FINITE ELEMENT MODELING OF A SUCTION CAISSON
SUBJECT TO MONOTONIC TENSILE LOADING

by

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ABSTRACT

Suction caissons are a proven method for securing floating oil and gas platforms in deep waters that have recently been garnering interest as a potentially more cost-effective, environmentally-friendly option to use in foundations for offshore wind turbines. However, due to complex interactions with saturated soils, the behavior of a suction caisson installed in dense sands under the unique loading regime of an offshore wind turbine is currently under vigorous numerical and experimental investigation.

This thesis concentrates on using finite element analysis (FEA) to simulate the soil-structure interaction (SSI) of a single suction caisson installed in dense sands subjected to vertical pull-out at various speeds. Using a numerical formulation that accounts for the displacement of soil as well as the pore-fluid pressure between soil grains, a two-dimensional axisymmetric finite element simulation of a suction caisson under tension (i.e. pull-out) is constructed using the program ABAQUS 6.14. Special, low-stiffness elements were added to simulate seepage into the gap formed between the lid and the soil inside the caisson as it was pulled from the seafloor. The development of suction pressure within the interior and exterior of the caisson in response to pull-out at various rates is examined and presented for a range of void ratios and tensile loading speeds. The total resistance to pull-out and the response of the soil inside and outside the suction caisson at each velocity is also examined.

As a basis for comparison, the observations of the numerical analysis are validated against the findings of an approximately 1:20 scale model test of a suction caisson undergoing tension loading at various rates. Overall, the results of the
numerical modeling show strong agreement with experimental results for maximum pull-out resistance and suction pressure. Increased pull-out velocities resulted in higher total pull-out resistance and increased suction. The influence of initial void ratio of the soil and the Young’s modulus of the water elements is examined and shown to have an effect on the peak suction pressure achieved during extraction.

Future work on the analysis presented in this thesis should focus on improving the modeling of interface friction to include fluid flow along the edges of the caisson in addition to implementing a three-field numerical formulation that accounts for pore fluid displacements and pressure, simultaneously.
Chapter 1
INTRODUCTION

1.1 Overview

Renewable energy provides hope for a sustainable future in which energy is not only available long-term, but also less damaging to the environment (International Energy Agency (IEA), 2018). Renewable energy technologies come in a variety of forms: solar, wind, and wave. Wind energy in particular is rapidly gaining footing in the energy sector (Global Wind Energy Council, 2017). However, some land-based wind energy projects encounter heavy opposition due to perceived aesthetic and noise-level impacts, and also can draw the ire of environmentalists seeking to protect migratory species of birds and bats that wind turbines are known to kill. Some of the disadvantages of on-land wind power are significantly reduced by siting wind turbines offshore, relatively far from coastal populations (reducing viewshed and noise concerns), where the wind blows stronger, more frequently and there are fewer migratory species of bird/bat that may be harmed by the operating wind project.

Today, more than 82% of all offshore wind turbines installed are founded on a single, large monopile or a group of smaller piles, which are driven into the subsurface using either dropped or vibrating hammers. This type of installation can generate extremely loud vibrations that place marine life at high risk of disturbance, injury, or even death (Andersson, et al., 2016). In the United States, concerns over harming indigenous marine mammal populations has resulted in stringent regulations regarding the installation of piled structures (NOAA, 2015). Currently, any developer
conducting pile driving in U.S. waters must have multiple whale spotters whose responsibility is to shut down all operations when a whale is spotted and to coordinate pile-driving operations around seasonal migration patterns of marine mammals (Regulations Governing the Take of Marine Mammals Incidental to Specified Activities, 50 C.F.R. pt. 2, 2017).

Suction caissons are named because of their method of installation, which may involve the use of suction pressure to assist in achieving optimal installation depth. They are effectively harmless to marine wildlife as they do not require vibrating or dropped hammers to install (van den Akker & van der Veen, 2013). Suction caissons are garnering interest in the offshore wind industry after a long, proven track record of use in the offshore oil and gas industry (Andersen, et al., 2005; Dean, 2010). However, unlike an oil and gas platform, offshore wind turbines are light and tall structures that experience large horizontal forces in proportion to vertical forces due to wind and waves acting along the length of the structure (Byrne, 2000; Achmus & Thieken, 2014; Vaitkunaite et al. 2016). The overturning moment generated by these forces acting at a distance from the foundation creates a rocking motion that places the upwind caisson under tension and the downwind caisson under compression, as shown in Figure 1.1 (Senders, 2008; Achmus & Thieken, 2014). This is a dangerous condition for an offshore structure to be in, for without some type of tensile resistance in the uplifting caisson, the entire structure will overturn (Senders, 2008).
In order to safely utilize suction caissons to support offshore wind turbines, accurate standards for the prediction of their tensile capacity under severe pull-out conditions must be known. However, since they are a relatively novel concept for supporting an offshore wind turbine, there are few analytical methods of predicting the
tensile capacity of suction caissons in high-permeability soils like dense sands (Senders, 2008; Vaitkunaite, 2016; Vaitkunaite et al. 2016; Sorensen et al. 2016). Instead of installation primarily in clay, future offshore wind projects sited in the North Sea and/or along the U.S. East Coast will likely be founded in sediments containing high-permeability, non-cohesive sands, which complicate the prediction of the tensile resistance to pull-out. One tool utilized by geotechnical engineers to simulate and develop analytical methods of predicting soil-structure interaction (SSI) is finite element (FE) modeling (Serbulea, 2013). The focus of this thesis is the development of a FE model that simulates the complex pull-out response at various rates of a suction caisson installed in dense sands.

By nature, any numerical model approximates real-world conditions and therefore requires validation to confirm that the numerical formulation is accurate (Helwany, 2007). In other words, before a model can be used to predict real-world load scenarios on full-size turbines, the numerical formulation (which defines the physical laws of the model) must be shown to match results from controlled experiments. Foundations strong enough to support an offshore wind turbine tend to be so large and expensive that it is not feasible for them to be tested in a controlled environment at full scale (Tran et al. 2004). Instead, civil engineers must rely on the observations and results of reduced scale-model tests to validate any proposed numerical formulations. Once a model has been shown to approximate the observations of scaled down conditions, common practice is to extrapolate the design to full size in order to predict soil-structure dynamics at a specific site (Houlsby et al. 2005b).
1.2 Problem Definition and Objectives

The overarching goal of this thesis project is to design, construct, and validate a FE model that simulates the pore-pressure response and load-displacement behavior of a suction caisson undergoing pull-out in dense sands. A numerical formulation that accounts for pore pressure and soil displacements is implemented in the FE modeling program, ABAQUS 6.14. The numerical formulation selected utilizes modeling capabilities that are available to any ABAQUS/CAE license holder. For validation, the results of the FE model are compared against the observations of a scale model test conducted by Vaitkunaite et al. (2016). This reduced scale-model testing campaign was conducted to serve as a reference for calibrating numerical models of suction caissons under tensile loading. To explore a wide range of tensile behaviors, Vaitkunaite et al. (2016) applied nine different pull-out rates while simultaneously recording changes in pore pressure at various points on the suction caisson, the displacement of the caisson itself, and the magnitude of the load applied by the pullout actuator to keep the displacement rate constant. The series of tests conducted by Vaitkunaite et al. (2016) distinguishes itself from others because it utilizes a scale model that is twice the size of any other previous pullout tests; an approximately 1:20 suction caisson with a lid diameter of 0.5 meters (m) and a skirt length of 0.25 m. This is beneficial to the overall design process as the closer that the testing apparatus is to actual size, the more readily the results can be extrapolated to actual foundations. The Vaitkunaite et al. (2016) scale experiment also distinguishes itself by making use of a large pressure tank capable of reproducing the hydrostatic conditions present in soil at seawater depths of 20 m. Finally and critically, the mechanical properties of the soil used in the Vaitkunaite et al. (2016) model tests are precisely known. Aalborg University, where the test campaign was conducted, used their own proprietary soil
mixture known as Baskarp Sand No. 15 that has been analyzed using geotechnical laboratory analysis (Ibsen, 2009; Sjelmo, 2012). As a result, the strength properties of the soil are well constrained and are easily implemented into a numerical model.

The primary objectives of this thesis project are to:

1. Design, construct, and execute FE models of the experimental conditions of a reduced-scale suction caisson in dense sand undergoing constant-rate pull-out at nine different rates.

2. Compare the results of the FE models against the observations of nine 1:20 scale model pull-out tests conducted by Vaitkunaite et al. (2016).

3. Evaluate the predictive accuracy of the proposed FE model numerical formulation in comparison to analytical methods proposed by Houlsby et al. (2005b) and Iskander et al. (2002).
Chapter 2

BACKGROUND AND THEORY

2.1 Value and Technical Aspects of Offshore Wind

2.1.1 Price of Offshore Wind in the U.S.

Offshore wind development is accelerating in Europe, China and the United States (U.S.) (Global Wind Energy Council, 2017). Europe has consistently led the way in developing offshore wind resources and, in 2017, Europe set a new record by erecting 612 new turbines, which correspond to approximately 3,148 megawatts (MW) of new generating capacity and is double the amount added in 2016 (WindEurope, 2018). The U.S. on the other hand, has not installed an offshore wind turbine since 2016 and only has six offshore wind turbines, totaling 30-MW of capacity at the Block Island Wind Farm in the Atlantic Ocean off the coast of Rhode Island (Mills, et al. 2018). Unlike in Europe, where the government, infrastructure, and supply lines have been streamlined to lower the costs of permitting, fabrication, and installation of an offshore wind project, offshore wind has had a very slow start-up period in the U.S. due to higher costs as a result of the expensive and complex permitting/siting process and a lack of existing infrastructure or supply chain (Grace, et al. 2017; Firestone et al. 2015).

The East Coast of the U.S. in particular is ideally situated for grid-scale offshore wind projects (Firestone et al. 2015). In terms of energy potential, the offshore wind resource off the East Coast of the U.S. is powerful enough to provide 4,574 terawatt-hours of energy a year, which is four times the electricity needs of the 14 states on the East Coast (Weissman et al. 2018). Of the 14 states that border the Atlantic Ocean, 12 of them have offshore wind potential that exceeds the electricity.
needs of that specific state (Weissman et al. 2018). From a water-depth perspective, the U.S. East Coast has favorable conditions for fixed (i.e. non-floating) offshore installations. The price of non-floating offshore structures increases dramatically with the depth of the water at the site and distance from shore (Rademakers, 2010). The relatively broad and shallow continental shelf of the Atlantic Ocean provides substantial space in economic water depths for fixed offshore wind projects (Figure 2.1).

The high wind resource, shallow continental shelf of the East Coast of the U.S. borders the third most populated coast in the world with 118.4 million residents, which represents 37% of the total U.S. population (U.S. Department of Energy (DOE), 2015). The East Coast, with cities such as New York, Philadelphia, Boston, and Washington D.C., demands more energy than any other region on Earth and is responsible for 34% of the total electric power consumed in the entire country (U.S. DOE, 2015). At present, these energy needs are met primarily with coal, petroleum, natural gas, nuclear, and hydropower, with just 4% of all East Coast energy production from renewable sources such as solar and onshore wind (U.S. DOE, 2015). Near highly populated cities along the East Coast, there is sparse free space to accommodate large-scale on-land wind turbine or photovoltaic arrays in response to increasing energy demands. Off the East Coast of the U.S. extending from Maine to Georgia, offshore wind offers an opportunity to meet increased demands for electrical power while reducing the dependency on energy from non-renewable sources in this critical area of the U.S.
Even though the U.S. is ~20 years behind Europe in terms of installed and operating offshore wind capacity, momentum is shifting in U.S. markets as the number of proposed offshore wind projects on the East Coast has steadily increased over the last 5 years (WindEurope, 2018; Weissman et al. 2018). At present, 14.2 gigawatts (GW) of U.S. offshore wind energy capacity have been proposed and are at various
stages of permitting and planning at sites along the East Coast, shown in Figure 2.1 (Weissman et al. 2018; Bureau of Ocean Energy Management (BOEM), 2018). Of the total 14.2 GW of proposed offshore wind capacity, 1.2 GW of offshore wind capacity from four proposed projects off the coasts of Rhode Island, Maryland, New York, and Massachusetts have secured long-term power purchase agreements (PPAs) to sell power into each respective state’s electricity market (Weissman et al. 2018; The Massachusetts Department of Energy Resources (DER), 2018). Due to the long-term loans required to finance up front, PPAs are necessary for the seller to guarantee revenue streams over the duration of the loan obtained to finance the project (Miller et al. 2017), and are advantageous to the buyer because they insure constant, or at least predictable, price of electricity. The price of the sale of electricity agreed upon through a given project’s PPA is based on the estimated levelized cost of energy (LCOE) of that project in order to determine the payback period of the capital investment and the profitability of the project (Miller et al. 2017). The LCOE is calculated by dividing the lifetime costs of an energy source by the lifetime energy production and is used to compare energy generating technologies of unequal life-spans, project size, capital costs, risk, return, and energy generating capacity (Short et al. 1995). Between 2012 and 2017, the price of energy from an offshore wind project agreed upon through PPAs ranged between $113 per megawatt-hour (MWh) to $160/MWh, which, as Figure 2.2 shows, was less competitive with existing energy sources (Lazard, 2017; Weissman et al. 2018). However, as of August 1st, 2018, Massachusetts’ Department of Energy Resources (DER) published a letter to the state’s Department of Public utilities regarding the proposed offshore wind project, Vineyard Wind, which stated that the planned project will provide Maine’s commonwealth with energy and
renewable energy credits (RECs) at a total levelized price of $65/MWh: a 42% drop from 2017 (Massachusetts DER, 2018). The dramatic reduction of the LCOE of Vineyard Wind Project means that offshore wind is now cheaper than the majority of non-renewable electricity generating methods, as shown in Figure 2.
Figure 2.2: LCOE of various energy sources in 2017 with added datapoint that reflects the 2018 offshore wind LCOE for Vineyard Wind Project (Lazard, 2017; Massachusetts DER, 2018).
2.1.2 Trends and Technology

A significant portion of the investment in an offshore wind project is devoted to designing, fabricating, transporting, and installing the turbine at an offshore site. The complexity of a given site depends on environmental conditions such as the water depth, wind conditions, storm and wave heights, distance from shore, and soil conditions (Rademakers, 2010). To keep installation costs low in response to increased project complexity, the current trend is to use the largest turbines available, thus requiring the fewest amount of foundation structures to be built and deployed.

