Shallow foundations for subsea structures: 
a comparison between design codes and numerical analysis

Supervisors:
PROF. M.A. HICKS
DR. F. PISANÒ
DR. R.B.J. BRINKGREVE
IR. R. BURGERS
DR. N.B. YENIGUL

Author:
B.A.M. VAN DE RIET

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Shallow foundations for subsea structures: a comparison between design codes and numerical analysis

by

Bart van de Riet

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Student number: 4050185
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Thesis committee: prof. M.A. Hicks, TU Delft
dr. F. Pisanò, TU Delft
dr. R.B.J. Brinkgreve, TU Delft and Plaxis bv.
ir. R. Burgers, Allseas bv.
dr. N.B. Yenigul, Allseas bv.

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An electronic version of this thesis is available at http://repository.tudelft.nl/.
Preface

This thesis report is the final result of my research to obtain the Master of Science degree in Civil Engineering with the specialism Geo-Engineering at Delft University of Technology. The topic of this research is the possible embedded conservatism in design codes used to make a foundation design.

The thesis is conducted at the department of Geo-Engineering of the Delft University of Technology in close collaboration with Allseas B.V. The target audience for this report is Allseas B.V. since this research is performed on behalf of Allseas B.V. in order to optimize the foundation design of subsea structures related to offshore pipelines. Other students with a specialism in Geo-Engineering which are interested in the design codes as used in practice by contractors in order to make a foundation design also are considered as target audience. For students which would like to contribute to this topic, I have given recommendations for possible further research.

Etten-Leur
August 14, 2016

Bart van de Riet
Acknowledgements

The past 11 months I have spend on this project and I would like to thank some of the people which made this master thesis result possible because of their support and help throughout the project.

First I would like to thank Allseas B.V. for introducing me to the topic and providing me a work place within the company. The structural design group in the pipeline department was a nice and interesting place to work during this project and I would like to thank everyone from this group for their time and help. In particular I would like to thank my supervisors from Allseas Raymond and Buket for their support and insights throughout this project.

Of course I also want to thank the other members from my graduation committee Federico Pisanò, Ronald Brinkgreve and Michael Hicks. Their input and feedback on my work during the committee meetings and individual sessions were very usefull.

Also I would like to thank my friends for the good times we had during the master study Geo-Engineering. Finally I want to thank my parents for their support in all kind of ways during the many years of study I enjoyed.
Subsea structures such as pipeline end termination (PLET) structures are currently designed according to the ISO:19901-4 and API RP 2A-WSD design codes by Allseas. It has become interesting to investigate the embedded conservatism in these design codes as not only the increasing demand of installations in larger water depths and/or on poor soil conditions, but also the use of larger pipeline diameters and the increase of the size and weight of subsea structures. This embedded conservatism could have a significant effect on the economical and operational aspects. The economical aspects can be explained by the costs for fabrication and offshore installation and the operational aspects by the operational limits of current vessels.

From a literature review is found that Terzaghi derived an equation for the bearing capacity of the soil for very specific situations. Afterwards several investigators including Brinch Hansen, Vesic and Meyerhof revised the solution in order to determine the bearing capacity of the soil for more general situations. Brinch Hansen is mostly followed by the ISO design code, while Vesic is followed by the API design code.

Two different safety approaches are found from an analytical analysis of the investigated design codes, namely the partial safety factor approach in the ISO design code and the global safety factor approach in the API design code. Both the investigated design codes only determine a safety factor against bearing failure and sliding failure in order to determine the safety of a foundation design.

In this study four case studies are investigated and a review of the site investigation reports is performed. Since no sufficient information regarding soil deformation parameters are provided to Allseas only the embedded conservatism in the design codes regarding soil strength is investigated in this study. For all four case studies an increasing shear strength profile is determined. This profile can however not be modeled in the API design code, therefore a constant shear strength profile which gives an equivalent soil bearing capacity as compared to the increasing shear strength profile is determined. Two methods using the numerical program PLAXIS are applied to determine a constant shear strength profile.

In this investigation the numerical program PLAXIS is used to determine the embedded conservatism in the ISO and API design codes. Two PLAXIS models are used, in the first model the partial safety factor approach is considered and partial load and material factors are applied to the input values. The results of the PLAXIS model are compared to the results from the ISO design code. In the second PLAXIS model the global safety factor approach is considered and the results are compared to the results from the API design code. The ratio between the safety factors against bearing and sliding failure as determined by PLAXIS and the design codes is used for the foundation designs of the four investigated case studies, which indicate that there is some embedded conservatism in the design codes. A sensitivity analysis is performed to investigate the influence of the aspect ratio and the embedment depth on the embedded conservatism. The results of these sensitivity analyses show that a variation in aspect ratio does not have an influence on the embedded conservatism, while embedment depths larger than 0.6 meter do have an influence on the embedded conservatism in both the ISO and API design codes. Three possible sources for the embedded conservatism in the design codes are found in the literature review; the assumption of superposition, the absence of an interdependance of V-H-M loading and the simplifications in situations for which the design codes are derived.

An example calculation is made in order to investigate the cost reduction for a design made by a numerical analysis instead of the analytical design codes. In this calculation is shown that the foundation area can be reduced by 47%. The smaller foundation area results in a cost reduction of 16% for the fabrication of the total subsea structure. An even larger reduction in installation costs could be achieved if the size of the structure can be optimized by a numerical analysis because of the high day rates for Allseas installation vessels.
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### Nomenclature

#### Latin symbols for geometry

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A)</td>
<td>Foundation area</td>
<td>(m^2)</td>
</tr>
<tr>
<td>(A')</td>
<td>Effective foundation area</td>
<td>(m^2)</td>
</tr>
<tr>
<td>(B)</td>
<td>Foundation width</td>
<td>(m)</td>
</tr>
<tr>
<td>(B')</td>
<td>Effective foundation width</td>
<td>(m)</td>
</tr>
<tr>
<td>(D)</td>
<td>Embedment depth</td>
<td>(m)</td>
</tr>
<tr>
<td>(e_1)</td>
<td>Eccentricity in length direction</td>
<td>(m)</td>
</tr>
<tr>
<td>(e_2)</td>
<td>Eccentricity in width direction</td>
<td>(m)</td>
</tr>
<tr>
<td>(h)</td>
<td>Soil layer thickness</td>
<td>(m)</td>
</tr>
<tr>
<td>(L)</td>
<td>Foundation length</td>
<td>(m)</td>
</tr>
<tr>
<td>(L')</td>
<td>Effective foundation length</td>
<td>(m)</td>
</tr>
<tr>
<td>(R)</td>
<td>Radius of foundation base</td>
<td>(m)</td>
</tr>
</tbody>
</table>

#### Latin symbols for soil parameters

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(C)</td>
<td>Compression index of the soil</td>
<td>[(\text{\text{-}})]</td>
</tr>
<tr>
<td>(c)</td>
<td>Cohesion</td>
<td>kPa</td>
</tr>
<tr>
<td>(E)</td>
<td>Young's modulus</td>
<td>MPa</td>
</tr>
<tr>
<td>(E_{\text{inc}})</td>
<td>Young's modulus increase in depth</td>
<td>MPa/m</td>
</tr>
<tr>
<td>(e_0)</td>
<td>Initial void ratio</td>
<td>[(\text{-})]</td>
</tr>
<tr>
<td>(G)</td>
<td>Elastic shear modulus</td>
<td>MPa</td>
</tr>
<tr>
<td>(k)</td>
<td>Undrained shear strength increase in depth</td>
<td>kPa/m</td>
</tr>
<tr>
<td>(N)</td>
<td>Number of load cycles</td>
<td>[(\text{-})]</td>
</tr>
<tr>
<td>(N_{eq})</td>
<td>Equivalent number of load cycles</td>
<td>[(\text{-})]</td>
</tr>
<tr>
<td>(s_u)</td>
<td>Undrained shear strength</td>
<td>kPa</td>
</tr>
<tr>
<td>(s_{u,m})</td>
<td>Undrained shear strength at surface level</td>
<td>kPa</td>
</tr>
<tr>
<td>(s_{u,ave})</td>
<td>Average shear strength over embedment depth</td>
<td>kPa</td>
</tr>
<tr>
<td>(s_{u,0})</td>
<td>Undrained shear strength at skirt tip level</td>
<td>kPa</td>
</tr>
</tbody>
</table>

#### Latin symbols for loading conditions

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(F_h)</td>
<td>Horizontal load</td>
<td>kN</td>
</tr>
<tr>
<td>(F_v)</td>
<td>Vertical load</td>
<td>kN</td>
</tr>
<tr>
<td>(H)</td>
<td>Horizontal load</td>
<td>kN</td>
</tr>
<tr>
<td>(H_u)</td>
<td>Ultimate horizontal load</td>
<td>kN</td>
</tr>
<tr>
<td>(M)</td>
<td>Moment load</td>
<td>kNm</td>
</tr>
<tr>
<td>(M_u)</td>
<td>Ultimate moment</td>
<td>kNm</td>
</tr>
<tr>
<td>(q_0)</td>
<td>Initial effective vertical stress</td>
<td>kPa</td>
</tr>
<tr>
<td>(\Delta q)</td>
<td>Added effective vertical stress</td>
<td>kPa</td>
</tr>
<tr>
<td>(T)</td>
<td>Torsion</td>
<td>kNm</td>
</tr>
<tr>
<td>(V)</td>
<td>Vertical load</td>
<td>kN</td>
</tr>
<tr>
<td>(V_u)</td>
<td>Ultimate vertical load</td>
<td>kN</td>
</tr>
</tbody>
</table>

#### Latin symbols for deformation

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(u_h)</td>
<td>Horizontal settlement</td>
<td>m</td>
</tr>
<tr>
<td>(u_{v,lt})</td>
<td>Long term vertical settlement</td>
<td>m</td>
</tr>
<tr>
<td>(u_{v,st})</td>
<td>Short term vertical settlement</td>
<td>m</td>
</tr>
</tbody>
</table>

#### Latin symbols for bearing and sliding capacity equations

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(b_c)</td>
<td>Foundation base inclination correction factor for cohesion</td>
<td>[(\text{-})]</td>
</tr>
<tr>
<td>(b_q)</td>
<td>Foundation base inclination correction factor for surcharge</td>
<td>[(\text{-})]</td>
</tr>
</tbody>
</table>
### Greek symbols for geometry

- $\beta$: Ground inclination angle
- $\mu$: Foundation base inclination

### Greek symbols for soil parameters

- $\gamma$: Unit weight soil
- $\gamma'$: Effective unit weight soil
- $\nu$: Poisson's ratio
- $\phi$: Friction angle
- $\phi'$: Dilatancy angle

### Greek symbols for loading conditions

- $\alpha$: Angle between long axis and horizontal load direction
- $\theta$: Load inclination angle
- $\tau$: Shear stress
- $\tau_a$: Average shear stress
- $\Delta\tau_a$: Additional shear stress
- $\tau_{cy}$: Cyclic shear stress
- $\tau_0$: Initial shear stress

### Greek symbols for safety factors

- $\gamma_e$: Partial environmental loading safety factor
- $\gamma_G$: Global safety factor
- $\gamma_g$: Partial permanent loading safety factor
- $\gamma_l$: Partial loading safety factor
- $\gamma_m$: Partial material safety factor
- $\gamma_q$: Partial variable loading safety factor
### Greek symbols for deformation

- $\gamma_a$: Average shear strain
- $\gamma_{cy}$: Cyclic shear strain
- $\theta_r$: Rotation of foundation due to overturning moments
- $\theta_t$: Rotation of foundation due to torsion

### Abbreviations

- API: American Petroleum Institute
- CPT: Cone penetration test
- DNV: Det Norske Veritas
- ESA: Effective stress approach
- DSS: Direct simple shear
- FEA: Finite element analysis
- FEM: Finite element method
- FLET: Flowline end termination
- HDD: Horizontal directional drilling
- IEC: International Electrotechnical Commission
- ISO: International Standardization Organisation
- LL: Liquid limit
- LRFD: Load and resistance factor design
- MC: Moisture content
- PL: Plastic limit
- PLEM: Pipeline end manifold
- PLET: Pipeline end termination
- ROV: Remote operated vehicle
- TSA: Total stress approach
- WSD: Working stress design
Introduction

Shallow foundations are used in the offshore industry for subsea structures in pipeline infrastructure. PipeLine End Termination, PLET, is the end part of an offshore pipeline and is investigated in this study. Figure 1.1 presents a part of a pipeline including four wellheads. These wellheads are connected to a hydrocarbon well and to a manifold by jumpers. The manifold is the yellow structure in the center of the wellheads and it collects the fluids from the wells and distributes it over different pipelines. Jumpers connect the manifold to the wellheads and the PLET structures, that are shown as the grey structures in figure 1.1. The jumper is a piece of pipeline capable of dissipating horizontal movements from the manifold or PLET structure due to the m-shape.

Figure 1.1: Overview of pipeline infrastructure with some subsea structures after [1]

Figure 1.2 shows a yellow PLET structure hanging on the red A-frame portal on the vessel Audacia. The PLET is connected to a 24 inch pipeline and is 12.5 meter long and 16.5 meter wide. The weight of the PLET is 134.5 tons, which is equivalent to 90 person cars. Although it is not obvious how huge the PLET structure from figure 1.1, one can see the scale in figure 1.2. The foundations of such PLET structures are the rectangular steel plates, which are also called a mud-mat.

1.1. Problem definition

Subsea structures such as PLET structures are currently designed according to design codes. It has become interesting to investigate the conservatism in the design codes mainly due to the increasing demand of installations in larger water depths and/or on poor soils, together with use of larger pipeline diameters and the increase of the size and weight of subsea structures. If there is conservatism in the design codes it is interesting to investigate how much conservatism there is as it has significant economical and operational aspects.
1.2. Research goals

The economical aspects can be explained by the fabrication costs of a structure and the offshore installation costs. The costs for the fabrication and installation of subsea structures can be reduced if the design can be optimized with respect to the conservatism in the design codes. The operational aspects can be explained by the operational limits of the current vessels. If the subsea structures keep increasing in size then the current vessels at some point will no longer be capable of installing subsea structures.

Hence, the main purpose of this research is to determine whether there is embedded conservatism in the design codes, and how to deal with the possible conservatism in them.

1.2. Research goals

There are some research question formulated in order investigate whether there is conservatism in the design codes for shallow foundation design for subsea structures and how to deal with such conservatism. These research questions can be listed as follows:

- Are the design codes conservative?
- Why are the design codes conservative?
- What are the factors in the codes that influence the design of shallow foundations?
- What is the effect of the factors in the codes on the design of a shallow foundation?
- Which codes result in more economical design under specific conditions?
- What are the differences between a design made by using the codes or using a numerical analysis?
1.3. Research scope
Since not everything can be investigated during this graduation project some limitations are set for this research study. The assumptions which are considered in this study are discussed below.

- The considered design codes in this study are the ISO 19901-4 and the API RP 2A-WSD since these are used within Allseas for the foundation design of subsea structures.
- In this study soil at the seabed is assumed to be a single homogeneous layer, similar to the assumptions included in the design codes used within Allseas so that it would be possible to determine whether there is conservatism embedded in the design codes.
- Very soft to soft clay soils conditions are considered in this study since foundation sizes in such soil conditions increase significantly.
- The static loads acting on the foundation are considered during the analysis performed to find the possible conservatism in the design codes. The dynamic and cyclic loads acting on the structure are not taken into account in the analysis. However their possible effect is replaced by an equivalent static load as suggested by design codes.
- The shapes of shallow foundations which are mentioned in the design codes are either rectangular, circular or an infinite strip foundation. Only the rectangular shaped shallow foundations are considered in this study due to the fact that the majority of subsea structures related to pipelines have a rectangular shaped structures. Structures with multiple footings as shown in figure 1.3 are not considered in this study since no guidelines for such foundations are provided in the investigated design codes.
- Only undrained analysis are performed, as clay soil conditions are taken into account in this study.
- The sensitivity of the soil parameters is determined by using a deterministic approach as in the design codes that are used within Allseas.
- Only the foundation design is considered in this investigation, the structural and installation aspect of the structure design are not taken into account. The complete PLET structure of case study 1 is shown in figure 1.4a and the considered PLET structure in this investigation is shown in figure 1.4b.
1.4. Thesis layout

The methodology followed during the graduation project will be discussed in this section. This thesis includes seven chapters whereas each chapter describes the methodology followed and the findings obtained during each step to accomplish the main objective of this study, namely the investigation of the possible embedded conservatism in the design codes, and how to deal with this conservatism.

Chapter 1 is the introduction of the research study. In this introduction the problem is defined, the research goals are listed and the research scope is discussed.

Chapter 2 provides a literature study regarding the recent developments of the subjects related to this research study. The subjects investigated in the literature study are the background of design codes, available safety approaches, the required site investigation, the bearing capacity theory for shallow foundations, the use of skirts in shallow foundations and different loading conditions that can act on an offshore foundation.

Chapter 3 presents the analytical analysis of the design codes.

Chapter 4 includes case studies considered in this research study. These case studies are projects from Allseas and the foundation design of these projects are used to investigate the possible embedded conservatism in the design codes. For these case studies the site investigation is reviewed and discussed and then values for soil parameters are determined based on the available data.

Chapter 5 provides the comparison between the analytical design codes used within Allseas and a numerical FEM program. The used numerical FEM program in this investigation is PLAXIS. For this comparison the case study discussed in chapter 4 are used.

Chapter 6 presents an example calculation regarding the economical aspects from the results of this study. Case study 1 is used to as an example to determine the possible cost reduction between a foundation design made according to the analytical design codes or the numerical program PLAXIS.

Chapter 7 presents the conclusions drawn from the research study. The results from the analytical analysis and the numerical analysis of the design codes are discussed and compared from the point of view of embedded conservatism in the design codes.

Finally chapter 8 gives recommendations on how to deal with the possible embedded conservatism in the design codes. This chapter gives practical recommendations on how to improve the foundation design with emphasis on how to deal with the possible conservatism in the design codes. Other recommendations for further research regarding embedded conservatism in design codes are also given in this chapter.
In this literature review the background of the investigated design codes and their safety approaches are investigated. Shallow and skirted foundations, the recent developments in research regarding these foundations, the offshore site investigation and offshore loading conditions are also investigated in this chapter.

2.1. Design codes and standards
The aim of standardization can be to make products, processes or services fit for its purpose by providing requirements, specifications or guidelines. Design codes and standards are developed to reduce the technical barriers for international trade and to increase economic growth [50] and [51]. The international developed design codes can be used by governments in order to save time, money and required expertise instead of developing their own design codes and standards. The design codes and standards can be used in a mandatory or voluntary way.

In this study two design codes are investigated to determine the possible conservatism embedded in them. The ISO 19901-4 [49] and the API RP 2A-WSD [12] are the design codes considered since these are used within Allseas for the foundation design of subsea structures.

2.2. Safety approaches
The design of structures must contain a safety factor to account for the uncertainty of variation in loads acting on a structure and resistance from the structure to fail. In general there are three different safety approaches [34], the global safety factor approach, the partial safety factor approach and the probabilistic safety approach.

The global safety factor approach, also called the working stress design approach (WSD), applies a single total safety factor $\gamma_G$ to the resistance against failure. The safety factor contains uncertainties from the loading and resistance. In geotechnical problems the resistance against failure is determined by the soil strength. The global safety approach is considered in the investigated design code API RP 2A-WSD. In equation 2.1 the global safety factor approach is shown. In this equation $R$ is the resistance against failure, $G$ is the permanent loading, $Q$ is the variable loading and $E$ is the environmental loading.

$$\frac{R}{\gamma_G} = G + Q + E$$

(2.1)

The partial safety factor approach, also called the load and resistance factor design (LRFD), applies separate safety factors on the loading and resistance sides. The resistance side in geotechnical problems is defined as the soil strength, this is usually reduced by a material safety factor $\gamma_m$. The loads are increased in this safety approach by a partial safety factor $\gamma_G$ for permanent loads, $\gamma_q$ for variable loads and $\gamma_e$ for environmental loads. The partial safety factor approach is considered in the investigated design code ISO 19901-4. In equation 2.2 the partial safety factor approach is shown. In this equation $R$ is the resistance against failure, $G$ is the
permanent loading, \( Q \) is the variable loading and \( E \) is the environmental loading.

\[
\frac{R}{\gamma_m} = \gamma_g G + \gamma_q Q + \gamma_e E
\] (2.2)

In the probabilistic safety approach the uncertainties of all loading and resistance parameters are simulated in multiple simulations. Each of these simulations gives a different response and corresponding global factor of safety. The results of all these simulations are used to construct a reliability curve from which a global factor of safety can be determined for a required corresponding probability of failure of the structure.

2.3. Offshore site investigation

Information regarding the values of the engineering soil parameters are required for the design of a structure. These engineering soil parameter values are determined from the site investigation results. The owner of the offshore structure usually is responsible for arranging the site investigation [23]. A typical offshore site investigation contains four stages: a desk study, a met-ocean investigation, a geophysical investigation and a geotechnical investigation [34]. During the desk study the available data regarding the soil and seabed in the project area is collected and analyzed. This information is used for the preliminary foundation design and the design of the framework for the required site investigation to complete the missing soil and seabed information. During the met-ocean investigation environmental data related to wind, waves, currents, tides, temperature, seasonal weather and ice is collected in the project area. The geophysical investigation is used to collect information regarding the water depth, seabed bathymetry and the existence of obstacles. In the geotechnical investigation soil samples are taken and soil tests are performed in order to obtain the data to determine the engineering soil parameter values required for the foundation design.

Performed soil tests during site investigation

During the geotechnical investigation the engineering soil parameters can be determined in two methods. In situ soil tests are performed at the location of an offshore structure or the soil is sampled and soil tests are performed on soil samples in the survey vessel or at an onshore laboratory.

The in situ soil tests used in the soil investigation are the cone penetration test, the T-bar, ball penetrometer and the vane shear test. The cone penetration test is used to characterize different soil layers and to determine the strength of the soil. The T-bar and ball penetrometer are used to measure the remoulded shear strength whereas the vane shear test is used to measure the soil shear strength. The soil sampling techniques used for pipeline development projects are usually the box core, the vibro core and the piston core. With the box core only the upper 0.5 meter of the soil, which is critical for pipelines, is sampled and with the vibro core and piston core techniques deeper soils are sampled for the inline structures. These soil samples then are used for laboratory tests where usually the shear strength, the unit weight, the Atterberg limits, the moisture content and the particle size distribution are determined. The soil strength and deformation parameters for static and cyclic loading are determined by using the oedometer test, DSS test (direct simple shear) and the triaxial test.

Missing soil parameter information

From actual offshore site investigations, it is seen that not always all the required soil data that are used to determine the required engineering soil parameters is collected during the site investigation. The missing soil parameter values can be determined by using correlations between soil parameters [69]. The engineering soil parameters usually are required to a depth of two times the width of rectangular subsea structures in order to design the foundation. This depth is usually not reached during an offshore site investigations and the gaps in the information about the engineering properties of the soil should be analysed and filled by the engineering judgement of a geotechnical engineer [69].

Site investigation for pipeline development projects

Pipeline development projects are used as case studies in this study, and the site investigation reports of these case studies are reviewed. The soil sampling and soil test locations from the site investigation which are close to the location of an offshore structure are preferred. However in pipeline development projects, the distance between the location of soil samples and soil in situ tests and the location of a structure is usually much larger compared to regular onshore site investigations. These larger distances are the result of the installation method, which has a large influence on the location of inline subsea structures, and the future developments
of the pipeline infrastructure, so the seabed bathymetry and soil properties of a larger area are investigated.

**Developments in site investigation**

New developments in survey technologies are currently aiming at increasing water depths and improving data recovery from soil sampling [70]. The new developments are aiming at increasing water depths since offshore developments are moving to larger water depths. The developments in improving the data recovery from soil tests and sampling techniques is to increase the quality of the determined engineering soil parameters for the foundation design. Next to these new developments the costs of the site investigation also is an important field of future developments since the mobilizing and operating costs of site investigation survey vessels is currently very high [34]. Operators already reduce the site investigation costs by splitting the mobilization costs for the survey vessels by planning site investigations close to each others new developments in one vessel tour. Another method of reducing the site investigation costs is investigated in the study of [75]. In this study the use of finite element methods instead of expensive physical soil testing to determined engineering soil parameter values is investigated.

If due to new developments the soil data recovery is improved and site investigations at larger water depths is possible, the determination of engineering soil parameter values can be further optimized. Also more soil data can be recovered if future site investigations are extended due to decreased site investigation costs. If, due to new developments in survey technologies, more and better soil data can be recovered for the same costs at possible larger water depths the uncertainty in engineering values for soil parameters could be decreased. Due to this possible decrease in uncertainty of engineering parameter values the safety factors in the foundation design could also be decreased.

### 2.4. Shallow foundations

Terzaghi [85] defined shallow foundations as "a foundation is considered a shallow foundation if the embedded depth of the foundation is less or equal to the least dimension of that foundation". The word "shallow" does not say anything about the embedding depth but about the embedment depth in relation to the foundation dimensions. Most of the foundation types, except pile foundations, are considered as shallow foundations.

In this section of the literature review the failure modes of shallow foundations, the classical bearing capacity theory and a more advanced failure envelope bearing capacity method are discussed. How skirts can improve the behaviour of shallow foundations and the recent developments in investigations regarding shallow skirted foundations are discussed in the following sections.

#### 2.4.1. Failure modes

Shallow foundation bearing capacity failure occurs usually if the soil supporting the shallow foundation fails in shear. There are three possible soil shear failure modes; general shear failure, local shear failure and punching shear failure [87] as shown in figure 2.1. The general shear failure is characterised by a good defined failure pattern consisting of a wedge and slip surface from the foundation edge to the surface. The local shear failure is characterised by a failure pattern, which is only clearly shown underneath the foundation. The failure pattern also consists of a wedge and slip surface, however the slip surface does not reach the surface. The punching shear failure is characterised by compression of the soil underneath the foundation and the surrounding soil is relative unaffected. Which of these three soil shear failure modes occurs depends on a combination of the soil properties, foundation geometry and loading conditions.

#### 2.4.2. Classical bearing capacity theory

The classical bearing capacity equation, as shown in equation 2.3, is proposed by Terzaghi [85] for shallow, infinitely long strip foundations under perfect vertical centric loading on a homogeneous single soil layer with an effective unit weight \( \gamma' \), friction angle \( \varphi \) and cohesion \( c \). The soil surface and the foundation base are also assumed to be perfectly horizontal. The surcharge \( q \) is defined as a loading equal to the weight of the soil above the foundation base. The equation consists of three terms, the first term dependens on the soil cohesion, the second on the surcharge and the third on the soil unit weight.

\[
q_{ult} = cN_c + qN_q + \frac{1}{2} \gamma'BN_T
\]  

\( (2.3) \)
2.4. Shallow foundations

(a) General shear failure  (b) Local shear failure  (c) Punch through failure

Figure 2.1: Bearing capacity failure modes after [87]

The classical bearing capacity equation as shown in equation 2.3 is later improved by Brinch Hansen, Meyerhof, Vesic and others into equation 2.4. In this equation the correction factors are included for the foundation shape, embedment, foundation base inclination, ground surface level inclination and loading inclination. An effective foundation is used to take the eccentricity of the loading into account.

\[ q_{ult} = cN_c s_c i_c b_c g_c + qN_q s_q i_q b_q g_q + \frac{1}{2} \gamma' B N_d s_d i_d b_d g_d \]  

(2.4)

The improved classical bearing capacity equation 2.4 takes inclined and eccentric loading conditions into account. The equation gives relative accurate results for mainly vertical loads acting on the shallow foundation, however for inclined and eccentric loadings the approach becomes unreliable due to the assumption of linear elasticity. For inclined and eccentric loading conditions the horizontal and moment loading becomes larger and they have an increasing influence on the bearing capacity of the shallow foundation. Due to linear elasticity the effect of the vertical, horizontal and moment loading components can be considered independently and the combined effect can be obtained by superposition.

2.4.3. Failure envelope for bearing capacity

The improved classical bearing capacity equation 2.4 is popular due to the simplicity and assumption of linear elasticity. From laboratory tests it is shown that the combined effect of vertical, horizontal and moment loading conditions can not be assessed independently because the effect of the different loading conditions are dependent on each other. By using a failure envelope and interaction equations where the different loading conditions are related to each other, the effects from combined loading conditions are assessed more accurate and reliable [83]. A failure envelope defines the loading conditions under which failure of the shallow foundation occurs. Any loading combination inside the failure envelope is considered safe. An example of a failure envelope is shown in figure 2.2. In this figure V, H and M the vertical, horizontal and moment loading and \( V_u \), \( H_u \) and \( M_u \) are the ultimate vertical, horizontal and moment loading for which the shallow foundation does not fail.

Figure 2.2: Failure envelope after [83]
2.4.4. Skirted shallow foundations

Skirted shallow foundations consist of a shallow foundation with thin skirts which penetrate in the soil. There is an increasing interest in using skirted shallow foundations as the skirted foundations have an improved behaviour compared to conventional shallow foundations and a short installation time compared to pile foundations [48]. An embedded pier foundation, as shown in figure 2.3, is investigated by physical model tests in the study of [24] to investigate the increase in bearing capacity and decrease in settlements of skirted foundations compared to a conventional shallow foundation. The results from the study indicate that the bearing capacity and settlements of the skirted and pier foundation are close but not the same.

