representing the conditions to be taken into account in the structural analyses.

The heterogeneity of the ground below the foundation and the unevenness of the contact surface should be considered in this assessment because of the risk of concentration of stresses they may cause. The hypotheses formulated about the stiffness of the ground and of the structure elements may influence this assessment.

In the case of a foundation equipped with skirts, the same principle of conservative assessment of the stress applied by the ground on the skirts is applicable, so it allows their structural design, including during the installation.

10.7.3. RÉSISTANCE TO SKIRT PENETRATION

The resistance to penetration of skirts during the installation of the foundation is the sum of the end bearing resistance base and of the friction mobilised on lateral walls.

The end bearing resistance may be assessed with the conventional formulations of bearing capacity of a strip footing, or on the basis of results from static penetrometer tests (PCPT) by applying a coefficient on the cone resistance.

Friction may be assessed from the mechanical properties of the materials being crossed and from the soil-skirt interface properties (determined notably with laboratory tests), or on the basis of results from static penetrometer tests (PCPT) by applying an empirical coefficient on the cone resistance.

Coefficients are proposed to assess end bearing resistance and friction from the PCPT results in DNVGL-RP-C212 (2017) for generic soils of the « sand » or « clay » type encountered in the North Sea. For soils of intermediate composition, and non-conventional soils, it is strongly recommended to apply methods that have been specifically developed for those material types. One may notably peruse ARGEMA-CLAROM (1994) for carbonate sands, and NF P 94-261 (2013) - Annex P for chalks.

The analyses of resistance to penetration shall pay caution to the lateral variability of the soil properties and to the potential presence of coarse materials that may cause a concentration of stresses or a premature refusal.

10.8. INSTALLATION

The installation stage of the gravity base foundation shall be handled as a particular situation for which the verification criteria against the ULS and SLS remain applicable nonetheless. The conditions to be verified cover all identifiable temporary situations related to the considered building stages. Thus, the verifications against the ULS as described in paragraph 10.3 remain pertinent. The environmental conditions applicable to the installation case differ from the conditions applicable over the structure lifespan. Extreme environmental conditions depend on the period and duration of installation (see: DLC 8.1 to 8.5 of DNVGL-ST-0437, 2016).

The laying of the foundation on the ground constitutes a specific case to be taken into account during the bearing capacity verifications against the ULS. The kinematic and dynamic effects should be integrated into the determination of the loads associated to this case.

According to DNV OS H204 (2013), it is recommended to verify that no gapping can occur under the entire set of load cases to be considered in the ULS for these temporary conditions. This verification of non-gapping of the footing is therefore made for the unfactored extreme loads that characterise the considered temporary situations of installation.

Furthermore, when getting close to the sea bottom, horizontal currents are generated, which are related to the descent speed of the foundation. It is appropriate to determine the limit speed of descent that allows avoiding the associated erosion phenomena (of the existing soil, or of the base course, if any) that would possibly compromise the flatness of the contact area.

When a base course must be set before laying the foundation (and before installing the anti-scour protection, if any) its stability in regard to the erosion phenomenon should be verified for the entire duration of exposure, in compliance with the environmental criteria that are applicable to this temporary situation.

In the case of a foundation equipped with skirts:

- the resistance to skirts penetration during the installation should be assessed in line with the principles mentioned in paragraph 10.7.3 in order to ensure that the vertical forces applied during the installation of the foundation are sufficient to guarantee a full penetration.
- the number and dimensions of the vents that allow draining the water from the compartments formed by the skirts should be adapted in function of the considered penetration speed (including hydrodynamic effects) during the installation of the foundation, in order to avoid any failure or piping phenomena related to an excessive overpressure in this (these) compartment(s).

10.9. DECOMMISSIONING

According to the regulatory framework applicable to an offshore wind turbine project, and to the requirements stated by the developer, the full removal of the gravity base foundation at the end of the operations may have to be considered at the early stages of design. In this case, analyses are required, notably by taking into account the selected method of removal and the environmental conditions applicable to these operations.

In the presence of skirts, the resistance to removal should be assessed by taking into account the increase of the ground resistance acting on the skirts over the structure lifespan, the suction forces that are potentially generated on the surface of the base as well as the possible presence of debris and marine growth on the structure.

Structural and mechanical equipment required to remove the foundations should be adequately designed in function of the considered conditions and the expected forces during the dismantling.

10.10. GROUND PREPARATION AND CONTACT TREATMENT

In the case of a relatively flat sea floor, the foundation may be directly laid on the ground without any prior preparation of the contact surface. For a foundation without skirts, the risk of erosion due to pumping induced by gapping under the footing should be addressed with the highest attention (paragraph 10.5 and chapter 11). The addition of skirts may, to some extent, allow avoiding the effect of erosion from pumping.

De facto, gravity foundations are often laid on an artificial platform, composed of engineered coarse materials that allow guaranteeing flatness of the layer and adequate draining conditions.

In-place surficial materials having poor properties may be removed by dredging if necessary, and possibly replaced.

In the case of an uneven ground, one may consider installing the foundation on the existing ground and then proceeding to underbase grouting in order to fill the voids and therefore guarantee a homogeneous contact over the entire area of the foundation. This alternative seems more specifically adapted in presence of outcropping rock, provided grout confinement be operationally ensured.

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11 SCOUR AND SEDIMENTARY MOBILITY

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11. SCOUR AND SEDIMENTARY MOBILITY

11.1. OVERVIEW

This chapter addresses the phenomena of erosion that are likely to occur around, or close to, the structures installed on the seabed. Such phenomena result from complex interactions between the flowing fluid (water), the marine ground and the structure that disturbs the initial motions of the fluid under the actions of currents and waves. The erosion may have a non-negligible impact on the capacity and performance (rigidity) of the foundations, up to the point where they may compromise operations. Forecasting the phenomena of erosion is a complex task that requires a highly interdisciplinary process (fluids mechanics, geotechnics, morphology). Setting up systems that allow limiting or delaying the erosion is an alternative that is often considered. Managing the risks relative to erosion is increasingly considered through probabilistic models.

11.2. SEDIMENTARY MOBILITY

Sedimentary mobility describes ground movements resulting from the morphodynamic conditions on the site in the absence of structure. As examples, the morphology changes of rivers estuary, the mobility of large dunes under the action of tide currents or also the gravity events of large flows of high-density fluids (mud) in underwater corridors can all be mentioned as such conditions.

These aspects are not taken into consideration within the present recommendations.

11.3. GENERALISED AND LOCAL SCOUR

The presence of a structure changes the flow conditions that were predominating in the absence of that structure, and may consequently generate new phenomena of erosion on the sea bottom. These phenomena, which may evolve with the reached level of erosion, can be classified into two categories depending on their geometry: generalised scour or local scour.

Generalised scour impacts a wide zone around the structure. The sedimentary losses are sufficiently large to modify stresses at depth within the foundation soil mass.

Local scour initiates in the immediate proximity of the structure. Even though it may be spectacular, its effects on the structure remain localised (absence of horizontal reaction on a pile, for instance).

Figure 11.1 clearly highlights the difference between generalised and local scour in the surroundings of a jacket founded on piles.

A list of references showing the impacts of scour on various types of structures is available in Luger et al. (2017).

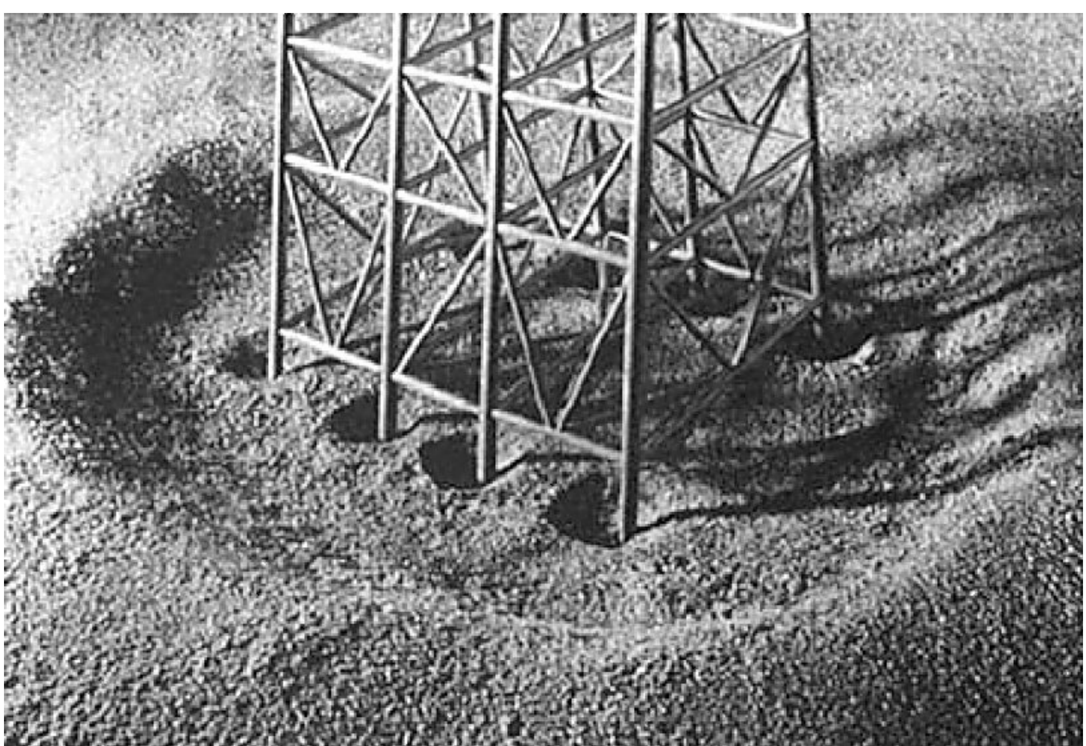

Figure 11.1: Generalised and local scour around a jacket-type platform founded on piles
(from Angus and Moore, 1982)

11.4. SEABED – FLUID – STRUCTURE INTERACTIONS

When a structure is installed on the seabed, it is subject to the hydrodynamic forces induced by currents and oscillating flows. Through its mere presence, the structure disturbs the movements of fluids and becomes an obstacle to the processes of sedimentary transport existing at the level of the seabed. What may result from this is the apparition of new mechanisms that will reconfigure the sea floor and modify its properties around and under the structure.