The production of a potential wind farm is directly proportional to the number of turbines that can fit in the available area. The array spacing of the wind farm determines the number of turbines that can fit within the proposed area and should be selected to most-effectively reduce losses due to wake effects (Manwell et al. 2009). One method of minimizing the effect that an upstream turbine has on a downstream turbine is to place them farther apart. Balancing tight spacing against aerodynamic losses can be done using best practices that suggest turbine spacing as a function of the diameter of the wind turbine rotor (Manwell et al. 2009). For example, studies on wind turbine wake done by Smith and Taylor (1991) show that smaller spacings of 2.5 diameters between turbines result in losses as high as 60%. These losses decrease with turbine spacing, reaching as low as 10% losses for wind turbines 15 diameters apart. However, the spacing geometry for a location should reflect the directional wind patterns of the wind regime, as each location has a unique wind regime.

Offshore wind turbines, which are less limited in size unlike land-based wind turbines, have attained truly epic proportions in the pursuit of greater efficiency and capacity. For example, the largest currently installed offshore wind turbine is the Vestas V164, which has a 220-m-height, a 164 m-rotor-diameter, and an 8 MW
nameplate capacity (Wittrup, 2014). However, even larger turbines are in development. GE announced in March 2018 plans to set a new world record for the largest offshore wind turbine by developing a 260-meter-tall, 12 MW, 214-m-rotor-diameter turbine, scheduled for testing summer 2019 and to be sold within the next 3 to 5 years of 2018 (Kellner and Petsche, 2018). Figure 2.3 below offers a sense of scale as to the size of modern offshore wind turbines.

![Figure 2.3: Size comparison and specifications of future 12-MW GE turbine (Kellner & Petsche, 2018).](image)

Wind turbine developers are able to harness more energy by increasing the height of the turbine tower so the turbine rotor can reach the faster winds at higher altitudes. With a taller tower, the size of the rotor itself has also increased for greater swept area and higher blade tip speed. With fewer turbines and therefore fewer foundations to install, larger turbines with higher energy generating capacities both
reduce costs while increasing the profit generated by selling the energy captured from higher speed, uninterrupted offshore winds.

While the upscaling of wind turbines is greatly increasing the energy generating efficiency and total capacity of a given wind farm, the mammoth task of keeping such a dynamic structure intact for more than 20 years at sea is made much more difficult by using larger turbines. Increasing the size of the turbine alters the frequency and magnitude of the loads on the turbine, requiring a more robust support structure that is best-suited to withstand any and all forces (Innwind.EU, 2015; Nikitas et al. 2016).

2.1.3 Offshore Loading Conditions

Larger turbines are a challenge to support in offshore conditions as the sheer size and dynamic nature of the structure requires significant foundation strength. The role of the foundation is to support the turbine tower, nacelle, and rotor under any load for the lifetime of the installation. However, a wind turbine is not a static structure by any means. For example, each blade of the Vestas V164 wind turbine weighs 35 tons and the entire rotor can spin as fast as 12 times per minute (Vestas Wind Systems A/S, 2011). Over the design life of the turbine, which can be 20 years or longer, the support structure will have to withstand over 100 million loading cycles from several sources. A structural engineer must predict the behavior of the structure in response to repeated loads, such as the spinning of the turbine rotor, or in response to extreme static loads, such as the force from an unusually large storm wave crashing against the substructure. As Figure 2.4 below illustrates, there are four main load sources acting on an offshore wind turbine in the form of aerodynamic drag on the structure and
rotating blades, waves in the form of hydrodynamic drag on the foundation, and the cyclical 1P and 3P loads exerted by the rotating blades (Nikitas et al. 2016).

Figure 2.4: A schematic drawing of the relevant forcing acting on an OWT (Nikitas et al. 2016).

2.1.3.1 Hydrodynamic and Aerodynamic Drag

Unlike a land-based wind turbine, an offshore wind turbine foundation must be designed specifically to withstand the load produced by waves and currents crashing and dragging against the foundation underwater. The magnitude of the force acting against the structure depends on wave height and wave period and is best quantified using numerical hydrodynamic modeling and/or site-specific wave measurements. In reality, wave height is random in both space and time (spatial and temporal) and are
best represented statistically using a power spectral density (PSD) function, such as the one used by Arany et al. in their 2015 study of North Sea wave conditions. A PSD function can be used when directly sampled, in-situ wave height and direction data are not available.

The foundation is acted on by the wave profile, which may contain a current component in addition to cyclical motion that is typical of a passing wave. The sum of forces along the foundation results in a moment at the mudline that can be analytically determined using hydrodynamic modeling. For monopile structures, shown above in Figure 2.4, the hydrodynamic drag is simpler to simulate and estimate accurately. However, for more complex jacket structures, which have individual, slender trusses arranged into a lattice, resolving the drag forces to single moments and components of forces at each leg is very complex, requiring sophisticated models and processing power (Innwind.EU, 2015).

Aerodynamic drag from wind is also difficult to quantify, requiring aerodynamic simulations to find and resolve the total force exerted on the structure to a moment at the mudline. Similar to wave drag, the accuracy of these simulations depends on the density of available on-site wind measurements, or lacking that, utilization of a reputable PSD function calculated from existing data (Arany et al. 2015)

2.1.3.2 1-P and 3-P Loading

Aerodynamic drag on the structure is not avoidable and is actually critical for the wind turbine to function. Fundamentally, a wind turbine is designed to capture the mechanical power of the moving air using drag on the airfoil-shaped blades. However, the rotating blades, which have considerable mass on larger turbines, create a load of
their own as a result of rotating through the air. It should be noted that the wind blowing past a turbine is not distributed evenly and that there is a component of random turbulence acting unevenly on the structure. When the rotating blades encounter such a turbulent pocket of air, the rotor is dragged unevenly as it passes through, creating a vibration that repeats with each rotation. In other words, as each blade rotates about the nacelle, a turbulent air stream will excite the blade each revolution as shown in Figure 2.5a. This first excitation frequency which is the same as the rotational speed of the rotor, is called 1-P. For three-bladed rotors, which are the most popular design in use today, a second excitation frequency results from each successive blade passing through the same turbulent air stream. Therefore, for three-bladed wind turbines, this second frequency is called 3-P and is just a multiple of 1-P. Figure 2.5b shows that for a wind turbine that operates in a range of wind speeds, the 1-P and 3-P excitations are spread across a band of frequencies with gaps in between. (Van der Tempel & Molenaar, 2002).
The loading frequencies of 1-P and 3-P excitations cannot overlap with the natural frequency of the supporting structure in order to avoid resonant response (Innwind.EU, 2015). A resonant response is a dangerous situation in which the structure’s natural vibrating frequency is excited by a cyclic load, such as the 1-P or 3-P loads discussed above. Resonant behavior can begin as a small vibration and grow into complete failure of the structure, producing displacements many times larger than if the mass was acted on by a static force of the same magnitude (Van der Tempel & Molenaar, 2002). The goal of a foundation designer is to engineer a support structure and foundation that fits within the “safe” range of frequencies (Innwind.EU, 2015). As a result, all existing offshore wind turbine foundations are soft-stiff foundations with
natural frequencies that are between the 1-P and 3-P rotor excitation frequencies (Arany et al. 2015).

However, meeting this design criteria is becoming increasingly difficult as the demand for ever larger wind turbines increases. As discussed previously, a taller turbine with a larger rotor diameter is more efficient and able to collect more energy over a given time interval. However, the consequence of a larger rotor is that the frequency of 1-P and 3-P excitation are lowered and shifted dangerously close to each other. As Figure 2.6 shows below, the range of safe frequencies for a supporting structure is extremely narrow.

Figure 2.6: Frequency diagram of recurring loads acting on the Siemens SWT-107-3.6 offshore wind turbine in 9 m/s average wind speed operating between 5 and 13 rpm. (Arany, Bhattacharya, Macdonald, & Hogan, 2015).
2.1.4 Support Structures and Foundations

The task of resisting all loads shown in Figure 2.6 in addition to supporting the weight of the turbine and tower above is given to the component of the offshore wind turbine known as the sub-structure. There are two categories of sub-structure, fixed and floating, of which each has two main components: the support structure, which is the section beneath the wind turbine tower that is exposed to the ocean and wave action, and the foundation, which is the actual footing that is secured in the soil at the seafloor. Figure 2.7 below shows several designs of monopile, jacket, and floating support structures that are secured to the seabed by either a gravity base, piles, or suction caisson foundation.

![Typical configurations of offshore wind turbine support structures and foundations](image)

Figure 2.7: Typical configurations of offshore wind turbine support structures and foundations, image modified from Bhattacharya (2014).
It should be noted that Figure 2.7 only covers the most basic support structure and foundation configurations, especially for jackets, which come in a wide variety of lattice orientations with three or more footings in the subsurface, as discussed in greater detail in Bhattacharya (2014). The same is true for floating sub-structures; only two design concepts are shown here despite the growing number of floating designs, discussed in detail in WindEurope’s (2017) report on floating offshore wind development (WindEurope, 2017). The decision for which type of support structure to use in combination with a particular footing is made based on site conditions. Each type of support structure has inherent advantages and disadvantages that the structural designer must consider when planning the construction of an offshore wind turbine. Similarly, there are also advantages and disadvantages of each footing concept that can be combined with the support structures. The suction caisson, which is the focus of this thesis, is compatible with all types of support structure shown in Figure 2.7.

2.2 Suction Caissons

Suction caissons (also referred to as skirted piles or suction buckets in literature) have two parts: a circular lid with diameter D and skirt that has a length, L, and thickness, t. (Figure 2.8). The proportions of the caisson, which are defined by the L/D ratio, vary depending on the soil conditions and expected loads. Caissons with a L/D ratio of 2.0 or higher are typically used for clayey soils under predominantly tensile loading while caissons with lower L/D ratios are better suited for sandy soil conditions and predominantly compressive loading (Houlsby et al. 2005b; Cotter, 2009).
2.2.1 Development and Use

The first suction caisson, which had a lid diameter of 0.45 m and a skirt length of 1.2 m, was used in 1958 to secure a small core sampler to the seafloor in 100 m deep water without heavy weights (Mackereth, 1958). In 1980, enlarged suction caissons with lid diameter of 3.8 meters and skirt lengths between 5 and 10 meters (L/D = 1.3 and 2.6) were tested by Hogervorst (1980) to be used as anchors for floating and fixed platforms. Ten years later, the suction caisson concept was applied to an actual foundation, the Gullfaks C platform, in the form of sixteen 22 m long concrete suction caissons supporting a heavy concrete base, as shown in Figure 2.9 below (Tjelta et al. 1990). The success of the Gullfaks C platform proved that the suction caisson concept was suitable for supporting fixed, heavy offshore structures in water 220 m deep (Tjelta et al. 1990; Tjelta, 2014). By 2004, more than 485 suction caissons in over 50 locations had been used as anchors for floating oil and gas platforms as well as foundations for fixed oil and gas platforms (Andersen et al. 2005).

Byrne (2000) and Houlsby et al. (2000) were the first to propose and test the suction caisson concept in the single footing and multiple footing configurations specifically for the purpose of supporting fixed offshore wind turbines in dense sands at seafloor depths between 10 and 30 meters. It was shown that the high horizontal and relatively low compressive loading of offshore wind turbines results in significant

Figure 2.8: Schematic of several L/D ratios of suction caissons for visualization.
tensile loads and overturning moments on the suction caisson footings (Byrne, 2000; Houlsby et al. 2000). Two years later, a prototype single footing suction caisson (D = 12 m, L = 6 m, L/D = 0.5) was installed offshore of Frederikshavn, Denmark and proven to adequately support a 3.0 MW Vestas V90 turbine for 3 years (Ibsen et al. 2006). Single footing suction caisson foundations were also used at Horns Rev 2 and Dogger Bank wind projects in 2009 and 2011, respectively, to support temporary meteorological masts for as long as six years (Holdsworth, 2015). In 2014, two model monopile suction caissons (L = 6 m, D = 8 m, L/D = 0.75 and L = 6 m, D = 4 m, L/D = 1.5) were installed and removed at three different sites a total of 29 times in just 24 days in order to investigate and de-risk the installation procedure in complex soil conditions (Tjelta, 2014). The same year, the first jacket concept which uses multiple suction caissons (L = 8 m, D = 8 m, L/D = 1.0) was installed as a prototype at the Borkum Riffgrund 1 wind project and shown to sufficiently support a 4.0 MW Siemens offshore wind turbine in 25 m of water (4C Offshore, 2018). May of 2018 the tripod suction caisson-jacket foundation was installed as the foundation for all 11 offshore wind turbines at the European Offshore Wind Deployment Center (EOWDC) in water 50 to 60 m deep. Installed in 15 hours, EOWDC set a record for the fastest installation as well as the largest offshore wind turbine supported by suction caissons and jacket foundation: an 8.4 MW Vestas V164.8.4 Turbine (Vattenfall, 2018). In addition, the Borkum Riffgrund 2 wind project completed installation of 20 suction caisson jacket foundations on July 30th, 2018 (Frith, 2018).
2.2.2 Installation

Suction caissons are able to secure offshore platforms to the seabed without dropped or vibrating hammers by using suction (Senders, 2008). Once the caisson is placed on the seafloor using a floating lift, the caisson will sink partially into the seabed under its own weight and force seawater out through an open valve at the top of the lid. To achieve optimal installation depth, the valve is attached to a pump that pulls water from the interior of the caisson, lowering the water pressure within the caisson. The net difference in pressure between the interior of the caisson and surrounding water pushes the caisson into the seafloor, but the soil mechanism that occurs at the same time depends on the soil type.

If the caisson is installed in sand, which has relatively large spaces between particles, the lower pressure within the caisson will instigate seepage from the soil beneath. The flow regime that develops within the soil reduces the friction acting
against the tip and sides of the caisson via a process called liquefaction, allowing the caisson to reach the proper installation depth. If the caisson is installed in clay, which is typical for offshore oil/gas platforms, seepage cannot develop because there is not enough space between soil grains. Without seepage to reduce friction, suction caissons installed in clays require higher suction pressures in order to reach optimal installation depth.

Removal of the suction caisson uses the same procedure in reverse; by pumping water into the caisson interior and lifting it out of the seabed. Installation or removal can be completed in as little as 12 hours, which is superior to the multi-day installation time for driven piles (Chatzivasileiou, 2014). However, there is a lack of experience with using suction caissons designed specifically for offshore wind turbines, which are very light and tall compared to a traditional oil and gas platform.