By applying skirts to shallow foundations the slip plane is forced down from the foundation edges to the skirt tips. Due to the increased slip plane length the bearing capacity is increased for skirted foundations [4], [16]. Bearing capacity is also increased as skirts are capable of transferring loads from the surface to deeper layers [4], [16], [33], [48]. If there is an increasing shear strength profile in the soil, skirts transfer loads to deeper stronger soil, which increases the bearing capacity of the foundation as shown in figure 2.4. In this figure is $s_{u,m}$ the shear strength at the surface, $s_{u,0}$ is the shear strength at the skirt tip level and a nondimensional ratio $kD/s_{u,m}$ presents the increase in shear strength.

The resistance against horizontal loading is increased by applying skirts in the foundation design compared to conventional shallow foundations [16], [32],[56]. The skirts increase the area, which pushes against the soil and skirts transfer horizontal forces to deeper layers with an increased shear strength. In figure 2.5 failure mechanisms are shown for skirted foundations under horizontal loading. In figure 2.5a the situation with an embedded shallow foundation without skirts is shown, figure 2.5b shows the passive wedge failure, figure 2.5c shows the tip-to-tip failure for an increasing shear strength profile and figure 2.5d shows the tip-to-tip failure for a constant shear strength profile. The slip plane can also go through a weak layer. Figure 2.6a presents...
the failure mechanism, where the failure plane goes through a weak layer below the skirted foundation and figure 2.6b presents the failure mechanism, where the skirts have penetrated the weak layer where the failure plane goes through.

![Figure 2.5: Effect of skirts on failure mechanism of a mudmat under pure sliding after [32]](image)

Figure 2.5: Effect of skirts on failure mechanism of a mudmat under pure sliding after [32]

The applied skirts provide better protection against scour due to a waterflow compared to shallow foundations [5] and they can also increase the resistance against moment loading and uplift forces due to the generation of suction in between the skirts [4], [16], [33].

The soil within the skirts is required to behave as a rigid body. This is achieved by constructing sufficient internal skirts below the foundation. Sufficient internal skirts need to be applied in order to prevent failure mechanisms involving deformations in the soil plug within the skirts to occur. These failure mechanisms lead to a reduction in the bearing capacity of the skirted foundation [4], [59]. In the study of [59] an investigation is performed to determine the required number of internal skirts in order to prevent failure mechanisms in the soil within the skirts. The results of the study are shown in a design graph shown in figure 2.7. In this figure the required number of internal skirts are shown on the y-axis for the embedment depth ratio $D/B$ at the x-axis. The embedment depth ratio is the embedment depth over the width of the structure. The degree of strength heterogeneity is shown in the figure by two lines, $kB/s_{u,m} = 0$ represents a constant shear strength profile and $kB/s_{u,m} = \infty$ represents an infinite strength increase. Strength increase values in between should be interpolated between the two lines.

![Figure 2.7: Required number of internal skirts after [59]](image)

Figure 2.7: Required number of internal skirts after [59]

### 2.4.5. Numerical analysis of bearing capacity

Errors in finite element analysis calculations fall in three broad categories: user error, modeling error and discretization error [71] and [77]. In the user error the software is used incorrectly. In the modeling error assumptions or simplifications are incorrect. The discretization error is due to an insufficient mesh discretization. The mesh discretization converts a continuous field into a finite number of elements. The discontinuity
2. Literature review

of the calculated result between one element to another is the discretization error. Due to this discretization error the calculated ultimate capacity of the soil is overpredicted in this investigation. The discretization error can be reduced by using a finer mesh, however this causes longer runtime for calculations. If the average element size would be made infinite small then the discretization error would also be infinitely small and vanishes. This is however not possible due to the limited computer power for the calculation. Other methods for increasing the accuracy and reducing the available error in finite element method programs are investigated. The combination of the finite element method and the material point method is studied in [58] and the improvement of the accuracy from a calculation by updating the shape and/or size of elements during the calculation is investigated in the study of [48]. The results of these studies show an increase of the accuracy and a decrease of the error.

2.4.6. Recent developments

Some of the recent developments regarding shallow foundations are discussed in this section of the literature review.

The assumption of linear elasticity for the classical bearing capacity equations 2.3 and 2.4 has limitations as the combination of vertical, horizontal and moment loading can not be assessed independently but the effect from combined loading is obtained by superposition. The effect of combined vertical, horizontal and moment loading in undrained conditions is investigated in different studies in combination with variations in foundation geometry and soil conditions. The results of these studies to the effect of the combined loading are shown in failure envelopes. The combined V-H-M loading in combination with a variation in increasing shear strength is investigated in the studies of [16], [42], [74], [78]. An increasing shear strength profile is used in this investigation. The rate of shear strength increase is determined in the site investigation review as shown in section 4. The combined V-H-M loading in combination with a variation in increasing shear strength and a variation in embedment ratio is investigated in the studies [33] and [88]. The critical skirt spacing under combined V-H-M loading in combination with a variation in increasing shear strength and embedment ratio is investigated in the study of [59]. The skirt spacing under combined V-H-M loading and a variation in skirt penetration is investigated in [14]. The effect of combined V-H-M loading and a variation in only embedment depth is investigated in the study of [15], and the effect of combined H-M loading and a variation in embedment depth is studied in [90]. In the study of [37] is the influence of combined V-H-M loading on 2 coupled shallow foundations investigated and in the study of [41] is for combined V-H-M loading the difference between the classical bearing capacity equations and the finite element analysis investigated.

The effect from variation of the embedment depth, the increase in shear strength and the aspect ratio of a shallow foundation are also investigated for situations where no combined vertical, horizontal and moment loading conditions are applied. In [21] is the effect of an increasing shear strength on the bearing capacity of shallow foundations investigated and in [84] is the influence of increasing shear strength on the bearing capacity of skirted foundation investigated. The variation in aspect ratio of the foundation in combination with a constant shear strength in depth is investigated in [76].

All these studies commonly investigate the combinations between V-H-M loading, increasing shear strength ratio, embedment ratio and aspect ratio’s. According to their findings the bearing capacity of a shallow foundation should increase for an increasing shear strength ratio, increasing embedment ratio considered to assess the effects on the bearing capacity of a shallow foundation.

In the study of [41] failure envelopes are derived from the classical bearing capacity equations and with finite element analyses. The study results show that the classical bearing capacity equations from the ISO design code [49] show a symmetrical behavior around the moment axis shown in figure 2.8. The classical bearing capacity equations are symmetrical because they do not take the difference between H:M and -H:M loading into account as shown in figure 2.9 while the results from finite element analysis show a non-symmetrical behaviour between H:M and -H:M loading. In this study both the H:M and -H:M situations are considered as load cases for the structures. The H:M loading correspond to the situation of Mode 1 in figure 2.9a and -H:M loading correspond to the situation of Mode 2 in figure 2.9b. From figure 2.8 is also seen that the results from the classical bearing capacity equations show significant less curvature compared to the results from the finite element analysis in both the H:M and -H:M area’s. This indicated that the use of the classical bearing capacity equations result in a significant under-prediction of the bearing capacity. The result in the study of
also indicated that the classical bearing capacity equations results in an underprediction of the bearing capacity under inclined loading and a new load inclination correction factor for the classical bearing capacity equation is proposed.

![Figure 2.8: Ultimate limit states under V-H-M loading: FEA vs ISO predictions after [41]](image)

![Figure 2.9: Non symmetric combined V-H-M loading after [86]](image)

Conventional methods of analyzing the bearing capacity of shallow foundations against horizontal loading usually are focussed on the effect of moment and horizontal loads on the vertical bearing capacity of the foundation. The response of the foundation under torsion and combined sliding and torsion is not investigated in such detail [67]. The resistance against sliding and torsion for shallow foundations is however investigated in the following studies. In [81] is the effect of horizontal loading on the sliding failure of skirted foundations investigated. The effects from horizontal and torsional loading on shallow foundations is investigated in [29] and [65]. From these studies is shown that the horizontal resistance against sliding of shallow and skirted foundations is reduced if the foundation is also loaded under torsion. The results from [89], a study about the influence of a combination of torsion and V-H-M loading proposes a modification of the failure envelope for V-H-M loading to account for the resultant reduction in the sliding capacity as a proportion of the ratio of applied torsional loading and ultimate torsional capacity of the shallow foundation. The effect of horizontal cyclic loading is investigated in [19].

The friction between the foundation base and the soil and the friction between the skirts and the soil has an influence on the bearing capacity of shallow foundations. The effect of the friction at the soil-foundation base interface and the foundation aspect ratio is investigated in [40]. The effect of skirt-soil interaction in combination with a variation in foundation embedment and foundation size is investigated in [36] and the effect of skirt-soil interaction in combination with a variation in increasing shear strength in the soil is investigated in [48].

Skirted foundation can generate suction below the foundation base. In numerical studies a skirted foundation can be simulated by applying a suction below a shallow foundation by using tension interface elements. In [86] is a skirted strip foundation under combined loading simulated by applying a simulated suction underneath the foundation. In [38] is also a skirted strip foundation simulated by applying a suction underneath a shallow foundation in combination with a variation of the embedment and in [39] is suction applied underneath a shallow foundation in combination with a variation in the aspect ratio of the foundation. Because of the generated suction underneath a skirted foundation there is a resistance against moment and uplift loading. The resistance against moment and uplift loading is investigated in [2], [3], [4], [60] and [83]. In this study the suction below a shallow skirted foundation is not taken into account because the mudmats of the inves-
tigated subsea structures are not capable of generating a suction underneath the structure. In the mudmat holes are constructed for installation purposes which prevent suction from occurring under moment loading.

The skirted foundations investigated in the discussed studies usually have a skirt penetration ratio in the range of $2 \leq D/B \leq 0.5$. The skirted foundations used for the investigated subsea PLET structures in this study have a much smaller skirt penetration, in the order of $0.05 \leq D/B \leq 0.2$. The aspect ratio for the investigated PLET structures in this study are in the range of $0.5 \leq B/L \leq 1$. An example of the skirted foundation of a PLET structure with short skirts is shown in figure 2.10. Subsea structures with a skirt penetration ratio and aspect ratio of the same order of magnitudes are investigated in the studies of [25], [27], [28], [32] and [59]. The loading conditions of the investigated shallow and skirted foundations in the literature are usually a combined V-H-M loading and a few investigations is torsion also taken into account. The loading conditions of subsea PLET structures is usually best defined by a combined $V − H^2 − M^2 − T$ loading condition instead of a $V$-H-M loading. In the $V − H^2 − M^2 − T$ loading condition six degrees of freedom are used in the bearing capacity calculation. This combined $V − H^2 − M^2 − T$ loading condition is investigated in [25], [26], [27] and [28].

![Figure 2.10: Skirted foundation after [28]](image)

2.5. Offshore loading conditions

The loading conditions for structures in onshore and offshore conditions are not comparable. For offshore structures the horizontal loads are much higher compared to onshore structures, which result in higher horizontal loads and moments on the foundation [66], [34]. It is also important to consider cyclic loading conditions for the design of offshore structures since the cyclic loads usually are one of the main design considerations for offshore structures [8].

2.5.1. General loading

Seven general load categories can be identified [23], based on a general overview of the different loads acting on onshore and offshore structures. These seven load categories are:

- **Dead loads** consist of loads which are constant over the lifetime of the structure such as own weight of the structure and permanent equipment or hydrostatic loading such as buoyancy.
- **Live loads** are considered as loads of varying magnitude during the lifetime of the structure such as temporary equipment, people or loads from operations.
- **Environmental loads** consist of wind, waves, currents, ice or temperature induced loads.
- **Motion and deformation loads** are considered as the secondary effects from loading due to the displacement of (a part of) the structure.
- **Accidental loads** are loads which can occur but it is not possible to determine when these loads will occur. Accidental loads consist of dropped objects, explosions, collisions or overloading.
- **Construction, transport and installation loads** are loads where forces are initiated at different locations in the structure compared to the force initiation of the structure at its final location.
- **Fatigue loads** are due to repeating loads during the lifetime of the structure.
2.5.2. Cyclic loading

Offshore structures are subjected to cyclic loads from wind, waves, currents and possibly ice and earthquakes [6]. These cyclic loads are important design considerations for offshore and onshore structures [8]. In general the difference between offshore and onshore cyclic loading is the cyclic load periods, offshore structures are subjected to loading periods of approximately 10 to 20 seconds and onshore structures are subjected to loading periods of approximately 1 second [6], [8]. The cyclic shear strength of the soil and the failure mode under cyclic loading are dependent on the stress path, the overconsolidation ratio, the combination of average and cyclic shear stresses and the number of load cycles [8], [9], [11].

For offshore structures where the effects of cyclic loading are significant some important foundation design aspects are investigated in studies of [6], [9], [11]. In these studies it is found that there is a significant difference between the static and cyclic bearing capacity of the soil. The settlement rate usually decreases in time under static loading. The settlement rate continues with approximately the same rate for a long time under cyclic loading. In figure 2.11 the difference between cyclic and static bearing capacity and displacements are shown.

The failure line below a cyclic loaded structure is shown in figure 2.12. On this failure line there are four locations where the cyclic strength and deformation properties of the soil can be investigated since the stress path is similar to a triaxial test and direct simple shear test [6], [9], [11]. In the figure \( \tau \) is the shear stress, \( \tau_0 \) the initial shear stress for a consolidated soil, \( \tau_{cy} \) the cyclic shear stress caused by cyclic loading, \( \tau_a \) is the average shear stress and \( \Delta\tau_a \) the additional shear stress due to the submerged weight of the structure. The average shear stress \( \tau_a \) is composed of the initial shear stress \( \tau_0 \) and the additional shear stress \( \Delta\tau_a \).

In order to determine the soil resistance to cyclic loading laboratory tests are performed. The actual cyclic loads acting on the structures vary from one cycle to another however due to practical reasons during a laboratory test the cyclic loads are kept constant. The irregular cyclic loading can be represented by constant cyclic loading during an equivalent number of load cycles \( N_{eq} \) since the magnitude and number of cyclic load cycles are related to each other [6], [8], [9], [10].

With laboratory tests the soil resistance against cyclic loading can be determined. There are three accumulation procedures which can be used as a memory of the effect of cyclic loading depending on the soil type and stress path:
- Cyclic shear strain accumulation procedure
- Pore pressure accumulation procedure
- Permanent shear strain procedure

Contour diagrams are the basis of practical foundation design of offshore structures. An example of contour diagrams are shown in figure 2.13 where the blue line are the failure modes for cyclic loading and the red dotted lines are the failure modes for static loading. These contour diagrams are based on a number of...
stress-controlled tests with different combinations of constant $\tau_a$ and $\tau_{cy}$ and the number of cycles before soil failure. In the tests failure is defined when $\gamma_a$ or $\gamma_{cy}$ reaches a certain value. The contour diagram can be constructed by using a direct simple shear soil test as shown in figure 2.13a or a triaxial soil test as shown in figure 2.13b.

The results from the laboratory tests are used to create a database with cyclic loaded soil data. This database can be used to gain a better understanding of the soil behaviour under combined static and cyclic loading. Furthermore the database can provide information for the determination of soil strength parameters prior to the site investigation, for determining the specifications of the site investigation and the laboratory testing, and for the interpretation of the cyclic loading test results.

The method of determining the soil strength decrease due to cyclic loading by using laboratory test is limited to the specific soil and loading conditions, which are tested. For different soils or loading conditions new laboratory tests have to be performed. Another method of determining the soil strength decrease is by using a theoretical model. In the study from [80] a densification model is created. This model is based on the assumption that there is no plastic volume change for cyclic loading under undrained conditions. Due to loading, the pore pressure increases and the effective stress decreases. Because of the decreasing effective stress the soil strength also decreases. Both the laboratory tests and a theoretical model can be used in order to determine the soil strength reduction due to cyclic loading.
The analytical analysis based on design codes is performed in this section. The considered design codes in this study are the API RP 2A-WSD [12] and the ISO 19901-4 [49], since these design codes are used within Allseas for the foundation design. The analytical solutions for the bearing capacity from three authors also are included in this analytical analysis because of their contribution to the research regarding the bearing capacity of shallow foundations. The three authors considered in this study are Brinch Hansen [45] [46], Meyerhof [62] [63] [64] and Vesic [87].

In the API design code a global safety factor approach is used and this global safety factor is applied after the ultimate bearing capacity of the soil is determined. The three authors do not use safety factors in the calculation of the ultimate bearing capacity of the soil since they have performed research to the ultimate bearing capacity rather than whether a foundation design is considered safe or not. In the ISO design code a partial safety factor approach is used and the partial safety factors are applied in the calculation of the bearing capacity of the soil. Both the partial material factor and partial load factor are set at $\gamma_m = 1.0$ and $\gamma_l = 1.0$ in this study in order to make a fair comparison with the API design code and the research results from the three considered authors.

Drained and undrained conditions in the design codes
In the API and ISO design codes equations are provided for either perfect drained or undrained conditions. Both the API and ISO design codes only consider pure clay in undrained conditions. Only the cohesion term of the shallow foundation bearing capacity equation 2.3 as shown in section 2 is used. The surcharge term is considered in undrained conditions, however no bearing capacity factors and correction factors are applied. The soil self weight term of the equation is neglected for undrained conditions in both the API and ISO design codes. The API and ISO design codes do not have a similar approach regarding drained conditions. In the ISO design code the cohesion term of the shallow foundation bearing capacity equation is used without correction factors, while these correction factors are applied on the soil self weight and the surcharge terms. The API design code uses the cohesion, surcharge and soil self weight terms with bearing capacity factors and correction factors for the bearing capacity of a shallow foundation in drained conditions. The equations as used by the three investigated authors also consider either perfect drained or undrained conditions, however for both conditions equation 2.4 is used. The design codes and the three authors consider drained conditions if $c = 0$ and undrained conditions as a special case of the drained conditions where $\phi = 0^\circ$.

3.1. Bearing capacity
3.1.1. Undrained conditions
In the API design code equation 3.1 is provided to calculate the ultimate bearing capacity of a shallow foundation in undrained conditions. In this equation a constant shear strength profile in depth is required. For the ISO design code two equations are provided to calculate the ultimate bearing capacity of the soil. Equation 3.2 uses a constant shear strength profile in depth and equation 3.3 uses an increasing shear strength profile. Only the cohesion term of the bearing capacity equations 2.3 and 2.4 is used in combination with the bearing capacity factors and correction factors by the provided equations in the API and ISO design code for
undrained conditions. The surcharge term is taken into account by the \( \gamma \cdot D \) in equation 3.1 for the API design code and by \( \gamma' \cdot D \) in equations 3.2 and 3.3 in the ISO design code. The three investigated authors use equation 2.4 with their proposed correction factors for undrained conditions. Only the cohesion and surcharge term of the equation are used due to the assumption of \( \varphi = 0 \) for undrained conditions.

\[
Q_{ult} = \left( s_u \cdot N_c \cdot K_c + \gamma \cdot D \right) \cdot A' \quad \text{- API with constant shear strength} \quad (3.1)
\]

\[
Q_{ult} = \left( N_c \cdot \left( \frac{s_u}{\gamma_m} \right) \cdot K_c + \gamma' \cdot D \right) \cdot A' \quad \text{- ISO with constant shear strength} \quad (3.2)
\]

\[
Q_{ult} = \left[ F \cdot \left( N_c \cdot s_u + \frac{k \cdot B'}{4} \right) \cdot \frac{K_c}{\gamma_m} + \gamma' \cdot D \right] \cdot A' \quad \text{- ISO with increasing shear strength} \quad (3.3)
\]

**Bearing capacity factor**

Table 3.1 shows the considered bearing capacity factors in the design codes and by the three authors for undrained conditions where \( c = s_u \) and \( \varphi = 0 \) are assumed. The factor \( N_c \) is defined as \( N_c = \pi + 2 = 5.14 \) by the three authors and in the ISO and API design code. No bearing capacity factors \( N_q \) and \( N_\gamma \) are provided in the design codes since these factors are not used in the provided equation to calculate the bearing capacity of the soil. The three authors present equations to calculate the bearing capacity factors \( N_q \) and \( N_\gamma \), but for \( \varphi = 0 \) the factor \( N_q \) becomes 1.0 and \( N_\gamma \) becomes zero.

<table>
<thead>
<tr>
<th></th>
<th>( N_c )</th>
<th>( N_q )</th>
<th>( N_\gamma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brinch Hansen</td>
<td>5.14</td>
<td>1.0</td>
<td>0</td>
</tr>
<tr>
<td>Meyerhof</td>
<td>5.14</td>
<td>1.0</td>
<td>0</td>
</tr>
<tr>
<td>Vesic</td>
<td>5.14</td>
<td>1.0</td>
<td>0</td>
</tr>
<tr>
<td>API RP 2A-WSD</td>
<td>5.14</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4</td>
<td>5.14</td>
<td>-</td>
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</tr>
</tbody>
</table>

Table 3.1: Summary bearing capacity factors for undrained conditions

**Correction factor**

Terzaghi derived bearing capacity equation 2.3 for a non embedded strip foundation on a homogeneous single soil layer with a horizontal surface under a centric and non inclined loading. Correction factors are determined later by Brinch Hansen, Vesic, Meyerhof and others in order to use the equation for more general situations. The three investigated authors multiplied their proposed correction factors as shown in equation 2.4.

Correction factors are only applied on the cohesion term of the bearing capacity equation for undrained conditions in the API design code. The correction factor \( K_c \) as used in the API design code is calculated by equation 3.4. The different correction factors for load inclination, foundation shape, embedment depth, surface slope and foundation base inclination are multiplied. This indicates that the correction factors have a value of 1.0 if there is no influence on the bearing capacity of the soil.

\[
K_c = i_c \cdot s_c \cdot d_c \cdot g_c \cdot b_c \quad (3.4)
\]

The ISO design code also only applies correction factors on the cohesion term of the bearing capacity equation for undrained conditions. The correction factor \( K_c \) as used in the ISO design code is calculated by equation 3.5. The correction factors for the foundation shape, embedment depth and load inclination are summed in this equation instead of multiplied. This indicates that the correction factors should have a value of zero if there is no influence on the bearing capacity from the correction factor. No correction factors for soil surface slope and foundation base inclination are used in the ISO design code.

\[
K_c = 1 + s_c + d_c - i_c \quad (3.5)
\]

**Correction for load eccentricity**

Terzaghi’s bearing capacity equation, as shown in equation 2.3 assumes a centric, vertical loading with no moments acting on the foundation. Moments acting on a foundation as shown in figure 3.1a can be converted to an eccentricity \( e \) and eccentric vertical load \( V \) acting on a shallow foundations as shown in figure 3.1b. The
bearing capacity of eccentric loaded foundations is reduced by assuming an equivalent centric load on a foundation area reduced by the eccentricity as shown for a rectangular shallow foundation in figure 3.2a, and for a circular shallow foundation in figure 3.2b. This method for correcting the bearing capacity of a shallow foundation under eccentric loading is used in the API and ISO design codes as well as by the three investigated authors.

Figure 3.1: Moment and eccentric loading after [12]

(a) Moment loading
(b) Eccentric loading

The length and width of an eccentric loaded rectangular foundation are reduced by equations 3.6 and 3.7 according to figure 3.2a in order to determine the reduced foundation area by equation 3.8 in the API design code. The area for an eccentric loaded rectangular foundation is in the ISO design code is only reduced in one direction and calculated by equation 3.9. The three authors Brinch Hansen, Meyerhof and Vesic reduce the foundation area in two directions for eccentric loading.

\[ L' = L - 2e_1 \]  
\[ B' = B - 2e_2 \]  
\[ A' = L' \cdot B' \quad \text{for API} \]  
\[ A' = B' \cdot L \quad \text{for ISO} \]

Corrections factors for load inclination

Table 3.2 presents the equations for the load inclination correction factors \( i_c \), \( i_q \) and \( i_\gamma \). From the table is seen that for correction factor \( i_c \) the API design code follows Vesic and the ISO design code follows Brinch Hansen. Correction factors \( i_q \) and \( i_\gamma \) are not provided in the API and ISO design codes since only \( i_c \) is applied in the bearing capacity equation of the design codes.

A square foundation is considered to analyze the effect of load inclination and different undrained shear strength values on the load inclination correction factor. Load inclination angle \( \theta \) is shown in figure 3.3a and a range of \( 0^\circ \leq \theta \leq 15^\circ \) is taken into account in this analysis since the load inclination of the four investigated case studies fall in this range.
3.1. Bearing capacity

Table 3.2 shows that the correction factor from Meyerhof is not dependent on the soil strength and foundation area, but only on the loading angle $\theta$. Vesic and the API design code also consider a factor $m$ in the correction factors. This factor $m$ is dependent on the foundation dimensions and load direction angle $\alpha$ as defined by equation 3.10. Loading direction $\alpha$ as shown in figure 3.3b is kept constant at $\alpha = 0^\circ$ in this analysis.

$$m = \frac{2 + \frac{L'}{B'}}{1 + \frac{L'}{B'}} \cos^2 \alpha + \frac{2 + \frac{B'}{L'}}{1 + \frac{B'}{L'}} \sin^2 \alpha$$

(3.10)

<table>
<thead>
<tr>
<th>$i_c$</th>
<th>$i_q$</th>
<th>$i_T$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brinch Hansen</td>
<td>0.5 - 0.5, $\sqrt{1 - \left(\frac{H}{B'L_s}\right)}$</td>
<td>$\left(1 - \frac{0.5}{90}\right)^2$</td>
</tr>
<tr>
<td>Meyerhof</td>
<td>$\left(1 - \frac{\theta^\circ}{90}\right)^2$</td>
<td>$\left(1 - \frac{\theta^\circ}{90}\right)^2$</td>
</tr>
<tr>
<td>Vesic</td>
<td>$\left(1 - \frac{mH}{B'L_sN_c}\right)$</td>
<td>$\left(1 - \frac{H}{V + B'L_s\cot \varphi}\right)^m$</td>
</tr>
<tr>
<td>API RP 2A-WSD</td>
<td>1 - $\frac{mH}{B'L_sN_c}$</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - constant $s_u$</td>
<td>0.5 - 0.5, $\sqrt{1 - \left(\frac{H}{B'L_s\gamma_m}\right)}$</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - increase $s_u$</td>
<td>0.5 - 0.5, $\sqrt{1 - \left(\frac{H}{B'L_s\gamma_m}\right)}$</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 3.2: Summary correction factor load inclination for undrained conditions

Figure 3.4 presents the results from the analysis of load inclination correction factor $i_c$. Meyerhof is the same in the plots for different shear strength values since Meyerhof is independent of shear strength. The load inclination correction factor from the API design code has a value of $i_c = 1.0$ for a loading angle of $\theta = 0^\circ$ which corresponds with equation 3.4 for $K_c$ where a value of 1.0 is required if a correction factor has no influence on the bearing capacity. For an increasing load inclination angle the influence on the bearing capacity increases and the API correction factor is decreasing. The influence on the bearing capacity becomes less for a stronger soil, which is seen in the plots by the decreasing rate of the correction factor for increasing shear strength. The ISO correction factor has a value of $i_c = 0$ for a loading angle of $\theta = 0^\circ$ which corresponds with equation 3.5 for $K_c$ that a value of zero is required if there is no influence on the bearing capacity. The ISO correction factor increases for an increasing load inclination angle. The correction factor rate of increase becomes less for an increasing shear strength. This corresponds to the theory that the correction factor value decreases for a stronger soil since the influence on the bearing capacity becomes smaller.
Correction factors for foundation shape

Since the bearing capacity equation is derived for a strip foundation, correction factors are applied for a foundation design consisting of a square, circular or rectangular shaped foundation. Table 3.3 presents the foundation shape correction factors investigated in this analysis. Correction factors $s_q$ and $s_f$ are not provided in the API and ISO design codes since these correction factors are not used in the bearing capacity equations for undrained conditions. The API design code follows Vesic, while the ISO design code does not follow any of the three investigated authors. The effective foundation area is used by Brinch Hansen, the API design code and the ISO design code as shown in table 3.3, while Meyerhof and Vesic use the real foundation area. However in this analysis no load eccentricity is assumed so there is no difference between the effective and total foundation dimensions.

Figure 3.5 presents the results from the analysis to the foundation shape correction factor $s_c$. An aspect ratio range of $0.5 \leq B/L \leq 1$ is considered since this range is of interest for the investigated foundation designs. In the plots is seen that if the aspect ratio is extrapolated for small aspect ratio values, which represent a strip foundation, the correction factor values of the API, Vesic and Meyerhof go to a value of $s_c = 1.0$ and correction factors values of Brinch Hansen and the ISO design code for a constant shear strength go to a value of $s_c = 0$. These extrapolated correction factor values correspond to equations 3.4 and 3.5 for $K_c$ which prescribes that the correction factor has no influence on the bearing capacity for a strip foundation. The increasing behavior
Table 3.3: Summary correction factor foundation shape for undrained conditions

<table>
<thead>
<tr>
<th></th>
<th>$s_c$</th>
<th>$s_q$</th>
<th>$s_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brinch Hansen</td>
<td>$0.2\frac{B}{L}$</td>
<td>$1 + \sin\phi\frac{B}{L}$</td>
<td>$1 - 0.4\frac{B}{L}$</td>
</tr>
<tr>
<td>Meyerhof</td>
<td>$1 + 0.2\tan^2(45^\circ + \frac{\phi}{2})\frac{B}{L}$</td>
<td>$1 + (\frac{B}{L})\tan\phi$</td>
<td>$1 - 0.4\frac{B}{L}$</td>
</tr>
<tr>
<td>Vesic</td>
<td>$1 + (\frac{B}{L})\frac{N_q}{N_c}$</td>
<td>$1 + (\frac{B}{L})\tan\phi$</td>
<td>$1 - 0.4\frac{B}{L}$</td>
</tr>
<tr>
<td>API RP 2A-WSD</td>
<td>$1 + (\frac{B}{L})\frac{N_q}{N_c}$</td>
<td>$1 + (\frac{B}{L})\tan\phi$</td>
<td>$1 - 0.4\frac{B}{L}$</td>
</tr>
<tr>
<td>ISO 19901-4 - constant $s_u$</td>
<td>$0.2(1 - 2i_c)\frac{B}{L}$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - increase $s_u$</td>
<td>$s_{cv}(1 - 2i_c)\frac{B}{L}$</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

for an increasing aspect ratio is similar for Brinch Hansen, Meyerhof, Vecis, the API design code and the ISO design code with a constant shear strength profile, except for the correction factor value due to the different equations 3.4 and 3.5 for $K_c$. The correction factor for the ISO design code with an increasing shear strength profile is decreasing for an increasing aspect ratio as shown in the figure. The rate of decrease becomes larger for an increasing shear strength increase which correspond to the theory that the influence of the correction factor on the bearing capacity is smaller for a stronger soil.