These mechanisms are complex, highly interdependent and non-linear. Their impact cannot be assessed by superposing the fluid-structure, fluid-ground and ground-structure elementary interactions. It is required to consider the interactions between the seabed, the fluids in motion and the structure as a global set of elements, as illustrated in Figure 11.2.

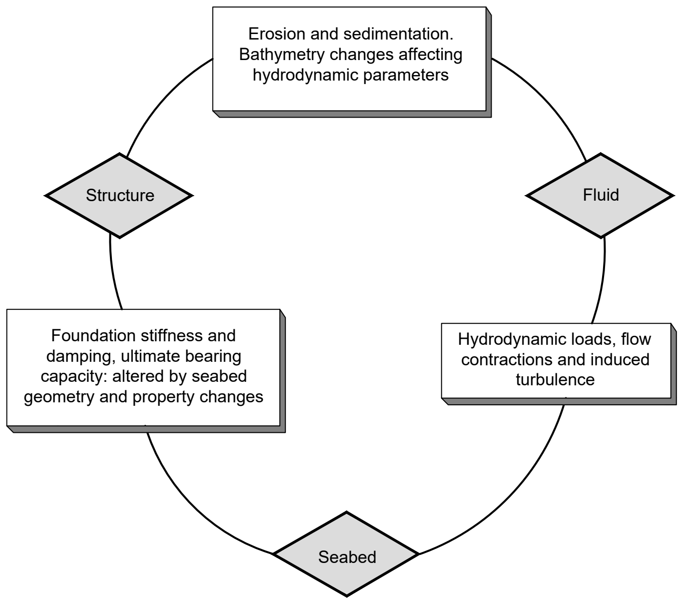

Figure 11.2: Seabed – structure - fluid interaction (from Luger et al., 2017)

11.5. FORECASTING SCOUR

Forecasting scour in the proximity of offshore structures remains an arduous and eminently interdisciplinary task, requiring experts from the hydrodynamical, geotechnical and dynamic morphology fields.

The approach is most often semi-empirical and calls on a combination of theoretical knowledge, of data collection and of modellings.

Scour initially occurs at the interface between water and ground. It is controlled by the equilibrium between the erosive force F and the resistance to erosion R. The erosive force is most often characterised by the shear stress generated by the motion of the fluid on the interface. The resistance force is characterised by the properties (grain size, cohesion, friction angle, viscosity) of the material that is in direct contact with the fluid. An in-depth knowledge of the properties at stake (fluid and ground) is paramount, and assumes having detailed hydrodynamic modellings and specific geotechnical measurements in layers of soil that are usually ignored during usual investigations. For sands and gravels, the method proposed by Soulsby et Whitehouse (1997) may be considered. For silts, muds, or when the structure or cementation effect of the material may be significant, it is recommended (Whitehouse et Harris, 2014) to collect samples in the presumed interaction zone and then proceed to laboratory erosion tests. In-situ erosion testing may also be considered.

The obstruction caused by the structure generates close to the seabed the apparition of a highly localised influence zone in which the movement conditions (F) are amplified by a factor M >1, which means that scour will always initiate at the contact of the structure, before increasing in volume.

Figure 11.3 summarises the initiation process of scour.

Several examples of assessment of the amplification factors M are available in the technical literature for various types of structures. For a particular project, the values of M may be determined with hydraulic physical models, or from numerical modellings.

The rate of erosion is a major parameter when defining the risk management approach. Its assessment is arduous and most often requires advanced studies that combine erosion tests, developing semi-empirical formulations, laboratory scour tests (physical modelling) and numerical modelling (often called CFD, for « Computational Fluid Dynamics »). The experience feedbacks that can be acquired through the study of scour on existing structures is a source of information that is particularly useful to confirm the validity of empirical and theoretical models.

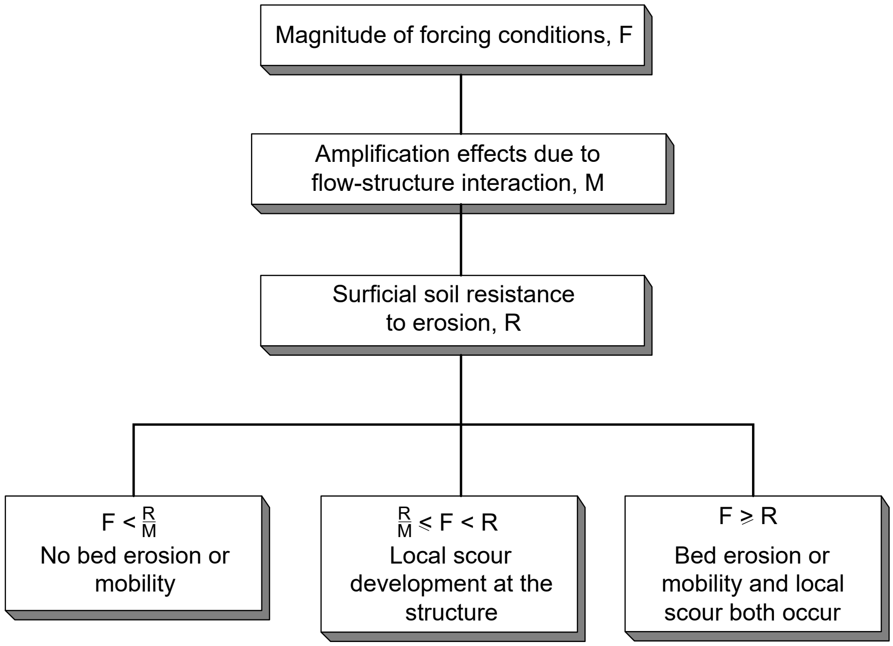

Figure 11.3: Initiation process of scour (from Luger et al., 2017)

11.6. CONSEQUENCES OF SCOUR

Scour may have more or less severe consequences, depending on the type of structure and the scope of the phenomenon.

As a non-comprehensive list, the following can be mentioned:

- reduction of the the stress-field within the foundation soil mass in the event of generalised scour;
- disappearance of the active/passive pressures around a pile/monopile over the scour height;
- disappearance of skin friction around a pile/monopile over the scour height;
- reduction of the rigidity in rotation of monopiles;
- disappearance of the passive resistances in front of the skirts of a gravity foundation;
- reduction of the bearing capacity under a gravity base with the possibility of scour occurring under the base, notably in presence of angular geometries (square foundation for instance).

In the case of gravity bases, scour may be coupled with phenomena of hydraulic flows under the skirts. These aspects are mentioned in paragraph 11.11.

11.7. RISK MANAGEMENT

Assessing scour around offshore structures, through an analytical and/or numerical method, allows determining its impact on the performance of the structure. If, in addition, the evolution of the phenomenon over time can be estimated, a true plan of risk management can be developed. It may allow withholding investments that will only be engaged when the necessity is truly established.

Three types of measures can be considered to avoid the adverse effects of scour:

- dimensioning the structure so that its safety is ensured in the conditions of maximum expected scour. For instance, the penetration length of a pile or of a monopile can be extended, or the gravity base may be equipped with peripheral skirts. This option is usually favoured when the scope of the expected scour remains limited and costs overruns are moderate;
- pre-installing anti-scour systems before the commissioning of the structure, so that the integrity of the sea floor and the structure is guaranteed over the longer term. This option requires short-term investments and is only justified if scour risks are particularly high and the consequences are highly detrimental to the structure performance;
- planning a reasonable reserve of capacity and implementing a maintenance and monitoring plan to observe the reality and evolution of scour. This option allows withholding investments and implementing anti-scour systems only when it is truly needed.

11.8. PROTECTION MEASURES

Protective measures against scour include:

- Rock dumping of blocks with sufficiently large dimensions to not be moved by hydrodynamic flows;

- installing cement mattresses, earth bags (called « geobags »), rock filled nets, tyre mats or frond mats…

Physical modellings are often required to assess the efficiency of these systems within the project conditions, and optimise design.

11.9. THE CASE OF PILES

In the case of piles, the single or combined impact of generalised and local scour is taken into account when assessing the bearing capacity and the axial and lateral performance of the pile by:

- reducing the effective vertical stress in the soil mass. ISO 19901-4 (2016) proposes a methodology whose principle is summarised in Figure 11.4 and in Figure 11.5. The effect of this reduction notably affects the limit skin frictions and the t-z and p-y transfer curves;
- assessing the effect of this reduction on the non-intrinsic parameters of the underlying materials, for instance on the values of cone resistance q_c;
- suppressing any soil-pile interaction (skin friction, transfer curves) over the whole height of the zone subject to scour.

