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Study and application of anchor systems into a benthic lander

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Abstract

This dissertation has the objective of developing an improved solution for the foundation of the benthic lander owned by INEGI. The current foundation comprises concrete ballast weights, which due to their weight can keep the structure immobile. However, this solution is limited since it must be detached from the lander at the decommissioning stage. After reviewing the different possibilities used in the offshore industry, the suction bucket foundation was the only one that fulfilled all the project requirements. The project requirements were: to have good installation precision, not need human labour during installation, to have an easy decommissioning procedure, and to be reusable.

The suction bucket foundation design is considered an interdisciplinary process since it involves mechanical and geotechnical project. It involved two main stages: determination of the structural loading regime and the verification of the installation process.

In the first stage, the loads were calculated for the lander's position in which the most demanding forces occur relative to the sea water current. It was verified that, in this position, the higher load regime occurs in the downstream suction buckets. Therefore, all the suction buckets were designed for this critical load. A conservative design method was used to ensure the structural integrity of the final bucket design.

In the second stage, the installation was evaluated. Since the suction bucket is embedded in the sand, its dimensions will also impact the soil capacity to failure. An iterative design method was used to achieve a possible design solution. It was observed that increasing the diameter, D , and the thickness, t , of the buckets also increases soil capacity. However, increasing the bucket length, L , could lead to tension forces in the sand to which it has no resistance.

The dimensions $D = 0.9$ m, $L = 1$ m, and $t = 0.030$ m were chosen for the bucket final design. For these dimensions, the length of the bucket can be totally embedded during suction installation.

These design dimensions were confirmed by FEM analysis, since no considerable deformation was detected or excessive stress, guarantying the lander's stability. Furthermore, the maximum stresses calculated during the simulation allowed to select a material for the suction buckets. The steel ASTM A36 was the material chosen resulting in a safety factor, SF , equal to 694. For this application, the safety factor, SF , is very high since a structural steel was chosen according to offshore wind turbine standard codes for a loading regime considerably lower than the ones registered in this industry.

Keywords: Offshore structures, Offshore foundations, Suction bucket, Wave and current loads, Sand, Mechanism-based methods, Bearing capacity, FEM analysis

Resumo

A presente dissertação tem como objetivo apresentar uma solução melhorada para a fundação do *lander* do INEGI. A solução atual consiste em lastros de betão que, devido ao seu peso, mantêm o *lander* imóvel no fundo do oceano. No entanto, esta solução apresenta limitações, uma vez que os lastros precisam de ser separados da estrutura aquando do processo de recuperação do *lander*. Após ponderar as diferentes soluções utilizadas na indústria *offshore*, os baldes de sucção são a única opção que cumpre na totalidade os requisitos do projeto. Os requisitos do projeto são: ter uma boa precisão de instalação, não necessitarem de mão de obra humana durante a instalação, ter um processo de recuperação simples e serem reutilizáveis.

O dimensionamento dos baldes de sucção é considerado um processo interdisciplinar, uma vez que envolve uma componente de projeto mecânico e de geotecnia. O seu dimensionamento envolveu duas etapas principais: a determinação do regime de carregamento e a verificação da instalação.

Na primeira etapa, calcularam-se as cargas para a posição do *lander* na qual atuam as forças de maior magnitude relativamente à corrente oceânica. Verificou-se que para esta posição, o carregamento crítico ocorre nos baldes mais a jusante do escoamento. Desta forma, todos os baldes foram dimensionados para a carga mais crítica. Um método de dimensionamento conservativo foi utilizado para garantir a integridade estrutural do balde.

Na segunda etapa, a instalação foi avaliada. Uma vez que os baldes de sucção se encontram embutidos na areia, as suas dimensões irão também influenciar a resistência do solo. Recorreu-se a um método iterativo para encontrar possíveis dimensões para o balde. Observou-se que um aumento do diâmetro, D , e da espessura, t , aumentavam a resistência do solo. No entanto, o aumento do comprimento do balde, L , poderia originar o aparecimento de forças de tração na areia, para as quais a areia não apresenta qualquer tipo de resistência. As dimensões $D = 0.9$ m, $L = 1$ m e $t = 0.030$ m foram escolhidas como sendo as dimensões finais para os baldes de sucção. Com estas dimensões, o balde pode ser totalmente embutido na areia recorrendo à instalação por sucção.

As dimensões do balde foram confirmadas através de análises por elementos finitos. Uma vez que nenhuma deformação considerável nem tensão excessiva foram detetadas, considerou-se a estabilidade dos baldes como garantida. Para além do mais, as tensões máximas calculadas durante as simulações permitiram selecionar um material para os baldes de sucção. O aço ASTM A36 foi escolhido resultando num fator de segurança, SF, igual a 694. Para esta aplicação, o valor do fator de segurança, SF, é muito elevado uma vez que foi escolhido um aço estrutural, de acordo com as normas usadas em turbinas eólicas *offshore*, para um carregamento consideravelmente inferior às solicitações verificadas nesta indústria.

Palavras-chave: Estruturas *offshore*, Fundações *offshore*, Baldes de sucção, Forças de ondulação e de corrente, Areia, Métodos *Mechanism-based*, Capacidade de suporte, Análise por elementos finitos

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João Pedro Alves

À memória da minha avó Joaquina

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List of abbreviations and symbols

Abbreviations

ALS	Accident Limit State
AMALIA	Algae-to-Market Lab IdeAs
API	American Petroleum Institute
ASTM	American Society for Testing and Materials
DNVGL	Det Norske Veritas and Germanischer Lloyd
ECMWF	European Centre for Medium-Range Weather Forecasts
EEZ	Exclusive Economic Zone
FEM	Finite Element Method
FLS	Fatigue Limit State
GBS	Gravity-based structure
IH	Hydrographic Institute of Portugal
INEGI	Institute of Science and Innovation in Mechanical and Industrial Engineering
IPMA	Portuguese Institute for Sea and Atmosphere
ISO	International Organization for Standardization
LAM	European Shelf Model
LRFD	Load and Resistance Factor Design
LRP	Load Reference Point
MATLAB	MATrix LABoratory
NASA	National Aeronautics and Space Administration
OWT	Offshore wind turbine
POM	Polyoxymethylene
ROV	Remotely operated vehicle
SBM	Single buoy mooring
SLS	Serviceability Limit State
SWAN	Simulating WAVes Nearshore
TLP	Tension Leg Platform
ULS	Ultimate Limit State
UNESCO	United Nations Educational, Scientific and Cultural Organization
WAM	WAve Model

General Notation

α, A	Scalar (italic)
$\mathbf{a}$	Vector (bold)

Subscripts

f	Failure
max	Maximum
min	Minimum

Latin Symbols

a	Ratio of excess pore pressure at tip of caisson skirt to beneath caisson base
A	Projected area
A_C	Wave crest height
$A_{caisson}$	Internal top area of suction bucket
A_{eff}	Effective area
A_T	Wave trough depth
B_e	Intermediate variable
B_{eff}	Effective width
B_H	Horizontal boundary extension
B_V	Vertical boundary extension
c	Wave phase velocity
c'	Effective peak cohesion of soil
c'_{cv}	Effective constant volume cohesion of soil
$\mathbf{C}$	Unit vector along the cylinder axis directed up or down
c_0, c_1, c_2	Constants
C_D	Drag coefficient
C_M	Inertia coefficient
d	Water depth
d_o	Reference depth
d_q, d_γ	Depth factors
D	Diameter or projected cross-sectional dimension
D_i	Inner diameter of suction bucket
D_o	Outer diameter of suction bucket
e	Eccentricity
e_v	Void ratio
E	Young's modulus
f	Force per unit length
f_D	Drag force component per unit length
f_I	Inertia force component per unit length
F_A^x	Design load in point A in x direction
F_A^y	Design load in point A in y direction
F_b	Buoyancy force
F_B^x	Design load in point B in x direction
F_B^y	Design load in point B in y direction
F_c	Total current force
F_d	Design load
F_g	Gravity force
F_k	Characteristic load

F_x	Force in x direction
F_y	Force in y direction
F_z	Force in z direction
g	Gravity acceleration
G	Toughness
G_s	Specific gravity of solid particles
h	Embedment depth
h_r	Roughness height
h_{side}	Embedment depth at which H_{side} is applied
H	Wave height
H_{base}	Horizontal load at the bucket base
H_c	Critical horizontal design load
H_d	Horizontal design load
H_{max}	Maximum wave height
H_{LRP}	Horizontal force at the load reference point
H_{side}	Horizontal load at the bucket side
i_q, i_γ	Load inclination factors
$\mathbf{i}$	Unit vector along x axis
$\mathbf{j}$	Unit vector along y axis
k	Wave number
k_f	Soil permeability ratio
k_i	Inner soil permeability
k_o	Outer soil permeability
k_s	Soil permeability
$\mathbf{k}$	Unit vector along z axis
K	Factor relating vertical stress to horizontal stress
K_0	Resting impulse coefficient
KC	Keulegan carpenter number
l	Length of cylinder
L	Length of suction bucket
L_e	Intermediate variable
L_{eff}	Effective length
m	Mass
M_{base}	Moment at the bucket base
M_{LRP}	Moment at the load reference point
M_x	Moment in x direction
M_y	Moment in y direction
M_z	Moment in z direction
n	Porosity
N	Order of the wave theory
N_q	Bearing capacity factor (overburden)
N_γ	Bearing capacity factor (self-weight)
N_z	Normal component of soil transmitted force
N'_z	Portion of the total normal force transmitted due to particle contacts
p_a	Atmospheric pressure
r	Radius of the circle that contains the suction bucket centres of gravity
R	Soil resistance acting on the suction bucket
R_A^x	Reaction in point A in x direction
R_A^y	Reaction in point A in y direction

R_B^x	Reaction in point B in x direction
R_B^y	Reaction in point B in y direction
Re	Reynolds number
s	Pressure inside the bucket with respect to the ambient seabed water pressure
s_q, s_γ	Shape factors
S	Wave steepness parameter
S_d	Design load effect
S_k	Characteristic load effect
S_r	Degree of saturation
SF	Safety factor
t	Suction bucket skirt wall thickness
T	Wave period
T_{min}	Minimum temperature
$T_{sea\ water}$	Temperature of the sea water
T_z	Tangential component of soil transmitted force
T_{zx}	Tangential component of soil transmitted force projected in x-axis
T_{zy}	Tangential component of soil transmitted force projected in y-axis
u	Horizontal water particle velocity
u_0	Pore pressure at the ground surface
u_c	Horizontal current velocity
$u_{c,tide}$	Tidal current velocity
$u_{c,wind}$	Wind generated current velocity
u_w	Horizontal wave velocity
U	Transmitted force through the pore water
U_R	Ursell number
v	Vertical water particle velocity
v_m	Maximum orbital particle velocity
v_t	Total flow velocity
V	Volume
V'	Effective vertical load
V_a	Volume of gaseous phase
V_{base}	Vertical load at the bucket base
V_c	Critical vertical design load
V_d	Vertical design force
V_{LRP}	Vertical force at the load reference point
V_s	Volume of solid phase
V_{side}	Vertical load at the bucket side
V_{ult}	Ultimate soil bearing capacity
V_v	Void volume
V_w	Volume of liquid phase
w	Water content
$\mathbf{w}$	Normal velocity vector
$\dot{\mathbf{w}}$	Normal acceleration vector
W_a	Weight of the gaseous phase
$W_{caisson}$	Weight of the suction bucket
W_s	Weight of the solid phase
W_w	Weight of the liquid phase
y	Distance from still water level
$y_{cg,lander}$	Centre of gravity of the lander's structure
Z_i	Inner stress enhancement parameter
Z_o	Outer stress enhancement parameter

Greek Symbols

α	Exponent
β	Direction of wave propagation measured from the positive x-axis
γ	Unit weight
γ'	Submerged unit weight
γ_d	Dry unit weight
γ_f	Safety factor
γ_s	Solid particles unit weight
δ	Angle of friction between the bucket wall and the soil
Δ	Deformation
Δ_r	Non-dimensional roughness
ζ	Angle of the cylinder axis to the vertical axis
η	Wave surface elevation
Θ	Wave phase
λ	Wavelength
μ	Shallow wave parameter
μ_s	Static friction coefficient
ν	Kinematic viscosity
ν_p	Poisson's ratio
ξ	Angle of the cylinder projection to the x-axis
ρ	Density
σ	Total stress
σ'	Effective total stress
σ_{end}	End bearing stress
σ_{h0}	Total horizontal geostatic stress
σ_{v0}	Total vertical geostatic stress
$\sigma_{VM,max}$	Maximum Von-Mises equivalent stress
σ_y	Yield strength
$\sigma_{1,max}$	Maximum principal stress
τ	Total shear stress
τ'	Effective total shear stress
ϕ	Peak angle of friction of the soil
ϕ'	Effective peak angle of friction of the soil
ϕ_{cv}	Constant volume angle of friction of the soil
ϕ'_{cv}	Effective constant volume angle of friction of the soil
ψ	Dilantancy angle
ω	Wave angular frequency

Chapter 1

Introduction

This chapter approaches the necessity of exploring the ocean and the lack of knowledge that humankind has about it to try to explain the work that must be done globally and in particular in each country. In this case Portugal was used as an example. Finally, the objectives and the structure of this thesis close the chapter.

1.1 Ocean exploration

Ocean exploration concerns the investigation and description of the ocean waters and the seabed. It plays a crucial role in ensuring that ocean resources are not only maintained but also managed sustainably so that they will be available for use by future generations.

It might be surprising but, according to UNESCO (United Nations Educational, Scientific and Cultural Organization), only 5% of the ocean has been explored and charted by humans [1]. Currently, humankind knows more about the moon and Mars, which have both been mapped completely, than Earth's oceans [2].

Our knowledge about the ocean is still limited, despite the fact that it covers more than 70% of the Earth's surface and is essential to maintaining life on our planet, including the air we breathe, the food we consume, weather and climate patterns.

However, scientists and engineers from all over the world have come together to make the entire ocean mapping a reality by 2030 with the "The Nippon Foundation-GEBCO Seabed 2030 Project" [3].

Figure 1.1 shows the most recent map of the seabed coverage within this project.

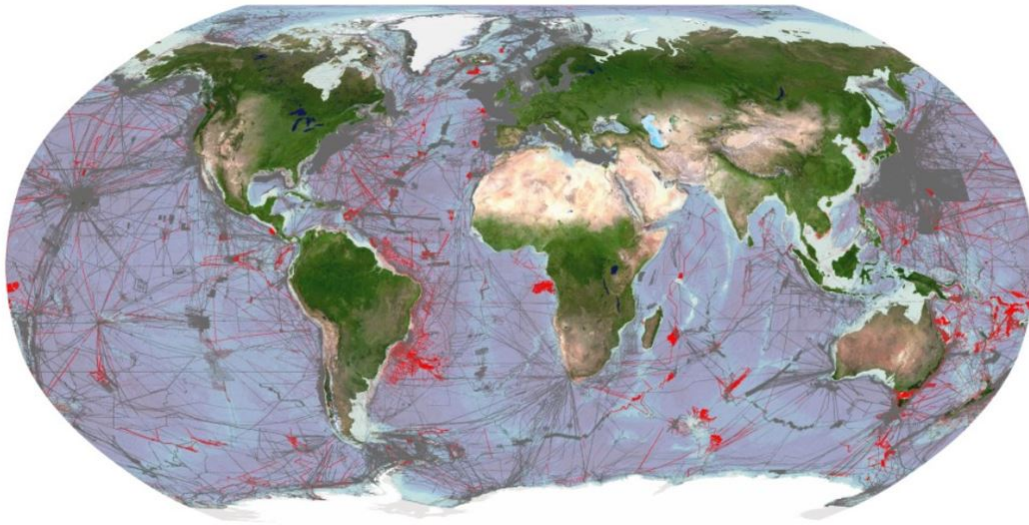


Figure 1.1 – Map of the seabed coverage within the Seabed 2030 Project. At grey colour the mapped area in the 2021 release. At red colour the additional coverage included in 2022 [3].

1.1.1 Portugal's strategic plan for sea

During the Portuguese discoveries, it was shown to the world that the unknowing sea was in fact a space of endless opportunities.

Due to the vast potential of sea biodiversity, geological resources, and minerals, Portugal saw in 2009 a chance to propose the expansion of its Exclusive Economic Zone (EEZ), as shown in Figure 1.2 [4].

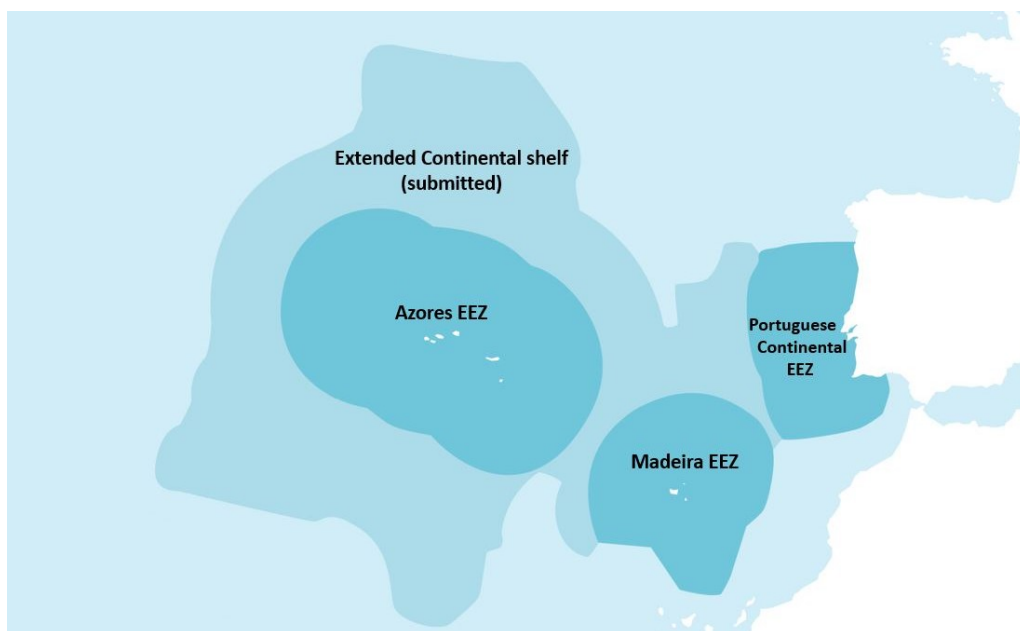


Figure 1.2 – Portugal's EEZ expansion proposal [4].

Portugal's land extension represents about 92 000 km². However, if its maritime dimension is considered (with the proposed extension), Portugal will have a total of 4 million km², meaning that its maritime area will be 40 times the land dimension. Although Portugal is still waiting for a decision of the United Nations about the extension of its continental shelf, several national strategic plans for the sea were developed to make the most out of the sea resources.

The most recent plan is called "Estratégia Nacional para o Mar 2021-2030" and has 10 main objectives. The first one is to "Fight the climate changes and the pollution and protect and restore the ecosystems" [5], as it will be seen in the next chapter is one of the lander's objective.

1.2 Objectives

The present thesis was proposed by INEGI - Institute of Science and Innovation in Mechanical and Industrial Engineering which had been developing novel solutions for ocean exploration. The most recent technology developed in the Institute is a benthic lander that, when equipped with a camera, could detect invasive macroalgae throughout the Iberian Coast, helping in the protection of the ecosystems (the first objective of the Portuguese plan for the sea).

The foundation type used to fix the lander to the seabed consists in concrete ballast weights, which due to their weight kept the structure immobile. However, when the lander was no longer needed at the site of installation, a diver was necessary to decouple the structure from ballast weights to recover it. Since recovering the lander was a demanding job, INEGI proposed the study and design of a novel foundation that could fix the structure to the seabed and fulfill the following requirements:

- Have a good installation precision;
- Do not need human labour during installation (for example, be ROV operated);
- Have an easy decommissioning procedure;
- Be reusable.

The objective of this thesis can be illustrated by Figure 1.3.

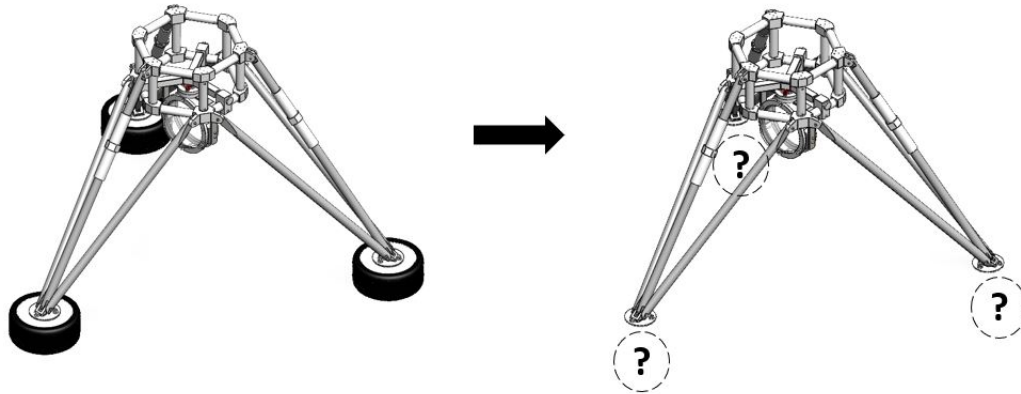


Figure 1.3 – Objective of this thesis - design of a novel lander's foundation.

More information about the lander project can be found in section 2.3.

1.3 Scope

The present dissertation is divided into six chapters.

Chapter 2 gathers information about the classification of offshore structures and materials. A brief introduction to the lander project is made.

Chapter 3 begins with an alternative concept for the lander's foundation. The structural loading is then calculated using DNVGL standard codes as reference.

Chapter 4 presents suction bucket and soil terminology for a better understanding of the design procedures. The suction bucket design is shown for the critical procedure stages and the final dimensions are obtained using an iterative method.

Chapter 5 shows a Finite Element Method (FEM) analysis of the stresses that the suction buckets undergo during its service period. The maximum stress values obtained in the analysis are then used to select a material for the suction buckets.

Chapter 6 closes this work with final comments and proposals for future works.

Chapter 2

Literature Review

This chapter aims to gather information about the classification of offshore structures. Therefore, it will be possible to classify the benthic lander and use the appropriate standard code to guide the design process. Additionally, information about the different types and materials of foundations was collected to choose an alternative to the one currently in use.

2.1 Introduction

Offshore structures/platforms are structures designed to work in sea environments. These structures are usually fabricated and assembled onshore, transported to an offshore site and then installed. Hence, the construction methods, materials, and design tolerances differ substantially from onshore to offshore structures.

Offshore structures are used in two main industries: oil and gas and offshore wind energy. Thereby, plenty of different standard codes are used to design offshore structures, depending on the industry. For example, the DNVGL (Det Norske Veritas (DNV) and Germanischer Lloyd (GL)) standard codes focus more in the offshore wind energy industry while API (American Petroleum Institute) standards focus more in the oil and gas industry.

2.2 Offshore Structures

Offshore structures can be classified as fixed or floating platforms regarding its method of connection to the sea bed. However, these definitions may vary between standard defining codes. Thus, ISO (International Organization for Standardization), DNVGL, and API standards definitions were compared.

Conforming to ISO Standards (ISO 19902:2007 and ISO 19904-1:2006) [6, 7],

Fixed structure is a structure that is bottom founded and transfers all actions on it to the sea floor.

Floating structure is a structure where the full weight is supported by buoyancy.

According to DNVGL Standard (GL IV-6-1) [8],

Fixed installations are installations permanently fixed by piling (pile foundation); or resting on the sea bed by action of gravity (gravity foundation); or that, with their excess of buoyancy, can connect to a base by tensioned anchoring elements (tension leg foundation).

Floating units are units connected to the sea bed by anchoring (mooring); or kept on position by dynamic positioning/propelling system and temporarily connected to sea by legs in jacked up condition (self elevating units).

As stated in API Standard (API RP 2A-WSD and API RP 2FPS) [9, 10],

Fixed platform is a platform extending above the water level and supported by the sea bed by means of piling, spread footings or other means with the intended purpose of remaining stationary over an extended period.

Floating platform consists in a buoyant hull, deck structure and station keeping system.

Since there is not an unanimous definition for this type of classification, some structures may be classified as fixed or floating depending on author's point of view. For example, Tension Leg Platforms (TLP) are considered by James F. Wilson [11] and Subrata Chakrabarti [12] as floating structures and by Mohamed A. El-Reedy [13] as fixed platforms.

Therefore, another term is frequently introduced in the literature to distinguish the different types of offshore structures. Beyond fixed and floating, the structure can also be classified as compliant; compliant means flexible [12, 14]. A compliant structure is a structure that extends to the ocean bottom and is directly attached to the seafloor by means of tension legs, piles, guidelines, flexible members or articulated joints.

For a more complete classification offshore platforms are divided in fixed, compliant and floating structures, as shown in Figure 2.1.

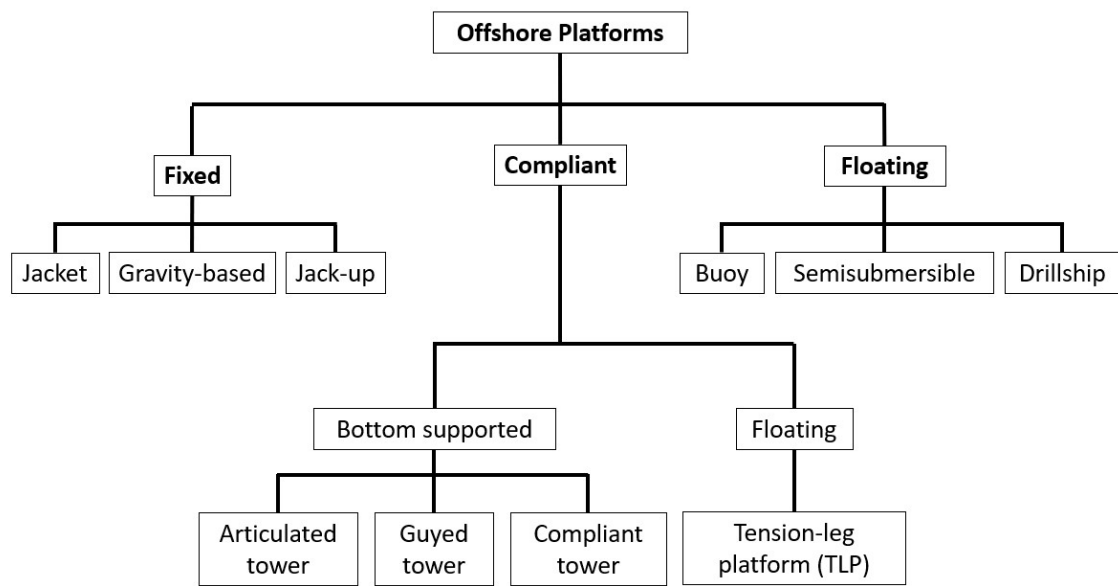


Figure 2.1 – Types of offshore platforms.

2.2.1 Fixed Structures

The following sections succinctly describe each type of fixed structures: jacket platform, jack-up platform, and gravity-based structure (GBS).

2.2.1.1 Jacket platform

Jacket structures, sometimes known as templates, are three-dimensional space frames composed of welded tube elements (legs and braces). Typically, the legs are inclined to improve stability under environmental stresses and associated overturning moments. The structure is fixed to the seafloor by piles that penetrate the soil. The piles are also tubular and are driven into the seafloor through the legs of the jacket. Typically, these platforms support a superstructure including drilling, production equipment, production equipment, and workover rigs. [15].

The use of these platforms has generally been limited to a water depth of 150-180 m in the harsh North Sea environment. In the more intermediate Gulf of Mexico environment some jacket platforms have been installed in deeper waters, having reached a depth of approximately 400 m [12].

A schematic and real representations of a jacket platform are shown in Figure 2.2.

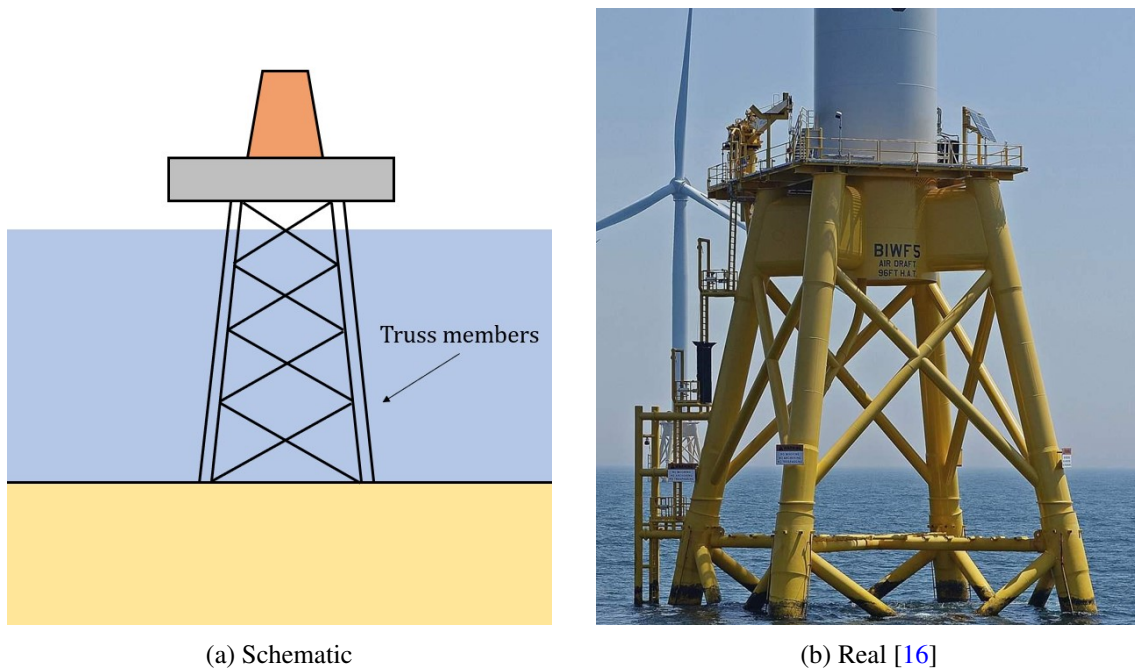


Figure 2.2 – Jacket platform.

2.2.1.2 Jack-up platform

Jack-up (self-elevating unit) platforms are typically three-legged having a deck supported on their legs, which are made of tubular truss members. The deck is buoyant. Jack-ups are utilized for exploratory drilling and are consequently built to move from location to location. The jack-up is towed while being supported by its own buoyancy. Once reaching the drilling location, the legs are lowered to the seabed and electric or hydraulic jacks are used to raise the deck above the water's surface. [17].

During operation, the structure behaves like a stationary platform. In transit, the legs stand well above the deck and the platform behaves like a floating platform.

Jack-up platforms are standard up to operating depths of 90 m; a few structures may operate to depths of 135 m in certain areas [14].

A schematic and real representations of a jack-up platform are shown in Figure 2.3.

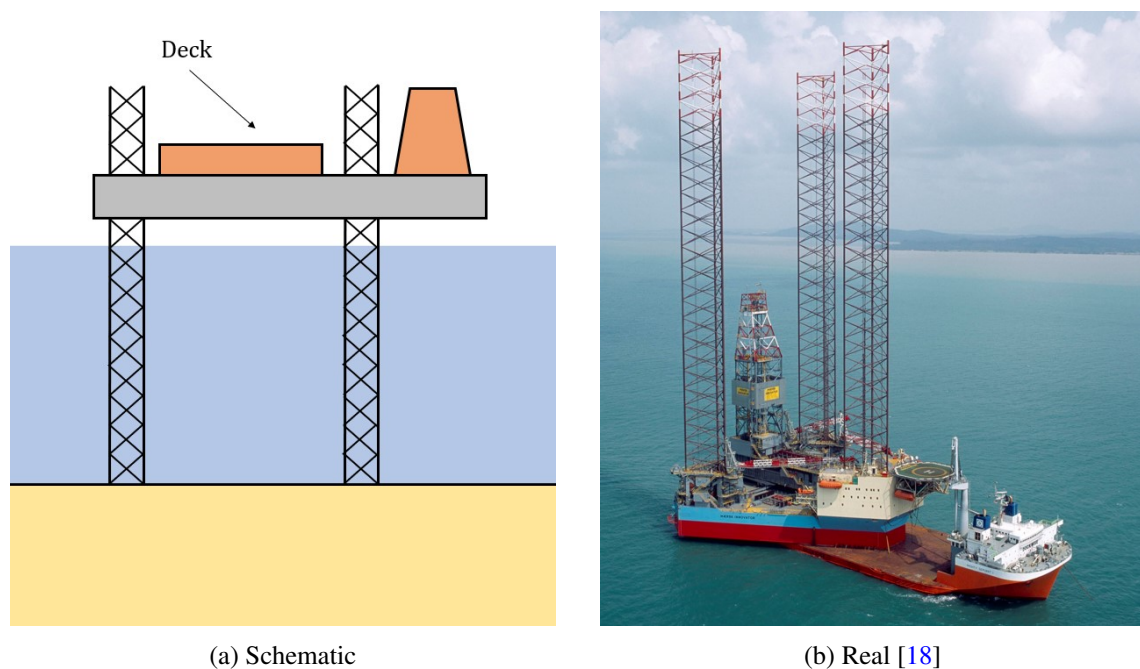


Figure 2.3 – Jack-up platform.

2.2.1.3 Gravity-based structure (GBS)

Gravity-based structures (GBSs) are placed on the sea bed and are held in place by their weight. No additional fixing is needed to hold GBSs in place. Upon construction they are towed in the upright position to the final destination and submerged. The deck (topside) can be also carried with the support structure. Since the gravity structures require large volume and high weight, concrete has been a common material for gravity structures. Increasing the size of the foundation also allows gravity-based structures to be made of steel. Unlike piling, the horizontal and vertical forces as well as the overturning moment are transferred directly to the seafloor. Therefore, the seafloor properties must be studied to confirm the possibility of an onsite GBS installation. GBSs are vulnerable to scour and sinkage.

This type of offshore structure is usually used in sheltered waters close to shore. These structures can be used up to depths of 330 m [19].

A schematic and real representations of a gravity-based structure are shown in Figure 2.4.

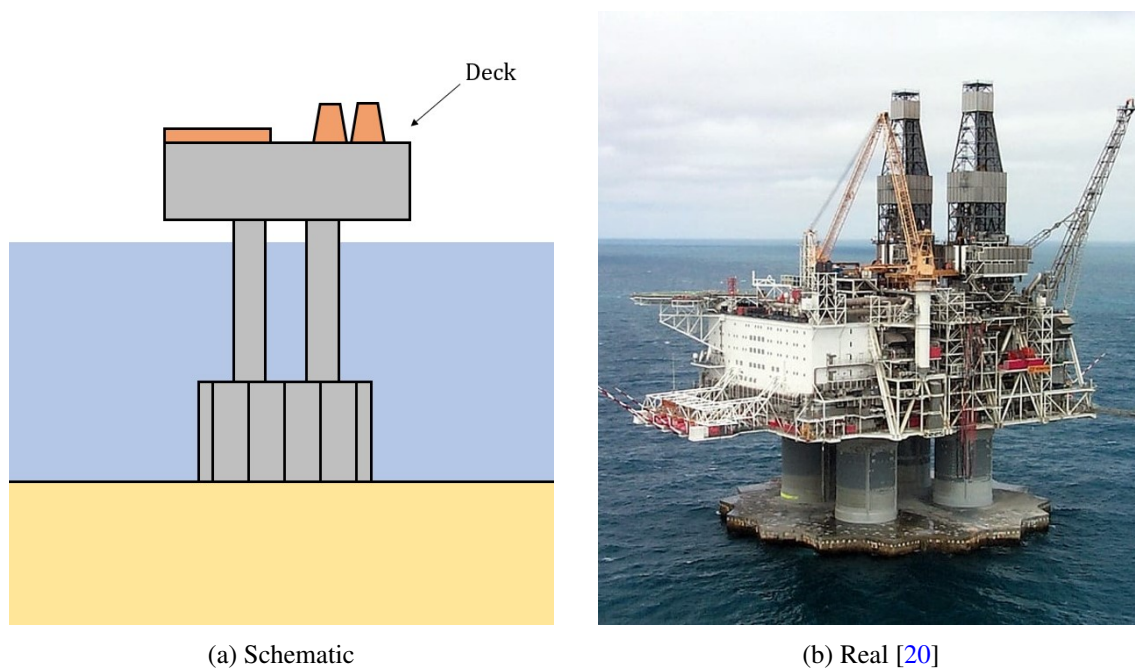


Figure 2.4 – Gravity-based structure.

2.2.2 Floating Structures

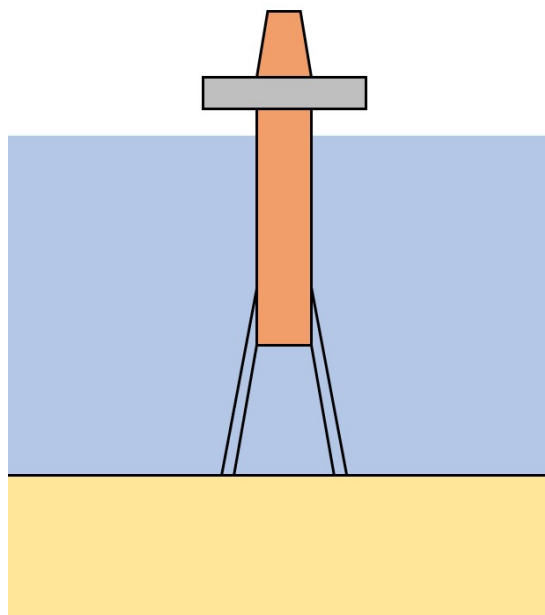
The following sections succinctly describe each type of floating structures: loading buoy and floating storage tank, drillship, and semi-submersible.

2.2.2.1 Loading Buoy and Floating Storage Tank

Single buoy moorings (SBMs) are utilized for anchoring tankers and transferring oil and gas from the production platform. The buoy has a round form and is made entirely of welds. Typically, they are secured in radial spread moorings with individual chains under tension, so that the buoy's restoring force increases linearly with displacement from its equilibrium position. SBMs are designed to be self-sufficient units with living quarters and a helicopter deck for use in particularly rough seas. In some instances, they offer temporary oil storage capacity. [14].

The SPAR buoy, named because of its spar-type configuration, is a floating cylinder designed for drilling, production and storage. Its buoyancy is responsible for supporting facilities above water surface. The centre of mass is located much below the centre of buoyancy, which restores and stabilizes the platform. The spar relies on a traditional mooring system, i.e., multiple catenary, slack or taut mooring lines, to maintain its position [19].

A schematic and real representations of a SPAR buoy are shown in Figure 2.5.



(a) Schematic



(b) Real [21]

Figure 2.5 – Spar buoy.

2.2.2.2 Drillship

Drillships are vessels with a drilling shaft near the centre plane and an above derrick (lifting device). Typically, drilling vessels are moored at depths of up to 300 m using a spread mooring configuration. Thus, just a minor shift in the location with respect to wind and wave direction is observed. This can be reduced if the vessel is positioned in the direction of the waves. Similar advantages are provided by dynamic positioning, which is utilized at sea depths ranging from 300 to more than 2000 m. This system enables precise and automatic control of the vessel's position and heading by combining riser angle indicators, taut wires, and acoustic beacons with an automatic control propulsion system [14].

A schematic and real representations of a drillship are shown in Figure 2.6.