![Figure 3.5: Foundation shape correction factor $s_c$ for different aspect ratio's and shear strength profiles](image-url)
Correction factors for embedment depth

Foundation embedment correction factors are applied in the bearing capacity equations in order to correct for an embedment depth. Table 3.4 presents the correction factors for undrained conditions. The ISO design codes do not provide correction factors \( d_q \) and \( d_\gamma \) since these are not used in the provided bearing capacity equations. The API design code does provide correction factors \( d_q \) and \( d_\gamma \), however for undrained conditions where \( \varphi = 0 \) is assumed the values are 1.0 and correction factors \( d_q \) and \( d_\gamma \) are not used in the provided bearing capacity equation. The presented embedment correction factors in table 3.4 are only valid for an embedment of \( D/B' < 1 \) which is the range for the investigated foundation designs in this study. For deeper penetrating foundations other correction factors exist, however these are not taken into account in this analysis. A square foundation with centric loading is considered in this analysis so the real and effective foundation areas are equal.

<table>
<thead>
<tr>
<th></th>
<th>( d_c )</th>
<th>( d_q )</th>
<th>( d_\gamma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brinch Hansen</td>
<td>( 1 + 0.4 \frac{D}{B} )</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Meyerhof</td>
<td>( 1 + 0.2 \sqrt{\tan^2(45^\circ + \varphi/2) + \varphi/2 \frac{D}{B}} )</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Vesic</td>
<td>( 1 + 0.4 \frac{D}{B} )</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>API RP 2A-WSD</td>
<td>( d_q - \frac{1-d_q}{N_v \tan \varphi} ) ( 1 + 2 \tan \varphi (1 - \sin \varphi)^2 \frac{D}{B} )</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - constant su</td>
<td>( 0.3 \arctan \left( \frac{D}{B} \right) )</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - increase su</td>
<td>( 0.3 \left( \frac{c_{u,1}}{c_{u,2}} \right) \arctan \left( \frac{D}{B} \right) )</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 3.4: Summary correction factor embedment depths for undrained conditions

Figure 3.6 presents the results for the analysis of the embedment correction factor \( d_c \). In the figure an increasing correction factor \( d_c \) is seen from Brinch Hansen, Vesic, Meyerhof and the ISO for an increasing embedment depth. The API design code has a constant value of \( d_c = 1.0 \) for undrained conditions as shown in the figure. An explanation for this correction factor behavior could be that no shear strength increase in the soil is taken into account in the API design code and the cohesion term of the bearing capacity is therefore not influenced by the embedment of a shallow foundation. The influence from the embedment on the bearing capacity is taken into account in the surcharge term of the bearing capacity equation. In the ISO design code a decreasing trend is seen for an increasing shear strength profile in the soil. An explanation for this behavior is the assumption that the embedment correction factor has smaller influence on the bearing capacity for a stronger soil.

Figure 3.6: Embedment depth \( d_c \) for different embedment ratios and shear strength increase values
Correction factors for ground inclination

Ground inclination correction factors \( g_c, g_q \) and \( g_\gamma \) are dependent on the ground surface inclination angle \( \beta \) as shown in figure 3.7, where \( \beta \) is in degrees. Table 3.5 presents the results for the correction factors. Meyerhof and the ISO design code do not provide ground inclination correction factors. In the API only \( g_c \) is provided since only this correction factors is used in the provided bearing capacity equation. Correction factor \( g_c \) is the same for Brinch Hansen, Vesic and the API design code.

<table>
<thead>
<tr>
<th></th>
<th>( g_c )</th>
<th>( g_q )</th>
<th>( g_\gamma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brinch Hansen</td>
<td>( 1 - \frac{2\beta}{\pi + 2} )</td>
<td>( (1 - 0,5 \tan \beta)^5 )</td>
<td>( (1 - 0,5 \tan \beta)^5 )</td>
</tr>
<tr>
<td>Meyerhof</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Vesic</td>
<td>( 1 - \frac{2\beta}{\pi + 2} )</td>
<td>( (1 - \tan \beta)^2 )</td>
<td>( (1 - \tan \beta)^2 )</td>
</tr>
<tr>
<td>API RP 2A-WSD</td>
<td>( 1 - \frac{2\beta}{\pi + 2} )</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - constant ( s_u )</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - increase ( s_u )</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 3.5: Summary correction factor ground inclination for undrained conditions

Figure 3.7: Ground and foundation base inclination after [12]

Figure 3.8 presents the results for the ground inclination correction factor \( g_c \). The ground inclination factor range \( 0^\circ \leq \beta \leq 10^\circ \) is taken into account in this analysis since the investigated case studies are installed on a seabed with a seabed inclination in this range. For an increasing ground inclination the correction factor \( g_c \) decreases and for a ground inclination of \( \beta = 0^\circ \) the correction factor is \( g_c = 1.0 \) which corresponds with the theory and the equation for \( K_c \) that the correction factor should not have an influence on the bearing capacity if there is no ground surface inclination.

Figure 3.8: Ground surface inclination factor \( g_c \) for different angles \( \beta \)
Correction factors for foundation base inclination

Foundation base inclination correction factors are only dependent on the foundation base inclination \( \mu \) as shown in figure 3.7, where \( \nu \) is in degrees. Table 3.6 presents the correction factors \( b_c \), \( b_q \) and \( b_\gamma \) for the three authors and the design codes. The correction factor \( b_c \) for Brinch Hansen, Vesic and the API design code are the same.

<table>
<thead>
<tr>
<th></th>
<th>( b_c )</th>
<th>( b_q )</th>
<th>( b_\gamma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brinch Hansen</td>
<td>( 1 - \frac{2\mu}{\pi + 2} )</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Meyerhof</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Vesic</td>
<td>( 1 - \frac{2\mu}{\pi + 2} )</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>API RP 2A-WSD</td>
<td>( 1 - \frac{2\mu}{\pi + 2} )</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - constant su</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>ISO 19901-4 - increase su</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 3.6: Summary correction factor foundation base inclination for undrained conditions

Figure 3.9 presents the results from the analysis to the foundation base inclination correction factor \( b_c \). In this analysis a range for the foundation base of \( 0^\circ \leq \mu \leq 10^\circ \) is used since the typical foundation base inclination the investigated structures falls in this range. In the figure it is seen that for an inclining foundation base the correction factor decreases. For a foundation base inclination of \( \mu = 0^\circ \) the correction factor has a value of \( b_c = 1.0 \). This corresponds to the theory and equation 3.4 for \( K_c \) that for a foundation base inclination of \( \mu = 0^\circ \) the correction factor has no influence on the bearing capacity.

![Foundation base correction factor \( b_c \)](image)

3.1.2. Drained conditions

An analytical analysis of the design codes for the bearing capacity in drained conditions is not presented in this report since drained soil conditions are not in the scope of this study.

3.2. Sliding capacity

The sliding capacity of a shallow foundation consists of the following three terms:

1. Resistance between the foundation base and the soil.
2. Resistance between the skirts and the surrounding soil.
3. Resistance on the skirts due to active and passive soil wedge forces.

Both API and ISO design codes assume a non embedded foundation for the provided sliding capacity equation where only the first term is used. The design codes mention that the second and third terms should be
3.2. Sliding capacity

taken into account during the design for an embedded foundation. However no equations are provided on how to calculate the sliding capacity for the second and third term. In the API design code also is prescribed that if a horizontal failure plane is not insured, other potential failure modes should be investigated with the mode giving the lowest lateral resistance selected as the design case. However in this analysis a horizontal failure plane is assumed as the occurring failure mode for sliding failure.

### 3.2.1. Undrained conditions

The influence from the three sliding capacity terms is investigated for case study 1 by a sensitivity analysis. Soil parameters from case study 1, as determined in the site investigation review in section 4, and a range of embedment depths from 0 to 1 meter are taken into account in this analysis. The results of the analysis are presented in figure 3.10, but first the used equations are discussed.

The API design code provides equation (3.11) to calculate the soil sliding capacity between the foundation base and the soil, which is the first term of the soil sliding capacity.

\[ H_{ult,1} = s_u \cdot B \cdot L : \text{for API} \]  

(3.11)

The ISO design code provided equation (3.12) to calculate the soil sliding capacity for a constant shear strength profile and equation (3.13) for the calculation of the soil sliding capacity for an increasing shear strength profile.

\[ H_{ult,1} = \left( \frac{S_u}{\gamma_m} \right) \cdot B \cdot L : \text{for ISO} \]  

(3.12)

\[ H_{ult,1} = \left( \frac{S_{u,0}}{\gamma_m} \right) \cdot B \cdot L : \text{for ISO} \]  

(3.13)

From soil mechanics theory it is found that it is appropriate to consider equations (3.14) and (3.15) for the calculation of the second and third term for the soil sliding capacity in the API design code. Equations (3.16) and (3.17) are appropriate to consider for the calculations of the second and third term for the soil sliding capacity in the ISO design code.

\[ H_{h,2} = 2 \cdot s_u \cdot L \cdot D : \text{for API} \]  

(3.14)

\[ H_{h,3} = B \cdot D \cdot 4 \cdot s_u : \text{for API} \]  

(3.15)

\[ H_{h,2} = 2 \cdot \left( \frac{S_{u,ave}}{\gamma_m} \right) \cdot L \cdot D : \text{for ISO} \]  

(3.16)

\[ H_{h,3} = B \cdot D \cdot 4 \cdot \left( \frac{S_{u,ave}}{\gamma_m} \right) : \text{for ISO} \]  

(3.17)

Equations (3.14) and (3.16) for the calculation of the second term are taking the friction between the two skirts on the sides of the foundation and the surrounding soil strength into account. In equation (3.14) as considered for the API design code contains a constant shear strength while equation (3.16) as considered for the ISO design code contains an average shear strength value for the embedment depth. For the calculation of the third term, which is the resistance on the skirts due to the active and passive wedge forces, the difference between the passive wedge as calculated in equation (3.19) and the active wedge as calculated in equation (3.20) is considered. These two equations are valid for the ISO design code, in order to use them for the API design code the average shear strength \( s_{u,ave} \) should be replaced by a constant shear strength \( s_u \). The result for the calculation of the difference between the passive and the active wedge force is shown in equations (3.18), (3.19) and (3.20) for the API and ISO design codes.

\[ H_{h,3} = H_{h,3,p} - H_{h,3,a} = B \cdot D \cdot 4 \cdot \left( \frac{S_{u,ave}}{\gamma_m} \right) : \text{for ISO} \]  

(3.18)

\[ H_{h,3,p} = \left( \sigma_v + 2 \cdot \left( \frac{S_{u,ave}}{\gamma_m} \right) \right) \cdot B \cdot D : \text{for ISO} \]  

(3.19)

\[ H_{h,3,a} = \left( \sigma_v - 2 \cdot \left( \frac{S_{u,ave}}{\gamma_m} \right) \right) \cdot B \cdot D : \text{for ISO} \]  

(3.20)

Some simplifications are considered for the calculation of the third term of the soil sliding capacity. The average shear strength value over the depth of the embedment is used for the ISO design code, while the exact
calculation would consist of an integration of the shear strength profile over the embedment depth. Since the simplification of using the average shear strength gives accurate enough results it is valid to use the simplification. This equilibrium between the passive and active wedge forces is derived for a strip foundation, thus in reality differences will occur due to the 3D effects of the foundation. The detachment of soil between the active wedge and the foundation is ignored in this analysis since clay is the investigated soil.

Figure 3.10 presents the results of the analysis for the sliding capacity terms for both the ISO and API design code. The red line represents the sliding capacity only for the first term as provided in the design codes. The blue line represents the sum of the first and second terms for the sliding capacity and the green line represents the sum of all three terms for the sliding capacity of the soil. The results for the ISO design code show a larger inclination which is caused by the increasing shear strength profile, while in the API design code a constant shear strength profile is used. From the results is seen that including the second and third term, which are the resistance between the skirts and the surrounding soil and the resistance on the skirts due to the active and passive soil wedge force, add significant sliding capacity to a foundation design.

![Figure 3.10: Results analysis to sliding capacity terms in undrained conditions](image)

### 3.2.2. Drained conditions

An analytical analysis of the design codes for the sliding capacity in drained conditions is not presented in this report since drained soil conditions are not in the scope of this study.

### 3.3. Settlement

In general five categories of settlements for subsea shallow foundations in the literature:

1. Immediate settlement
2. Consolidation settlement
3. Creep
4. Settlement due to cyclic loading
5. Differential settlement

In the API design code equations for the calculation of immediate settlements, consolidation settlements and differential settlements are provided for a circular, rigid shallow foundation in isotropic and homogeneous soil conditions. The provided equation for the immediate settlement in vertical direction is shown in equation 3.21 and in horizontal direction in equation 3.22. Consolidation settlement in vertical direction is calculated by equation 3.23. For the differential settlement equation 3.24 is provided to calculate the rotation of a shallow foundation due to moments, torque and eccentricity. The foundation rotation due to differential horizontal loads is calculated by equation 3.25 in the API design code. In the ISO design code no equations are provided to calculate the settlement of a shallow foundation. The ISO design code gives references to standard
soil mechanics text books in order to calculate settlements, which are similar to the settlement equations provided in the API design code. So in practice there is no difference between the API and ISO design codes regarding settlement calculations.

\[ u_{v,sl} = \left( \frac{1 - \nu}{4GR} \right) V \]  
\[ u_h = \left( \frac{7 - 8\nu}{32(1 - \nu)GR} \right) H \]  
\[ u_{v,lt} = \frac{hC}{1 + e_0} \log \frac{\sigma_0 + \Delta q}{\sigma_0} \]  
\[ \theta_r = \left( \frac{3(1 - \nu)}{8GR^3} \right) M \]  
\[ \theta_t = \left( \frac{3}{16GR^3} \right) T \]  

In order to determine the foundation settlements with the equations from the API and ISO design codes loading conditions, soil parameters and model parameters are required. The loading condition are the vertical and horizontal loads and the overturning and torsional moments. The required soil parameters are the initial void ratio, the compression index, the layer thickness and the initial and added effective stresses. The model parameters are the elastic shear modulus and the poisson's ratio.

Since such limited information regarding the soil stiffness parameters is available from the performed site investigation review, the study regarding the possible embedded conservatism in the settlement calculations of the design codes API RP 2A-WSD and ISO 19901-4 is not investigated in this study. More accurate information regarding the soil stiffness parameters is required for such a study.

3.4. Other aspects

3.4.1. Dynamic and cyclic loading

The before discussed equations are only applicable under static or equivalent static loading. Equivalent static loading is the maximum cyclic loading used as a static load. If dynamic or cyclic loads are imposed at the foundation the API and ISO design codes prescribe that these loads should be analysed by a dynamic or cyclic analysis. In both design codes no guidance is given on how to perform a dynamic or cyclic analysis.

3.4.2. Hydraulic instability

Scour

In the API and ISO design codes it is prescribed that positive measures should be taken to prevent erosion and undercutting of the soil beneath or near the shallow foundation due to scour. Examples of such measures are the application of skirts penetrating the erodible layers, scour-resistant materials placed around the edges of the foundation or performing sediment transport studies.

Piping

The API and ISO design codes describes that the shallow foundation should be designed such that the creation of excessive hydraulic gradients which can lead to piping, are prevented.

3.4.3. Installation

The API and ISO design codes describes that the shallow foundation should be installed without excessive disturbance to the supporting soil, and multiple set-downs due to wave motions should be prevented. Since Allseas uses vessels with state of the art heave-compensation systems for the installation of subsea structures these installation issues are not considered to be a problem and are not investigated further in this study.
3.5. Conclusions

From the analytical analysis of the API RP 2A-WSD and ISO 19901-4 design codes several conclusions can be drawn. These conclusions are discussed in this section of the report.

Some limitations should be considered while using the API and ISO design codes for a foundation design. The used bearing capacity equations in the API and ISO design codes are mainly based on the bearing capacity equations from Vesic and Brinch Hansen. These bearing capacity equations from Vesic and Brinch Hansen are derived for idealized conditions with a single homogeneous soil layer of perfectly plastic material. Only for static or equivalent static loads the equations are valid. The use of these equations should be reconsidered in situations with the following conditions:

- Highly nonhomogeneous or anisotropic soils.
- Loading conditions deviate considerably from the simple conditions assumed in the stability equations.
- Loading rates are such that the conditions are not clearly drained or undrained.
- Foundation geometry is highly irregular.

When the use of the stability equations from the API and ISO design codes is not justified, a more refined analysis such as a numerical analysis or scaled physical model test should be considered.

A comparison between the API RP 2A-WSD and the ISO 19901-4 design codes shows the following differences between the two design codes:

- The API design code utilizes a global safety factor approach while the ISO design code utilizes a partial safety factor approach.
- Only a constant shear strength profile can be used in the API design code while a constant or increasing shear strength profile can be used in the ISO design code.
- The stability equations for the bearing capacity of the API design code are mainly derived from the author Vesic while these equations in the ISO design code are mainly derived from the author Brinch Hansen.
- The applied correction factors on the cohesion term of the bearing considered in the ISO design code while they are considered in the API design code.
- The effective area, which is used due to eccentric loading in the API design code is reduced in two directions while in the ISO design code it is reduced only in the width direction.
Case studies

Case studies, with the actual site investigation data, are used for the comparison between the analytical analysis from the design codes and the numerical analysis performed by the FEM program PLAXIS. These cases are actual projects, designed and installed by Allseas in the recent years. First the case studies are introduced in this section, hereafter the performed site investigation is reviewed and engineering parameter values are determined.

4.1. Case study 1 and 2

4.1.1. Introduction

The first and second case study as used in this investigation are part of a project off the coast of Israel. A subsea gas transportation system which connects a deepwater gas field to a gas production plant in Israel is developed by Allseas for this project. Figure 4.1 presents the location of the project and and figure 4.2 provides an overview of the subsea gas transportation system.

Figure 4.1: Location of case study 1 and 2 after [1]
Figure 4.2 shows the new subsea gas transportation system which consists of pipelines as shown as the red lines and offshore platform 1. The yellow pipelines and offshore platform 2 are an already installed gas transportation system. The hydrocarbon field with wellheads is located in the top left of the figure and 10 inch flowlines are installed between the wellhead and the manifold located in the center of the gas field. At the wellhead and manifold ends of the flowlines FLET (flowline end terminations) structures are installed in J-mode. The manifold is connected to offshore platform 1 by two 150 km long 16 inch pipelines with 4.5 inch piggyback lines. At both ends of the two pipelines PLET (pipeline end terminations) structures are installed in J-mode. For an explanation of the J-mode installation process a reference is made to appendix E. The piggyback lines are used to transport fluids to the manifold for flow assurance in the pipelines. The installation of the flowlines and pipelines with piggyback lines is done by Allseas pipelay vessels Solitaire and Audacia and the J-mode installation of the FLET and PLET structures is done by Audacia.

The gas field is located at approximately 1,700 meter waterdepth and the water level decreases to 230 meter at the location of platform 1. The hydrocarbon products are processed in platform 1 whereafter they are transported to an onshore processing plant by a bundle of a 10 inch and two 6 inch pipelines. This pipeline bundle is initiated at the onshore landfall by a beach pull-in through an existing conduit for a 24 inch pipeline. The bundle of pipelines is at sea trenched to 60 meter water depth.

For the project the following structures are installed:
- 2 deep water PLET structures at approximately 1,660 meter water depth.
- 2 shallow water PLET structures at approximately 230 meter water depth.
- 10 deep water FLET structures at a water depth ranging from 1,650 to 1,700 meter.

### 4.1.2. Subsea structures

Three subsea structure foundation designs are made for all the subsea structures in this project. For the two deep water PLET structures one foundation design is made. The dimension of this foundation design are 10 meter wide and 11 meter long. For the FLET structures two foundation designs are made, one for the 5 FLET structures located at the wellheads and one for the 5 FLET structures located at the manifold. A 8.6 meter wide and 8.9 meter long FLET foundation design is made for the wellhead FLETs and a 7 meter wide and 8 meter long foundation design is made for the manifold FLETs. The design for the two shallow water PLET structures is not made by Allseas and these subsea structures are therefore not included in this study. The foundation design of the deep water PLET structures and wellhead FLET structures are used as case study 1 and 2 in this study. Figure 4.3 presents a visualisation of the considered subsea structures:
• **Case study 1** - The foundation design for the 16 inch PLET structures in deep water at approximately 1,650 meter water depth. Figure 4.3a presents an impression of these structures.
• **Case study 2** - The foundation design for the 10 inch FLET structures near the wellheads in the gas field at approximately 1,700 meter water depth. Figure 4.3b presents an impression of these structures.

4.1.3. Site investigation data

During the site investigation, geophysical and geotechnical investigations are performed. The bathymetry of the site is investigated by a sidescan and multi-beam during the geophysical investigation. In the geotechnical investigation CPT tests are performed and soil samples taken for soil tests. The soil samples are collected by box cores, piston cores and jumbo piston cores.

The site investigation results show that the seabed consists of very soft clays for water depths above 1,100 meter and soft silty clays in water depths below 1,100 meter. The result of the geotechnical investigation indicate that the clay at the seabed consists of two layers. The top clay layer is extremely soft and contains shell fragments and the clay layer directly below is slightly firmer.
The site investigation for the first and second case study is divided in two sections, the gas field area and the pipeline route to offshore platform 1. The pipeline route from the offshore platform 1 to the onshore processing plant is not included in the site investigation.

Hydrocarbon field
During the geophysical investigation, a sidescan and a multi-beam are used to investigate the bathymetry of the seabed. During the geophysical investigation, 95 sidescan contacts are identified. Two of these objects are indicated as old wells, 15 as possible anchor locations and 78 as unidentified objects. Ridges up to 800 meter wide and 65 meter high are observed. During the geophysical investigation an area of approximately 11 km by 14 km is investigated.

The performed geotechnical investigation in the hydrocarbon field consisted of one 20 meter deep CPT. Also 12 piston core samples of 6 meter, 1 jumbo piston core sample of 20 meter and 13 box core samples of 0.40 meter are taken. These soil samples are further investigated in geotechnical soil tests.

Pipeline route
The pipeline route from the gas field to the offshore platform is investigated during the geophysical investigation with a sidescan and multi-beam. They are used to investigate the bathymetry of the pipeline route from a water depth of 1.648 meter to 237 meter water depth. During the geophysical investigation depressions, reworked slide areas, slope failure, faults, a shipwreck and gravity creeping geohazards are found.

In the geotechnical investigation of the pipeline route no in-situ soil tests are performed, 20 box core samples and 6 piston core samples are taken. The piston core samples are all taken near the gas field manifold and the offshore platform where the deep water and shallow water PLET structures are installed. In between, on the pipeline route, only box core samples are taken. These box core samples are used for the pipeline design.

Soil sample locations
During the geotechnical investigation, soil samples are taken at different locations. These soil samples are used for soil tests on the survey vessel and in an onshore laboratory. Table A.1 in appendix A presents all soil sampling locations, the water depth at the soil sampling location, the penetration in the soil and how much of the penetration is recovered.

The distance from the soil sampling locations to the locations of the PLET and FLET structures is calculated and shown in table A.2 of appendix A. These distances are used to determine which soil samples are used for the determination of the engineering values for the required soil parameters.

Table A.2 in appendix A presents in red the used soil sampling locations for the determination of the engineering values for the soil parameters of case study 1 and 2. The soil parameter values for case study 1 are determined from soil sampling location 26, a piston core sample, and the CPT location where a 20 meter deep CPT test is performed. The piston core at soil location 26 is located closest to the PLET structures. Soil sampling location 14 also is located relative close to the PLET structures however this is a disturbed box sample of the upper 0.40 meter of the seabed. The soil sample does not provide sufficient information for the determination of soil parameter values. Other soil sampling locations all are located at significant larger distance from the PLET structures and are not used for the determination of soil parameter values for the foundation design of case study 1.

For the determination of the engineering parameter values for the foundation design of case study 2 soil sampling locations 1, 2, 4, 5 and 6 are used since these locations are located closest to the location of the FLET structures. Other soil sampling locations also are within 100 meter distance from the FLET structures, however these are disturbed box core samples and they only sample the upper 0.40 meter of the seabed soil. The soil sampling locations with deeper soil samples all are located at a minimum distance of 1,000 meter.

Table 4.1 presents a summary of the used soil sampling locations for the determination of the engineering soil parameter values for case studies 1 and 2. Soil samples from other locations are not used because of the distance between their location and the location of the investigated subsea structures or because they do not provide sufficient information.
4.1.4. Soil data case 1

The retrieved soil sample from soil location 26 is used to perform soil tests for the determination of soil parameter values for case study 1. This section presents the results of these soil tests.

### Undrained shear strength

Figure 4.4 presents the results for the undrained shear strength $s_u$ of the performed soil tests and the CPT. Four soil tests are performed on the soil sample, namely a torvane, handvane, lab vane and triaxial test. The test results show some scatter however a general increasing trend in depth is seen.

![Figure 4.4: Soil data for undrained shear strength $s_u$ as used for case study 1](image)

### Remolded undrained shear strength

The remolded undrained shear strength is the strength of remolded soil, this remolded state is reached after the soil has failed. The hand vane and lab vane are the used soil test to investigate the remolded undrained shear strength of the soil. Figure 4.5 presents the results of the soil tests where an increasing trend with depth is seen, similar to the undrained shear strength. Below approximately 5 meter depth a decrease in the remolded shear strength is seen.

![Figure 4.5: Remolded undrained shear strength as used for case study 1](image)
4.1. Case study 1 and 2

Submerged unit weight
Figure 4.6 presents the results for the submerged unit weight of the soil at six different depths. The submerged unit weight results show a slightly decreasing trend in the upper 5 meter. At 7 meter depth a test result with a higher submerged unit weight is shown. However since only 1 test result which deviates from the trend is seen, no conclusions can be drawn.
Moisture content
Figure 4.7 presents the results for the moisture content at different soil depths. The moisture content in the upper 5 meter is relatively constant in depth while at 6.5 meter depth lower moisture content test results are seen.

![Figure 4.7: Soil data for moisture content as used for case study 1](image)

Particle type distribution
Only one particle size test is performed on the soil sample. The distribution of particle sizes in the soil is determined in this test, however the distribution of the different particle size classes is not shown in the geotechnical report. Only the particle type distribution as the percentage of clay, silt and sand is shown in the geotechnical report. Figure 4.8 presents the results for the particle type distribution. The soil at 6 meter depth consists of roughly 60 percent clay, 35 percent silt and a few percent sand particles.

![Figure 4.8: Soil data for particle type distribution as used for case study 1](image)
Atterberg limits
Figure 4.9 presents the Atterberg limits for the soil at different depths. The liquid limit, plastic limit and plasticity index are shown in the figure for case study 1.

4.1.5. Determination of engineering soil parameter values case 1
The results from the soil tests are used to determine the engineering soil parameter values for case study 1. These engineering soil parameter values are used for the foundation design of case study 1. In this section the determination of the soil parameter values for case study 1 is discussed.

Undrained shear strength
Figure 4.10 presents the results from the CPT and other performed soil tests. The determined profile for the engineering parameter value of the undrained shear strength $s_u$ is shown in the figure by the red line.

In the literature some relations between different soil testing techniques for the undrained shear strength are found in order to reduce the scatter in the soil data. In [54] it is found that the lab vane should produce slightly higher results for the undrained shear strength compared to the torvane test because the labvane is a more operator controlled technique. This relation between the labvane and the torvane is however not seen in the soil test results in figure 4.10. In [54] it is discussed that the data scatter from the lab vane and torvane is higher compared to other shear strength testing techniques due to the small volume of the tested soil samples. This larger data scatter is shown in figure 4.10. Another relation found in [54] is the relation between the in situ measured shear strength and the shear strength from the UU triaxial test, which should be approximately equal. The results in the figure do no correspond with this relation between the in situ measured shear strength and the result from the UU triaxial test.