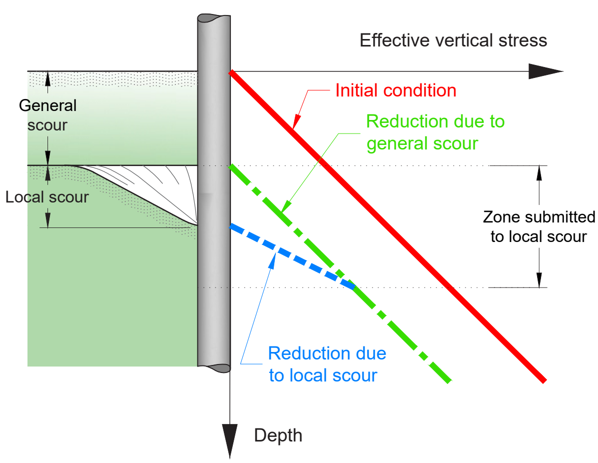

Figure 11.4: Principle of reduction of effective vertical stress in the presence of scour, according to ISO 19901-4 (2016) - Homogeneous soil

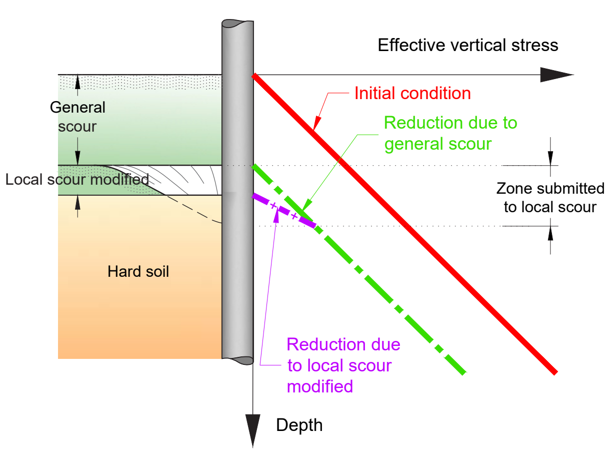

Figure 11.5: Principle of reduction of effective vertical stress in the presence of scour, according to ISO 19901-4 (2016) – Two-layer configuration

11.10. THE CASE OF MONOPILES

The different hydrodynamic mechanisms generated by the presence of a monopile are represented schematically in Figure 11.6.

In the absence of anti-scour protection, the development of scour around the monopile should be assessed and taken into account within the design process of the foundation (Whitehouse et al., 2011 a, b). The risk of scour should be systematically assessed in the presence of sandy or silty materials in surface, but erosion phenomena have also been witnessed on clayey materials (experience feedbacks of the Barrow wind farm in the United Kingdom, presented by Whitehouse et al., 2011).

Given the large diameter of monopiles, and their low slenderness, scour may quickly have a significant impact on their response in terms of capacity and lateral stiffness.

When the lateral displacements of the monopile have an amplitude that is large enough to generate a gap between the ground and the monopile (Kallehave et al., 2015, mention an amplitude larger than 3% of the diameter), flushes between the pile and the ground may initiate erosion and remoulding phenomena on the material. Their impact on the lateral response shall be assessed.

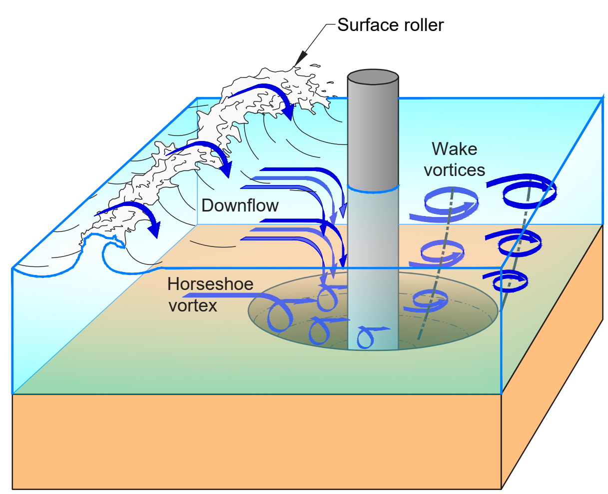

Figure 11.6 : Mechanisms causing scour around a monopile

The potential effect of anti-scour equipments on the foundations stiffnesses should be considered. Kallehave et al. (2015) report an increase of 5% of the estimated natural frequencies because of the anti-scour equipments installed on sands around the monopiles of the Horn's Reef wind farm (Denmark).

11.11. THE CASE OF GRAVITY BASES

One can distinguish:

- the case of a base laid on a substratum identified as being non-erodible (rock for instance);
- the case of a base laid on a preformed platform. In that case, the materials that support the base are selected and installed in such a way that any risk of erosion and ground-fluid interaction is avoided (paragraph 10.10);
- the case of a base designed to be directly laid on a loose/soft material. In that case, the phenomena of ground-fluid-structure interaction may prove to be highly complex.

In the case of a base designed to be directly installed on a loose/soft material (sand, silt, sand-silt mixes, or even clay) the scour that results from the mere presence of the base can potentially develop at the bottom of the base. However, the phenomenon will most likely be highly amplified due to the rocking movements of the base, which generate horizontal flushes that increase with the development of gapping between base and ground. These aspects are addressed in paragraphs 10.5, 10.8 and 10.10.

In order to limit the scope of these phenomena, bases are most often equipped with peripheral skirts. In that case, the scour per se that may develop in front of the skirts should be assessed, and its impact on the stability of the base should be taken into account (reduction of front reaction on the skirts, reduction of bearing capacity and of rocking stiffness). Furthermore, it is appropriate to ensure the hydraulic stability of the base-skirts system by assessing the risks of occurrence of piping along the skirts and of internal erosion under the base caused by a pumping effect due to the rotation of the base induced by strong swell.

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APPENDIX A

LOADS TABLE

APPENDIX A

LOADS TABLE

The table below summarises all the combinations of environmental loads E under IEC 61400-3 (2009).

Design situation	DLC	Wind conditions	Waves	Wind and waves directionality	Sea currents	Water level	Other conditions
1) Power production	1.1	NTM $V_{in} < V_{hub} < V_{out}$ RNA	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	For extrapolation of extreme loads on a RNA
	1.2	NTM $V_{in} < V_{hub} < V_{out}$	NSS Joint probable distribution of H_s, T_p, V_{hub}	COD, MUL	No currents	NWLR ou $\geq$ MSL	
	1.3	ETM $V_{in} < V_{hub} < V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	
	1.4	ECD $V_{hub} = V_r - 2$ m/s, V_r, $V_r + 2$ m/s	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	MIS, Wind direction	NCM	MSL	
	1.5	EWS $V_{in} < V_{hub} < V_{out}$	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	
	1.6a	NTM $V_{in} < V_{hub} < V_{out}$	SSS $H_s = H_{s,SSS}$	COD, UNI	NCM	NWLR	
	1.6b	NTM $V_{in} < V_{hub} < V_{out}$	SWH $H = H_{SWH}$	COD, UNI	NCM	NWLR	
2) Power production plus occurrence of fault	2.1	NTM $V_{in} < V_{hub} < V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	Control system fault or loss of electrical network
	2.2	NTM $V_{in} < V_{hub} < V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	Protection system or preceding internal electrical fault
	2.3	EOG $V_{hub} = V_r \pm 2$ m/s and V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	External or internal electrical fault, including loss of electrical network
	2.4	NTM $V_{in} < V_{hub} < V_{out}$	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	No currents	NWLR or $\geq$ MSL	Control protection, or electrical system faults, including loss of electrical network
3) Start up	3.1	NWP $V_{in} < V_{hub} < V_{out}$	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	No currents	NWLR or $\geq$ MSL	
	3.2	EOG $V_{hub} = V_{in}$, $V_r \pm$ 2m/s and V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	
	3.3	EDC_1 $V_{hub} = V_{in}$, $V_r \pm$ 2m/s and V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	MIS, Wind direction change	NCM	MSL	

Design situation	DLC	Wind conditions	Waves	Wind and waves directionality	Sea currents	Water level	Other conditions
4) Normal shutdown	4.1	NWP $V_{in} < V_{hub} < V_{out}$	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	No currents	NWLR or $\geq$ MSL	
	4.2	EOG $V_{hub} = V_r \pm 2$ m/s and V_{out}	NSS (or NWH) $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	
5) Emergency shutdown	5.1	NTM $V_{hub} = V_r \pm 2$ m/s and V_{out}	NSS $H_s = E[H_s \| V_{hub}]$	COD, UNI	NCM	MSL	
6) Parked (standing still or idling)	6.1a	EWM Turbulent wind model $V_{hub} = k_1 V_{ref}$	ESS $H_s = k_2 H_{s50}$	MIS, MUL	ECM	EWLR	
	6.1b	EWM Steady wind model $V(z_{hub}) = V_{e50}$	RWH $H = H_{red50}$	MIS, MUL	ECM	EWLR	
	6.1c	RWM Steady wind model $V(z_{hub}) = V_{red50}$	RWH $H = H_{red50}$	MIS, MUL	ECM	EWLR	
	6.2a	EWM Turbulent wind model $V(z_{hub}) = V_{e50}$	ESS $H_s = k_2 H_{s50}$	MIS, MUL	ECM	EWLR	Loss of electrical network
	6.2b	EWM Steady wind model $V(z_{hub}) = V_{e50}$	RWH $H = H_{red50}$	MIS, MUL	ECM	EWLR	Loss of electrical network
	6.3a	EWM Turbulent wind model $V_{hub} = k_1 V_1$	ESS $H_s = k_2 H_{s1}$	MIS, MUL	ECM	EWLR	Extreme yaw misalignment
	6.3b	EWM Steady wind model $V(z_{hub}) = V_{e1}$	RWH $H = H_{red1}$	MIS, MUL	ECM	EWLR	Extreme yaw misalignment
	6.4	NTM $V_{hub} < 0{,}7\, V_{ref}$	NSS joint probable distribution of H_s, T_p, V_{hub}	COD, MUL	No currents	NWLR or $\geq$ MSL	
7) Parked and fault conditions	7.1a	EWM Turbulent wind model $V_{hub} = k_1 V_1$	ESS $H_s = k_2 H_{s1}$	MIS, MUL	ECM	NWLR	
	7.1b	EWM Steady wind model $V(z_{hub}) = V_{e1}$	RWH $H = H_{red1}$	MIS, MUL	ECM	NWLR	
8) Transport, assembly, maintenance and repair	8.1	To be stated by the manufacturer					
	8.2a	EWM Turbulent wind model $V_{hub} = k_1 V_1$	ESS $H_s = k2\, H_{s1}$	COD, UNI	ECM	NWLR	
	8.2b	EWM Steady wind model $V_{hub} = V_{e1}$	RWH $H = H_{red1}$	COD, UNI	ECM	NWLR	
	8.2c	RWM Steady wind model $V(z_{hub}) = V_{red1}$	EWH $H = H_1$	COD, UNI	ECM	NWLR	
	8.3	NTM $V_{hub} < 0{,}7\, V_{ref}$	NSS joint probable distribution of H_s, T_p, V_{hub}	COD, MUL	No currents	NWLR or $\geq$ MSL	No grid during installation period