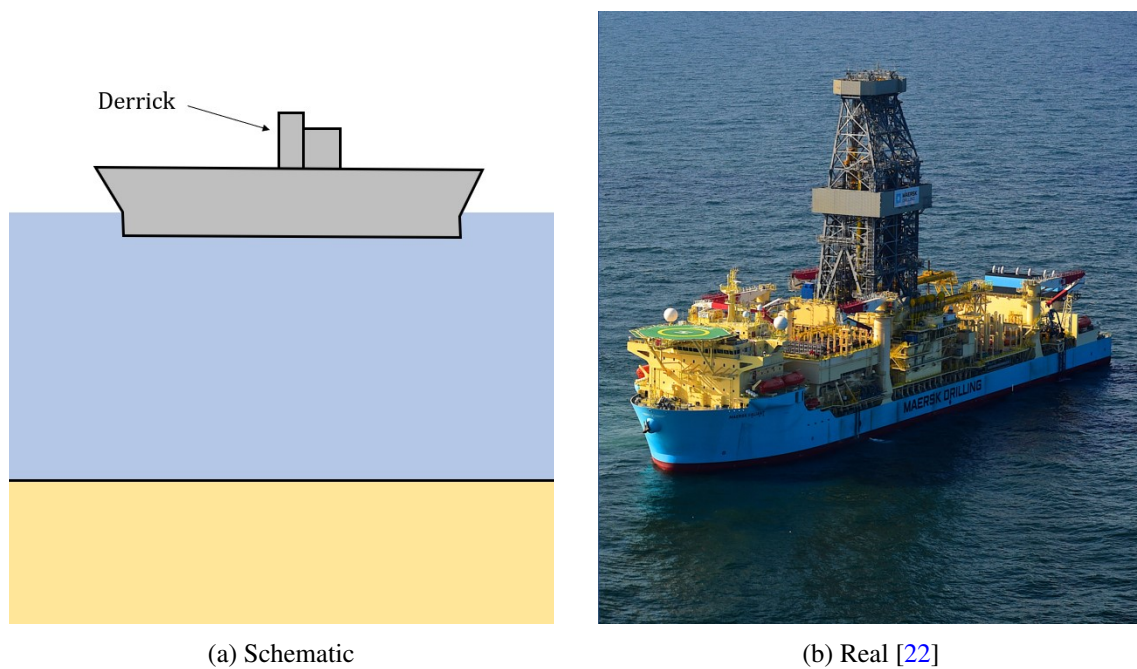


Figure 2.6 – Drillship.

2.2.2.3 Semi-submersible

Semi-submersibles are multi-legged floating structures with a large deck (topside). These legs are interconnected at the bottom underwater with horizontal buoyant members called pontoons [12]. When submerged for stationing and operations, the legs connecting the pontoons to the upper deck present a lower water plane area, thereby attracting less wave loads and reducing the motions [19].

Despite higher construction and operating costs in comparison with drillships or jack-up platforms, semisubmersibles are used in many offshore technology applications, such as exploratory drilling activities in areas of rough seas, crane and pipelaying operations, and as floating production systems [14]. These platforms are used in depths up to 2000 m [19].

A schematic and real representations of a semi-submersible are shown in Figure 2.7.

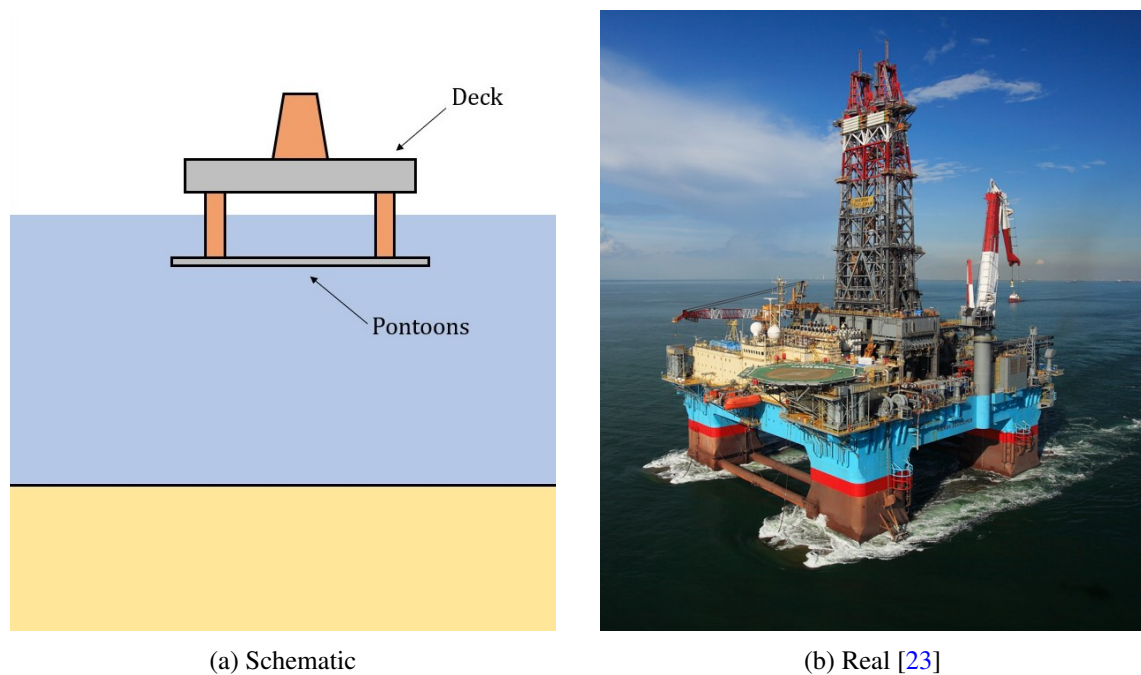


Figure 2.7 – Semi-Submersible.

2.2.3 Compliant Structures

The following sections succinctly describe each type of compliant structures: tension-leg platform (TLP), guyed tower, compliant tower, and articulated tower.

2.2.3.1 Tension-Leg Platform (TLP)

Tension-Leg Platform (TLP) is a flexible structure that is vertically anchored by taut mooring lines known as tendons (or tethers). The structure is vertically constrained, preventing vertical (heave) and rotational movement (pitch and roll). It is flexible in the horizontal direction, permitting lateral movement (surge and sway) [12].

TLP technology retains many of the benefits of fixed platforms while reducing production costs in water depths up to 1500 m. Its production and maintenance processes resemble those of stationary platforms. However, TLPs are sensitive to weight and may have restrictions on carrying big weights [14].

A schematic and real representations of a tension-leg platform are shown in Figure 2.8.

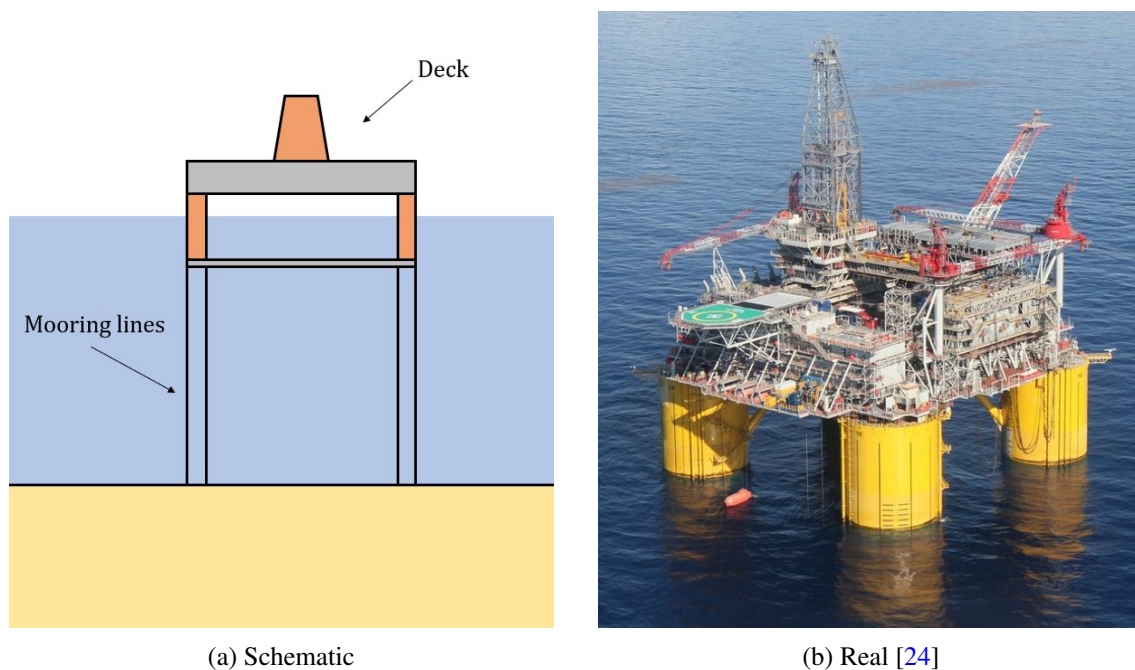
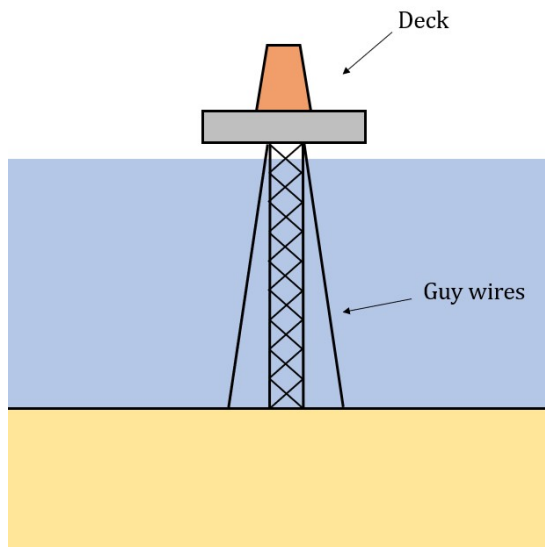


Figure 2.8 – Tension-Leg Platform.

2.2.3.2 Guyed Tower

A guyed tower is a thin structure composed of truss members that rests on the seafloor and is supported by symmetrical arrangements of catenary guy wires. In deep, unfavorable waters, where environmental loads on gravity base or jacket-type constructions are prohibitively high, a guyed tower may be an appropriate solution. In calm seas, the upper part is composed of a lead cable that works as a strong spring. The bottom part is made up of a hefty chain with clump weights that are raised off the bottom during high winds and act as a soft spring, making the tower more compliant [12].

A schematic and real representations of a guyed tower are shown in Figure 2.9.



(a) Schematic



(b) Real [25]

Figure 2.9 – Guyed Tower.

2.2.3.3 Compliant Tower

Compliant (Piled) Tower is a slender construction comparable to conventional platforms that goes from the surface to the ocean floor. However, instead of grouping piles near the centre of the tower, which offers very little resistance to overturning, they are shifted to the outside of the construction and increased in number. In deepwater design, these piles are continued in ungrouped guides to a point midway up the tower, where they are connected. Therefore, the piles can provide sufficient overturning resistance and preserve a sufficient axial "spring" characteristic [14].

In deeper water, weight savings from using compliant piled tower with fixed base structure overcomes pivot-related expenses and provide the possibility of expanding the concept's economic range over 900 m [14].

A schematic and real representations of a compliant tower are shown in Figure 2.10.

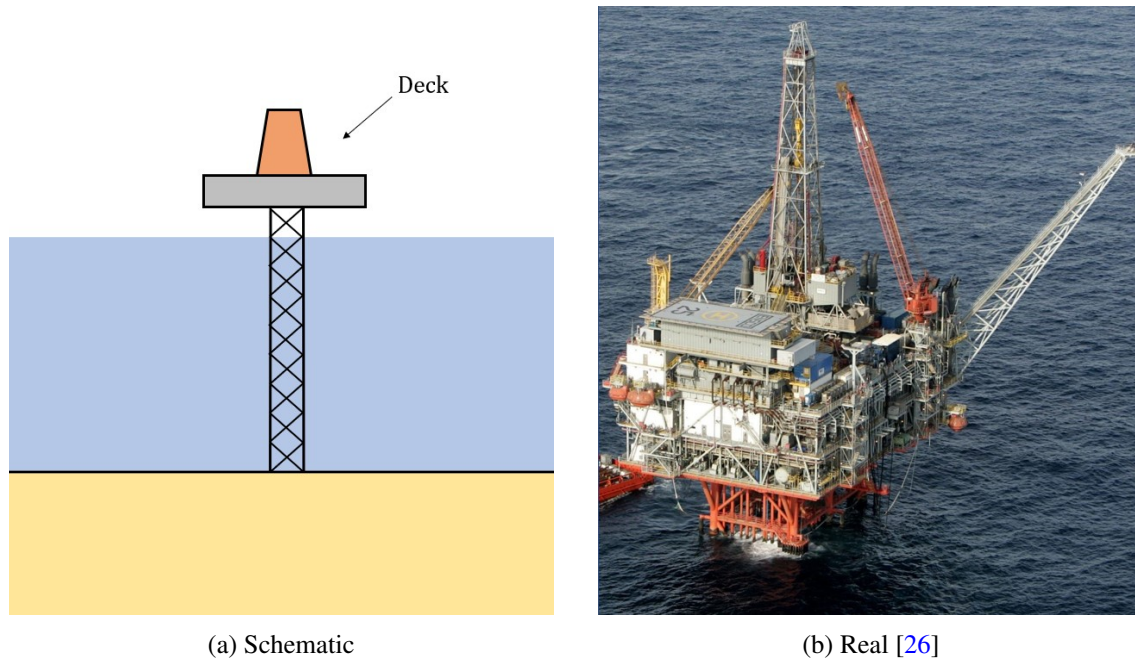
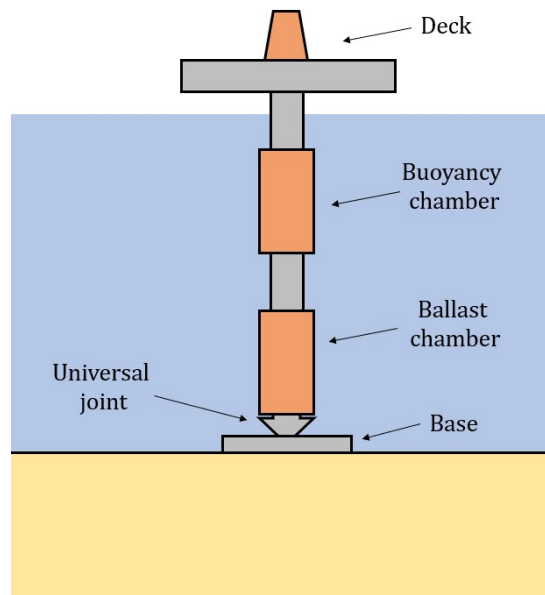


Figure 2.10 – Compliant Tower.

2.2.3.4 Articulated Tower

Articulated Tower is a vertical tower that allows free movement about its base due to a cardan joint at its base (universal joint). The foundation beneath the universal joint on the seafloor may be supported by gravity or piles. At the surface of the tower is a massive buoyancy chamber that provides a substantial restoring force (moment). On the top of the tower, above the surface, there is a deck and a fluid swivel. In deeper seas, it is frequently useful to insert a second articulation at a mid-depth [14].

A schematic representation of a articulated tower is shown in Figure 2.11.



(a) Schematic

Figure 2.11 – Articulated tower.

2.2.4 Foundation types

A foundation is a structural element which connects the structure to the soil. It is responsible for transmitting the loads to the underlying soil. Typically, foundations fall into two categories: shallow and deep (or piled) [27]. By definition, shallow foundation embedment depth, L , is less than its minimum lateral dimension, B . The deep foundation embedment depth is much greater than its minimum lateral dimension. The distinction $L/B = 1$ to 3 for a deep foundation is made for convenience, as illustrated in Figure 2.12.

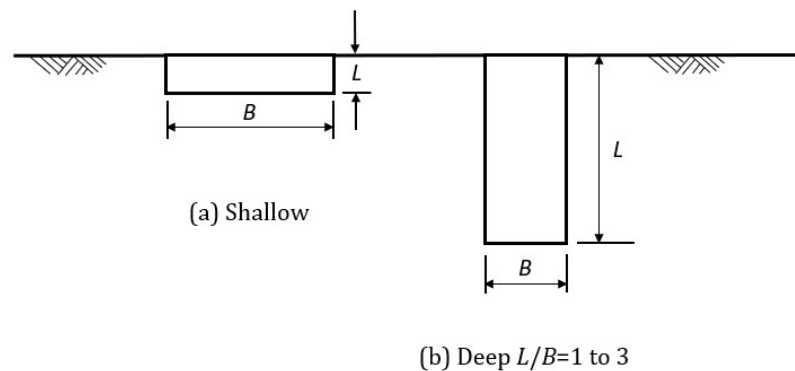


Figure 2.12 – Definition of shallow and deep foundations. Adapted from [27].

However, the terms shallow and deep may be confusing when applied to underwater foundations since the same foundation may be designed to penetrate either partly or fully in seabed. Thus, it could have different classifications according to these definitions as shown in Figure 2.13.

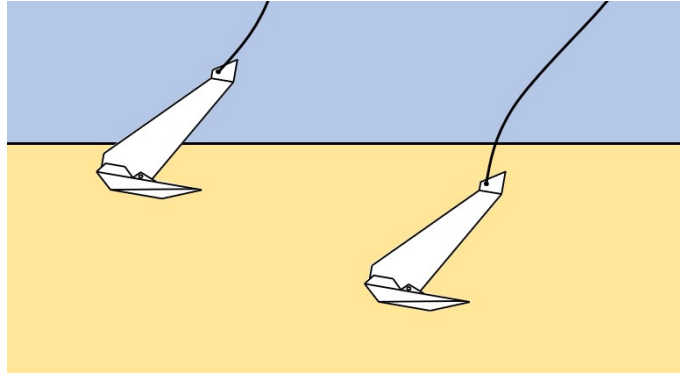


Figure 2.13 – Drag anchor partly (shallow foundation) and fully embedded (deep foundation).

Therefore, to broadly represent all the different types of foundations a classification similar to what was proposed by Landris T. Lee et al will be used [28].

Underwater foundations may be classified into the following categories: improved-site, gravity-based, and pinned. The pinned foundations can be subdivided in driven piles, drilled and grouted piles, suction piles and drag embedded anchors as presented in Figure 2.14.

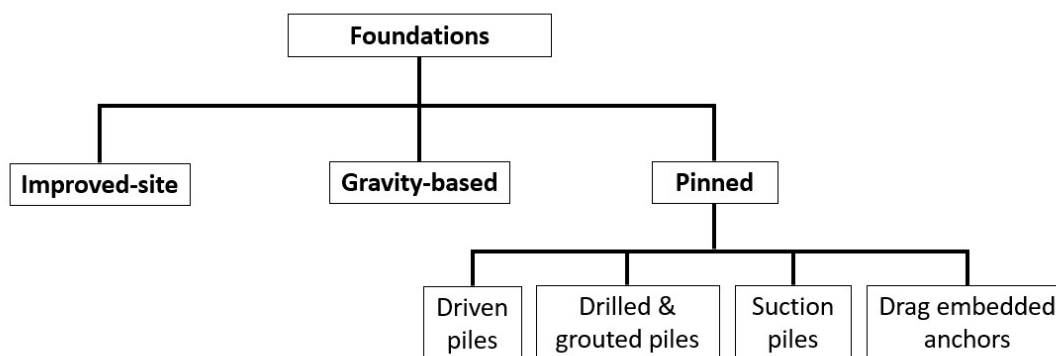


Figure 2.14 – Types of foundations.

Nonetheless, the traditional classification (shallow and deep foundations) will be used throughout this document to better understand some equations' terminology and applicability.

2.2.4.1 Improved-site

An improved-site foundation consists of a levelled surface (excavated or backfilled) above modified or natural soil and/or rock material. Soil alteration includes preloading, dredging, backfill, levelling, consolidation, and other engineered adjustments. [28].

This may be the least complex of all foundations because the foundation itself is not connected to the structure. Figure 2.15 illustrates one example of an improved-site foundation.

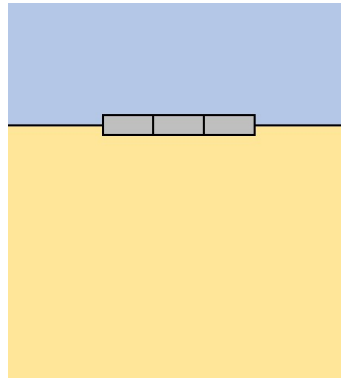


Figure 2.15 – Improved-site foundation: concrete pads.

2.2.4.2 Gravity-based

Although gravity structures are considered as a fixed platform in section 2.2.1.3, they may be also regarded as a type of foundation. For gravity-based foundations, Figure 2.16, the structure is connected to the foundation and loads bear down directly on the uppermost layers. This type of foundation is designed to resist the imposed compression, tension, overturning, lateral, and dynamic forces.

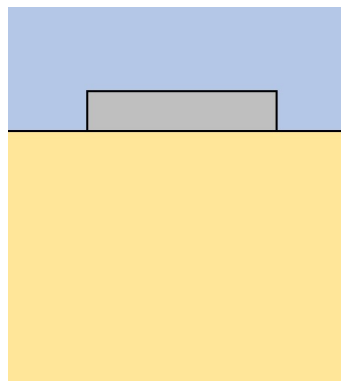


Figure 2.16 – Gravity-based foundation.

2.2.4.3 Pinned

Pinned foundations are constructed to achieve more lateral stability, to give greater uplift resistance, to provide greater resistance to scouring compared to gravity-based foundations, and to reduce the potential for differential settling of specific structures. This document explains the four most common forms of pinned offshore foundations: driven piles, drilled and grouted piles, self-penetrating (suction) piles, and drag embedded anchors [17].

Driven piles Driven piles are tubular members driven into the ground using a pile hammer, creating a stable foundation for underwater or overwater structures, Figure 2.17. Driven pile foundations can be made of concrete, steel or timber. These piles are prefabricated before placing at the construction site. When driven piles are made of concrete, they are precast. The pile length, diameter and the wall thickness depend on the soil characteristics and the magnitude of design loads [17].

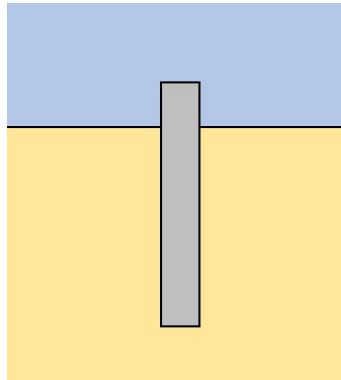


Figure 2.17 – Driven piles.

Drilled and grouted piles The concept of drilled and grouted steel piles, shown in Figure 2.18, has been successfully implemented in offshore applications. Typically, a hole is made to a certain depth through the leg of a jacket construction into the seafloor. Then, a pile is placed through the leg of the jacket and dropped into the drilled hole. Cement is then injected downward through and around the pile to fill the space between the pile and the sides of the seabed hole. The grout cement is pumped until the annulus between the jacket leg and the pile is filled. Through the grout annulus between the pile and the inside wall of the jacket leg, structural continuity and load transmission are established between the jacket and the pile [17].

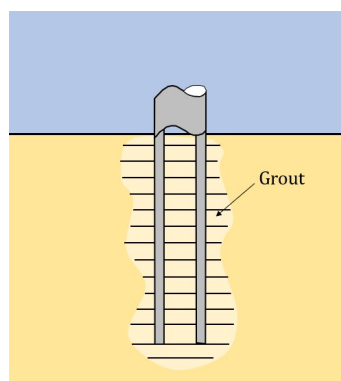


Figure 2.18 – Drilled and grouted piles.

Self-penetrating (suction) piles Self-penetrating piles (also called buckets, skirts, suction caissons, anchor piles or suction anchors) are made of an hollow cylinder, usually made of steel, having the top end closed and the bottom end opened, Figure 2.19. The suction pile is placed on the sea bottom and the entrapped water is pumped out to create an underpressure relative to the surrounding water pressure, which causes the pile or caisson foundation to penetrate into the seabed. Suction piles are now widely being used in offshore industries for anchoring large offshore floating facilities to the seafloor [17].

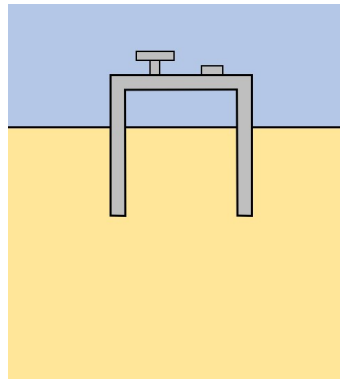


Figure 2.19 – Suction pile.

Drag embedded anchors Drag embedded anchors are designed to penetrate either partly or fully in the seabed, Figure 2.20. Their holding capacity is directly related to embedment depth. It is very well suited for resisting large horizontal loads, but it does not perform well for large vertical loads [17].

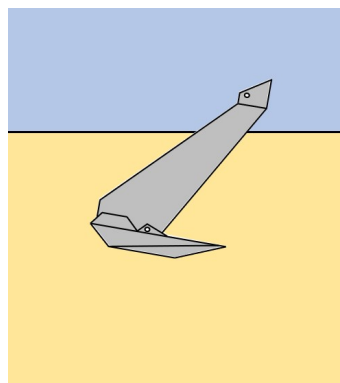


Figure 2.20 – Drag embedded anchor.

A brief summary of advantages and disadvantages of the different types of foundation is presented in Table 2.1 [28, 29].

Table 2.1 – Advantages and disadvantages of different types of foundation [28, 29].

Foundations		Advantages	Disadvantages
Improved-site		• Improve soil properties for structure settlement • Simple geometry	• Implies numerous soil modifications • Requires a considerable hard soil underbase
Gravity-base		• Shorter installation time than pinned foundations • Simple installation process	• Requires medium to hard soil conditions • Difficult to remove upon decommissioning
Pinned	Driven piles	• Withstand high bending and tensile stresses • Multiple pile materials and equipment available	• Difficult to remove upon decommissioning • Soil disturbance can affect surrounding structures
	Drilled and grouted piles	• Pile material is not subject to handling or driving stress • Easily adjusted lengths to match the bearing stratum • Construction noise and vibration are reduced	• Requires special equipment and inspection • Possible end-bearing reduction in sandy stratum • Difficult to remove upon decommissioning
	Suction piles	• Less equipment required for installation • Shorter installation time comparing to piles • May be fairly easily removed or relocated	• Highly dependent on soil conditions • Excessive differential pressure may cause soil heave • Requires ROV (remotely operated vehicle) for installation
	Drag embedded anchors	• Simple installation process • Recoverable during decommissioning	• Installation and orientation difficult to assure • Requires ROV (remotely operated vehicle) for installation

2.2.5 Foundation materials

Materials used in offshore structures must withstand extreme conditions such as corrosive sea actions and dynamic cyclic loads over a wide temperature range. Therefore, standard codes and special requirements are imposed on material properties. The materials used for offshore structural applications are structural steel, concrete, and grout. Although these are acknowledged in all different standard codes, their classification may vary from each other. Thus, ISO 19902:2007, DNVGL-OS-C101, and API RP 2A-WSD standards codes are compared [6, 9, 30].

2.2.5.1 Structural steel

Structural steel is one of the most common material in offshore industry due to its strength and long operating life cycle.

The main characteristics of steels are strength and toughness. Strength is the primary parameter for structural steel and the yield strength is used to determine the resistance of structural components. Toughness becomes more important as the magnitudes of varying actions increase and as service temperatures decrease since its directly related with fatigue fracture control.

In one hand, steel may be grouped according to its strength level. DNVGL and API standards codes group steel in three different categories while ISO standard establishes five different groups.

According to DNVGL and API codes steel can be divided into the following strength groups [9, 30]:

Group I designates mild steels with specified minimum yield strengths of 280 MPa or less. Carbon equivalent is generally 0.40% or less.

Group II designates intermediate strength steels with specified minimum yields of over 280 MPa through 360 MPa. Carbon equivalent ranges up to 0.45% and these steels require the use of low hydrogen welding processes.

Group III designates high strength steels with specified minimum yield strengths in excess of 360 MPa.

ISO standard code extends a little bit further this classification by adding two more groups to the previously mentioned [6]. They can be defined as:

Group IV designates high strength steels with specified minimum yield strengths in range of 400 to 455 MPa.

Group V designates steels with specified minimum yield strength above 500 MPa together with high levels of impact toughness and weldability.

In another hand, steel may be grouped according to its toughness level. Although, DNVGL standard code doesn't make this distinction because it divides structural steel in grades according to its weldability, API and ISO standard codes consider structural steel toughness classes. According to API code steel can be divided into the following class groups:

Class A steels are suitable for use at subfreezing temperatures. Steels enumerated herein as Class A can generally meet the Charpy requirements at temperatures ranging from -40°C to -20°C .

Class B steels are suitable for use where thickness, cold work, restraint, stress concentration, impact loading, and/or lacking of redundancy indicate the need for improved notch toughness. Class B can generally meet these Charpy requirements at temperatures ranging from 0° to 10°C .

Class C steels are those which have a history of successful application in welded structures at service temperatures above freezing, but for which impact tests are not specified. Such steels are applicable to primary structural members involving limited thickness, moderate forming, low restraint, modest stress concentration, quasi-static loading and structural redundancy such that an isolated fracture would not be catastrophic. Examples of such applications are piling, jacket braces and legs, and deck beams and legs.

According to the steel function, it is recommended a minimum group and class category. This recommendation can be made by two different approaches: material category approach and design class approach. After deciding these two parameters a steel may be selected.

2.2.5.2 Cement grout

Cement grout is normally used to fill the space between the piles and the surrounding structure. Grout mixtures are made of cement, water, finer sand and some chemical additives [31]. The offshore grouting process is unusual in that only one chance is available to achieve a satisfactory connection. Therefore, it is imperative that all measures possible be taken to ensure that the grout and grouting equipment used in the process are satisfactory prior to pumping the grout.

2.2.5.3 Concrete

The concrete mix used in belled piles should be selected on the basis of shear strength, bond strength and workability for underwater placement including cohesiveness and flowability. The concrete mix may be made with aggregate and sand, or with sand only. According to API RP 2A-WSD, the water-cement ratio should be less than 0.45. If aggregate is used, the aggregates should be small and rounded, the sand content should be 45% or greater, the cement content should be not less than 445 kg/m^3 [9].

2.3 AMALIA project

The Algae-to-Market Lab IdeAs (AMALIA) project started in 2016 and ended in 2019 [32]. This project counted with the participation of higher education institutions, companies, and research units, in particular INEGI. In the northwest of the Iberian Peninsula, exotic seaweeds are

causing major ecological problems in underwater environments. Therefore, this project aimed to take advantage of this threat by enhancing their use in innovative food products, cosmetics, and pharmaceuticals. To help observe the presence of seaweeds in situ, INEGI has developed an innovative benthic lander to collect data about seaweeds growth and further comprehend its characteristics [33].

2.3.1 AMALIA lander

The AMALIA lander, shown in Figure 2.21, was designed to act as a tripod, so it could adjust to ocean floor irregularities. Since macroalgae flourish mainly in the photic zone ($< 200\text{m}$), the lander was designed for shallow-water applications. In addition, it was required to have a reduced projected area to reduce the drag effect, which is why it has an open-frame shape. The lander was produced in a modular way to ease transport and allow a simpler replacement of broken and specialized parts [33].

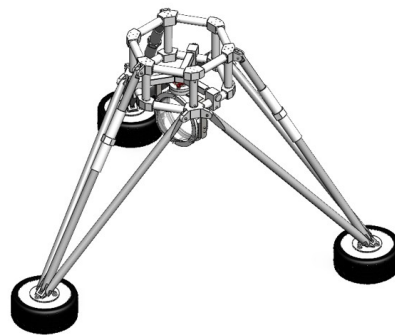
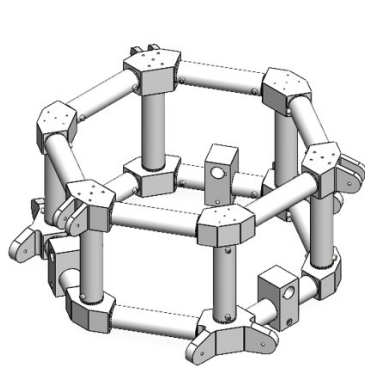


Figure 2.21 – AMALIA's benthic lander [33].

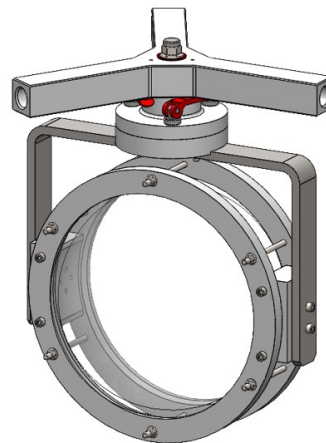
The lander can be divided in five parts, presented in Figure 2.22:

- The cage, main structure of the lander
- The glass sphere coupled to the cage where data collecting equipment is kept
- The rods
- The feet which make the connection between the rods and the ballast weights
- The ballast weights

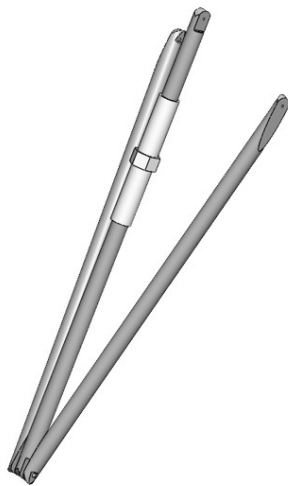
The first three parts are made of ERTACETAL C, commercial name of polyoxymethylene (POM), due to its low density and satisfactory mechanical properties. This material choice makes the lander not only easy to transport but also mechanically sturdy. The ballast weights are concrete pieces enclosed in a tire casing while the sphere is made of glass [33].



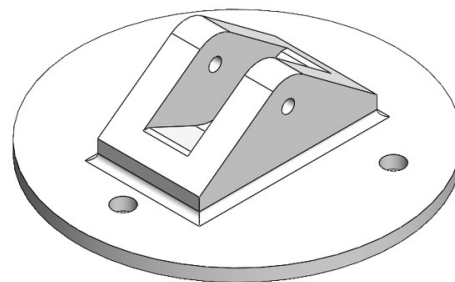
(a) Cage



(b) Glass sphere



(c) Rods



(d) Foot



(e) Ballast weights

Figure 2.22 – Components of AMALIA's benthic lander [33].

This lander version uses a gravity-based type of foundation, since the ballast weights are responsible for keeping the structure in contact with the seabed. The ballast weights are fixed to the lander's feet through screws which require human intervention specially during the recovery phase, since it is necessary to detach the structure from the ballast weights in order to recover it.

As mentioned before in section 1.2 the objective of this thesis is to design an alternative foundation to the lander that doesn't require human assistance (e.g a diver) since technical divers may work up to 100 meters, being impossible to recover the structure if it is installed at a higher depth.

2.3.2 AMALIA mission

The southernmost cove of the Berlenga grande island, also known as Cova do Sono, Figure 2.23 was selected as the site for the installation of the AMALIA lander. The Portuguese city of Peniche is close to the Berlengas islands. Cova do Sono was picked as the site for early testing because it is a restricted area and has a number of invasive macroalgae that are typical throughout the northwestern Iberian Coast.



Figure 2.23 – Cova do Sono (red icon), Berlenga grande island.

The installation took place at a depth of 10 m, just 50 m from the barrier that separates the shore of Peniche from the island of Berlenga grande. For additional monitoring, a sensor for measuring water quality was placed. The glass surface was subjected to routine inspections and cleaning procedures [33].

Chapter 3

Concept and Loads

This chapter aims to find an alternative concept for the lander's foundation with the information gathered in the previous chapter. Once chosen, the loads acting on the structure and that the foundation must withstand are calculated, using DNVGL standard codes as reference.

3.1 Concept

The main objective of this thesis is to replace the current lander's foundation type with one that could accomplish the project requirements:

- Have a good installation precision;
- Do not need human labour during installation (for example, be ROV operated);
- Have an easy decommissioning procedure;
- Be reusable.

A decision matrix was applied, see Table 3.1, to choose one of the foundations presented in subsection 2.2.4.

Table 3.1 – Concept decision matrix.

	Improved-site	Gravity-base	Pinned			
			Driven piles	Drilled piles	Suction piles	Drag anchor
Installation precision	✓	✓	✓	✓	✓	
Human labour not needed					✓	✓
Easy decommissioning					✓	✓
Recoverable					✓	✓

As a result, suction piles were the foundation type chosen to replace the current in use since they fulfil all the project requirements. In Figure 3.1 is shown the concept change.

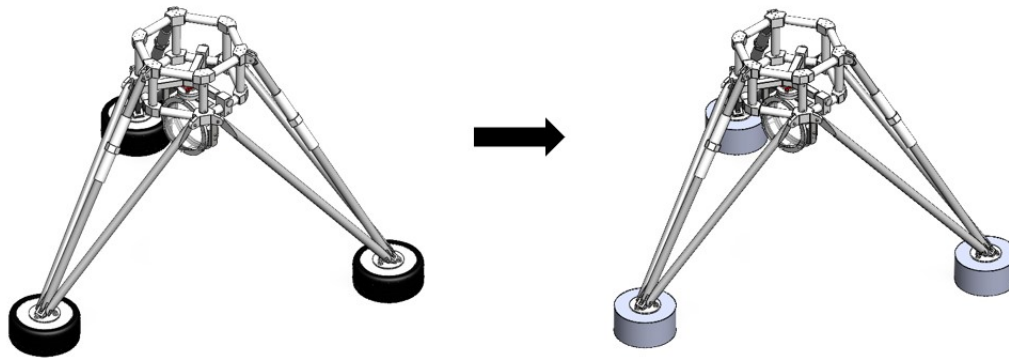


Figure 3.1 – Concept change from gravity-based (left) to suction buckets (right) foundation.

Before starting the suction buckets design, it is necessary to understand which forces are acting in the structure since suction buckets must withstand them during service.

3.2 Structural loading

In recent years, the offshore wind turbine (OWT) industry has discussed suction bucket design in workshops [34] and has considered it a novel solution [35] for traditional foundations. Hence, the DNVGL standard codes, which are related to this industry, will be applied throughout this work because they are on par with this new trend.

Although the lander may assume different configurations before deployment by using the threaded connection positioned in the middle rod of each rod trio, only one will be the object of study. The orientation studied corresponds to the one that leads to a greater projected area and therefore leads to larger induced forces. Figure 3.2 exhibits the lander's dimensions for the tallest configuration. Note that the foundation is not assumed since the objective is to study structural loading.

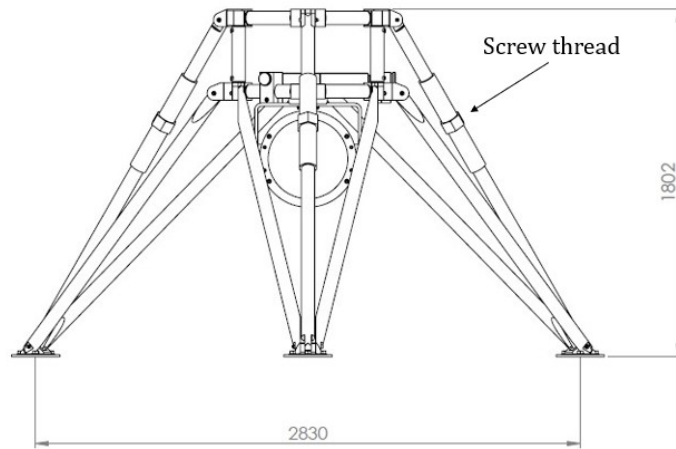


Figure 3.2 – Lander dimensions (in mm).

The following tables present some relevant data for the project. Table 3.2 shows AMALIA's lander component's mass and Table 3.3 shows lander and environment conditions data. The following data was obtained using the lander's CAD model in *Solidworks*.

Table 3.2 – AMALIA's lander component's mass.

Component	Component mass (Kg)	Quantity	Total mass (Kg)
Cage	48.60	1	48.60
Glass sphere	18.89	1	18.89
Rods trio	29.23	3	87.69
Foot	1.14	3	3.42
Lander			158.60

Table 3.3 – Data about the lander and environment conditions.

Data	
Lander mass (m)	158.60 Kg
Lander volume (V)	0.13 m ³
Structural lander centre of gravity ($y_{cg,lander}$)	1 m (positive from base of feet)
Sea water density ($\rho_{sea\ water}$)	1025 Kg/m ³
Sea water kinematic viscosity ($\nu_{sea\ water}$)	1.18 × 10 ⁻⁶ m ² /s
Sea water temperature ($T_{sea\ water}$)	15°C
Gravity acceleration (g)	9.81 m/s ²

Since the lander is underwater during its service life, gravity, buoyancy, wave, and current loads are appropriate for its structural loading calculation.

3.2.1 Gravity and buoyancy loads

Using Newton's second law, it is possible to calculate the structure weight, $F_{g,structure}$:

$$\begin{aligned} F_{g,structure} &= m \times g \Rightarrow \\ \Rightarrow F_{g,structure} &= 158.60 \times 9.81 \\ &= 1555.87 \text{ N.} \end{aligned} \quad (3.1)$$

The lander is undersea during its service life. Thus, buoyancy loads will act alongside gravity. By Archimedes' principle, the buoyancy load, $F_{b,structure}$, is given by:

$$\begin{aligned} F_{b,structure} &= \rho_{sea\ water} \times V \times g \Rightarrow \\ \Rightarrow F_{b,structure} &= 1025 \times 0.13 \times 9.81 \\ &= 1307.18 \text{ N.} \end{aligned} \quad (3.2)$$

Figure 3.3 displays the gravity and buoyancy force vectors.