For the API design code a constant shear strength profile in depth is required while for the ISO design code either a linear increasing shear strength profile in depth or a constant shear strength profile is required. Figure 4.10 presents the determined increasing shear strength profile as the red line. From a linear regression analysis a value of $R^2 = 0.869$ is found for the linear increasing shear strength profile on the shear strength data. The $R$-value indicates that the determined increasing shear strength profile fits the data well. The determined constant shear strength profile as required in the API design code is shown as the green line in the figure.
The value for the constant shear strength profile is determined as $s_u = 6.1 \text{kPa}$. The method of determining the constant shear strength profile is shown in appendix B. The increasing shear strength profile is described by equation 4.1. In this equation the value of the undrained shear strength at a specific depth is determined by parameter $s_{um}$, the value of the undrained shear strength at the seabed, parameter $k$, the value of the undrained shear strength increase in depth and parameter $z$, the depth in the soil.

$$s_u = s_{um} + k \cdot z$$  \hspace{1cm} (4.1)

The determined engineering values for $s_{um}$ and $k$, as shown in figure 4.10 are:
- $s_{um} = 3 \text{kPa}$ is the undrained shear strength at the seabed level
- $k = 2.1 \text{kPa} / \text{m}$ is the increase of $s_u$ in depth

**Overconsolidation ratio**

Figure 4.10 shows that the soil is overconsolidated by the value of the undrained shear strength at the seabed level which does not go through the origin. This overconsolidation is not investigated during the site investigation, however by using equation 4.2 the OCR can be determined from the undrained shear strength. The determined overconsolidation ratio from this equation for the determined linear increasing shear strength profile is $OCR = 2.5$.

$$s^OC_u = 0.22 \cdot \sigma'_{v} \cdot OCR^{0.8}$$  \hspace{1cm} (4.2)

**Submerged unit weight**

The engineering value for the submerged unit weight is determined as a constant value because the design codes require a constant soil unit weight value in depth. Figure 4.11 presents the soil test results by the blue triangles and the determined submerged unit weight as the red line. The submerged unit weight as used for case study 1 is determined as $\gamma = 5.6 \text{kN/m}^3$. 

![Figure 4.10: Engineering soil parameter values for the undrained shear strength $s_u$ as used for case study 1](image-url)


Moisture content

Figure 4.12 presents the results of the soil tests by the blue triangles and the determined engineering parameter value for the moisture content by the red line. In the upper 5 meter the moisture content value is determined as a constant value of 80%. Below 5 meter the moisture content value is determined as decreasing to a value of 50% at a depth of 7 meter below the seabed.

Non-linear increase undrained shear strength

Figure 4.9 presents the results of the Atterberg Limits. The Atterberg Limits are a basic measure of the critical water content of a fine grained soil. The soil may appear in different states depending on the actual moisture content (MC) in the soil, the liquid limit (LL) and the plastic limit (PL). The different states the soil can appear in are listed below.

\[
\begin{align*}
MC > LL & \quad \text{- Soil behaviour is liquid} \\
LL > MC > PL & \quad \text{- Soil behaviour is plastic} \\
PL > MC & \quad \text{- Soil behaviour is solid}
\end{align*}
\]

Figure 4.13 presents the Atterberg Limits and the engineering value for the moisture content. In the upper 4 meter the moisture content lies above the liquid limit, which indicate a liquid soil behavior. Below 4 meter
depth the moisture content lies below the liquid limit which indicate a plastic soil behavior. The transition from liquid to plastic behavior of the soil indicate a bi-linear undrained shear strength profile instead of a linear increasing profile. In the upper 4 meter soil the increase in shear stength is less than in the soil below 4 meter depth. From this site investigation the different shear strength increase values can not be determined however it is shown that there is a difference in the shear strength increas above and below approximately 4 meter soil depth.

**Multiple soil layers**

In the geophysical investigation two clay layers were found, the top layer is extremely soft and contains shell fragments while the clay layer directly below is slightly firmer. The results from the geotechnical investigation also indicate two soil layers in the investigated soil depth. Most of the investigated soil parameter have a change in the trend at approximately 4 to 5 meter depth. Figures 4.11 and 4.12 shown below approximately 4 to 5 meter depth an increase in submerged unit weight and a decrease in moisture content. If the moisture content decreases, it indicates a denser soil which would indeed have a higher unit weight. Figure 4.13 presents the moisture content and the Atterberg Limits, also a stiffer soil layer below 4 meter depth is indicated compared to the upper 4 meter due to the change from liquid to plastic behavior. All these soil tests results indicate two clay layer where the lower clay layer is firmer. Contradictory to a firmer lower clay layer, the results for the undrained shear strength as shown in figure 4.10 show no bi-linear behavior for the undrained shear strength over the investigated depth.

**4.1.6. Soil data case 2**

The foundation design for case study 2 is used for the 5 FLET structures near the wellheads. Soil samples taken withing 500 meter of each of the investigated FLET structures are used for the determination of the engineering soil parameter values for case study 2. Table 4.2 presents the distances between the used soil samples and the FLET structures.

<table>
<thead>
<tr>
<th>Distance soil sample to subsea structure in [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil location 1</td>
</tr>
<tr>
<td>Soil location 2</td>
</tr>
<tr>
<td>Soil location 4</td>
</tr>
<tr>
<td>Soil location 5</td>
</tr>
<tr>
<td>Soil location 6</td>
</tr>
</tbody>
</table>

Table 4.2: Distance between locations FLET structures and the used site investigation locations
**Undrained shear strength**

Figure 4.14 presents the results of the soil tests for the undrained shear strength from the torvane, hand vane, lab vane and triaxial test. The soil test results show a large scatter which can be explained by the distance between the different used soil locations as presented in table 4.2.

![Graph showing undrained shear strength](image)

**Remolded undrained shear strength**

The remolded undrained shear strength is the undrained shear strength of the soil after failure. The hand vane and lab vane soil tests are performed in order to determine the engineering soil parameter values. Figure 4.15 presents the results from the soil tests. Some large scatter, similar to the test results for the undrained shear strength, is seen in the figure. This large data scatter can be explained by the distances between the soil locations.

![Graph showing remolded undrained shear strength](image)
4. Case studies

Submerged unit weight
The submerged unit weight of the soil is determined for different depths on 5 soil samples. Figure 4.16 presents the results from the soil tests.

![Submerged unit weight graph](image)

**Figure 4.16: Soil data for submerged unit weight $\gamma$ as used for case study 2**

Moisture content
Figure 4.17 presents the results for the moisture content as tested on different soil depths in the soil samples collected during the geotechnical investigation. A relative constant moisture content is seen over the depth, however also some scatter in the results is shown which can be explained by the distance between the investigated FLET locations.

![Moisture content graph](image)

**Figure 4.17: Soil data for moisture content as used for case study 2**
4.1. Case study 1 and 2

**Particle type distribution**
The particle size distribution of the soil is determined at different depths in the soil samples. Not the particle size distribution but the particle type distributions is shown in the geotechnical report provided to Allseas. Figure 4.18 presents the results for the particle type distribution where the particle type distribution of sand, clay and silt is shown in percentages. Relative constant percentages are shown from the results for the investigated depth of the soil.

![Particle type distribution graph](image)

**Atterberg limits**
Figure 4.19 presents the results for the Atterberg Limits. From the results is shown that there is an increasing trend in the plastic limit, liquid limit and plasticity limit. There also is some scatter shown in the results, this can be explained by the distance between the five soil sampling locations as used for the determination of engineering soil parameter values for case study 2.

![Atterberg limits graph](image)
4.1.7. Determination of engineering soil parameter values case 2

The results from soil tests performed during the geotechnical investigation are used to determine the engineering soil parameter values for as used for the foundation design of case study 2. In this section the determined engineering values of soil parameters for the second case study discussed.

Undrained shear strength

Figure 4.20 presents the results of the soil tests for the undrained shear strength, the determined increasing engineering shear strength profile as shown by the red line and the determined constant shear strength profile as shown by the green line.

In the literature some relations between different soil testing techniques are described. In [54] is found that the torvane and lab vane techniques show relative large scatter in the soil test results due to the small soil sample sizes. This large scatter also is shown in the results from the geotechnical soil tests. Other soil test techniques show less scatter in the results because larger soil samples are used. In [54] also is found that the undrained shear strength results of an UU triaxial test and in-situ tests should have similar test result values and that the lab vane gives slightly higher results compared to the torvane testing technique. The UU triaxial and in-situ results show however not a similar result, as shown in figure 4.20. The results from the lab vane also are not slightly higher compared to the torvane results.

The determined engineering value for the undrained shear strength parameter as shown as the red line in figure 4.20 is increasing in depth. From a linear regression analysis a value of $R^2 = 0.523$ is found. This low $R^2$-value is due to the large scatter in test result data. The linear increase in depth is characterised by equation 4.1 in which $s_{u,m}$ the value of the undrained shear strength at the seabed is, $k$ is the value of the undrained shear strength increase in depth and $s_u$ is the value for the undrained shear strength in depth. The determined values $s_{u,m}$ and $k$ for the determined engineering parameter values as shown in figure 4.20 are:

- $s_{u,m} = 1.35kPa$ is the undrained shear strength at the seabed level
- $k = 2.8kPa/m$ is the increase of $s_u$ in depth

![Figure 4.20: Soil data for undrained shear strength $s_u$ as used for case study 2](image)

The value of the constant undrained shear strength value, shown as the green line in figure 4.20, is determined as $s_u = 4.2kPa$. The method of the determination of this value is explained in appendix B.
Overconsolidation ratio
Figure 4.20 shows that the soil is overconsolidated because the determined increasing shear strength profile, as shown by the red line, does not go through the origin. The overconsolidation ratio is not investigated during the geotechnical investigation, however it can be determined with equation 4.2 from the determined linear increasing shear strength profile. From equation 4.2 is found that the overconsolidation ratio for the soil is \( OCR = 2.9 \).

Submerged unit weight
Figure 4.21 presents the results and the determined engineering value for the submerged unit weight. The data indicates a decrease of the submerged unit weight in depth however since the design codes requires a constant value in depth. Therefore the engineering value for the submerged unit weight is determined as a constant value of \( \gamma = 4.5 \, kN/m^3 \).

Moisture content
Figure 4.22 presents the results from the soil tests for the moisture content and the determined engineering value for the moisture content for case study 2. Some scatter in the results is seen, however a constant value in depth of 75% is determined as the moisture content for case study 2.
Non-linear increase of undrained shear strength

Figure 4.23 presents the Atterberg Limits and the determined engineering value for the moisture content of case study 2. In the upper 1.5 meter the moisture content lies above the liquid limit and below 1.5 meter the moisture content lies below the liquid limit. A soil with a moisture content above the liquid limit behaves as a liquid soil while a soil with a moisture content in between the liquid and the plastic limit behaves as a plastic soil. This change in soil behaviour indicate two soil layers in the investigated soil depth. In the upper 1.5 meter a weaker soil is found compared to the soil below 1.5 meter. The increase in undrained shear strength as shown in figure 4.20 is determined as a linear increase in depth, however the Atterberg Limits and the moisture content indicate a bi-linear increase of the undrained shear strength in depth. This bi-linear relation can however not be used in the design codes since either a constant or linear increasing shear strength profile in depth is required by the design codes.

![Figure 4.23: Engineering soil parameter for Atterberg Limits and moisture content used for case study 2](image)

4.1.8. Determination of stiffness parameters case study 1 and 2

Settlement calculations according to the design codes requires two model parameters; poisson ratio $\nu$ and shear modulus $G$ as shown in equations 3.21 to 3.25. The poisson ratio is $\nu = 0.5$ under undrained conditions, however in order to prevent numerical problems in the FEM program PLAXIS a value of $\nu = 0.495$ is assumed for the poisson ratio. These numerical problems occur by a poisson ratio of $\nu < 0.5$ because a division by 0 appears in the used equations. Shear modulus $G$ is direct related to the Young's modulus $E$, which can be determined by either soil tests performed during the site investigation or correlations with other soil parameters.

The compression test and triaxial test are soil tests capable of determining the Young's modulus $E$. These tests are however not performed during the site investigation and can not be used for the determination of the Young's modulus for soil located below case 1 and 2.

The Young's modulus can also be determined by using correlations with other soil parameters. From the literature correlations with the plastic limit $w_p$, liquid limit, $w_l$, plasticity index $I_p$, void ratio $e$, saturation $S$, cone tip resistance $q_c$, the over consolidation ratio OCR and the undrained shear strength $s_u$ are found in [13], [20], [31],[52], [55] and [61]. Most of these correlations however can not be used in this study since the required soil parameters as used in the correlation which the Young's modulus are not investigated during the site investigation or not provided to Allseas.

An usually found relation in the literature between the Young's modulus $E$ and undrained shear strength $s_u$ is shown in equation 4.3.

$$E = \mu \cdot s_u$$  (4.3)
In [13] the correlation $E = (100 - 500) \cdot s_u$ is found for soft clays which are investigated in this study. In [20] the correlation $E \approx (275 - 400) \cdot s_u$ is found for a clay with a slightly higher moisture content compared to case studies 1 and 2. The correlation of $E \approx (300 - 375) \cdot s_u$ is found in a design graph from [55] for a similar clay as found from the site investigation for case study 1 and 2. In [13] the correlation $E = 350 \cdot s_u$ is found for soils comparable to case study 1 and 2. From these correlations found in the literature a Young's modulus of $E = 350 \cdot s_u$ is determined for case study 1 and 2.

Due to the large uncertainty of the Young's modulus the possible embedded conservatism in the design codes regarding settlements is not investigated in this study. However during modeling with the FEM program PLAXIS the Young's modulus and poisson ratio are required. A Young's modulus of $E = 350 \cdot s_u$ and a poisson ratio of $\nu = 0.495$ are used in PLAXIS.
4.2. Case study 3 and 4

4.2.1. Introduction
Case study 3 and 4, used in this study, are part of a project of the coast of Brazil. The projects consists of a 307 km long gas pipeline, i.e. the green and red line as presented in figure 4.24. The pipeline is installed in water depths ranging from 0 to 2,296 meter. The project is divided into two sections, the deep water section as shown by the red line in figure 4.24 and a shallow water section as shown by the green line in the figure. The deep water section consists of 196 km pipeline with a diameter of 20 inch connecting PLET-001 at 2,190 meter water depth to PLET-002 at 650 meter water depth. The shallow water section consists of a 111 km 24 inch pipeline connecting PLET-003 at 650 meter water depth to the shore of Brazil. The shore approach is made by a pipeline installed with a horizontal directional drilling (HDD) technique.

![Figure 4.24: Location of case study 3 and 4 after [1]](image)

4.2.2. Subsea structures
The project consists of 21 designed subsea structures as presented in the project overview in figure 4.25. In general there are five different types of subsea structures designed in the project. The first type of subsea structures are the ILT and ILY structures. These structures provide hubs where additional parts of pipeline infrastructure can be connected, 9 of these structures are designed for the project. The difference between the ILT and ILY structure is the connection to the pipeline, this can be in a T-joint or Y-joint connection. Both the ILT and ILY structures are installed in S-mode by an Allseas installation vessel. The second type of subsea structures are the in-line valves which can close a part of the pipeline. Three in-line valves are designed for the project and they also are installed in S-mode. The third subsea structure is the pipeline diameter reducer. This structure reduces the diameter of the pipeline from 24 to 20 inch. Only one pipeline reducer is designed for the project. The PLEM (pipeline end manifold) structures are the fourth type of subsea structures designed for the project. These PLEM structures are not directly connected to the pipeline, they are connected through jumpers to the hubs located at the ILT, ILY or PLET structures which are connected to the pipeline. Five PLEM structures are designed for this project. The last type of subsea structures designed for this project are the PLET structures. Three PLET structures are designed for the project and they are located at the ends of the pipeline. The PLET structures are installed in J-mode by an Allseas vessel.
For this study only 2 of the 21 subsea structures are used as case studies. The 2 considered structures are:

- **Case study 3** - 20 inch PLET-001 as shown in figure 4.26a. This structure is located at 2,190 meter water depth and connected to a 20 inch pipeline. The foundation design of this subsea structure consists of a 13 meter long and 10 wide foundation.
- **Case study 4** - 24 inch PLET-002 and 24 inch PLET-003 are similar structures and shown in figure 4.26b. These structures are located at 650 meter water depth and connected to a 24 inch pipeline. The foundation design for these structures consists of a 11 meter long and 10 meter wide foundation.

### 4.2.3. Site investigation data

During the geotechnical investigation 52 CPTU were performed, 31 locations were sampled by a vibrocore, 13 T-Bar tests are performed and 67 soil locations are sampled with a Kullenberg sampler. The interpretation of the CPTU suggests very loose to dense sands underlain by clay at the shallow section of the pipeline route. At the deeper sections the main encountered sediments consisted of very soft clays. The interpretation of the results from the soil tests performed on the vibrocore samples support the CPTU interpretation.
The distance between the locations of the CPTU, T-Bar test, soil sampling and the locations of subsea structures PLET-001, PLET-002 and PLET-003 are determined and only the results from soil tests from soil locations relatively close to the subsea structures locations are used for the determination of the engineering soil parameter values. Table 4.3 presents the used soil locations and the distance between the performed soil tests and the three subsea structures.

<table>
<thead>
<tr>
<th>Soil locations</th>
<th>PLET-001</th>
<th>PLET-002</th>
<th>PLET-003</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT locations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>35</td>
<td>149,320</td>
<td>149,353</td>
</tr>
<tr>
<td>2</td>
<td>154,665</td>
<td>5,718</td>
<td>5,687</td>
</tr>
<tr>
<td>3</td>
<td>146,570</td>
<td>2,956</td>
<td>2,986</td>
</tr>
<tr>
<td>4</td>
<td>144,319</td>
<td>5,408</td>
<td>5,439</td>
</tr>
<tr>
<td>5</td>
<td>150,139</td>
<td>923</td>
<td>893</td>
</tr>
<tr>
<td>Soil sample locations</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>9,970</td>
<td>143,952</td>
<td>143,984</td>
</tr>
<tr>
<td>7</td>
<td>9,839</td>
<td>141,607</td>
<td>141,639</td>
</tr>
</tbody>
</table>

Table 4.3: Distance between used soil test locations and subsea structures for case study 3 and 4

4.2.4. Soil data case 3

Undrained shear strength

The CPT used to determine the undrained shear strength profile for case study 3 is located at CPT location 1 at a distance of 35 meter from case study 3 and presented in figure 4.27. The second closest soil location where soil data regarding the undrained shear strength is available is located at approximately 90 km distance. The soil data from that locations shows a similar trend for the undrained shear strength profile however due to the distance between the subsea structure and this soil location it is not considered for the determination of the engineering parameter value for the undrained shear strength of case study 3.

![Figure 4.27: Soil data for undrained shear strength $s_u$ as used for case study 3](image)
Soil unit weight
Figure 4.28 presents the results for the soil unit weight from soil tests performed on soil samples from soil locations 6 and 7. These soil locations are located at approximately 10 km distance from case study 3. This distance is relatively large, however since no closer soil data is available from the performed site investigation, the soil test results for a soil sample from soil locations 6 and 7 are used to determine the engineering values for the soil unit weight of case study 3.

![Figure 4.28: Soil data for unit weight $\gamma$ as used for case study 3](image)

4.2.5. Determination engineering soil parameter values case 3
Undrained shear strength The soil test results for the undrained shear strength as shown in figure 4.27 show an increasing trend with an increasing depth. The red line as presented in figure 4.29 is the determined undrained shear strength profile in depth for case study 3. The CPT data indicate an undrained shear strength of approximately zero at the seabed level, however since the design codes require a linear increasing shear strength this is not shown in the determined engineering value for the undrained shear strength.

![Figure 4.29: Engineering soil parameter for undrained shear strength $s_u$ as used for case study 3](image)
The determined undrained shear strength profile can be characterised by equation 4.1 where \( s_{u,m} = 3.3 \text{kPa} \) at the seabed level and an increase of undrained shear strength in depth of \( k = 1.6 \text{kPa/m} \) are used. From a linear regression analysis a value of \( R^2 = 0.824 \) is found. This \( R^2 \)-value indicates that the determined increasing engineering shear strength value fits the data well, which is also seen in figure 4.29.

The constant shear strength profile, shown as the green line in figure 4.29, is determined as \( s_u = 6.1 \text{kPa} \). This constant shear strength profile is required for the API design code and the method of the determination of this constant shear strength profile is discussed in appendix B.

**Overconsolidation ratio**

Figure 4.29 shows that the shear strength at the seabed is zero which indicates that the soil is not overconsolidated. The determined increasing shear strength profile however does not go through the origin and since this determined increasing shear strength profile is used in the calculations the corresponding overconsolidation ratio is determined by equation 4.2. From this equation an overconsolidation ratio of \( OCR = 2 \) is found for case study 3.

**Soil unit weight**

The determined engineering value for the unit weight of the soil for case study 3 is presented in figure 4.30 by the red line. A constant value of \( \gamma = 14.5 \text{kN/m}^3 \) over the investigated depth of the soil is determined.

![Figure 4.30: Engineering soil parameter for unit weight \( \gamma \) as used for case study 3](image)

**4.2.6. Soil data case 4**

**Undrained shear strength**

The undrained shear strength of the soil for case study 4 is determined from four CPTs at soil locations 2, 3, 4 and 5. The CPT from soil locations 5 is within 1 km distance from the location of case study 4, the other cpts are located at larger distances but within 6 km. The results from the four CPTs show a similar increasing trend in the depth as presented in figure 4.31.

**Unit weight**

Figure 4.32 presents the CPT results from CPT location 5 which is used for the determination of the soil unit weight value for case study 4. The results for the soil unit weight test show a relative constant value in the investigated soil depth.
4.2. Case study 3 and 4

4.2.7. Determination engineering soil parameter values case 4

Undrained shear strength

Figure 4.33 presents the determined engineering parameter value for the undrained shear strength by the black line. For the determination of this shear strength profile all four CPT results are used, however since the CPT at soil location 5 is located closest to the location of case study 4 this CPT result is given more weight compared to the other CPT results. The determined undrained shear strength profile can be characterised by equation 4.1 where the value at the seabed of $s_{u,m} = 4.8kPa$ and an increase of undrained shear strength in depth of $k = 2kPa/m$ are used. From a linear regression analysis a value of $R^2 = 0.831$ is found. This $R^2$-value indicates that the determined increasing shear strength profile fits the data well, as shown in figure 4.33.
The constant shear strength as required for the API design code is determined as $s_u = 8.1$ kPa and the method of this determination is discussed in appendix B. Figure 4.33 presents the determined constant engineering shear strength profile by the blue line.

Figure 4.33: Engineering soil parameter for undrained shear strength $s_u$ as used for case study 4

**Unit weight**

Figure 4.34 presents the determined engineering value of $\gamma = 16 \text{kN/m}^3$ for the soil unit weight of case study 4.

Figure 4.34: Engineering soil parameter for unit weight $\gamma$ used for case study 4
Overconsolidation ratio
Figure 4.33 shows that the CPT results go through the origin of the plot which indicate that there is no overconsolidation of the soil. The determined linear increasing shear strength value however does not go through the origin and since the determined increasing shear strength profile is used in the design codes the corresponding overconsolidation ratio is determined by equation 4.2. From this equation an overconsolidation ratio of \( OCR = 2.7 \) is found for case study 4.

4.2.8. Determination stiffness parameters case 3 and 4
The soil stiffness parameters can be determined by soil tests or correlations with other soil parameters. From the site investigation information of case study 3 and 4 only the undrained shear strength profile and the soil unit weight are provided to Allseas. The client provided the correlation as shown in equation 4.4 in order to determine the Young's modulus \( E \). With the received soil information it is not possible to validate this correlation since no information regarding soil tests or other soil parameters used in correlations are provided to Allseas. Therefore the correlation in equation 4.4 is used to determine the Young’s modulus in the FEM calculations with PLAXIS and a value of \( \nu = 0.495 \) is used for the poisson ratio in the undrained soil conditions.

\[
E = 300 \cdot s_u
\]  
(4.4)
Comparison design codes and Plaxis

5.1. Introduction
In this section the study to the possible embedded conservatism in the investigated design codes is presented. The methodology of the investigation will first be explained, then a summary of the input parameters found in the site investigation review and the Allseas structure design reports will be presented. Hereafter the investigation, the results and the conclusions are discussed.

5.2. Methodology
Four case studies are investigated to determine possible embedded conservatism in the design codes. From an investigation to the possibility to influence the foundation design it is found that only the aspect ratio and the embedment depth are design parameters that can be influenced by Allseas during foundation design. In order to find the sensitivity in relation with the aspect ratio and the embedment depth of the foundation design, these parameters are varied in this investigation.

In the ISO design code the partial safety factor approach is applied whereas in the API design code the global safety factor approach is applied. In order to compare the results from the analytical design codes with a numerical finite element method program analyses have been performed by PLAXIS. Two PLAXIS models are used to compare the result of analytical models with numerical models. In the first model the partial safety factor approach is applied in order to calculate the safety factor. The results from this PLAXIS model are compared to the results from the ISO design code. In the second PLAXIS model the global safety factor approach is applied and the results are compared to the results from the API design code.

The investigated design codes only consider two failure modes; bearing failure and sliding failure, and determine the safety factor against these failure modes. The two PLAXIS models are only used to calculate the safety factor against bearing failure and sliding failure. A foundation design made by PLAXIS can be further optimized, but since the foundation design then can not be compared to the foundation design according to the analytical design codes this is not done in this study.

Guidance to calculate the deformation of a shallow foundation is given in the investigated design codes. The deformation is not investigated in this study due to the limited available soil deformation parameters. In this study only the possible embedded conservatism regarding the strength of the soil is investigated.

5.3. Input parameters
The four different types of input parameters distinguished in this investigation are: soil parameters, model parameters, structure geometry parameters and loading conditions. These four input parameters are discussed in this paragraph.
5.3. Input parameters

Soil parameters
The soil parameters are determined during the site investigation review discussed in section 4. The determined soil parameter values are kept constant during the investigation since these values cannot be influenced by Allseas in the foundation design. The required soil parameters in this investigation are the shear strength at the seabed level, the shear strength increase in depth and the unit weight of soil.

From the site investigation review an increasing shear strength profile is found for the four case studies, however for the API design code a constant shear strength profile is required. Two methods are used to determine the constant shear strength profile: the soil reaction force method and the soil failure depth method. Both these methods require the use of a PLAXIS model. In the soil reaction force method a constant shear strength profile is determined which produces an equal soil reaction force compared to a soil with an increasing shear strength for an applied vertical soil displacement. In the soil failure depth method a displacement is applied to soils with a constant and increasing shear strength profile. The depth of the failure mechanisms is investigated in both the soil masses. If no PLAXIS model is available to determine a constant shear strength profile from an increasing shear strength profile a third method, the 2:1 approximation method is investigated. For the details and a discussion of the three methods is referred to Appendix B. The soil parameter values as used in this investigation are shown in table 5.1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear strength value at seabed</td>
<td>$s_{u,m}$</td>
<td>[kPa]</td>
<td>3</td>
<td>1.35</td>
<td>3.3</td>
<td>4.8</td>
</tr>
<tr>
<td>Shear strength increase</td>
<td>$k$</td>
<td>[kPa/m]</td>
<td>2.1</td>
<td>2.8</td>
<td>1.6</td>
<td>2</td>
</tr>
<tr>
<td>Constant shear strength value</td>
<td>$s_u$</td>
<td>[kPa]</td>
<td>6.1</td>
<td>4.2</td>
<td>6.1</td>
<td>8.4</td>
</tr>
<tr>
<td>Soil unit weight</td>
<td>$\gamma$</td>
<td>[kN/m³]</td>
<td>15.6</td>
<td>14.5</td>
<td>14.5</td>
<td>16</td>
</tr>
</tbody>
</table>

Table 5.1: Summary soil input parameters

Model parameters
The model parameters are the required parameters for the used soil model. In this analysis the Mohr-Coulomb soil model is considered, which is equal to the Tresca model for a friction angle of zero degrees for the investigated cohesive soils. The Mohr-Coulomb soil model is not an advanced soil model, however it requires only few soil parameters which can be determined from the performed site investigation. More advanced soil models require more parameters which are not investigated in the performed site investigation for the considered case studies. Table 5.2 presents the model parameters used in this study.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
<td>$E$</td>
<td>[MPa]</td>
<td>1050</td>
<td>475</td>
<td>990</td>
<td>1440</td>
</tr>
<tr>
<td>Young’s modulus increase</td>
<td>$E_{inc}$</td>
<td>[MPa/m]</td>
<td>735</td>
<td>980</td>
<td>480</td>
<td>600</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>$\nu$</td>
<td>[-]</td>
<td>0.495</td>
<td>0.495</td>
<td>0.495</td>
<td>0.495</td>
</tr>
</tbody>
</table>

Table 5.2: Summary model input parameters

Structure geometry parameters
The structure geometry parameters define the geometry of the structure. These parameters can be influenced by Allseas in the foundation design and these parameters are therefore varied in the investigation. The aspect ratio is varied from 0.5 to 1.0 and the embedment from 0 meter to 1.0 meter. These ranges for the aspect ratio and the embedment depth are found to be realistic values for the investigated structure types. Since foundation design is the scope of this study only the length, width and embedment depth of the shallow foundation designs are investigated. Other geometry parameters which have an influence on the structural and installation design are not considered as they do not fall within the framework of this thesis. The structure geometry parameters for the four case studies found in the Allseas design reports are shown in table 5.3.
5. Comparison design codes and Plaxis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Unit</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>$L$</td>
<td>[m]</td>
<td>11</td>
<td>8.9</td>
<td>13</td>
<td>13</td>
</tr>
<tr>
<td>Width</td>
<td>$B$</td>
<td>[m]</td>
<td>10</td>
<td>8.6</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Embedment</td>
<td>$D$</td>
<td>[m]</td>
<td>0.3</td>
<td>0.3</td>
<td>0.5</td>
<td>0.4</td>
</tr>
<tr>
<td>Foundation base tilt</td>
<td>$\beta$</td>
<td>[°]</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Aspect ratio</td>
<td>$B/L$</td>
<td>[-]</td>
<td>0.91</td>
<td>0.97</td>
<td>0.77</td>
<td>0.77</td>
</tr>
</tbody>
</table>

Table 5.3: Summary structure geometry input parameters

**Loading conditions**

The loading conditions define the loads acting on the subsea structures investigated in this study. In the four case studies similar H-V-M loading conditions are found in the Allseas structure design reports. The loading conditions used in this study are shown in table 5.4.