THE FOLLOWING ABBREVIATIONS ARE USED IN THE LOADS TABLE :

COD	co-directional (see: NF EN 61400 -3 (2016) § 6.4.1)
DLC	design load case
ECD	extreme coherent gust, with direction change (see: IEC 61400-1)
ECM	extreme current model (see: IEC 61400 -3 (2016) § 6.4.2.5)
EDC	extreme direction change (see: IEC 61400-1)
EOG	extreme operating gust (see: IEC 61400-1)
ESS	extreme sea state (see: IEC 61400 -3 (2016) § 6.4.1.5)
EWH	extreme wave height (see: IEC 61400 -3 (2016) § 6.4.1.6)
EWLR	extreme water level range (see: IEC 61400 -3 (2016) § 6.4.3.2)
EWM	extreme wind speed model (see: IEC 61400-1)
EWS	extreme wind shear (see: IEC 61400-1)
MIS	misaligned (see: IEC 61400 -3 (2016) § 6.4.1)
MSL	mean sea level (see: IEC 61400 -3 (2016) § 6.4.3)
MUL	multi-directional (see: IEC 61400 -3 (2016) § 6.4.1)
NCM	normal current model (see: IEC 61400 -3 (2016) § 6.4.2.4)
NSS	normal sea state (see: IEC 61400 -3 (2016) § 6.4.1.1)
NTM	normal turbulence model (see: IEC 61400-1)
NWH	normal wave height (see: IEC 61400 -3 (2016) § 6.4.1.2)
NWLR	normal water level range (see: IEC 61400 -3 (2016) § 6.4.3.1)
NWP	normal wind profile model (see: IEC 61400-1)
RNA	rotor-nacelle assembly
RWH	reduced wave height (see: IEC 61400 -3 (2016) § 6.4.1.7)
RWM	reduced wind speed model (see: 6.3)
SSS	state of severe sea state (see: IEC 61400 -3 (2016) § 6.4.1.3)
SWH	severe wave height (see: IEC 61400 -3 (2016) § 6.4.1.4)
UNI	uni-directional (see: IEC 61400 -3 (2016) § 6.4.1)
V_r +-2m/s	sensitivity to all wind speeds in the range shall be analysed
F	fatigue (see: IEC 61400 -3 (2016) § 7.6.3)
U	ultimate strength (see: IEC 61400 -3 (2016) § 7.6.2)
N	normal
A	abnormal
T	transport and erection

APPENDIX B

ANNULAR BASES

B1. OVERVIEW

B1.1. INTRODUCTION

Gravity bases with an annular contact surface with the ground are commonly used.

Considering an annulus allows increasing the contact surface keeping the external radius constant but increasing stresses on ground. On soils with good mechanical characteristics, taking into account the annulus may become a financial alternative to a solution where the external diameter would be increased to meet the criteria of minimum contact surface.

The purpose of this appendix is to provide the elements that will allow identifying the benefits and consequences of taking into account an annulus during the pre-design process.

B1.2. CONDITIONS OF USE

In order to be able to consider a base as an annulus, there should not be any effective transfer of vertical pressure to the ground, including through the water that could be trapped under the base, in the central part that is not in direct contact with the ground.

B1.3. EVOLUTION OF THE GEOMETRIC PROPERTIES

Figure B.2 presents the variation of the geometric properties (inertia I and section S) of an annular foundation, versus those of a circular foundation.

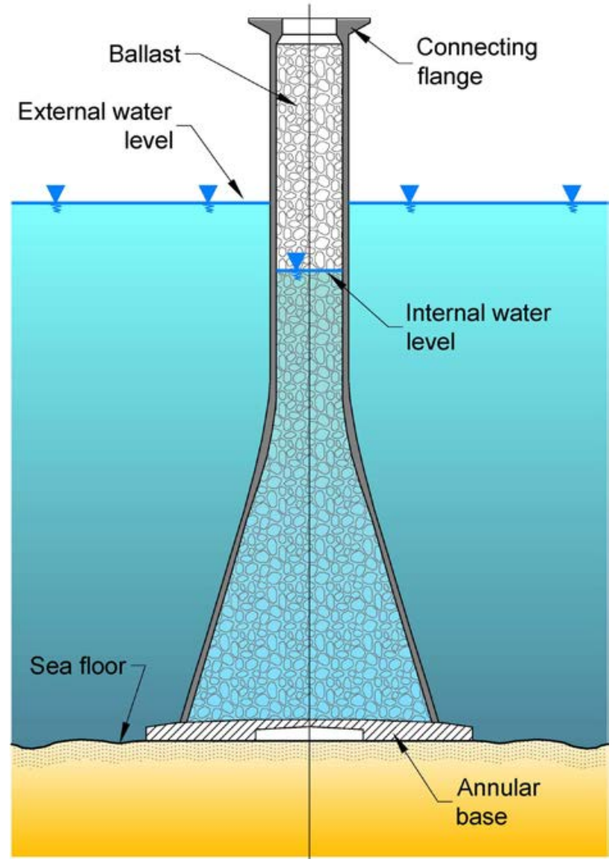

Figure B.1

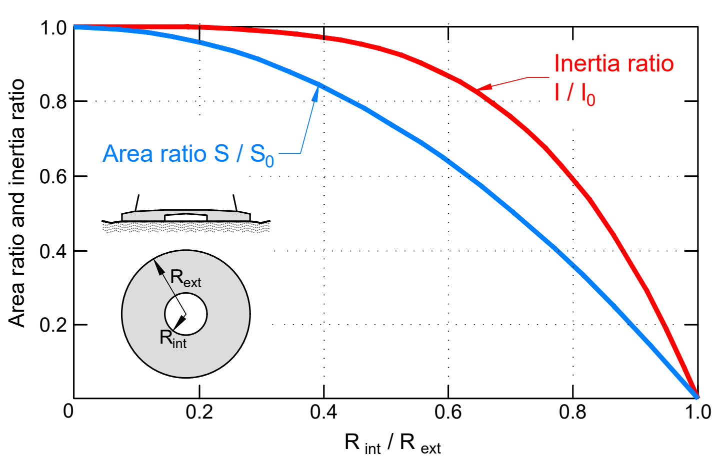

Figure B.2

I_0 ($I_0 = \pi.D_{ext}^4/64$) and S_0 respectively designate the inertia and the bearing surface of a full circular base with a diameter $D_{ext} = 2.R_{ext}$.

B2. CONTACT SURFACE : TRIANGULAR DIAGRAM

B2.1. INTRODUCTION

This diagram, illustrated in Figure B.3, is used to determine the contact length (= length under compression) and the contact surface (= base area under compression). The model consists in approximating the ground reaction under the foundation with a trapezoidal diagram, which implicitly assumes the foundation as being infinitely rigid and the ground as being assimilated to a distribution of homogeneous vertical springs.

Thus, edge effects, which play a favourable role on the stability of the foundation to overturning, are ignored. These effects, highlighted through a modelling derived from the elasticity theory (ground assimilated as a continuous elastic environment) are greater when the foundation rigidity increases. For a real ground, these edge effects are limited by plastic deformations.

Figure B.4 introduces the notions of contact surface and contact length in the case of an annular foundation.