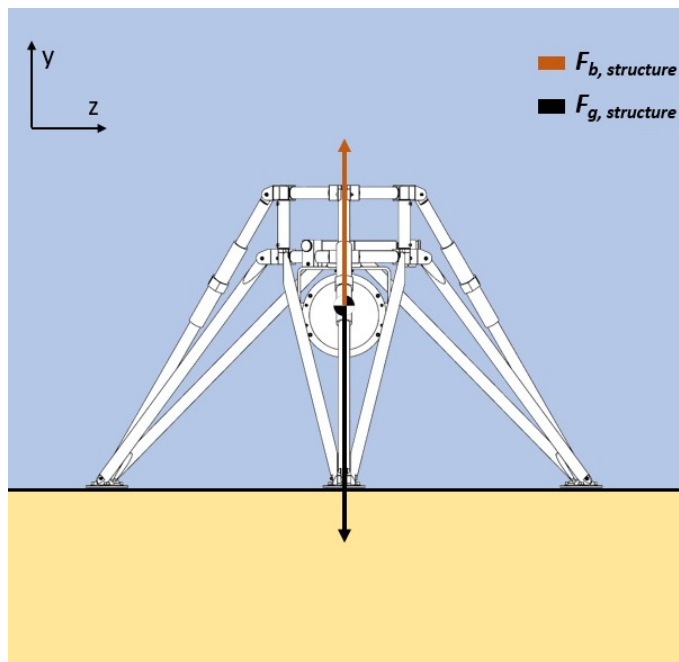


Figure 3.3 – Gravity and buoyancy force vectors.

3.2.2 Wave loads

To determine the wave loads acting on the structure it is necessary to know the waves' characteristics, such as their period, height, velocity and acceleration. These may vary with geographic position, tides, wind velocity, direction, and other parameters. Thereby, trying to parameterize a wave becomes a complex task.

There are two techniques to characterize wave phenomena: deterministic design wave and stochastic methods.

Deterministic regular wave methods are sufficient for a quasi-static response of structures. The wavelength, wave period, and wave height fully characterize these types of waves.

Stochastic methods are used to model the sea surface required by structures with a significant dynamic response. The wave frequency spectrum with a significant wave height, a representative frequency, a mean propagation direction, and a spreading function characterizes a sea state. The sea state is usually assumed to be a stationary random process.

According to a research report on the structural modelling of deep water jack up structures led by KW Ltd to the Health and Safety Executive of the UK government, the reaction of fixed structures is essentially quasi-static. The dynamic response is only interesting for the fatigue-limit state of individual structural members [36].

The AMALIA's lander is a fixed structure since it is supported by its suction buckets, transferring the loads directly to the seabed. Thus, the lander reaction will be considered quasi-static. Therefore, the deterministic regular wave method is the design method most adequate for this application.

3.2.2.1 Regular wave characteristics

A regular wave is the most simple way to characterize a wave. However, it is commonly used to design offshore structures. It can be completely defined by three parameters. They are [12]:

1. Period (T), time taken for two successive crests to pass a stationary point.
2. Height (H), vertical distance between the crest and the following trough.
3. Water depth (d), vertical distance from mean ocean floor to mean water level.

With these three parameters it is possible to define all the other wave parameters, such as:

- Wavelength (λ), distance between successive crests. This can be deduced using the dispersion relation.
- Phase velocity (c), propagation velocity of the wave and is denoted $c = \lambda/T$.
- Wave angular frequency: $\omega = 2\pi/T$.
- Wave number: $k = 2\pi/\lambda$.

- Wave crest height (A_C), distance from still water level to the crest.
- Wave trough depth (A_T), distance from still water level to the trough.

The dispersion relation is the relationship between wave period, T , wavelength, λ , and wave height, H , for a water depth, d .

Some regular wave properties mentioned before are represented in Figure 3.4.

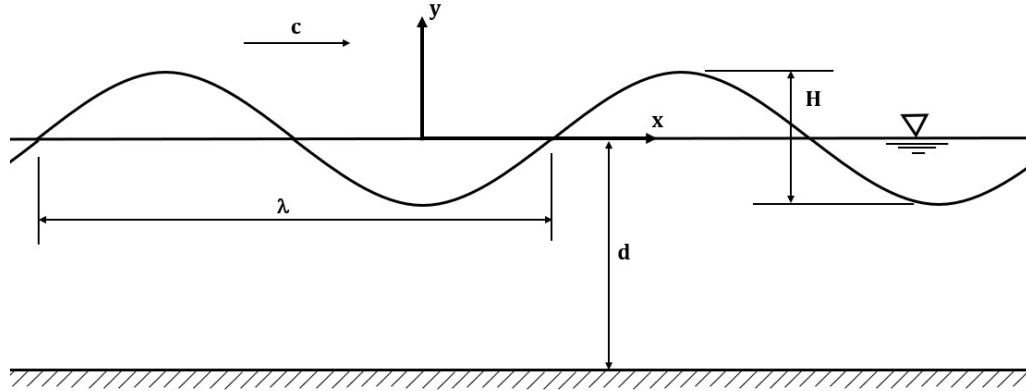


Figure 3.4 – Regular wave properties.

Nonlinear regular waves are asymmetric because $A_C > A_T$.

3.2.2.2 Regular wave theories

There are several wave theories that can be used to describe wave characteristics. The following theories are regular wave theories, meaning that the wave form is permanent (properties are invariant) from cycle to cycle.

Linear wave theory

The most simple and applied theory is the linear wave theory. It can also be called the small amplitude wave theory, sinusoidal wave theory, and Airy theory. This theory is applied if wave height, H , is assumed to be substantially smaller than the wavelength, λ and the water depth, d .

The surface elevation is given by

$$\eta = \frac{H}{2} \cos \Theta, \quad (3.3)$$

where $\Theta = k(x \cos \beta + z \sin \beta) - \omega t$ is the phase and β is the direction of propagation measured from the positive x-axis [37].

The dispersion relationship for linear waves is given by

$$T = \left[\frac{g}{2\pi\lambda} \tanh\left(\frac{2\pi d}{\lambda}\right) \right]^{-1/2}. \quad (3.4)$$

For regular linear waves the wave crest height, A_C , is equal to the wave trough height, A_T .

Stokes wave theory

The linear wave theory describes the wave as having a single frequency and regular shape. Stokes wave theory proposed that it is the result of adding linear solutions with high-order solutions. For example, second-order Stokes wave theory provides two components for the wave amplitude, the first at the wave frequency and the second at twice the wave frequency. Figure 3.5 shows these properties.

The surface elevation profile for a regular second-order Stokes wave is given by

$$\eta = \frac{H}{2} \cos \Theta + \frac{\pi H^2}{8\lambda} \frac{\cosh kd}{\sinh^3 kd} [2 + \cosh 2kd] \cos 2\Theta, \quad (3.5)$$

where $\Theta = k(x \cos \beta + z \sin \beta) - \omega t$ [37]. The linear dispersion relation holds for second-order Stokes waves.

Nonlinear regular waves are asymmetric because $A_C > A_T$.

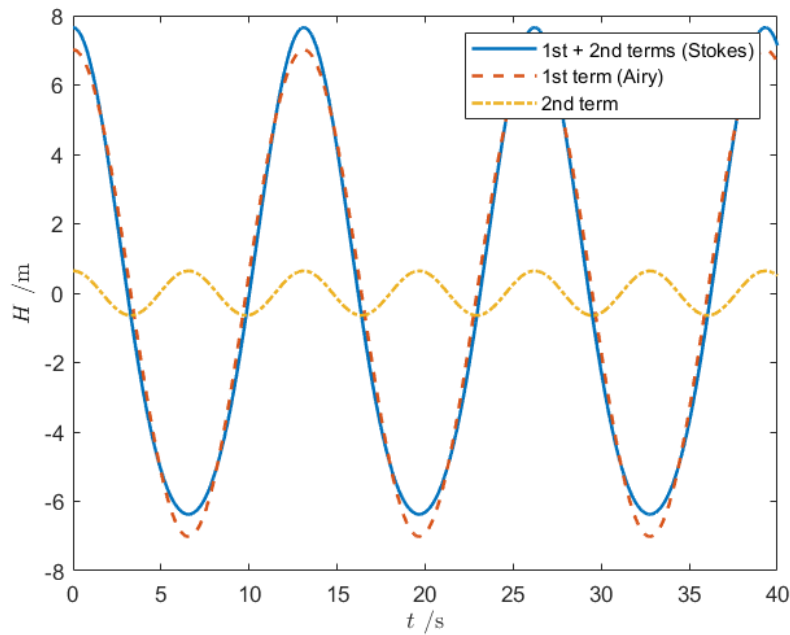


Figure 3.5 – Time history of first and second order components.

Cnoidal wave theory

The cnoidal wave is a periodic wave with sharp crests separated by wide troughs, as shown in Figure 3.6. Cnoidal wave has crest to wave height ratio between 0.635 and 1. It describes finite amplitude long waves of permanent form in shallow waters [38].

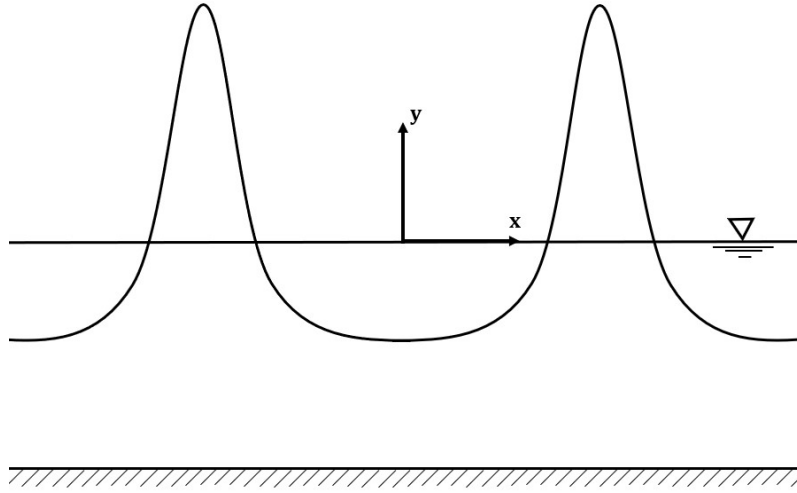


Figure 3.6 – Cnoidal wave.

Stream Function Theory

The stream function is a purely numerical procedure for approximating a given wave profile.

There are two types of stream function theory. The first is called the regular stream function theory and is based on a periodic wave with a permanent form but asymmetric crest to trough [12]. The second one is irregular therefore the waveform has no restrictions. Thus, this section will not cover it.

The regular stream function theory describes the stream function in a moving coordinate system. According to this theory, the stream function describes the wave [37]:

$$\psi(x, y) = cy + \sum_{n=1}^N X(n) \sinh nk(y+d) \cos nkx, \quad (3.6)$$

where N determines the order of the theory and c is the wave celerity. The expression for ψ is written in a moving coordinate system so that time does not enter in the equation explicitly.

For $N = 1$, the stream function theory reduces to linear wave theory.

3.2.2.3 Applicability of wave theories

Three wave parameters determine which wave theory is suitable for a specific problem. These are the wave height H , the wave period T and the water depth d . They are used to define three non-dimensional parameters which determine the different wave theories' validity,

- Wave steepness parameter: $S = 2\pi \frac{H}{gT^2} = \frac{H}{\lambda_0}$.
- Shallow water parameter: $\mu = 2\pi \frac{d}{gT^2} = \frac{d}{\lambda_0}$.
- Ursell number: $U_R = \frac{H\lambda^2}{d^3}$.

where λ_0 and k_0 are the linear water wavelength and wave number corresponding for wave period T [37]. The three parameters are not independent. When two parameters are defined, the third is uniquely determined. The relation is:

$$U_R = \frac{S}{\mu^3}.$$

Thus, it is necessary to know the three wave parameters that will define the wave acting on the structure. The parameters that define the most demanding wave were chosen because they lead to a conservative structural design.

Since the AMALIA mission was in Portugal, see section 2.3.2, a future expedition will probably be in the same country. Therefore, the wave parameters were defined using the Portuguese marine data.

In Portugal, the Hydrographic Institute of Portugal (IH) has the mission of contributing to the country's development in the scientific areas and defence of the marine environment. It has several buoys that permanently gather information about ocean and wind conditions. Thereby, comparing the results of ondograph buoys, Figure 3.7 and Table 3.4, it is possible to infer which region has the harshest waves.

According to a study led by Rita Esteves, Leixões region has the harshest conditions after comparing the buoys collected data [39].



Figure 3.7 – Ondograph buoys location. Adapted from [40].

Table 3.4 – Leixões, Sines and Faro ondograph buoys information [40].

Buoys	Depth (m)	Latitude (N)	Longitude (W)
Leixões	83	41°19'00"	08°59'00"
Sines	97	37°55'15"	08°55'44"
Faro	93	36°54'16"	07°53'54"

The information collected from the Leixões buoy was asked to the Hydrographic Institute of Portugal. However, it was not conceded for this project since these buoys are protocolled.

The Portuguese Institute for Sea and Atmosphere (IPMA), responsible for the weather and sea state forecasts, was found as an alternative to obtain the wave parameters.

Three numerical models provide the foundation for IPMA's sea state predictions. The Wave Model (WAM), a worldwide model with a spatial resolution of 0.25° , and the limited area model European Shelf Model (LAM), which covers most of the North Atlantic with a resolution of 0.1° , are two such models created by the European Centre for Medium-Range Weather Forecasts (ECMWF). A spectrum discretization of 36 frequencies and 36 orientations is present in both models. The Simulating WAVes Nearshore (SWAN) model, with a 0.05° horizontal resolution and a spectrum discretization of 36 frequencies and 36 orientations, was recently applied by IPMA in the area next to the Portuguese mainland [41].

The predictions are in good agreement when compared with IH's buoys data, using the three numeric models. However, a tendency to overestimate the wave height (H) and underestimate the wave period (T) was shown [42].

The wave parameters were requested to IPMA referring to at least one year. For that reason, it was possible to detect the seasonal effects on wave parameters. IPMA provided data referring to 2019 and 2020. Moreover, since the data was available in `.grib` format instead of `.xls` format, a Python script was developed to read and analyse it. The Appendix A shows the source code. Figure 3.8 represents the idea behind it. The script aims to represent graphically wave height and period with time. After that, it determines the maximum wave height and the respective wave period with the data provided.

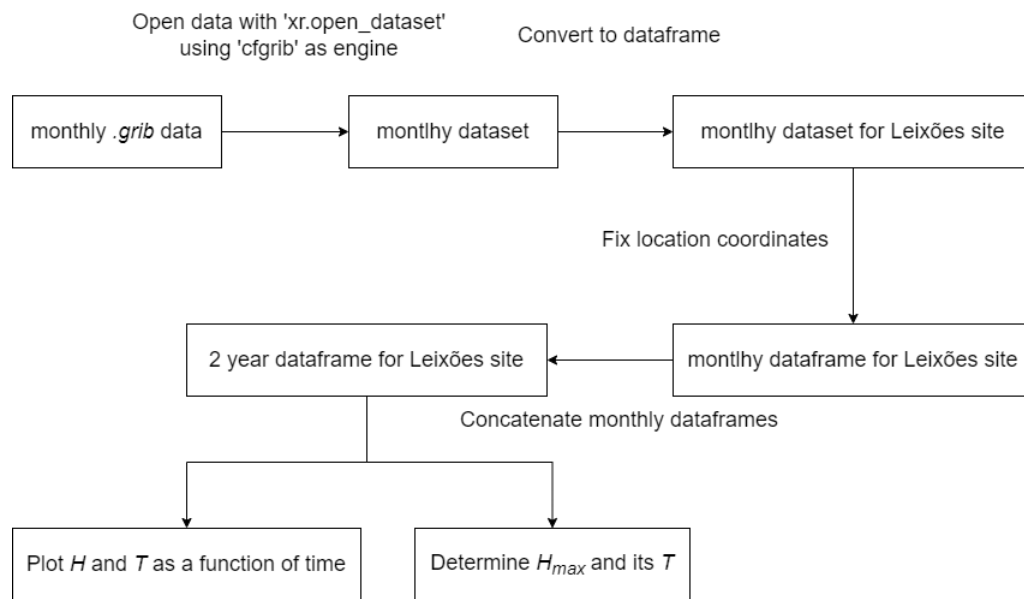
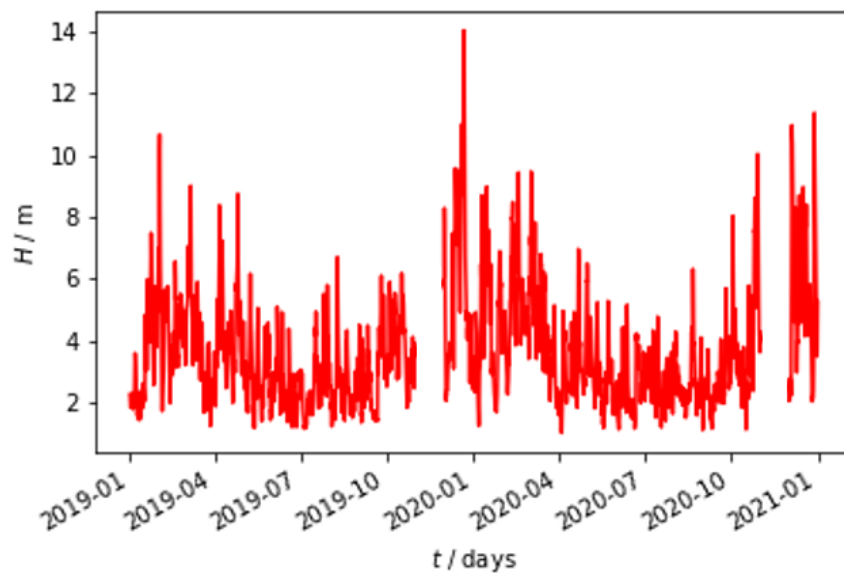


Figure 3.8 – Python script.

The results are shown in Figure 3.9 and Figure 3.10.

Figure 3.9 – Wave height, H , at Leixões buoy between 2019 and 2020.

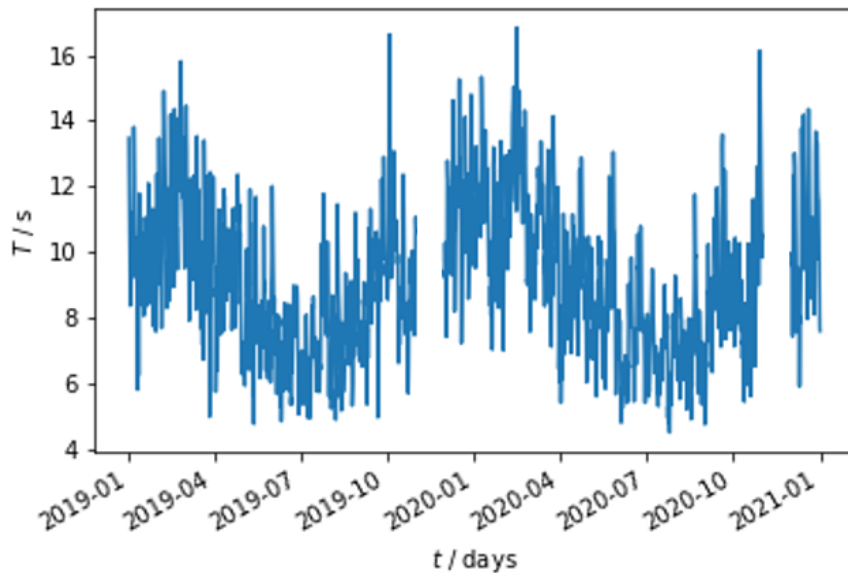


Figure 3.10 – Wave period, T , at Leixões buoy between 2019 and 2020.

The maximum wave height was registered in 22nd December of 2019 at 00:00 h. It had:

- $H_{max} = 14.03$ m,
- $T = 12.43$ s.

Consequently, the wave parameters chosen are (they are represented in the imperial system of units to be able to use Figure 3.11):

- $T = 12.43$ s,
- $H = 14.03$ m = 46.03 ft,
- $d = 200$ m = 656.17 ft, which is a project requirement.

The DNV-RP-C205 standard distinguishes the validity of different stokes wave theories by suggesting a chart based on the previously mentioned non-dimensional parameters. In order to decide which wave theory is recommended, it is necessary to calculate the following parameters:

- $\frac{H}{T^2} = \frac{46.03}{12.43^2} = 0.298$ ft/s²,
- $\frac{d}{T^2} = \frac{656.17}{12.43^2} = 4.247$ ft/s².

According to Figure 3.11, the most suitable theory for this project is the 2nd or 3rd order stokes theory. For simplicity, the 2nd order stokes theory is the theory chosen.

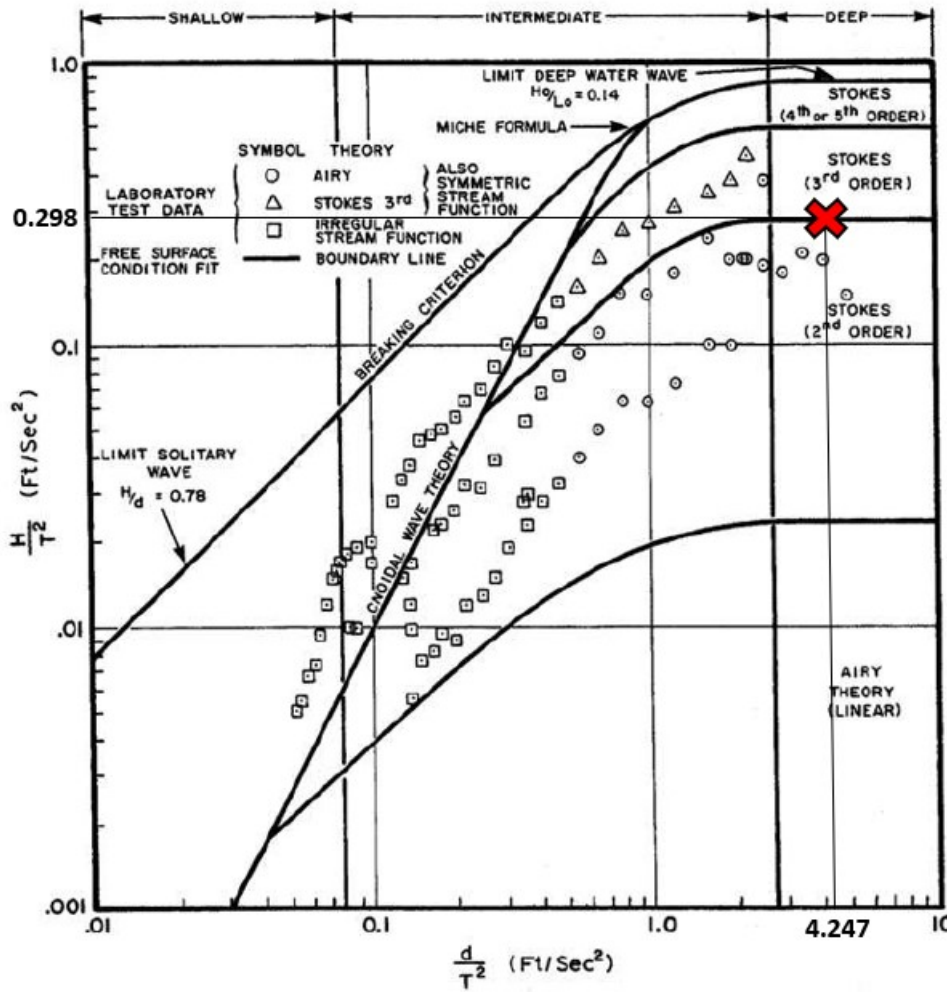


Figure 3.11 – Validity of different wave theories. Adapted from DNV-RP-C205 standard [37].

Table 3.5 gathers the parameter formulas for Airy wave theory and Stokes second-order wave theory. If the ratio d/λ is larger than 0.5 and $\tanh(kd)$ is close to 1, the deep water condition may be considered [12].

Using the dispersion equation 3.4, also valid for second-order Stokes theory, for the wave parameters already discussed, the wavelength is:

$$\lambda = 241.22 \text{ m.}$$

Since,

$$\frac{d}{\lambda} = \frac{200}{241.22} = 0.83, \text{ and } 0.83 > 0.5,$$

the deep water linear term may be chosen as the first-order term.

According to the expressions in Table 3.5, it was elaborated a MATLAB script, see Appendix B, to better analyse the wave properties.

Table 3.5 – Formulas for Airy wave theory and Stokes second-order wave theory. Adapted from DNV-RP-C205 standard [37].

Parameter	Airy wave theory		Stokes second-order wave theory	
	General water depth	Deep water		
Surface elevation, η	$\frac{H}{2} \cos \Theta$	$\frac{H}{2} \cos \Theta$	$\eta_\ell + \frac{\pi H^2}{8\lambda} \frac{\cosh kd}{\sinh^3 kd} [2 + \cosh 2kd] \cos 2\Theta$	
Horizontal particle velocity, u	$\frac{\pi H}{T} \frac{\cosh[k(y+d)]}{\sinh(kd)} \cos \Theta$	$\frac{\pi H}{T} e^{ky} \cos \Theta$	$u_\ell + \frac{3}{4c} \left(\frac{\pi H}{T} \right)^2 \frac{\cosh[2k(y+d)]}{\sinh^4(kd)} \cos 2\Theta$	
Vertical particle velocity, v	$\frac{\pi H}{T} \frac{\sinh[k(y+d)]}{\sinh(kd)} \sin \Theta$	$\frac{\pi H}{T} e^{ky} \sin \Theta$	$w_\ell + \frac{3}{4c} \left(\frac{\pi H}{T} \right)^2 \frac{\sinh[2k(y+d)]}{\sinh^4(kd)} \sin 2\Theta$	
Horizontal particle acceleration, $\dot{u}$	$\frac{2\pi^2 H}{T^2} \frac{\cosh[k(y+d)]}{\sinh(kd)} \sin \Theta$	$\frac{2\pi^2 H}{T^2} e^{ky} \sin \Theta$	$\dot{u}_\ell + \frac{3\pi}{\lambda} \left(\frac{\pi H}{T} \right)^2 \frac{\cosh[2k(y+d)]}{\sinh^4(kd)} \sin 2\Theta$	
Vertical particle acceleration, $\dot{v}$	$-\frac{2\pi^2 H}{T^2} \frac{\sinh[k(y+d)]}{\sinh(kd)} \cos \Theta$	$-\frac{2\pi^2 H}{T^2} e^{ky} \cos \Theta$	$\dot{w}_\ell - \frac{3\pi}{\lambda} \left(\frac{\pi H}{T} \right)^2 \frac{\sinh[2k(y+d)]}{\sinh^4(kd)} \cos 2\Theta$	

Using the MATLAB script in Appendix B, the velocity and acceleration quantities were compared for each theory for lander centre of gravity. According to Figures 3.12 and 3.14, the difference between both theories is insignificant, due to the lower magnitude of stokes second-order term shown in Figures 3.13 and 3.15. Hence, the Airy wave theory will be used to determine the wave parameters.

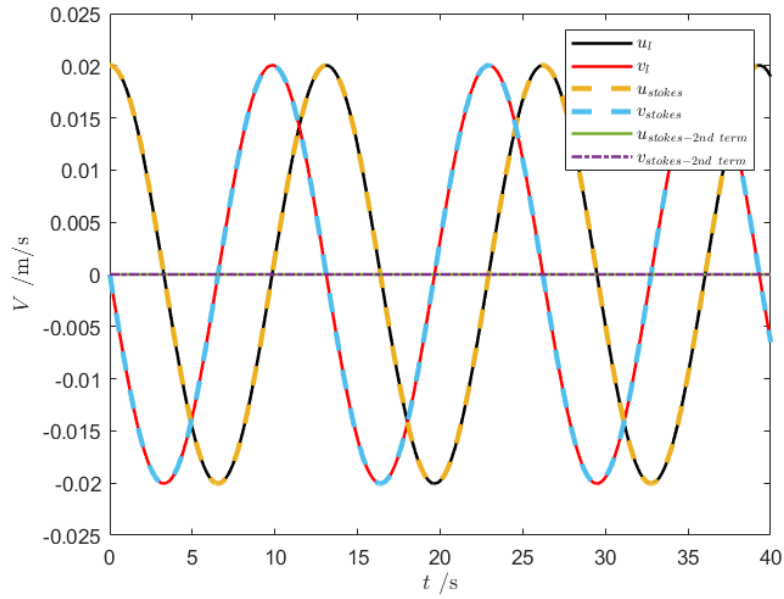


Figure 3.12 – Comparison between linear and second-order Stokes theories regarding water velocity.

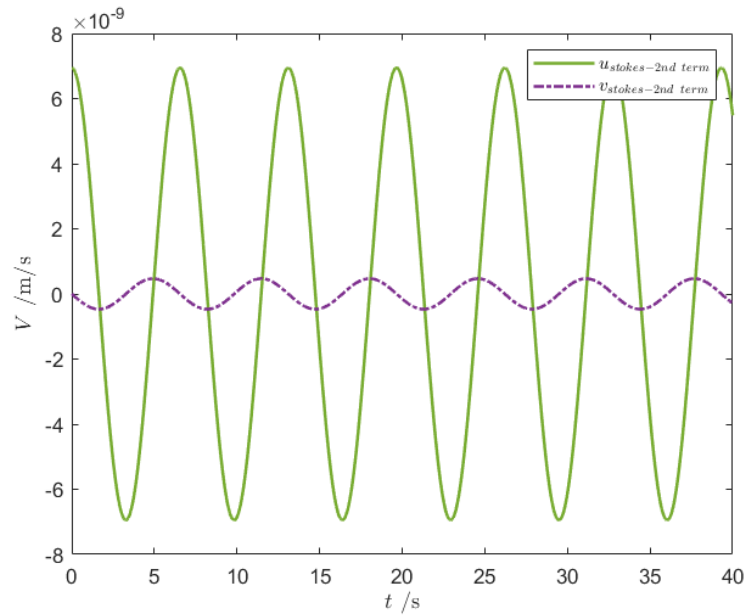


Figure 3.13 – Second-order Stokes water velocity term.

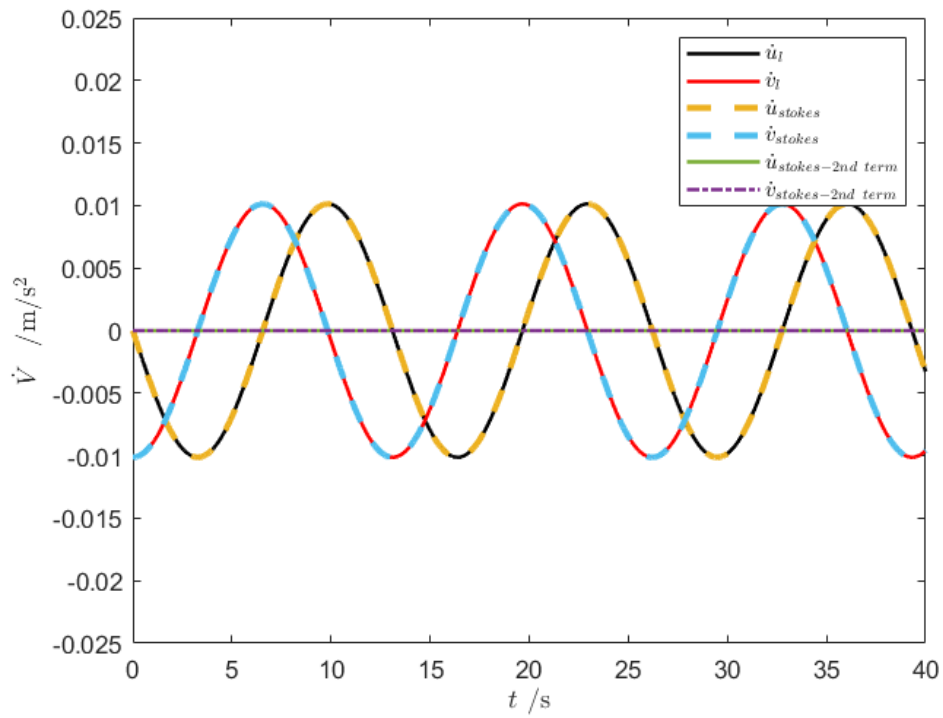


Figure 3.14 – Comparison between linear and second-order Stokes theories regarding water acceleration.

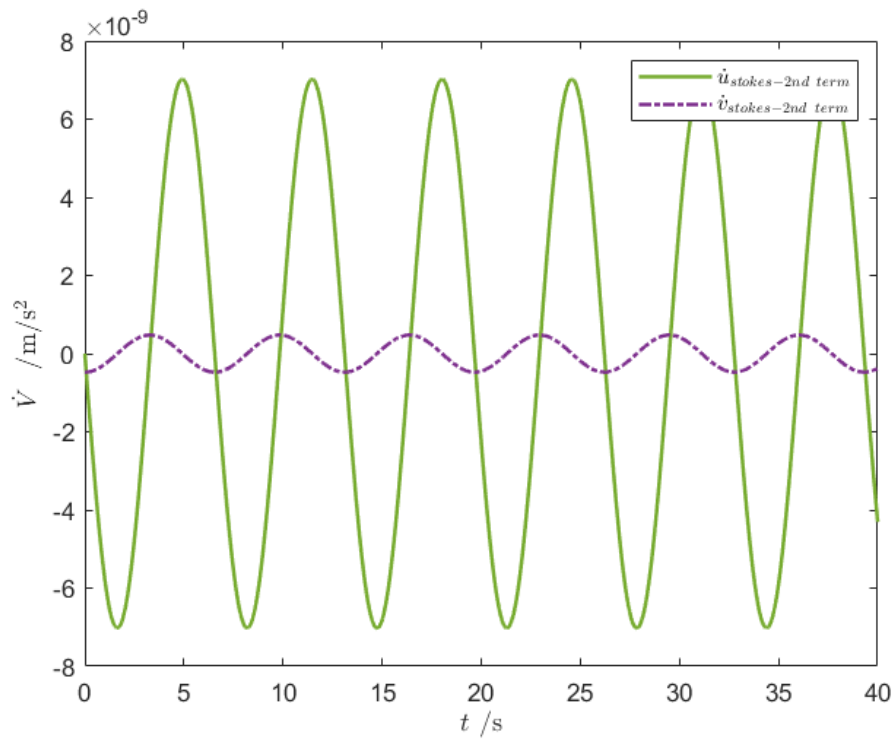


Figure 3.15 – Second-order Stokes water acceleration term.

3.2.2.4 Morison's equation

Wave loads can be calculated using Morison's load formula, which is the sum of an inertia force, proportional to acceleration and a drag force, being proportional to the square of velocity. It is applicable for thin structural parts with cross-sectional dimensions small enough to allow the gradients of fluid particle velocities and accelerations in the direction normal to the member to be neglected.

Morison's load formula is applicable when the following condition is satisfied:

$$\lambda > 5D,$$

where λ is the wavelength and D is the member diameter or other projected cross-sectional dimension.

Comparing the wavelength value with the different member's diameters, shown in Table 3.6, it is possible to conclude that the Morison's load formula is applicable in every part of the lander:

$$241.22 > 5 \times 0.07,$$

$$241.22 > 0.35.$$

Table 3.6 – Lander structural component's dimensions.

Component	Diameter (mm)	Length (mm)
Middle rod	70	2042
Lateral rod	70	1878
Cage vertical member	70	298
Cage horizontal member	70	275

The Morison's load formula was developed to describe the horizontal forces acting on a vertical (fixed) pile which extends from the seabed through the free surface, as shown in Figure 3.16. In this figure the y -axis is shown as s -axis since it is measured from the ocean bottom ($s = y + d$).

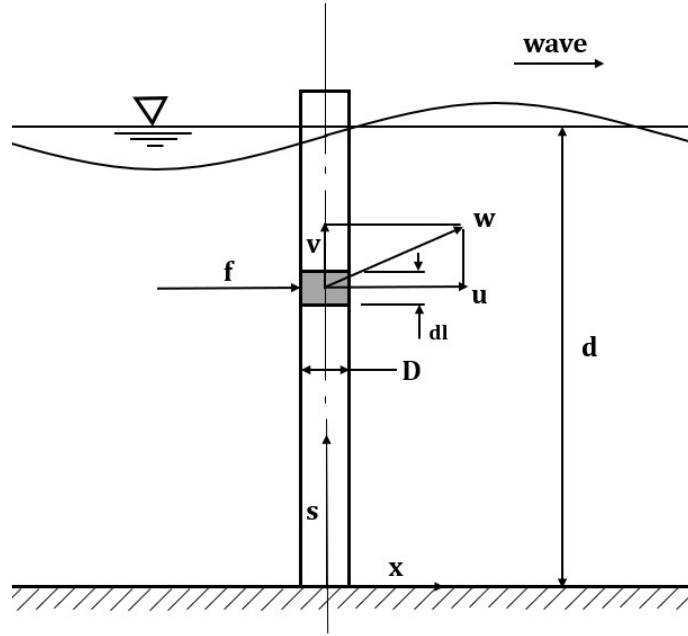


Figure 3.16 – Definition sketch for wave forces on small diameter cylinder. Adapted from [38].

The Morison's equation is written as:

$$f = f_I + f_D,$$

$$f = C_M A_L \frac{\partial u}{\partial t} + C_D A_D |u|u, \quad (3.7)$$

in which f is force per unit length of the vertical cylinder, C_M is the inertia coefficient, C_D is the drag coefficient, and u is the instantaneous water particle velocity defined by a suitable wave theory,

$$A_L = \rho \frac{\pi}{4} D^2,$$

and

$$A_D = \frac{1}{2} \rho D.$$

For the example given in Figure 3.16, the total force on the cylinder is given by the integral

$$F = \int_0^d f ds = \int_0^d \left[C_M A_L \frac{\partial u}{\partial t} + C_D A_D |u|u \right] ds. \quad (3.8)$$

The instantaneous velocity and acceleration may be considered as vector or scalar quantities since they are in one plane having the same direction. The Morison's equation implicitly assumes that the horizontal force is only function of the horizontal water particle velocity and acceleration normal to the cylinder. The vertical and tangential components are ignored in the evaluation. Thus,

the force on the vertical cylinder may be thought as a normal force whose horizontal component is equal to its magnitude and vertical component is zero.

This original Morison's equation is empirical and has evolved through different applications including structure orientation, moving structures, currents in presence of waves, among others.

Since the lander's structural members are tilted, a modified Morison's equation must be considered to calculate the wave forces acting on the lander.

3.2.2.5 Modified Morison's equation

The modified Morison's equation that will be analysed considers an inclined cylinder as shown in Figure 3.17. Again, in this figure the y -axis is shown as s -axis since it is measured from the ocean bottom ($s = y + d$).

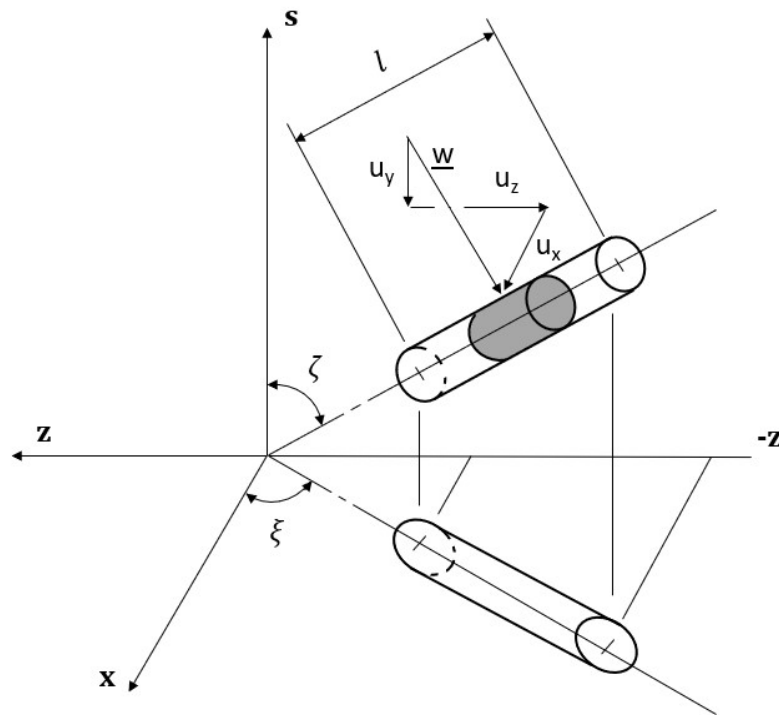


Figure 3.17 – Definition sketch of modified Morison's equation application on an inclined cylinder. Adapted from [38].