<table>
<thead>
<tr>
<th>Load</th>
<th>Unit</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_x$</td>
<td>[kN]</td>
<td>37</td>
<td>-58</td>
<td>-45</td>
<td>-74</td>
</tr>
<tr>
<td>$F_y$</td>
<td>[kN]</td>
<td>-42</td>
<td>-63</td>
<td>64</td>
<td>125</td>
</tr>
<tr>
<td>$F_z$</td>
<td>[kN]</td>
<td>-887</td>
<td>-627</td>
<td>-1190</td>
<td>-1229</td>
</tr>
<tr>
<td>$M_x$</td>
<td>[kN/m]</td>
<td>1308</td>
<td>714</td>
<td>-1490</td>
<td>-1420</td>
</tr>
<tr>
<td>$M_y$</td>
<td>[kN/m]</td>
<td>203</td>
<td>-357</td>
<td>1064</td>
<td>889</td>
</tr>
<tr>
<td>$M_z$</td>
<td>[kN/m]</td>
<td>235</td>
<td>203</td>
<td>472</td>
<td>707</td>
</tr>
</tbody>
</table>

Table 5.4: Summary loading conditions input

The axis coordinate system used for all four case study structures is shown in figure 5.1. The origin of the axis coordinate system is for all case studies located at the reference point in the foundation design. This reference point where the origin of the axis coordinate system is located is at the skirt tip level, in the geometrical center of the foundation.

![Figure 5.1: Coordinate system structure, figure after [1]](image-url)
All the investigated loads are acting on the reference point in the case study structures. Load eccentricities and moments are summed and applied as moments on the reference point in the structure. The considered loads and moments acting on the structures in this study originate from the mudmat, the pipeline, the jumper and the running tool.

- **Mudmat** - The self weight of the mudmat generates a vertical load. Due to the seabed slope also small horizontal loads are present.
- **Pipeline** - The self weight of the pipeline generates a vertical and small horizontal load due to the seabed slope. The axial expansion of the pipeline does not cause a large horizontal load acting on the structures since in case studies 1 and 2 a sliding frame is used to allow the pipeline an axial expansion of maximum 1.40 meter. In case study 3 and 4 the shallow foundation is allowed to move in horizontal direction due to the axial expansion of the pipeline. The residual torsional moment in the pipeline after installation is due to the installation process. During the installation the pipeline is plastically deformed in the overbend. The residual curvature causes a rotation because the bent pipeline wants to roll over.
- **Jumper** - After installation of a jumper the PLET and a hydrocarbon well are connected. At the jumper connection forces and moments from the jumper are acting on the structure.
- **Running tool** - The running tool is a piece of equipment used for the installation of the jumper to the structure. Due to the self weight vertical loads are acting on the structure foundation.

The loads acting on the structures which are not taken into account in this study are installation loads, environmental loads and cyclic and earthquake loads.

- **Installation loads** - Installation loads are not considered in this study since these are mainly influencing the structural design which is not investigated in this study. The installation loads described in the design codes are related to slamming the structure on the seabed and multiple set downs due to waves. The installation loads might have an adverse effect on the soil structure, however Allseas installation vessels are equipped with state-of-the-art motion control installation equipment so the effects from these installation loads are minimised.
- **Environmental loads** - The loads due to wind, waves and currents might have a large influence on the foundation design but these loads are not considered in this investigation due to the fact that there is no exposure to wind and the PLET structures are installed in deep water where no influence from waves is encountered. In line with the Allseas structure design reports the loading from the current is due to the limited structure height not considered within this study.
- **Cyclic and earthquake loads** - Cyclic and earthquake loads are not taken into account in this study since significant information regarding the soil cyclic and dynamic parameters is not available.

5.4. Investigation

In order to investigate the possible embedded conservatism in the ISO:19901-4 and API RP 2A-WSD design codes, the results from these two design codes are compared to the results from two PLAXIS models. A distinction is made between the global safety factor approach, which is used in the API RP 2A-WSD design code, and the partial safety factor approach which is used in the ISO 19901-4 design code.

5.4.1. PLAXIS model

In the FEM program PLAXIS a shallow foundation is modelled as a rigid rectangular plate. The use of a rigid rectangular plate is applicable since the designed foundation for the subsea structures is much stiffer compared to the surrounding soil according to [18], [30], [44] and [47]. The real foundation design consists of a steel plate with skirts positioned to the bottom. In the PLAXIS model however, a rectangular box with a depth equal to the skirt and plate height is modeled due to the variation in aspect ratio and embedment depth in this investigation. A model with the real structure geometry leads to extensive time needed to change the model for all different situations. Figure 5.2 illustrates the real geometry of the foundation. From this figure is seen that it is not feasible to modify the real geometry for all different situations within the time frame of this study. The simplification of the foundation model into a box is valid since enough inner skirts are designed for the actual foundation designs of the four investigated case studies according to [4] and [59]. The reader is referred to Appendix D for the description of all the used assumptions and the details of the PLAXIS model.
Determination of the safety factor

The $c - \tan(\phi)$ reduction method and the load increasing method are two available methods to determine a safety factor in PLAXIS. Both methods start with the same three calculation steps; the initial step where only the soil mass is modeled, the installation step where the shallow foundation is installed without any applied loads and the loading step where the loads from the Allseas design reports are applied on the foundation.

The $c - \tan(\phi)$ method is an option available within PLAXIS where the safety factor $\Delta M_{sf}$ is determined by PLAXIS itself. In this method the strength parameters of the soil, the undrained shear strength $s_u$ in this study, is reduced in steps until the soil fails. The safety factor is hereafter determined as the ratio of the initial soil strength over the soil strength at failure as shown in equation 5.1.

$$S.F. = \Delta M_{sf} = \frac{s_{u, input}}{s_{u, reduced}}$$

(5.1)

The load increasing method is a manual method to determine the safety factor in PLAXIS. In this method the calculation step ‘S.F. load increase’ is created following the loading step. In this ‘S.F. load increase’ step a load is increased until the soil fails. The ultimate load is hereafter calculated by using the calculated $\Delta M_{stage}$ value from the ‘S.F. load increase’ step. The $\Delta M_{stage}$ indicates how much of the load increase between two calculation steps is reached. From a $\Delta M_{stage}$-displacement graph it is possible to calculate the ultimate load for the shallow foundation. The safety factor in this method is hereafter determined as shown in equation 5.2.

$$S.F. = \frac{F_{ultimate}}{F_{initial}}$$

(5.2)

The load $F_{initial}$ is the load taken from the Allseas structure design reports and load $F_{ultimate}$ is the ultimate load where the soil failure start to occur. The main difference between the two methods to determine the safety factor is the ratio between the applied loads acting on the foundation. In the $c - \tan(\phi)$ method the ratio between the applied loads remains the same. In the manual load increasing method the ratio between the applied loads changes because one of the loads is increased until soil failure.

In this study only static loads are used since in the Allseas foundation design reports no distinction is made between the static and cyclic or dynamic loads. More over, the investigated design codes also do not provide guidance on how to use cyclic and dynamic loads in a calculation. They only mention that if cyclic and dynamic loads are present a more advanced calculation method should be used.

5.4.2. Foundation design according to API RP 2A-WSD

In the API RP 2A-WSD design code it is stated that shallow foundations should have an adequate margin of safety against failure under the design loading conditions. In the API design code two failure modes are indicated; bearing failure and sliding failure. A global safety factor approach is used in the API RP 2A-WSD design code, which means that all the loads and material parameters that are used in the PLAXIS model should be unfactored.
According to the API design code the safety factor against bearing failure should be equal or larger than 2.0 and the safety factor against sliding failure should be equal or larger than 1.5. Figure 5.3 presents the failure contour based on the API design code. The foundation design is considered safe in the green area and not safe in the red area, when the safety factors against bearing failure and sliding failure are determined and plotted in figure 5.3.

The safety factors against bearing failure and sliding failure in the PLAXIS model which is compared to the API design code are determined with the load increasing method. This method is used instead of the $c - \tan(\phi)$ method where the soil strength is reduced and the ratio between the different loads is kept the constant, while this is not the case in the API design code.

**Bearing failure**
The ultimate vertical loading which a shallow foundation can support under undrained conditions is determined in the API design code using equation 5.3. The analytical analysis of this equation for the ultimate vertical load is investigated in section 3.

$$Q_{ult} = (s_u \cdot N_c \cdot K_c + \gamma \cdot D) \cdot A'$$  \hspace{1cm} (5.3)

The safety factor against bearing failure is determined as the ratio of the ultimate vertical loading over the actual vertical load as shown in equation 5.4.

$$S.F. = \frac{Q_{ult}}{F_z}$$  \hspace{1cm} (5.4)

**Sliding failure**
The capacity of a shallow foundation against sliding failure in undrained conditions in the API design code is determined as shown in equation 5.5. An analytical analysis of this equation is found in section 3 of this report.

$$H_{ult} = s_u \cdot A + 2 \cdot s_u \cdot L \cdot D + 4 \cdot s_u \cdot B \cdot D$$  \hspace{1cm} (5.5)

The safety factor against sliding failure in the API design code is determined as the ratio between the ultimate horizontal loading and the actual horizontal loading as shown in equation 5.6.

$$S.F. = \frac{H_{ult}}{F_h}$$  \hspace{1cm} (5.6)
5.4.3. Foundation design according to ISO:19901-4

The stability of a shallow foundation should be analysed by limit equilibrium methods ensuring equilibrium between design actions and design resistance according to the ISO design code. This design code is based on the partial safety factor approach, where partial load factor $\gamma_l$ and partial material factor $\gamma_m$ are considered. A partial material factor of $\gamma_m = 1.25$ is considered, or $\gamma_m = 1.5$ in the absence of horizontal loading. For the loading action a partial load factor of $\gamma_l = 1.3$ is considered. In the ISO design code two failure modes are indicated; bearing failure and sliding failure.

According to the ISO design code the safety factor against bearing failure should be equal or larger than 1.0 and the safety factor against sliding failure should also be equal or larger than 1.0 as the design code considers the partial safety factor approach. The partial safety factors are already applied on the loads and material parameters in the calculation. In figure 5.4 a contour plot is shown for the ISO design code. The foundation design is considered safe in the green area and not safe in the red area, when the safety factors against bearing failure and sliding failure are determined and plotted in the figure.

![Figure 5.4: Failure contour for ISO design code](image)

In the PLAXIS model used for the comparison with the ISO design code the safety factors are determined with the load increasing method. This method is used instead of the $c - \tan(\phi)$ method where the soil strength is decreased the ratio between the different loads is constant in the $c - \tan(\phi)$ method, while this is not the case in the ISO design code.

**Bearing failure**

The ultimate vertical loading which a shallow foundation can support in undrained conditions with an increasing shear strength profile is determined in the ISO design code as shown in equation 5.7. The analytical analysis of this equation and the equation for a situation with a constant shear strength profile is discussed in section 3.

$$Q_{ult} = F \cdot \left( N_c \cdot s_{u0} + \frac{k \cdot B' \cdot K_c}{4} \right) \frac{K_c}{\gamma_m} + \gamma' \cdot D \cdot A' \left(5.7\right)$$

The safety factor against bearing failure is determined as the ratio of the ultimate vertical loading over the actual vertical loading as shown in equation 5.8.

$$S.F. = \frac{Q_{ult}}{F_z} \left(5.8\right)$$
5.4. Investigation

Sliding failure

The capacity of a shallow foundation against sliding failure in undrained conditions with an increasing shear strength profile is determined as shown in equation 5.9. The analytical analysis of this equation and for the equation applicable for situations with a constant shear strength profile are discussed in section 3.

\[
H_{ult} = \left( \frac{s_{u,0}}{Y_m} \right) \cdot A + 2 \cdot \left( \frac{s_{u,ave}}{Y_m} \right) \cdot L \cdot D + 4 \cdot \left( \frac{s_{u,ave}}{Y_m} \right) \cdot B \cdot D
\]  

(5.9)

The safety factor against sliding failure in the ISO design code is determined as the ratio between the ultimate horizontal loading and the actual horizontal loading as shown in equation 5.10.

\[
S.F. = \frac{H_{ult}}{F_h}
\]  

(5.10)

FEM input parameters

In the ISO design code a partial safety factor approach is applied. The partial material factors \( \gamma_m = 1.25 \) is used to reduce the soil strength parameters and the partial load factor \( \gamma_f = 1.3 \) is used to increase the load acting on the soil. In order to compare the results from the ISO design code and the PLAXIS model, the partial material and load factor should also be applied on the PLAXIS input parameters. In table 5.5 the parameters input values are shown for the PLAXIS model which is compared to the ISO design code.

<table>
<thead>
<tr>
<th>Soil parameters</th>
<th>Symbol</th>
<th>Unit</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear strength value at seabed</td>
<td>( s_u )</td>
<td>[kPa]</td>
<td>2.4</td>
<td>1.08</td>
<td>2.64</td>
<td>3.84</td>
</tr>
<tr>
<td>Shear strength increase</td>
<td>( k )</td>
<td>[kPa/m]</td>
<td>1.68</td>
<td>2.24</td>
<td>1.28</td>
<td>1.6</td>
</tr>
<tr>
<td>Soil unit weight</td>
<td>( \gamma )</td>
<td>[kN/m³]</td>
<td>12.5</td>
<td>11.6</td>
<td>11.6</td>
<td>12.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Model parameters</th>
<th>Symbol</th>
<th>Unit</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
<td>( E )</td>
<td>[MPa]</td>
<td>840</td>
<td>380</td>
<td>800</td>
<td>1150</td>
</tr>
<tr>
<td>Young’s modulus increase</td>
<td>( E_{inc} )</td>
<td>[MPa/m]</td>
<td>590</td>
<td>785</td>
<td>385</td>
<td>480</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loading conditions</th>
<th>Symbol</th>
<th>Unit</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal</td>
<td>( F_x )</td>
<td>[kN]</td>
<td>48</td>
<td>-75</td>
<td>-59</td>
<td>-97</td>
</tr>
<tr>
<td>Vertical</td>
<td>( F_y )</td>
<td>[kN]</td>
<td>-55</td>
<td>-82</td>
<td>83</td>
<td>162</td>
</tr>
<tr>
<td>Moment</td>
<td>( M_x )</td>
<td>[kNm]</td>
<td>-1153</td>
<td>-815</td>
<td>-1547</td>
<td>-1598</td>
</tr>
<tr>
<td>Moment</td>
<td>( M_y )</td>
<td>[kNm]</td>
<td>1700</td>
<td>928</td>
<td>-1937</td>
<td>-1846</td>
</tr>
<tr>
<td>Torsion</td>
<td>( M_z )</td>
<td>[kNm]</td>
<td>306</td>
<td>264</td>
<td>614</td>
<td>919</td>
</tr>
</tbody>
</table>

Table 5.5: FEM input parameters for partial safety factor approach

In PLAXIS it is possible to use Design Approaches [72]. By using the Design Approaches the unfactored input parameters can be used in PLAXIS. A shadow data set of input values is created in PLAXIS where the unfactored input values are multiplied or divided by selected partial safety factors. The use of Design Approaches is in particular useful when the same database of partial safety factors is used in multiple projects. In this study however the Design Approaches are not used and the input data is already factored before it is inserted in PLAXIS for simplicity.

5.4.4. Determination safety factor

If the applied vertical or horizontal load acting on the foundation is increased enough for soil failure to occur the ultimate load capacity of the soil can be determined in PLAXIS with the \( \Sigma M_{stage} \) value. In theory the \( \Sigma M_{stage} \) line should become horizontal after the soil fails, however this behavior is not seen in the PLAXIS results as shown in figure 5.5. In PLAXIS 2D the \( \Sigma M_{stage} \) becomes horizontal, but in PLAXIS 3D as used in this study the value of \( \Sigma M_{stage} \) keeps increasing due to the shape of the mesh elements. The graph starts in the origin and after the big bend in the graph the soil fails, the increasing part of the graph after the bend is an overestimation of the ultimate bearing capacity by PLAXIS. Failure is in this investigation not defined.
as the $\Sigma M$ stage value on the horizontal part of the graph but if the $\Sigma M$ stage value in between two numerical calculation steps becomes less than 0.0005. After this point a little load increase causes a large increase in displacements and the soil is considered as failed. The value 0.0005 is selected after an analysis of different situations and for this value the failure point for all situations lies on the right of the big bend, where soil failure starts. In the green part of the graph in figure 5.5 the $\Sigma M$ stage is increasing with more than 0.0005 in between two numerical calculation steps and this part of the graph is thus defined as no soil failure. In the red part of the graph the $\Sigma M$ stage is increasing with less than 0.0005 in between two numerical calculation steps and the soil is thus defined as failing. The numerical calculation in PLAXIS is stopped after 250 numerical steps. If the number of numerical steps is increased the graphs will continue and the failure part, shown as red in the figure, will become larger, but the transition between green and red will not be affected.

![Figure 5.5: Determination $\Sigma M$ stage value for soil failure](image)

### 5.5. Investigation results case study 1

The results for the investigation of case study 1 are presented in this section. First a sensitivity analysis for a range of aspect ratio's and embedment depths is performed for the safety factors against bearing and sliding failure from the design codes and the PLAXIS models. Hereafter the safety of the actual foundation design for case study 1 and sources of overprediction in the PLAXIS models are discussed.

#### 5.5.1. Bearing failure

**Aspect ratio**

Table 5.6 presents length and width dimensions considered for different aspect ratio's in case study 1. The actual foundation design area of case study 1 is 110 square meter and this area is kept constant for the other aspect ratio's. Only 2 decimals can be inserted in PLAXIS for the geometry input values of length and width, therefore a truncation error is introduced in the geometry input values and the area of the foundation design is not exactly 110 square meter for all aspect ratios. However this small variation in foundation area is not significant as shown in later sections. Therefore no further measures are taken to reduce the error in the foundation area for a variation in aspect ratio's.

Figure 5.6 shows the results of the analysis in terms of the safety factor against bearing failure for a range of aspect ratio's. In the left plot the results for the partial safety factor approach are presented to compare the ISO design code and the PLAXIS model, whereas in the right plot the results for the global safety factor approach are presented to compare the API design code and the PLAXIS model.
### 5.5. Investigation results case study 1

<table>
<thead>
<tr>
<th>Aspect ratio [-]</th>
<th>Length [m]</th>
<th>Width [m]</th>
<th>Area $[m^2]$</th>
<th>Area difference $[m^2]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>14.83</td>
<td>7.42</td>
<td>110.0386</td>
<td>0.03860</td>
</tr>
<tr>
<td>0.55</td>
<td>14.14</td>
<td>7.78</td>
<td>110.0092</td>
<td>0.00920</td>
</tr>
<tr>
<td>0.60</td>
<td>13.54</td>
<td>8.12</td>
<td>109.9448</td>
<td>0.05520</td>
</tr>
<tr>
<td>0.65</td>
<td>13.01</td>
<td>8.46</td>
<td>110.0646</td>
<td>0.06460</td>
</tr>
<tr>
<td>0.70</td>
<td>12.54</td>
<td>8.78</td>
<td>110.1012</td>
<td>0.10120</td>
</tr>
<tr>
<td>0.75</td>
<td>12.11</td>
<td>9.08</td>
<td>109.9588</td>
<td>0.04120</td>
</tr>
<tr>
<td>0.80</td>
<td>11.73</td>
<td>9.38</td>
<td>110.0274</td>
<td>0.02740</td>
</tr>
<tr>
<td>0.85</td>
<td>11.38</td>
<td>9.67</td>
<td>110.0446</td>
<td>0.04460</td>
</tr>
<tr>
<td>0.90</td>
<td>11.06</td>
<td>9.95</td>
<td>110.0470</td>
<td>0.04700</td>
</tr>
<tr>
<td>0.91</td>
<td>11</td>
<td>10</td>
<td>110</td>
<td>0</td>
</tr>
<tr>
<td>0.95</td>
<td>10.76</td>
<td>10.22</td>
<td>109.9672</td>
<td>0.03280</td>
</tr>
<tr>
<td>1.00</td>
<td>10.49</td>
<td>10.49</td>
<td>110.0401</td>
<td>0.04010</td>
</tr>
</tbody>
</table>

Table 5.6: Foundation length and width dimensions for different aspect ratios

The safety factor against bearing failure from the ISO design code tends to decrease as the aspect ratio increases. This decrease is small and a safety factor of approximately $S.F. = 2$ is considered for the range of aspect ratios. A constant safety factor of $S.F. = 4$ is found in the PLAXIS model for the range of aspect ratios. A ratio of 2 between the safety factor against bearing failure found in the ISO design code and the PLAXIS model is considered. From the results can be concluded that no significant effect for a variation in aspect ratio is found on the safety factor against bearing failure.

The comparison between the results of the API design code and PLAXIS shows that for the global safety factor approach the safety factor against bearing failure determined in the API design code is relatively constant for a range of aspect ratios. The calculated safety factor in the API design code is approximately $S.F. = 3.5$, and $S.F. = 5.8$ for the PLAXIS model respectively. A ratio of 1.7 between the calculated safety factors against bearing failure for a range of aspect ratios from the API design code and the PLAXIS model is considered. From the results can be concluded that no significant effect for a variation in aspect ratio is found on the safety factor against bearing failure.

There is some fluctuation in the safety factor values determined by PLAXIS as seen in figure 5.6. This fluctuations can be explained by the need to create a new mesh in the PLAXIS model every time the foundation is changed for a different aspect ratio. Due to these different meshes there is some variation in the calculated ultimate capacity by PLAXIS and thus in the determined safety factor.
Embedment depth

Figure 5.7 presents the results of the analysis performed to determine the influence of the embedment on the safety factor against bearing failure. The left plot shows the comparison between results from the ISO design code and the PLAXIS model. For an embedment depth of 0 to 0.6 meter a constant ratio of 2.2 is considered between the safety factor against bearing failure in the ISO design code and the PLAXIS model, while for an embedment larger than 0.6 meter the ratio increases. A possible explanation for this behavior could be the assumption of superposition in the used equation to calculate the bearing capacity of the soil in the ISO design code. This simplification deviates from the reality and in the PLAXIS model the embedment is modeled more realistic.

![Partial safety factor approach](image1)

![Global safety factor approach](image2)

Figure 5.7: Comparison safety factor for bearing failure for embedment range of case study 1

A similar trend as for the partial safety factor approach is seen in the right plot of figure 5.7 for the results of the analysis using the global safety factor approach. For an embedment depth from 0 to 0.6 meter a constant ratio of 1.7 is considered, while for larger embedment depths an increasing ratio is considered between the safety factor against bearing failure from the API design code and the PLAXIS model.

5.5.2. Sliding failure

For the investigation to the sliding failure the same foundation structure is used as for the investigation to the bearing failure in the PLAXIS model. The used length and width dimensions for the different aspect ratio’s are shown in table 5.6.

Aspect ratio

Figure 5.8 shows that the safety factor against sliding failure stays constant for a variation in aspect ratio for both the partial and global safety factor approaches either using the design codes or a PLAXIS model. A ratio of 1.7 is considered between the safety factor against sliding failure determined in the ISO design code and a PLAXIS model. For the comparison between the API design code and a PLAXIS model a ratio of 1.6 is considered.
5.5. Investigation results case study 1

**Embedment depth**

Figure 5.9 presents the results in terms of the safety factors against sliding failure for a range of embedment depths for case study 1. Both the partial and global safety factor approach show an increasing ratio between the design codes and the PLAXIS models. A ratio from 1.8 to 2.8 is considered for the comparison between the ISO design code and a PLAXIS model, while a ratio from 1.6 to 2.2 is considered between the API design code and a PLAXIS model. An explanation for the increasing ratio could be the assumption of superposition for the soil sliding capacity in the design codes as discussed in the analytical analysis of the design codes in section 3. This superposition deviates from reality and in the PLAXIS model the soil sliding capacity is determined more realistically.

**5.5.3. Safety foundation design**

In this section the safety factors from the comparison of the ISO and API design codes and the PLAXIS models are used for the actual foundation design of case study 1. In the actual foundation design an aspect ratio of 0.91 and an embedment depth of 0.3 meter are considered.
Safety foundation design according to API RP 2A-WSD

A foundation design is considered to be safe according to the API design code if the safety factor against bearing failure is minimum 2.0 and the safety factor against sliding failure is minimum 1.5. A safe foundation design is indicated in figure 5.10 by the green area and an unsafe foundation design by the red area.

In figure 5.10 is shown that the foundation design made by the API design code has a safety factor against bearing failure of $S.F. = 3.4$ and a safety factor of $S.F. = 18.8$ against sliding failure. This foundation design lies in the green area and the foundation design is considered to be safe according to the API design code. The determined safety factors from the PLAXIS model are for bearing failure $S.F. = 5.5$ and for sliding failure $S.F. = 28.9$. This result also lies in the green area and is also considered to be safe according to the API design code. A ratio of 1.6 is considered between the safety factor against bearing failure if the API design code and the PLAXIS model are compared and a factor of 1.5 between the safety factors against sliding failure.

From the considered ratio’s between the safety factors from the API design code and the PLAXIS model can be concluded that there is conservatism present in the used equations to calculate the safety factors against bearing failure and sliding failure as given in the API design code.

![Figure 5.10: Determination safety of foundation design according to API RP 2A-WSD design code for case study 1](image)

Safety foundation design according to ISO:19901-4

A foundation design is considered to be safe according to the ISO design code if the safety factors against bearing and sliding failure are equal or larger than 1.0. A safe foundation design is indicated in figure 5.11 by the green area and an unsafe foundation design by the red area.

The safety factor against bearing failure for the foundation design made by the ISO design code is $S.F. = 1.8$ and for sliding failure $S.F. = 6.9$. The safety factor against bearing failure determined by the PLAXIS model is $S.F. = 4.1$ and against sliding failure $S.F. = 12.5$. Both the foundation designs made according to the ISO design code and the PLAXIS model lie in the green area of figure 5.11 and are considered to be safe foundation designs. A ratio of 2 between the safety factor against bearing failure is considered when the ISO design code and the PLAXIS model are compared and a ratio of 1.8 is considered for sliding failure.

From the considered ratio’s between the safety factors from the ISO design code and the PLAXIS model can be concluded that there is conservatism present in the equations to calculate the safety factors against bearing failure and sliding failure provided in the ISO design code.
5.5.4. Overprediction PLAXIS results
The safety factor against bearing failure and sliding failure for the actual foundation design of case study 1 are determined by the API and ISO design codes and the PLAXIS models. It is shown that the safety factors determined by the PLAXIS models are higher compared to the safety factor values determined in the codes.

The ratio's between the safety factors determined in the design codes and the PLAXIS models are an over-estimation because the ultimate soil capacity for bearing and sliding is overpredicted in the PLAXIS models. This overprediction does not result into problems since the numerical results are more realistic while the overprediction is only little. This overestimation can be reduced by optimizing the PLAXIS models. In this study the optimization of the PLAXIS model is investigated by means of the mesh discretization error, modeling the soil-structure interaction and the determination of soil failure from PLAXIS results.

Mesh discretization error
The mesh discretization error is one of the three broad categories of errors in finite element analysis as found in the literature review in section 2. Due to the mesh discretization the continuous field is converted into a finite number of elements. The mesh discretization error is the discontinuity of the calculated result between different elements. Due to this mesh discretization error the ultimate capacity of the soil is overpredicted in a finite element analysis. By increasing the number of used elements the mesh discretization error decreases. In section 5.5.5 an analysis is performed to investigate the effect of an increasing number of used elements on the calculated ultimate soil capacity for case study 1.

Soil-structure interaction
The soil-structure interaction can be defined in a PLAXIS model by creating interface elements between the soil and the structure. The roughness of the interaction can be specified by the factor $R_{\text{inter}}$ in the PLAXIS model. By default this factor is set at a value of $R_{\text{inter}} = 1.0$. For this value the interface does not have a reduced strength compared to the surrounding soil. In this investigation the default value of $R_{\text{inter}} = 1.0$ is used since no detailed information was known. This default value is however an optimistic value which overpredicts the soil-structure interaction behavior. The effect of using a reduced interface strength could be analysed for the calculations of the safety factors in case study 1. This analysis is however not performed due to the limited amount of time available for this study.

Determination soil failure from PLAXIS results
In this study soil failure is defined when the $\Sigma M_{\text{stage}}$ value increase between two numerical steps becomes less than 0.0005. The used value is the result from an analysis, but no scientific background is found for this value. In section 5.5.6 an analysis to another method with a scientific background is performed.
5.5.5. Analysis mesh discretization

The mesh discretization error causes a small overprediction of the ultimate capacity of the soil in a finite element analysis. An indication of the accuracy of the solution can be seen in a convergence study where the number of elements in the mesh is increased. The result of multiple simulations with an increase in mesh elements is plotted in a graph the results should converge to a stable value. No industry hard rules are established to assess whether the used mesh discretization gives a result where the required accuracy is reached [71].

In PLAXIS it is possible to automatically generate a mesh. In this automatic option the mesh density can be very coarse, coarse, medium, fine and very fine. In this mesh discretization analysis all these automatically generated mesh densities are used in the PLAXIS model to calculate the safety factor against bearing failure and sliding failure.

Bearing failure

Figure 5.12 shows the results of the mesh discretization analysis for the safety factor against bearing failure of case study 1. The partial and global safety factor approach show a convergence of the results to a stable value for an increasing number of mesh elements. In this study a fine mesh density is used because of the required calculation time. The fine mesh is indicated in figure 5.12 by the circle with the x in it. The real safety factor against bearing failure is not known, however from the results of the mesh discretization analysis can be concluded that the result for the very fine mesh are approximately the real values for both the partial and global safety factor approach. A difference between the fine mesh as used in this study and the approximate real value found in the mesh discretization analysis can be calculated. The safety factor against bearing failure is overpredicted by 20% in the partial safety factor approach and by 8% in the global safety factor approach.