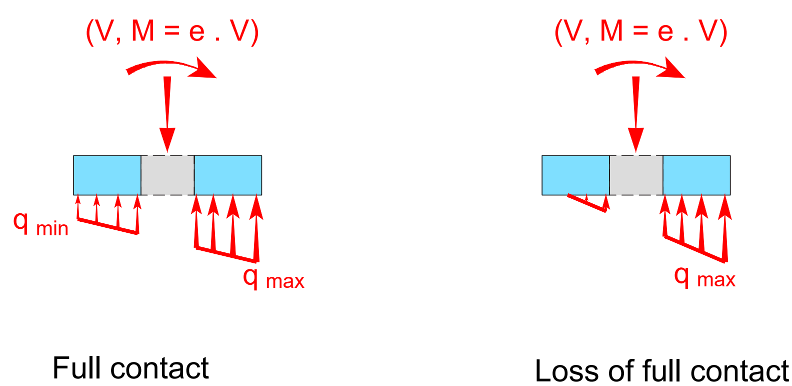

Figure B.3

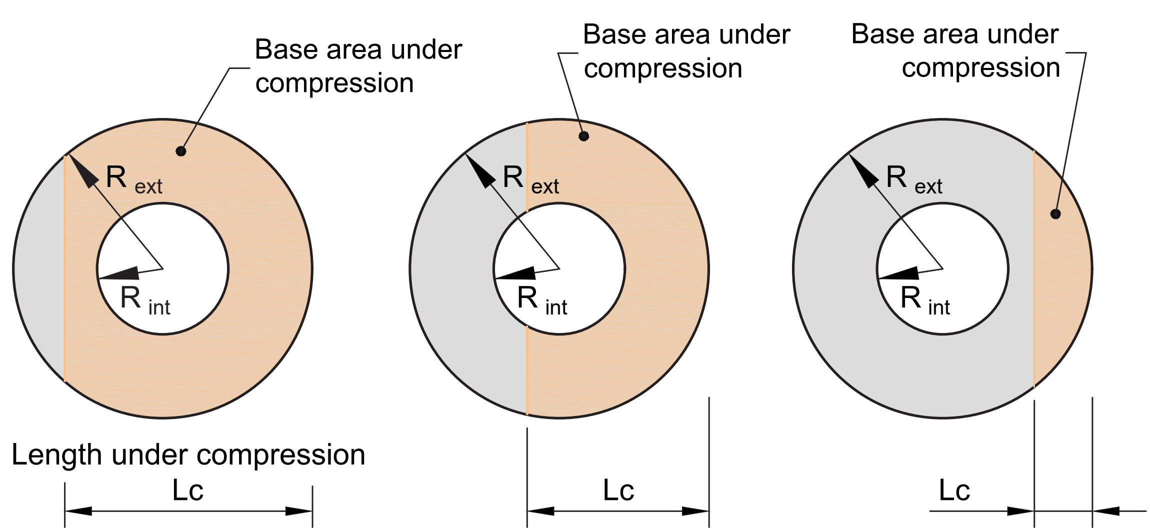

Figure B.4

B2.2. CONTACT LENGTH AND MAXIMUM STRESS

Researching the pressure diagram (assumed to obey a trapezoidal or triangular shape in the event of gapping) under the foundation is a process made by writing the static equilibrium of the foundation, i.e. two equations (equilibrium of vertical forces and equilibrium of moments) with two unknowns: q_{min} and q_{max} when there is no gapping, L_c and q_{max} when there is gapping.

Figure B.5 presents the variation of contact length in function of the relative eccentricity (e/R_{ext} ratio) for different R_{int}/R_{ext} ratios. One should note in particular that the effect of an annular shape is not significant for internal radiuses when they are smaller than $0.3 \, . \, R_{ext}$.

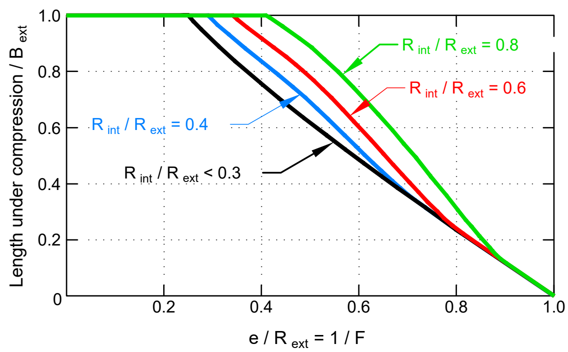

Figure B.5

It is useful to remind that the relative eccentricity e/R_{ext} is merely the inverse of the safety coefficient between the overturning moment (= $R_{ext}.V$) and the applied moment (= $e.V$), i.e., $e/R_{ext} = 1/F$.

The model also allows determining the evolution of stresses q_{min} and q_{max} in function of eccentricity. Then, the notion of reference stress is introduced, which is usually defined as follows : $q_{ref} = (q_{min}+3.q_{max})/4$. Figure B.6 presents the evolution of this reference stress normalised by the mean stress q_0 (before gapping) under a circular foundation $q_0 = V/S_0$.

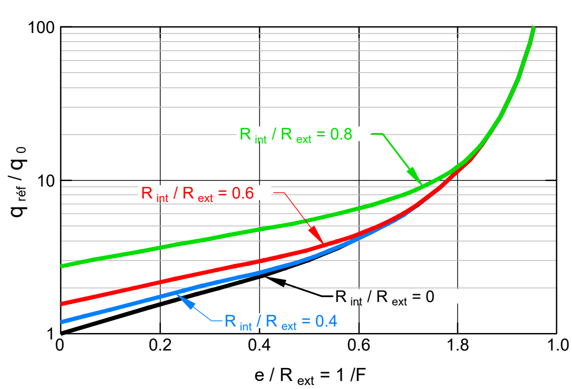

Figure B.6

B2.3. CONTACT SURFACE

Figure B.7 presents the variation of the contact surface in function of the relative eccentricity (e/R_{ext} ratio), for different ratios : $R_{int}/R_{ext} = D_{int}/D_{ext}$.

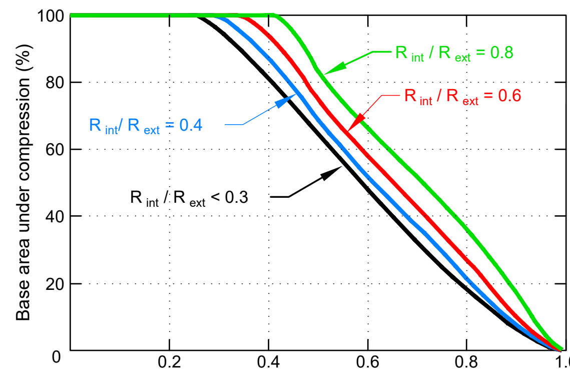

Figure B.7

Figure B.8 provides an alternative presentation of the evolution of the percentage of contact surface in function of the safety coefficient to overturning F.

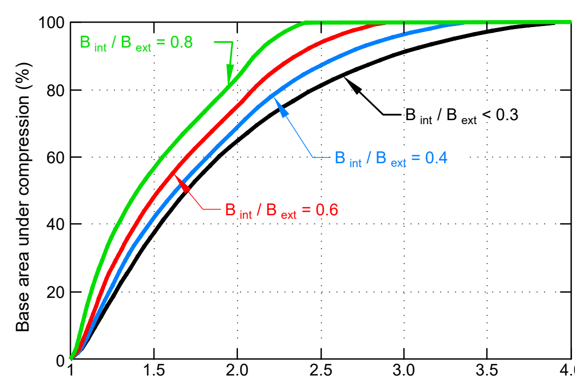

Figure B.8

B2.4. CHARACTERISTIC ECCENTRICITIES

Figure B.9 presents characteristic eccentricities corresponding to a contact ratio of 50% and 100% respectively.

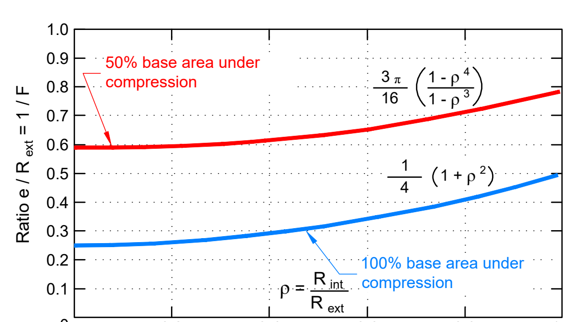

Figure B.9

B3. EFFECTIVE SURFACE

B3.1. INTRODUCTION

The model of « effective surface » is used to determine the equivalent rectangular bearing surface that allows carrying out verifications of bearing capacity taking into account the coupling effects (H, V, T). The diagram is built by assuming a uniform distribution of the interaction pressure on the effective surface. This surface is researched in a way that guarantees the equivalency of forces and moments throughout the foundation as seen in Figure B.10 (left: without gapping; right: with gapping).

B3.2. PRINCIPLE OF THE PROPOSED APPROACH

The proposed approach is an extrapolation of the lune model used in the case of a full circular foundation. The external border of the diagram matches the contour of the base. The inner border is sought in a way it meets the equivalency of moments and forces.

The principle is illustrated in Figure B.11.

G : gravity centre of the lune (brown area)

e : distance of G to the centre of the ring

x_m : distance of the intersection between the lune and the outer border of the ring to the centre of the ring

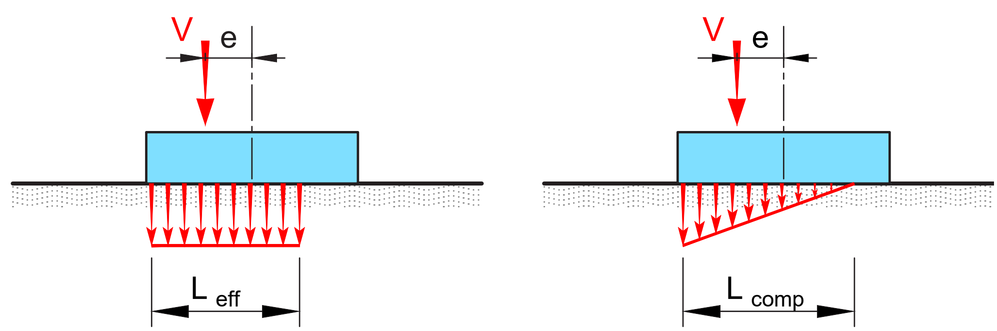

Figure B.10

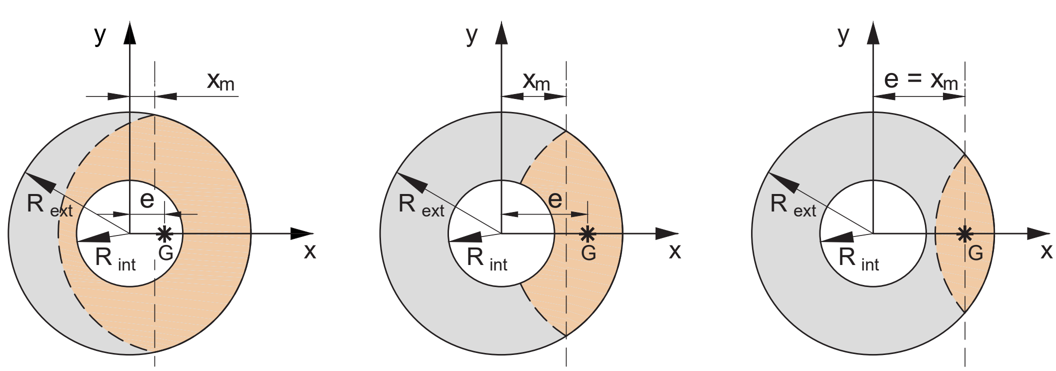

Figure B.11

B3.3. EFFECTIVE SURFACE

Putting into equation the proposed model (Figure B.11) allows determining the evolution of the effective surface in function of the relative eccentricity, illustrated in Figure B.12. The effective surface is here normalised to the total gross area S_0 of a full disk.