2.2.3 Drainage Conditions

The load-bearing behavior of a suction caisson under tensile load post-installation depends on several factors, collectively known as the drainage conditions. Drainage conditions are dependent on the permeability of the soil, the rate of the load, and the dimensions of the caisson, which determines the length of the seepage path that pore water can take into the caisson (Houlsby et al. 2005b; Senders, 2008) Figure 2.10 below shows the three possible responses of a suction caisson resisting pullout in undrained, intermediate, and fully drained conditions (Achmus & Thieken, 2014).
2.2.3.1 Fine Grained Soils

Historically, suction buckets have been used by the offshore oil and gas industry in the form of temporary or permanent anchors for floating platforms in cohesive, low-permeability soils like clay (Cotter, 2009). The permeability of a soil refers to the degree at which the soil allows fluid to pass through the soil matrix (Verruijt, 2012). A high-permeability soil, like sands and gravels with large grains and gaps in between, allows for seepage of this pore fluid through the soil matrix in response to a pressure gradient. In clayey and silty soils with low-permeability, pore fluid flows very slowly and seepage is negligible (Verruijt, 2012; Chatzivasileiou, 2014). Most offshore oil and gas platforms secured with suction caissons, due to the nature of the formation of hydrocarbons, are located on clay-rich sea beds (Cotter, 2009; Chatzivasileiou, 2014). In such scenarios, the behavior of the caisson under pullout is considered undrained: suction pressure generated by the uplift of the caisson...
firmly pulls the soil inside the caisson upwards at the same rate, preventing the formation of a gap inside the lid and drawing surrounding soil into the bucket (Achmus & Thieken, 2014). This behavior, termed reverse-bearing, provides the strongest resistance to pull-out and is the preferred tensile response for securing an offshore foundation (Senders, 2008; Cotter, 2009). However, as mentioned previously, the drainage conditions are not determined by permeability of the soil alone, but also the load velocity. Conversely, under slow loading, suction caissons in clay will exhibit fully-drained behavior, resisting pull-out via friction along the caisson skirt alone (Cotter, 2009).

2.2.3.2 Coarse Grained Soils

Unlike fine grained soils like clay, which experience fully drained conditions under slower tensile loads, coarse grained soils have considerably higher permeability as large spaces between soil grains allow for water to travel easily through the soil. As a result, when the caisson is acted on by a tensile force, the suction pressure that would otherwise prevent gap formation is dissipated at the same rate at which it forms. In other words, the suction pressure inside the caisson is relieved by pore water seeping into the gap between the soil and the lid of the caisson, leaving friction along the inner and outer skirt as the only force acting opposite to the pull-out load. Compared to the undrained scenario, fully drained conditions offer the least resistance to pull-out.

More frequently, however, suction caissons installed in dense sands will yield an intermediate response that is a mixture of fully drained and undrained conditions. In intermediate conditions, raising the lid in uplift generates negative pore pressure within the interior of the caisson. The negative pore pressure instigates seepage into
the gap of the caisson from the saturated soil below, reducing the suction force that resists the pull-out. As a result, the total resistance to pull-out at any given time is the sum of the friction on the skirt, the weight of the soil plug and caisson being lifted, and suction pressure within the caisson interior. As Figure 2.10 shows, an intermediate response results in pull-out resistance that is between the ideal, undrained case and the weak, fully drained scenario.

Each of these forces and their individual contribution to pull-out resistance varies non-linearly over the duration of a given tensile load. The behavior of suction caissons under tensile loads in dense sands has been under both experimental and numerical investigation in order to more accurately quantify the tensile response of suction caissons.

2.2.4 Scale Model Investigations of Suction Caissons

The first published investigation into the behavior of suction caissons under purely tensile loads was conducted by Iskander et al. (1993). While previous studies on the tensile behavior of suction caissons have been conducted prior to Iskander et al. (1993), the results of those studies were not published in their entirety in order to protect proprietary techniques. In 2000, Byrne (2000) conducted small-scale tests on a suction caisson in dense sands specifically for offshore wind turbines (L = 50 mm, D = 150 mm, L/D = 0.33). Byrne (2000) tested the response of the caisson to pull-out loading as well as cyclical vertical loading. Byrne (2000) found that there was a close correlation between the behavior of the caisson under cyclical loads and the response of the caisson to monotonic pull-out. In addition, he noted that increasing the pull-out rate also increased the resistance of the caisson to extraction, but only after large amounts of heave had been achieved (Byrne, 2000). These results were confirmed by
Feld (2001), who conducted similar pull-out tests at various rates with a different dimension caisson (L = 100 mm, D = 200 mm, L/D = 0.5). Feld (2001) also noted that the resistance of the caisson to pull-out was significantly greater if cyclical loading is applied beforehand.

Houlsby et al. (2005b) performed pull-out tests on a larger scale model caisson (L = 180 mm, D = 280 mm, L/D = 0.64) at pull-out rates between 5 mm/s and 100 mm/s. It was observed that decreasing the permeability of the soil or increasing the pull-out rate resulted in a strong increase in the total resistance to pull-out (Houlsby et al. 2005b). However, the suction pressures observed during the scale model tests were later determined to be artificially large as a result of the loading conditions applied by Houlsby et al. (2005). Based on his findings, Houlsby et al. (2005b) developed an expression for predicting the ultimate pull-out resistance and peak suction pressure acting on a suction caisson that are presented in a later section. One year later, Houlsby et al. (2006) conducted tests on an even larger scale model caisson (L = 1000 mm, D = 1500 mm, L/D = 0.66) in a prepared field test site in Scotland. The field test confirmed that partially drained conditions provide a greater resistance to pull-out than fully-drained conditions, but require a significant amount of extraction before the resistance can be mobilized.

Senders (2008) designed and executed a centrifuge test campaign in which five caissons with L/D ratios, varying from 0.5 to 1.63, were subjected to pull-out. The lengths of the five model caissons varied from 60 mm to 114 mm, while the diameter was between 49 mm and 120 mm (Senders, 2008). The results of the centrifuge testing showed that the pull-out rate has a greater influence on resistance to pull-out than the initial stiffness (Senders, 2008). Senders (2008) developed a conceptual model with a
large number of input parameters that have proven to be difficult to assess and reproduce (Achmus & Thieken, 2014). Zhu et al. (2011) also conducted scale model tests on a scale model caisson (L = 600 mm, D = 1.0 m) but is not published in English.

Vaitkunaite et al. (2016) conducted pull-out tests on the largest scale model caissons in a pressure vessel to date (L = 250 mm, D = 500 mm, L/D = 0.5). The design and results of these tests are discussed in a later section.

2.2.5 Numerical Modeling Investigations of Suction Caissons Under Tensile Loads

2.2.5.1 Cao et al. (2002)

Cao et al. (2002) developed and constructed a FE model of the tensile response of a suction caisson installed in clay. Using 1:100 scale laboratory centrifuge tests conducted by Cao et al. (2001) on a small suction caisson (L = 0.18 m, D = 0.0517 m, L/D = 3.5) as validation, the FE model constructed by Cao et al. (2002) was built using the program ABAQUS and formulated in axisymmetric space. As Figure 2.11 shows, the right-most boundary and the axis of symmetry of the finite element mesh were fixed in the horizontal direction, while the base of the model is fixed in both horizontal and vertical directions.
Figure 2.11: Finite Element (FE) mesh and boundary conditions (Cao et al. 2002).

The stress-strain behavior of the porous soil was modeled using a Modified Cam-Clay plasticity model with parameters calculated using the results of laboratory consolidation tests. The interaction between the deformable soil and the steel caisson was simulated using a Coulomb friction law in tandem with the contact surface approach available in ABAQUS, which automatically generates appropriate contact elements and is thought to be the most appropriate technique for simulating pipe/soil interaction based on the study done by Popescu (1999).

The key feature of the model designed by Cao et al. (2002) was the ability to account for suction mobilization, which is known to make up a significant contribution of the total resistance to pull-out. Accounting for suction, which occurs as a result of gap formation under the lid of the suction caisson during extraction, required the
implementation of a thin layer of poro-elastic soft material beneath the caisson lid (Figure 2.11). This layer of poro-elastic material was intended to simulate water that fills the expanding gap during extraction. The poro- indicates that the material is pore-pressure coupled and able to account for changes in pore pressure as well as stress, while the elastic indicates that the material has a linear stress-strain response. The stiffness, or Young’s modulus of the elastic material is chosen to be a very small value in order to simulate water. These softer elements were directly bonded to both the soil as well as the caisson in axisymmetric space. Cao et al. (2002) is the first published FE analysis that utilizes this technique to transfer the force due to negative pore pressures (i.e., suction) to the underside of the caisson.

In comparison against the laboratory centrifuge tests, the FE model proposed by Cao et al. (2002) performed adequately. Figure 2.12a and 2.12b below show a direct comparison of the results for total tensile resistance as well as mobilized suction.

Figure 2.12: Comparison of (a) Total Pullout Force Development and (b) Passive Suction Development (Cao et al. 2002).
2.2.5.2 Mana et al. (2013)

Mana et al. (2013) developed a FE model in ABAQUS of suction caissons with L/D ratios of 0.1, 0.2, 0.3, 0.5, and 1.0 to investigate the seepage and pore pressure conditions during pull-out at various velocities. The geometry and boundary conditions of the model were similar to Cao et al. (2002), as both were formulated in axisymmetric space (Figure 2.13), however Mana et al. (2013) adopted a higher mesh density.

![Figure 2.13: FE mesh and boundary conditions (Mana et al. 2013).](image)

The porous soil was modeled using continuum elements defined as isotropically, linearly elastic and weightless. The caisson was assigned rigid body properties to ease model computation time and the coupling of stress and pore fluid was modeled using Biot-type three-dimensional consolidation (Biot, 1941). Interface behavior between the caisson and the porous soil was modeled using surface to surface
discretization. The tangential friction along all interfaces was selected as perfectly smooth (i.e. no transfer of shear stress from the caisson to the soil) to provide an upper limit to the magnitude of displacements. By disabling friction effects, the total resistance to extraction was reduced unrealistically. However, in order to utilize the ABAQUS built-in pore-pressure formulation, a frictionless and therefore unrealistic interface had to be used in tandem with the pore pressure formulation for, the ABAQUS solver struggles to converge on a solution in this specific situation (Kaliakin, 2018). This inconsistency is a regrettable but necessary to achieving the goals of this modeling campaign, as the ABAQUS built-in pore-pressure formulation is required to investigate the seepage and pore pressure changes in response to tensile extraction of varying magnitudes. The interface surfaces between the soil and caisson were also prevented from passing through each other in the direction normal to their surfaces. In the style of Cao et al. (2002) soft, poro-elastic elements representing water were added just beneath the lid of the caisson (Figure 2.13). Loading was applied to the caisson at magnitudes of 10%, 20%, 30%, 40%, and 60% of the maximum uplift pressure observed during centrifuge tests conducted by Mana et al. (2012).

The results of the FE model were used to develop an expression for seepage length and velocity as a function of the caisson L/D ratio and the magnitude of the extraction force. The equations obtained from the FE model were then used to back-calculate centrifuge testing conducted by Mana et al. (2012) on a 1:200 scale-model suction caisson installed in lightly over consolidated clay. Upon comparison, it was concluded that the equation developed for seepage velocity using the FE model showed strong agreement with the centrifuge tests conducted at lower loading
magnitudes. As Figure 2.14 shows, seepage velocities observed during centrifuge pullouts at higher magnitude uplift loads were much higher than those predicted by the equation derived from the FE model.

Figure 2.14: Comparison of the normalized rate of foundation displacements under various loads between centrifuge results and equation derived from FE model (Mana et al. 2013).

2.2.5.3 Achmus & Thieken (2014)

Achmus & Thieken (2014) performed a FE analysis in ABAQUS of a full-size suction caisson (L = 9 m, D = 12 m, L/D = 0.75) undergoing pull-out at rates of 0.01 mm/s, 0.1 mm/s, 0.5 mm/s, 1 mm/s, 10 mm/s, and 100 mm/s. The model was developed in axisymmetric space with a mesh density greater than Mana et al. (2013) and Cao et al. (2002), as shown in Figure 2.15.
The porous soil material was modeled using a hypo-plastic constitutive model developed by von Wolffersdorff (1996). The parameters of the von Wolffersdorff constitutive model were determined through laboratory experiments on IGT sand. The soil permeability was modeled using the highly accurate Kozeny-Carman approach, which accounts for the void ratio, viscosity of the wetting fluid, and the grain size distribution of the soil (Carrier, 2003). In the style of Cao et al. (2002), poro-elastic elements with a low Young’s modulus were added under the lid of the caisson to allow for transfer of suction pressure (Achmus & Thieken, 2014). Interaction between the suction caisson and the porous soil was initialized using a
similar surface-to-surface discretization as Cao et al. (2002), with maximum shear stress along the interface modeled using a Coulomb friction law (Achmus & Thieken, 2014).

A unique aspect of the FE model developed by Achmus & Thieken (2014) was the application of an upper and lower boundary solution for allowed seepage and liquefaction. Liquefaction in non-cohesive soils like sands occurs if the pore seepage (i.e., the hydraulic gradient) through the soil matrix becomes too great, causing the total loss of effective stress. Achmus & Thieken (2014) proposed an upper bound solution for the mobilization of seepage where no liquefaction was considered. When no liquefaction was considered, the FE model was permitted to simulate the mobilization of high hydraulic gradients in the porous soil elements that would have caused soil failure in real life (Achmus & Thieken, 2014). As a lower bound solution that considers liquefaction, the FE model was “checked” and fixed to a maximum allowed value every time that the critical hydraulic gradient for liquefaction was exceeded (Achmus & Thieken, 2014). The behavior of the FE model upper and lower boundary solutions under various rates of pull-out are shown in Figure 2.16a and 2.16b (Achmus & Thieken, 2014).
Figure 2.16: Total resistance to pull-out and suction pressure beneath the caisson lid for (a) upper bound solution and (b) lower bound solution (Achmus & Thieken, 2014).

The results of both upper and lower boundary solutions agree fundamentally with basic suction caisson dynamics observed in experiments: i.e., the resistance to pull-out increases sharply with the pull-out rate and the greatest amount of resistance is mobilized after a large amount of heave (Houlsby et al. 2005b; Achmus & Thieken, 2014). In addition, at small extraction rates, the suction pressure beneath the lid of the caisson was negligible (Achmus & Thieken, 2014).
For validation, the FE model constructed by Achmus & Thieken (2014) was adapted to match the dimensions of a scale model experiment conducted by Iskander et al. (1993), which used a small-scale suction caisson (L = 194 mm, D = 100 mm, L/D = 1.94). As Figure 2.17 shows, the FE model performed reasonably well at approximating the shape and magnitude of the load-displacement curve, but struggled to reproduce the erratic pore pressure changes at various points along the interior of the caisson observed by Iskander et al. (1993).

![Figure 2.17: Comparison of FE results and model test results from Achmus & Thieken (2014) and Iskander et al. (1993).](image)

2.3 Saturated Soil Properties, Mechanics, and Modeling

2.3.1 Definition and Parameters

Soil mechanics, by definition, is the science of equilibrium and motion of soil bodies, where soil is defined as the product of weathering of rocks into smaller pieces (Verruijt, 2012). Given enough time, natural processes such as freezing and melting water, rain, wind, and exposure to the sun break down solid rock into particles that can
then be transported long distances through fluvial processes, glaciers, or even wind. During transport, internal friction acting on broken fragments of rock also contributes to the reduction of soil grain size. Naturally formed, each soil is unique and categorized by the relative amounts of different sized particles that they contain over a unit volume. There is a wide variety of soil types that can be differentiated using various soil classification systems, however for the most part, they are separated by grain size (Table 1.1).