![Mesh discretization analysis for bearing failure of case study 1](image)

Figure 5.12: Mesh discretization analysis for bearing failure of case study 1

Sliding failure

Figure 5.13 shows the results of the analysis for the safety factor against sliding failure. The circle with the x in it represents the fine mesh which is used in this study because of the required calculation time. In the results from both the partial and global safety factor approaches is shown that the results converge to a stable value. The real safety factor against sliding failure for both the partial and global safety factor approaches is not known, however from the mesh discretization analysis can be concluded that the very fine mesh results are approximately the real values for both the partial and global safety factor approaches. The overprediction is the difference between the fine mesh as used in this study and the approximately real values. This overprediction is determined as 12% for the partial safety factor approach and 10% for the global safety factor approach.

Safety foundation design

Figure 5.14 presents the results from the mesh discretization analysis. In both the partial safety factor approach and the global safety factor approach is seen that the calculated safety factors against bearing and

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sliding failure decrease for an increasing mesh density in the PLAXIS model. This is seen in figure 5.14 by the south west movement of the results for an increasing mesh density.

From the results of the mesh discretization analysis can be concluded that the accuracy of the mesh for the calculation of the safety factor against bearing failure for the partial safety factor approach could be optimized since the result of the fine mesh, as used in this study, deviates 20% from the real safety factor value. The accuracy for the safety factor against bearing failure for the global safety factor approach and the safety factors against sliding failure for both partial and global safety factor approaches results are within approximately 10% of the real safety factors, which is an acceptable accuracy.
5.5.6. Determination soil failure from PLAXIS results

From theory is known that if soil failure occurs large deformations are seen while the load is not increased. For the used soil failure criteria as explained in section 5.4.4 which is used in this study no scientific background is found. An analysis for different situations is performed and for a value of 0.0005 the point of soil failure was always located at the right of the big bend as shown in the ΣMstage-u graph. However this approach overestimates the real soil capacity since the part of the graph right of the big bend is increasing due to the shape of the mesh elements, this behavior will not occur in reality.

Figure 5.15 presents an alternative method with a more scientific background which could be used to determine soil failure from the PLAXIS results. In the figure the same PLAXIS results are shown as in section 5.4.4, however the soil failure criteria is different. Both graphs show the same, only the right graph is zoomed in on the critical part of the graph. In the alternative method tangent lines for the elastic and the plastic part of the graph are constructed as shown by the black lines. The point where these tangent lines cross is the soil failure point from the theory for a linear elastic perfectly plastic soil. The right graph in figure 5.15 is zoomed in. The black dashed line shows the point from the PLAXIS results which has an equal ΣMstage value as where the 2 tangent lines cross. This point in the graph from PLAXIS results is the soil failure point. Results on the left are shown in green as the present no soil failure, and results on the right are shown in red as they present soil failure.

For this specific case soil failure occurs in the alternative method at a point with a ΣMstage value of 0.4755, while in the method used in this study a ΣMstage value of 0.488 was found for the point of soil failure. Table 5.7 presents the differences between this alternative method and the method as used in this investigation to determine the point of soil failure. From the differences is shown that there is no significant difference between the alternative method and the method as used in this investigation. The alternative method will give a slightly lower safety factor compared to the used method in this study.

<table>
<thead>
<tr>
<th>Method</th>
<th>ΣMstage value [-]</th>
<th>Ultimate soil load [kN]</th>
<th>Safety factor [-]</th>
<th>Ratio [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Used method</td>
<td>0.488</td>
<td>4846.1</td>
<td>5.46</td>
<td>1.83</td>
</tr>
<tr>
<td>Alternative method</td>
<td>0.4755</td>
<td>4844.7</td>
<td>5.35</td>
<td>1.79</td>
</tr>
</tbody>
</table>

Table 5.7: Comparison alternative method with used method for soil failure determination
5.5.7. Analysis foundation area
The influence of the foundation shape, in terms of the aspect ratio, is investigated in this thesis. The results indicate that the shape of the foundation has no influence on the ratio between the results from the design codes and a PLAXIS model. From this can be concluded that the foundation shape has no influence on the embedded conservatism in the design codes. In this section it is investigated whether the foundation size has an influence on the embedded conservatism in the design codes.

The foundation design of case study 1 is 0.3 meter embedded, 11 meter long, 10 meter wide and has an area of 110 square meter. In order to investigate the influence of the foundation area on the embedded conservatism in the design codes the foundation area is decreased in steps of 10%. Figure 5.16 presents the results of this analysis. These results show slightly increasing ratio’s for decreasing foundation area’s from which can be concluded that the foundation area has no significant influence on the embedded conservatism in the design codes. The increasing trend in the results can be explained by the 0.3 meter embedment for the foundation design of case study 1. If no embedment is present the ratio is constant for different foundation area’s.

5.6. Investigation results case study 2, 3 and 4
5.6.1. Bearing failure
For the results of the safety factors against bearing failure for case studies 2, 3 and 4 a reference is made to Appendix C. In this Appendix the results from the API and ISO design codes and the results from the PLAXIS models are compared and discussed.

5.6.2. Sliding failure
A reference is made to Appendix C for the results of the safety factors against sliding failure for case studies 2, 3 and 4. In this Appendix the results from the API and ISO design codes and the PLAXIS models are compared and discussed.

5.6.3. Safety foundation design
The results for the safety factors against bearing failure and sliding failure as shown in Appendix C are used to determine whether the actual foundation design of case studies 2, 3 and 4 are either safe or unsafe. Also is discussed in this section what ratio is found between the results of the safety factors found in the API and ISO design codes and a PLAXIS model for the actual foundation design of case studies 2, 3 and 4.
Case study 2
Figure 5.17 presents the determined safety factors against bearing failure and sliding failure for the partial and global safety factor approaches according to the design codes and a PLAXIS model for the actual design of case study 2. The actual foundation design of case study 2 consists of an aspect ratio of 0.97 and an embedment depth of 0.3 meter. In the partial safety factor approach the foundation design is not considered as a safe since the safety factor against bearing failure is 0.81, while a safety factor of 1.0 is required. In the global safety factor approach both the results from the API design code and the PLAXIS model are determined as a safe foundation design.

![Graph showing safety factors for case study 2](image)

In the partial safety factor approach a ratio of 4.1 is considered between the safety factors against bearing failure and a ratio of 2.1 between the safety factors against sliding failure. In the global safety factor approach a ratio of 1.8 is considered for the safety factors against bearing failure and a ratio of 1.5 between the safety factors against sliding failure. The ratio between the safety factors against bearing failure for the partial safety factor approach is much higher compared to the results from case study 1, but the other considered ratio’s correspond to the results from case study 1. An explanation for the significant different ratio of the partial safety factor approach for case study 2 is given in section 5.7.

Case study 3
Figure 5.18 presents the results for the actual foundation design of case study 3 for both the partial and global safety factor approach. The foundation design is considered to be safe according to both the API and ISO design codes. The actual foundation design of case study 3 consists of an aspect ratio of 0.77 and an embedment of 0.5 meter.

In the partial safety factor approach a ratio of 2.1 is considered between the results for the safety factors against bearing failure and a ratio of 1.9 for the results between the safety factors against sliding failure. In the global safety factor approach a ratio of 1.8 is considered between the results of the safety factors against bearing failure and a ratio of 1.6 between the results for the safety factors against sliding failure. The considered ratio’s correspond with the ratio’s from case studies 1 and 2. Also is indicated by the considered ratio’s that more conservatism is embedded in the ISO design code compared to the API design code.
5.6. Investigation results case study 2, 3 and 4

Figure 5.18: Determination safety of foundation design according to ISO and API design codes for case study 3

Case study 4

Figure 5.19 presents the results for the actual foundation design of case study 4 for both the partial and global safety factor approach. The actual foundation design of case study 4 consists of an aspect ratio of 0.77 and an embedment depth of 0.4 meter. In figure 5.19 is seen that the results from the design codes and the PLAXIS models are located in the green area and thus considered as safe foundation designs.

In the partial safety factor approach a ratio of 2.1 is considered between the results for the safety factors against bearing failure and a ratio of 1.6 is considered between the results for the safety factors against sliding failure. In the global safety factor approach a ratio of 1.8 is considered between the results for the safety factor against bearing failure and a ratio of 1.6 between the results for the safety factor against sliding failure. The considered ratio’s correspond to the results from case studies 1, 2 and 3. From the considered ratio’s it can be concluded that more conservatism is embedded in the ISO design code compared to the API design code.

Figure 5.19: Determination safety of foundation design according to ISO and API design codes for case study 4
5.7. Comparison ratio's

Ratio's between the results for the safety factors against bearing and sliding failure for the design codes and the PLAXIS models are determined for the actual foundation designs of the four case studies and for the sensitivity analysis for the range of aspect ratio's and embedment depths. The shown ratio's in this section are defined as the ratio of the safety factor against either bearing or sliding failure from a PLAXIS model over the safety factor against bearing or sliding failure from the design codes.

5.7.1. Bearing failure

Figure 5.20 presents the ratio's between the results of the safety factors against bearing failure for the four case studies. A constant ratio of 2.1 between the results for case studies 1, 3 and 4 is considered for the partial safety factor approach. The ratio for case study 2 deviates significantly from the other investigated case studies. The source for this deviating behavior is found and explained in section 5.7.3.

For the global safety factor approach a constant ratio of 1.9 between the results of the safety factors against bearing failure is considered. From the considered ratio's can be concluded that more conservatism is embedded in the ISO design code because the ratio's between the results for the partial safety factor approach are higher compared to the global safety factor approach.

![Graph](image.png)

Figure 5.20: Comparison ratio's between design codes and PLAXIS model for bearing failure of the actual foundation design for case studies 1, 2, 3 and 4

**Aspect ratio**

Figure 5.21 presents the ratio's between the results for a range of aspect ratio's of the four case studies. For the partial safety factor approach a constant ratio of 2 is considered between the results of case studies 1, 3 and 4 for the range of aspect ratio's. The ratio between the results for case study 2 deviates significantly due to the effect of the load inclination correction factor in the ISO design code calculation. In the global safety factor approach a constant ratio of 1.6 to 2.3 is considered between the results of the four case studies.
5.7. Comparison ratio's

Figure 5.21: Comparison found ratio's between design codes and PLAXIS model for bearing failure of aspect ratio range

**Embedment**

Figure 5.22 presents the ratio's between the results for a range of embedment depths for the four case studies. For the partial safety factor approach a constant ratio of 2.1 is considered for an embedment depth up to 0.6 meter for case studies 1, 3 and 4. For larger embedment depths the ratio between the results is increasing. The ratio for the results of case study 2 deviate significantly from the other case studies, this behavior can be explained by the influence of the load inclination correction factor in the ISO design code calculation.

For the global safety factor approach a relative constant ratio between the results is considered for an embedment depth from 0 to 0.6 meter. For larger embedment depths the ratio is increasing. The increasing behavior of the ratio between the results for embedment depths larger than 0.6 meter could be explained by the assumption of superposition in the design codes. Using superposition to model the soil behavior for cohesion and surcharge terms in the bearing capacity equation is not a good representation of the reality. This behavior is modeled more realistically in PLAXIS as shown by the increasing ratio between the results.
5.7.2. Sliding failure

Figure 5.23 presents the ratio between the results of the safety factors against sliding failure for the actual foundation designs of the four case studies. For the global safety factor approach a ratio of 1.6 is considered for the four investigated case studies. In the partial safety factor approach ratio's between 1.6 and 2.1 are seen in the plot. From the ratio's it can be concluded that more conservatism is embedded in the ISO design code for the determination of the safety factor against sliding failure because the considered ratio's in figure 5.23 are higher compared to the global safety factor approach.

![Graph showing ratios for sliding failure](image)

Figure 5.23: Comparison ratio's between design codes and PLAXIS model for sliding failure of the actual foundation design for case studies 1, 2, 3 and 4

Aspect ratio

Figure 5.24 presents the ratio's between the results for a range of aspect ratio's of the four case studies. In the partial safety factor approach ratio's between 1.5 and 1.9 are considered for case studies 1, 3 and 4. The ratio's between the results from case study 2 deviate significant from the ratio's found in the other three case studies.

![Graph showing ratios for aspect ratio](image)

Figure 5.24: Comparison found ratio's between design codes and PLAXIS model for sliding failure of aspect ratio range

In the global safety factor approach a constant ratio of 1.6 is considered for all case studies. From the results shown in the figure it can be concluded that the aspect ratio has no influence on the embedded conser-
vatism in the equations for the soil sliding capacity in the design codes. The ratio’s for the partial safety factor approach are higher compared to the global safety factor approach. This indicates that there is more conservatism embedded in the ISO design code.

**Embedment**

Figure 5.25 presents the ratio between the results for a range of embedment depths for the investigated case studies. In the partial safety factor approach a relative constant ratio between 1.7 and 2.2 is considered for an embedment depth less than 0.5 meter. For embedment depths larger than 0.5 meter the ratio between the results are increasing significantly. In the global safety factor approach a similar behavior is seen in the figure. For an embedment depth from 0 to 0.5 meter a constant ratio of 1.5 is considered and for larger embedment depths the ratio is increasing. A possible explanation for the increasing ratio from 0.5 meter embedment could be the assumption of superposition in the design codes which is not a good representation of the reality. In PLAXIS the embedment is modeled more realistic which is shown for larger embedment depths as shown from the results in figure 5.25.

![Figure 5.25: Comparison found ratio’s between design codes and PLAXIS model for sliding failure of embedment depth range](image)

### 5.7.3. Deviating results case study 2

The ratio’s between the safety factors against bearing failure as found in the PLAXIS model and ISO design code for case study 2 show significantly different values. The source for these different values are found in the load inclination correction factor.

In the equation for the calculation of the bearing capacity of the soil in the ISO design code a correction factor $K_c$ is used as shown in red in equation 5.11. This factor contains the correction factors for the foundation shape, foundation embedment and for the load inclination. This load inclination correction factor is shown in red in equation 5.12. In the analytical analysis in section 3 is presented that if there is no load inclination the load inclination correction factor $i_c$ should have no influence on the bearing capacity of the soil and the value should be zero.

$$Q_{ult} = \left[ F \cdot \left( N_c \cdot s_{u,0} + \frac{k \cdot B^2}{4} \right) \frac{K_c}{\gamma_m} + \gamma'D \right] A'$$  \hspace{1cm} (5.11)

$$K_c = 1 + s_c + d_c - i_c$$  \hspace{1cm} (5.12)

Table 5.8 presents the load inclination and the value of the load inclination correction factor for the four investigated case studies. From this table is seen that case study 1 and 3 have a similar load inclination and the values for the load inclination correction factors also are similar. For case study 2 and 4 the load inclination also is relative similar however the load inclination correction factors are significantly different. The source for the significantly difference in the load inclination correction factors for case studies 2 and 4 is found in the
equation for this factor as shown in equation 5.13. The equation does not contain the horizontal and vertical loads but the horizontal load and resistance against vertical loading of the soil. This resistance against vertical loading contains the length and width dimensions of the foundation design and the undrained shear strength of the soil. The foundation design of case study 2 has the smallest foundation area and the undrained shear strength of the soil is the smallest. This is the source for the large load inclination correction factor for case study 2 which has a significant influence on the bearing capacity of the soil.

\[
i_c = 0.5 - 0.5 \sqrt{1 - \left( \frac{H}{B'L s_{u,0} / \gamma_m} \right)} \tag{5.13}
\]

In order to investigate the influence of the load inclination correction factor \(i_c\) in the ISO design code for case study 2 equation 5.13 is replaced by the fixed value 0.119. This is the value for the load inclination correction factor of case study 4 which has a similar load inclination compared to case study 2. The result for the ratio between the safety factor from the ISO design code and the PLAXIS model for the foundation design of case study 2 for the adjusted load inclination correction factor is presented in figure 5.26 by the black mark. The ratio for case study 2 decreases to a value of 2.2 if the adjusted load inclination correction factor is used. This ratio is in the range of the results from the other investigated case studies.

<table>
<thead>
<tr>
<th>Load inclination [°]</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
<th>Case 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load inclination correction factor [−]</td>
<td>3.6</td>
<td>7.8</td>
<td>3.8</td>
<td>6.7</td>
</tr>
<tr>
<td>0.087</td>
<td>0.341</td>
<td>0.083</td>
<td>0.119</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.8: Comparison load angle and load angle correction factors for investigated case studies

Figure 5.26: Comparison ratio’s between design codes and PLAXIS model for bearing failure of the actual foundation design for case studies 1, 2, 3 and 4 with an adjusted partial safety factor approach for case study 2
The results for the ratio for case study 2 with an adjusted load inclination correction factor are presented in figure 5.27 for both a range of aspect ratio's and embedment depths. In the plots is seen that the ratio for case study 2 for a range of aspect ratio's decreases to a similar ratio as the other investigated case studies. For embedment depths between 0 and 0.6 meter the ratio as found with the adjusted load inclination correction factor for case study 2 decreases to similar values as found for the other three investigated case studies.

Figure 5.27: Comparison ratio's between ISO design code and PLAXIS model for bearing failure of the actual foundation design for case studies 1, 2, 3 and 4 with an adjusted case study 2.
6 Cost reduction example

The results found in this study indicate that there is embedded conservatism in the design codes as used by Allseas for the foundation design of subsea structures. As a result of this embedded conservatism the foundation design can be optimized for future projects which is beneficial for the economical and operational aspects. The main goal is to reduce the costs by optimizing the foundation design by implementing the results from this study. In order to give a first approximation regarding the possible cost reduction for future projects case study 1 is used as an example. Figure 6.1 presents a visualisation of case study 1.

![Figure 6.1: PLET structure case study 1 as used in the cost reduction calculation, figure after [1]](image)

The structure from case study 1 consists of two main elements; piping and structural parts. The piping consists of pipelines, valves and connectors. The piping is shown in figure 6.2a as the grey structures. The costs of the piping are mainly dependent on the required type and number of welds. The costs for the fabrication of the piping are not influenced by the possible embedded conservatism in the foundation design. The structural parts of the PLET are shown in figure 6.2b in yellow. The structural parts consist of the mudmat, yoke, piping frame and piggy back supports. The costs for the fabrication of the structural parts are determined by unit rates for steel structures. These unit rates do change in time due to steel prices, the complexity of the mudmat and the market situation. For this example calculation an unit rate of €7.50 is used to determine the fabrication costs.
By optimizing the foundation design regarding the embedded conservatims as found in this study only the mudmat dimensions can be reduced. The other structural steel elements in the PLET are not affected by the embedded conservatism in the investigated design codes. Figure 6.3a presents the mudmat in yellow and the other structural steel elements in blue. Figure 6.3b presents the mudmat of the PLET structure which is investigated in this study.

The foundation design for case study 1 according to the API design code consists of a 10 meter wide, 11 meter long and 0.3 meter embedded foundation. The mudmat weight is 19.5 t and the weight of the other structural steel is also 19.5 t. The fabrication costs for all structural steel is €292,500. The costs for the piping are estimated by engineering experience as 33% of the total fabrication costs, or €146,250. The total fabrication costs for the PLET structure of case study 1 are €438,750.

The foundation design for case study 1 is also determined by a PLAXIS model. The embedment is kept at 0.3 meter and only the length and width dimensions are reduced until the same safety factor against bearing failure as compared to the API design code is found. The safety factor against bearing failure is used instead of the safety factor against sliding failure because it is the governing failure mode. A 7.29 meter wide, 8.02 meter long and 0.3 meter embedded foundation design is found by the PLAXIS model. This foundation design has a foundation area of 58.47 square meter which is a reduction of 47% compared to the 110 square meter of the actual foundation design. In this example a reduction of 47% of the mudmat weight is assumed since the embedment depth is the same, however a more detailed investigating should be performed to asses the exact weight reduction due to the reduced mudmat dimensions. In this study this detailed investigation is not performed since the structural and installation aspects should be taken into account and these are not within the scope of this study. The costs to fabricate the mudmat from the foundation design as determined by PLAXIS are €77,659 which is a reduction of €68,591. The real reduction in fabrication costs is 16% since the piping and other structural steel will not reduce significantly due to the reduced foundation area.
In order to set the cost reduction for the structure fabrication in perspective, they should be compared to the operational installation costs. The investigated PLET structures are installed on the seabed by Allseas installation vessels which have day rates in the order of €250,000 to €500,000 depending on e.g. the type of work and the market situation. If the size of the foundation for the PLET structure can be reduced, a possible larger time window for the installation is created. In the installation operation an even larger cost reduction can be achieved, compared to the fabrication costs, if the foundation design of the PLET structures can be optimized.
Conclusions

The conclusions drawn based on the analysis performed during this study to investigate the possible embedded conservatism in the ISO:19901-4 and API RP 2A-WSD design codes are presented by answering the research questions formulated in the introduction of this thesis.

**Are the design codes conservative?**
The embedded conservatism in the ISO and API design codes in this study is investigated by comparing the analytical design codes and a numerical finite element program PLAXIS. In this study the safety factors against bearing failure and sliding failure according to the design codes and PLAXIS models are compared and the ratio between them is determined. Analysis results show that the ratio between the safety factors against bearing and sliding failure based on the design codes and PLAXIS models lay in the range of 1.5 and 2.3. Since for given soil and load conditions PLAXIS results being minimal 1.5 times higher, this is a significant indication of embedded conservatism in the investigated design codes. The exact embedded conservatism in the design codes is slightly lower than the ratio as found in this study since there always is some small overestimation for the ultimate soil capacity in a finite element analysis.

**Why are the design codes conservative?**
In this study the method of determining the safety factors against bearing and sliding failure in the PLAXIS models is based on the calculation methods prescribed by the design codes. All the other limitations to the use of the design codes also are applied to the PLAXIS model. Therefore only the difference in calculation between the analytical design codes and the numerical program PLAXIS can have an influence on the embedded conservatism in the design codes. In the performed literature review three possible sources of conservatism are found; the assumption of superposition, the absence of an interdependence of V-H-M loading and the limitations in situation for which the design codes are derived.

The assumption of superposition is used by Terzaghi to calculate the bearing capacity of the soil. In the equation derived by Terzaghi the cohesion term, the surcharge term and the soil self weight term are summed by superposition. This summation is not a correct representation of the reality and leads to a conservative approach in the calculation method. In this study only the cohesion term and the surcharge term are summed as only embedded foundations in a clay soil are investigated.

There is an absence of an interdependence between vertical, horizontal and moment loading in both the ISO and API design codes. This is not a correct representation of the reality because there is an interdependence between these different loads and moments. Hence, negligence of lack of interdependence is another reason for conservatism introduced in the calculation method.

The bearing capacity equations in the API and ISO design codes are based on the equations from Brinch Hansen and Vesic. These equations are extended versions from the original bearing capacity equation from Terzaghi. Terzaghi’s equation is derived for a non embedded, infinite long strip foundation, loaded centrical by a perfectly vertical load on a single soil layer. Brinch Hansen and Vesic have modified Terzaghi’s equation by introduction of correction factors in order to incorporate effects of different foundation shapes, em-
bedment and inclined and eccentric loading to represent the general situations more realistic. However the modified equation from Vesic and Brinch Hansen still oversimplify the reality and are only valid for relative simple situations. Design codes are based on these equations and due to the oversimplification embedded conservatism or optimism could be present in the design codes.

**What are the factors in the design codes that influence the design of shallow foundations?**

Factors that influence the foundation design are found in the equations prescribed in the design codes. These factors are found in soil parameters, geometrical structure parameters and loading conditions. For the soil parameters in undrained conditions the undrained shear strength $s_u$, undrained shear strength increase $k$, unit weight of soil $\gamma$ are considered to have an effect on the foundation design. The geometrical structure parameters length $L$, width $B$ and embedment depth $D$ also have an effect on the foundation design. The loading conditions from horizontal loading $H$, vertical loading $V$ and moment loading $M$ have an influence on the foundation design of shallow foundations.

**What is the effect of the factors in the design codes on the design of a shallow foundation?**

The effect in terms of embedded conservatism from the geometrical structure parameters is determined from the analysis results in this study. The aspect ratio, which is the ratio of width over length, has no influence on the ratio between the results from the design codes and the PLAXIS models, and therefore no influence on the embedded conservatism. The foundation area also has no influence on the embedded conservatism in the design codes. The embedment is the only investigated geometrical structure parameter which has an effect on the embedded conservatism in the design codes. From the results in the analysis is shown that for an embedment larger than approximately 0.5 a 0.6 meter the ratio between the results from the PLAXIS model and the design codes increase significantly for the investigated structures.

No sensitivity analyses are performed to the effect of soil parameters and loading conditions on the possible embedded conservatism in the design codes. Therefore no conclusions can be drawn about their influence on the embedded conservatism in the design codes.

**Which design code results in a more economical design under specific conditions?**

In this investigation two design approaches are considered; the partial safety factor approach and the global safety factor approach. In the partial safety factor approach the ISO design code is compared to a PLAXIS model which also uses a partial safety factor approach and in the global safety factor approach the API design code is compared to a PLAXIS model which uses a global safety factor approach. The conservatism in the design codes is determined as the ratio's between the calculated safety factors in the ISO and API design codes and the safety factors calculated in the two PLAXIS models. The results of this study show that for the investigated situations the ratio's between the safety factors from the ISO design code and a PLAXIS model are higher compared to the ratio's between the safety factors from the API design code and a PLAXIS model. The higher ratio between the ISO results and the results from the partial safety factor approach PLAXIS model indicate the presence of more embedded conservatism in the ISO design code compared to the API design code, where the ISO would give a less economical design. However no conclusions regarding the comparison between the embedded conservatism in two investigated design codes can be drawn because of the different safety approaches as used in the design codes no fair comparison can be made.

A note to the comparison between the two design codes is that in the performed site investigation for the investigated case studies an increasing undrained shear strength profile is found. It is not possible to use this increasing shear strength profile in the API design code. An additional calculation step is performed in Appendix B in order to convert the increasing shear strength profile to a constant shear strength profile. Two different methods are used for this additional calculation step and they show comparable results. Therefore is assumed that the influence from this additional calculation step has no significant influence on the embedded conservatism in the design codes.

**What are the differences between a foundation design made by using the design codes or using a numerical analysis?**

In this study a numerical analysis using the finite element program PLAXIS is performed with multiple limitations. The used PLAXIS models are constructed such that they only calculate the safety factor against bearing failure and sliding failure similar to the approach as provided by the investigated design codes. Only these
two failure modes are considered in the investigated design codes. By using PLAXIS it is possible to perform more and more advanced analyses, however these are not performed in this study in order to make a fair comparison with the design codes. Only the difference in the used calculation method is investigated in this study. An example of a more advanced analysis performed by PLAXIS is using an effective stress approach instead of the total stress approach as used in this study. The total stress approach could give more conservative results compared to the effective stress method, but since the effective stress method cannot be used in analytical methods this method is not used in the investigated design codes. Another example of a more advanced analysis performed by PLAXIS is the use of more advanced soil failure models instead of using the Mohr-Coulomb soil failure model which is a first order approximation for soil behavior and provided in the investigated design codes.

The design codes approach the result from the conservative side so the results are always on the safe side. The finite element method program PLAXIS approaches the result from the optimistic side since there is some small overprediction of the ultimate soil capacity. This overprediction is due to a mesh discretization error, a non optimized soil structure interaction and determination of soil failure from the PLAXIS results. This overprediction in the finite element analysis results is however small, but the results are more realistic compared to the results from the calculation method as provided in the investigated design codes.
Recommendations

8.1. Recommendations on how to deal with conservatism in the codes

Practical guidelines for foundation design

Some guidelines for a foundation design can be given from the results of this study. For an embedment of more than 0.5 meter for a foundation design the increase of embedded conservatism in the design codes should be taken into account in the design process. Changing the aspect ratio of a foundation design has no influence on the embedded conservatism and can thus not reduce the embedded conservatism in the foundation design.

Proof of foundation design safety

A designer needs to prove to a client that the foundation design is safe. The design codes are made to optimize this process, as the committee which has made the design codes already proved that a design according to the design codes is a safe design. However the results of this study has shown that there is some embedded overconservatism in the design codes.

A practical example of when the results from this study could be used. At the start of a design process not all the loads acting on a foundation, soil parameter values and the structure geometry parameter values are exactly known so assumptions have to be made. If the loads, soil parameter values and the structure geometry of the foundation design are known later in the design process it could be possible that the foundation design would be unsafe according to the design codes. With the information from this study regarding the embedded conservatism in the investigated design codes there is some space for a discussion with the client about the safety of the foundation design. For a foundation design which falls just below the boundary of a safe design and is therefore considered as an unsafe foundation design, a discussion with the client could be held in order to convince the client that the foundation design is in reality a safe design due to the embedded conservatism in the design codes. By convincing the client time and money could be saved which otherwise had to be spend on making a new foundation design.

Required undrained shear strength profile in the design codes

From the site investigation review an increasing undrained shear strength is found for the four investigated case studies. It is not possible to use this increasing undrained shear strength profile in the API design code because a constant undrained shear strength profile is required. It is recommended to use a design code which is capable of using the found undrained shear strength profile from the performed site investigation results if no advanced program such as PLAXIS is available. Using a design code capable of modeling the shear strength profile as found in a site investigation provides a better representation of the reality since no additional calculation step is required in order to convert the shear strength profile. The PLAXIS program, as used in this study to convert the shear strength profile, gives accurate results but if no advanced program is available to convert the shear strength profile additional uncertainty is introduced in the calculation. In this study an analytical method to convert the shear strength profile is investigated and the results deviated significantly compared to the results found in the PLAXIS program.
Provided soil parameters by client
In the four investigated case studies in this study the site investigation has been performed by an independent company commissioned by the client. The client hereafter requested a foundation design according to a certain design code. Only the required soil parameters for the requested design code are provided to Allseas. For a numerical analysis with advanced soil failure models more soil parameters are required which are not available. Either a simple numerical analysis is performed or an advanced numerical analysis with more advanced soil failure models but with input information with a larger uncertainty. In order to optimize the foundation design a discussion with the client after the initial desk study would be recommended in order to determine which soil parameters are required for an optimized numerical analysis later in the design process. From a discussion with the client an optimized site investigation program can be designed.