Writing the Morison equation for an inclined cylinder in terms of the normal velocity and acceleration vectors extends its application.

$$\mathbf{f} = C_M A_L \dot{\mathbf{w}} + C_D A_D |\mathbf{w}| \mathbf{w}, \quad (3.9)$$

where a bold character denotes vector and $\mathbf{w}$, $\dot{\mathbf{w}}$ are the components of velocity and acceleration normal to the inclined cylinder.

The normal velocity vector, $\mathbf{w}$, is written as

$$\mathbf{w} = \mathbf{C} \times [(\mathbf{u}\mathbf{i} + \mathbf{v}\mathbf{j}) \times \mathbf{C}], \quad (3.10)$$

where $\mathbf{C}$ is the unit vector along the cylinder axis so that

$$\mathbf{C} = C_x\mathbf{i} + C_y\mathbf{j} + C_z\mathbf{k}, \quad (3.11)$$

u, v are the horizontal and vertical water particle velocity, and $\mathbf{i}, \mathbf{j}, \mathbf{k}$ are the unit vectors along the cartesian coordinate axes, x, y , and z respectively.

Admitting a spherical coordinate system, then

$$C_x = \sin \zeta \cos \xi, \quad C_y = \cos \zeta, \quad C_z = \sin \zeta \sin \xi, \quad (3.12)$$

in which ζ is the angle of the cylinder axis to the vertical axis, and ξ is the angle of the cylinder projection to the x -axis.

Considering that $\mathbf{w}$ is written as

$$\mathbf{w} = u_x\mathbf{i} + u_y\mathbf{j} + u_z\mathbf{k}, \quad (3.13)$$

and by doing the cross product in equation 3.9, the velocity components along x, y and z are

$$u_x = u - C_x(C_x u + C_y v), \quad (3.14)$$

$$u_y = v - C_y(C_x u + C_y v), \quad (3.15)$$

$$u_z = -C_z(C_x u + C_y v). \quad (3.16)$$

Thereby, on a randomly oriented cylinder the forces per unit length are calculated in the x, y , and z directions respectively using the following expressions

$$f_x = C_M A_L \dot{u}_x + C_D A_D |w| u_x, \quad (3.17)$$

$$f_y = C_M A_L \dot{u}_y + C_D A_D |w| u_y, \quad (3.18)$$

$$f_z = C_M A_L \dot{u}_z + C_D A_D |w| u_z. \quad (3.19)$$

By integrating the equations along each projected cylinder dimension, it is possible to obtain the total force for each direction.

Due to the fact that the tangential component is neglected and the forces are assumed to act normal to the cylinder, the inertia and drag coefficients along x , y , and z axes are equal in equations (3.17) – (3.19).

The variations of C_M and C_D are function of Reynolds number, Re , the Keulegan-Carpenter number, KC , and the roughness, Δ_r :

$$C_D = C_D(Re, KC, \Delta_r),$$

$$C_M = C_M(Re, KC, \Delta_r).$$

The parameters are defined as

- Reynolds number: $Re = v_t D / \nu$.
- Keulegan-Carpenter number: $KC = v_m T / D$.
- Non-dimensional roughness: $\Delta_r = h_r / D$.

where:

D = diameter (m); T = wave period (s); h_r = roughness height (m); v_t = total flow velocity (m/s); ν = fluid kinematic viscosity (m^2/s); v_m = maximum orbital particle velocity (m/s).

In general, these coefficients are experimentally determined. Several studies were conducted to achieve these coefficients values. However, its applicability is often limited due to assumptions made.

In one hand, the experimental work led by Keulegan and Carpenter is of limited applicability since it does not cover the behaviour of an orbital wave field (waves characterized by horizontal and vertical velocities) [43].

In another hand, other studies cover orbital wave fields interactions with structures, but only for vertical orientations [44].

Since the majority of lander's cylinders are inclined and the wave profile is characterized by an orbital field because horizontal and vertical velocities are regarded, the studies previously mentioned are not applicable.

Based on all experimental studies, recommended practical values were established and may be applied in every situation. Table 3.7 shows the recommended practical coefficient values for smooth cylinders [14] since the lander's cylinders are assumed to be smooth.

Table 3.7 – Recommended practical values of C_M and C_D coefficients for smooth cylinders [14].

	$Re < 10^5$		$Re > 10^5$	
$KC < 10$	$C_M = 2.0$	$C_D = 1.2$	$C_M = 2.0$	$C_D = 0.6$
$KC \geq 10$	$C_M = 1.5$	$C_D = 1.2$	$C_M = 1.5$	$C_D = 0.6$

3.2.3 Current loads

Current is typically regarded as time-invariant, when it comes to offshore structures design, and is represented by its mean value. However, its strength may vary depending on water depth. It causes a pressure distribution around the offshore structure member, which creates a constant drag force in the flow direction.

The current load is written as:

$$F_c = \frac{1}{2} \rho C_D A u_c^2, \quad (3.20)$$

in which F_c is the total current force, ρ the sea water density, C_D the drag coefficient, A the projected area and u_c the uniform current velocity.

The current velocity profile is established by the sum of each component present, such as wind generated, tidal and other type of currents,

$$u_c(y) = u_{c,wind}(y) + u_{c,tide}(y) + \dots \quad (3.21)$$

The wind generated current profile can be taken as linear profile from $y = -d_0$ to still water level,

$$u_{c,wind}(y) = u_{c,wind}(0) \left(\frac{d_0 + y}{d_0} \right) \text{ for } -d_0 \leq y \leq 0, \quad (3.22)$$

or a slab profile,

$$u_{c,wind}(y) = u_{c,wind}(0) \text{ for } -d_0 \leq y \leq 0. \quad (3.23)$$

However, wind generated current may be assumed to vanish at a distance below the reference depth:

$$u_{c,wind}(y) = 0 \text{ for } y < -d_0, \quad (3.24)$$

where $u_{c,wind}(y)$ is the wind generated current velocity at level y , y the distance from still water level (positive upwards) and $d_0 = 50$ m is the reference depth for wind generated current [37].

The tidal current profile velocity may be modelled as a simple power law, assuming uni-directional current:

$$u_{c,tide}(y) = u_{c,tide}(0) \left(\frac{d + y}{d} \right)^\alpha \text{ for } y \leq 0, \quad (3.25)$$

where $u_{c,tide}(y)$ is the tide current velocity at level y , y is the distance from still water level (positive upwards), d is the water depth to still water level (taken positive) and α is an exponent, typically $\alpha = 1/7$.

In Figure 3.18 is depicted the total current profile and its different components.

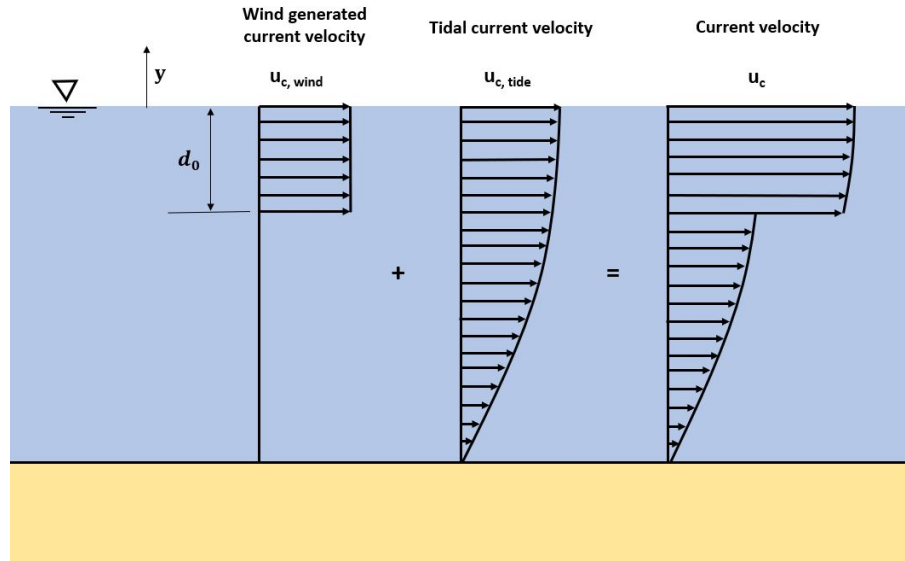


Figure 3.18 – Total current velocity profile and its components.

3.2.4 Wave and current loads

Despite being possible to calculate wave and current loads separately, when both effects are relevant it is incorrect to consider the total force as their sum.

In other words, if the total drag force is calculated by adding separately the different drag components it may under predicts the total drag force. Therefore, if both types of forces are relevant (wave and current forces), velocities must be added first and the total drag force calculated afterwards.

Thus, to considerate both effects, the modified Morison's equation should be applied with horizontal water particle velocity, u , being the sum of wave and current horizontal velocities, u_w and u_c . To prevent the previous mistake, the standard codes, such as the DNVGL-RP-C205, apply the (modified) Morison's equation to the horizontal water velocity composed by wave and current components [37]:

$$u = u_w + u_c. \quad (3.26)$$

When both effects are in the same direction, it is considered the most demanding situation for the structure. Therefore, this situation will be considered.

Moreover, the structural loading varies with wave and current applied direction.

According to Yeong-Hoon Jeong et al, the most vulnerable situation, see Figure 3.19, corresponds to the one in which one of the buckets is subjected to pull out, as shown in Figure 3.20 [45].

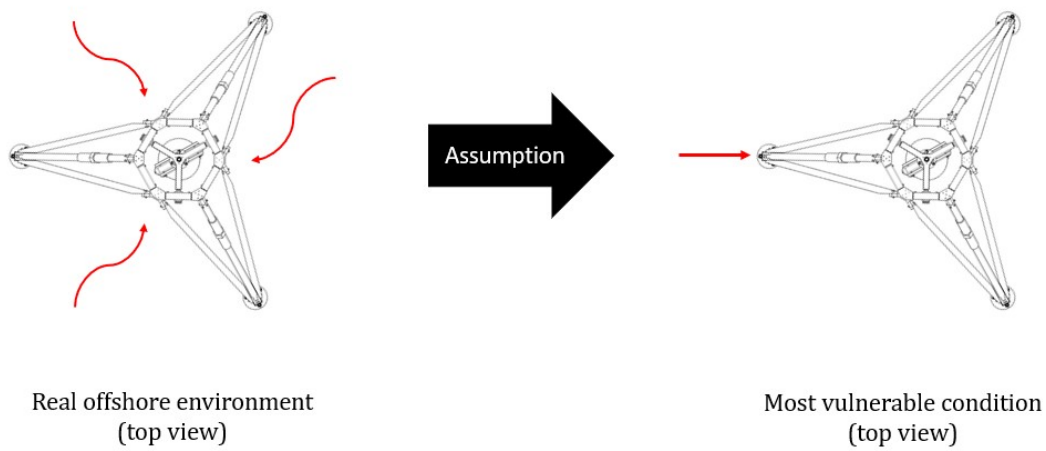


Figure 3.19 – Assumption of the most vulnerable position [45].

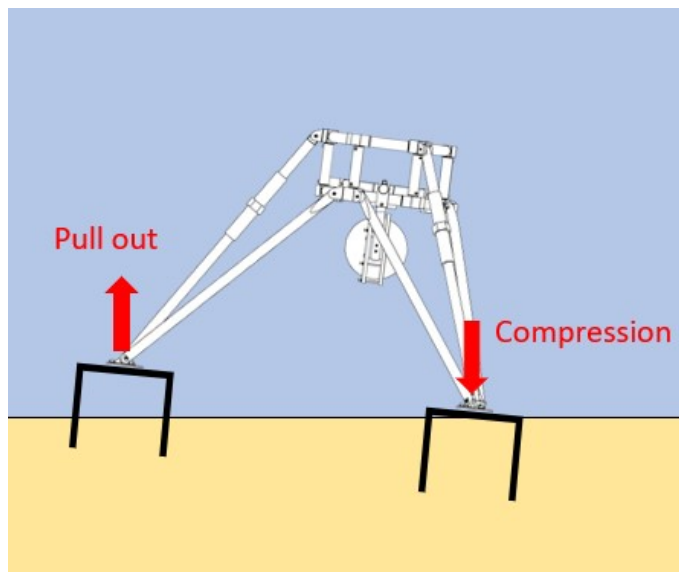


Figure 3.20 – Most vulnerable position (side view).

Table 3.8 comprises the wave properties for the selected case in section 3.2.2.3 that will be needed to determine the kinematic properties and inertia and drag coefficients, before the load calculation.

Table 3.8 – Wave properties for the selected case.

Data	
Period (T)	12.43 s
Height (H)	14.03 m
Water depth (d)	200 m
Wavelength (λ)	241.22 m
Phase velocity (c)	19.41 m/s
Wave angular frequency (ω)	0.51 rad/s
Wave number (k)	0.026 rad/m
Wave crest height (A_C)	7.015 m
Wave trough depth (A_T)	7.015 m

The water velocities will be calculated for lander centre of gravity. It is approximately at 1 m from sea floor. Therefore, the depth used in further calculation is given by,

$$\begin{aligned}
 y_{lander} &= d + y_{cg,lander} \\
 &= -200 + 1 \\
 &= -199 \text{ m.}
 \end{aligned}$$

The horizontal wave velocity, u_w , is given by the deep water linear term as evidenced by Figure 3.12,

$$\begin{aligned}
 u_w(y) &= \frac{\pi H}{T} e^{ky} \cos \Theta \Rightarrow \\
 \Rightarrow u_w(-199) &= \frac{\pi \times 14.03}{12.43} e^{0.026 \times (-199)} \cos \Theta \\
 &= 0.02 \cos \Theta \text{ m/s.}
 \end{aligned}$$

Water current measurements of horizontal sea surface velocities vary throughout the year, however measurements vary between 0.1-1.0 m/s [46]. Thereby, the maximum measured current surface velocity was considered for each component, $u_{c,wind}(0) = u_{c,tide}(0) = 1.0 \text{ m/s}$.

Using equation 3.24,

$$u_{c,wind}(y) = 0 \text{ m/s, since } -199 < -50.$$

By equation 3.25,

$$\begin{aligned}
 u_{c,tide}(y) &= u_{c,tide}(0) \left(\frac{d+y}{d} \right)^\alpha \text{ for } y \leq 0 \Rightarrow \\
 \Rightarrow u_{c,tide}(-199) &= 1.0 \left(\frac{200-199}{200} \right)^{\frac{1}{7}} \\
 &= 0.47 \text{ m/s.}
 \end{aligned}$$

Reminding the equation 3.18, the horizontal current velocity, u_c , is given by,

$$\begin{aligned}
 u_c(y) &= u_{c,wind}(y) + u_{c,tide}(y) \Rightarrow \\
 \Rightarrow u_c(-199) &= u_{c,wind}(-199) + u_{c,tide}(-199) \\
 &= 0 + 0.47 \\
 &= 0.47 \text{ m/s.}
 \end{aligned}$$

Finally, the horizontal water velocity, u , is given by equation 3.26,

$$\begin{aligned}
 u(y) &= u_w(y) + u_c(y) \Rightarrow \\
 \Rightarrow u(-199) &= 0.02 \cos \Theta + 0.47 \text{ m/s,}
 \end{aligned}$$

and $u_{max}(y)$ for $y = -199$ is maximum when $\cos \Theta = 1$,

$$\begin{aligned}
 u_{max}(-199) &= 0.02 \times (1) + 0.47 \\
 &= 0.49 \text{ m/s.}
 \end{aligned}$$

After horizontal water particle velocity determination, u , the vertical wave velocity will be determined, v . Similarly to horizontal wave velocity, the vertical wave velocity is given by the deep water linear term as evidenced by Figure 3.12,

$$\begin{aligned}
 v(y) &= \frac{\pi H}{T} e^{ky} \sin \Theta \Rightarrow \\
 \Rightarrow v(-199) &= \frac{\pi \times 14.03}{12.43} e^{0.026 \times (-199)} \sin \Theta \\
 &= 0.02 \sin \Theta \text{ m/s,}
 \end{aligned}$$

and $v_{max}(y)$ for $y = -199$ is maximum when $\sin \Theta = 1$,

$$\begin{aligned} v_{max}(-199) &= 0.02 \times (1) \\ &= 0.02 \text{ m/s.} \end{aligned}$$

Once both water velocities are known, it is possible to determine the inertia and drag coefficients. To determine them, first it is necessary to determine Reynolds number, Re , and Keulegan-Carpenter number, KC , quantities. Non-dimensional roughness will not be determined since lander's cylinders are considered as being smooth.

Therefore,

$$\begin{aligned} Re &= \frac{vD}{\nu} \Rightarrow \\ \Rightarrow Re &= \frac{0.49 \times 0.070}{1.18 \times 10^{-6}} \\ &= 29067.80, \end{aligned}$$

and

$$\begin{aligned} KC &= \frac{v_m T}{D} \Rightarrow \\ \Rightarrow KC &= \frac{0.02 \times 12.43}{0.070} \\ &= 3.55. \end{aligned}$$

According to Table 3.7, and because $Re < 10^5$ and $KC < 10$ the inertia and drag coefficients assume the following values:

- $C_M = 2.0$,
- $C_D = 1.2$.

A MATLAB script, see Appendix C, was elaborated to calculate the forces since some integrals do not have analytical solution. Thereby, the numerical integration function of MATLAB makes it possible to obtain a solution. The script validity was confirmed since similar results were obtained in a previous structural study [47].

For a better understating of the results, the lander structural cylinders were numbered, as shown in Figure 3.21.

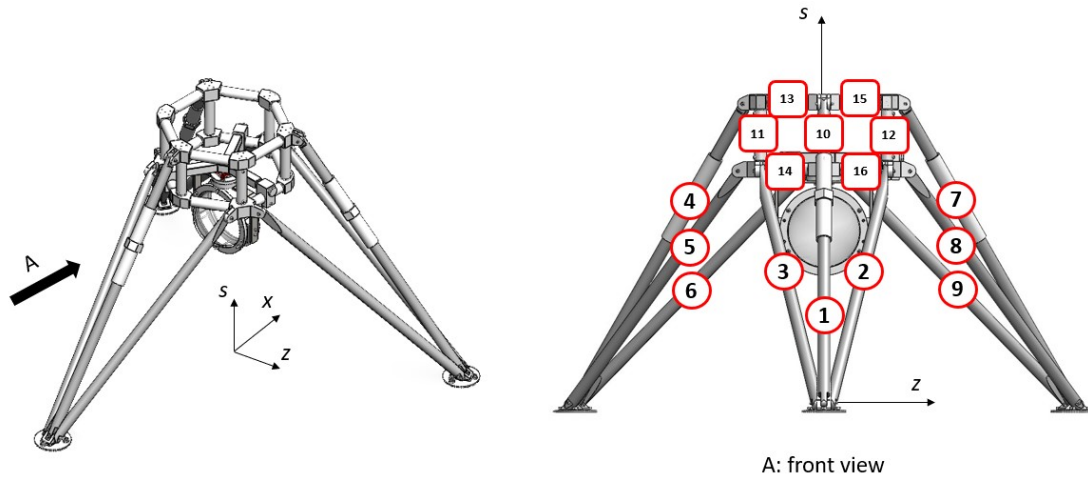


Figure 3.21 – Lander structural cylinders numbered.

The different components of total force acting in each cylinder are displayed in Table 3.9.

Table 3.9 – Different components of total force for each cylinder.

No. cylinder	Length, l (m)	$\zeta(^{\circ})$	$\xi(^{\circ})$	$F_{x,max}$ (N)	$F_{y,max}$ (N)	$F_{z,max}$ (N)
1	2.042	33.49	0	10.18	-6.74	0
2	1.878	44.62	-14.09	5.13	-4.66	1.07
3	1.878	44.62	14.09	5.13	-4.66	-1.07
4	2.042	33.49	240	15.37	3.89	-2.34
5	1.878	44.62	225.91	8.49	3.96	-2.78
6	1.878	44.62	254.09	11.99	1.86	-1.78
7	2.042	33.49	120	15.37	3.89	2.34
8	1.878	44.62	134.09	8.49	3.96	2.78
9	1.878	44.62	105.91	11.99	1.86	1.78
10-12	0.298	0	0	1.88	0	0
13-14	0.298	90	-120	0	0	0
15-16	0.298	90	120	0	0	0
Total				97.78	3.36	0

The set of forces acting on each cylinder can be replaced by equivalent forces and moments acting on lander's centre of mass if they are statically equivalent. The equivalent forces are calculated in the last line of Table 3.9. The equivalent moment components are presented in Table 3.10.

Table 3.10 – Equivalent moment components for lander's centre of mass.

No. cylinder	$CG_{lander} - CG_{cylinder}$			$M_{x,CG}$ (Nm)	$M_{y,CG}$ (Nm)	$M_{z,CG}$ (Nm)
	x (m)	y (m)	z (m)			
1	-1.08	0.15	0	0	0	8.81
2	-0.99	0.34	-0.18	0.84	0.92	6.36
3	-0.99	0.34	0.18	-0.84	-0.92	6.36
4	0.54	0.15	0.95	3.70	-14.60	4.41
5	0.36	0.34	0.95	3.76	-8.07	4.31
6	0.63	0.34	0.68	1.27	8.15	5.25
7	0.54	0.15	-0.95	-3.70	14.60	4.41
8	0.36	0.34	-0.95	-3.76	8.07	4.31
9	0.63	0.34	-0.68	-1.27	8.15	5.25
10	-0.42	-0.46	0	0	0	0.87
11	-0.21	-0.46	0.36	0	-0.68	-0.87
12	-0.21	-0.46	-0.36	0	0.68	-0.87
13	-0.34	-0.69	0.18	0	0	-0.87
14	-0.34	-0.31	0.18	0	0	0
15	-0.34	-0.69	-0.18	0	0	0
16	-0.34	-0.31	-0.18	0	0	0
Total				0	0	46.86

In Figure 3.22 is shown the equivalent forces and moment vectors.

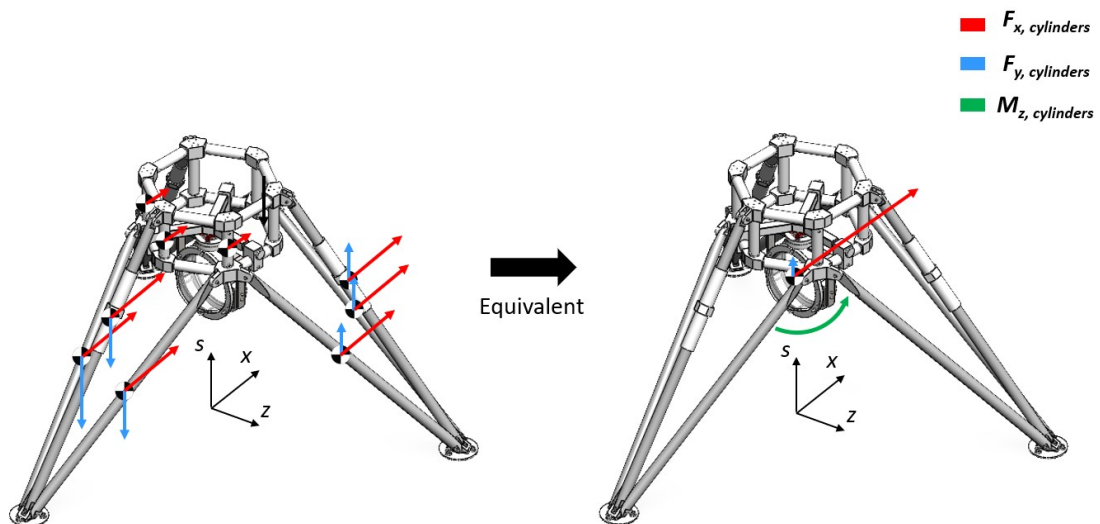


Figure 3.22 – Equivalent forces and moment represented in lander's centre of mass.

According to the table results, it is possible to infer that the force acting in the wave and current direction, F_x , is predominant. Due to the structure symmetry, the total transverse force component, F_z , is null. It is also possible to observe a very small force in the vertical direction, F_y , which is negligible.

As pictured in Figure 3.23, F_x , which is the predominant force acting on the cylinder is practically equal to its drag component since the inertia component is almost null. The inertia component is approximately zero because it was assumed a quasi-static response, which implies that inertia forces are negligible. Similar results were obtained for the other cylinders.

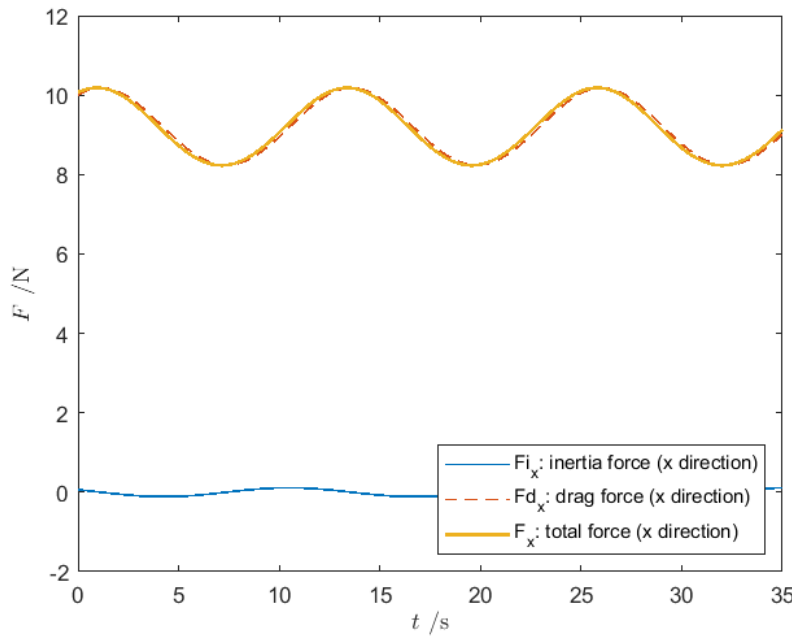


Figure 3.23 – Components of force in x direction for cylinder no. 1.

To end the wave and current forces calculation, it is necessary to know the force acting on the glass sphere. Since, F_x is the predominant acting force and it is almost equal to its drag component, the force acting on the glass sphere is given by equation 3.20. Note that A is the glass projected area and the diameter of the glass sphere is $D = 0.5$ m. According to NASA (National Aeronautics and Space Administration), the sphere drag coefficient may vary between 0.5 and 0.7. For this application was chosen an intermediate value, 0.6 [48].

Therefore, the force acting in the glass sphere is given by

$$\begin{aligned}
 F_{x,sphere} &= \frac{1}{2} \rho C_D A u_{max}^2 \Rightarrow \\
 \Rightarrow F_{x,sphere} &= \frac{1}{2} \times 1025 \times 0.6 \times \frac{\pi \times 0.5^2}{4} \times 0.49^2 \\
 &= 14.50 \text{ N.}
 \end{aligned}$$

In Figure 3.24 is displayed the wave and current force vectors acting on cylinders and on the glass sphere.

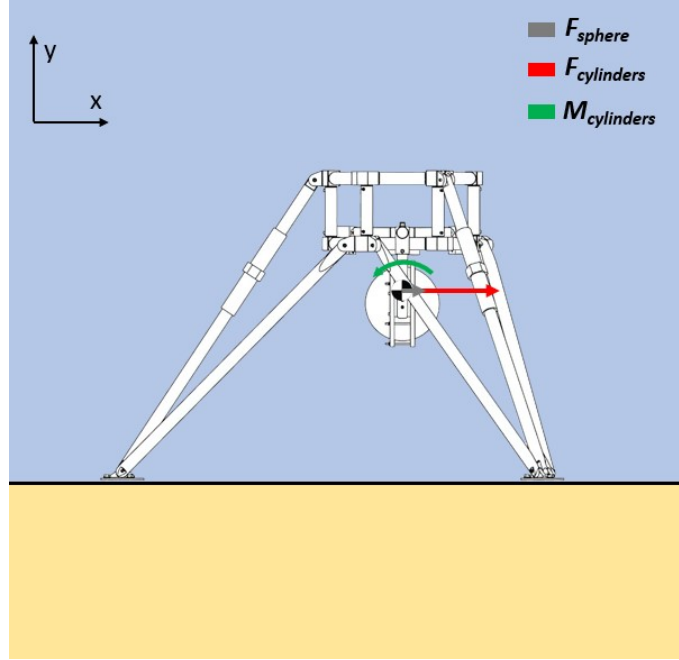


Figure 3.24 – Wave and current force vectors acting on cylinders and the glass sphere.

3.3 Foundation loading

According to figures 3.3 and 3.24, total force/moment vectors can be obtained by adding the force/moments vectors in each direction.

By adding the gravity, $F_{g,structure}$, and buoyancy loads, $F_{b,structure}$, in y axis direction,

$$\begin{aligned} F_y &= F_{g,structure} + F_{b,structure} \Rightarrow \\ \Rightarrow F_y &= -1555.87 + 1307.18 \\ &= -248.69 \text{ N.} \end{aligned} \quad (3.27)$$

By adding the water forces for cylinders and the glass sphere in x axis direction,

$$\begin{aligned} F_x &= F_{x,cylinders} + F_{x,sphere} \Rightarrow \\ \Rightarrow F_x &= \sum_{i=1}^{16} F_{x,max}^i + F_{x,sphere} \\ &= 97.78 + 14.50 \\ &= 112.28 \text{ N.} \end{aligned} \quad (3.28)$$

The total moment is equal to the equivalent moment caused by cylinder forces,

$$M_z = 48.86 \text{ Nm.} \quad (3.29)$$

In Figure 3.25 is displayed the total force vectors in x and y directions and moment vector.

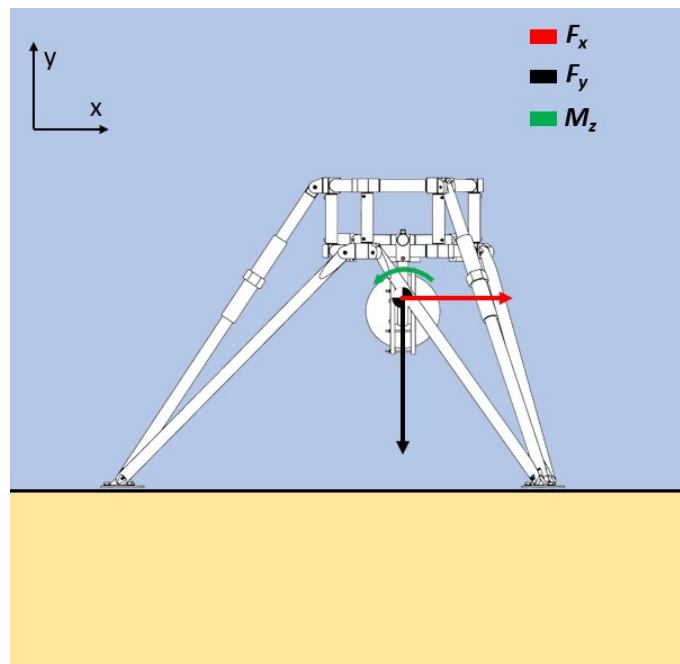


Figure 3.25 – Total force vectors in x and y directions.

To determine the forces acting on top of the buckets, it is necessary to know previously the reactions at the structure supports. To specify that, it was elaborated a free body diagram of all forces acting on the structure, shown in Figure 3.26. Note that point B stands for two supports.

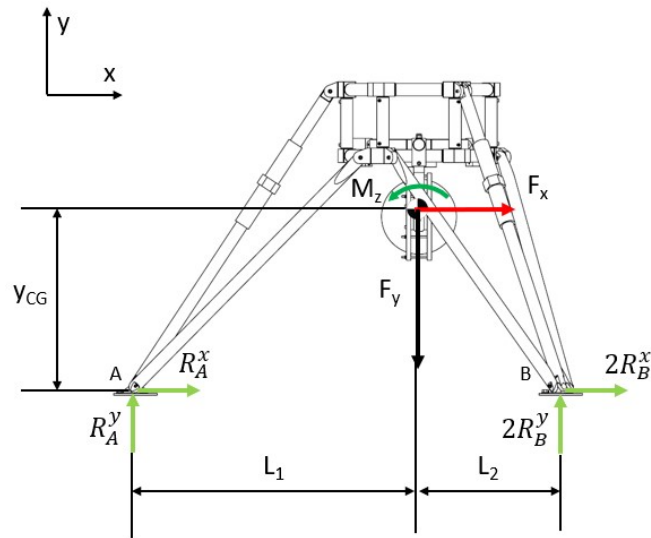


Figure 3.26 – Free body diagram.

Thereby, the equilibrium equations are written as,

$$\sum F_x = 0 \Rightarrow F_x + R_A^x + 2R_B^x = 0, \quad (3.30)$$

$$\sum F_y = 0 \Rightarrow -F_y + R_A^y + 2R_B^y = 0, \quad (3.31)$$

$$\sum M_A = 0 \Rightarrow M_z - F_x \cdot y_{CG} - F_y \cdot L_1 + 2R_B^y \cdot (L_1 + L_2) = 0. \quad (3.32)$$

However, there are 4 unknowns for 3 equilibrium equations.

This leads to a hyperstatic problem. Due to the lander complex geometry, the support reactions could be determined using a structural design software without doing any type of simplification.

To simplify the problem and obtain an isostatic problem (4 unknowns for 4 equilibrium equations) in order to determine an analytic solution, it was assumed that,

$$R_B^x = -\frac{F_x}{3}, \quad (3.33)$$

since later on it will be applied safety factors to the design loads.

The parameters values pictured in Figure 3.26 are gather in Table 3.11.

Table 3.11 – Free body diagram parameters values.

Parameter	
F_x	112.28 N
F_y	248.69 N
M_z	48.86 Nm
y_{CG}	1 m
L_1	1.64 m
L_2	0.82 m

Solving the system of equations in order to the unknowns,

$$\begin{cases} F_x + R_A^x + 2R_B^x = 0 \\ -F_y + R_A^y + 2R_B^y = 0 \\ M_z - F_x \cdot y_{CG} - F_y \cdot L_1 + 2R_B^y \cdot (L_1 + L_2) = 0 \\ R_B^x = -\frac{F_x}{3} \end{cases} \quad (3.34)$$

these are determined,

$$\begin{cases} R_A^x = -37.43 \text{ N} \\ R_A^y = 43.42 \text{ N} \\ R_B^x = -37.43 \text{ N} \\ R_B^y = 102.63 \text{ N} \end{cases} \quad (3.35)$$

The support reaction are shown in Figure 3.27.

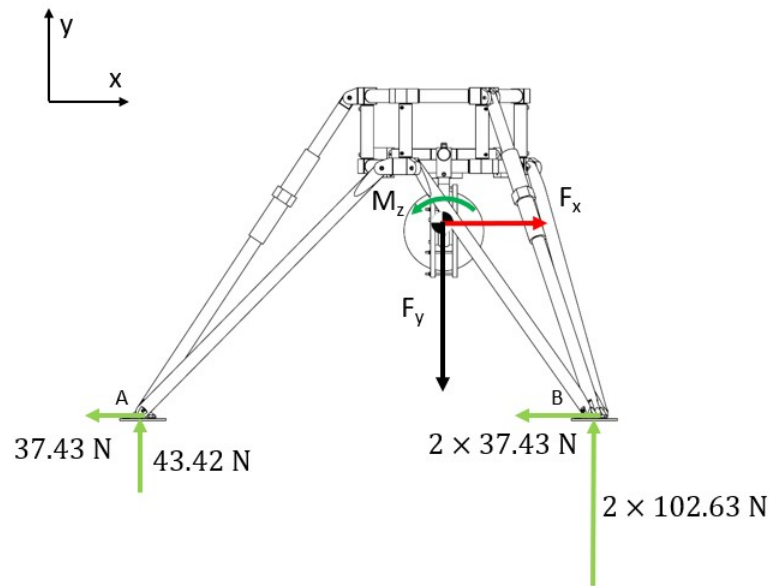


Figure 3.27 – Support reactions.

Applying Newton's third law, the forces acting on buckets top are equal in magnitude to the support reactions but with the opposite direction, as shown in Figure 3.28.

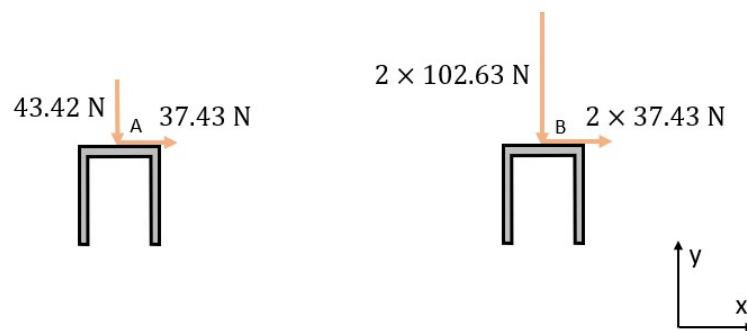


Figure 3.28 – Suction buckets top loads.

To achieve a conservative design, safety factors must be applied to these forces. In the next section, it will be used the most adequate safety factor to achieve design loads values.

3.3.1 Limit states

A limit state is a state beyond which a structure or structural component fails to fulfil the requirements of the design. The limit states generally considered for suctions buckets are [49]:

- Ultimate Limit State (ULS)
- Accident Limit State (ALS)
- Fatigue Limit State (FLS)
- Serviceability Limit State (SLS)

Ultimate Limit States (ULS) correspond to maximum load-carrying resistance. Can be considered as ULS the following examples:

- loss of structural resistance
- loss of static equilibrium of the structure, or part of it, considered as a rigid body, e.g. overturning
- failure of critical components of the structure caused by exceeding the ultimate resistance

Accident Limit States (ALS) correspond to the highest load-carrying capacity for unusual accidental loads or the post-accident integrity for structures that have been damaged.

Can be considered as ALS the following examples:

- structural damage caused by accidental loads
- ultimate resistance of damaged structures

Fatigue Limit States (FLS) correspond to structural failure caused by repeated/cyclic loading.

Can be considered as FLS the following examples:

- cumulative damage due to repeated loads

Serviceability Limit States (SLS) correspond to tolerance criteria applicable to normal use, generally the maximum allowable displacements and/or stiffness.

Can be considered as SLS the following examples:

- deflections that may alter the effect of the acting forces
- excessive vibrations producing discomfort or affecting non-structural components
- durability

3.3.2 Safety Method

In Europeans waters, a Load and Resistance Factor Design (LRFD) method is recommend for designing suction buckets [49]. This method approaches the target safety level by applying safety factors, γ_f , to characteristic values, such as characteristic load effect, S_k and characteristic load, F_k .

There are two methods for establishing the design load effect, S_d associated with a particular load, F.

1. The design load effect, S_d is obtained by multiplication of the characteristic load effect, S_k by a specified load factor, γ_f

$$S_d = \gamma_f S_k, \quad (3.36)$$

where the characteristic load effect S_k is determined in a structural analysis for the characteristic load, F_k .

2. The design load effect, S_d is obtained from a structural analysis for the design load, F_d is obtained by multiplication of the characteristic load, F_k by a specified load factor, γ_f

$$F_d = \gamma_f F_k. \quad (3.37)$$

The result of the structural analysis previously carried out was expressed as a characteristic load, F_k . Therefore, the second method was chosen to determine the design load, F_d .

The load factors should be selected according to the limit state for which the suction buckets are being designed and the type of loads acting on them. According to the previous type of limit states mentioned before, the suction bucket will be designed for a ULS since the main objective is to ensure that a static equilibrium is achieved even in the most hostile conditions.

Relatively to the type of loads acting on the structure, as seen in sections before, they are permanent (gravity and buoyancy loads) and variable and environmental (wave and current loads).

Thus, the highest partial safety factor will be chosen to ensure the structure safety.

According to Table 3.12 transcribed from DNV-ST-0437 standard code, the highest partial safety factor suitable for these conditions is [50]:

$$\gamma_f = 1.35.$$

Table 3.12 – Partial safety factors (DNV-ST-0437) [50].