Table 2.1: General soil types based on grain size (Verruijt, 2012).

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Grain size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum</td>
</tr>
<tr>
<td>Clay</td>
<td>-</td>
</tr>
<tr>
<td>Silt</td>
<td>0.002</td>
</tr>
<tr>
<td>Sand</td>
<td>0.063</td>
</tr>
<tr>
<td>Gravel</td>
<td>2.000</td>
</tr>
<tr>
<td>Stones</td>
<td>63.00</td>
</tr>
</tbody>
</table>

As a porous material, soils can be described as a mixture of solid, liquid, and gas particles. Soils are categorized by the relative amounts of different sized particles that they contain over a unit volume. The void ratio $e$ is the ratio of the volume of voids filled with air or water ($V_{voids}$) to the volume of solid soil particles ($V_{solids}$).

$$e = \frac{V_{voids}}{V_{solids}} \quad (1)$$

The minimum and maximum void ratio of a sample are measured in a laboratory setting to quantify the density ratio of a given sample. The density ratio, or “relative density”, is defined by the following relationship:

$$D_R = \frac{e_{max} - e}{e_{max} - e_{min}} \quad (2)$$
where $e_{max}$ is the void ratio of the sample under the loosest state and $e_{min}$ is the void ratio of the sample when densely compacted. The density ratio is always between 0 and 1, with small values indicating soil that is easily densified and values close to 1 indicating that there is little room for further compaction of voids.

The saturation of a soil refers to how much of the voids are filled with water instead of air. Saturated soils, specifically, are composed of solid grains with voids completely filled by liquid water. Soils located on the seafloor are fully saturated, with no gaseous air present and are the primary focus of offshore geotechnical design (Dean, 2010). The total unit weight ($\gamma$) of a saturated soil sample is the sum of the force due to gravity ($W$) acting on the soil and water per unit volume ($V$), which is equivalent to the sum of the soil ($\rho_s$) and water ($\rho_w$) density times the acceleration due to gravity, $g$.

$$
\gamma = (\rho_s + \rho_w)g = \frac{W}{V}
$$

\subsection*{2.3.2 Pore Pressure and Effective Stress}

In terms of a complete soil unit comprised of solid grains and spaces filled with fluid, the pores are assumed to be mutually connected and constitute a single, continuous body of fluid. Pressure may be transferred through the inter-granular fluid or, in the presence of a pressure gradient, may cause the pore-fluid to flow. When there is no flow of pore fluid, the pressure in the water (called pore pressure) is determined by the depth below the surface of the water. Since shear stress cannot be transferred by water, the shape of the container is irrelevant and only the depth beneath the water’s surface ($d$) and the unit weight of the fluid ($\gamma_w$) determine the pore pressure ($u$) at a given depth using the following relationship.

$$
u = \gamma_w \cdot d$$
The presence of water, specifically between soil grains, dramatically alters stress transfer within the soil, for pore water between soil grains is also under pressure. In a soil unit under pressure (Figure 2.18), the overburden that threatens to collapse the soil sample is balanced by the pore pressure in the intergranular water and the stresses between individual soil particles. The stress between particles is generated by friction and normal forces at the small contact points between irregularly shaped grains and completely determines the deformation of the soil body (i.e., the soil only changes shape when soil grains actually slide over/past each other).

Figure 2.18: Diagram illustrating the effective stress principle in a saturated cohesionless soil (Atkinson, 2000).

The isotropic pressure of the saturating water acts in a normal direction to the exterior of all soil grains and, since water has no shear strength (e.g., you cannot make a pile of water; it must have a container), pressurized pore fluid in between grains reduces the ability of the soil to resist shear deformations. It is common practice is to treat the total stress \( \sigma \) acting in a larger soil body as a combination of the stress between individual soil grains, known as the effective stress \( \sigma' \), and the hydrostatic stress exerted by intergranular fluid \( u \); i.e.,
\[ \sigma = \sigma' + u \] (5)

The effective stress principle was introduced by Karl Terzaghi (1936) and applies to normal stresses, as shear stresses are only transmitted through the soil skeleton. In three dimensions, the shear stress components acting on the x, y, and z-plane are determined by the effective tangential stress between soil grains, alone.

\[ \sigma_{xx} = \sigma'_{xx} + u \]
\[ \sigma_{yy} = \sigma'_{yy} + u \]
\[ \sigma_{zz} = \sigma'_{zz} + u \]
\[ \tau_{yz} = \sigma'_{yz} \]
\[ \tau_{zx} = \sigma'_{zx} \]
\[ \tau_{xy} = \sigma'_{xy} \]

Figure 2.19: Terzaghi’s equations for applying the effective stress principle in three dimensions.

This formulation assumes that the effective stress which governs the sliding of grains and the overall motion of the soil body occurs on very small contact surfaces and that the individual soil grains themselves are extremely stiff in relation to the soil sample as a whole. By assuming the soil particles are incompressible, the effective stress principle is not exact, but instead offers a very good approximation of the behavior of larger units of soil. In practice, the total stress and pore pressure are typically well-known based on the weight of the soil, external loads, and the height of the water table. The effective stresses are estimated by subtracting the known values for pore pressure and total stress. The effective stress principle is used in later chapters.
to provide an estimate of the stress state when calculating the initial pore pressure and effective stress distributions of the scale model test conducted by Vaitkunaite et al. (2016).

2.3.3 Mechanical Behavior

The mechanical behavior of soils is complex in comparison to the rock that it originates from. For example, soils can be compacted and excavated with tools or by hand, unlike rock which must be loosened or broken into smaller pieces. Most materials are linearly elastic and will deform a certain amount under applied stress before returning to the original shape. Hooke’s law holds that the stress and strain are proportional for linearly elastic materials. Thus, if the applied stress is doubled, the deformation is doubled. Soils, however, exhibit non-linear, inelastic behavior under compressive stress: the more the soil is compressed, the stiffer it becomes. For example, soil under little stress, such as the sand at the top of a wind-blown dune, can be displaced with light pressure from the fingers or foot. However, the deeper sand that comprises the base of the dune is very strong, supporting the tremendous weight of the dune above due to being under high stress. The relative strength of soils under compaction makes them suitable for supporting buildings and other structures, however, special considerations must be made to avoid placing soil under shear loading.

Under shear stress, which acts parallel to a given surface unlike compressive (normal) stress, soils lose strength and can eventually fail. The clearest example of this is an unsupported pile of sand. The low shear strength of the sand prevents the slope of the sides of the sand pile from exceeding 30 to 40 degrees as, any steeper will cause sand grains to fall down the side. Failure is possible within the sand pile itself if the
Sample is tilted past some critical angle (i.e., placed under shear stress). Failure of soil in response to shear stress is often accompanied by volume expansion, known as dilation. Most materials that fail in shear will simply deform without changing volume. Dense configurations of larger grained soils, such as sand, exhibit an increase in volume during shear deformation (Figure 2.20).

Figure 2.20: Conceptual diagram of volumetric strain of soil sample known as dilation in response to shear stress (Verruijt, 2012).

Dilatancy is the phenomena responsible for the formation of a dry spot under the feet of beach-going pedestrian near the waterline. Common sense says that walking on damp sand should squeeze the water from the sand and create a small puddle around the foot. However, instead, walking on damp sand creates a dry spot in the sand around the foot. The cause of this counterintuitive behavior is the dilation of the sand: the weight of the human on the foot causes the sand along the edges of the foot to deform under shear and to expand in volume. The extra space between sand grains draws pore water from the surface of the sand into new voids, making the sand next to the foot appear drier.

2.3.4 Continuum Finite Element Modeling

Applying FE modeling and other numerical techniques to the difficult task of predicting soil mechanics allows geotechnical engineers to solve otherwise
unapproachable problems, such as a suction caisson undergoing rapid extraction (Helwany, 2007). A key caveat of FE modeling of soils, however, is that the soil is approximated as a larger unit. In other words, instead of defining material laws that describe the behavior of individual soil grains interacting on small-scales, FE constitutive models are fitted to laboratory testing data that quantifies the mechanical behavior of soil specimens composed of many soil grains (Helwany, 2007). As a result, most FE models which make use of a constitutive routine assume that the small-scale particle interactions can be approximated as continuous stresses and strains on a unit element as shown in Figure 2.21 (Helwany, 2007).

Figure 2.21: (a) Soil composition of discrete particles and voids filled with fluid and air (b) three-dimensional finite element used to represent a unit element of soil under continuous stresses (Helwany, 2007).

2.3.4.1 Mohr-Coulomb Elasto-plasticity

One widely used constitutive model that accounts for the dilative and plastic behavior of dense sands is Mohr-Coulomb elasto-plasticity. The premise of the Mohr-
Coulomb routine is that soil behavior can be elastic or plastic depending on the stress state. As described in Helwany (2007), elastic materials will deform linearly in response to the magnitude of the applied stress and return to their original dimensions once the stress is removed.

Soils are one of the many types of materials that transition to plastic behavior if the stress state exceeds a particular yield criterion (Helwany, 2007; Dean, 2010; Verruijt, 2012). Materials that have exceeded the yield criterion and enter the plastic regime will deform permanently in response to additional stress (Helwany, 2007; Dean, 2010). Consider, as a simple example, a steel beam. Under low stress conditions, the beam will bend in response to loads, but ultimately return to the original shape. If the stress on the beam is increased enough to reach the plastic yield criterion, the beam will begin to yield plastically and deform as long as the stress is still applied. Once the stress on the beam is removed, a portion of the strain accumulated during the plastic regime will remain as it is permanent. The yield criterion, which defines the stress conditions that merit a transition from elastic behavior to inelastic plastic behavior, is therefore critical for simulating non-linear materials like soils (Helwany, 2007; Dean, 2010, Verruijt, 2012).

The response of cohesionless soils like dense sands under monotonic loads is reasonably well predicted by the Mohr-Coulomb yield function below:

\[ \tau_f = c + \sigma' \tan(\varphi) \]  

(6)

where \( \tau_f \) is the maximum shear stress acting on a plane, \( \sigma' \) is the normal effective stress acting on the same plane, \( \varphi \) is the angle of internal friction, and \( c \) is the cohesion intercept (Dean, 2010; ABAQUS, 2014). The maximum shear stress is a linear
function of the applied normal effective stress that can be plotted in shear stress and normal stress space as shown in Figure 2.22 below (ABAQUS, 2014).

![Figure 2.22: Mohr-coulomb yield criterion, adopted from (ABAQUS, 2014).](image)

The parameters of the yield function (cohesion, angle of internal friction, and normal effective stress) are fitted to experimental data of the soil sample under triaxial testing (Dean, 2010; Ibsen et al. 2009). During testing, different confining stresses are applied until the sample yields plastically. The three principal stresses, \( \sigma_1 \), \( \sigma_2 \), and \( \sigma_3 \) are plotted as a Mohr circle (Figure 2.22) which touches the yield function at the onset of plastic behavior. The straight line defined by the yield function acts as a dividing line between elastic behavior and plastic behavior. In three dimensions, the yield function is defined for each principal stress direction. At any given time, if the principal stresses acting on a unit of soil are plotted as a Mohr circle which touches or exceeds the yield function, the mechanical behavior of the element is switched to plastic and allowed to deform and flow.

Once a unit of soil enters the plastic regime, the direction and magnitude of the plastic strain is a vector determined by the gradient of the plastic flow potential. The plastic flow potential can be defined as associated or non-associated based on the
amount of dilation expected to occur in the soil. The dilation angle ($\psi$) is a property of larger-grained soils such as dense sands in which the volume of a given sample increases in response to shear stress (Figure 2.20). Stresses on isotropically consolidated clays and other cohesive soils are well approximated using an associated flow rule (Helwany, 2007). In clays, which have a dilation angle of zero, plastic deformation occurs perpendicular to the Mohr-Coulomb yield function, which has a slope equal to the angle of internal friction ($\phi$). Dense sands, however, often exhibit dilation during plastic flow and are more accurately described using a non-associated flow potential (Helwany, 2007). The amount of volumetric expansion due to shearing of soil grains is accounted for using the angle of dilation ($\psi$) (Verruijt, 2012). To invoke the non-associative flow potential, the angle of dilation must be a different value than the angle of internal friction (AB AQUS, 2014).

### 2.4 Design of Scale Model Test Conducted by Vaitkunaite et al. (2016)

The overarching goal of this thesis is to simulate the experimental conditions of the scale model experiment conducted by Vaitkunaite et al. (2016). The purpose of the scale model experiment was to investigate the mobilization of pore pressure and tensile resistance when a small-scale suction caisson is extracted from saturated dense sands at various constant velocities. The experiment was designed with validation of FE modeling in mind as a specific use for the results. To apply FE modeling techniques for approximating the mechanical behavior of soils, the dimensions and relevant soil properties of the experiment must be well known.
2.4.1 Equipment Geometry

At the time of the writing of this thesis, the small-scale suction caisson used in the Vaitkunaite et al. (2016) pull-out testing is the largest scale model to undergo this type of testing, with a lid diameter of 500 millimeters (mm), a skirt length of 250 mm, and a thickness of 2 mm. Figure 2.23 below shows the layout of the 11 transducers, labeled PP1 through PP11, used to measure the intergranular fluid pressure at various depths along the caisson skirt.

Figure 2.23: Diagram of the suction caisson model dimensions (in mm) used in pull-out testing with locations of pore pressure transducers (Vaitkunaite, 2015)

The setup for testing suction caissons includes a large pressure vessel that simulates the pressure of 20 m of water, shown in Figure 2.24. Loads can be applied to the small-scale caisson via the hydraulic piston, which can be used to extract or install the caisson at user-specified velocities (Vaitkunaite et al. 2016). A load cell measures
and modulates the load applied to maintain the user-specified velocity, while two displacement transducers record the distance extracted or installed (Vaitkunaite et al. 2016).

![Diagram of experimental setup](image)

Figure 2.24: Diagram of the experimental setup used to apply constant-velocity pull-out loads, shown to scale (Vaitkunaite, 2015).

### 2.4.2 Soil Properties

The sand selected for the Vaitkunaite et al. (2016) testing was Aalborg University Sand No. 1 (or Baskarp Sand No. 15) which has undergone extensive laboratory testing to determine the grain distribution, strength, and pore fluid properties (Hedegaard and Borup, 1993; Ibsen and Bodker, 1994; Ibsen et al. 2009; Sjelmo, 2012). The saturated unit weight was determined to be 20.0 kN/mm³ by Ibsen et al. (2009).
2.4.2.1 Hydraulic Conductivity and Void Ratio

The hydraulic conductivity, or permeability, of Baskarp Sand No.15 was measured by Sjelmo (2012) using a falling head apparatus and determined to have a strong dependence on the void ratio. Figure 2.25 shows the testing data for permeability as a function of void ratio fitted to a quadratic function. The quadratic function displayed on in Figure 2.25 was used to approximate the permeability of a given soil across the observed range of void ratios in Baskarp Sand No.15, which is between 0.549 and 0.858. In the next chapter of this thesis, this equation is used to apply hydraulic conductivity to the FE model.