Use of design codes is quick and easy
The use of the ISO and API design codes is relative quick and easy because of the simplifications in the design codes. Due to these simplifications the use of the design codes should only be considered in relatively simple foundation designs and because of the simplifications the calculations are relatively simple and fast. The use of design codes is a good option for a preliminary design, but for the final foundation design or a difficult foundation design a more advanced analysis should be utilized in order to optimize the foundation design. The result of the analysis in this study shows that the foundation design could be optimized by using programs capable of performing numerical analysis.

8.2. Recommendations for further research

Investigate more case studies
In this study only four case studies are investigated. If the foundation design of the four case studies is compared only little differences are seen. A recommendation for further research is to extend this investigation and include more case studies to investigate whether the results are also valid for a broader range of subsea structure foundation designs.

Investigate conservatism regarding deformation of foundation design
Only the soil strength is investigated in this study due to the limited available information regarding the soil deformation parameters. For further research a study to the conservatism regarding the deformation in the design codes is suggested because for subsea pipeline related structures the deformation of the soil can be the governing criteria.

Investigate influence of more advanced soil failure models
The Mohr-Coulomb failure model is in this investigation the used soil failure model due to the limited availability of soil parameters. For more advanced soil failure models more soil parameters are required which are not provided by the client. Further research is recommended to investigate, for the investigated types of subsea structures, the influence of using more advanced soil failure models with input parameters which have a larger uncertainty compared to the simple Mohr-Coulomb failure model with input parameter which have less uncertainty. The missing soil parameters could be derived from correlations or an engineering experience. The uncertainty of the missing soil parameters is larger but the results from the more advanced soil failure models could give an optimized design compared to the use of the simple Mohr-Coulomb failure model.

Compare foundation design by design codes by PLAXIS without limitations
In this investigation not all the capabilities of PLAXIS are used, only the safety factor against bearing failure and sliding failure are investigated in PLAXIS. It is recommended to perform a study which compares a foundation design according to the design codes and a foundation design made by PLAXIS without limiting the use of the program in order to make a fair comparison as is done in this study.

Investigate API RP 2GEO
The API RP 2A-WSD design code is published in 2002 and the ISO:19901-4 design code in 2003. The API RP 2GEO design code is combination of the API RP 2A-WSD and ISO:19901-4 design code, it is published in 2011 and updated in 2014. It is recommended to extend this study and investigate the conservatism in the API RP 2GEO design code, which applies a global safety factor approach and has the capability to model an increas-
8. Recommendations

In the investigated ISO and API design codes only simple foundation shapes are considered. The foundation design of in-line structures installed in s-mode consist usually of multiple footings instead of one footing. In the design codes no guidelines on how to calculate the safety of such a foundation design is offered. It is useful to develop guidelines which can be used in addition to the design codes for a foundation design consisting of multiple footings.
Soil data case studies

In this appendix soil data from the site investigation for case studies 1 and 2 is shown in two tables. Table A.1 presents the soil locations, waterdepth at the soil locations, penetration of the boreholes at the soil locations and recovery of the boreholes at the soil locations.

The distances between the different structures of case studies 1 and 2 and the soil locations of the site investigation are presented in table A.2.
<table>
<thead>
<tr>
<th>Hydrocarbon field</th>
<th>Water depth [m]</th>
<th>Penetration [m]</th>
<th>Recovery [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil location 1</td>
<td>1,674</td>
<td>6.00</td>
<td>4.50</td>
</tr>
<tr>
<td>Soil location 2</td>
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<td>6.00</td>
<td>4.19</td>
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Table A.1: Site investigation data
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Table A.2: Distance locations subsea structures compared to locations site investigation
Constant shear strength determination

The engineering soil parameter values for four case studies are determined in section 4. The undrained shear strength is determined as a linear increasing profile for the four investigated case studies. This linear increasing profile can be used in the FEM program PLAXIS and the ISO 19901-4 design code, but the API RP 2A-WSD design code requires a constant shear strength value over the depth of the soil. In this appendix three methods to determine a constant shear strength profile are discussed and utilized. Hereafter the results of the methods are compared and finally a constant shear strength profile for the four case studies is determined.

B.1. Used methods
B.1.1. The 2:1 approximation method
A simple method to determine the increase of vertical stress in depth without the use of advanced computer programs is the 2:1 approximation method. This method is described in [13] and [22] and the basic principle of the method is presented in figure B.1. The method assumes a stress dissipation with depth in the form of a trapezoid with inclined sides under an angle 2:1. The purpose of this method is to approximate the actual “pressure bulb” stress increase beneath a footing. The advantage of the 2:1 approximation method is the simplicity and the main disadvantage of the method is the assumption that the stress beneath the center and the edge of the footing are equal, while the stress underneath the edge of the footing in reality is lower compared to the center of the footing.

Figure B.1: The 2:1 approximation method for stresses in the soil after [87]

Figure B.1 shows that the stresses in the soil due to load $q$ from a structure, decrease in depth due to the increasing influenced area of soil. The stresses due to the self weight of the soil are increasing in depth. The distribution of stresses in the soil due to the loading from the a structure and the soil self weight are shown in figure B.2.

The constant shear strength profile is assumed to be the average value of the increasing shear strength profile over a certain soil depth. This soil depth is determined from the two graphs as shown in figure B.2. If the stresses due to the structure load and the soil self weight are summed it is possible to investigate until which depth the structure load has a significant effect on the total stresses in the soil. In this study a value of 25% of the total stresses is used to define when the structure load has no longer a significant effect on the total
stresses in the soil. The 25% value is found from an analysis where this method was compared to the other 2 used methods which use an advanced computer program to determine the constant shear strength profile in the soil. Hereafter the constant shear strength profile is defined as the average of the increasing shear strength profile between the seabed level and the depth where the subsea structure load no longer has a significant effect on the total stresses in the soil.

![Stress graph](image)

Figure B.2: Stress in soil due to soil self weight in left figure and stress in soil due to vertical load of structure in right figure.

An important note to the 2:1 approximation method is that the method does not take into account any information regarding strength inhomogeneity. The method is derived from homogeneous soil cases, and originally not intended to convert an increasing shear strength profile to a constant shear strength profile.

### B.1.2. PLAXIS soil reaction force method

The second method to determine the constant shear strength profile uses the finite element program PLAXIS. A shallow foundation is modelled on a soil with an increasing shear strength profile as determined in the site investigation review. A very stiff shallow foundation is modelled by applying a prediscribed vertical displacement. From the soil reaction force $F_y$ and the soil displacement $u$ underneath the center of the foundation a $F_y - u$ graph is constructed. For the applied vertical displacement a displacement of 4 meter is used, since for this value the soil will fail and a clear limit value for the soil reaction force is shown in the $F_y - u$ graph. Hereafter the shallow foundation is modelled again in a soil with a constant shear strength profile and the same vertical displacement of 4 meter is applied. A $F_y - u$ graph is made for soil masses with different constant shear strength values and these graphs are compared to the $F_y - u$ graph from the soil with an increasing shear strength profile.

![Mesh diagram](image)

Figure B.3: Initial and deformed mesh after applied vertical displacement used in the PLAXIS soil reaction force method

The initial mesh and the deformed mesh after the applied displacement are presented in figure B.3. The applied vertical displacement is represented in the figures as the purple arrows.
B.1.3. PLAXIS failure depth method
The FEM program PLAXIS is also used in the third method. In the PLAXIS soil reaction force method only the soil reaction force is investigated. In this method, the PLAXIS failure depth method, the failure depth in the soil due to an applied vertical displacement is also investigated. For this method a soil mass with a constant shear strength profile, determined from the second method, the PLAXIS soil reaction force method, an a soil mass with an increasing shear strength profile as determined in the site investigation review in section 4 are used. The failure mechanisms in the two soil masses are compared and for the increasing shear strength profile the failure depth is determined. This depth is hereafter used to determine a constant shear strength profile by calculating the a constant shear strength profile as the average of the increasing shear strength profile between the seabed level and the failure depth.

B.1.4. Determination constant $s_u$ profile
The three methods to determine the constant shear strength profile give for soils with low shear strength increase values relative similar results. If the shear strength increase in depth becomes larger the difference between the first method which does not use the PLAXIS program and the second and third methods which do use the PLAXIS program, becomes significant larger. The constant shear strength profile as used in the API RP 2A-WSD design code is defined as the average of the 'PLAXIS soil reaction force method' and the 'PLAXIS failure depth method' because both the methods take strength inhomogeneity into account. Since the 2:1 approximation method does not take strength inhomogeneity into account this method is not used to determine the constant shear strength profile. The results of the method is however compared with the other 2 methods in order to show how accurate the result is if no advanced FEM program is available and only the simple 2:1 approximation method would be used to determine a constant shear strength profile.

B.2. Case study 1
The three methods as discussed before are used to determine a constant shear strength profile for case study 1. Hereafter the results from the three methods are compared and a constant shear strength profile is determined. The increasing shear strength profile for case study 1 is determined in section 4 and can be described by: $s_u = 3 + 2.1 \cdot d$.

![Figure B.4: Influence vertical foundation load on stress in soil according to 2:1 approximation method for case 1](image)

B.2.1. The 2:1 approach method - Case 1
In the literature is found that at a depth of $4 \cdot B$ all the stresses in the soil due to a shallow strip foundation are dissipated and no more settlement occur at larger depths. The width of the shallow foundation in case study 1 is 10 meter, so the stress distribution in the soil to 40 meter depths is investigated. In the left graph of figure B.4 is shown what the percentage of stresses in the soil due to the vertical subsea structure load is compared to the summed total stress in the soil from the soil self weight and the vertical structure load until 40 meter depth. The right graph in figure B.4 presents the influence in the upper 10 meter soil, in the figure is seen that below 2.8 meter depth the vertical structure loading contributes less than 25% to the total stresses in the soil.
The constant shear strength profile, according to the 2:1 approximation method, is determined in figure B.5. The black dashed line at 2.8 meter depth shows the influence depth until where the structure vertical load has a significant influence on the total stresses in the soil. The red line represents the determined increasing shear strength profile and the green line is the determined constant shear strength profile determined from the 2:1 approximation method. This determined constant shear strength profile has a value of $s_u = 5.9 \text{kPa}$.

![Figure B.5: Determination constant shear strength value according to 2:1 approximation method for case 1](image)

**B.2.2. Plaxis soil reaction force method - Case 1**

A shallow foundation of 10 meter wide is modelled in PLAXIS on a soil with an increasing shear strength profile of $s_u = 3 + 2.1 \cdot d$. The soil reaction force for the shallow foundation is 351 kN. Hereafter a constant shear strength profile is modelled in the soil with different shear strength values ranging from $s_u = 1 \text{kPa}$ to $s_u = 15 \text{kPa}$. The modelled results are shown in figure B.6. A soil with a constant shear strength profile of $s_u = 6.7 \text{kPa}$ has an equal bearing capacity compared to the soil with an increasing shear strength profile.

![Figure B.6: $F_y - u$ plot for different shear strength profiles for case study 1](image)
B.2.3. PLAXIS failure depth method - Case 1
For this method the same PLAXIS model is used as for the second method to determine the constant shear strength profile. The applied displacement representing the stiff shallow foundation is applied from 30 meter to 40 meter and indicated in figures B.7 to B.9. The failure mechanism in the soil with a constant shear strength profile of $s_u = 6.7$ kPa is presented in figure B.7, the general shear failure mechanism is clearly shown in this figure. The failure mechanism in the soil with an increasing shear strength profile is shown in figure B.8. In this figure no clear failure mechanism is shown, however if the scale bars from figure B.7 and B.8 are compared it is shown that they contain different deformation ranges. If the scale bar for the soil with an increasing shear strength profile is adjusted to the same range as for the soil with a constant shear strength profile as seen in figure B.7, also a general shear failure mechanism is seen in figure B.9.

![Figure B.7: Shading plot for constant shear strength profile with $s_u = 6.7$ kPa](image1)

![Figure B.8: Shading plot for increasing $s_u$ profile](image2)
If the failure mechanisms as shown in figures B.7 for the constant shear strength profile and figure B.9 for the increasing shear strength profile are compared, different failure mechanisms are seen. The failure mechanism for the soil with a constant shear strength profile reaches deeper in the soil and is much wider compared to the failure mechanism of the soil with an increasing shear strength profile. The shallower failure mechanism for the soil with an increasing shear strength profile seems correct since the undrained shear strength in the increasing shear strength profile is already higher than the constant shear strength profile at 1.8 meter depth. The failure mechanism in figure B.7 for the constant shear strength profile reaches approximately 7 meter deep and for the increasing shear strength profile in figure B.9 approximately 2.7 meter depth.

The found failure mechanism depth of 2.7 meter for the increasing shear strength level is used to determine the constant shear strength value. Figure B.10 presents the soil test results for the undrained shear strength of case 1 in black marks. The increasing shear strength profile as determined in section 4 is shown as the red line. The dashed black horizontal line is the failure depth of 2.7 meter found in this method. The average of the increasing shear strength profile over 2.7 meter depth is $s_u = 5.8$ kPa. The constant shear strength profile of $s_u = 5.8$ is shown as the blue line in the figure.
B.2.4. Constant $s_u$ determination

In order to determine a constant shear strength profile for case study 1 for the API RP 2A-WSD design codes three different methods are used. The result from the first method is highly dependent on two assumptions. The assumption that the stresses are dissipating under a constant angle of 2:1 in depth is only valid for shallow depths and the assumption of the 25% influence level below which the influence on the total stresses is not significant, is very dependent on engineering judgement. The 25% influence level is however investigated for different situations and the 25% seems to give a realistic assumption for the influence depth of 2.8 meter which is close to the failure depth determined by PLAXIS. The found constant shear strength value from the 2:1 approximation method is $s_u = 5.9$ kPa. In the second method, the PLAXIS soil reaction force method, only the soil reaction force is investigated and the developed failure mechanism is not studied. The found constant shear strength profile from the PLAXIS soil reaction force method has a value of $s_u = 6.7$ kPa. In the third method is the developed failure mechanism investigated and the failure depth is determined as 2.7 meter. The constant shear strength profile corresponding to that failure depth has a value of $s_u = 5.8$ kPa.

The results from the three used methods to determine the constant shear strength profile for case study 1 are shown in figure B.11 and the results of the three methods are relative close to each other. The determined constant shear strength profile for case study 1 is $s_u = 6.1$ kPa, which is the average of $s_u = 6.7$ kPa and $s_u = 5.8$ kPa.

![Figure B.11: Determination constant shear strength value for case 1](image-url)
## B.3. Case study 2

In this section the three discussed methods are utilized to determine a constant shear strength profile for case study 2. Hereafter the results of the three methods are compared and a constant shear strength value is determined which is used in the API RP 2A-WSD design code. The increasing shear strength profile is already determined in section 4 and can be described by: \( s_u = 1.35 + 2.8 \cdot d \).

### B.3.1. The 2:1 approach method - Case 2

The width of the shallow foundation of case study 2 is 8.6 meter and from the literature is known that for a shallow strip foundation all the stresses in the soil are dissipated at a depth of \( 4 \cdot B \). So in this case study the soil is investigated to 35 meter depth. The percentage of stresses in the soil due to the vertical structure load compared to the combined soil self weight stress and the stress due to the vertical structure load is shown in figure B.12. In the left figure the influence to a depth of 35 meter is shown and on the right only the upper 10 meter is shown. In the right figure also is shown that at a depth of 4.3 meter the influence from the structure vertical loading is less than 25%.

![Figure B.12: Influence vertical foundation load on stress in soil for case 2](image1.jpg)

![Figure B.13: Determination constant shear strength value with 2:1 approximation method for case 2](image2.jpg)
The constant shear strength profile according to the 2:1 approximation method is determined in figure B.13. The constant shear strength value is determined as the average of the increasing shear strength profile between the seabed level and the influence depth determined in figure B.13. The influence depth was determined at 4.3 meter depth, however no information regarding the undrained shear strength is known below 4 meter depth. Therefore an influence depth of 4 meter is used to determined the constant shear strength profile in this method. The constant shear strength value is determined as $s_u = 7$ kPa.

B.3.2. Plaxis soil reaction force method - Case 2
A shallow foundation of 8.6 meter wide is modelled in PLAXIS on a soil with an increasing shear strength profile of $s_u = 1.35 + 2.8 \cdot d$. A soil vertical reaction force of 203 kN is found. Hereafter a constant shear strength profile is modelled in the soil with different shear strength values ranging from $s_u = 1$ kPa to $s_u = 15$ kPa. The constant shear strength profile of $s_u = 4.5$ kPa has a similar vertical soil reaction force as the increasing shear strength profile. All the results of the modelling are shown in figure B.14.

![Figure B.14: $F_y - u$ plot for different shear strength profiles for case study 2](image)

B.3.3. PLAXIS failure depth method - Case 2
For the third method, the PLAXIS failure depth method, the same PLAXIS model is used as for the second PLAXIS soil reaction force method. A foundation of 8.6 meter wide is modeled in PLAXIS from 25.8 meter to 34.4 meter as shown in figures B.15 to B.17. The failure mechanism in the soil with a constant shear strength value of $s_u = 4.5$ kPa is shown in figure B.15. In this figure the general shear failure mechanism is clearly shown. The failure mechanism in the soil with an increasing shear strength value is shown in figure B.16. In this figure the failure mechanism is not clearly shown. A comparison between figures B.15 and B.16 also shows a clear difference between the scale bars. In figure B.17 the scale bar is adjusted to the same scale as for figure B.15 and a general shear failure mechanism is seen. If the failure mechanisms of figures B.15 and B.17 are compared, it is seen that there are some large differences between them.

In figure B.15 the soil has a constant shear strength profile and the failure depth of the failure mechanism is 6 meter. The failure mechanism shown in figure B.17 for the increasing shear strength profile is seen at 1.8 meter depth.

The failure depth of 1.8 meter as shown in figure B.17 is used in figure B.18 in order to determine the constant shear strength profile with the third method, the PLAXIS failure depth method. The constant shear strength value is determined as the average value of the increasing shear strength profile between the seabed level and the dashed line which represents the failure depth at 1.8 meter depth. The constant shear strength is determined as $s_u = 3.9$ kPa.
Figure B.15: Shading plot for constant shear strength profile with \( s_u = 4.5 \) kPa

Figure B.16: Shading plot for increasing \( s_u \) profile

Figure B.17: Shading plot for increasing \( s_u \) profile with adjusted scale equal to figure B.15
**B.3.4. Constant $s_u$ determination**

In order to determine a constant shear strength profile for case study 2 for the API RP 2A-WSD design code three different methods are used. The result from the first method, the 2:1 approximation method, is highly dependent on two assumptions. The first assumption is the angle of 2:1 in which the stresses dissipate in the soil and the second assumption is the 25% influence level. The 2:1 angle is a common used value for hand calculations and the 25% is an engineering judgement which is investigated for different situations. It seems to be a realistic assumption in most situations, however for case study 2 there is a relative large difference if the influence depth of 4.3 meter found in the 2:1 approximation method and the failure depth of 1.8 meter found in the PLAXIS failure depth method are compared. The result of the 2:1 approximation method is a constant shear strength value of $s_u = 7$ kPa. In the second method, the PLAXIS soil reaction force method, only the vertical soil reaction forces are investigated and a shear strength profile of $s_u = 4.5$ kPa gives a similar vertical soil reaction force compared to the increasing shear strength profile under an applied displacement of 4 meter. In the PLAXIS failure depth method the failure depth in the soil is investigated with PLAXIS. The increasing shear strength profile has a failure depth of 1.8 meter from which a constant shear strength profile of $s_u = 3.9$ kPa is determined. The results from the three methods to determine a constant shear strength profile are shown in figure B.19. It is shown that the 2:1 approximation method results in a higher constant shear strength profile compared to the PLAXIS soil reaction force method and the PLAXIS failure depth method. This higher value can be explained due to the higher shear strength increase value of $k = 2.8$ kPa/m.

The determined constant shear strength profile has a value of $s_u = 4.2$ kPa. This is the average of the PLAXIS soil reaction force method and the PLAXIS failure depth method.
B.4. Case study 3

The three methods to determine a constant shear strength profile as discussed before are utilized in this section for case study 3. The results of the three methods are discussed and compared whereafter a constant shear strength profile for the soil of case study 3 is determined. The increasing shear strength profile for case study 3 is determined in section 4 and can be described by: \( s_u = 3.3 + 1.6 \cdot d \).

B.4.1. The 2:1 approach method - Case 3

The foundation of case study 3 is 11 meter wide and from the literature is known that all the stresses in the soil due to the structure loading are dissipated at a depth of \( 4 \cdot B \), which is 44 meter depth for this case study. So the soil is investigated to a depth of 44 meter as shown in the left graph of figure B.20. In the figure is shown what the percentage of stresses in the soil due to the vertical structure load is compared to the summed total stress in the soil from the soil self weight and the vertical structure load. In the right graph in figure B.20 only the upper 10 meter is shown and the depth corresponding to the 25% influence. This influence depth is 3.1 meter according to the 2:1 approximation method for case study 3.
The influence depth of 3.1 meter found in figure B.20 is used in figure B.21 to determine the constant shear strength value for the 2:1 approximation method. The constant shear strength profile is determined as the average of the increasing shear strength profile between the seabed level and the influence depth of 3.1 meter. The constant shear strength profile according to the 2:1 approximation method is determined at $s_u = 5.8$ kPa for case study 3.

B.4.2. Plaxis soil reaction force method - Case 3
A very stiff shallow foundation of 11 meter width is modelled in PLAXIS by an applied displacement of 4 meter. First is this displacement applied on a soil with an increasing shear strength profile of $s_u = 3.3 + 1.6 \cdot d$. The vertical soil reaction force found in PLAXIS is 375 kN. Hereafter is the prediscribed displacement applied on a soil with a constant shear strength profile ranging from $s_u = 1$ kPa to $s_u = 15$ kPa. A similar vertical soil reaction force is found in a soil with a constant shear strength of $s_u = 6.7$ kPa. The results of the modelling with PLAXIS are shown in the $F_y - u$ plot in figure B.22.
B.4. Case study 3

B.4.3. PLAXIS failure depth method - Case 3

In the PLAXIS failure depth method the same PLAXIS model from the second method, the PLAXIS soil reaction force method, is used with the same applied displacement of 4 meter only now the failure depth of the failure mechanism is investigated instead of the reaction force of the soil. First is the constant shear strength value of $s_u = 6.7$ kPa is modelled in PLAXIS and the incremental displacements are shown in figure B.23. A clear general shear failure mechanism is shown in the figure. The failure depth of the failure mechanism is 7.5 meter depth as shown in the figure. Hereafter the increasing shear strength profile of $s_u = 3.3 + 1.6 \cdot d$ is modelled. The incremental displacements in the increasing shear strength profile soil are shown in figure B.24. In this figure no clear failure mechanism is shown.

If the scale bars of figures B.23 and B.24 are compared it is shown that they are different. If the scale of the increasing shear strength profile is adjusted to a similar scale of the constant shear strength profile as shown in figure B.23, the increasing shear strength profile with the adjusted scaling is shown in figure B.25. A clear failure mechanism is shown in the figure. However if the failure mechanisms of the constant shear strength profile and the increasing shear strength profile, figures B.23 and B.25, are compared it is clearly shown that...
the failure mechanisms are not similar. The failure mechanism of the constant shear strength profile reaches deeper in the soil and is wider compared to the failure mechanism of the increasing shear strength profile. The failure depth of the increasing shear strength profile failure mechanism is 2.75 meter depth as shown in figure B.25.

The failure depth of 2.75 meter, found in PLAXIS, is used in figure B.26 in order to determine the constant shear strength profile for case study 3 with the PLAXIS failure depth method. The increasing shear strength profile is shown in red in figure B.26 and the constant shear strength profile is shown in blue. The constant shear strength profile determined by the PLAXIS failure depth method has a value of \( s_u = 5.5 \text{ kPa} \).
B.4.4. Constant $s_u$ determination

Three methods are used to determine the constant shear strength profile in the soil for case study 3. The results of these three methods are shown in figure B.27 where the increasing shear strength profile is shown in red. The constant shear strength profile determined by the 2:1 approximation method is shown in green and has a value of $s_u = 5.8$ kPa. The constant shear strength profile determined by the PLAXIS soil reaction force method is shown in yellow and has a value of $s_u = 6.7$ kPa. The constant shear strength profile determined by the PLAXIS failure depth method is shown in blue and has a value of $s_u = 5.5$ kPa. In figure B.27 is shown that the three determined values for the constant shear strength value are relative close to each other. The final determined value for the constant shear strength profile for case study 3 is calculated as the average of the PLAXIS soil reaction force method and the PLAXIS failure depth method. The determined constant shear strength profile for case study 3 is $s_u = 6.1$ kPa, as shown as the magenta line in figure B.27.
B.5. Case study 4
The three methods to determine a constant shear strength profile are discussed before and used in this section for case study 4. The results of the three methods are here discussed and compared and hereafter is the constant shear strength profile for case study 4 determined. The increasing shear strength profile for case study 4 is determined in section 4 and can be described by \( s_u = 4.8 + 2 \cdot d \).

B.5.1. The 2:1 approach method - Case 4
The width of the foundation in case study 4 is 11 meter. From the literature is known that all the stresses are dissipated in the soil at a depth of \( 4 \cdot B \), which corresponds with 44 meter for case study 4. In figure B.28 is in the left figure shown what the percentage of the stresses in the soil due to the vertical structure load is compared to the summed total stresses in the soil from the soil self weight and the vertical structure load until 44 meter depth. In the right plot in figure B.28 is the upper 10 meter shown and which influence depth corresponds to the 25% influence boundary. The influence depth is indicated by the dashed black line at 2.4 meter depth.
The influence depth of 2.4 meter found in figure B.28 is used in figure B.29 to determine the constant shear strength profile with the 2:1 approximation method. The increasing shear strength profile is indicated by the red line and the influence depth is indicated by the black, dashed line. The constant shear strength value is determined as the average of the increasing shear strength value over the depth from the seabed level to the influence depth. A constant shear strength value of $s_u = 7.2$ kPa is determined by the 2:1 approximation method and shown as the green line in figure B.29.
B.5.2. Plaxis soil reaction force method - Case 4

A shallow foundation of 11 meter wide is modelled in PLAXIS on a soil with an increasing shear strength profile of $s_u = 4.8 + 2 \cdot d$. The soil reaction force found in PLAXIS is 518 kN. Hereafter a constant shear strength profile of modelled in the soil with different shear strength values ranging from $s_u = 1$ kPa to $s_u = 15$ kPa. The modelled results are shown in figure B.30. The result of the PLAXIS soil reaction force method is a constant shear strength value of $s_u = 9$ kPa.

![Figure B.30: F_y – d plot for different shear strength profiles for case study 4](image)

B.5.3. PLAXIS failure depth method - Case 4

The same PLAXIS model from the PLAXIS soil reaction force method is used in this PLAXIS failure depth method with the same applied displacement of 4 meter. The predescribed displacement is applied from 33 meter to 44 meter as shown in figures B.31 to B.33. First a soil with a constant shear strength of $s_u = 9$ kPa is modelled. The incremental displacements in the soil are shown in figure B.31 and the failure depth is at 7.25 meter depth.

![Figure B.31: Shading plot for constant shear strength profile with $s_u = 9$ kPa](image)
Hereafter is a soil with an increasing shear strength profile modelled in PLAXIS. The incremental displacements in the soil are shown in figure B.32. In this figure no clear failure mechanism is shown in comparison to the soil with a constant shear strength profile, as shown in figure B.31. If the scale bars of both graphs are investigated is shown that there is a significant difference in them. If the scale of figure B.32 is adjusted to the same scale as figure B.31 a clear failure mechanism is shown in figure B.33. In this figure is a failure mechanism with a failure depth of 3 meter shown.

The failure depth of 3 meter as shown in figure B.33 is used in figure B.34 to determined a constant shear strength profile with the PLAXIS failure depth method. In this figure is the increasing shear strength, as determined in section 4, shown as the red line and the failure depth is shown as the black, dashed line. The constant shear strength is calculated as the average of the increasing shear strength profile over the depth.
between the seabed level and the failure depth level. The determined constant shear strength profile with the PLAXIS failure depth method is $s_u = 7.8$ kPa. This constant shear strength profile is shown as the blue line in the figure.

![Figure B.34: Determination constant shear strength value with the PLAXIS failure depth method for case 4](image)

**B.5.4. Constant $s_u$ determination**

Three methods are used to determine the constant shear strength profile in the soil for case study 4. The results of these three methods are shown in figure B.35. The already determined increasing shear strength profile is shown as the red line. The constant shear strength profile determined by the 2:1 approximation method is shown as the green line. The constant shear strength profile determined by the PLAXIS soil reaction force method is shown as the yellow line and the constant shear strength profile determined by the PLAXIS failure depth method is shown as the blue line.