ULS				FLS, ALS, SLS	
Permanent*		Variable and environmental		Permanent*	Variable
Favourable	Unfavourable	Normal	Abnormal****		
0.9**	1.1**	1.35***	1.1	1.0	1.0

* Permanent loads include dead loads and pretension loads for support structure. A load is favourable when a reduced value of this load results in an increased load effect in the structure.

** Factors for permanent loads in ULS may be taken as 1.0 if appropriate measures are taken.

*** May be reduced for specific design load cases.

**** Abnormal denotes situations with serious failures of unlikely environmental conditions.

Applying the partial design factor to the bucket top loads,

$$\begin{cases} F_A^x = -37.43 \times 1.35 = -50.53 \text{ N} \\ F_A^y = 43.42 \times 1.35 = 58.62 \text{ N} \\ F_B^x = -37.43 \times 1.35 = -50.53 \text{ N} \\ F_B^y = 102.63 \times 1.35 = 138.55 \text{ N} \end{cases} \quad (3.38)$$

The design loads for each suction bucket are shown in Figure 3.29.

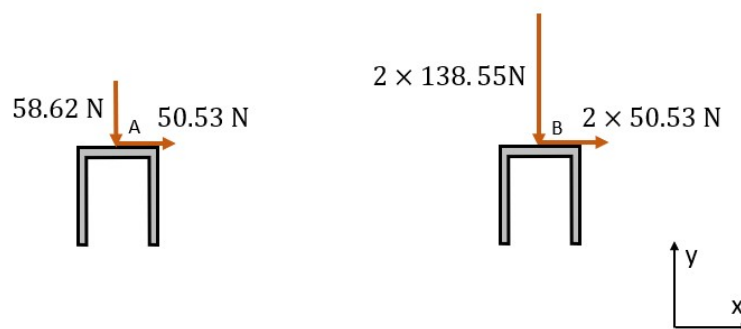


Figure 3.29 – Suction buckets design loads.

Chapter 4

Suction bucket design

This chapter begins by presenting the suction bucket and soil terminology for a better understanding of the design procedures. The suction bucket design process is described for the critical procedure stages and the final dimensions are calculated following an iterative method.

4.1 Suction bucket

To design a suction bucket it is necessary to know its terminology and the way it works.

4.1.1 Terminology

The widely used terms related to suction buckets are described in Figure 4.1.

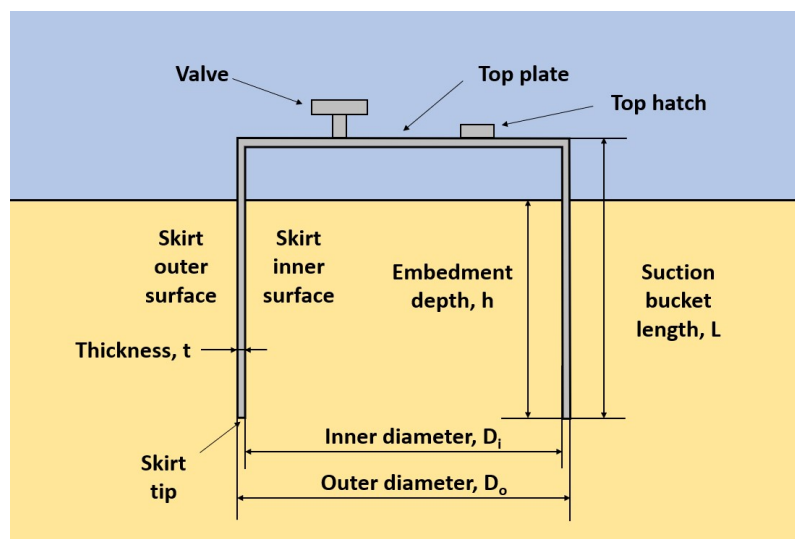


Figure 4.1 – Suction bucket terminology. Adapted from [51].

4.1.2 Procedure stages

The procedure stages concerning the suction bucket comprise all the phases from its release until its recovery. These may be divided into four stages:

- Deployment
- Installation
- In-service
- Decommissioning

The deployment stage corresponds to the release of the suction bucket to the marine site. Generally, steel wires assist the downward movement to grant an effective positioning. During this period, the valves are open to let the water flow out.

The installation stage is one of the critical stages. If this stage fails, the suction bucket can not be in its working position. There are two steps in the installation stage. The first step relates to foundation penetration into the soil by self-weight. During this step, the valves are kept open to avoid overpressure inside the buckets. When the self-weight penetration stops, the valves are closed and the second step is applied. In the second step, the water is pumped out of the bucket interior (suction-assisted penetration), which creates a net pressure difference that "sucks" (penetrates) the bucket into the soil to the targeted depth.

The in-service stage starts after the bucket achieves the targeted depth. Similarly to the previous one, this is also a critical stage since the suction bucket must withstand the loads transferred by the structure. The valves remain closed during this stage.

The decommissioning stage is applied when it is no longer necessary to have a structure and its foundation doing their job. During this stage, it detaches from the seabed since water is pumped back inside the bucket. During this operation, the valves are closed.

A ROV generally opens and closes all the valves. The pump system can be incorporated into the ROV or it can be an independent system.

These 4 stages are represented in Figure 4.2.

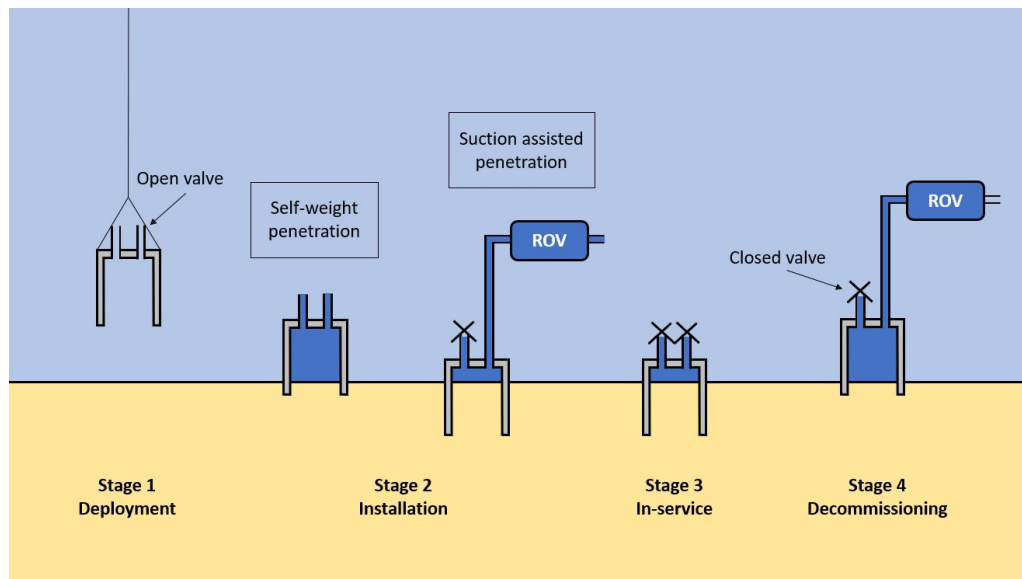


Figure 4.2 – Procedure stages.

4.2 Soil

To better understand soil behaviour, its properties and the different types were studied.

4.2.1 Terminology

Soils consist of three phases (solid, liquid and gas), as shown in Figure 4.3. There are several parameters that describe their physical state that are not used in the study of other materials. These quantities are defined in Table 4.1 and their definition involves the volumes of the three phases (solid volume, V_s ; liquid volume, V_w ; gas volume, V_a) and the weights of the solid and liquid phases (respectively W_s and W_w). The weight of the gaseous phase is considered negligible. Note that the volume of voids, V_v , is the sum of liquid and gas phases volumes [52].

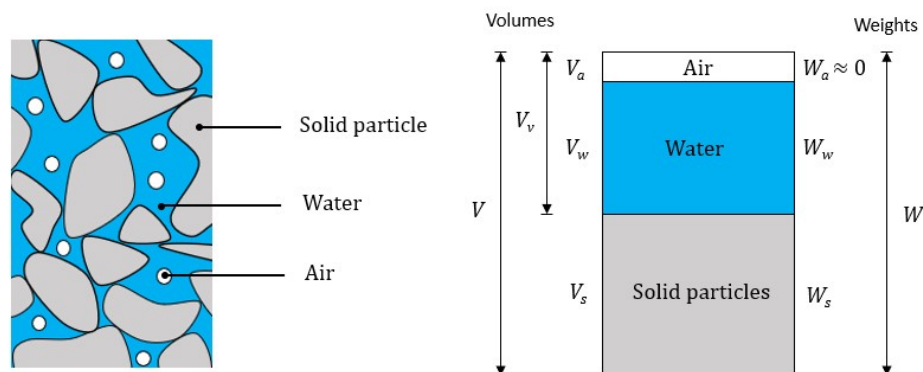
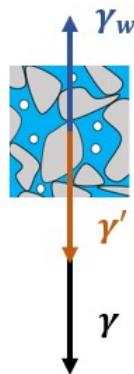


Figure 4.3 – Soil phases and parameters.

Table 4.1 – Definition of soil parameters.

Parameter	Symbol	Definition	Units	Observations
Void ratio	e_v	$\frac{V_v}{V_s}$	-	-
Porosity	n	$\frac{V_v}{V}$	-	Express in %.
Degree of saturation	S_r	$\frac{V_w}{V_v}$	-	Express in %.
Water content	w	$\frac{W_w}{W_s}$	-	Express in %.
Unit weight	γ	$\frac{W_s + W_w}{V}$	kN/m ³	When $S_r = 100$ %, it is called saturated unit weight, γ_{sat} .
Dry unit weight	γ_d	$\frac{W_s}{V}$	kN/m ³	-
Submerged unit weight	γ'	$\gamma - \gamma_w$	kN/m ³	γ_w is water unit weight.
Solid particles unit weight	γ_s	$\frac{W_s}{V_s}$	kN/m ³	Usually between 25.5-27.5 [52].
Specific Gravity of Solid Particles	G_s	$\frac{\gamma_s}{\gamma_w}$	-	Usually between 2.6-2.8 [52].

To better understand the definition of γ' , it is important to highlight that 1 m³ of submerged soil is subjected to buoyancy effects equal to γ_w ; the submerged unit weight is the result of gravity and effective forces acting in 1 m³ of soil, as shown in Figure 4.4.

Figure 4.4 – Submerged unit weight, γ' , definition.

All defined quantities are related to each other. For example, the higher the void ratio, the higher the porosity and vice versa. The greater these two quantities are, the lower the unit weight, dry unit weight, and submerged unit weight. The higher the water content for a given void ratio, the higher the degree of saturation and the unit weight will be [52].

4.2.2 Types of soil

The soil behaviour differs between high-permeable and low-permeable soils. Both have a different response when suction-assisted penetration starts at the installation stage.

In high-permeable soils (sand), a seepage flow is generated during the assisted penetration, decreasing the effective stresses and reducing the friction at the skirt's inner and outer sides and tip.

In low-permeable soils (clay), it is not developed a seepage flow as the low permeability avoids any flow regime within the soil layer during installation. Therefore, it is not induced a substantial change in the soil resistance. In this case, the installation process is only due to the pressure difference that provides an effective downward force to push the bucket into the clay.

Figure 4.5 represents the different types of soil behaviour during installation.

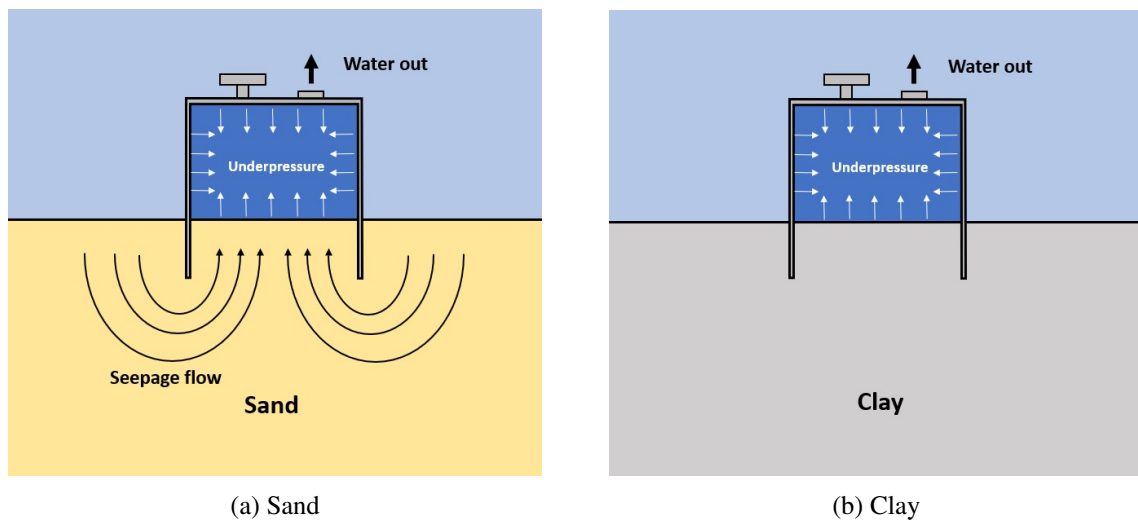


Figure 4.5 – Installation in different types of soil.

Before starting the suction bucket design, knowing the most probable type of soil the lander will face during installation is fundamental. It is crucial since the design methods for the two soil types are distinct, according to Byron Byrne and Guy Houlsby [53, 54].

As seen in section 3.2.2.3, the Leixões coast was the marine site chosen for this study since it poses the most adverse sea conditions in the Portuguese coast.

Thus, it was possible to find the substrate type using the EUSeaMap 2021, a comprehensive and ready-to-use broad-scale seabed habitat map of Europe. The 2021 edition is the fifth iteration and was created based on previous iterations [55]. Figure 4.6 presents this map.

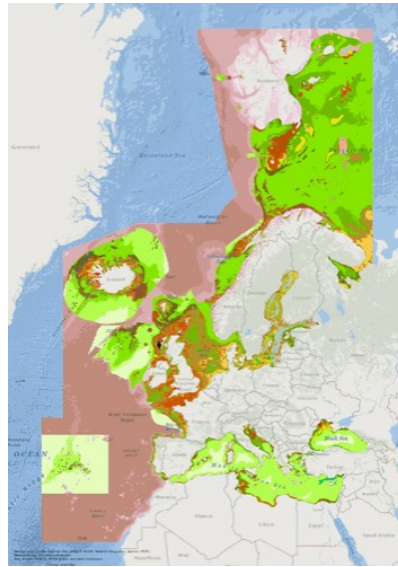


Figure 4.6 – EUSeaMap2021 [55].

A closer view is possible due to the detailed map of EUSeaMap2021 shown in Figure 4.7.

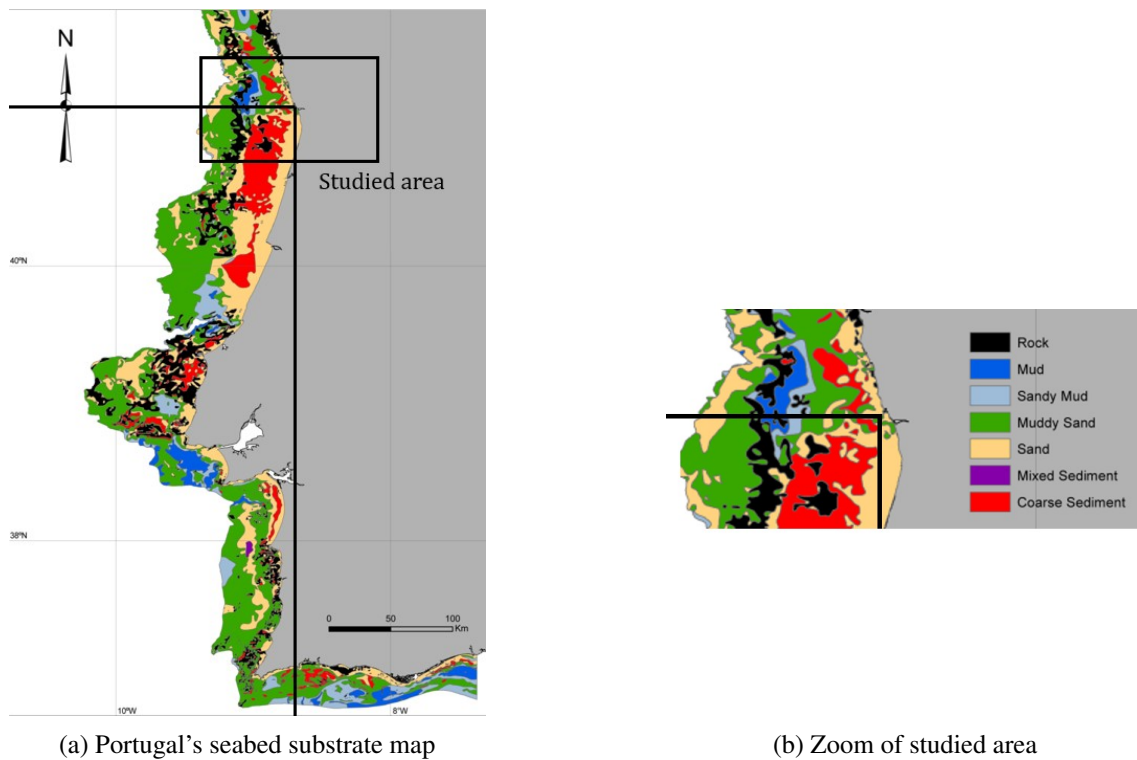


Figure 4.7 – Detailed map of EUSeaMap 2021.

As seen in the zoom image, the Leixões coast is mainly composed of muddy sand (green), sand (yellow) and coarse sediments (red). Therefore, installation design procedures in the sand were chosen for designing the suction buckets.

4.2.3 Stresses

Since soil masses are made up of particles, the notion of stress is more complex than in continuous environments, whether solid or fluid.

Figure 4.8 shows a soil mass in static equilibrium under the action of its own weight and forces applied to the surface. The system origin is located at the surface. The vertical axis is the z -axis with a positive downward direction. The x -axis will be considered in the plane of the figure and the y -axis perpendicular to it.

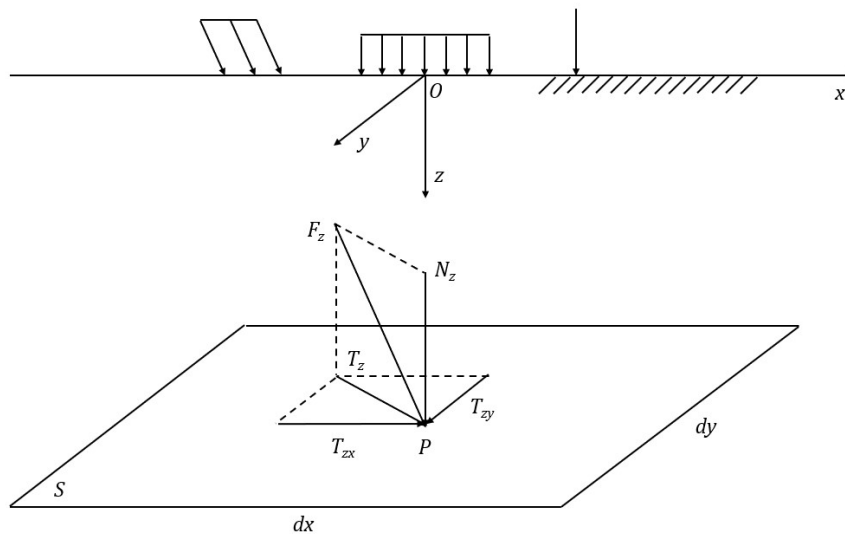


Figure 4.8 – Transmitted forces through a generic section in an emerged soil mass [52].

A horizontal section, S , in the immediate vicinity of a point P is considered. It is assumed that this section has dimensions dx and dy small enough that it is acceptable to admit that the force acting on it is uniformly distributed, but large enough that they are much larger than the diameter of the soil particles. It is also considered that S makes small deviations from its median plane so that it does not section any particle, but rather passes through the points of contact.

Through section S one part of the soil mass transmits a force, F_z , to the other, this force is transmitted through the points of contact. It can be decomposed into a normal force, N_z , and a tangential force, T_z , to the section. T_z can also be decomposed into two forces on the plane containing S , parallel to the Ox and Oy axes, respectively, T_{zx} and T_{zy} .

The stresses in section S are defined as:

$$\sigma = \sigma_z = \frac{N_z}{dx dy}, \quad (4.1)$$

$$\tau_z = \frac{T_z}{dx dy}, \quad (4.2)$$

$$\tau_{zx} = \frac{T_{zx}}{dx dy}, \quad (4.3)$$

$$\tau_{zy} = \frac{T_{zy}}{dx dy}. \quad (4.4)$$

From these equations it can be concluded that the stress concept in soil mechanics is macroscopic, since stresses are obtained by dividing the forces applied by the total area. In soil mechanics context, normal compressive stresses are considered positive.

It is assumed hereafter that the soil mass is totally submerged and the water pressure of the soil pores in point P has a value of u , see Figure 4.9.

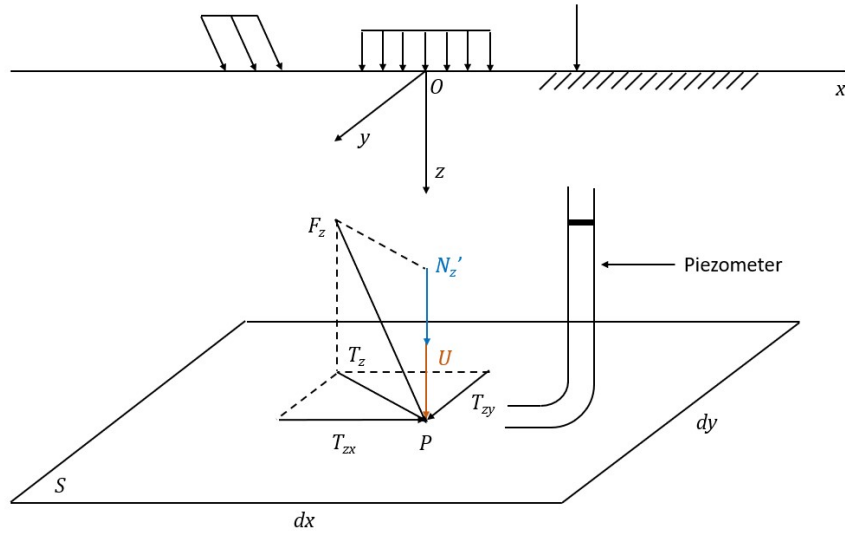


Figure 4.9 – Transmitted forces through a generic section in a submerged soil mass [52].

This value can be measured using a piezometer. Considering that the pores represent almost the entire area of the section, a force U is transmitted through the pore water and can be written as

$$U = u dx dy. \quad (4.5)$$

Note that the force is entirely normal, since in fluids the stresses act in the normal direction for every section. Hence, the portion of the total normal force transmitted due to particle contacts will be the force N'_z ,

$$N'_z = N_z - U. \quad (4.6)$$

Dividing both terms by $dx dy$,

$$\frac{N'_z}{dx dy} = \frac{N_z}{dx dy} - \frac{U}{dx dy}, \quad (4.7)$$

and attending the equations 4.1 and 4.5,

$$\frac{N'_z}{dx dy} = \sigma - u. \quad (4.8)$$

The first member of the equation is consistent with the concept of stress induced above for soils (and called macroscopic): it represents the force transmitted by the contacts between particles divided by the cross-sectional area. This stress is called effective stress, σ' . Thus,

$$\sigma' = \sigma - u. \quad (4.9)$$

This equation represents the principle of effective stress: in soils the effective stress is equal to the total stress minus the pore water pressure.

However, the stresses in soil masses can be seen through another perspective. These can be divided in geostatic and induced stresses. The geostatic stresses are the natural stresses existing in rock and soil structure due to self-weight. The induced stresses are associated with actions imposed by structures over or inside the soil masses.

4.2.3.1 Geostatic stresses

Figure 4.10 shows an homogeneous soil mass of horizontal surface with unit weight, γ , constant in depth. It is considered that the groundwater is at rest with its upper level coincident with the ground surface. It is also considered an infinitesimal neighborhood of P with three pairs of faces perpendicular to the coordinate axes (in the figure only one facet is represented for each mentioned pair). On these faces the tangential stresses are zero, that is, the stresses are purely normal; the faces are called principal faces, the stresses acting on them are called principal stresses; the directions of these stresses are the principal stress directions.

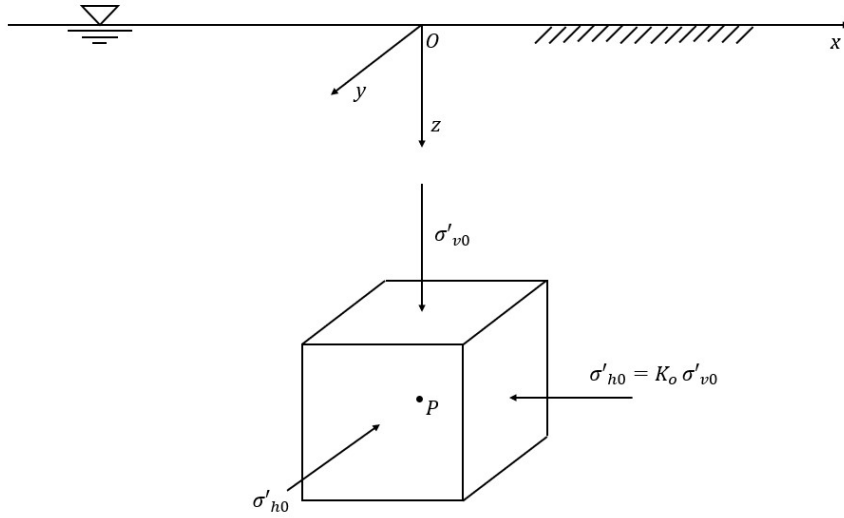


Figure 4.10 – Principal effective geostatic stresses [52].

The total vertical stress in P acts on the horizontal faces and is

$$\sigma_{v0} = \gamma z. \quad (4.10)$$

The "zero" index will be used to symbolize that the stress is exclusively due to its own weight, that is, it is a geostatic stress. On the other hand, with the water in hydrostatic conditions and the phreatic level at the ground surface, the pore water pressure at P is

$$u_0 = \gamma_w z, \quad (4.11)$$

therefore, the effective vertical geostatic stress is

$$\sigma'_{v0} = (\gamma - \gamma_w)z = \gamma' z, \quad (4.12)$$

in which γ' is the submerged unit weight.

On the vertical faces the horizontal stresses will naturally be equal to each other. Unlike the vertical stresses, their calculation does not result from purely gravitational considerations. For this purpose it is necessary to introduce a parameter - the resting impulse coefficient, K_0 - which is equal to the ratio of the effective horizontal geostatic stress to the effective vertical geostatic stress at the same point:

$$K_0 = \frac{\sigma'_{h0}}{\sigma'_{v0}}. \quad (4.13)$$

Thus, the effective and total horizontal geostatic stresses are, respectively:

$$\sigma'_{h0} = K_0 \sigma'_{v0} = K_0 \gamma' z, \quad (4.14)$$

$$\sigma_{h0} = \sigma'_{h0} + u_0 = K_0 \gamma' z + \gamma_w z. \quad (4.15)$$

The K_0 coefficient depends essentially on the geological history of the soil mass, which determines the evolution of the stresses experienced by it. Therefore, there are no theoretical methods for the determination of K_0 .

Its evaluation in each case is, therefore, carried out through tests, such as *in situ* tests, based on empirical correlations with other parameters.

However, it can be said that in geologically very recent sedimentary soil masses, K_0 is well below 1, often not far from 0.5 [52].

4.2.3.2 Induced stresses

Ground-supported structures will modify the geostatic stress state in a given region and its surroundings. This problem can be solved considering the equilibrium, compatibility and constitutive equations. There are several solutions to solve this problem, being the Boussinesq and Flamant the most used analytical solutions in soil mechanics. In both solutions it is possible to deduce that a ground loading enhances the vertical stress.

However, in this study other type of solutions will be used to represent this stress enhancement.

4.2.4 Mohr-Coulomb failure criteria

The majority of soils experience failure according to Mohr-Coulomb criteria. According to this criteria, a material (in this case, a soil) breaks at a point when a given ratio between the normal component and the tangential component of stress is surpassed. This ratio is expressed by:

$$\tau_f = c' + \sigma'_f \tan \phi', \quad (4.16)$$

in which ϕ' is the angle of friction of soil or angle of shear resistance and c' is a parameter that quantifies the cohesion of the soil. The sub-index f of stress components refers to failure. The use of c' and ϕ' symbols (and not simply c and ϕ) results from the fact that the Mohr-Coulomb criteria only applies to effective stresses. Hence, these parameters are also called effective cohesion and effective angle of friction of soil [52].

The Figure 4.11 represents the Mohr circle for a stress state and the respective Mohr-Coulomb failure criteria envelope.

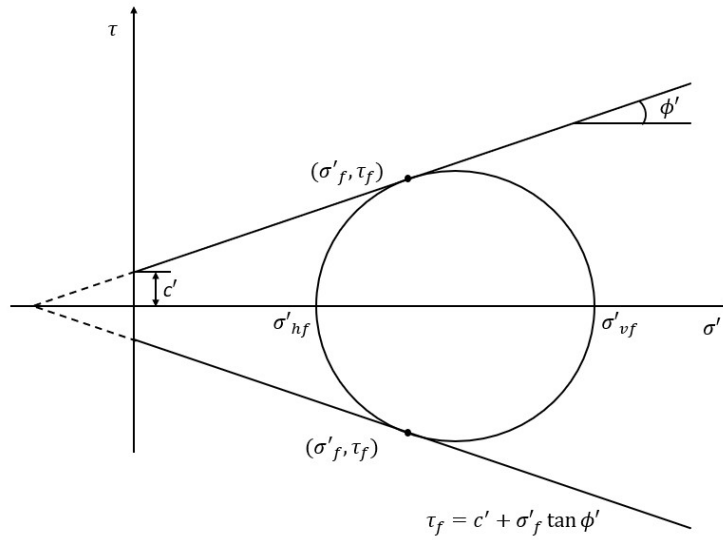


Figure 4.11 – Soil failure for a stress state according to Mohr-Coulomb criteria [52].

Note that the envelope on the left side of the vertical axis has been interrupted, because soils have practically no tensile strength.

The soil resistance parameters, c' and ϕ' can be obtained by analysing experimental results obtained by triaxial shear tests or direct simple shear tests (DSS), for example [52]. For sand and granular soils it is known that cohesion is equal to zero, $c' = 0$.

Depending on whether the sand is more or less compacted during triaxial tests, it presents two angles of friction of soil: peak, ϕ' , and the constant volume, ϕ'_{cv} , angles. In Figure 4.12 this phenomena is shown for the compacted sand sample since the difference between the two angles is bigger. For loose sand the peak, ϕ' , and the constant volume, ϕ'_{cv} , angles are equal since no significant peak stress is verified in the triaxial compression test.

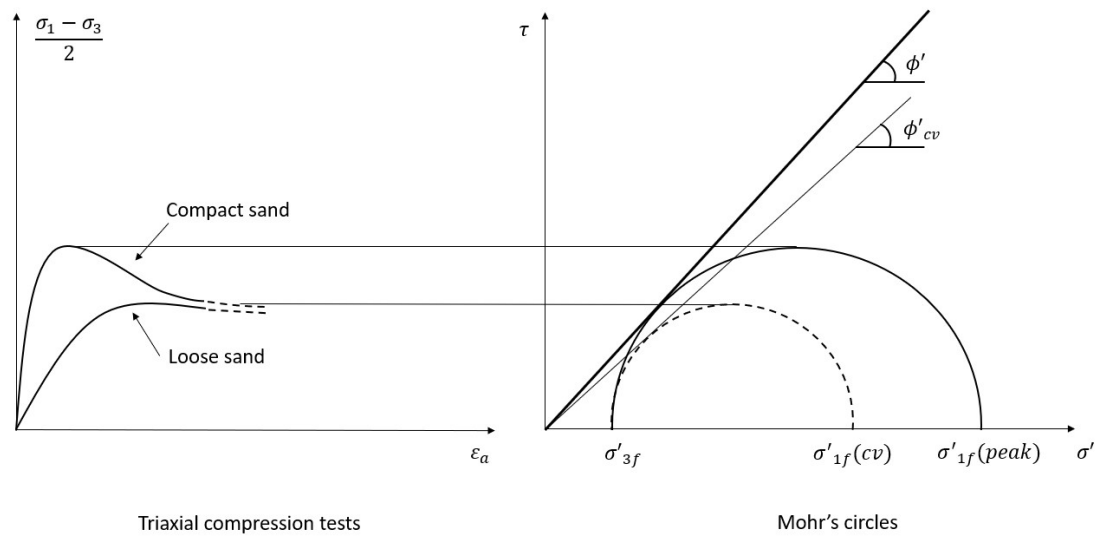


Figure 4.12 – Triaxial compression tests and Mohr circles of compact and loose sand tests.

Since it was not possible to obtain a sample of the sand at 200 meters depth because it is a complex and time consuming task, tabled values must be considered.

In Table 4.2 values for the two types of angles of friction of soil are shown.

Table 4.2 – Values for peak and constant volume angles of friction for granular soils [52].

Soil type	$\phi'_{cv} (^\circ)$	$\phi' (^\circ)$	
		Medium compact	Compact
Silts (not plastics)	26	28	30
Fine to medium sand badly graded	26 to 30	30 to 34	32 to 36
Sand well graded	30 to 34	34 to 40	38 to 46
Sand and gravel	32 to 36	36 to 42	40 to 48

However, to fully define the Mohr-Coulomb failure criteria, it is necessary to determine the dilatancy angle, ψ . This angle expresses the volume change with shear stresses and it is correlated with the peak, ϕ' , and the constant volume, ϕ'_{cv} angles by [52]:

$$\phi' = \phi'_{cv} + \psi. \quad (4.17)$$

4.3 Installation

The installation design calculations are important to:

- Predict the response to vertical load without the application of suction when the self-weight of the caisson and structure are applied, i.e the "self-weight penetration".
- Predict the suction-penetration response, i.e the suction pressures required to reach target penetration.
- Anticipate specific risks to successful installation and put in place appropriate measures to reduce those risks.

The design calculations are based on "Suction Installed Caisson Foundations for Offshore Wind: Design Guidelines" [51].

The basic equation for the installation calculations for (a) and (b) is:

$$V' + s \cdot A_{caisson} = R_{inside} + R_{outside} + R_{tip}, \quad (4.18)$$

where V' is the effective vertical load (accounting for buoyancy), s is the suction applied during suction bucket installation (pressure outside minus pressure inside caisson), $A_{caisson}$ is the internal top area of the caisson and R is the soil resistance acting on the bucket.

The results of the installation calculations are usually presented in graphical format, including a plot of penetration resistance against depth and a plot of required suction against penetration depth, as shown in Figure 4.13.

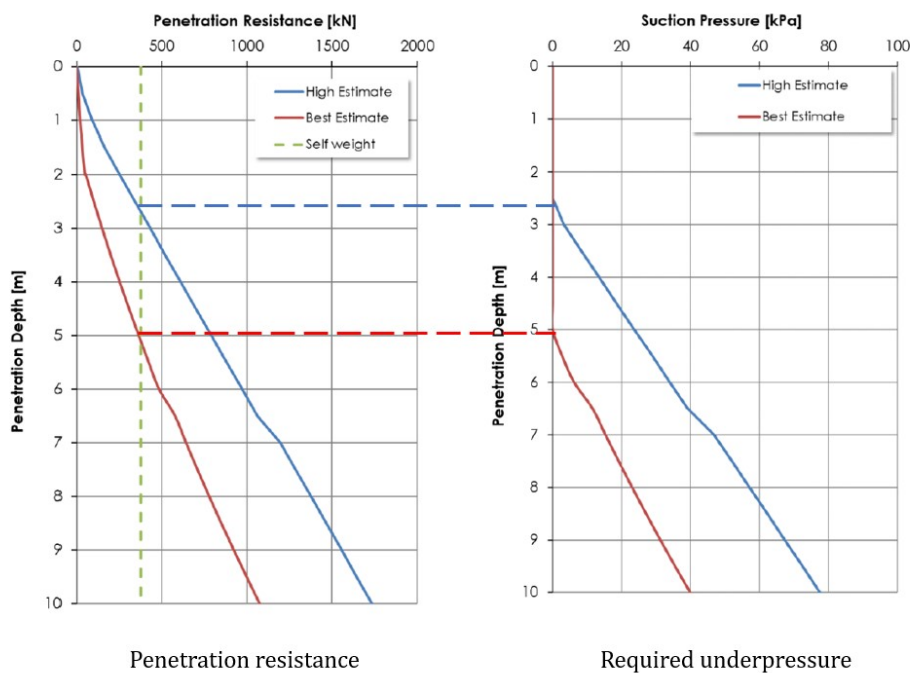


Figure 4.13 – Example penetration resistance plots. Adapted from [51].

There are two main types of calculation methods for caisson installation. The first is based on using geotechnical properties of the soil, determined either by *in-situ* tests or laboratory tests, together with an approximate mechanism-based calculation of the relevant loads on the caisson. The second is based on using cone penetration test (CPT) data. Empirical factors are applied directly to CPT measurements to convert these directly to estimate the loads on the caisson.

4.3.1 Mechanism-based methods for sand

The mechanism-based methods use the standard geotechnical parameters determined by in-situ or laboratory testing.

For sand (case in study) these parameters are:

- the angle of friction, ϕ' ;
- the submerged unit weight of the soil, γ' ;
- the soil permeability, k_s (for flow rate calculations).

The design methods are based on essentially the same principles as those used for estimating the capacity of driven piles: the vertical load on the bucket is resisted by friction on the sides (inside and outside) together with end bearing on the annular rim.

4.3.2 CPT-based methods

The cone penetration test consists of piezocone, a cylindrical penetrometer with a 60° conical tip, to which a load cell is attached. This measures the resistance to penetration, which is usually expressed as the force per unit area, q_c [56], as shown in Figure 4.14.

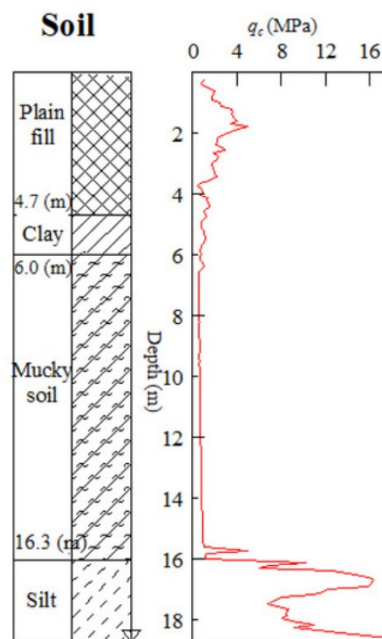


Figure 4.14 – CPT test. Adapted from [57].

With this result, the relation between self-weight/suction penetration with depth can be deduced after applying friction factors.

Since collecting an *in-situ* sample or beginning a CPT test at 200 meters are time consuming tasks within the project period, neither these methods can be accurately used since no empirical data is available. However, the mechanism-based method will be used since it is easier to find similar sand properties than find similar CPT tests for the region in study.

Table 4.3 shows the sand properties used in this study. It was considered that the sand was loose and well graded because the seabed is a soil geological recent nearshore. Therefore, the minimum value for ϕ_{cv} was chosen from Table 4.2. Furthermore, the angles of soil friction were assumed to be equal. A typical value for submerged unit weight, γ' , was also used [53]. Since values for the permeability factor vary between 2.0 to 3.0, an average value was used, $k_s = 2.5$ [53]. The dilatancy angle, ψ , can be determined using equation 4.17.

Table 4.3 – Assumed sand properties.

Parameter	
$\phi'(^{\circ})$	30 °
$\phi'_{cv}(^{\circ})$	30 °
γ'	8.5 kN/m ³
k	2.5
ψ	0

4.3.3 Self-weight penetration (SWP)

During this phase, the weight of the suction bucket is responsible for its penetration. The maximum depth is achieved when the soil friction equals the weight of the suction bucket. V' is the net downward vertical load and corresponds to the weight of the structure and caisson less the buoyancy effects that is supported by one of the three caissons.