Figure 2.25: Hydraulic conductivity vs. void ratio for Baskarp Sand No.15 (Sjelmo, 2012).
2.4.2.2 Density Index

The density index (or density ratio) of the soil was measured on a test-by-test basis by Vaitkunaite et al. (2016) after each soil sample had been compacted and prepared accordingly. The density ratio was estimated using a cone penetrometer testing (CPT) procedure developed by Larsen (2008), in tandem with a numerical expression for density ratio as a function of CPT resistance developed by Ibsen et al. (2009). The result of the pre-extraction CPT testing is an estimate of the density ratio for that particular test (Table 2.1), which is used to calculate the strength parameters of the soil.

Table 2.2: Density ratio of each pull-out test calculated from CPT testing on the soil before undergoing caisson installation and subsequent extraction at a constant rate (Vaitkunaite et al. 2016).

<table>
<thead>
<tr>
<th>Pull-out test (mm/s)</th>
<th>Density Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>v0.01</td>
<td>79</td>
</tr>
<tr>
<td>v0.05</td>
<td>85</td>
</tr>
<tr>
<td>v0.1</td>
<td>86</td>
</tr>
<tr>
<td>v1</td>
<td>90</td>
</tr>
<tr>
<td>v10</td>
<td>90</td>
</tr>
<tr>
<td>v22</td>
<td>83</td>
</tr>
<tr>
<td>v27</td>
<td>85</td>
</tr>
<tr>
<td>v47</td>
<td>83</td>
</tr>
<tr>
<td>v98</td>
<td>82</td>
</tr>
<tr>
<td>v152</td>
<td>84</td>
</tr>
</tbody>
</table>

2.4.2.3 Mohr-Coulomb Parameters

A common technique for quantifying the strength properties of a soil is to calibrate the sand to fit a Mohr-Coulomb elastoplastic constitutive model using a combination of cone penetration testing and drained triaxial testing (Helwany, 2007; Dean, 2010; Serbulea, 2013). The sand utilized in the scale model experiments
conducted by Vaitkunaite et al. (2016) underwent Mohr-Coulomb parameter calibration to accurately determine the strength properties of the soil (Ibsen et al. 2009). The results of this analysis are presented in the form of a series of equations that can be used to calculate all five Mohr-Coulomb parameters using just two known values: the specific density ratio (DR) and confining pressure (σ) (Ibsen et al. 2009). The confining pressure is always equal to the absolute pressure within the tank, which is 200 kPa to simulate 20 meters of water plus, 100 kPa of atmospheric pressure. Vaitkunaite et al. (2016) took great care in determining the density ratio of the soil before each test such that the Young’s modulus (E), Poisson’s ratio (ν), friction angle (φ), dilation angle (ψ), and cohesion (c) could be applied to each model using the following equations proposed by Ibsen et al. (2009).

Young’s modulus (MPa)

\[ E = (0.6322 \times 10^{-3}) \cdot (D_R)^{2.507} + 10.920 \]  \hspace{1cm} (7)

Poisson’s ratio (ν)

\[ ν = 0.250 \]  \hspace{1cm} (8)

Friction angle (°)

\[ φ = 0.110 \cdot D_R + 32.3° \]  \hspace{1cm} (9)

Cohesion (MPa)

\[ c = (0.032 \times 10^{-3}) \cdot D_R + (3.520 \times 10^{-3}) \]  \hspace{1cm} (10)

Dilation angle (°)

\[ ψ = 0.195 \cdot D_R + 14.9 \cdot σ^{-0.0976} - 9.95° \]  \hspace{1cm} (11)
2.4.3 Testing Program

Before pull-out testing could be conducted by Vaitkunaite et al. (2016), a 600 mm thick sample of Baskarp Sand No.15 had to be prepared: first by loosening the soil using pressurized water and then compacting the sample with a vibrating rod. Once compacted, the soil sample was subjected to several rounds of cone penetrometer testing to calculate the density ratio of the sand (Vaitkunaite et al. 2016). Next, the model suction caisson was hydraulically rammed into the soil at a slow speed of 0.05 mm/s with the valve on the top of the caisson lid opened to allow water to leave the caisson interior (Vaitkunaite et al. 2016). Installation was halted once the interior lid of the caisson contacted the soil and the distance installed was carefully recorded for each test (Table 2.2). After the tank was sealed and pressurized to 300 kPa, pull-out testing was carried out using the displacement-controlled load actuator at 10 different extraction rates (Vaitkunaite et al. 2016).

Table 2.3: Depth of installation of the suction caisson at the start of each pull-out test (Vaitkunaite et al. 2016).

<table>
<thead>
<tr>
<th>Pull-out test (mm/s)</th>
<th>Installation depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>v0.01</td>
<td>241</td>
</tr>
<tr>
<td>v0.05</td>
<td>239</td>
</tr>
<tr>
<td>v0.1</td>
<td>241</td>
</tr>
<tr>
<td>v1</td>
<td>242</td>
</tr>
<tr>
<td>v10</td>
<td>242</td>
</tr>
<tr>
<td>v22</td>
<td>236</td>
</tr>
<tr>
<td>v27</td>
<td>239</td>
</tr>
<tr>
<td>v47</td>
<td>236</td>
</tr>
<tr>
<td>v98</td>
<td>239</td>
</tr>
<tr>
<td>v152</td>
<td>236</td>
</tr>
</tbody>
</table>
2.4.4 Results of Testing Campaign

The results of the pull-out tests were presented by Vaitkunaite et al. 2016 in the form of load-displacement plots and tabular results. In this manner, the development of the total tensile load as well as the mobilized suction load in relation to the extraction distance could be examined. The data for the total load required to maintain the extraction velocity was collected from the load cell and plotted as a function of the extraction distance obtained from the displacement transducer (Vaitkunaite et al. 2016). The suction pressure \( s \) was estimated as the differential pressure difference between the data collected by the pore pressure transducers under the caisson lid and the pressure within the tank using the following equation:

\[
s = p_{atm} + p_{tank} - \text{p}_{abs}
\]  

(13)

where \( \text{p}_{abs} \) is the measured pressure obtained from pore pressure transducer, \( p_{atm} \) is the atmospheric pressure of 100 kPa, and \( p_{tank} \) is the applied pressure within the tank of 200 kPa (Vaitkunaite et al. 2016). The above equation was organized so the suction pressure was positive. The total load, as a result of the differential suction pressure \( (F_s) \), was calculated using the following relationship:

\[
F_s = s \cdot \pi \left( \frac{D_i}{2} \right)^2
\]  

(14)

where \( s \) is the suction pressure determined from the previous equation and \( \pi (D_i/2)^2 \) is the area of the caisson lid that is acted on by the suction pressure calculated using \( D_i \), which is the internal diameter of the caisson lid, equal to 500 mm (Vaitkunaite et al. 2016).

2.4.4.1 Tensile Load vs. Displacement

Table 2.3 and Figure 2.4 below provide a comprehensive overview of the Vaitkunaite et al. 2016 test results. Figure 2.26 shows the total tensile load as a
function of the pull-out distance for Test v1 through Test v152. The most obvious trend was the parallel increase of tensile load with pull-out rate: the higher the pull-out velocity, the greater the total tensile load required to extract the caisson at said constant pull-out velocity (Vaitkunaite et al. 2016).

Figure 2.26: Load-displacement results for tests v1 through v152 (Vaitkunaite et al. 2016).
The maximum tensile load achieved by each pull-out test (Table 2.3) highlights another pattern: the higher the peak tensile load, the greater the extraction distance required to fully mobilize said peak load. For example, Test v1 developed a relatively small peak tensile load of 4.1 kN after 0.7 mm of pull-out at 1 mm/s, while Test v152 was reported as developing a peak tensile load of 75.17 kN after 68.2 mm of pull-out at 152 mm/s.

Table 2.4: Overview of the observed magnitude of peak tensile load and the distance the caisson was extracted to achieve said peak loads (Vaitkunaite et al. 2016).

<table>
<thead>
<tr>
<th>Pull-out test (mm/s)</th>
<th>Peak Tensile Load (kN)</th>
<th>Uplift at peak (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>v0.01</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>v0.05</td>
<td>2.70</td>
<td>0.7</td>
</tr>
<tr>
<td>v0.1</td>
<td>4.10</td>
<td>0.7</td>
</tr>
<tr>
<td>v1</td>
<td>8.00</td>
<td>3.6</td>
</tr>
<tr>
<td>v10</td>
<td>30.80</td>
<td>16</td>
</tr>
<tr>
<td>v22</td>
<td>44.07</td>
<td>14.7</td>
</tr>
<tr>
<td>v27</td>
<td>48.84</td>
<td>14.3</td>
</tr>
<tr>
<td>v47</td>
<td>65.36</td>
<td>48.8</td>
</tr>
<tr>
<td>v98</td>
<td>71.65</td>
<td>60.5</td>
</tr>
<tr>
<td>v152</td>
<td>75.17</td>
<td>68.2</td>
</tr>
</tbody>
</table>

2.4.4.2 Pore Pressure Suction Development

The results for suction development and suction load were determined using the 11 pore pressure transducers placed along the inner and outer skirts of the model caisson. During extraction, the pore pressure drops associated with suction pressure were always greatest on the interior of the caisson and smallest along the outside skirt (Vaitkunaite et al. 2016). The three slowest tests, v0.01, v0.05, and v0.1, were reported as mobilizing a negligible amount of suction as any changes in pore pressure were too small for the pore pressure transducers to resolve (Vaitkunaite et al. 2016).
Lack of suction mobilization confirmed basic load-bearing theory that these three slow pull-out tests exhibited drained behavior (Vaitkunaite et al. 2016). Table 2.4 shows how for the remainder of the tests, the magnitude of the suction load increased in magnitude with the pull-out velocity. It should be noted that the peak suction load showed only a small increase between test v98 and v152 for it is believed that cavitation occurred during both of these tests (Vaitkunaite et al. 2016). When compared against the total tensile loads presented in Table 2.3, it is clear that the contribution to the total pull-out resistance due to suction is significant for moderate and high extraction velocities.

Table 2.5: Peak suction load and percentage of total load for each pull-out test (Vaitkunaite et al. 2016).

<table>
<thead>
<tr>
<th>Pull-out test (mm/s)</th>
<th>Peak Suction Load (kN)</th>
<th>Percent of total load (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>v0.01</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>v0.05</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>v0.1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>v1</td>
<td>3.98</td>
<td>49.8</td>
</tr>
<tr>
<td>v10</td>
<td>24.64</td>
<td>80.0</td>
</tr>
<tr>
<td>v22</td>
<td>35.37</td>
<td>80.3</td>
</tr>
<tr>
<td>v27</td>
<td>37.71</td>
<td>77.2</td>
</tr>
<tr>
<td>v47</td>
<td>48.87</td>
<td>74.8</td>
</tr>
<tr>
<td>v98</td>
<td>53.45</td>
<td>74.6</td>
</tr>
<tr>
<td>v152</td>
<td>56.17</td>
<td>74.7</td>
</tr>
</tbody>
</table>

2.4.5 Comparison Against Analytical Methods of Houlsby et al. (2005b) and Iskander et al. (2002)

For purposes of evaluating the accuracy of two analytical methods, Vaitkunaite et al. (2016) performed back-calculations of peak suction pressure, total tensile load, and suction force acting on the lid for comparison against observed experimental results. The analytical methods discussed in this section are evaluated as alternatives
to the FEM constructed in this thesis as a method of predicting total tensile capacity as well as contribution due to suction.

### 2.4.5.1 Peak Suction Pressure

One analytical method evaluated is the prediction of peak suction pressure as a function of pull-out rate through the following equation proposed by Houlsby et al. (2005b):

\[
 s = -\frac{\pi \gamma_w \frac{dh}{dt}}{4F k_0 dt}
\]

where \( s \) is peak suction pressure, \( F \) is a dimensionless flow factor, \( \gamma_w \) is the unit weight of seawater, \( k_0 \) is the hydraulic conductivity, \( D \) is the diameter of the caisson, \( \frac{dh}{dt} \) is the pull-out rate. The unit weight of seawater (\( \gamma_w \)), the diameter of the caisson (\( D \)), and the hydraulic conductivity (\( k_0 \)) were constants with values of 1.009×10^{-5} \( \text{N/mm}^3 \), 500 mm, and 0.074 mm/s, respectively. The dimensionless flow factor (\( F \)) was estimated based on the dimensions of the caisson using the following equation, also proposed by Houlsby et al. (2005):

\[
 F = 1.75 + 1.9e^{-5h/D}
\]

where \( h \) is the height of the caisson. Using this equation with a caisson height (\( h \)) of 250 mm and a diameter (\( D \)) of 500 mm yields a dimensionless flow factor of 1.90596.

Figure 2.27 shows how the analytical method proposed by Houlsby et al. (2005b) clearly overpredicts the suction pressure beneath the caisson lid, especially for higher pull-out rates. The first data point, \( v = 1 \text{ mm/s} \), yields the closest prediction of 27.3 kPa of suction, but this value is still 17% higher than the observed suction of 23.4 kPa.
Figure 2.27: Peak suction $s$ back-calculated through the use of Equation 8 (Vaitkunaite et al. 2016).

2.4.5.2 Peak Tensile Capacity

Houlsby et al. (2005b) proposed the following equation to estimate the total tensile capacity in the presence of suction:

$$F_T = -sA \left(1 + \frac{4}{D_i d} (K \tan \delta) \left(\frac{D_o (m^2 - 1)}{4(K \tan \delta)}\right)^2 \left(e^{\frac{4d(K \tan \delta)}{D_i}} - 1 + \frac{4d(K \tan \delta)}{D_i}\right)\right)$$

(17)

where $s$ is suction pressure (estimated using Eq. 8 above or using experimental observations), $A$ is area of the lid, $d$ is the installation depth, $K$ is the coefficient of lateral earth pressure, $\delta$ is the interface friction angle, $D_o$ and $D_i$ are the outer and inner diameters of the caisson, and $m$ is a constant equal to 1.5.

Since the analytical expression for suction pressure (Eq. 15) over-predicted the experimental data by several orders of magnitude, the expression for total tensile capacity above (Eq. 17) also over-estimated experimental data by multiple orders of magnitude. However, as Figure 2.28 shows, if the suction pressure, $s$, was adjusted to
match the experimental observations of Vaitkunaite et al. (2016), then Eq. 17 yielded values of tensile capacity that strongly agreement with experiment.