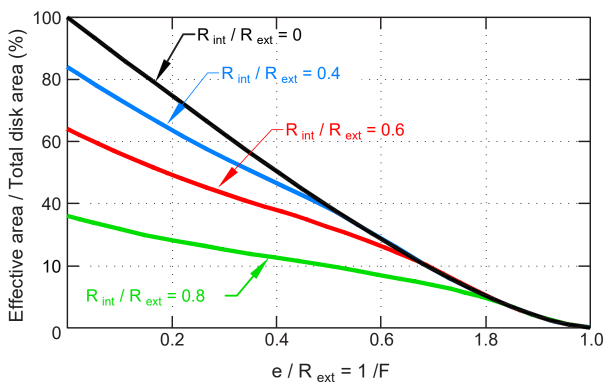

Figure B.12

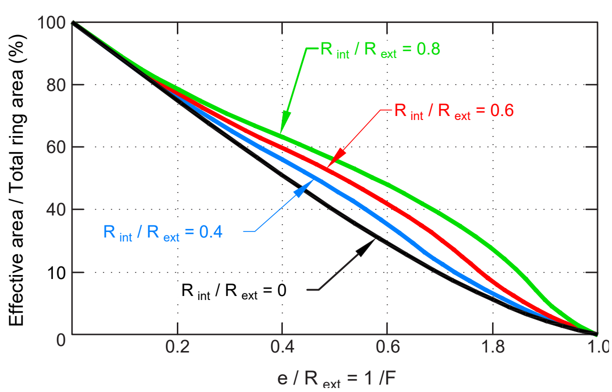

Figure B.13

Figure B.13 provides an alternative representation of the previous result, by normalising the effective surface to the total surface of the considered annulus.

B3.4 EQUIVALENT LENGTH AND WIDTH

Once the effective surface is obtained, it is usually used as a basis to define an equivalent rectangular surface $B_{eq}.L_{eq}$ that guarantees the same slenderness ratio $L_{eq}/B_{eq} = L_{max}/B_{max}$, the same surface $L_{eq}.B_{eq} = S_{eff}$, and the same centre of gravity (G), according to the indications of figure B.14. These equivalent dimensions allow carrying out stability verifications on the basis of usual analytical or semi-analytical models.

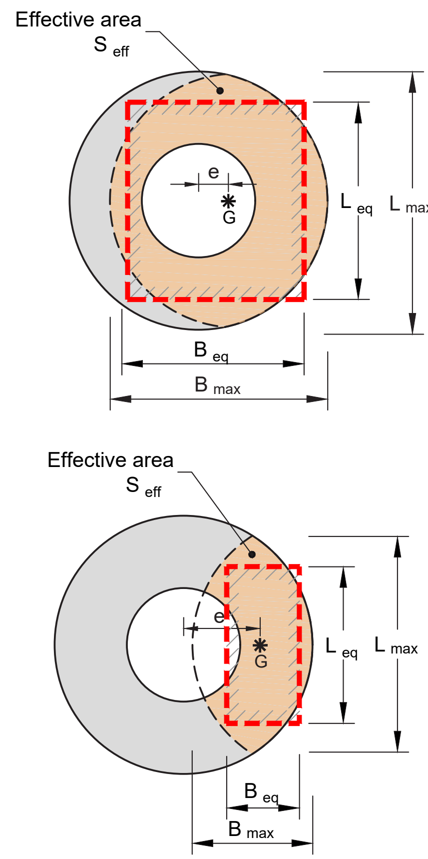

Figure B.14

Figure B.15 presents the variation of equivalent length B_{eq} in function of the relative eccentricity e/R_{ext}. The value of B_{eq} is normalised to the outer diameter of the annulus B_{ext}.

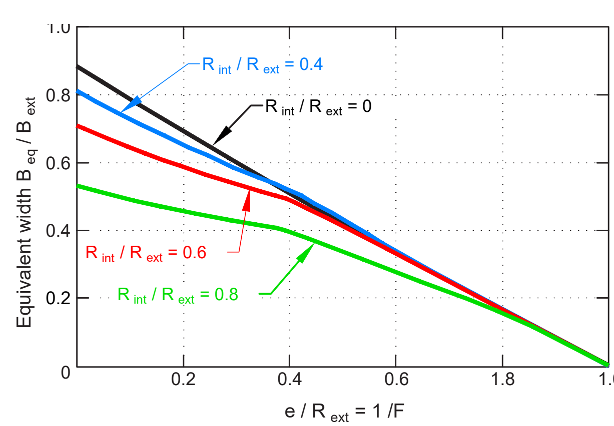

Figure B.15

Figure B.16 presents the variation of the equivalent length L_{eq} in function of the relative eccentricity e/R_{ext}. The observed change of slope on the obtained curves highlights the start of the interaction with the central disk.

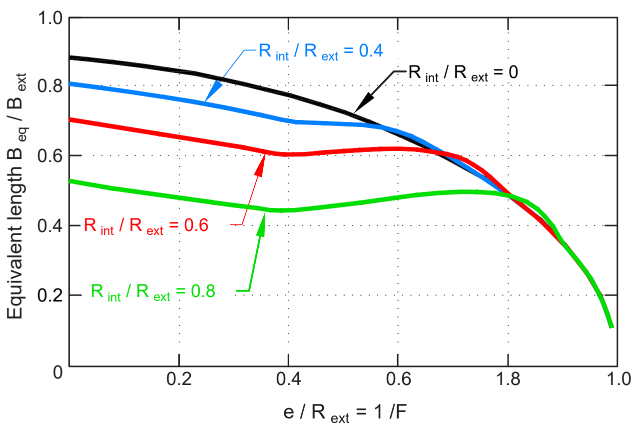

Figure B.16

Crédit photo : EDF- renouvelables / Blyth

APPENDIX C

EXAMPLE OF MODELLING OF A GRAVITY BASE BY FINITE ELEMENTS

APPENDIX C

EXAMPLE OF MODELLING OF A GRAVITY BASE BY FINITE ELEMENTS

During the stage of detailed design, the use of an advanced modelling by finite elements of the foundation behaviour is unavoidable. This is a complex task which requires a thorough process.

The principle of a possible modelling procedure is described below. This type of model allows to simultaneously:

- verify and optimise the stability (ULS);
- analyse the displacements in service (SLS);
- analyse stiffnesses under fatigue loadings (FLS).

Other approaches may be considered.

The model proposed here is based on the implementation of a specific coding that allows expanding the model of cyclic shear strength under an element-by-element formulation in which a unique stress-strain relation is prescribed for each element from the cyclic loading rate specific to this element. The stress-strain relations are defined over the whole range of strains, from the very small strains (domain of G_{max} or G_0) to the failure strains (usually above 15%).

Typically, the study involves a series of steps and iterations. For the verification of the ULS, the following procedure may be followed:

Step 1: building of the 3D model of the foundation and of the ground;

Step 2: initial analysis to determine the distribution of initial effective stresses (σ'_{h0}, σ'_{v0}) under the foundation, resulting from the initial in-situ stress state and from the consolidation stresses due to the weight of the foundation;

Step 3: within the model, definition of a monotonous stress-strain relation for each element (or group of elements) based on the specific soil data and on the applicable S_u/σ'_{v0} ratio. Implementation of the model under maximum (cycle peak) and minimum (cycle trough) loads;

Step 4: for each load cycle (peak and trough), extraction of the inferred shear stress on the most critical shear surface; assessment of the cyclic loading rate that is representative of each element;

Step 5: for each element, definition of the applicable stress-strain cyclic relation, with, as a basis, the value of the cyclic loading rate that is applicable to the element (or to the group), the type of soil, the relevant S_u/σ'_{v0} ratio and the equivalent number of cycles. Reiteration of the previous load cases (peak and trough). Proportional increase of the peak load until failure and calculation of the partial material coefficient;

Step 6: extraction of shear stresses on the most critical shear surfaces for peak and trough loads; assessment of the updated cyclic loading rates;

Step 7: update of the cyclic stress-strain relation on the basis of the new cyclic loading rates and iteration of calculations; if needed, the task is repeated until a satisfying convergence on material factors is obtained;

Step 8: extraction of foundation displacements and reactions under the base for the desired loading levels.

The procedure described above is easily transposable to the verification of SLS cases (determination of displacements under different loading cases) or FLS cases (determination of the foundation cyclic rigidity).