The soil resistance on the caisson is calculated as the sum of friction outside and inside of the skirt, and at the end bearing on the caisson tip. The frictional terms are calculated by considering the vertical effective stress adjacent to the caisson and then assuming that horizontal effective stress is the vertical effective stress times a factor K . Assuming that the angle of friction between the caisson wall and the soil is δ , the shear stress acting on the caisson is $\sigma'_v K \tan \delta$ [53], as shown in Figure 4.15.

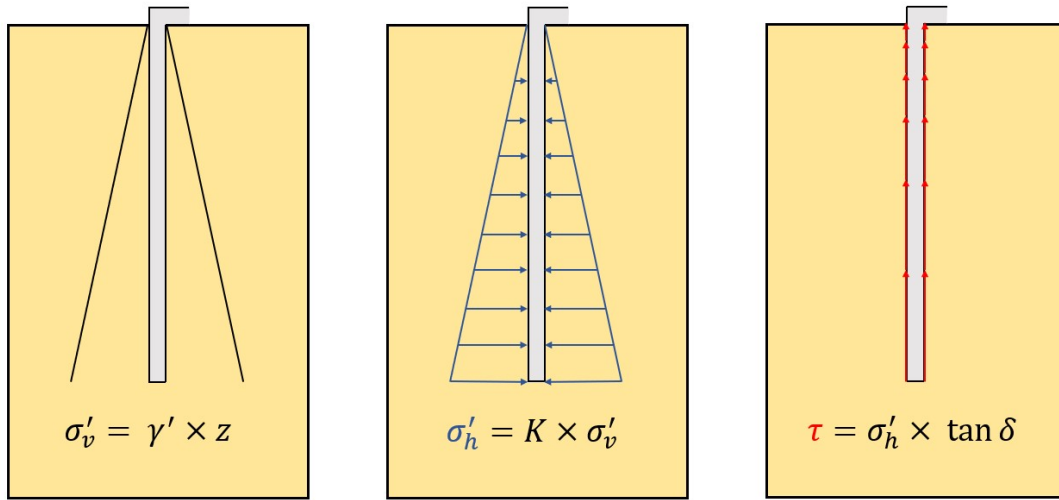


Figure 4.15 – Stress correlations.

Assuming that:

- the vertical effective stress is constant across the section of the caisson;
- there is a zone between diameters D_o and $D_m = mD_o$ in which the vertical stress is enhanced through the action of the downward friction from the caisson;
- within this zone the enhanced vertical stress does not vary with radial coordinate;
- there is no shear stress on vertical planes at diameter D_m .

For the case where m is taken as constant, the equilibrium equation can be expressed as [53]:

$$\begin{aligned}
 V' = & \gamma' Z_o^2 \left[\exp\left(\frac{h}{Z_o}\right) - 1 - \left(\frac{h}{Z_o}\right) \right] (K \tan \delta)_o (\pi D_o) \\
 & + \gamma' Z_i^2 \left[\exp\left(\frac{h}{Z_i}\right) - 1 - \left(\frac{h}{Z_i}\right) \right] (K \tan \delta)_i (\pi D_i) \cdot \\
 & + \sigma'_{\text{end}} (\pi D t)
 \end{aligned} \quad (4.19)$$

The stress enhancement referred in section 4.2.3.2 is expressed by using a parameter Z . The inner, Z_i , and outer, Z_o , parameters can be expressed as:

$$Z_i = \frac{D_i}{4(K \tan \delta)_i}, \quad (4.20)$$

$$Z_o = \frac{D_o (m^2 - 1)}{4(K \tan \delta)_o}. \quad (4.21)$$

Assuming a symmetrical stress distribution on the caisson tip, the effective end bearing stress, σ'_{end} , is expressed as:

$$\sigma'_{\text{end}} = \sigma'_{\text{vo}} N_q + \gamma' t N_\gamma, \quad (4.22)$$

where N_q is the bearing capacity factor (overburden) and N_γ is the bearing capacity factor (self-weight).

These factors depend exclusively on the sand angle of friction, ϕ , and can be expressed as [58]:

$$N_q = e^{\pi \tan \phi} \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2} \right), \quad (4.23)$$

$$N_\gamma = e^{\frac{1}{6}(\pi + 3\pi^2 \tan \phi)} (\tan \phi)^{\frac{2\pi}{5}}. \quad (4.24)$$

For the first iteration calculation, the following dimensions were taken as the suction buckets dimensions. The diameter chosen was similar to the diameter of the ballast weight, whereas the length was assumed to be almost twice the diameter to overcome any erosion problems that may occur in the surrounding sand, such as scouring, leading to less structural support. In Table 4.4 the suction bucket dimensions are presented.

Table 4.4 – Suction bucket dimensions (first iteration).

Parameter	
Diameter (D)	0.495 m
Length (L)	1 m
Thickness (t)	5 mm
Inner diameter (D_i)	0.49 m
Outer diameter (D_o)	0.5 m
Volume (V)	0.00872 m ³

With the information of Tables 4.3 and 4.4, it is possible to determine the factors from equation 4.19. Predictions depend significantly on the choice of the factor m , and values of approximately 1.5 have been found to be appropriate [51]. Hence, $m = 1.5$ will be used in this study. The foundation-soil interface angle of friction, δ , can be correlated with the soil angle of friction, ϕ' , depending on suction bucket material and roughness. Table 4.5 shows the different values for the ratio δ/ϕ' [58].

Table 4.5 – Ratio δ/ϕ' [58].

Interface	δ/ϕ'
Sand/rough concrete (bored pile)	1.0
Sand/smooth concrete (driven pile)	0.8 to 1.0
Sand/rough steel (corrugated steel)	0.7 to 0.9
Sand/smooth steel	0.5 to 0.7
Sand/timber	0.8 to 0.9

Steel was the material chosen for this application, however, its roughness is unknown. Assuming that the steel pipe is provided with a smooth finish, the lowest ratio was used, therefore $\delta/\phi' = 0.5$ is the ratio chosen.

Since the seabed soil was considered geologically recent, the factor relating vertical stress to horizontal stress, K , is assumed to be approximately equal to K_0 , the resting impulse coefficient, i.e $K = K_0 = 0.5$.

Table 4.6 shows the values of the equation parameters for this application.

Table 4.6 – Values of the equation parameters (first iteration).

Parameter	
N_q	18.40
N_γ	14.62
m	1.5
K	0.5
ϕ'	30°
δ/ϕ'	0.5
δ	15°
$(K \tan \delta)_i$	0.13
$(K \tan \delta)_o$	0.13
Z_i	0.94
Z_o	1.20

V' , as mentioned before, is the downward vertical force that corresponds to the weight of the lander (structure and caissons) acting on one bucket less the buoyancy effects. The structural weight and buoyancy effects were calculated in section 3.2.1. Since the type of steel will be chosen in the end of the stress calculations, an average value for its density was chosen, $\rho_{steel} = 7900 \text{ kg/m}^3$ [59].

Gravity, $F_{g,bucket}$, and buoyancy, $F_{b,bucket}$, loads of each caisson can be expressed as:

$$\begin{aligned}
F_{g,bucket} &= \rho_{steel} \times V_{bucket} \times g \Rightarrow \\
\Rightarrow F_{g,bucket} &= 7900 \times 0.00872 \times 9.81 \\
&= 675.79 \text{ N},
\end{aligned} \tag{4.25}$$

$$\begin{aligned}
F_{b,bucket} &= \rho_{sea \text{ water}} \times V_{bucket} \times g \Rightarrow \\
\Rightarrow F_{b,bucket} &= 1025 \times 0.00872 \times 9.81 \\
&= 87.68 \text{ N}.
\end{aligned} \tag{4.26}$$

Thus, the total gravity and buoyancy loads acting on the lander with the buckets is given by:

$$\begin{aligned}
F_{g,lander} &= F_{g,structure} + 3 \times F_{g,bucket} \\
\Rightarrow F_{g,lander} &= 1555.87 + 3 \times 675.79 \\
&= 3583.24 \text{ N},
\end{aligned} \tag{4.27}$$

$$\begin{aligned}
F_{b,lander} &= F_{b,structure} + 3 \times F_{b,bucket} \\
\Rightarrow F_{b,lander} &= 1307.18 + 3 \times 87.68 \\
&= 1570.22 \text{ N}.
\end{aligned} \tag{4.28}$$

Hence,

$$\begin{aligned}
V' &= (F_{g,lander} - F_{b,lander}) / 3 \\
\Rightarrow V' &= (3583.24 - 1570.22) / 3 \\
&= 671.01 \text{ N}.
\end{aligned} \tag{4.29}$$

To solve equation 4.19 in order to the embedment depth, h , it was created a MATLAB script since its solution requires an iterative method. The script is shown in Appendix D.

After running the script for the parameters previously determined, the graph presented in Figure 4.16 was obtained. Note that the y-axis is represented with a reverse direction.

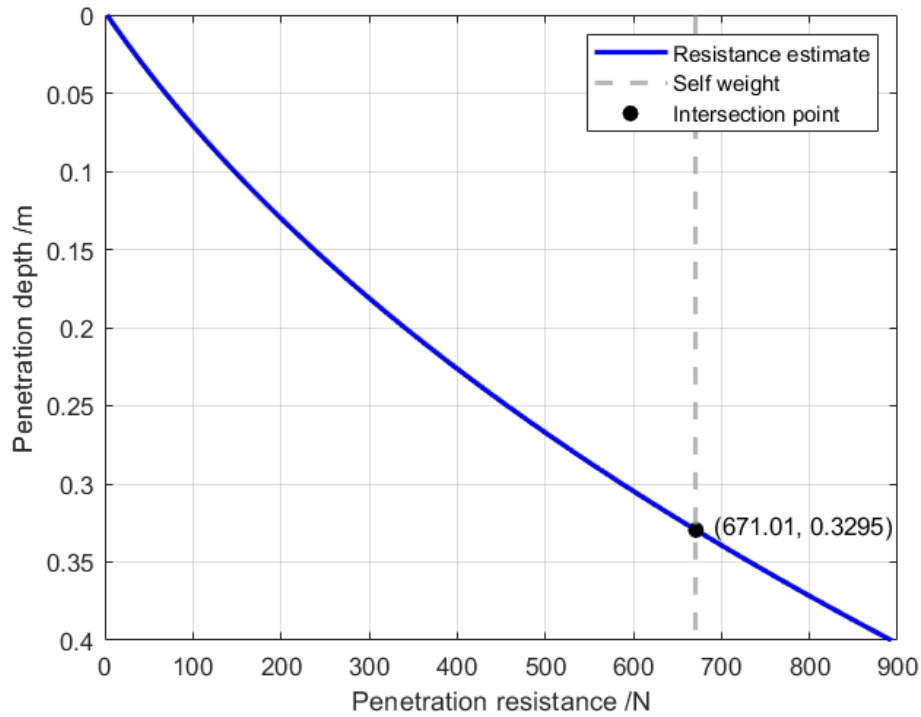


Figure 4.16 – Self-weight penetration design graph (first iteration).

According to Figure 4.16, the suction buckets achieve an embedment depth, h , of approximately 33 cm due to their own weight plus the structure weight.

4.3.4 Suction penetration

After the self-weight penetration phase is concluded, the suction phase starts in order to achieve the final embedment depth, i.e the in-service position. The pressure inside the caisson is represented as s with respect to the ambient seabed water pressure. Therefore, the absolute pressure in the caisson is $p_a + \gamma_w h_w - s$, where p_a is the atmospheric pressure.

In this design phase, it was assumed that:

- the excess pore pressure at the tip of the caisson is as . The factor a is the ratio of excess pore pressure at tip of caisson skirt to beneath caisson base. Thus, the absolute pressure is $p_a + \gamma_w(h_w + h) - as$;
- the distribution of pore pressure on the inside and outside of the caisson is linear with depth;
- the internal vertical effective stress is reduced sufficiently so that the failure mechanism involves movement of soil entirely inwards.

For the special case where m is taken as a constant, the equilibrium equation can be expressed as [53]:

$$\begin{aligned}
V' + s \left(\frac{\pi D_i^2}{4} \right) &= \left(\gamma' + \frac{as}{h} \right) Z_o^2 \left[\exp \left(\frac{h}{Z_o} \right) - 1 - \left(\frac{h}{Z_o} \right) \right] \\
&\times (K \tan \delta)_o (\pi D_o) \\
&+ \left(\gamma' - \frac{(1-a)s}{h} \right) Z_i^2 \left[\exp \left(\frac{h}{Z_i} \right) - 1 - \left(\frac{h}{Z_i} \right) \right] \\
&\times (K \tan \delta)_i (\pi D_i) \\
&+ \left\{ \left[\gamma' - \frac{(1-a)s}{h} \right] Z_i \left[\exp \left(\frac{h}{Z_i} \right) - 1 \right] N_q + \gamma' t N_\gamma \right\} \\
&\times (\pi D t).
\end{aligned} \tag{4.30}$$

Note that the vertical stresses inside and outside the bucket are the same as before, however γ' is replaced by $\gamma' + as/h$ outside the caisson and by $\gamma' - (1-a)s/h$ inside the bucket.

As suction is applied there is the possibility that the sand within the caisson might become loosened and thus exhibit a higher permeability, leading to changes in the factor a . Thus, this factor is function of h/D . Permeability can be expressed as being the ratio k_f , between the inner permeability, k_i , and the outer permeability, k_o , $k_f = k_i/k_o$. For simplicity, since no empirical data was available, it was considered that the inner and the outer permeabilities are equal and therefore $k_f = 1$.

The following equations are used to define the factor a :

$$a_1 = c_0 - c_1 \left[1 - \exp \left(-\frac{h}{c_2 D} \right) \right], \tag{4.31}$$

$$a = \frac{a_1 k_f}{(1 - a_1) + a_1}, \tag{4.32}$$

with the values $c_0 = 0.45$, $c_1 = 0.36$, $c_2 = 0.48$.

During this phase, some limitations may be encountered. For example, as the suction is increased, the upward hydraulic gradient on the inside of the caisson approaches the value at which a piping failure might be induced. Soil piping is the formation of underground tunnels due to subsurface soil erosion [60], as depicted in Figure 4.17.

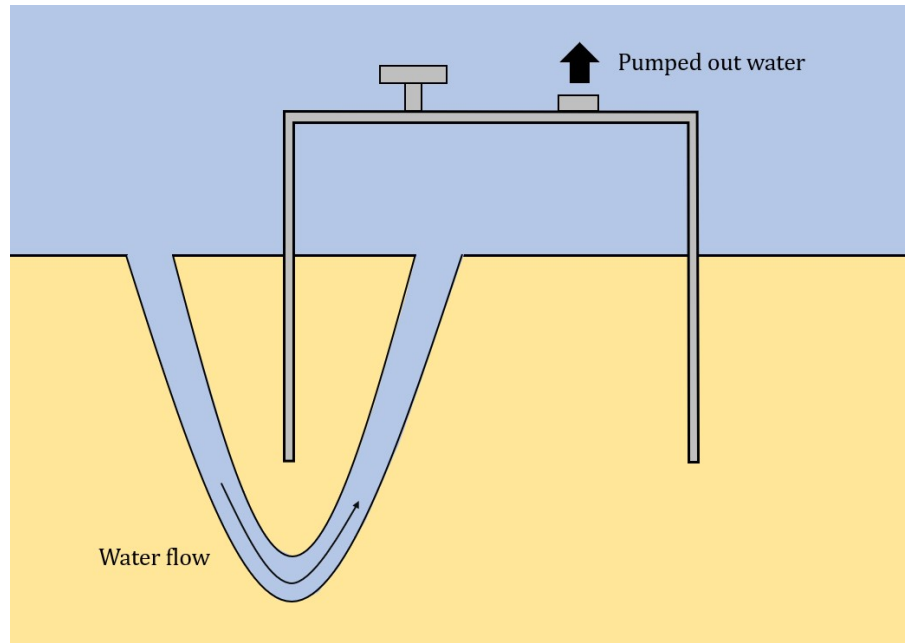


Figure 4.17 – Soil piping.

If piping occurs, any attempts to increase the suction further will induce a major inflow of water into the caisson, but without significant further penetration. Solving equation 4.33 for h determines the depth for which soil piping occurs.

$$V' + \frac{\gamma' h}{(1-a)} \frac{\pi D_i^2}{4} = \frac{\gamma'}{(1-a)} Z_o^2 \left[\exp\left(\frac{h}{Z_o}\right) - 1 - \left(\frac{h}{Z_o}\right) \right] \quad (4.33)$$

$$\times (K \tan \delta)_o (\pi D_o) + (\gamma' t N_\gamma) (\pi D t).$$

Similarly to the previous section, these equations required iterative methods to be solved, thus a MATLAB script was also used to solve them. The script is shown in appendix D.

The results obtained are shown in Figure 4.18 as a design graph. According to this figure, the limit of penetration depth before soil failure is approximately 1.59 m.

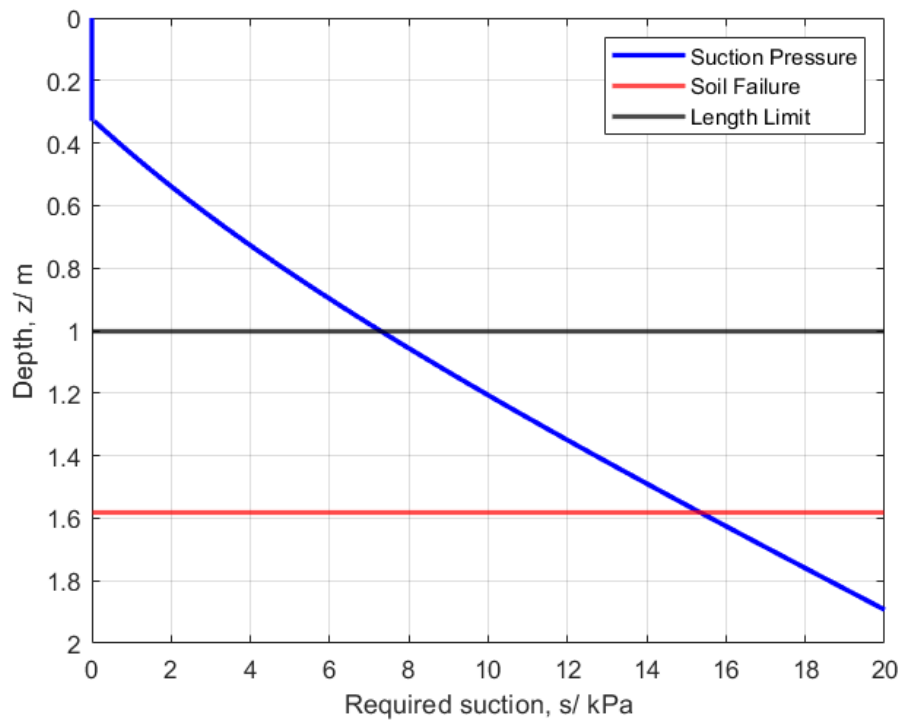


Figure 4.18 – Suction penetration design graph (first iteration).

As seen in Figure 4.18, when suction is not applied, the maximum depth reached is equal to the maximum self-weight embedment of 33 cm. For this particular case, with the aid of suction assisted penetration, the maximum embedment depth is limited by the caisson length and not by the depth at which soil piping occurs. Therefore, if a suction pressure of, approximately, 7.2 kPa is achieved the suction bucket can be totally embedded.

4.4 In-service

According to Figure 3.27, the downstream footing experience the most significant compression loads and the upstream footing reduced compression load. Note that upstream footing in other environmental conditions could experience tension uplift. Horizontal loads were assumed to be equally shared across the foundations.

Therefore, the compression capacity of the most heavily loaded downstream footing must be verified.

4.4.1 Load regime

To verify if the soil compression capacity is sufficient to hold the foundation loads, the critical load regime must be analysed. In this case, it corresponds to the load regime in downstream buckets, as seen in Figure 4.19.

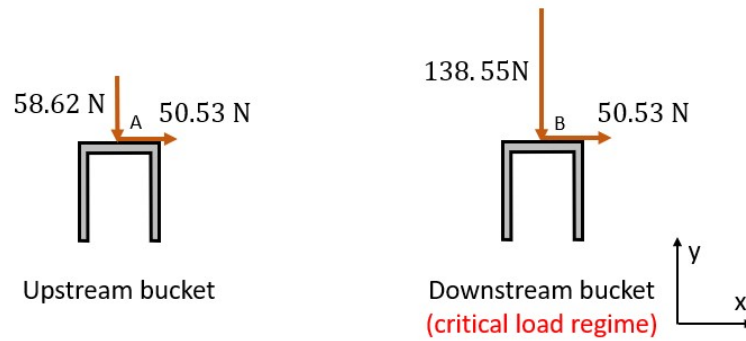


Figure 4.19 – Critical load regime.

Therefore, the critical loads that will be considered in the suction bucket design are:

- $V_c = 138.55 \text{ N}$,
- $H_c = 50.53 \text{ N}$.

4.4.2 Bearing capacity

Suction bucket foundations, given their geometry and possibility of different embedment depths, are not easily treated as shallow foundations nor as deep foundations; they are classified as intermediate foundations [51].

For conceptual design, classical bearing capacity methods for shallow foundations may be adapted (e.g. DNVGL-RPC212 (2017)). However, the effect of embedment (typically $L/D > 0.5$) and the strong coupling between the horizontal and moment resistance makes their application not straightforward [51].

Nevertheless, the method outlined in "Suction Installed Caisson Foundations for Offshore Wind: Design Guidelines" [51], that adapts the classical bearing capacity method for shallow foundations, will be used to evaluate the soil bearing capacity.

In classical bearing capacity method, all horizontal and vertical forces acting on the foundation are transferred to the foundation base and combined into resultant forces H and V in the horizontal and vertical directions, respectively, at the foundation-soil interface.

It is assumed that H and V are design forces, i.e. they are factored forces, which are characteristic forces that have been multiplied by their relevant load factor γ_f . This is indicated by index d in the bearing capacity expressions, hence H_d and V_d . The load centre, denoted LC, is the point where the resultant of H_d and V_d intersects the foundation-soil interface, as shown in Figure 4.20 [37].

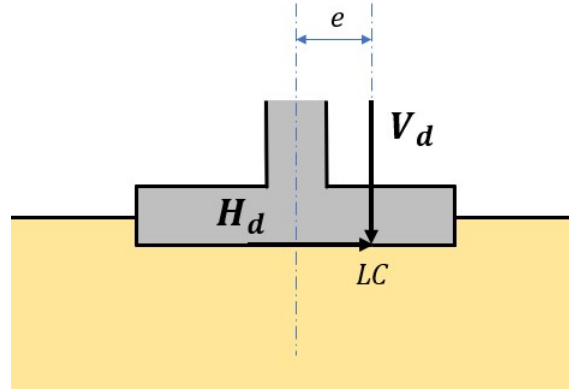


Figure 4.20 – Loading of a shallow foundation. Adapted from [61].

The load centre LC implies an eccentricity e of the vertical force V_d relative to the centre line of the foundation. The eccentricity is calculated as:

$$e = \frac{M_d}{V_d}, \quad (4.34)$$

where M_d is the resulting design overturning moment about the foundation-soil interface.

If the applied moment on the base of the caisson is not under consideration, the effective area, breadth and length are given simply as:

$$A_{eff} = A = \frac{\pi D^2}{4}, \quad (4.35)$$

and

$$B_{eff} = L_{eff} = D. \quad (4.36)$$

If the moment is considered the soil capacity is then calculated for a symmetric foundation with a reduced “effective area”, with this vertical load applied at its centre. This procedure is always conservative, as it ignores part of the foundation capacity. The effective area, breadth and length of the foundation need to be calculated, and, for a circular foundation, these are given by:

$$A_{eff} = 2 \left(\frac{D^2}{4} \arccos \left(\frac{2e}{D} \right) - e \sqrt{\frac{D^2}{4} - e^2} \right). \quad (4.37)$$

Making use of the intermediate variables $B_e = D - 2e$ and $L_e = \sqrt{D^2 - (D - B_e)^2}$:

$$L_{eff} = \sqrt{A_{eff} \frac{L_e}{B_e}}, \quad (4.38)$$

$$B_{eff} = L_{eff} \frac{B_e}{L_e} = \sqrt{A_{eff} \frac{B_e}{L_e}}. \quad (4.39)$$

These quantities are shown in Figure 4.21.

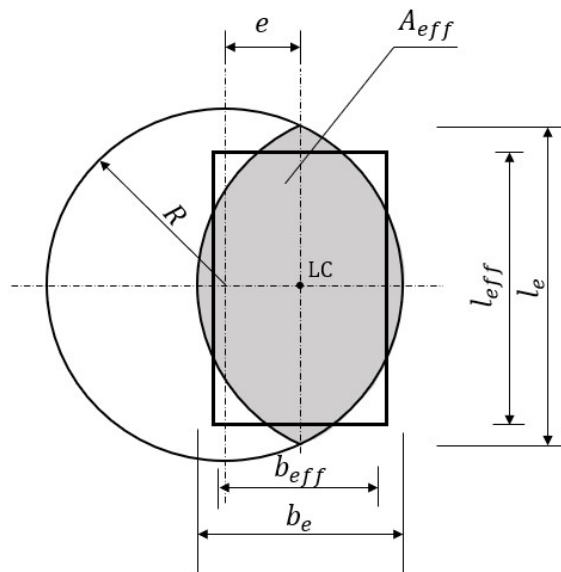


Figure 4.21 – Circular footing with effective foundation area marked out. Adapted from [61].

However, when skirts are used, the base level (the foundation-soil interface is taken to be at the skirt tip level rather than at the seabed) [37]. Thus, modifications are necessary to convert the solutions so that they refer to the Load Reference Point (LRP) at the mudline level. This involves converting the loads and take into account the contribution of external friction and lateral pressure on the skirt.

The conversion method used in "Suction Installed Caisson Foundations for Offshore Wind: Design Guidelines" is shown in Figure 4.22.

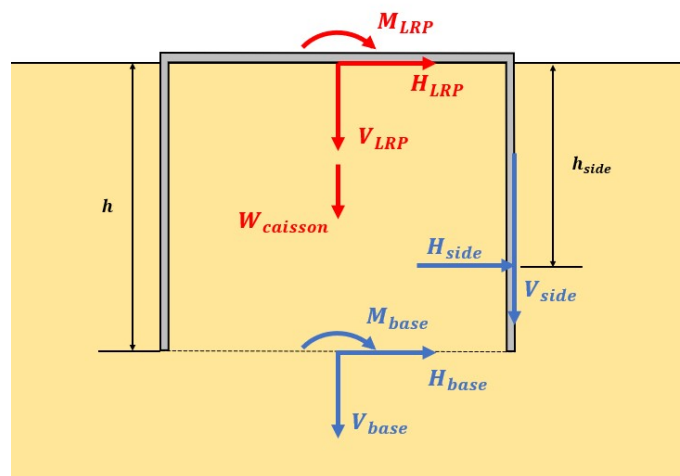


Figure 4.22 – Forces considered in the conversion method. Adapted from [51].

The forces represented in red act on the suction bucket, whereas the forces represented in blue act on the sand.

The forces shown in Figure 4.22 are related by the following equations:

$$V_{LRP} + W_{caisson} = V_{base} + V_{side}, \quad (4.40)$$

$$H_{LRP} = H_{base} + H_{side}, \quad (4.41)$$

$$M_{LRP} = M_{base} - h_{side}H_{side} - hH_{base}. \quad (4.42)$$

In this case, the eccentricity, e , is calculated as:

$$e = \frac{M_{base}}{V_{base}}. \quad (4.43)$$

The term $W_{caisson}$ should include the buoyant weight of the soil trapped in the caisson $\frac{\pi D_i^2 h \gamma'}{4}$. Thus,

$$W_{caisson} = (\rho_{steel} - \rho_{sea\ water}) \times V_{bucket} \times g + \frac{\pi D_i^2 h \gamma'}{4}. \quad (4.44)$$

In sand, the vertical load capacity in compression (downward load) on the side of the caisson may be estimated as:

$$V_{side} = (\pi D h) \frac{\gamma' h}{2} K \tan \delta. \quad (4.45)$$

According to the guidelines, active and passive pressures may be assumed on the sides of the caisson and the horizontal capacity may be estimated as:

$$H_{side} = \frac{\gamma' h^2 D}{2} (K_p - K_a). \quad (4.46)$$

The Earth pressure coefficients may be estimated from the standard expressions $K_p = \frac{1}{K_a} = \frac{1 + \sin \phi'}{1 - \sin \phi'}$.

The position of the effective depth of action of the horizontal load may be estimated as

$$h_{side} = \frac{2h}{3}. \quad (4.47)$$

Multi-caisson structures only permit very small rotations of an individual footing, thus the moment developed on them may not be significant as far as ultimate capacity is concerned [51]. Since the lander is a multi-caisson structure this simplification is assumed. Thus,

$$M_{LRP} = 0 \text{ Nm}. \quad (4.48)$$

For drained conditions (sand) at the skirt tip level:

$$V_{ult} = A_{eff} \left(\frac{1}{2} \gamma' B_{eff} N_\gamma s_\gamma d_\gamma i_\gamma + \gamma' h N_q s_q d_q i_q \right), \quad (4.49)$$

where N_q and N_γ are conventional bearing capacity factors given by:

$$N_q = \tan^2 \left(\frac{\pi}{4} + \frac{\phi'}{2} \right) e^{\pi \tan \phi'}, \quad (4.50)$$

$$N_\gamma = 1.5 (N_q - 1) \tan \phi' \text{ (this being an approximate formula).} \quad (4.51)$$

i_q and i_γ are load inclination factors given by:

$$i_q = 1 - 0.5 \left(\frac{H_{base}}{V_{base}} \right)^5, \quad (4.52)$$

$$i_\gamma = 1 - 0.7 \left(\frac{H_{base}}{V_{base}} \right)^5. \quad (4.53)$$

If the horizontal load is not under consideration the formulae may be used with $i_q = i_\gamma = 1$.

s_q and s_γ are shape factors given by:

$$s_q = 1 + i_q \left(\frac{B_{eff}}{L_{eff}} \right) \sin \phi', \quad (4.54)$$

$$s_\gamma = 1 - 0.4 i_\gamma \left(\frac{B_{eff}}{L_{eff}} \right). \quad (4.55)$$

d_q is a depth factor given by:

$$d_q = 1 + 1.2 \left(\frac{h}{B_{eff}} \right) \tan \phi' (1 - \sin \phi')^2. \quad (4.56)$$

The depth factor d_γ is per definition equal to unity, thus $d_\gamma = 1$.

For the values of the first iteration of the suction bucket and for the soil conditions previously stated, the bearing capacity parameter values are presented in Table 4.7.

Table 4.7 – Bearing capacity parameter values (first iteration).

Parameter	
V_{LRP}	138.55 N
H_{LRP}	50.53 N
M_{LRP}	0 Nm
h	1 m
$W_{caisson}$	2190.88 N
V_{side}	885.45 N
H_{side}	5610 N
h_{side}	0.67 m
V_{base}	1443.97 N
H_{base}	5559.47 N *
M_{base}	1819.47 Nm *
e	1.26 m *
V_{ult}	not available

* Using the previous equations these values should be negative. However, for design purposes only their absolute values are considered.

Since it was not possible to calculate the ultimate bearing capacity, V_{ult} , because the eccentricity, $e = 1.26$ m, is larger than the outer radius of the bucket, $D_o/2 = 0.25$ m, an iterative method was followed. This method is later described.

4.5 Deployment

The deployment phase includes the suction bucket lowering and the subsequent touchdown in the seabed. This phase should be carefully controlled, especially in the case of an erodible or soft soil.

To minimize overpressure within the suction bucket, during the deployment phase, the bucket should be vented so that air and water can escape through it, as shown in Figure 4.23.



Figure 4.23 – Vent of the suction bucket. Adapted from [62].

The effectiveness of the vent is critical when the caisson approaches the seabed since the landing impact may cause wash-out/erosion beneath the rim. If erosion beneath the rim occurs, a successful seal for the suction application may be more difficult to attain [51].

The vent design is not within the scope of the present thesis, however information about this topic can be founded in DNVGL-RP-N103 code.

4.6 Decommissioning

The decommissioning corresponds to the suction buckets extraction when their function is no longer required. Its extraction can be achieved by the reverse of the installation procedure, where water is pumped into the suction bucket to create an 'overpressure' which pushes it out of the seabed [51].

The required overpressure is calculated in the same way as the installation procedure. During this stage, all vents must be closed and other sources of leakage avoided to develop sufficient overpressure to extract the suction bucket.

4.7 Iterations and final result

During the in-service design, it was concluded that the first iteration dimensions of the suction bucket were not appropriate for the load regime. Hence, an iterative method was developed to achieve possible solutions.

The iterative method is shown in Figure 4.24.

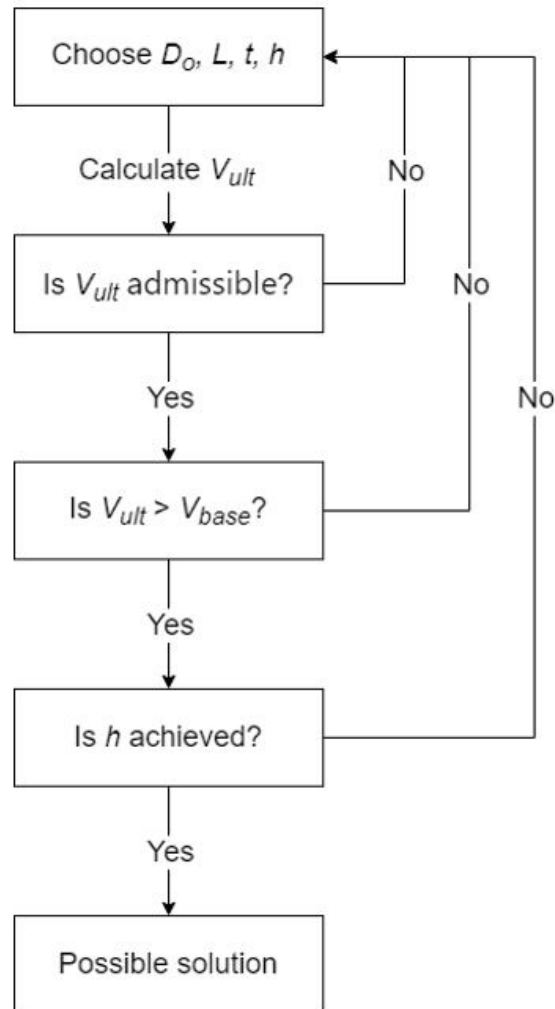


Figure 4.24 – Iterative method.

Firstly, values of D_o , L , t and h are assumed. V_{ult} is calculated according to these values. If V_{ult} is a positive real number, then it is considered to be admissible. The next step consists in comparing the ultimate soil capacity, V_{ult} , with the soil vertical loading, V_{base} . If $V_{ult} > V_{base}$, then the soil does not fail. Lastly, the installation process is verified by confirming that the assumed embedment depth, h , can be achieved. If it is confirmed, a possible solution was found. However, if any of these steps are not verified, new values for D_o , L , t and h should be considered.

Table 4.8 shows the values of the first phase of iterations. The first phase consists in a sensitivity analysis that determines how bearing capacity parameters are affected based on changes in other design variables (input variables).

Table 4.8 – Iterations of suction bucket dimensions (first phase).

No.	Design variables				Bearing capacity parameters				
	D_o (m)	L (m)	t (m)	h (m)	V_{base} (N)	H_{base} (N)	M_{base} (Nm)	e (m)	V_{ult} (N)
1	0.5	1	0.005	1	1444	5559	1819	1.26	-
2	1	1	0.005	1	6215	11226	3708	0.60	-
3	1.5	1	0.005	1	14457	16893	5597	0.39	-
4	2	1	0.005	1	26169	22559	7486	0.29	432996
5	0.5	2	0.005	2	915	22389	14859	16.24	-
6	0.5	5	0.005	5	-11298	-	-	-	-
7	0.5	10	0.005	10	-67071	-	-	-	-
8	0.5	1	0.010	1	1960	5503	1801	0.92	-
9	0.5	1	0.050	1	5593	5049	1649	0.29	-
10	0.5	1	0.100	1	8976	4483	1461	0.16	8065
11	0.5	1	0.150	1	11221	3916	1272	0.11	11897

According to Table 4.8 the increase of the outer diameter, D_o , and the bucket thickness, t , lead eventually to a possible solution, being the diameter increase the one with bigger impact on soil resistance. The increase of the embedment depth, h , and the consequently increase of the bucket length, L , lead to higher soil lateral resistance, thus a higher traction vertical force on the soil. This case will not be analysed since the sand has almost no resistance to traction loads.

A second phase of iterative calculations focused on varying the outer diameter, D_o , and thickness, t , variables sequentially until an optimized possible solution was achieved. Table 4.9 shows the values of the second phase of iterations.

Table 4.9 – Iterations of suction bucket dimensions (second phase).

No.	Design variables				Bearing capacity parameters				
	D_o (m)	L (m)	t (m)	h (m)	V_{base} (N)	H_{base} (N)	M_{base} (Nm)	e (m)	V_{ult} (N)
12	1	1	0.150	1	32797	9583	3161	0.10	125406
13	1	1	0.100	1	25261	10149	3349	0.13	122258
14	1	1	0.050	1	16056	10716	3538	0.22	86276
15	1	1	0.030	1	11872	10943	3614	0.30	37954
16	1	1	0.020	1	9667	11056	3652	0.38	187
17	0.9	1	0.030	1	9944	9809	3236	0.33	15890
18	0.8	1	0.030	1	8182	8676	2858	0.35	3287

According to the Table 4.9, a possible solution is a bucket with the following dimensions and embedment depth:

$$D_o = 0.9 \text{ m}, L = 1 \text{ m}, t = 0.030 \text{ m}, h = 1 \text{ m}.$$

As shown in Figures 4.25 and 4.26 these design dimensions can achieve the assumed embedment depth before soil piping occurs.

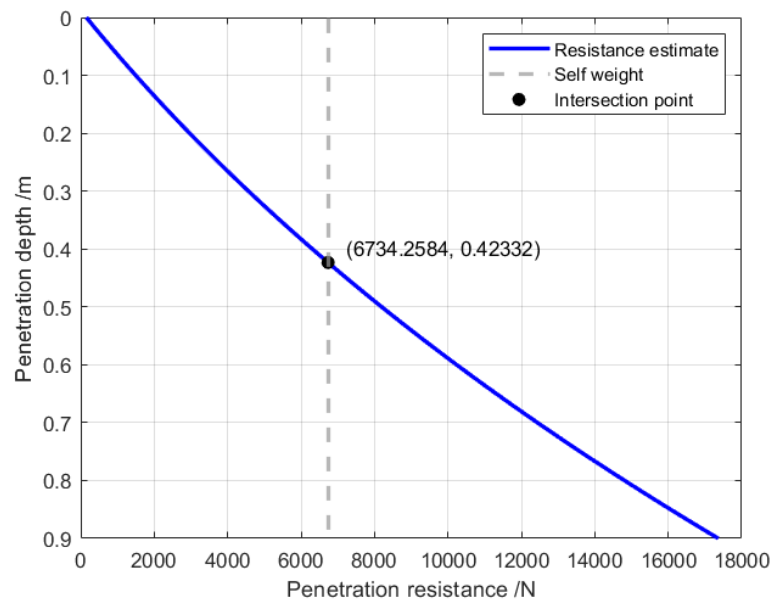


Figure 4.25 – Self-weight penetration design graph (final dimensions).

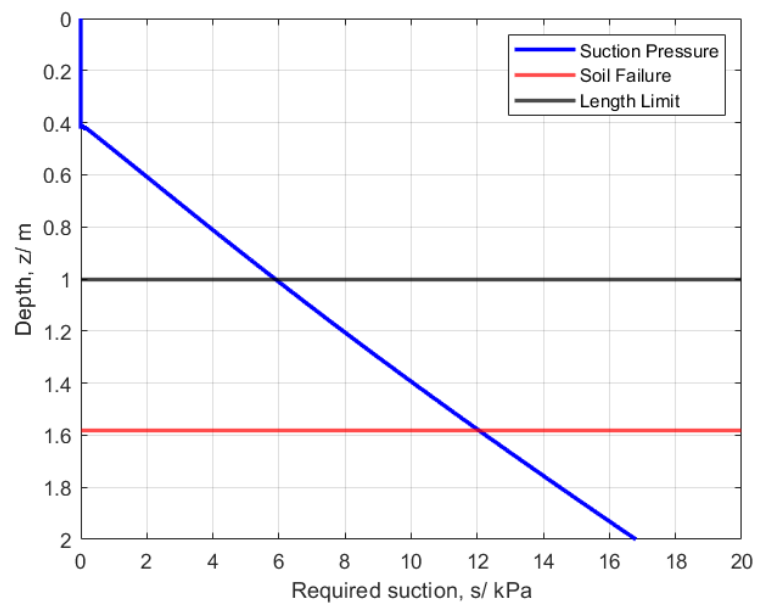


Figure 4.26 – Suction penetration design graph (final dimensions).