Another method for estimating the total tensile capacity, proposed by Iskander et al. (2002), calculated the total pull-out load as a sum of the forces listed in the equation below.

\[ F_T = F_s + W_b + W_{plug} + F_{fr,o} \]  (18)

where \( F_T \) is the total force required to uplift the caisson at a given pull-out rate, \( F_s \) is the force due to suction, \( W_b \) is the self-weight of the caisson, \( W_{plug} \) is the weight of the soil within the caisson interior, and \( F_{fr,o} \) is the friction acting on the outer skirt of the caisson. In this experiment, the self-weight of the caisson was always zero and the weight of the soil plug was estimated as 450 Newtons. The friction contribution to the pull-out resistance was estimated using the following equation, also proposed by Iskander et al. (2002):

\[ F_{fr,o} = -\frac{d}{2}A_o(\gamma' + i \cdot \gamma_w)K\tan \delta \]  (19)

where \( d \) is the length of the caisson actually installed into the soil, \( \gamma' \) is the effective unit weight of the soil, \( \gamma_w \) is the unit weight of water, \( A_o \) is the area of the outside caisson skirt that was installed to a depth, \( d, K \) is the coefficient of lateral earth pressure, \( \delta \) is the interface friction angle, and \( i \) is the downward vertical pressure gradient along the outside of the caisson skirt. Houlsby et al. (2005b) suggested that a typical value for \( K\tan \delta \) was 0.5.

Unlike the method proposed by Houlsby et al. (2005b), which used constants that are readily obtainable from the dimensions and soil properties of the experiment, Eq. 18 proposed by Iskander et al. (2002) required experimental data to estimate the contribution due to suction \( (F_s) \) and the pressure gradient term \( (i) \) of Eq. 18. The
suction force on the lid was estimated using experimental data of Vaitkunaite et al. (2016) substituted into Eq. 13 to obtain a load in Newtons. The pressure gradient acting along the caisson exterior \( i \) was calculated using recorded pore pressure data from the transducers distributed along the outer skirt of the caisson (Vaitkunaite et al. 2016). Figure 2.28 below shows that this analytical expression under-predicted experimental observations.

![Figure 2.28: Predicted and measured tensile resistance at different pull-out rates (Vaitkunaite et al. 2016).](image)

2.4.6 Summary

The experimental findings of the scale-model testing conducted by Vaitkunaite et al. (2016) provide valuable insights into the load-bearing response of suction caissons in deep water experiencing extraction at various rates. The express purpose of the experimental campaign was to provide a reference for FE models, such as the one presented in this thesis. At the time of the writing of this thesis, no such FE model has been attempted, despite the fact that the experimental testing conducted by Vaitkunaite...
et al. (2016) utilizes one of the largest pressure testing facilities in the world. Furthermore, the ability to simulate pressures of 20 m of seawater during extraction tests was unique to this testing campaign. Since two analytical methods were shown to roughly approximate the findings of the scale model experiment, the next logical step is to construct an FE model to perform the same task. In the following chapter, the application of the FE method to the problem of simulating the findings of Vaitkunaite et al. (2016) is covered in detail.
Chapter 3

FINITE ELEMENT MODELING

This chapter details the specifications and aspects of the finite element (FE) model developed to reproduce experimental conditions of scale model pull-out tests conducted by Vaitkunaite et al. (2016). The model was constructed, executed, and processed using ABAQUS/CAE Version 6.14.

3.1 Model Geometry and Discretization

3.1.1 Unit Convention

ABAQUS/CAE is able to use any system of units interchangeably, as long as the user is consistent with conversions (ABAQUS, 2014). The model presented in this thesis used SI (mm) units for all numerical quantities (Table 3.1). SI (mm) was selected to keep the simulation well-conditioned. ABAQUS documentation recommends that SI (mm) be used for models in which the stiffness modulus is on the order of MPa and model dimensions are on the order of millimeters, which was the case for the model presented in this thesis (ABAQUS, 2014).

Table 3.1: Unit systems used by ABAQUS/CAE 6.14 (ABAQUS, 2014).

<table>
<thead>
<tr>
<th>Quantity</th>
<th>SI</th>
<th>SI(mm)</th>
<th>SI</th>
<th>US Unit(ft)</th>
<th>US Unit(inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>m</td>
<td>mm</td>
<td>m</td>
<td>ft</td>
<td>in</td>
</tr>
<tr>
<td>Force</td>
<td>N</td>
<td>N</td>
<td>kN</td>
<td>lbf</td>
<td>lbf/ft</td>
</tr>
<tr>
<td>Mass</td>
<td>kg</td>
<td>tonne(10^3kg)</td>
<td>tonne</td>
<td>slug</td>
<td>lbft^2/in</td>
</tr>
<tr>
<td>Time</td>
<td>s</td>
<td>s</td>
<td>s</td>
<td>s</td>
<td>s</td>
</tr>
<tr>
<td>Stress</td>
<td>Pa (N/m²)</td>
<td>MPa (N/mm²)</td>
<td>kPa</td>
<td>lbf/ft²</td>
<td>psi (lbft/in²)</td>
</tr>
<tr>
<td>Energy</td>
<td>J</td>
<td>mJ (10^-³J)</td>
<td>KJ</td>
<td>lbft</td>
<td>inlb</td>
</tr>
<tr>
<td>Density</td>
<td>kg/m³</td>
<td>tonne/mm³</td>
<td>tonne/m³</td>
<td>slug/ft³</td>
<td>lbft²/in³</td>
</tr>
</tbody>
</table>
3.1.2 Axisymmetric Space

If there was a computer with unlimited processing ability, a FE model of a saturated soil would ideally have as many finite elements as there are soil grains and molecules of water to be as accurate as possible. Unfortunately, with access to only limited computational power (a single laptop in this case), there must be a balance between computation cost (due to a high number of elements) and the inaccuracy of the model (due to too few elements) (ABAQUS, 2014). One technique to reduce the number of elements is to take advantage of symmetries, which is possible when considering a suction caisson under tension. In the scale model tests conducted by Vaitkunaite et al. (2016), the caisson only underwent vertical displacement and no lateral displacement, which means that all forces and displacements were symmetric about the center axis of the caisson. To take advantage of this, instead of simulating a 3-dimensional soil base, which would require a higher number of finite elements (20,000 or more), it was possible to utilize a much more efficient 2-dimensional axisymmetric model that arrives at the same result, but with only 8,000 elements. The model geometry was simple in the axisymmetric case and only has two parts; the deformable saturated soil base and the suction caisson.

3.1.3 Suction Caisson

The simulated caisson dimensions were selected to match the small-scale caisson used by Vaitkunaite et al. (2016), except for the thickness of the caisson and the shape of the skirt tip. The caisson was modeled with an inner wall skirt length \((L)\) of 250 meters, an inner lid diameter \((D_i)\) of 500 millimeters, but an artificial thickness of 6 millimeters (Figure 3.1). Vaitkunaite et al. 2016 used a caisson with a thickness of 2 millimeters. It has been shown that giving the suction caisson an artificially thicker
The wall does not significantly affect the ultimate solution, but can greatly ease convergence of the model and increase computational efficiency (Achmus & Thieken, 2014; Mana et al. 2013; Cerfontaine, 2014). The shape of the caisson was also modified in order to avoid a specific drawback of ABAQUS; sharp corners often cause unrealistic stress concentrations that can lead to convergence problems (ABAQUS, 2014; Cerfontaine, 2014). To avoid this, the tip of the caisson was rounded (rather than left square) as shown in the cutout of Figure 3.1a. Since the focus of this model was to simulate soil mechanics rather than the material response of the steel caisson itself, the caisson was defined as a rigid body and constrained to the motion of the reference node shown in Figure 3.1a and 3.1b. Rigid body properties were assigned via constraint, which is discussed in greater detail in Section 3.5: Constraints.

Figure 3.1: Screenshots of (a) the section sketch and dimensions of the caisson in axisymmetric space and (b) visualization of the suction caisson geometry and mesh rotated about the axis of symmetry.
3.1.4 Soil Base

The initial geometry of the model was selected such that the caisson was already installed as installation is not being investigated in this study. As a result, the soil base was dimensioned with a cutout that the caisson fits in, which allowed the soil base to be in full contact with the interior of the suction caisson skirt and lid at the start of simulation (Figure 3.2a and 3.2b). The dimensions of the soil base supporting the caisson were selected to imitate the installation distance reported by Vaitkunaite et al. (2016), listed in Table 2.2 in the previous chapter.

Figure 3.2a shows the model geometry for Test v0.05 specifically, which was installed a distance \(d_{\text{inst}}\) of 239 mm out of a possible 250 mm skirt depth \(D\). A unique model was constructed for each test in order to most accurately reproduce the installation depth reported by Vaitkunaite et al. (2016). The dimensions of the soil base were selected to be large enough to eliminate any possible influence of the boundaries of the model (Mana et al. 2013; Achmus & Thieken, 2014; Sorensen et al. 2016). For the caisson used in this thesis, which had a relatively short skirt length relative to the lid diameter \(L/D\) ratio of 0.5, the soil base was chosen to have a depth \(d_{\text{soil}}\) of 1500 millimeters and a radius of 1500 millimeters.
3.1.5 Water Elements

A key design element of this model was the implementation of a 1.0 mm thick layer of extremely low-stiffness elements that mechanically resemble water. The purpose of these elements was to allow for the expansion of the gap under the lid while maintaining the net suction force acting on the caisson lid. This technique has proven to reliably simulate the negative pore pressure induced during suction caisson pull-out (Mana et al., 2014; Achmus & Thieken, 2014; Sorensen et al., 2016).

In order to ensure proper transference of suction pressure from porous soil below to the caisson lid, the water elements were directly bonded to the soil elements. Full bonding was enabled by initializing the water elements as a partition of the main soil base part. Figure 3.3b and 3.3c shows how the top 1 millimeter of the soil part was
partitioned and meshed separately from the larger soil base, but was still initialized as the same part.

### 3.1.6 Assembly and Mesh

Figure 3.3 below shows the geometry and mesh discretization of the full model assembly, composed of just two parts; the suction caisson and the soil base, which included partitioned water elements at the lid. The orientation of the parts was initialized such that the interior lid and skirt of the caisson were in full contact with the soil base. The mesh density was lowest along the bottom and right boundaries of the soil base, where the least deformation was expected. Mesh density was highest directly beneath the caisson tip and along the skirt of the caisson, where the soil was expected to deform significantly as the caisson is extracted at various rates. The shape of the mesh near the tip of the caisson (Figure 3.3c) was chosen in order to reduce stress concentrations under the tip of the caisson (Cerfontaine, 2014). The mesh of the suction caisson part, which was assigned rigid properties and therefore could not deform, was discretized to align precisely with the nodes of the soil base in order to ease simulation start-up and convergence of initial contact conditions (Sorensen et al. 2016).

#### 3.1.6.1 Mesh Sensitivity

Gourvenec and Randolph (2010) and Mana et al. (2013) determined that the effect of mesh sensitivity on pore pressure mobilization and total tensile capacity was minimal for 2-D axisymmetric FE models of caissons with skirt wall thickness that is less than 0.02 times the caisson diameter. Since this model was constructed with a small skirt thickness ($t$) relative to the caisson diameter ($t/D = 0.012$), the size of the
mesh has very little effect on the results for pore pressure and tensile capacity. The mesh size along the caisson-soil interface was discretized to be 0.025 times the caisson diameter, which was the same proportion used by Achmus & Thieken (2014), Mana et al. (2013), and Gourvenec and Randolph (2010).

![Figure 3.3](image)

Figure 3.3: (a) Screenshot of full model assembly in axisymmetric space with (b), (c), and (d) showing magnified views of areas with high mesh density and the water elements highlighted in red. In this configuration, the model has 8166 elements and 9315 nodes.

### 3.2 Coupled Pore Pressure and Displacement Formulation

The primary focus of this model was to accurately model the behavior of a saturated soil, which is comprised of soil grains with gaps between each grain 100 percent filled with water (i.e. there is no air between grains, only water). Saturated soils represent a challenge to model numerically due to the complex interaction between the pore-fluid and the soil grains that make up the soil skeleton (Helwany, 2007; Dean, 2010; Serbulea, 2013). Continuum elements have been shown to approximate the behavior of soils if the pore pressure can also be accounted for
simultaneously (Helwany, 2007; Serbulea, 2013). A quasi-static u-p formulation was selected, where the u refers the displacement of the soil skeleton and p is the pore-fluid pressure. The basic theory behind a u-p formulation is that the total stress within any finite element is the sum of the effective stress borne by friction and normal force between soil grains and the pressure of the pore-fluid (Verruijt, 2012; ABAQUS, 2014; Sorensen et al. 2016). Using the u-p formulation, ABAQUS solver not only tracks the displacement and stress acting on the soil skeleton, but adds a degree of freedom to the corner node of each element such that the analysis is able to track changes in pore-fluid pressure by applying the effective stress principle (ABAQUS, 2014). The u-p formulation was readily applicable in ABAQUS/CAE in the form of a pore pressure-displacement coupled analysis (ABAQUS, 2014; Sorensen et al. 2016).

3.2.1 Element Type

The quasi-static u-p formulation is built-in to ABAQUS/CAE and available to any license holder (Helwany, 2007; Serbulea, 2013; ABAQUS, 2014). To couple pore pressure and displacement, the elements that were to represent saturated media (e.g. the soil base and water elements in this model) had to be assigned the proper element type. ABAQUS 6.14 comes pre-loaded with an extensive element library that includes a pore-pressure enabled element type for axisymmetric models with the designation CAX4P (ABAQUS, 2014). The “C” stands for continuum element family, the “AX” indicates that the element is designed to be used for axisymmetric models, “4” indicates the number of nodes (one per corner of each square element), and the “P” indicates that the u-p formulation and effective stress principle are applied (ABAQUS, 2014). Selecting the CAX4P element type adds an additional degree of freedom to the corner nodes of each element. Normal CAX4 elements have four corner nodes with
just two active degrees of freedom: displacement in the x- and y- directions, whereas CAX4P corner nodes have pore pressure as an additional active degree of freedom (ABAQUS, 2014).

### 3.2.2 Analysis Steps

In order to utilize the u-p formulation, a compatible numerical analysis had to be selected, for most of ABAQUS/CAE built-in analysis procedures are not designed to account for pore pressure degrees of freedom (ABAQUS, 2014). There are two built-in analysis procedures provided by ABAQUS/CAE that are compatible with CAX4P elements: *GEOSTATIC analysis and transient (also called quasi-static) *SOILS analysis (ABAQUS, 2014). To successfully reproduce the experimental conditions of Vaitkunaite et al. (2016), both analysis types were required in addition to an initial step, in which the model geometry, boundary conditions, predefined fields, and interactions between contacting surfaces were all originally defined.