LIST OF ACRONYMS

LIST OF SYMBOLS

LATIN SCRIPTS

GREEK SCRIPTS

LIST OF ACRONYMS

- ALS : Accidental Limit State
- API : American Petroleum Institute
- ARGEMA : Association de Recherche en GEotechnique MArine (*Research Association of Marine Geotechnics*)
- ASTM : American Society for Testing Materials
- BE : Best Estimate
- BRGM : Bureau des Recherches Géologiques et Minières (French Geological Survey)
- BS : British Standards
- BSH : Bundesamt für Seeschifffahrt und Hydrographie (*Federal Maritime and Hydrographic Agency of Germany*)
- CAU : Anisotropically consolidated triaxial test with undrained loading
- CAD : Anisotropically consolidated triaxial test with drained loading
- CAU_C : Anisotropically consolidated triaxial test with undrained loading, in compression
- CAU_E : Anisotropically consolidated triaxial test with undrained loading, in extension
- CE : Conservative Estimate
- CFMS : Comité Français de Mécanique des Sols et de Géotechnique (*French Committee of Soils Mechanics and Geotechnics)*
- CK_0U : Anisotropically consolidated triaxial test (under K_0 conditions) with undrained loading
- CK_0D : Anisotropically consolidated triaxial test (under K_0 conditions) with drained loading
- CID : Isotropically consolidated triaxial test with drained loading
- CIRIA : Construction Industry Research and Information Association
- CIU : Isotropically consolidated triaxial test with undrained loading
- CLAROM : Club pour Les Actions de Recherche sur les Ouvrages en Mer (*Club for Research Activities on Offshore Structures*)
- CNL : Shear box test under Constant Normal Loading
- CNS : Shear box test under Constant Normal Stiffness
- CPT : Cone Penetration Test
- CPTU : Cone Penetration Test with pore pressure measurement (piezocone)
- CSS : Cyclic direct Simple Shear
- CV : Shear box test under Constant Volume
- DLC : Design Load Case
- DNV : Det Norske Veritas
- DNV GL : Det Norske Veritas Germanischer Lloyd
- DSS : Direct Simple Shear
- DTS : Desk Top Study
- EN : European Standard
- FEED : Front End Engineering Design
- FLS : Fatigue Limit State
- GIS : Geographical Information System

- GL : Germanischer Lloyd
- GSI : Geological Strength Index
- HE : High Estimate
- HPDT High Pressure Dilatometer Test
- HR : High Resolution (seismic reflection)
- ICL : Imperial College London
- ICP : Imperial College Pile
- IEC : International Electrotechnical Commission
- IFP : Institut Français du Pétrole (*French Petroleum Institute*)
- Ifremer : Institut Français de Recherche pour l'Exploitation de la MER (*French Research Institute for Exploitation of the Sea*)
- I_s 50 : Point Load Test Index
- ISO : International Standard Organisation
- ISSMGE : International Society of Soil Mechanics and Geotechnical Engineering
- LE : Low Estimate
- LDD : Load Duration Distribution
- MASW : Multichannel Analysis of Surface Waves
- MBES : Multibeam Echosounder System
- NF : French Standard (Norme Française)
- NF EN : European Standard with the status of a French Standard
- NGI : Norwegian Geotechnical Institute
- OCR : Overconsolidation Ratio
- OE : Optimistic Estimate
- PAF : Self-boring Pressuremeter
- PISA : PIle Soil Analysis project
- PMT : Ménard Pressuremeter Test
- PCPT : see CPTU
- RQD : Rock Quality Designation
- RMR : Rock Mass Rating
- SCPT : Seismic Cone Penetration Test (with waves velocities measurement)
- SCPTU : Seismic Cone Penetration Test, with pore pressure measurement
- SHOM : Service Hydrographique et Océanographique de la Marine (*French Naval Hydrographic and Oceanographic Service*)
- SLS : Serviceability Limit State
- SOLCYP : Collaborative project on Piles under Cyclic Loading
- SRD : Soil Resistance to Driving
- SSS : Side Scan Sonar
- SUT : Society for Underwater Technology
- TLP : Tension Leg Platform
- TX : Triaxial Test
- TX_C : Triaxial Test, in compression

- TX_E : Triaxial Test, in extension
- UCS : Unconfined Compressive Strength
- UHR : Ultra High Resolution (seismic reflection)
- ULS : Ultimate Limit State
- UU : Unconsolidated Triaxial Test, with Undrained loading
- UWA : University of Western Australia
- UXO : UneXploded Ordnances
- VST : Vane Shear Test

LIST OF SYMBOLS

LATIN SCRIPTS

Script	Unit	Description
a	[m]	Thickness of the grout annulus in a pre-drilled pile
a_0	[-]	Non-dimensional frequency
A_{eff}	[m²]	Effective area of a shallow foundation (Meyerhof's method)
A_p	[m²]	Cross sectional section of a pile
A_{tot}	[m²]	Total area of a shallow foundation
B	[m]	Diameter of a pile or of a circular foundation/width of a rectangular foundation
B' or B_{eff}	[m]	Effective diameter or effective width of a foundation
B_{ext}	[m]	Outer diameter of a pile
B_{int}	[m]	Inner diameter of a tubular pile
B_q	[-]	Pore pressure ratio determined from CPTU
B_0	[m]	Diameter of a pile of reference
c'	[Pa]	Effective cohesion
c_{pi}	[-]	Tangential compressibility index
c_{pl}	[-]	Limit compressibility index (carbonate sands)
c_u	[Pa]	Undrained cohesion
C_v	[m²/s]	Vertical consolidation coefficient
C_c	[-]	Compression index
C_r	[-]	Recompression index
CR	[-]	Relative stiffness coefficient of a pile under lateral loading
C_U	[-]	Uniformity coefficient
C_α	[-]	Secondary consolidation coefficient (compression)
D	[m]	Pile penetration length
e	[m]	Thickness of a tubular pile $e = (B_{ext} - B_{int})/2$
e	[m]	Eccentricity
e	[-]	Void ratio
e_0	[-]	Void ratio under σ'_{vo} stress

Script	Unit	Description
E	[Pa]	Young's modulus
E_{intact}	[Pa]	Young's modulus of an intact rock sample
E_{mass}	[Pa]	Young's modulus of an in-situ rock mass
E_p	[Pa]	Young's modulus of the pile
E_pI_p	[N.m²]	Flexural pile stiffness
E_s	[Pa]	Young's modulus of a homogeneous and isotropic soil
E_0 or (E_{max})	[Pa]	Young's modulus at very small strains
E_{50}	[Pa]	Secant Young's modulus at 50% of ultimate strength
E_M	[Pa]	Pressuremeter modulus determined from a MENARD pressuremeter test
f	[-]	Frequency of the cyclic loads
f_s	[Pa]	Skin friction measured on the sleeve of a cone penetrometer
$f_{s\ lim}$	[Pa]	Limit skin friction on pile wall (maximum authorised value)
$F(e)$	[-]	Function depending of the void ratio when expressing G_0
F_R	[-]	Normalised friction ratio (%) (cone penetrometer)
g	[m/s²]	Acceleration of the earth gravity field
G	[Pa]	Shear modulus
G_{cy}	[Pa]	Cyclic shear modulus measured over a loading cycle
G_{intact}	[Pa]	Shear modulus of an intact rock sample
G_{mass}	[Pa]	Shear modulus of an in-situ rock mass
G_o (or G_{max})	[Pa]	Shear modulus at very small strains
G_{50}	[Pa]	Secant shear modulus at 50% of the ultimate strength
H	[N]	Lateral load
H_c	[N]	Half-amplitude of the cyclic lateral load
H_{lim}	[N]	Conventional limit lateral load
H_m	[N]	Average lateral load
H_{max}	[N]	Maximum cyclic lateral load
H_{min}	[N]	Minimum cyclic lateral load
I_p	[m⁴]	Moment of inertia of a pile
I_C	[-]	Consistency index
I_D	[-]	Density index
I_L	[-]	Liquidity index
I_P	[-]	Plasticity index
I_R	[-]	Rigidity index (Poulos and Davies, 1980)
j	[s/m]	Smith's dynamic amplification factor (or damping factor) during driving
j_m	[-]	Rock mass factor (for assessing rock mass moduli)
j_p	[s/m]	Dynamic amplification factor on toe resistance
j_s	[s/m]	Dynamic amplification factor on skin friction resistance
k	[-]	Exponent depending on I_P when expressing G_0

Script	Unit	Description
k	[-]	Factor applied to the ground E_s Young's modulus (Figure 8.9)
k	[Pa/m]	Gradient of increase of the lineic reaction modulus with depth
k_a and k_s	[-]	Coefficients used in expressing the pile lateral displacement under a cyclic lateral load
k_c	[-]	Bearing capacity factor (static penetrometer test)
k_h	[m/s]	Horizontal Darcy's permeability
k_i or k_s	[Pa/m]	Coefficient of horizontal subgrade reaction
k_p	[-]	Bearing capacity factor (pressuremeter test)
k_q	[Pa/m]	Axial transfer rigidity of an axial load under the base of a pile according to NF P 94-262 (2012)
k_v	[m/s]	Vertical Darcy's permeability
k_τ	[Pa/m]	Local transfer rigidity of an axial load along a pile according to NF P 94-262 (2012)
K or K_{axial}	[N/m]	Local axial transfer stiffness of a pile
K_i or K_s	[Pa]	Modulus of horizontal subgrade reaction ($K_i = B \cdot k_i$; $K_s = B \cdot k_s$)
K_0	[-]	Coefficient of earth pressure at rest
K_{Hx}	[N/m]	Stiffness in horizontal translation along the x axis
K_{Hy}	[N/m]	Stiffness in horizontal translation along the y axis
K_V	[N/m]	Stiffness in vertical translation along the z axis
K_{Mx}	[N.m/rad]	Stiffness in rotation around the x axis
K_{My}	[N.m/rad]	Stiffness in rotation around the y axis
K_T	[N.m/rad]	Stiffness in torsion
$K_{M,0}$	[N.m/rad]	Stiffness in rotation of a shallow foundation in the absence of gapping
l_0	[m]	Transfer length (pile)
L	[m]	Length of a rectangular foundation
L' or L_{eff}	[m]	Effective length of a foundation
m	[-]	Exponent when expressing ultimate friction at the grout-rock interface
LI	[-]	Liquidity index
m	[-]	Exponent of the power function in the expression of the pile displacement under cyclic lateral loadings
m_c	[-]	Mass factor on resistances (penetrometer data)
m_l	[-]	Mass factor on resistances (laboratory data)
m_p	[-]	Mass factor on resistances (pressuremeter data)
M	[Pa]	Constrained (oedometric) modulus
M	[N.m]	Overturning moment
M_u	[N.m]	Ultimate moment leading to overturning
M_B	[N.m/rad]	Rotational resistance at the base of the pile
n	[-]	Porosity
N	[-]	Number of cycles
N_c	[-]	Correlation coefficient between q_c and S_u
N_{eq}	[-]	Equivalent number of cycles determined by applying Miner's rule
N_f	[-]	Number of cycles to reach a failure criterion