Thus, these dimensions were considered as the final design dimensions for the suction buckets. Figure 4.27 shows the final suction bucket design when coupled with the lander structure.

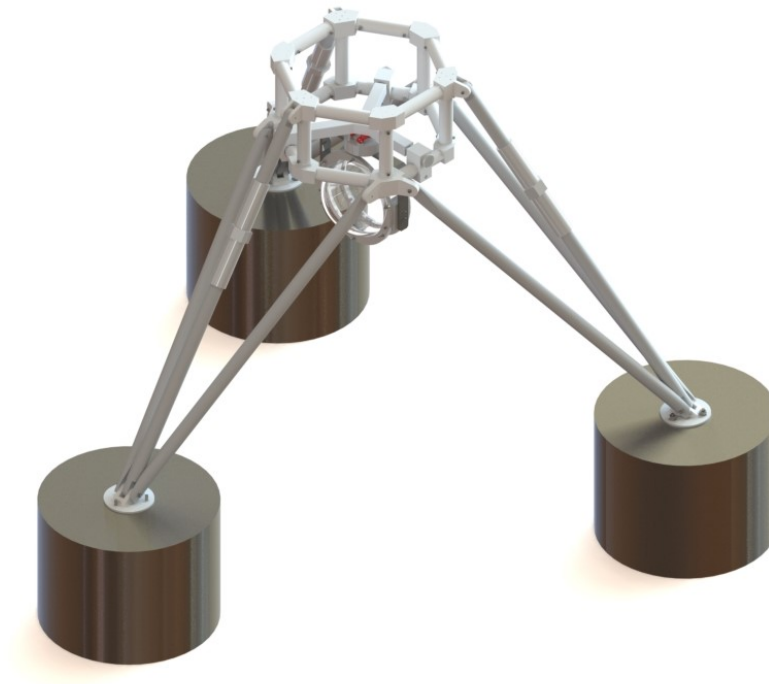


Figure 4.27 – Final suction bucket design (photorealistic image).

The technical drawing of the final suction bucket design is shown in Appendix E.

Chapter 5

FEM analysis

This chapter aims to use Finite Element Method (FEM) as a tool to know which stresses the suction buckets undergo during its service period. The steps since the model simplification until the application of the loads and boundary conditions are described. After obtaining the maximum stress values, a material for the suction buckets is selected.

5.1 Simplifications

For a detailed stress analysis, as the lander (composed by the structure as well as the suction buckets) and the soil should be modeled. However, this analysis proved to be a time-consuming simulation. Since the objective is to analyse the stresses in the suction buckets, only the buckets and the soil were modeled. For this reason, the forces acting on the structure were represented as acting on the lander's centre of gravity, as shown in Figure 5.1.

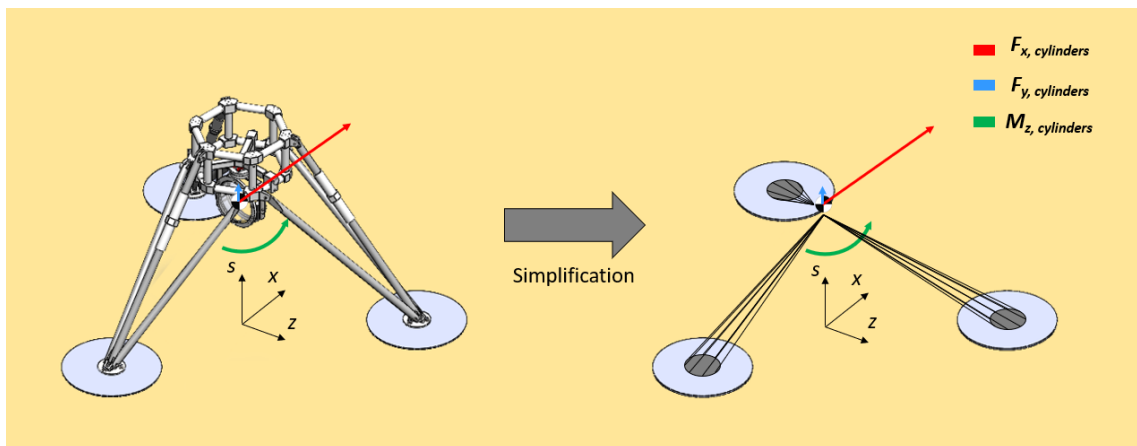


Figure 5.1 – Model simplification.

5.2 Model geometry

Initially, the tripod bucket foundation and the soil were modeled using *Solidworks*. Figure 5.2 shows the boundary extensions of the soil domain. B_H and B_V denote the vertical and the horizontal boundary extents from the center of the bucket foundation and the ground surface, respectively. For the tripod bucket foundation, these boundaries were remotely extended at $B_H = 3.5D_o + r$ and $B_V = 2.5D_o + r$ [63].

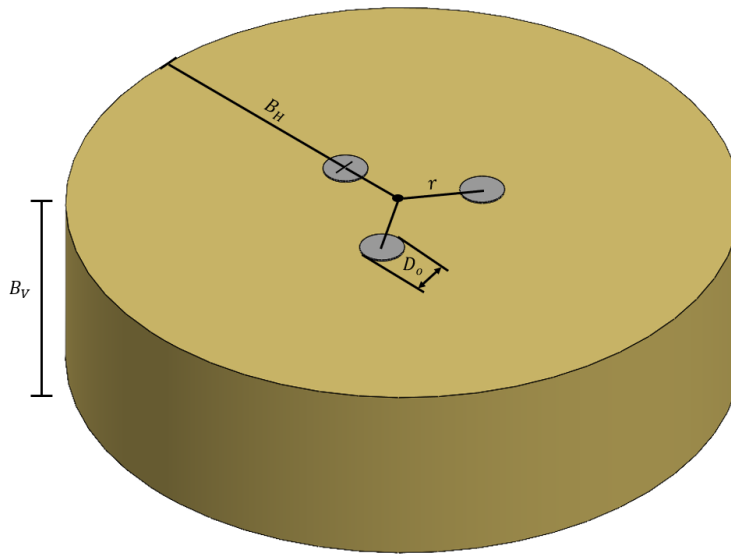


Figure 5.2 – Model of tripod bucket foundation and the soil in *Solidworks*.

This model was converted to *Ansys* using *SpaceClaim*, as shown in Figure 5.3.

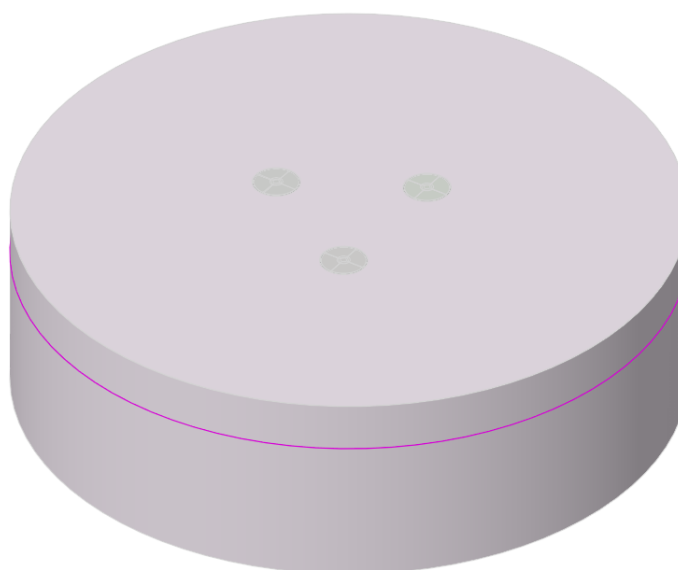


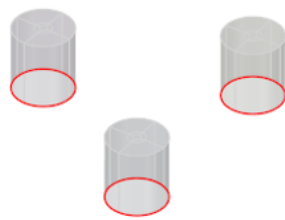
Figure 5.3 – Model of tripod bucket foundation and the soil in *Ansys*.

The values of geometric parameters are presented in Table 5.1.

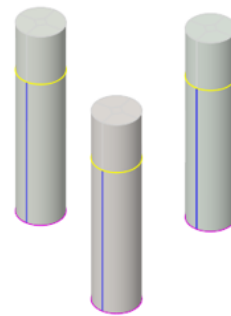
Table 5.1 – Geometric parameters.

Parameter	
D_o	0.9 m
L	1 m
t	0.030 m
r	1.6 m
B_H	4.75 m
B_V	3.85 m

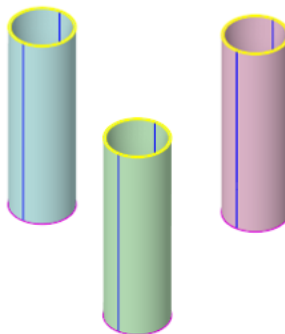
Within *SpaceClaim*, the sand was divided into 3 parts for easier mesh control: inner, intermediate and outer sand. The different model bodies are shown in Figure 5.4.



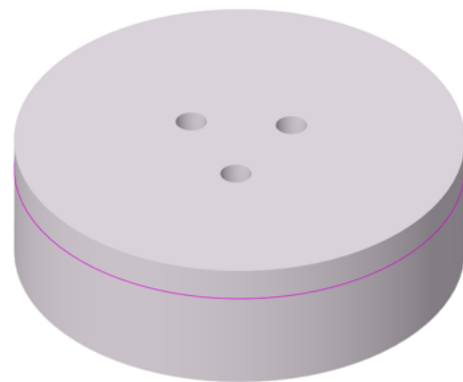
(a) Suction buckets



(b) Inner sand



(c) Intermediate sand



(d) Outer sand

Figure 5.4 – Different model bodies in *Ansys*.

5.3 Material properties and parameters

The suction buckets were modeled using the default structural steel from *Ansys*. However, its density was modified to cover water buoyancy effects.

The soil was modeled using a Mohr-Coulomb elasto-plastic model. The Young modulus and Poisson's ratio of the soil were considered as being the average values for a loose sand according to New Jersey Institute of Technology [64].

The material properties necessary to define the model are presented in Table 5.2.

Table 5.2 – Material properties.

Suction buckets (steel)	Young's modulus (E)	200 GPa
	Poisson's ratio (ν_p)	0.3
	Density (ρ)	6900 Kg/m ³
Soil (Loose sand)	Young's modulus (E)	17 MPa
	Poisson's ratio (ν_p)	0.3
	Density (ρ)	1885.8 Kg/m ³
	Initial Inner Friction Angle (ϕ')	30 °
	Residual Inner Friction Angle (ϕ'_{cv})	30 °
	Initial cohesion (c')	0 Pa *
	Residual cohesion (c'_{cv})	0 Pa *
	Dilatancy angle (ψ)	0 °

* Although the theoretical value of sand cohesion is 0, small values of c' are used to avoid numerical instabilities [65]. In this case, it was assumed that $c' = 0.1$ MPa and $c'_{cv} = 0.1$ MPa, after solving the simulation for $c' = 0.001$ MPa and $c' = 0.01$ MPa, where convergence problems were detected.

5.4 Mesh

Several partitions were made in the different bodies to achieve a structured mesh in the zone of interest, i.e suction buckets and their surroundings. Before doing the mesh, the topology of the sand parts was shared so that the nodes at each interface were coincident.

In this study, four different quality meshes were used. The parameters of the meshes are presented in Table 5.3.

Table 5.3 – Parameters of the different types of mesh.

Type	Nodes	Elements	Characteristic length (mm)	Skewness		
			Average	Min	Max	Average
Coarse	191237	45618	44.608	8.733e-2	0.999	0.715
Medium	273386	66384	31.593	1.200e-6	0.536	0.094
Fine	330960	81840	28.942	1.657e-7	0.571	0.108
Very fine	408389	100418	25.885	1.681e-7	0.535	0.095

As the mesh gets finer, the number of nodes and elements increases. Skewness is a mesh quality parameter that was used to control the solution accuracy. It is defined as the difference between the shape of the cell and the shape of an equilateral cell of equivalent volume [66]. This parameter ranges values between 0 (optimal cells) and 1 (highly skewed cells). Medium and fine meshes were considered very good meshes since their average skewness is approximately 0.

The different medium meshes are shown in Figure 5.5 for each body.

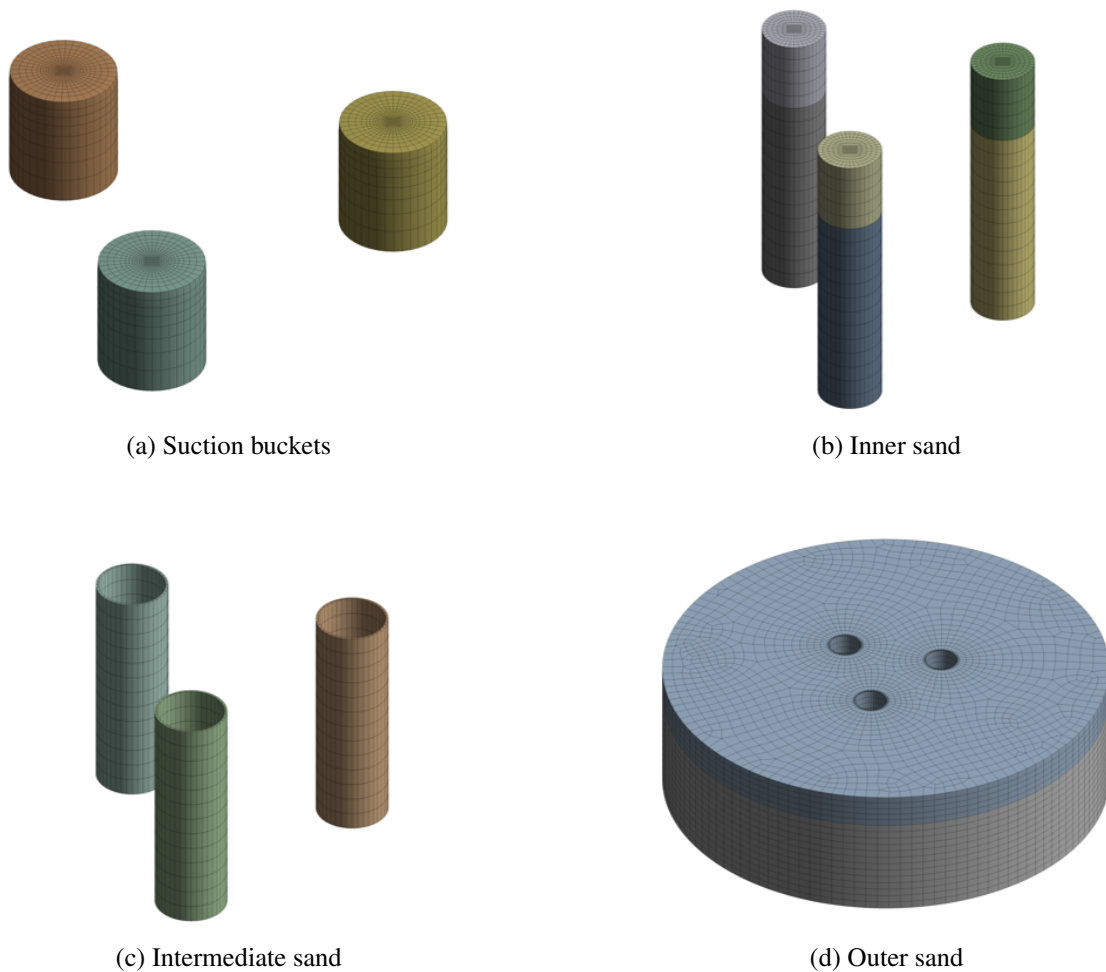


Figure 5.5 – Medium meshes of different bodies.

5.4.1 Element Type

The mesh used to discretize the sand used SOLID186 element type. SOLID186 is a higher order 3D 20-node solid element that exhibits quadratic displacement behavior. The element is defined by 20 nodes having three degrees of freedom per node: translations in the nodal x, y, and z directions [67].

The mesh used to discretize the suction buckets used SHELL181 element type. SHELL 181 is a 4-node element with six degrees of freedom at each node: translations in the x, y, and z directions, and rotations about the x, y, and z-axes. This element type is for general use since it includes the linear effects of transverse shear deformation [68].

5.4.2 Contacts

Before running the simulation, several pairs of contacts were defined between:

- Suction bucket top faces and inner sand top faces
- Suction bucket outer lateral faces and outer sand lateral faces
- Suction bucket inner lateral faces and inner sand lateral faces
- Suction bucket bottom faces and intermediate sand top faces

It was chosen a frictional type of contact whose friction coefficient, μ_s , is defined by:

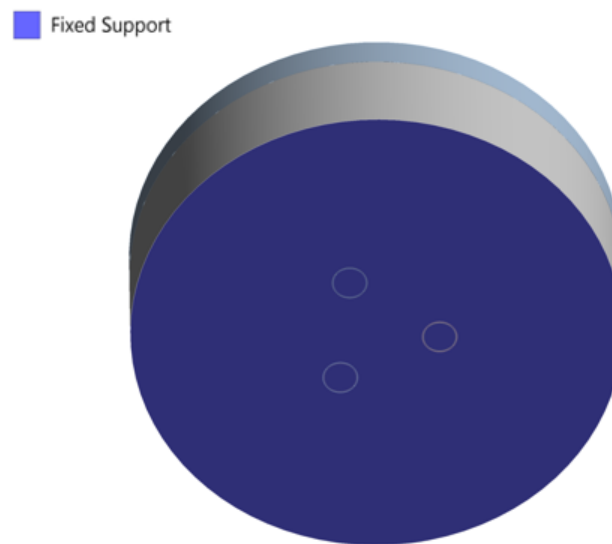
$$\mu_s = \tan \delta \quad (5.1)$$

being δ the angle of friction of the interface suction bucket-soil, previously defined as $\delta = 15^\circ$. Therefore, the friction coefficient is:

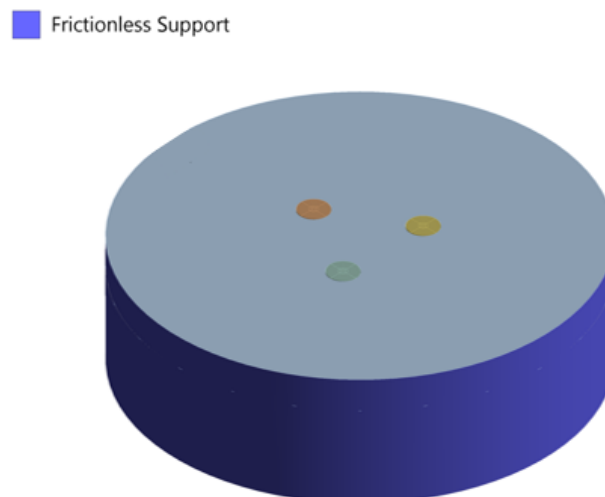
$$\mu_s = 0.27$$

5.5 Loads and boundary conditions

All the boundary conditions were applied into the sand. The bottom sand face was considered fixed while the lateral faces were considered as being frictionless supported, i.e they cannot move radially, as shown in Figure 5.6.



(a) Fixed support



(b) Frictionless support

Figure 5.6 – Boundary conditions.

Loads such as gravity and environmental forces, hydrostatic pressure (inside and outside of the suction bucket) and environmental moment were considered in this simulation as shown in Figure 5.7. Note that the environmental force is applied in the contact area between the suction bucket and the feet of the lander.

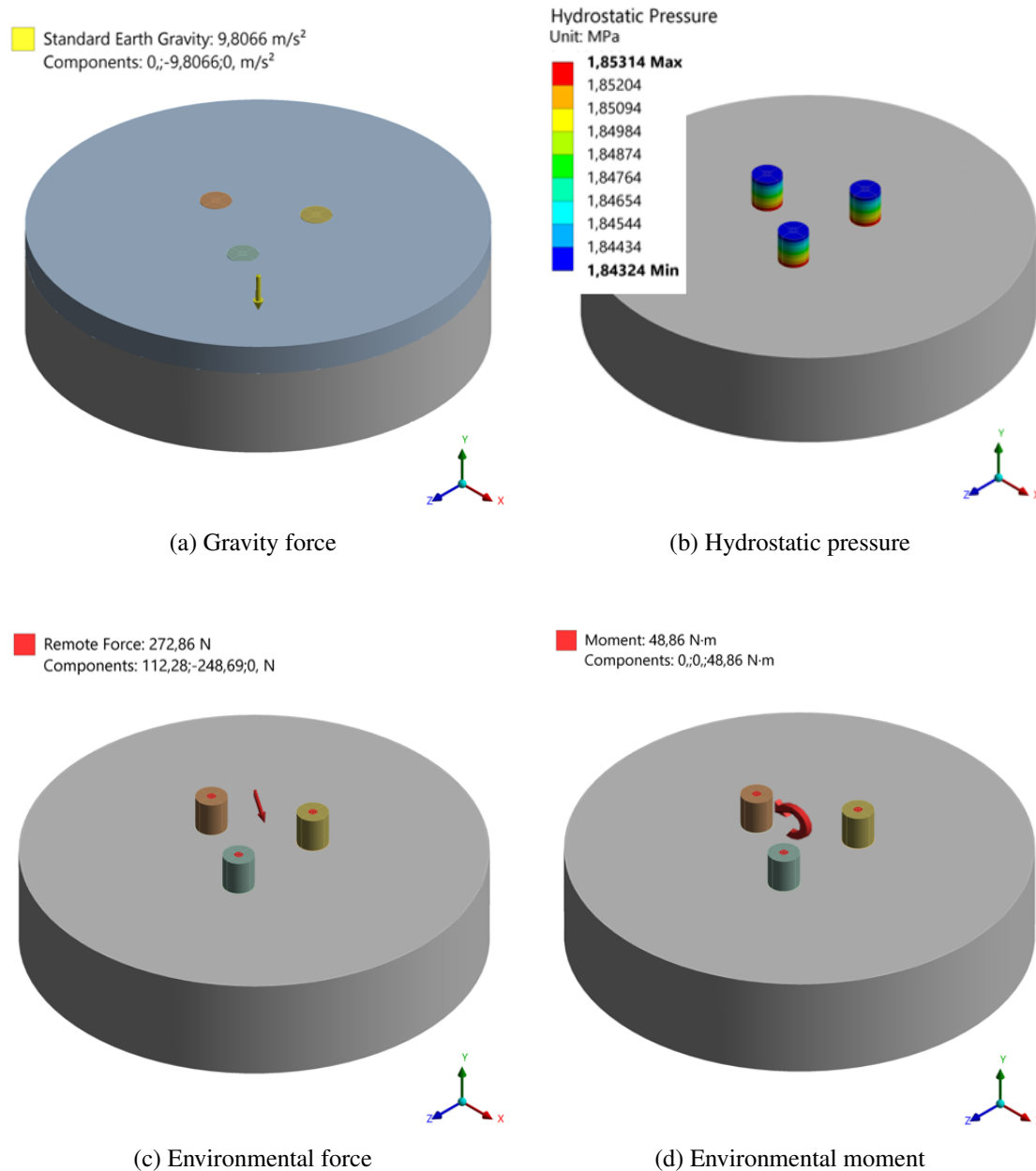


Figure 5.7 – Analysis settings.

5.6 Results

The results shown in this section were obtained for the very fine mesh, since they are more accurate. The von-Mises equivalent stresses for the sand and suction buckets and only for the suction buckets are shown in Figures 5.8 and 5.9, respectively.

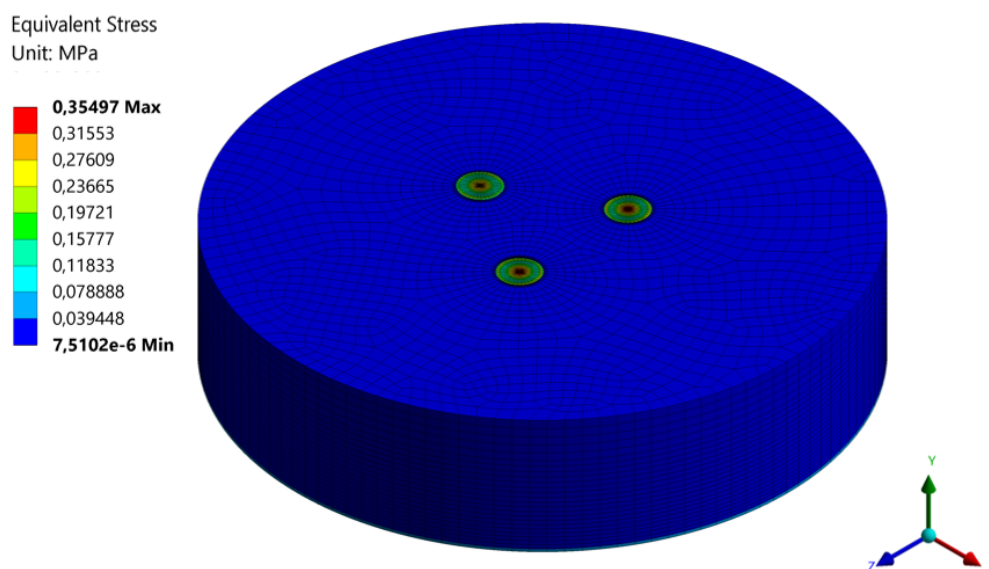


Figure 5.8 – von-Mises equivalent stresses in the suction buckets and sand (very fine mesh).

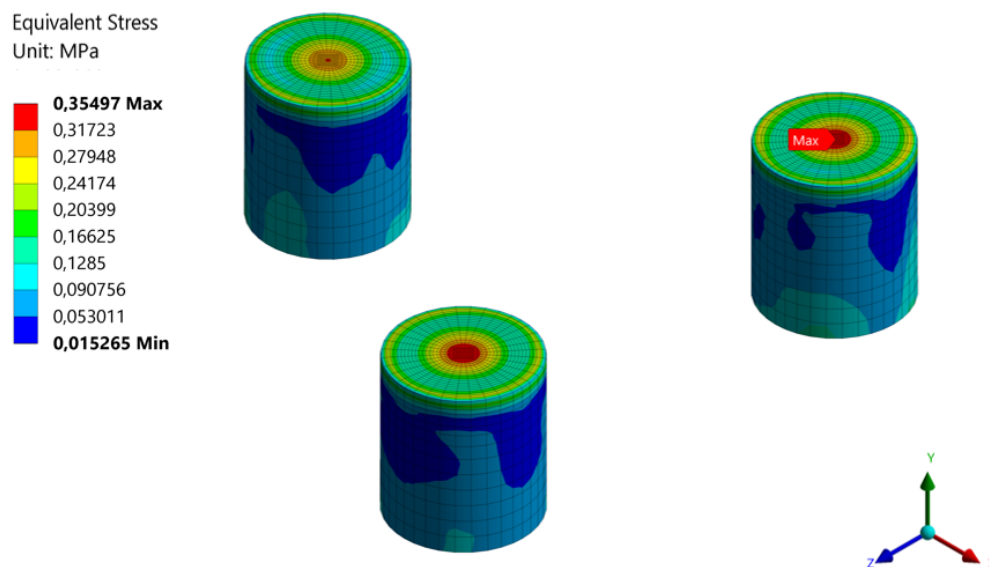


Figure 5.9 – von-Mises equivalent stresses in the suction buckets (very fine mesh).

The results obtained in Figure 5.9 confirm that the critical loading regime is verified on the downstream suction buckets, as seen in section 4.4.1, and is equal to $\sigma_{VM,max} = 0.36$ MPa.

The maximum principal equivalent stresses for the sand and suction buckets and only for the suction buckets are shown in Figures 5.10 and 5.11, respectively.

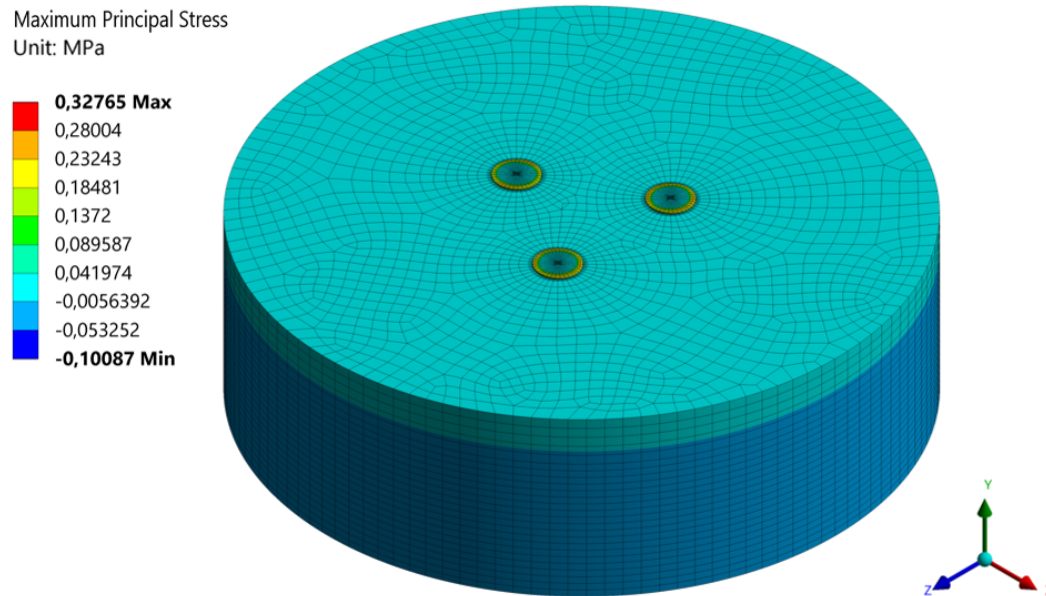


Figure 5.10 – Maximum principal stresses in the suction buckets and sand (very fine mesh).

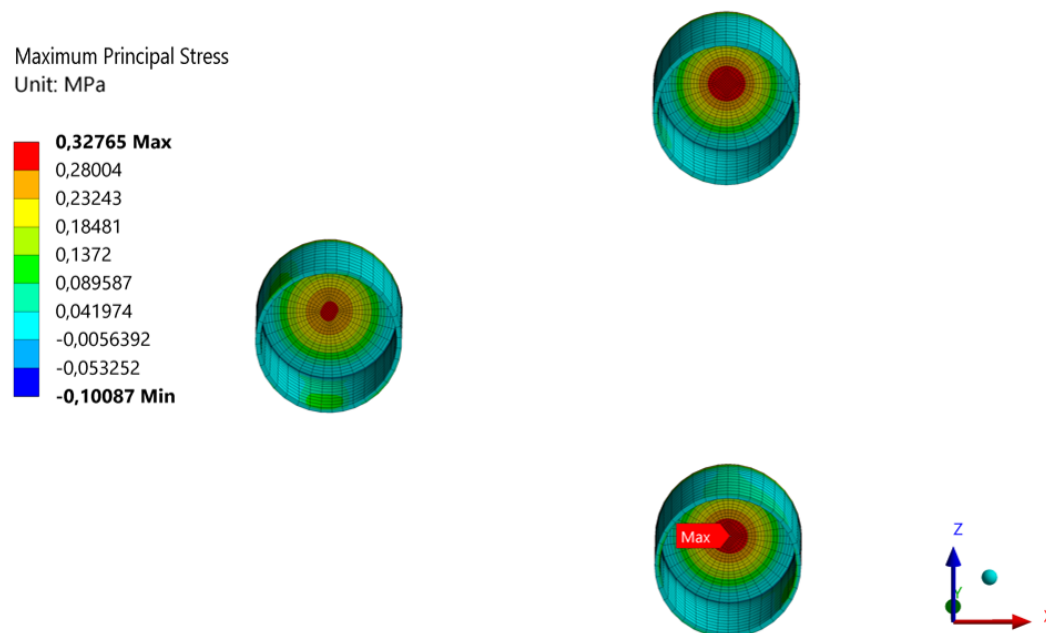


Figure 5.11 – Maximum principal stresses in the suction buckets (very fine mesh).

The results obtained in Figure 5.11 confirm that the critical loading regime is verified on the downstream suction buckets, as seen in section 4.4.1, and is equal to $\sigma_{1,max} = 0.33$ MPa.

The sand and the suction buckets total deformation is shown in Figure 5.12.

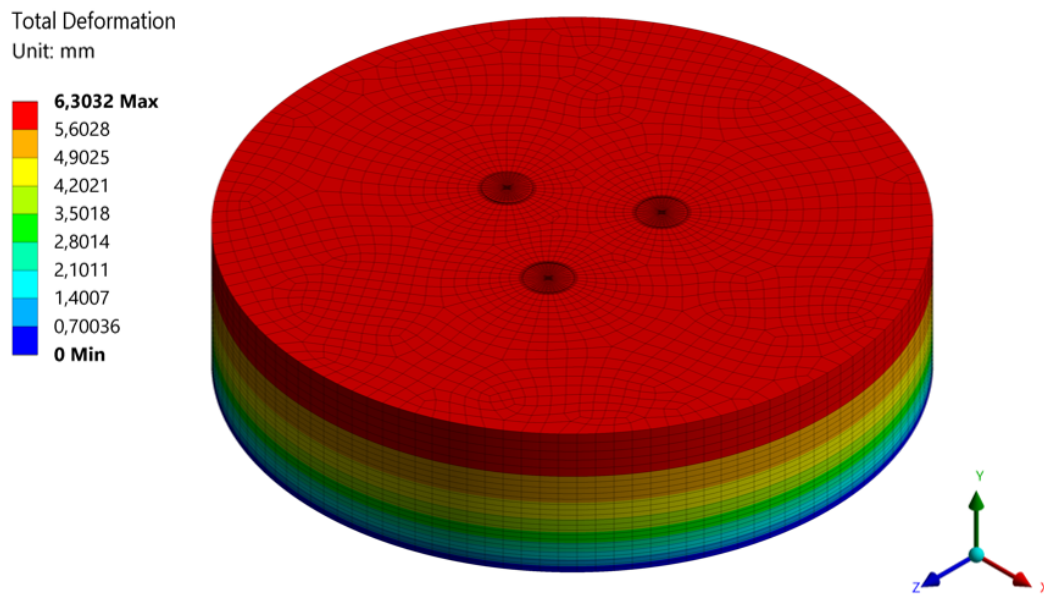


Figure 5.12 – Total deformation in the suction buckets and sand (very fine mesh).

According to Figure 5.12, no significant deformation is verified, being the maximum deformation, $\Delta = 6.3$ mm. Thus, the lander stability is assured.

5.6.1 Convergence study

The convergence study used the four types of mesh mentioned in section 5.4. This convergence study was led for two different points of the upstream bucket, shown in Figure 5.13. These points were chosen since they belong to the geometry partitions. Therefore, they are always a node in the same position of the mesh.

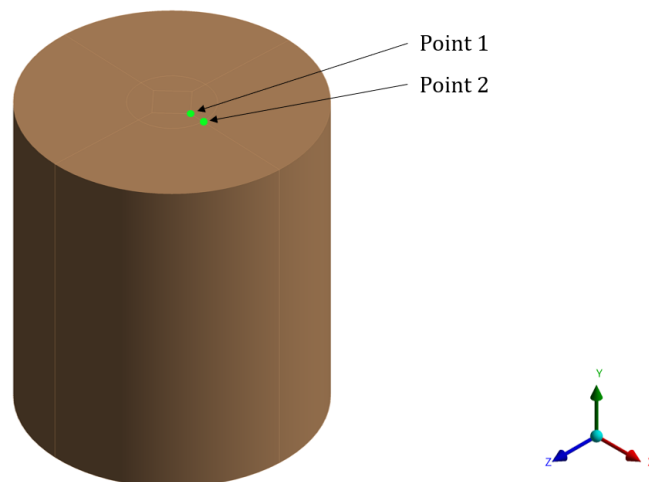


Figure 5.13 – Points of the convergence study.

The results of the convergence study are presented in Table 5.4.

Table 5.4 – Convergence study of von-Mises equivalent stress for two points.

Mesh no.	Type of mesh	Point 1		Point 2	
		Eq. stress (MPa)	Change (%)	Eq. stress (MPa)	Change (%)
1	Coarse	0.30234		0.26378	
2	Medium	0.30190	-0.15	0.26760	1.49
3	Fine	0.29777	-1.37	0.26431	-1.23
4	Very fine	0.29798	0.07	0.26519	0.33

The results are also shown graphically in Figure 5.14.

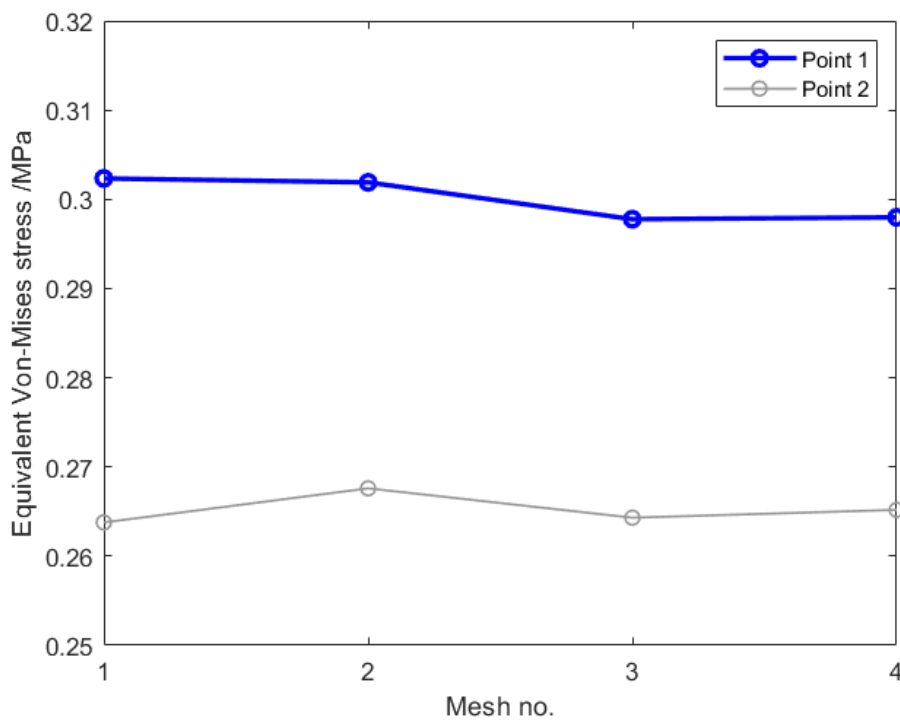


Figure 5.14 – Mesh convergence.

According to the results in Table 5.4 and Figure 5.14, the mesh refinements show mesh convergence since smaller relative variations between mesh refinements were calculated, lower than 1%.

5.7 Material selection

As mentioned in section 2.2.5, material's strength and toughness must be defined to choose an appropriate material.

Since steel is the most used material in offshore foundations due to its mechanical properties, this was the material chosen for the suction buckets.

According to Figures 5.9 and 5.11, the maximum von-Mises equivalent stress, $\sigma_{VM,max}$, and the maximum principal stress, $\sigma_{1,max}$ in the suction bucket are respectively:

$$\sigma_{VM,max} = 0.36 \text{ MPa},$$

$$\sigma_{1,max} = 0.33 \text{ MPa}.$$

Therefore, a steel from group I whose minimum yield strength is 280 MPa or less is sufficient for this application.

The lander once in its service position will operate at temperatures above freezing (0°C), since the temperature at the deep ocean (below 200 m) is in average 4°C [69]. In this position, the structure will be subjected to quasi-static loading. Therefore and according to the different steel classes, steel class C is the most appropriate for the suction bucket.

The steels that fall in this category as well as their properties are shown in Table 5.5.

Table 5.5 – Structural steels. Adapted from [9].

Structural steels	Yield strength (MPa)	Tensile strength (MPa)
ASTM A36 (up to 5.08 cm)	250	400-550
ASTM A131 (up to 1.27 cm)	235	400-490
ASTM A285 (up to 1.90 cm)	205	380-515

The steel bucket is composed by a seamless pipe and a circular plate of 3 cm thickness that are welded to each other. Thus, ASTM A36 is the only steel suitable for this application.

Additional properties of steel ASTM A36 can be found in Table 5.6.