#### 3.2.2.1 Geostatic Step

Before any simulations of pull-out could take place, the continuum elements representing the soil of the model had to be pre-stressed, as if the soil were real and exposed to gravity and water pressure like the samples utilized by Vaitkunaite et al. (2016). In the scale model experiments, the soil was compacted and pressurized to simulate sand under 20 meters of water (Vaitkunaite et al. 2016). As a result, the initial state of the soil just before pull-out was under pressure from self-weight of the soil, the gravity induced weight of the caisson resting in place, and pressure applied to the top of the soil and caisson lid that simulates 20 meters of seawater. Active and passive lateral earth pressure are known to influence the bearing behavior of suction
caissons and therefore must be included (Stapelfeldt, 2015). Since the geostatic stress state was not reported by Vaitkunaite et al. (2016), an approximation was made.

ABAQUS/CAE has a built-in geostatic step in which the ABAQUS solver attempts to find equilibrium between the applied loads, boundary conditions, and user-defined in-situ stresses (ABAQUS, 2014). The geostatic step also is compatible with pore pressure elements, which means that the solution of the geostatic step applies the effective stress principle in the calculation of the initial stress state (ABAQUS, 2014). Once the geostatic stress within the loaded soil was known, the calculated stress and pore pressure distribution were used as the initial conditions of the final step of the FE analysis; pull-out.

3.2.2.2 Pull-out Step

Once the initial stress state was calculated using a geostatic step, the stress and pore pressure distribution was exported to a separate model, which was responsible for executing the pull-out simulation. The high displacements during the pull-out of the suction caisson were expected to be accompanied by rapid pore pressure changes (Vaitkunaite et al. 2016). The transient *SOILS analysis was selected as the analysis procedure to account for varying seepage velocity in addition to changes in volume of fluid present in continuum elements (ABAQUS, 2014). Since the time increments in this analysis were anticipated to vary over several orders of magnitude, automatic time incrementation was selected as opposed to fixed incrementation, which encountered convergence problems during rapid pore pressure changes (ABAQUS, 2014). In order to reproduce the findings of Vaitkunaite et al. (2016), extraction simulations had to pull the caisson as much as 100 millimeters. Since the velocity of testing varied from 0.05 mm/s to 152 mm/s, the length of simulation to achieve 100 millimeters of pull-
out varied from 2000 seconds to just 0.66 seconds. To mitigate the large differences in simulation duration, the length of the pull-step was set to 10,000 seconds for all tests to ensure adequate displacement could be achieved. However, even though a much longer total step time was utilized, the average simulated uplift was just 31.119 millimeters due to numerical instability. More details on the performance of the model during each pull-out test are included in Chapter 4.

3.3 Material Properties and Parameters

3.3.1 Density

All parts of the model were exposed to the force of gravity and, therefore, had an associated density. Typical values for the density of steel and the density of water were used for the caisson and water parts (Achmus & Thieken, 2014; Sorensen et al., 2016; Mana et al., 2013). The density of the soil was obtained from the report of the laboratory test conducted by Ibsen et al. (2009). Since the suction caisson was constrained as a rigid body (discussed later in Section 3.4.1.), only the soil and water elements under the caisson lid were defined using material laws other than density.

Table 3.2: Density values assigned to corresponding model parts obtained from Vaitkunaite et al. (2016) and Ibsen et al. (2009).

<table>
<thead>
<tr>
<th>Part</th>
<th>Density (tonne/mm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suction caisson</td>
<td>7.951*10⁻⁹</td>
</tr>
<tr>
<td>Soil</td>
<td>2.038*10⁻⁹</td>
</tr>
<tr>
<td>Water elements</td>
<td>1.000*10⁻⁹</td>
</tr>
</tbody>
</table>
3.3.2 **Elastic Properties of Water Elements**

The water elements under the lid of the caisson were assigned elastic properties only, defined using Hooke’s isotropic elasticity. The purpose of the water elements is to transfer pore pressure suction from the porous soil to the rigid caisson without unduly affecting the overall resistance of the caisson (Mana et al. 2013; Achmus & Thieken 2014; Sorensen, 2016). As a linearly elastic material, the expansion of the water elements under the lid is expected to unrealistically increase the total resistance to extraction (Mana et al. 2013). To minimize the excess load caused by extension of the water elements, the water elements were assigned very low values for the Young’s modulus on the order of 0.01 to 0.0001 MPa with a Poisson’s ratio of zero. To investigate the effect of Young’s modulus on the overall solution, each simulation had three additional model versions in which the Young’s modulus of the water elements was varied from 0.01 to 0.0001 MPa.

3.3.3 **Mohr-Coulomb Elasto-plastic Parameters**

In the previous chapter, the equations for the Young’s modulus (E), Poisson’s ratio (ν), friction angle (φ), dilation angle (ψ), and cohesion (c) were presented as functions of the density ratio (DR) and confining pressure (σ) (Ibsen et al. 2009). The soil part (excluding the water elements) was assigned Mohr-coulomb parameters calculated using the density ratio of that specific test (Table 3.3).
Table 3.3: Calculated Mohr-coulomb parameters for the six different density ratio samples used in pull-testing. To account for the effect of varying Young’s modulus, friction angle, and dilation angle, individual models were created for each test version and density ratio.

<table>
<thead>
<tr>
<th>Density Ratio (%)</th>
<th>Test Version</th>
<th>DR</th>
<th>82</th>
<th>83</th>
<th>84</th>
<th>85</th>
<th>86</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus (MPa)</td>
<td>E</td>
<td>50.620</td>
<td>51.845</td>
<td>53.092</td>
<td>54.362</td>
<td>55.655</td>
<td>61.055</td>
<td></td>
</tr>
<tr>
<td>Poisson’s Ratio (-)</td>
<td>ν</td>
<td>0.250</td>
<td>0.250</td>
<td>0.250</td>
<td>0.250</td>
<td>0.250</td>
<td>0.250</td>
<td></td>
</tr>
<tr>
<td>Friction Angle (°)</td>
<td>φ</td>
<td>41.430</td>
<td>41.430</td>
<td>41.540</td>
<td>41.650</td>
<td>41.760</td>
<td>42.200</td>
<td></td>
</tr>
<tr>
<td>Dilation Angle (°)</td>
<td>ϕ</td>
<td>14.579</td>
<td>14.774</td>
<td>14.969</td>
<td>15.164</td>
<td>15.359</td>
<td>20.081</td>
<td></td>
</tr>
<tr>
<td>Cohesion (MPa)</td>
<td>c</td>
<td>0.006144</td>
<td>0.006176</td>
<td>0.006208</td>
<td>0.006240</td>
<td>0.006272</td>
<td>0.006400</td>
<td></td>
</tr>
</tbody>
</table>

3.3.4 Permeability

In addition to the strength properties of the soil, additional care was taken by Vaitkunaite et al. (2016) to quantify the permeability ($k$) of the soil as a function of the void ratio ($e$). In ABAQUS/CAE, isotropic permeability, which relates the flow rate through soil and the pore pressure gradient in all directions, can be defined as a function of the void ratio using experimental data (ABAQUS, 2014). The relationship between the permeability and void ratio for Baskarp Sand No.15 was presented as a quadratic equation, which is printed in Chapter 2 (Sjelmo, 2012).

The permeability was implemented in ABAQUS using tabular data entry of the values in Table 3.4 below. Once input into ABAQUS, the permeability was set by the initial void ratio at the beginning of analysis. During the analysis, as the void ratio of elements changed with volume expansion or contraction, the permeability of the element was assigned a value below based on the corresponding void ratio.
Table 3.4: Permeability for void ratios of 0.540 to 0.870 (Sjelmo, 2012).

<table>
<thead>
<tr>
<th>$e$ (-)</th>
<th>$k$ (mm/s)</th>
<th>$e$ (-)</th>
<th>$k$ (mm/s)</th>
<th>$e$ (-)</th>
<th>$k$ (mm/s)</th>
<th>$e$ (-)</th>
<th>$k$ (mm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.540</td>
<td>0.059088</td>
<td>0.625</td>
<td>0.085625</td>
<td>0.710</td>
<td>0.121988</td>
<td>0.795</td>
<td>0.168177</td>
</tr>
<tr>
<td>0.545</td>
<td>0.060377</td>
<td>0.630</td>
<td>0.087492</td>
<td>0.715</td>
<td>0.124433</td>
<td>0.800</td>
<td>0.171200</td>
</tr>
<tr>
<td>0.550</td>
<td>0.061700</td>
<td>0.635</td>
<td>0.089393</td>
<td>0.720</td>
<td>0.126912</td>
<td>0.805</td>
<td>0.174257</td>
</tr>
<tr>
<td>0.555</td>
<td>0.063057</td>
<td>0.640</td>
<td>0.091328</td>
<td>0.725</td>
<td>0.129425</td>
<td>0.810</td>
<td>0.177348</td>
</tr>
<tr>
<td>0.560</td>
<td>0.064448</td>
<td>0.645</td>
<td>0.093297</td>
<td>0.730</td>
<td>0.131972</td>
<td>0.815</td>
<td>0.180473</td>
</tr>
<tr>
<td>0.565</td>
<td>0.065873</td>
<td>0.650</td>
<td>0.095300</td>
<td>0.735</td>
<td>0.134553</td>
<td>0.820</td>
<td>0.183632</td>
</tr>
<tr>
<td>0.570</td>
<td>0.067332</td>
<td>0.655</td>
<td>0.097337</td>
<td>0.740</td>
<td>0.137168</td>
<td>0.825</td>
<td>0.186825</td>
</tr>
<tr>
<td>0.575</td>
<td>0.068825</td>
<td>0.660</td>
<td>0.099408</td>
<td>0.745</td>
<td>0.139817</td>
<td>0.830</td>
<td>0.190052</td>
</tr>
<tr>
<td>0.580</td>
<td>0.070352</td>
<td>0.665</td>
<td>0.102513</td>
<td>0.750</td>
<td>0.142590</td>
<td>0.835</td>
<td>0.193313</td>
</tr>
<tr>
<td>0.585</td>
<td>0.071913</td>
<td>0.670</td>
<td>0.106652</td>
<td>0.755</td>
<td>0.145217</td>
<td>0.840</td>
<td>0.196608</td>
</tr>
<tr>
<td>0.590</td>
<td>0.073508</td>
<td>0.675</td>
<td>0.108825</td>
<td>0.760</td>
<td>0.147968</td>
<td>0.845</td>
<td>0.199937</td>
</tr>
<tr>
<td>0.595</td>
<td>0.075137</td>
<td>0.680</td>
<td>0.110832</td>
<td>0.765</td>
<td>0.150753</td>
<td>0.850</td>
<td>0.203500</td>
</tr>
<tr>
<td>0.600</td>
<td>0.076800</td>
<td>0.685</td>
<td>0.112727</td>
<td>0.770</td>
<td>0.152572</td>
<td>0.855</td>
<td>0.206697</td>
</tr>
<tr>
<td>0.605</td>
<td>0.078497</td>
<td>0.690</td>
<td>0.112584</td>
<td>0.775</td>
<td>0.154625</td>
<td>0.860</td>
<td>0.210128</td>
</tr>
<tr>
<td>0.610</td>
<td>0.080228</td>
<td>0.695</td>
<td>0.114857</td>
<td>0.780</td>
<td>0.159312</td>
<td>0.865</td>
<td>0.213593</td>
</tr>
<tr>
<td>0.615</td>
<td>0.081953</td>
<td>0.700</td>
<td>0.117200</td>
<td>0.785</td>
<td>0.162333</td>
<td>0.870</td>
<td>0.217092</td>
</tr>
<tr>
<td>0.620</td>
<td>0.083792</td>
<td>0.705</td>
<td>0.119577</td>
<td>0.790</td>
<td>0.165188</td>
<td>0.875</td>
<td>0.220766</td>
</tr>
</tbody>
</table>

It should be noted that the water elements just beneath the lid of the caisson were not assigned the same permeability relationship. Since the void ratio of the water elements was expected to change by several orders of magnitude as a result of rapid expansion, the void ratios listed in Table 3.4 do not cover the correct values (Mana et al. 2013). In addition, since water is much more permeable than soil, the water elements were assigned just one value for permeability of 1000 mm/s at a corresponding void ratio of 1.3 (Mana et al. 2013).

3.4 Boundary Conditions and Constraints

3.4.1 Suction Caisson Rigid Body Constraint

The focus of this FE model and the reference scale model testing conducted by Vaitkunaite et al. (2016) was the simulation and observation of pore pressure mobilization and tensile capacity within the soil itself, ignoring any deformation that occurs within the steel caisson (Vaitkunaite et al. 2016). To save time on model simulations, the elements assigned to the caisson part were constrained as a rigid body.
such that, the ABAQUS solver did not attempt to calculate the material response of the steel caisson in response to encountered stresses (ABAQUS, 2014). The constraint was applied to the body elements of the caisson part, which was then constrained to the motion of a single reference point (ABAQUS, 2014). Though the location of the reference point is irrelevant, for all models in this thesis the reference point was placed 100 millimeters directly above the axis of symmetry and the centerline of the caisson for easier visualization (Figure 3.1 and 3.3).

3.4.2 Water Elements Tie Constraint

To simulate the effect of suction pressure and volume expansion of elements acting on the interior of the caisson, a tie constraint was used. This technique was used in similar FE models constructed by Cao et al. (2002), Mana et al. (2013), Achmus & Thieken (2014), Cerfontaine (2014) and Sorensen et al. (2016). Without a tie constraint, no suction is mobilized because there is no interaction between the caisson and porous soil below (Cao et al. 2000; Mana et al. 2013; Achmus & Thieken, 2014). To enforce pore pressure continuity, a tie constraint was formulated using the nodes along the bottom of the caisson lid as a master surface and the nodes along the top of the water layer as a slave surface (Figure 3.4). The distance between the nodes was constrained to a value of zero for all analysis steps to ensure that the motion of the caisson and generated suction are was directly transferred to the transfer of pore pressure suction and expansion of the porous media as the caisson was extracted.
3.4.3 Soil and Suction Caisson Displacement Boundary Conditions

In the model experiment conducted by Vaitkunaite et al. (2016), when the caisson was extracted from the soil at various rates, the edges and boundary of the soil sample were held in place against the motion of the caisson by gravity and the confines of the pressure vessel. In modeling space, the soil base also had to be fixed using boundary conditions to provide reaction forces against the pull-out deformation (ABAQUS, 2014). To fix the soil base in the x- and y- directions, a displacement/rotation boundary condition was placed on the base of the soil (Figure 3.5). Using an identical displacement/rotation boundary condition, the right boundary (in axisymmetric space) of the soil base was prevented from displacing in the x-direction, but allowed to deform freely in the y-direction.