Script	Unit	Description
N_k	[-]	Correlation coefficient between q_n and S_u
OCR	[-]	Overconsolidation ratio [OCR= σ'_p /σ'_{vo}]
p	[Pa]	Lateral pressure within a p-y analysis
p_l*	[Pa]	Net limit pressure obtained from a MENARD pressuremeter test
p_{ult}	[Pa]	Ultimate lateral pressure
p(z)	[Pa]	Lateral pressure of the ground at depth z
p'	[Pa]	Effective isotropic stress (average) [= (σ'_1 + 2σ'_3)/3] ou [= (σ'_1 + σ'_2 + σ'_3)/3]
p'_g	[Pa]	Effective pressure applied by a grout column before setting
P_{ult}	[N/m]	Ultimate lateral reaction that can be mobilised
P(z)	[N/m]	Lateral reaction of the ground at depth z [P(z) = B . p(z)]
q	[Pa]	Mobilised stress under pile toe according to NF P 94-262 (2012)
q	[Pa]	Main deviatoric stress [= σ'_1 - σ'_3] or [= σ_1 - σ_3]
q_c	[Pa]	Cone resistance measured with a CPT
q_n	[Pa]	Net cone resistance (CPT)
q_p	[Pa]	Stress that can be mobilised under pile toe
$q_{p\ lim}$	[Pa]	Limit stress that can be mobilised under pile toe
q_t	[Pa]	Corrected cone resistance (CPT)
Q	[N]	Load (axial or lateral)
Q_c	[N]	Cyclic component or half-amplitude of the cyclic load
Q_m	[N]	Mean load or mean component of the cyclic load
Q_{max}	[N]	Maximum cyclic load
Q_{min}	[N]	Minimum cyclic load
Q_p	[m]	Reversible displacement (« quake ») at pile toe during driving
Q_s	[m]	Reversible local displacement (« quake ») on pile wall during driving
Q_t	[-]	Normalised cone resistance [= q_n /σ'_{vo}]
R	[m]	Radius of a pile, or of a circular foundation (R= B/2)
R_d	[N]	Design resistance
R_{dy}	[N]	Dynamic resistance developed during pile driving
R_{kc}	[N]	Characteristic cyclic resistance for a given event (static resistance degraded by cycles)
R_{ks}	[N]	Characteristic static resistance
R_{ks1}	[N]	Characteristic static resistance obtained from the axial capacity calculation
R_{ks2}	[N]	Characteristic static resistance obtained from the axial displacement calculation
R_{st}	[N]	Static ground resistance (obtained from SRD) on the ground-pile interface
s	[m]	Axial displacement (vertical) of a pile at a given depth, according to NF P 94-262 (2012)
S_d	[N]	Design load
s_u	[Pa]	Undrained shear strength
s_u^{DSS}	[Pa]	Undrained shear strength obtained from direct simple shear testing
s_u^C	[Pa]	Undrained shear strength obtained from triaxial testing in compression
s_u^E	[Pa]	Undrained shear strength obtained from triaxial testing in extension
SRD	[N]	Static component of the resistance to driving

Script	Unit	Description
t	[s]	Time
t_{max}	[Pa]	Maximum friction value in a t-z load transfer analysis
$t(z)$	[Pa]	Axial unit skin friction at depth z
T	[s]	Period of cycles
T	[N.m]	Moment of torsion
T_b	[N]	Shear strength at the base of a pile
$T(z)$	[N/m]	Axial friction per unit length at depth z [$T(z) = \pi . D . t(z)$]
u	[Pa]	Pore pressure
u_{cy}	[Pa]	Cyclic pore pressure
u_p	[Pa]	Permanent pore pressure
u_m	[Pa]	Mean pore pressure
U	[-]	Degree of consolidation
v	[m/s]	Particle displacement velocity of a pile component during driving
V	[N]	Vertical load (axial)
V_F	[m/s]	Propagation velocity of compression waves within a bedrock
V_L	[m/s]	Propagation velocity of compression waves within an intact rock sample, in laboratory
V_p	[m/s]	Propagation velocity of compression waves
V_s	[m/s]	Propagation velocity of shear waves
w	[-]	Moisture content
w_L	[-]	Liquid limit
w_P	[-]	Plastic limit
X_d	[N]	Design static resistance of the ground material
X_{kc}	[N]	Characteristic cyclic resistance of the ground material
X_{ks}	[N]	Characteristic static resistance of the ground material
y	[m]	Lateral local displacement of a pile
y_c	[m]	Limit value of the lateral local displacement in a p-y analysis
y_1	[m]	Displacement of the pile toe during the first load under H_{max}
y_N	[m]	Displacement of the pile toe at the N^{th} cycle under H_{max}
z	[m]	Depth under seabed
z	[m]	Axial local displacement (vertical) of a pile at a given depth in a t-z transfer analysis
z_0	[m]	Reference depth when estimating the initial stiffness of a p-y curve

GREEK SCRIPTS

Script	Unit	Description
α	[-]	Adhesion factor at grout-rock interface
β	[-]	Reduction factor (function of the j_m mass factor) at grout-rock interface
β	[-]	Hysteretic damping coefficient
ε	[-]	Strain
γ_h	[kN/m^3]	Wet unit weight of a soil
γ_d	[kN/m^3]	Dry unit weight of a soil
γ_s	[kN/m^3]	Unit weight of solid particles
γ_w	[kN/m$_3$]	Unit weight of water
γ'	[kN/m^3]	Submerged unit weight of a soil
γ	[-]	Shear strain - Distortion
γ_m	[-]	Mean shear strain - Mean distortion
γ_{cy}	[-]	Cyclic shear strain - Cyclic distortion
γ_F	[-]	Partial factor on loads
γ_M	[-]	Partial factor on materials
γ_R	[-]	Partial factor on resistances
δ	[°]	Friction angle at soil-pile or soil-grout interface
δ_{cv}	[°]	Friction angle at constant volume at soil-pile interface
δ_r	[°]	Residual friction angle at soil-pile or soil-skirt interface
ε_1, ε_2, ε_3	[-]	Strains in principal plane
ε_v	[-]	Volumetric strain
ε_{50}	[-]	Strain at 50% of ultimate strength
θ	[rad]	Rotation of the pile or of the foundation under lateral loading
θ_0	[rad]	Reference rotation = rotation of a shallow foundation at initiation of gapping
v	[-]	Poisson's coefficient
ρ	[kg/m^3]	Unit mass of soil
ρ_d	[kg/m^3]	Dry unit mass of soil
ρ_s	[kg/m^3]	Unit mass of particles
ρ_w	[kg/m^3]	Unit mass of water
σ	[Pa]	Total stress
σ_c	[Pa]	Unconfined compressive strength
σ_{ho}	[Pa]	Total horizontal stress due to overburden
σ_{vo}	[Pa]	Total vertical stress due to overburden
σ'	[Pa]	Intergranular effective stress
σ'_h	[Pa]	Effective horizontal stress
σ'_{ho}	[Pa]	In-situ effective horizontal stress
σ'_p	[Pa]	Preconsolidation pressure
σ'_v	[Pa]	Effective vertical stress
σ'_{vo}	[Pa]	Effective vertical stress due to overburden

Script	Unit	Description
σ'_{vc}	[Pa]	Effective vertical consolidation stress
$\sigma_1, \sigma_2, \sigma_3$	[Pa]	Principal stresses
$\sigma'_1, \sigma'_2, \sigma'_3$	[Pa]	Effective principal stresses
τ	[Pa]	Shear stress
τ	[Pa]	Axial unit skin friction mobilised at ground-pile interface according to NF P 94-262 (2012)
τ_B	[-]	Shear stress under the base of a pile
τ_0	[Pa]	Initial shear stress of the ground prior to installing the structure
τ_{us}	[kPa]	Ultimate shear stress under a monotonous/static loading
τ_m	[Pa]	Mean shear stress
τ_{cy}	[Pa]	Cyclic shear stress
$\tau_{f,\ cy}$	[Pa]	Cyclic undrained shear strength
$\tau_{s\ ult}$	[Pa]	Ultimate unit skin friction at ground-grout interface
τ_{sf}	[Pa]	Average axial unit skin friction over the height of the shaft
φ'	[°]	Effective friction angle
φ'_{cv}	[°]	Friction angle at constant volume of the ground
Δu	[Pa]	Additional pore pressure
ΔH	[N]	Additional lateral load
$\Delta\gamma_m$	[-]	Additional shear strain (distortion)
$\Delta\tau_m$	[Pa]	Additional mean shear stress

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