Table 5.6 – ASTM A36 properties [70].

Properties	
Young's modulus (E)	200 - 212 GPa
Poisson's ratio (ν_p)	0,26 - 0,289
Toughness (G)	11.2 - 16.2 kJ/m ²
Minimum service temperature (T_{min})	-54.9 - -44.9 °C

Since the ASTM A36 properties are very similar to the assumed steel properties necessary to run the simulation, it is still considered valid.

The safety factor, SF , using this type of material is:

$$\begin{aligned} SF &= \frac{\sigma_y}{\sigma_{VM,max}} \\ \Rightarrow SF &= \frac{250}{0.36} \\ &= 694. \end{aligned} \tag{5.2}$$

where, σ_y is the steel's yield strength in MPa and $\sigma_{VM,max}$ is the steel's maximum von-Mises equivalent stress calculated in MPa. The value of the safety factor is considered to be very high when comparing with offshore wind turbine foundations, that could present values of $SF = 1.3$ [71]. For this application, the safety factor, SF , is very high since a structural steel was chosen according to offshore wind turbine standard codes for a loading regime considerably lower than the ones registered in this industry.

Although the selected steel can withstand the loading regime, it may suffer from corrosive sea actions. Hence, cathodic protection may be necessary. This protection is usually granted using "sacrificial anodes" that, as the name suggests, are corroded instead of the structural steel [30]. The anode design is not within the scope of this thesis, however more information can be found in DNVGL-OS-C101 standard code.

Chapter 6

Conclusions and Future Work

This chapter closes this work with final comments and proposals for future works.

6.1 Conclusions

The main objective of this dissertation was achieved since an alternative foundation was found to INEGI's lander. After reviewing the literature, the suction buckets foundation type fulfilled all the project requirements.

The method used to design the suction buckets was based on the "Suction Installed Caisson Foundations for Offshore Wind" guidelines [51]. However, in these guidelines the structural loading is supposed to be known. Therefore, the DNV-RP-C205 standard code [37] and the book "Hydrodynamics of Offshore Structures" [38] were used to determine the environmental loads. This methodology has proven to be adequate for designing the suction buckets since it considers all the environmental conditions and possible soil failure mechanisms.

During the design process, it was observed that increasing the diameter, D , and the thickness, t , the soil capacity increased as well. However, increasing the bucket length, L , could lead to tension forces in the sand to which it has no resistance. In the end, the dimensions $D = 0.9$ m, $L = 1$ m, and $t = 0.030$ m were chosen for the bucket final design. For these dimensions the length of the bucket can be totally embedded during suction installation. This design can be in the future optimized for the minimum use of material, however it assures that the bucket has a conservative design.

The final design dimensions were confirmed by FEM analysis, since no considerable deformation was detected, guarantying the lander's stability. The maximum deformation detected was $\Delta = 6.3$ mm. Furthermore, the maximum stresses calculated during the simulation allowed to select a material for the suction buckets. The steel ASTM A36 was the material chosen with a safety factor, SF , equal to 694. For this application, the safety factor, SF , is very high since a structural steel

was chosen according to offshore wind turbine standard codes for a loading regime considerably lower than the ones registered in this industry.

It is important to refer that suction buckets are a fairly recent development in the offshore industry when compared to monopiles and gravity-based foundations, for example. The guidelines referred in this study have a little more than a couple years and therefore a lot of investigation is being made to try to fully understand the mechanisms behind this type of foundation. As shown in this work, it has many advantages regarding offshore applications and the recently increase in research about this topic proves that it is a technology with future.

6.2 Future Work

During the design process, assumptions and simplifications were made to find an analytical solution. However, some of these choices could lead to a less accurate solution. Thus, some notes and suggestions are made so that a better design can be achieved in the future.

Although the water loads were calculated using the modified Morison's equation, this equation does not consider the transversal and tangential force components. A better solution would be to use a CFD simulation to attain more accurate values for the forces acting on the lander.

During this work, the sand properties were approximated by average values since no soil sample was available. The soil properties have a huge influence on the suction bucket design and a slight change can be the difference between a possible solution and a not admissible one. Thus, for a more accurate design of the suction bucket, a soil sample must be used to find out the soil properties of the area where it will be installed.

If a complete suction bucket design is required, then it is also necessary to design the hatch/vent through which the water can flow during the downward movement. Furthermore, other design limitations could have also been considered to confirm the final design. Cavitation inside the bucket, buckling of the steel buckets and other types of soil failure (squeezing, plug uplift) are examples of phenomenons for which the suction bucket should be designed. In this thesis, only the main phenomenons were considered. Lastly, the final suction bucket design should be also verified for the situation where the suction buckets support the total structure weight out of the water.

To help in the installation of the lander, monitoring equipment can be added to the structure. For example, if the lander lands in an irregular soil, a gyroscope with a pressure sensor would be helpful to identify which suction bucket requires more suction pressure to level the lander. Load cells could also be installed in the lander's rods so that the theoretical solutions could be compared with empirical data and therefore validate the design calculations.

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Appendix A

IPMA's data - Python script

The following script was developed in Jupyter Notebook, therefore each line break refers to a new block command. It gathers the monthly data provided by IPMA and plots the wave height, H , and the wave period, T , for a defined location. It also determines the maximum wave height and the respective wave period.

```
1 import xarray as xr
2 import numpy as np
3 import matplotlib.pyplot as plt
4 import pygrib
5 import pandas as pd
6
7 ds_jan_19 = xr.open_dataset('folder path', engine="cfgrib")
8 ds_feb_19 = xr.open_dataset('folder path', engine="cfgrib")
9 ds_mar_19 = xr.open_dataset('folder path', engine="cfgrib")
10 ds_apr_19 = xr.open_dataset('folder path', engine="cfgrib")
11 ds_may_19 = xr.open_dataset('folder path', engine="cfgrib")
12 ds_jun_19 = xr.open_dataset('folder path', engine="cfgrib")
13 ds_jul_19 = xr.open_dataset('folder path', engine="cfgrib")
14 ds_aug_19 = xr.open_dataset('folder path', engine="cfgrib")
15 ds_sep_19 = xr.open_dataset('folder path', engine="cfgrib")
16 ds_oct_19 = xr.open_dataset('folder path', engine="cfgrib")
17 ds_nov_19 = xr.open_dataset('folder path', engine="cfgrib")
18 ds_dec_19 = xr.open_dataset('folder path', engine="cfgrib")
19 ds_jan_20 = xr.open_dataset('folder path', engine="cfgrib")
20 ds_feb_20 = xr.open_dataset('folder path', engine="cfgrib")
21 ds_mar_20 = xr.open_dataset('folder path', engine="cfgrib")
22 ds_apr_20 = xr.open_dataset('folder path', engine="cfgrib")
23 ds_may_20 = xr.open_dataset('folder path', engine="cfgrib")
24 ds_jun_20 = xr.open_dataset('folder path', engine="cfgrib")
25 ds_jul_20 = xr.open_dataset('folder path', engine="cfgrib")
26 ds_aug_20 = xr.open_dataset('folder path', engine="cfgrib")
27 ds_sep_20 = xr.open_dataset('folder path', engine="cfgrib")
28 ds_oct_20 = xr.open_dataset('folder path', engine="cfgrib")
```

```
29 ds_nov_20 = xr.open_dataset('folder path', engine="cfgrib")
30 ds_dec_20 = xr.open_dataset('folder path', engine="cfgrib")
31
32 leixoes_jan_19 = ds_jan_19.isel(latitude=26, longitude=18)
33 leixoes_feb_19 = ds_feb_19.isel(latitude=26, longitude=18)
34 leixoes_mar_19 = ds_mar_19.isel(latitude=26, longitude=18)
35 leixoes_apr_19 = ds_apr_19.isel(latitude=26, longitude=18)
36 leixoes_may_19 = ds_may_19.isel(latitude=26, longitude=18)
37 leixoes_jun_19 = ds_jun_19.isel(latitude=26, longitude=18)
38 leixoes_jul_19 = ds_jul_19.isel(latitude=26, longitude=18)
39 leixoes_aug_19 = ds_aug_19.isel(latitude=26, longitude=18)
40 leixoes_sep_19 = ds_sep_19.isel(latitude=26, longitude=18)
41 leixoes_oct_19 = ds_oct_19.isel(latitude=26, longitude=18)
42 leixoes_nov_19 = ds_nov_19.isel(latitude=26, longitude=18)
43 leixoes_dec_19 = ds_dec_19.isel(latitude=26, longitude=18)
44 leixoes_jan_20 = ds_jan_20.isel(latitude=26, longitude=18)
45 leixoes_feb_20 = ds_feb_20.isel(latitude=26, longitude=18)
46 leixoes_mar_20 = ds_mar_20.isel(latitude=26, longitude=18)
47 leixoes_apr_20 = ds_apr_20.isel(latitude=26, longitude=18)
48 leixoes_may_20 = ds_may_20.isel(latitude=26, longitude=18)
49 leixoes_jun_20 = ds_jun_20.isel(latitude=26, longitude=18)
50 leixoes_jul_20 = ds_jul_20.isel(latitude=26, longitude=18)
51 leixoes_aug_20 = ds_aug_20.isel(latitude=26, longitude=18)
52 leixoes_sep_20 = ds_sep_20.isel(latitude=26, longitude=18)
53 leixoes_oct_20 = ds_oct_20.isel(latitude=26, longitude=18)
54 leixoes_nov_20 = ds_nov_20.isel(latitude=26, longitude=18)
55 leixoes_dec_20 = ds_dec_20.isel(latitude=26, longitude=18)
56
57 df_jan_19 = leixoes_jan_19.to_dataframe()
58 df_feb_19 = leixoes_feb_19.to_dataframe()
59 df_mar_19 = leixoes_mar_19.to_dataframe()
60 df_apr_19 = leixoes_apr_19.to_dataframe()
61 df_may_19 = leixoes_may_19.to_dataframe()
62 df_jun_19 = leixoes_jun_19.to_dataframe()
63 df_jul_19 = leixoes_jul_19.to_dataframe()
64 df_aug_19 = leixoes_aug_19.to_dataframe()
65 df_sep_19 = leixoes_sep_19.to_dataframe()
66 df_oct_19 = leixoes_oct_19.to_dataframe()
67 df_nov_19 = leixoes_nov_19.to_dataframe()
68 df_dec_19 = leixoes_dec_19.to_dataframe()
69 df_jan_20 = leixoes_jan_20.to_dataframe()
70 df_feb_20 = leixoes_feb_20.to_dataframe()
71 df_mar_20 = leixoes_mar_20.to_dataframe()
72 df_apr_20 = leixoes_apr_20.to_dataframe()
73 df_may_20 = leixoes_may_20.to_dataframe()
74 df_jun_20 = leixoes_jun_20.to_dataframe()
75 df_jul_20 = leixoes_jul_20.to_dataframe()
76 df_aug_20 = leixoes_aug_20.to_dataframe()
77 df_sep_20 = leixoes_sep_20.to_dataframe()
```

```
78 df_oct_20 = leixoes_oct_20.to_dataframe()
79 df_nov_20 = leixoes_nov_20.to_dataframe()
80 df_dec_20 = leixoes_dec_20.to_dataframe()
81
82 df_final = pd.concat([df_jan_19, df_feb_19, df_mar_19, df_apr_19, df_may_19,
                        df_jun_19, df_jul_19, df_aug_19, df_sep_19, df_oct_19, df_dec_19, df_jan_20,
                        df_feb_20, df_mar_20, df_apr_20, df_may_20, df_jun_20, df_jul_20, df_aug_20,
                        df_sep_20, df_oct_20, df_dec_20])
83
84 ax = plt.gca()
85 df_final.plot(kind='line', y='hmax', color='red', ax=ax, legend=None)
86 plt.ylabel('H / m')
87 plt.xlabel('t / h')
88 plt.savefig('folder path\\Hmax.png')
89 plt.savefig('folder path\\IPMA\\Hmax.pdf')
90 plt.show()
91
92 ax = plt.gca()
93 df_final.plot(kind='line', y='mwp', ax=ax, legend=None)
94 plt.ylabel('T / s')
95 plt.xlabel('t / h')
96 plt.savefig('folder path\\IPMA\\Tm.png')
97 plt.savefig('folder path\\IPMA\\Tm.pdf')
98 plt.show()
99
100 df_final.loc[df_final['hmax'].idxmax()]
```


Appendix B

Wave theories - MATLAB script

This script calculates the wave velocity and acceleration quantities according to different wave theories.

```
1 function F = dispersion(x)
2 g = 9.81;
3
4 % wave parameters
5 d = 200;           % depth (m)
6 T = 12.43;         % wave period (s)
7
8 F = ((g/(2*pi*x))*tanh(2*pi*d/x))^(1/2)-T;
9 end
10
11 % physical constants
12
13 g = 9.81 ;         %gravity acceleration (m/s^2)
14
15 % wave parameters
16
17 H = 14.03 ;        % height (m)
18 T = 12.43 ;        % wave period (s)
19 d = 200 ;          % depth (m)
20
21 ivalue = 200;       % first iteration value
22 lambda = fsolve(@dispersion,ivalue) % wave length (m)
23
24 c = lambda/T        % phase velocity (m/s)
25 k = 2*pi/lambda;    % wave number (rad/m)
26 w = 2*pi/T;         % wave angular frequency (Hz)
27 y = -198.7;         % distance from mean free surface (m)
28
29 t = 0:0.01:5*T;
30 x = 0:0.01:5*T;
31 teta = k*x-w*t;     % beta = 0
```

```

32 % Airy solution
33
34 %% Surface elevation
35
36 eta_linear = H/2*cos(teta);
37
38 %% particle velocity
39
40 % general
41 %u = ((pi*H)/T)*(cosh(k*(y+d))/sinh(k*d))*cos(teta);
42 %v = ((pi*H)/T)*(sinh(k*(y+d))/sinh(k*d))*sin(teta);
43
44 % deep
45 u = ((pi*H)/T)*exp(k*y)*cos(teta);
46 v = ((pi*H)/T)*exp(k*y)*sin(teta);
47
48 %% particle acceleration
49
50 % general
51 %u_dot = ((2*pi^2*H)/T^2)*(cosh(k*(y+d))/sinh(k*d))*sin(teta);
52 %v_dot = ((-2*pi^2*H)/T^2)*(sinh(k*(y+d))/sinh(k*d))*cos(teta);
53
54 %deep
55 u_dot = ((2*pi^2*H)/T^2)*exp(k*y)*sin(teta);
56 v_dot = ((-2*pi^2*H)/T^2)*exp(k*y)*cos(teta);
57
58 % Stokes' 2nd Order
59
60 %% Surface elevation
61
62 eta_2nd = ((pi*H^2)/(8*lambda))*(cosh(k*d)/(sinh(k*d)^3))*(2+cosh(2*k*d))*cos(2*
    teta);
63 eta_stokes = eta_linear + eta_2nd;
64
65 %% particle velocity
66
67 % general
68 %u_stokes = ((pi*H)/T)*(cosh(k*(y+d))/sinh(k*d))*cos(teta) + (3/(4*c))*((pi*H)/T)
    ^2*(cosh(2*k*(y+d))/(sinh(k*d))^4)*cos(2*teta);
69 %v_stokes = ((pi*H)/T)*(sinh(k*(y+d))/sinh(k*d))*sin(teta) + (3/(4*c))*((pi*H)/T)
    ^2*(sinh(2*k*(y+d))/(sinh(k*d))^4)*sin(2*teta);
70 %u2_stokes = (3/(4*c))*((pi*H)/T)^2*(cosh(2*k*(y+d))/(sinh(k*d))^4)*cos(2*teta);
71 %v2_stokes = (3/(4*c))*((pi*H)/T)^2*(sinh(2*k*(y+d))/(sinh(k*d))^4)*sin(2*teta);
72
73 % deep
74 u_stokes = ((pi*H)/T)*exp(k*y)*cos(teta) + (3/(4*c))*((pi*H)/T)^2*(cosh(2*k*(y+d))
    /(sinh(k*d))^4)*cos(2*teta);
75 v_stokes = ((pi*H)/T)*exp(k*y)*sin(teta) + (3/(4*c))*((pi*H)/T)^2*(sinh(2*k*(y+d))
    /(sinh(k*d))^4)*sin(2*teta);

```

```

76 u2_stokes = (3/(4*c))*((pi*H)/T)^2*(cosh(2*k*(y+d))/(sinh(k*d))^4)*cos(2*teta);
77 v2_stokes = (3/(4*c))*((pi*H)/T)^2*(sinh(2*k*(y+d))/(sinh(k*d))^4)*sin(2*teta);
78
79 %% particle acceleration
80
81 % general
82 %u_dot_stokes = ((2*pi^2*H)/T^2)*(cosh(k*(y+d))/sinh(k*d))*sin(teta) + ((3*pi)/
    lambda)*((pi*H)/T)^2*(cosh(2*k*(y+d))/sinh(k*d)^4)*sin(2*teta);
83 %v_dot_stokes = ((-2*pi^2*H)/T^2)*(sinh(k*(y+d))/sinh(k*d))*cos(teta) + (-3*pi)/
    lambda)*((pi*H)/T)^2*(sinh(2*k*(y+d))/sinh(k*d)^4)*cos(2*teta);
84 %u2_dot_stokes = (3*pi)/lambda*((pi*H)/T)^2*(cosh(2*k*(y+d))/sinh(k*d)^4)*sin(2*
    teta);
85 %v2_dot_stokes = (-3*pi)/lambda*((pi*H)/T)^2*(sinh(2*k*(y+d))/sinh(k*d)^4)*cos(2*
    teta);
86
87 % deep
88 u_dot_stokes = ((2*pi^2*H)/T^2)*exp(k*y)*sin(teta) + ((3*pi)/lambda)*((pi*H)/T)^2*(
    cosh(2*k*(y+d))/sinh(k*d)^4)*sin(2*teta);
89 v_dot_stokes = ((-2*pi^2*H)/T^2)*exp(k*y)*cos(teta) + (-3*pi)/lambda*((pi*H)/T)
    ^2*(sinh(2*k*(y+d))/sinh(k*d)^4)*cos(2*teta);
90 u2_dot_stokes = ((3*pi)/lambda)*((pi*H)/T)^2*(cosh(2*k*(y+d))/sinh(k*d)^4)*sin(2*
    teta);
91 v2_dot_stokes = ((-3*pi)/lambda)*((pi*H)/T)^2*(sinh(2*k*(y+d))/sinh(k*d)^4)*cos(2*
    teta);
92
93 % plots
94
95 % surface elevation
96
97 figure()
98 plot(t,eta_stokes,'LineWidth',1.5)
99 hold on
100 plot(t,eta_linear,'--','LineWidth',1.5)
101 hold on
102 plot(t,eta_2nd,'-.','LineWidth',1.5)
103 ylabel('H /m','Interpreter','latex')
104 xlabel('t /s','Interpreter','latex')
105 legend('1st + 2nd terms (Stokes)','1st term (Airy)','2nd term')
106 xlim([0, 40])
107
108 % velocities
109
110 figure()
111 plot(t,u,'LineWidth',1.5,'Color','black')
112 hold on
113 plot(t,v,'LineWidth',1.5,'Color','#FF0000')
114 hold on
115 plot(t,u_stokes,'--','LineWidth',2.5,'Color','#EDB120')
116 hold on

```

```

117 plot(t,v_stokes,'--','LineWidth',2.5,'Color','#4DBEEE')
118 hold on
119 plot(t,u2_stokes,'LineWidth',1.5,'Color','#77AC30')
120 hold on
121 plot(t,v2_stokes,'-.','LineWidth',1.5,'Color','#7E2F8E')
122 hold on
123 ylabel('V /m/s','Interpreter','latex')
124 xlabel('t /s','Interpreter','latex')
125 legend('u_l','v_l','u_stokes','v_stokes','u_stokes-2nd term','v_stokes-2nd term','Interpreter','latex')
126 xlim([0, 40])
127
128 figure()
129 plot(t,u2_stokes,'LineWidth',1.5,'Color','#77AC30')
130 hold on
131 plot(t,v2_stokes,'-.','LineWidth',1.5,'Color','#7E2F8E')
132 ylabel('V /m/s','Interpreter','latex')
133 xlabel('t /s','Interpreter','latex')
134 legend('u_stokes-2nd term','v_stokes-2nd term','Interpreter','latex')
135 xlim([0, 40])
136
137 % acellerations
138
139 figure()
140 plot(t,u_dot,'LineWidth',1.5,'Color','black')
141 hold on
142 plot(t,v_dot,'LineWidth',1.5,'Color','#FF0000')
143 hold on
144 plot(t,u_dot_stokes,'--','LineWidth',2.5,'Color','#EDB120')
145 hold on
146 plot(t,v_dot_stokes,'--','LineWidth',2.5,'Color','#4DBEEE')
147 hold on
148 plot(t,u2_dot_stokes,'LineWidth',1.5,'Color','#77AC30')
149 hold on
150 plot(t,v2_dot_stokes,'-.','LineWidth',1.5,'Color','#7E2F8E')
151 hold on
152 ylabel('V ~ /m/s^2','Interpreter','latex')
153 xlabel('t /s','Interpreter','latex')
154 legend('u_l','v_l','u_stokes','v_stokes','u_stokes-2nd term','v_stokes-2nd term','Interpreter','latex')
155 xlim([0, 40])
156
157 figure()
158 plot(t,u2_dot_stokes,'LineWidth',1.5,'Color','#77AC30')
159 hold on
160 plot(t,v2_dot_stokes,'-.','LineWidth',1.5,'Color','#7E2F8E')
161 ylabel('V ~ /m/s^2','Interpreter','latex')
162 xlabel('t /s','Interpreter','latex')
163 legend('u_stokes-2nd term','v_stokes-2nd term','Interpreter','latex')
164 xlim([0, 40])

```

Appendix C

Water forces - MATLAB script

Before running this script, it is necessary to verify if "Optimization Toolbox" is installed. This script calculates the water forces acting on an inclined cylinder through numerical integration.

```
1 function F = dispersion(x)
2 g = 9.81;
3
4 % wave parameters
5 d = 200;          % depth (m)          %150
6 T = 12.43;        % wave period (s)    %12
7
8 F = ((g/(2*pi*x))*tanh(2*pi*d/x))^(1/2)-T;
9
10 end
11
12
13 % physical constants
14
15 g = 9.81;          %gravity accelaration (m/s^2)
16
17 % wave parameters
18
19 H = 14.03;         % wave height (m)
20 T = 12.43;         % wave period (s)
21 d = 200;           % wave depth (m)
22
23 ivalue = 200;      % first iteration value
24 lambda = fsolve(@dispersion,ivalue); % wave length (m)
25
26 c = lambda/T;      % phase velocity (m/s)
27 k = 2*pi/lambda;   % wave number (rad/m)
28 omega = 2*pi/T;    % wave angular frequency (rad/s)
29
30 t = 0:T/100:3*T;   % time (s)
31 x = 0:lambda/100:lambda; % distance of propagation (m)
```

```

32
33 % current parameters
34
35 alpha = 1/7;           % exponent
36 Uco = 1.0;             % current velocity at the sea level, y=0 (m/s)
37
38 % cylinder parameters
39
40 zeta = 90 * (pi/180);   % angle of the cylinder axis to the vertical axis (
    rad)
41 xi = 120 * (pi/180);    % angle of the cylinder projection to the x-axis (rad)
42 l = 0.298;              % cylinder length (m)
43 D = 0.07;              % cylinder diameter (m)
44 hx = l*(sin(zeta)*cos(xi)); % cylinder projected length in x-axis (m)
45 hy = l*(cos(zeta));      % cylinder projected length in y-axis (m)
46 hz = l*(sin(zeta)*sin(xi)); % cylinder projected length in z-axis (m)
47
48 Cx = sin(zeta)*cos(xi);
49 Cy = cos(zeta);
50 Cz = sin(zeta)*sin(xi);
51
52 % Particle and current velocity
53
54 y = -0;                % distance from mean free surface (m)
55
56 u_w = zeros(length(t),1);
57 u_c = zeros(length(t),1);
58 u = zeros(length(t),1);
59 v = zeros(length(t),1);
60 ux = zeros(length(t),1);
61 uy = zeros(length(t),1);
62 uz = zeros(length(t),1);
63 w = zeros(length(t),1);
64
65 for i = 1:length(t)
66
67     teta = k*x(1)-omega*t(i);
68
69     % Stokes 2nd order reduced to the 1st term -> Airy
70     %% particle velocity
71
72     u_w(i,1) = ((pi*H)/T)*exp(k*y)*cos(teta);
73     u_c(i,1) = Uco*((y+d)/d)^alpha;
74     u(i,1) = u_w(i,1) + u_c(i,1);
75     v(i,1) = ((pi*H)/T)*exp(k*y)*sin(teta);
76
77     ux(i,1) = u(i,1)-Cx*(Cx*u(i,1)+Cy*v(i,1));
78     uy(i,1) = v(i,1)-Cy*(Cx*u(i,1)+Cy*v(i,1));
79     uz(i,1) = -Cz*(Cx*u(i,1)+Cy*v(i,1));

```

```

80
81     w(i,1) = sqrt(ux(i,1)^2+uy(i,1)^2+uz(i,1)^2);
82
83     %% particle acceleration
84
85     %u_dot(i,j) = ((2*pi^2*H)/T^2)*exp(k*y)*sin(teta);
86     %v_dot(i,j) = ((-2*pi^2*H)/T^2)*exp(k*y)*cos(teta);
87 end
88
89 % horizontal and verticle particle velocities
90
91 figure
92 plot(t,u_w(:,1),'--','LineWidth',2)
93 hold on
94 plot(t,u_c(:,1),'--','LineWidth',2)
95 hold on
96 plot(t,u(:,1),'LineWidth',1)
97 hold on
98 plot(t,v(:,1),'LineWidth',1)
99 legend('u_w','u_c','u','v')
100
101 % projection of horizontal velocity in the 3 axis
102
103 figure
104 plot(t,ux(:,1),'LineWidth',1)
105 hold on
106 plot(t,uy(:,1),'LineWidth',1)
107 hold on
108 plot(t,uz(:,1),'--','LineWidth',2)
109 legend('u_x','u_y','u_z')
110
111 % contribution of each component on normal velocity, w
112
113 figure
114 plot(t,w(:,1),'LineWidth',1)
115 hold on
116 plot(t,sqrt(ux(:,1).^2),'--','LineWidth',2.5)
117 hold on
118 plot(t,sqrt(uy(:,1).^2),'-.','LineWidth',1.5)
119 hold on
120 plot(t,sqrt(uz(:,1).^2),'LineWidth',2.5)
121 legend('w','ux','uy','uz')
122 title(['Cx = ' num2str(Cx) ' Tau = ' num2str(zeta*(180/pi)) ' Xi = ' num2str(xi
    *(180/pi))])
123
124 clear y
125
126 % Forces induced
127

```

```

128 Fi_x = zeros(length(t),1);
129 Fd_x = zeros(length(t),1);
130 F_x = zeros(length(t),1);
131 Fi_y = zeros(length(t),1);
132 Fd_y = zeros(length(t),1);
133 F_y = zeros(length(t),1);
134 Fi_z = zeros(length(t),1);
135 Fd_z = zeros(length(t),1);
136 F_z = zeros(length(t),1);
137
138 p = 1025; % sea water density (kg/m^3)
139 Ai = p*(pi/4)*D^2;
140 Ad = (1/2)*p*D;
141
142 Cm = 2.0; % inertia coefficient
143 Cd = 1.3; % drag coefficient
144
145 for i = 1:length(t)
146
147     teta = k*x(1)-omega*t(i);
148
149     %int_u = (pi*H/T)*(exp(-k*d)*(exp(k*h)-1)/k)*cos(teta) + ...;
150     %int_v = (pi*H/T)*(exp(-k*d)*(exp(k*h)-1)/k)*sin(teta);
151
152     int_u_dot_x = ((2*pi^2*H)/(T^2*k))*(exp(k*(-d+hy))-exp(k*(-d)))*sin(teta);
153     int_v_dot_x = ((-2*pi^2*H)/(T^2*k))*(exp(k*(-d+hy))-exp(k*(-d)))*cos(teta);
154
155     int_u_dot_y = ((2*pi^2*H)/(T^2*k))*(exp(k*(-d+hy))-exp(k*(-d)))*sin(teta);
156     int_v_dot_y = ((-2*pi^2*H)/(T^2*k))*(exp(k*(-d+hy))-exp(k*(-d)))*cos(teta);
157
158     int_u_dot_z = ((2*pi^2*H)/(T^2*k))*(exp(k*(-d+hy))-exp(k*(-d)))*sin(teta);
159     int_v_dot_z = ((-2*pi^2*H)/(T^2*k))*(exp(k*(-d+hy))-exp(k*(-d)))*cos(teta);
160
161
162     u_velocity = @(y) ((pi*H)/T)*exp(k*y)*cos(teta) + Uco*((y+d)/d).^alpha;
163     v_velocity = @(y) ((pi*H)/T)*exp(k*y)*sin(teta);
164
165     u_x = @(y) (u_velocity(y)-Cx*(Cx*u_velocity(y)+Cy*v_velocity(y)));
166     u_y = @(y) (v_velocity(y)-Cy*(Cx*u_velocity(y)+Cy*v_velocity(y)));
167     u_z = @(y) (-Cz*(Cx*u_velocity(y)+Cy*v_velocity(y)));
168
169     f_int_w_x = @(y) (sqrt((u_x(y)).^2+(u_y(y)).^2+(u_z(y)).^2).*u_x(y));
170     f_int_w_y = @(y) (sqrt((u_x(y)).^2+(u_y(y)).^2+(u_z(y)).^2).*u_y(y));
171     f_int_w_z = @(y) (sqrt((u_x(y)).^2+(u_y(y)).^2+(u_z(y)).^2).*u_z(y));
172
173
174     Fi_x(i,1) = Cm*Ai*((1-Cx^2)*int_u_dot_x-Cx*Cy*int_v_dot_x);
175     int_w_x = integral(@(y) f_int_w_x(y),-d,-d+hy);
176     Fd_x(i,1) = Cd*Ad*int_w_x;

```

```

177
178     F_x(i,1) = Fi_x(i,1) + Fd_x(i,1);
179
180     Fi_y(i,1) = Cm*Ai*((1-Cy^2)*int_v_dot_y-Cx*Cy*int_u_dot_y);
181     int_w_y = integral(@(y) f_int_w_y(y),-d,-d+hy);
182     Fd_y(i,1) = Cd*Ad*int_w_y;
183
184     F_y(i,1) = Fi_y(i,1) + Fd_y(i,1);
185
186     Fi_z(i,1) = Cm*Ai*(-Cx*Cz*int_u_dot_z-Cy*Cz*int_v_dot_z);
187     int_w_z = integral(@(y) f_int_w_z(y),-d,-d+hy);
188     Fd_z(i,1) = Cd*Ad*int_w_z;
189
190     F_z(i,1) = Fi_z(i,1) + Fd_z(i,1);
191
192 end
193
194 figure
195 plot(t,Fi_x(:,1))
196 hold on
197 plot(t,Fd_x(:,1))
198 hold on
199 plot(t,F_x(:,1))
200 legend('Fi_x','Fd_x','F_x')
201
202 figure
203 plot(t,Fi_y(:,1))
204 hold on
205 plot(t,Fd_y(:,1))
206 hold on
207 plot(t,F_y(:,1))
208 legend('Fi_y','Fd_y','F_y')
209
210 figure
211 plot(t,Fi_z(:,1))
212 hold on
213 plot(t,Fd_z(:,1))
214 hold on
215 plot(t,F_z(:,1))
216 legend('Fi_z','Fd_z','F_z')
217
218 max(abs(F_x))
219 max(abs(F_y))
220 max(abs(F_z))

```


Appendix D

Installation design - MATLAB script

Before running this code, it is necessary to verify if "Optimization Toolbox" is installed. This script calculates the bucket self-weight penetration, the suction-assisted penetration, and the soil bearing capacity.

```
1 % Suction Caisson Dimensions
2
3 Do = 0.9; % Outer Diameter (m)
4 L = 1; % Length (m)
5 t = 0.030; % Thickness (m)
6 Di = Do-2*t; % Inner Diameter (m)
7 D = (Do+Di)/2; % Average Diameter (m)
8
9 % Effective vertical load
10
11 g = 9.81; % Gravity acceleration (m/s^2)
12 rho_steel = 7900; % Steel density (kg/m^3)
13 rho_sea_water = 1025; % Sea water density (kg/m^3)
14 V_disco = ((pi*Do^2)/4)*t;
15 V_tubo = ((pi*Do^2)/4 - (pi*Di^2)/4)*(L-t);
16 V_bucket = V_disco + V_tubo; % Suction bucket volume (m^3)
17 Fg_bucket = rho_steel*V_bucket*g;
18 Fb_bucket = rho_sea_water*V_bucket*g;
19 Fg_lander = 1555.87 + 3*Fg_bucket;
20 Fb_lander = 1307.18 + 3*Fb_bucket;
21 V = (Fg_lander-Fb_lander)/3; % Effective vertical load (N)
22
23 % Sand properties
24
25 phi = 30 * (pi/180); % Soil angle of friction (rad)
26 gama = 8.5 *10^3; % Effective unit weight (N/m^3)
27 %K = ; % Factor relating vertical stress to
    horizontal stress
28 %delta = ; % Interface angle of friction (rad)
29 ktani = 0.13;
```

```

30 ktano = 0.13;
31 % Other parameteres
32
33 m = 1.5;
34 Zi = Di/(4*ktani);
35 Zo = Do*(m^2-1)/(4*ktano);
36 Nq = exp(pi*tan(phi))*(tan(pi/4+phi/2))^2;
37 Ngama = exp(1/6*(pi+3*pi^2*tan(phi)))*(tan(phi))^(2*pi/5);
38
39 h = 0:0.01:5*L;
40 R = gama*Zo^2*(exp(h/Zo)-1-(h/Zo))*ktano*pi*Do + gama*Zi^2*(exp(h/Zi)-1-(h/Zi))*
    ktani*pi*Di + (gama*Zo*(exp(h/Zo)-1)*Nq + gama*t/2*Ngama)*pi*D*t;
41
42 hivalue = 1; % first iteration value
43 h_sw = fsolve(@selfweight,hivalue); % self-weight penetration (m)
44
45 grayColor = [.6 .6 .6];
46
47 R_safety = R.*(3/2);
48
49 figure()
50 plot(R,h,'LineWidth',2,'Color','blue')
51 hold on
52 xline(V,'--','LineWidth',2,'Color',grayColor)
53 hold on
54 plot(V,h_sw,'o','Color','black','MarkerFaceColor','black')
55 set(gca,'Ydir','reverse')
56 xlabel('Penetration resistance /N')
57 ylabel('Penetration depth /m')
58 xlim([0 1])
59 ylim([0 0.9])
60 legend('Resistance estimate','Self weight','Intersection point')
61 text(V+500,h_sw-0.025,['(' num2str(V) ', ' num2str(h_sw) ')'])
62 grid()
63
64
65 hi = h_sw:0.01:20*L;
66 s = zeros(1,length(hi));
67
68 for j = 1:length(hi) % first iteration value
69     fun = @(s)suction(s,hi(j));
70     s(j) = fsolve(fun,1); % suction pressure (Pa)
71 end
72
73 hmax = fsolve(@maxdepth,hivalue);
74
75 figure()
76 plot([0;0],[0;h_sw],'LineWidth',2,'Color','b')
77 hold on

```

```

78 plot(s*10^(-3),hi,'LineWidth', 2,'Color','b')
79 hold on
80 yline(hmax,'LineWidth', 2,'Color','red')
81 hold on
82 yline(L,'LineWidth', 2,'Color','black')
83 hold on
84 set(gca,'Ydir','reverse')
85 xlabel('Required suction, s/ kPa')
86 ylabel('Depth, z/ m')
87 xlim([0 20])
88 ylim([0 1.5*L])
89 legend('Suction Pressure','','Soil Failure','Length Limit')
90 grid()
91
92 %soil piping
93 figure()
94 plot([0;0],[0;h_sw],'LineWidth', 2,'Color','b')
95 hold on
96 plot(s*10^(-3),hi,'LineWidth', 2,'Color','b')
97 hold on
98 yline(hmax,'LineWidth', 2,'Color','red')
99 hold on
100 yline(L,'LineWidth', 2,'Color','black')
101 hold on
102 set(gca,'Ydir','reverse')
103 xlabel('Required suction, s/ kPa')
104 ylabel('Depth, z/ m')
105 xlim([0 20])
106 ylim([0 2])
107 legend('Suction Pressure','','Soil Failure','Length Limit')
108 grid()
109
110
111 function H = selfweight(h)
112
113 % Loads
114
115 V = 6734; % Effective vertical load (N)
116
117 % Suction Caisson Dimensions
118
119 Do = 0.9; % External Diameter (m)
120 L = 1; % Length (m)
121 t = 0.030; % Thickness (m)
122 Di = Do-2*t; % Internal Diameter (m)
123 D = (Do+Di)/2; % Average Diameter (m)
124
125 % Sand properties
126

```

```

127 phi = 30 * (pi/180); % Soil angle of friction (rad)
128 gama = 8.5 * 10^3; % Effective unit weight (N/m^3)
129 %K = ; % Factor relating vertical stress to horizontal stress
130 %delta = ; % Interface angle of friction (rad)
131 ktani = 0.13;
132 ktano = 0.13;
133
134 % Other parameteres
135
136 m = 1.5;
137 Zi = Di/(4*ktani);
138 Zo = Do*(m^2-1)/(4*ktano);
139 Nq = exp(pi*tan(phi))*(tan(pi/4+phi/2))^2;
140 Ngama = exp(1/6*(pi+3*pi^2*tan(phi)))*(tan(phi))^(2*pi/5);
141 %sigma_vi = gama*Zi*(exp(z./Zi)-1);
142 %sigma_vo = gama*Zo*(exp(z./Zo)-1);
143 %sigma_end = gama*Zo*(exp(h/Zo)-1)*Nq + gama*t/2*Ngama;
144
145 H = gama*Zo^2*(exp(h/Zo)-1-(h/Zo))*ktano*pi*Do + gama*Zi^2*(exp(h/Zi)-1-(h/Zi))*
    ktani*pi*Di + (gama*Zo*(exp(h/Zo)-1)*Nq + gama*t/2*Ngama)*pi*D*t - V;
146
147 end
148
149
150 function S = suction(s,h)
151
152 % Loads
153
154 V = 6734; % Effective vertical load (N)
155
156 % Suction Caisson Dimensions
157
158 Do = 0.9; % External Diameter (m)
159 %L = 1; % Length (m)
160 t = 0.030; % Thickness (m)
161 Di = Do-2*t; % Internal Diameter (m)
162 D = (Do+Di)/2; % Average Diameter (m)
163
164 % Sand properties
165
166 phi = 30 * (pi/180); % Soil angle of friction (rad)
167 gama = 8.5 * 10^3; % Effective unit weight (N/m^3)
168 %K = ; % Factor relating vertical stress to horizontal stress
169 %delta = ; % Interface angle of friction (rad)
170 ktani = 0.13;
171 ktano = 0.13;
172
173 % Other parameteres
174

```

```

175 m = 1.5;
176 Zi = Di/(4*ktani);
177 Zo = Do*(m^2-1)/(4*ktano);
178 Nq = exp(pi*tan(phi))*(tan(pi/4+phi/2))^2;
179 Ngama = exp(1/6*(pi+3*pi^2*tan(phi)))*(tan(phi))^(2*pi/5);
180
181 c0 = 0.45;
182 c1 = 0.36;
183 c2 = 0.48;
184 ki = 2.5;
185 ko = 2.5;
186 kf = ki/ko;
187
188 a1 = @(h) (c0-c1*(1-exp(-h/(c2*D))));
189 a = @(h) ((a1(h)*kf)/((1-a1(h))+a1(h)));
190
191
192 S = (gama+(a(h)*s)/h)*Zo^2*(exp(h/Zo)-1-(h/Zo))*ktano*pi*Do + (gama-((1-a(h))*s)/h)
    *Zi^2*(exp(h/Zi)-1-(h/Zi))*ktani*pi*Di + ((gama-((1-a(h))*s)/h)*Zi*(exp(h/Zi)
    -1)*Nq + gama*t*Ngama)*pi*D*t - V - s*(pi*Di^2)/4;
193
194 end

```

Appendix E

Technical drawing of the suction bucket

When the file is fully shown in A4 format, the scale on the technical drawing is correct.