The three determined constant shear strength profiles all are relative comparable. The determined constant shear strength profile is calculated as the average of the results from the PLAXIS soil reaction force method and the PLAXIS failure depth method. The results from the two used methods give constant shear strength values of $s_u = 9$ kPa and $s_u = 7.8$ kPa. The average of these two methods gives a constant shear strength profile of $s_u = 8.4$ kPa, shown as the magenta line in figure B.35.
Figure B.35: Determination constant shear strength value for case 4
The results from the PLAXIS model for case studies 2, 3 and 4 are shown and discussed in this Appendix. For all four investigated case study a sensitivity analysis is performed for a range of aspect ratio’s and embedment depths for the safety factor against bearing failure and sliding failure.

C.1. Case study 2

In the actual foundation design of case study 2 the embedment is 0.3 meter and the aspect ratio is 0.97. In this investigation an embedment range from 0 to 1.0 meter is investigated since this range covers the realistic embedment depth values for the investigated type of structures. For the analysis where the embedment depth is varied the aspect ratio is kept constant at 0.97. The investigated range of aspect ratio’s is varied from 0.5 to 1.0 and during this analysis the embedment depth is kept constant at 0.3 meter.

C.1.1. Bearing failure

Aspect ratio

The length of the actual foundation design of case study 2 is 8.9 meter and the width 8.6 meter. Table C.1 presents the used foundation length and width dimensions for the sensitivity analysis to the range of aspect ratio’s for case study 2. Due to a maximum precision of 2 decimals in the geometry input values in the PLAXIS model a truncation error is introduced. The area of the used foundation designs for the different aspect ratio’s and the foundation area difference compared to the actual foundation design are presented in table C.1. The difference in foundation area do not have a significant influence on the PLAXIS results and no further measures are taken to reduce the introduced truncation error.

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<th>Area [m²]</th>
<th>Area difference [m²]</th>
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Table C.1: Foundation length and width dimensions for different aspect ratio’s of case 2
Figure C.1 presents the safety factors against bearing failure for the partial safety factor approach in the left graph and for the global safety factor approach in the right graph. A ratio from 4 to 5.5 between the safety factors against bearing failure in the partial safety factor approach is shown. For the global safety factor approach a ratio of 2.1 is seen in the graph.

**Embedment depth**

Figure C.2 presents the results for the range of embedment depths for case study 2. The results for the partial safety factor approach as presented in the left graph. A relative constant ratio of 4 is seen for an embedment from 0 to 0.6 meter. For larger embedment depths the ratio between the safety factors against bearing failure from the PLAXIS model and the ISO design code increases until a ratio of 7.1 for an embedment of 1.0 meter. For the global safety factor approach as shown in the right graph a constant ratio of 2 is seen for an embedment depth from 0 to 0.6 meter. For increasing embedment depths the ratio also increases.
C.1.2. Sliding failure  

Aspect ratio  
Figure C.3 presents the results for the partial and global safety factor approaches for the safety factor against sliding failure. In the partial safety factor approach a constant ratio of 2.4 is seen for the range of investigated aspect ratio’s. For the global safety factor approach a constant ratio of 1.6 is seen for the same range of aspect ratio’s.

![Partial safety factor approach](image1) ![Global safety factor approach](image2)  

Figure C.3: Comparison safety factor for sliding failure for aspect ratio range of case study 2

Embedment depth  
The results for the partial and global safety factor approaches for the safety factors against sliding failure for a range of embedment depths are presented in figure C.4. A non-linear ratio between the results from the PLAXIS models and the design codes is seen. For the partial safety factor approach the ratio ranges from 1.9 to 5.3 and for the global safety factor approach the ratio between the PLAXIS and API results range from 1.5 to 3.0.

![Partial safety factor approach](image3) ![Global safety factor approach](image4)  

Figure C.4: Comparison safety factor for sliding failure for embedment range of case study 2
C.2. Case study 3

In the actual foundation design of case study 3 the embedment is 0.5 meter and the aspect ratio is 0.77. In this investigation an embedment range from 0 to 1.0 meter is investigated since this range covers the realistic embedment depth values for the investigated type of structures. An aspect ratio range from 0.5 to 1.0 is also investigated since this is a realistic range for the investigated type of subsea structures. For the sensitivity analysis of a range of aspect ratios the embedment depth is kept constant at 0.5 meter, while for the sensitivity analysis to a range of embedment depths a constant aspect ratio of 0.77 is considered.

C.2.1. Bearing failure

Aspect ratio

Table C.2 presents the used length and width dimensions for the different aspect ratios as used in the sensitivity analysis in this study. Due to a maximum precision of 2 decimals in the geometry input values of the PLAXIS model a truncation error is introduced. The foundation area and difference in area compared to the actual foundation design for all aspect ratios are shown in table C.2. The actual foundation design for case study 3 consists of a 13 meter long and 10 meter wide foundation with an embedment of 0.5 meter.

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Table C.2: Foundation length and width dimensions for different aspect ratios of case 3

Figure C.5 presents the results for the partial and global safety factor approaches for the safety factors against bearing failure for case study 3. In the partial safety factor approach a constant ratio of 2 is seen for the investigated range of aspect ratios. For the global safety factor approach a constant range of 1.8 is seen.
**Embedment depth**

Figure C.6 presents the results for the partial and global safety factor approaches for the safety factor against bearing failure for a range of embedment depths. In the partial safety factor approach a constant ratio of 2.1 is seen for an embedment range in between 0 and 0.6 meter. For larger embedment depths the ratio between the safety factors from the PLAXIS model and the ISO design code increases to a ratio of 3.5 for an embedment of 1.0 meter. For the global safety factor approach a constant ratio of 1.9 is seen in the figure for an embedment depth range in between 0 and 0.6 meter. For larger embedment depths the ratio becomes larger until a ratio of 2.3 for an embedment depth of 1.0 meter.

![Graph showing safety factor vs embedment depth](image1)

**C.2.2. Sliding failure**

**Aspect ratio**

Figure C.7 presents the results for the partial and global safety factor approaches for the safety factors against sliding failure for case study 3. In the partial safety factor approach a constant ratio of 1.8 is seen for the investigated range of aspect ratio's. For the global safety factor approach a constant ratio of 1.6 is seen for the same range of aspect ratio's.

![Graph showing safety factor vs aspect ratio](image2)
**Embedment depth**

Figure C.8 presents the results for the partial and global safety factor approaches for the safety factors against sliding failure for a range of embedment depths. In both safety factor approaches a non-linear increasing ratio between the determined safety factors in the PLAXIS models and the design codes are seen. In the partial safety factor approach a ratio from 1.7 to 3.1 is seen for the investigated range of embedment depths, while for the global safety factor approach a range from 1.5 to 2.3 is seen.

**C.3. Case study 4**

In the actual foundation design of case study 4 the embedment is 0.4 meter and the aspect ratio is 0.77. In this investigation an embedment range from 0 to 1.0 meter is investigated since this range covers the realistic embedment depth values for the investigated type of structures. For the analysis where the embedment depth is varied the aspect ratio is kept constant at 0.77. The investigated range of aspect ratio’s is varied from 0.5 to 1.0 and during this analysis the embedment depth is kept constant at 0.4 meter.

**C.3.1. Bearing failure**

**Aspect ratio**

<table>
<thead>
<tr>
<th>Aspect ratio [-]</th>
<th>Length [m]</th>
<th>Width [m]</th>
<th>Area $[m^2]$</th>
<th>Area difference $[m^2]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>16.12</td>
<td>8.06</td>
<td>129.9272</td>
<td>0.07280</td>
</tr>
<tr>
<td>0.55</td>
<td>15.38</td>
<td>8.46</td>
<td>130.0994</td>
<td>0.09942</td>
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<tr>
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<td>14.72</td>
<td>8.83</td>
<td>130.0070</td>
<td>0.00704</td>
</tr>
<tr>
<td>0.65</td>
<td>14.14</td>
<td>9.19</td>
<td>129.9607</td>
<td>0.03926</td>
</tr>
<tr>
<td>0.70</td>
<td>13.63</td>
<td>9.54</td>
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<td>0.04383</td>
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<tr>
<td>0.75</td>
<td>13.17</td>
<td>9.88</td>
<td>130.0867</td>
<td>0.08667</td>
</tr>
<tr>
<td>0.77</td>
<td>13</td>
<td>10</td>
<td>130</td>
<td>0</td>
</tr>
<tr>
<td>0.80</td>
<td>12.75</td>
<td>10.20</td>
<td>130.0500</td>
<td>0.05000</td>
</tr>
<tr>
<td>0.85</td>
<td>12.37</td>
<td>10.51</td>
<td>130.0644</td>
<td>0.06436</td>
</tr>
<tr>
<td>0.90</td>
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<td>10.82</td>
<td>130.0320</td>
<td>0.03236</td>
</tr>
<tr>
<td>0.95</td>
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<td>11.12</td>
<td>130.0455</td>
<td>0.04550</td>
</tr>
<tr>
<td>1.00</td>
<td>11.40</td>
<td>11.40</td>
<td>129.9600</td>
<td>0.04000</td>
</tr>
</tbody>
</table>

Table C.3: Foundation length and width dimensions for different aspect ratio’s of case 4
Table C.3 presents the used foundation length and width dimensions as used for the sensitivity analysis to a range of aspect ratios for case study 4. The foundation designs with different aspect ratios have slightly different areas due to a truncation error in the input values for the length and width dimensions in the PLAXIS model since there is a maximum of 2 decimals for the geometry input values. In table C.3 the area of the foundation design for the different aspect ratios and the area difference with the actual foundation design is presented.

Figure C.9 presents the results from the partial and global safety factor approaches for the safety factors against bearing failure for a range of aspect ratios. In the partial safety factor results a constant ratio of 2.1 is seen for the investigated range of aspect ratios, while for the global safety factor approach a constant ratio of 1.8 is seen.

**Embedment depth**

Figure C.10 presents the results for the partial and global safety factors against bearing failure for a range of embedment depths. In the partial safety factor approach a relative constant ratio of 2.1 is seen for an embedment depth between 0 and 0.6 meter. For larger embedment depths the ratio increases until a ratio of
3.2 for an embedment of 1.0 meter. For the global safety factor approach a constant ratio of 1.9 is seen for an embedment depth from 0 to 0.6 meter. For larger embedment depths the ratio increases to a value of 2.3 for an embedment of 1.0 meter.

C.3.2. Sliding failure

Aspect ratio

The results for the safety factor against sliding resistance for a range of aspect ratios of case study 4 are presented in figure C.11. For the partial safety factor approach a constant ratio of 1.6 is seen for the investigated range of aspect ratios, while for the global safety factor approach the same aspect ratio range a constant ratio of 1.5 is found.

Embedment depth

Figure C.12 presents the results for the partial and global safety factors against sliding failure for a range of embedment depths for case study 4. In both the safety factor approaches a non-linear behavior is seen for the ratio over the investigated embedment depth. In the partial safety factor approach the ratio ranges from 1.5 to 3.2 and for the global safety factor approach from 1.4 to 2.4.
C.4. Results safety factor against bearing failure

In this section the results from the calculation of the safety factor against bearing failure by the PLAXIS models and the design codes are shown. The results from the four case studies are plotted in one graph in order to investigate the difference in results between the investigated case studies.

C.4.1. Design code results for bearing failure

Aspect ratio

Figure C.15 presents the results for the partial and global safety factor approaches of the safety factors against bearing failure for a range of aspect ratio's for the four investigated case studies. The results for case study 2, 3 and 4 show a similar behavior for the investigated range of aspect ratio's, while case study 1 shows a different behavior. This different behavior can be explained by the large eccentricities due to the loading of the foundation. An extra hub for the piggyback line is located in the corner of the foundation area and causes these large eccentricities in case study 1.

Also should be noted that the results for the safety factors for case study 2 as shown in figure C.15 as found in the ISO design code are not considered as safe foundation designs for the investigated range of aspect ratio's. The results from the API design code indicate that only an aspect ratio from 0.9 to 1.0 provide a safe foundation design.

![Comparison safety factor against bearing failure for aspect ratio range of all case studies from design codes](image)

Embedment depth

Figure C.16 presents the results from the design codes for the safety factors against bearing failure for the partial and global safety factor approach. The four investigated case studies show a similar behavior for a range of embedment depths, the safety factor increases for an increasing embedment depth. Case study 2 only provides a safe foundation design for a minimal embedment of 0.5 meter for the partial safety factor approach or a minimal embedment of 0.2 meter for the global safety factor approach.
C.4. Results safety factor against bearing failure

C.4.2. PLAXIS results for bearing failure

Aspect ratio
The results from the PLAXIS models for the safety factors against bearing failure for the partial and global safety factor approach are presented in figure C.15 for a range of aspect ratio’s. A similar behavior is seen for the four investigated case studies. These PLAXIS results show more variation compared to the design code results which were located more on a straight line.

Embedment depth
Figure C.16 presents the results from the PLAXIS models for the safety factors against bearing failure for both the partial and global safety factor approaches for a range of embedment depths. Case studies 1, 3 and 4 show a similar behavior in the partial safety factor approach and case study 2 shows a different behavior for an embedment depth larger than 0.7 meter. For the global safety factor approach all four investigated case studies show a similar behavior. From an embedment depth of 0.7 meter an increase in safety factors against bearing failure is seen in the 2 graphs in figure C.16.
C.5. Results safety factor against sliding failure

The results from the calculation of the safety factor against sliding failure are presented in this section of the Appendix. Results from the four case studies are plotted in one graph in order to investigate the difference in results between the investigated case studies.

C.5.1. Design code results for sliding failure

Aspect ratio

Figure C.19 presents the results for the partial and global safety factor approaches for a range of aspect ratio’s. The results of the four investigated case studies show similar behavior for both partial and global safety factor approaches as shown in the figure. The different aspect ratio’s do not have an influence on the safety factors against sliding failure as determined in the design codes for the four case studies.
C.5. Results safety factor against sliding failure

Embedment depth
Figure C.20 presents the results for the partial and global safety factors of the four investigated case studies. The results of all case studies for both the partial and global safety factor approach show a similar increasing behavior for an increasing embedment depth.

C.5.2. PLAXIS results for sliding failure

Aspect ratio
The results for both the partial and global safety factor approach of the four investigated case studies are presented in figure C.19. A constant safety factor for the four case studies in the partial and global safety factor approach are seen from the results. More variation in the result behavior is seen compared to the results from the design codes.
Embedment depth

Figure C.20 presents the results for the partial and global safety factor approaches for the four investigated case studies for a range of embedment depths. The case studies show a similar behavior, the safety factor against sliding failure increases more for embedment depths larger than 0.7 meter. Only the results for case study 2 in the partial safety factor approach shows a slightly different behavior for embedment depths larger than 0.7 meter, the safety factor increases more compared to the other case studies.

![Diagram showing safety factor against sliding failure for embedment depth range of all case studies from design codes](image)

Figure C.20: Comparison safety factor against sliding failure for embedment depth range of all case studies from design codes
D.1. Introduction
The PLAXIS model as used in the investigation to the possible embedded conservatism, and the assumptions in the model are discussed in this appendix. The discussed model and its assumptions is applicable for case study 1, however the PLAXIS models for case studies 2, 3 and 4 have similar assumptions. The general information regarding the model is first discussed, hereafter modeling of the ground, structures and loads are discussed. Finally the assumptions in meshing and the calculation are discussed in this appendix.

D.2. General information

D.2.1. Model contours
Before the foundation can be modeled the model contours are defined. The model contours are boundaries in the model space in x- and y-direction which are chosen by a rule of thumb for general shear failure. According to this rule of thumb, the model contours should be minimal $3 \cdot B$ of the structure width in all horizontal directions. The corresponding model contour values as used in this study are $x_{\text{min}} = 0$ m, $x_{\text{max}} = 80$ m, $y_{\text{min}} = 0$ m and $y_{\text{max}} = 80$ m. The soil mass with the defined contours for this PLAXIS model are shown in figure D.1. The model contours in z-direction are later defined in the borehole profile.

D.2.2. Water conditions
Steady-state water conditions are assumed in the PLAXIS model since the ground water flow is neglected in this study. The ground water level in the soil is known from the site investigation, however since a total stress analysis is used in this study there is no difference between the total and effective stresses in the soil. The
ground water level is thus not important in this study and is therefore chosen at 25 meter below the seabed in the PLAXIS model.

**D.3. Modelling ground**

**D.3.1. Soil layering**

The soil layering in the model domain is defined by creating borehole profiles. PLAXIS interpolates the soil layers in between the different boreholes. In this investigation a single homogeneous soil layer is modelled since not enough information is known from the site investigation regarding the soil heterogeneity and different soil layers. In the site investigation only the upper soil layer is investigated. This layer is extended to a model depth of $z_{\text{min}} = -25\,\text{m}$, and the soil properties below the investigated depth in the site investigation are extrapolated.

**D.3.2. Soil failure model**

In this study the Mohr-Coulomb model is the used soil failure model in the single homogeneous soil layer. The Mohr-Coulomb soil model is a first order approximation for the soil behaviour and it requires only five soil parameters. The two required stiffness parameters are the Young’s modulus $E$ and the poisson’s ratio $\nu$, the three required strength parameters are the cohesion $c$, the friction angle $\phi$ and the dilatancy angle $\psi$. In undrained materials, the Mohr-Coulomb model uses a friction angle of $\phi = 0^\circ$ and a cohesion of $c = s_u$ to enable a direct control of the undrained shear strength. A limitation of the Mohr-Coulomb model is that in an undrained material the increase of shear strength with consolidation is not automatically included.

The use of the less-advanced Mohr-Coulomb soil model instead of more advanced soil models is due to the availability of soil parameters. The used case studies are designed by using design codes as requested by the client. These design codes only require few soil parameters which are investigated in the site investigation. The Mohr-Coulomb soil model uses the investigated parameters and can therefore be used in this investigation. The soil parameters required for the more advanced soil models are not investigated during the site investigation.

**D.3.3. Drainage type**

Undrained behaviour is investigated in this study because only subsea structures located at clay soils are included in this investigation. In PLAXIS it is possible to use three undrained analysis, Undrained A, B and C. In this investigation the drainage type ‘Undrained C’, which is an undrained total stress analysis with undrained parameters is used since only the undrained parameters are investigated during the site investigation. A limitation of this analysis type is that it does not predict the generated pore water pressures and it is therefore not useful for consolidation analyses. Since consolidation is not investigated in this study, this limitation does not give any problems during the analyses.

**D.4. Modelling foundation structures**

**D.4.1. Structural plate elements**

Plates are structural elements used to model thin two-dimensional structures in the ground with a significant flexural rigidity. The plate structures can be used to model shallow foundations and the horizontal, vertical and moment loading acting on them. From the stiffness of the foundation and the soil, the internal forces in the plates can be determined and the soil-structure interaction can be determined. In this investigation only the foundation design is considered and the subsea structure is assumed to be very stiff compared to the surrounding soil and strong enough to resist the applied loads.

**D.4.2. Modelling soil-structure interaction**

The internal structural forces in the investigated case studies are not relevant output data because in this investigation only the foundation design is investigated. Deformations of the foundation are not relevant output data in this study since the foundation is assumed to be very stiff compared to the surrounding soil. The shallow foundation can be modelled as a very stiff structure due to the ratio between the stiffness of the structure and the soil according to [18], [30], [44] and [47].
If however a plate with a much higher stiffness compared to the soil stiffness is modeled in PLAXIS, the global stiffness matrix may deteriorate. In PLAXIS an alternative option is provided to model such stiff structures and prevent problems with the global stiffness matrix. In this alternative option ‘rigid bodies’ are created based on predefined geometry components. The created rigid body is shown as the grey structure in figure D.2. Rigid body elements are advanced features in PLAXIS and therefore there are some limitations to be considered before the use of these elements in situations where a prescribed displacement or dynamic behaviour is modeled. Since these situations are not investigated in this study the limitations in the use of rigid bodies are not applicable and they can be used in this model.

The actual foundation of the case study structure consists of a steel plate with skirts as shown in figure D.3. Since the foundation aspect ratio and the embedment depth are varied in this study adjusting this structure geometry consumes too much time. Instead a rectangular block is used to model the steel plate with the skirts. This is appropriate due to the sufficient amount of inner skirts in the foundation design according to [4] and [59]. These inner skirts prevent failure of the soil within the skirts if the foundation is loaded.

The strength reduction at the interface between the foundation and the soil is not known exactly. This reduction can be adjusted in the PLAXIS model by creating an interface element between the foundation and the soil. First a rigid body is created as shown in figure D.4a and hereafter positive interfaces are created around the sides of the rigid body where there is an interface between the steel skirts and the surrounding soil as shown in figure D.4b. The strength reduction of the interface between the foundation and the soil can be adjusted by the $R_{inter}$ parameter in PLAXIS. The exact value required in this study is not known, so a default value of $R_{inter} = 1.0$ is used in this investigation. This value gives in reality an overestimation of the calculated soil capacity, so a conservative value of $R_{inter} = 0.5$ could be used instead of the optimistic value of $R_{inter} = 1.0$ in order to investigate a conservative value for the soil capacity which is in reality an underprediction. All other interface parameters are set at their default values in this study.
D.5. Modelling loads

Hereafter the loads acting on the structure are modeled in PLAXIS. From the Allseas structure design reports the loads acting on the foundation are known. The created rigid body has a reference point in the center of the foundation and the forces and moments in x-, y- and z-direction acting on the structure are applied at the reference point of the rigid body. These forces and moments are all static loads. The design codes are not capable of modeling cyclic and dynamic loads, so no cyclic or dynamic loads are modeled in this investigation.

The modeled shallow foundation is given a density equal to the modeled soil. By doing this the self weight of the foundation can be applied as a vertical loading acting on the structure while the foundation does not float in the installation phase where no loads are applied on the foundation. In reality the skirted foundation does not replace the soil, it encloses the soil by the skirts. So using a density equal to the soil for the rectangular box used as shallow foundation and applying the foundation self weight as a load is a valid method of modeling this situation in PLAXIS.

D.6. Mesh generation

In PLAXIS it is possible to automatically generate a mesh in the soil mass. Automatic element distributions ranging from very coarse to very fine mesh discretizations as shown in figure D.5, or a manual element distribution can be used in PLAXIS for the mesh generation. In this investigation a fine mesh as shown in figure D.5a is used. The other mesh discretizations are only used in the sensitivity analysis on the mesh discretization. In interesting areas the mesh can be refined by a local mesh refinement. In this investigation the mesh is locally refined below the foundation. This is shown in the figure by the mesh element sizes, they decrease towards the center of the soil mass where the foundation is located. It is however also possible to let PLAXIS determine where a local mesh refinement is required with the ‘enhanced mesh refinement’ option. This is due to the relative simple structure geometry not used in this investigation.

D.6.1. Updated mesh

In conventional finite element analysis, the influence of the mesh geometry change during the loading of the structure is neglected. This is usually a good approximation for situations with relative small deformations. If the deformed mesh has a significant influence it is necessary to take this effect into account. It is possible in PLAXIS to take the deformation of the mesh into account by using an updated mesh analysis where the program updates the mesh automatically. The updated mesh analysis is in this study not used since only relative small deformation occur. The required application of the updated mesh analysis option in the study can be checked by investigating the deformed mesh after the safety factor calculation phase. An example of such a deformed mesh is shown in figure D.6 of the results subsection in this appendix.
D.7. Calculation

D.7.1. Calculation phases

Bearing failure phases
The loads acting on the shallow foundation for bearing failure are modeled in four steps in this model: the initial phase, the installation phase, the loading phase and the safety factor calculation phase. In the initial phase only the ground is modeled and the stresses and strains due to the soil self weight are calculated. In the installation phase the foundation is modeled without any loads. In the loading phase the loading conditions, from the Allseas structure design reports, which are acting on the foundation are applied in the model. In the safety factor calculation phase the loading conditions are increased so that the safety factor can be determined.

Sliding failure phases
In the model for the sliding failure also four loading steps are used in this model: the initial phase, the installation phase, the loading phase and the safety factor calculation phase. The initial and installation phase are similar to these phases for the bearing failure model. The loading phase applies the horizontal loads from the Allseas structure design reports to the foundation and in the safety factor calculation phase one of the
horizontal loads is increased until sliding failure occurs.

D.7.2. Calculation type
The used calculation type in the initial phase is the K0 procedure, in the installation phase, loading phase and the safety factor calculation phase the plastic calculation type is the used calculation type since no consolidation, dynamic behaviour and time-dependent behaviour is considered in this study.

D.7.3. Loading type
The used loading type in the initial phase, installation phase, loading phase and safety factor calculation phases is the staged construction type.

D.7.4. Deformation control parameters
Skirted foundations usually generate suction below the foundation due to uplift and moment loading. In the investigated structures in this study no suction is generated because holes are made in the foundation plate for installation issues. Therefore the suction is ignored in all calculation phases.

After the initial phase where only the ground without foundation is modelled some settlements are seen due to gravity loading. In the installation phase the displacements are reset to zero since the settlements due to gravity loading are no required output information. In the calculation phases following the installation phase the displacements are not reset to zero because the accumulation of settlements due to the foundation installation are required output information.

The numerical control parameters are set at their default values for all the calculation phases in this study.

(a) Phase 1 - Soil mass  
(b) Phase 2 - Installation foundation

(c) Phase 3 - Loading phase  
(d) Phase 4 - S.F. calculation

Figure D.6: Deformed mesh for loading phases of bearing failure in PLAXIS model
D.8. Results

In the results section of this appendix only the deformed meshes for the two different failure modes are shown. Other results of the PLAXIS model are shown in appendix C.

**Deformed mesh for bearing failure**

Figure D.6 presents the deformed mesh for the four calculation phases in the bearing failure calculation. Figure D.6a presents the initial mesh as shown in the calculation phase where only the soil is modeled. The blue rectangle indicates the location of the shallow foundation. In the second calculation phase the shallow foundation is installed as shown in figure D.6b where the foundation is indicated as the green rectangle. Since no loads are applied at the foundation no mesh deformations are seen. After the third calculation step where loads are applied at the foundation some mesh deformations are seen as shown in figure D.6c. The blue boxframe is the original location of the shallow foundation. After the fourth calculation phase where the soil is loaded until failure larger deformations are seen in figure D.6d. It should be noted that the scale of the deformations of the third and fourth phase are not the same. The deformations in the fourth phase, where the soil is loaded until failure, are much higher compared to the third phase in the model.

**Deformed mesh for sliding failure**

The deformed mesh for the four calculation phases in the sliding failure calculation are shown in figure D.7. The initial mesh where only the soil is modelled is shown in figure D.7a. The blue rectangle again is the foundation location. The mesh in the installation phase also is not deformed since no loads are applied on the foundation as in figure D.7b. In the loading phase the mesh is deformed due to the applied horizontal loads as shown in figure D.7c. After the safety factor calculation phase sliding failure has occurred in the soil and the mesh deformation has increased. This is shown in figure D.7d. It should be noted that the scale in figure D.7d is much larger compared to the scale in figure D.7c. Due to the large scale the deformation in the second horizontal direction in figure D.7d is not visible.
The J-mode installation of subsea PLET and FLET structures will be explained and discussed in this section.

The first step in the J-mode installation process is the recovery of the pipeline head. The pipeline head is connected to a crane by a ROV whereafter the pipeline head is lifted to the vessel as shown in figure E.1. In the figure the pipeline is shown as the white pipe connected to the lifting equipment. The thicker part of the pipeline is a roller guide, which covers the connection during the installation of the pipeline over the stinger of the pipelay vessel. The yellow box is a buoyancy block which is connected to the pipeline head by a short cable. The buoyancy block lifts the cable from the seabed so this cable can be connected to the hook from the crane by a ROV.

Figure E.1: Recovery pipeline head from [1]
The next step in the J-mode installation is the removal of the roller guide from the pipeline. First is the pipeline lifted to the platform from where people can cut the bolts from the roller guide. The roller guide covers the bolted connection which is shown in figure E.3. The removal of the roller guide is shown in figure E.2. If the roller guide is removed, the pipeline is placed in the hangoff seat as shown in figure E.3. Hereafter the pipeline can be released from the lifting equipment as shown in figure E.4.
If the pipeline is released from the cranes lifting equipment, it is prepared for the connection with the subsea structure. The bolts in the bolted connection as shown in figure E.3 are removed and the oversized piece of pipe which is installed over the actual pipeline is removed. Hereafter the pipeline is prepared for the connection with the subsea structure. The pipe is beveled as shown in figure E.5 before welding it to the structure.

Hereafter the structure is lifted on the A-frame as shown in figure E.6. The subsea structure is the yellow structure in the figure and the A-frame is the red structure. This A-frame is used later to lower the subsea structure with the pipeline on the seabed.
Before the subsea structure and the pipeline are connected, they are lined up as shown in figure E.8 and figure E.9. If they are lined up the structure and the pipeline are welded together. Beside the welded connection at the beveled part of the pipeline there is a bolted connection between the pipeline and the structure. The lower part of the bolted connection as shown in figure E.3 is bolted to the horizontal yellow beam of the structures as shown in figure E.9 behind the white helmets. During the connecting of the structure to the pipeline, the pipeline remains in the hangoff seat as shown in figure E.10 and figure E.11.

After installation of the structure and the pipeline on the seabed the lifting equipement is disconnected and the installation of the subsea structure is completed.
Figure E.8: Lining up of the structure and the pipeline shown from above from [1]

Figure E.9: Lining up of the structure and the pipeline from [1]
Figure E.10: Connection of the structure to the pipeline from [1]

Figure E.11: Installation of the structure and pipeline on the seabed by the A-frame from [1]
[1] Allseas Pipeline Engineering department portal


[31] Fugro, (2008), *Cone Penetration Interpretation*, FEBV/CDE/APP/012, p 1-11


[51] International Standards Organisation and International Electrotechnical Commission *Using and referencing ISO and IEC standards to support public policy*


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