The rigid body motion of the caisson, which was controlled by the reference node along the axis of symmetry, was also constrained against horizontal motion.
Since the model is formulated in axisymmetric space, the reference point, which was directly above the axis of symmetry, could not be allowed to displace any amount in the x-direction for this would be non-physical in axisymmetric space. For the same reasons, the rotation of the reference node about the z-axis was also fixed. If left unfixed, rotations occurred during pull-out simulation that resulted in the folding of the rigid caisson at the center. Applying a boundary condition that fixes both rotation about the z-axis and displacement in the x-direction prevented such motion. Finally, the soil nodes along the axis of symmetry were prevented from moving in the x-direction as this motion is also non-physical in axisymmetric space.

Figure 3.5: Diagram of model assembly in 2-D space showing applied boundary conditions.
3.4.4 Pore Pressure Boundary Conditions

To initialize the pore pressure distribution within the saturated soil base, boundary conditions were used. During all experimental tests by Vaitkunaite et al. (2016), the pressure inside the tank was set to a total pressure of approximately 300 kPa, which simulated the pore pressure distribution found in saturated soil under 20 meters of seawater. The pore pressure was fixed at a node using boundary conditions to simulate the effects of a free draining surface along the top, sides, and base of the soil part (Figure 3.5). The pore pressure at the top, sides, and bottom of the soil part were calculated using the following equations:

\[ PP_{\text{bottom}} = \gamma_w \cdot (H_w + d_{\text{soil}}) \]  \hspace{1cm} (20)

\[ PP_{\text{top}} = \gamma_w \cdot (H_w) \]  \hspace{1cm} (21)

\[ PP_{\text{side}}(y) = \gamma_w \cdot (H_w + d_{\text{soil}} - y) \text{ for } 0 \leq y \leq 1500 \]  \hspace{1cm} (22)

where \( H_w \) is the depth of the water above the soil part equal to 20,000 mm, \( \gamma_w \) is the unit weight of the water equal to \( 9.81 \times 10^{-6} \) N/mm\(^3\), \( d_{\text{soil}} \) is the depth of the soil part equal to 1,500 mm, and \( y \) is the depth in millimeters used to calculate the pore pressure at any depth along the side boundary of the soil part. The equation for the pore pressure along the side of the soil part is a function of the height above the soil part bottom, \( y \). The side pore pressure function was applied as a boundary condition using an analytical field (ABAQUS, 2014).

All pore pressure boundary conditions were active during both the geostatic and pull-out steps of the analysis. However, it should be noted that the top pore pressure boundary condition was only applied to the soil adjacent to the caisson lid (as opposed to applying the boundary condition to the soil under the caisson lid as well) to ensure that the only possible seepage path was through the soil base and along the boundaries of the caisson.
3.4.5 Pull-out Boundary Condition

In the scale model testing by Vaitkunaite et al. (2016), the actuator responsible for pulling out the scale model caisson was controlled by displacement rather than load magnitude. In other words, the displacement rate was kept constant while the resultant force required to maintain the current pull-out rate was recorded as a function of the amount of displacement achieved (Vaitkunaite et al. 2016). The same concept was applied in ABAQUS/CAE during the pull-out step using a displacement-controlled boundary condition applied to the reference node of the caisson (Figure 3.5). Since the caisson was constrained as a rigid body to the motion of the reference node, any displacements applied to the reference node were also applied to the caisson. In total, there were nine pull-out velocities (in mm/s) investigated in this analysis that correspond to tests conducted by Vaitkunaite et al. (2016): 0.05, 0.1, 1.0, 10, 22, 27, 47, 98, and 152 mm/s. Each model version therefore had a constant-velocity boundary condition that corresponds to the pull-out velocity. In the context of the model, the boundary condition was applied in the positive y-direction, which corresponds to vertical pull-out.

To ease simulation convergence, the pull-out boundary condition was applied gradually using a user-defined amplitude function. Otherwise, the analysis solver attempted to instantaneously apply a constant velocity to the initially stationary caisson, which usually resulted in simulation termination. To gradually increase the rate of pull-out ($u$) from zero to the desired extraction velocity ($v$) over 2 milliseconds (ms), the following piecewise amplitude function was assigned to the pull-out boundary condition:

$$
\dot{u}(t) = \begin{cases} 
(0.5 - 0.5 \cos \left( \frac{\pi}{2 \text{ms}} \right)) v & \text{for } t < 2 \text{ ms} \\
\frac{v}{2} & \text{for } t \geq 2 \text{ ms}
\end{cases}
$$

(23)
This amplitude function was identical to the one used in the FE model constructed by Sorensen et al. (2016), which was shown to improve simulation start-up and convergence. After the initial ramp over 2 ms, the velocity was kept constant until simulation termination.

3.5 Loads

In order to reproduce the geostatic stress and pore pressure distribution of pre-stressed soil, four external loads were balanced over the course of the geostatic step before any pull-out simulation was initiated:

2. Gravity-induced weight of caisson resting in place.
3. Pressure applied to the top of the soil base to simulate the weight of 20 meters of seawater and one atmosphere of air pressure.
4. Pressure applied to the top of the suction caisson lid to simulate the weight of 20 meters of seawater and one atmosphere of air pressure.

Gravity acting on the saturated soil and suction caisson was simulated using a body force. The magnitude of a body force is specified in ABAQUS in units of force per unit volume and is calculated by multiplying the density of the material (listed in Table 3.3) by the acceleration due to gravity, which is 9,810 mm/s². The body force was implemented in the negative y-direction. Loads 3 and 4 above were initialized using a mechanical pressure load type. The magnitude of the pressure across the top of the soil and caisson lid was uniformly distributed and equal to the sum of the pressure of air and water. The pressure due to the 20 meters of water was approximately 0.200 MPa and the pressure of one atmosphere was approximately 0.100 MPa: magnitude
0.300 MPa pressure acting on the lid of the caisson and surrounding soil. Each of the 4 loads above was active during the geostatic step, during which the internal stresses of the soil were calculated. The self-weight of the caisson is deactivated for the pull-step since Vaitkunaite et al. (2016) zeroed the weight of the caisson on the load actuator.

3.6 Predefined Fields

3.6.1 Saturation

Predefined fields are used to provide initial values for certain variables (ABAQUS, 2014). When using CAX4P elements, specifically, one of the required initial quantities is the saturation, which defines the percentage of the volume of the porous material is composed of liquid (ABAQUS, 2014). In this model, the soil was assumed to be 100% saturated with fluid water with no gaseous component (Vaitkunaite et al. 2016). As a result, a saturation of 1.0 was assigned to all elements in the model, with the exception of the rigid caisson part which lacked an active pore pressure degree of freedom.

3.6.2 Void Ratio

Similar to the saturation property, the void ratio is another initial quantity that must be predefined in order to use CAX4P elements (ABAQUS, 2014). The void ratio is defined as the ratio of the volume of voids to the volume of soil particles, which is measured using laboratory testing of the soil sample in question (Sjelmo, 2012). Since tabular permeability was applied in all models (Section 3.3.4) the initial void ratio determines the permeability of the soil at the start-up of simulation. To investigate the influence of initial void ratio (and therefore the initial permeability), a parametric
study was conducted by varying the initial void ratio between values of 0.549 and 0.858 for all nine pull-out test velocities.

3.6.3 Initial Stress Distribution of the Geostatic Step

Since the initial geostatic stress state was not reported by Vaitkunaite et al. (2016), an approximation was made using the built in *GEOSTATIC analysis. This type of analysis requires an initial prediction of the geostatic stress that is close to the expected solution (Roberts & Britto, 2012; ABAQUS, 2014). The more accurate the initial stress prediction is, the fewer corrections must be made during analysis when balancing the applied loads and pore pressure boundary conditions against the internal stresses within the soil (Roberts & Britto, 2012; ABAQUS, 2014). The estimate of the initial stress was implemented in ABAQUS using a geostatic stress field, which is a special type of predefined field developed specifically for this geostatic analysis (ABAQUS, 2014).

To accurately simulate a geostatic stress state, the stress field was applied uniformly in the x-direction, but defined as increasing in pressure with depth into the soil base (Helwany, 2007; Dean, 2010; Serbulea, 2013). The stress field in the y-direction increases with depth linearly, based on two user-provided stress coordinates. Figure 3.6 shows the user provided stress and pore pressure distribution used to calculate the initial geostatic step.
The horizontal stress is defined using the lateral earth pressure coefficient ($K_0$) which is calculated using the following equation developed by Jaky (1944):

$$K_0 = 1 - \sin(\varphi)$$  \hspace{1cm} (24)

where ($\varphi$) is the angle of internal friction, calculated using the density ratio specified by Vaitkunaite et al. (2016) for each test (Table 3.5). The results of the geostatic step and the accuracy of the estimates for the initial effective stress field are discussed in Chapter 4.

Table 3.5: Model specifications for the nine geostatic analyses conducted.

<table>
<thead>
<tr>
<th>Pull-out test (mm/s)</th>
<th>Installation depth (mm)</th>
<th>Density Ratio (%)</th>
<th>Friction Angle ($^\circ$)</th>
<th>Coeff. Lateral Earth Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>239</td>
<td>85</td>
<td>41.650</td>
<td>0.335421</td>
</tr>
<tr>
<td>0.1</td>
<td>241</td>
<td>86</td>
<td>41.760</td>
<td>0.333988</td>
</tr>
<tr>
<td>1</td>
<td>242</td>
<td>90</td>
<td>32.300</td>
<td>0.465648</td>
</tr>
<tr>
<td>10</td>
<td>242</td>
<td>90</td>
<td>32.300</td>
<td>0.465648</td>
</tr>
<tr>
<td>22</td>
<td>236</td>
<td>83</td>
<td>41.430</td>
<td>0.338295</td>
</tr>
<tr>
<td>27</td>
<td>239</td>
<td>85</td>
<td>41.650</td>
<td>0.335421</td>
</tr>
<tr>
<td>47</td>
<td>236</td>
<td>83</td>
<td>41.430</td>
<td>0.338295</td>
</tr>
<tr>
<td>98</td>
<td>239</td>
<td>82</td>
<td>41.430</td>
<td>0.338295</td>
</tr>
<tr>
<td>152</td>
<td>236</td>
<td>84</td>
<td>41.540</td>
<td>0.336857</td>
</tr>
</tbody>
</table>
3.6.4 Imported Stress Distribution of Pull-out Step

The geostatic stress state determined during the first analysis step was used as the initial conditions of the pull-step through an output database (.odb) predefined field. As long as the odb of the geostatic step existed within the current ABAQUS working directory, the final effective stress and the pore pressure distribution calculated by the geostatic step were imported into a separate model that performed the pull-out analysis. In this manner, only one geostatic model was constructed and executed for each load test rate, drastically saving computation time by eliminating the need to re-run a geostatic step each model. It should be noted that importing the results to a new model was not successful if the mesh discretization of the geostatic model and the pull-out model did not match exactly.

3.7 Interactions and Contact Formulation

In a numerical model such as the one developed in this thesis, without a defined contact formulation between identified surfaces, the parts of the model will not interact with each other (ABAQUS, 2014). Before any contact formulations or interaction properties could be applied, the surfaces of the parts that may interact over the course of the simulation were identified and defined as contact pairs (ABAQUS, 2014). In this model, three surfaces of the suction caisson were expected to be in contact with the deformable soil base: the lid, the inner wall of the skirt, and the outer wall and tip of the skirt. As shown in Figure 3.6 below, each surface on the suction caisson part was assigned a corresponding contact pair on the soil base part. The soil was in direct contact with the inner skirt, outer skirt, and tip of the suction caisson while the underside of the suction caisson lid was in contact with the thin layer of water elements. Self-contact was also defined along the inner and outer soil of the
caisson skirt, but only for the pull-out step. Self-contact was needed during the pull-
step because the soil on either side of the caisson skirt was expected to collapse into
the gap left by the caisson skirt as it was extracted from the soil base.

The surfaces in contact were discretized into surface pairs using the master-
slave surface relationship. As this model only had two parts, of which one was
constrained as a rigid body, the master slave surface in contact was always the rigid
suction caisson (ABAQUS, 2014). For the self-contact condition, which was not
active during the geostatic step, only one surface definition was required: the inner and
outer walls along the caisson skirt.

![Diagram showing the three contact pair definitions rotated about the axis of rotation to display the entire caisson in 2-d space.](image)

The contact formulation and interface parameters for each of the contact pairs
is listed in Table 3.6 below. During the geostatic step, penalty friction behavior was
assigned to the inner and outer skirt contact pairs in order to build tangential stress
within the soil elements. During the pull-out step, the tangential behavior was
switched to fully-smooth, frictionless behavior to simulate steady seepage of pore fluid along the interface between the skirt wall and soil (Mana et al. 2013).

Table 3.6: Interaction and contact parameters for each interface shown in Figure 3.6.

<table>
<thead>
<tr>
<th>Contact pair (Master/Slave)</th>
<th>Sliding formulation</th>
<th>Tangential behavior</th>
<th>Normal behavior</th>
<th>Separation after contact</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Geostatic Step</td>
<td>Pull-out step</td>
<td>Geostatic Step</td>
<td>Pull-out step</td>
</tr>
<tr>
<td>Self-Contact (skirt)</td>
<td>(Inactive)</td>
<td>Finite sliding</td>
<td>(Inactive)</td>
<td>Frictionless</td>
</tr>
<tr>
<td>Outer skirt and tip/soil</td>
<td>Small sliding</td>
<td>Finite sliding</td>
<td>Isotropic penalty friction (Fr. Coeff. = 0.38)</td>
<td>Frictionless</td>
</tr>
<tr>
<td>interface</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner skirt/soil interface</td>
<td>Small sliding</td>
<td>Isotropic penalty friction (Fr. Coeff. = 0.38)</td>
<td>Frictionless</td>
<td>Hard contact</td>
</tr>
<tr>
<td>Lid/water interface</td>
<td>Small sliding</td>
<td>Frictionless</td>
<td></td>
<td>Hard contact</td>
</tr>
</tbody>
</table>

3.8 Analysis Output and Processing

To view the results of a simulation, a field output request was created in order to define which variables were written to the output database file (ABAQUS, 2014). Since the overarching goal of this model design was to replicate the observations and results of Vaitkunaite et al. (2016), the following variables requested by the field output are used to reproduce the reported soil displacement, pore pressure observations, back-calculated suction force, and total pull-out load as a function of uplift:

- U - Nodal displacements, x- and y- component
- S - Stress,
- POR - Pore pressure
- RF - Reaction force at fixed/reference nodes, x- and y- component
- VOIDR - Void ratio
Each of the above variables was requested to be output from the entire model domain for every increment of the geostatic and pull-out analyses.

3.8.1 Soil and Suction Caisson Displacement

To quantify the motion of soil in and around the caisson during pull-out simulations, the variable for displacement (U) was requested to be output from all deformable soil elements that comprise the soil base part. The displacement of any node (U) had a component for motion in the x-direction (U1) and a component for motion in the y-direction (U2) in order to track the motion of any given node in axisymmetric space (ABAQUS, 2014). The uplift of the caisson, which was used as the x-coordinate when plotting results for total tensile capacity, pore pressure, and suction pressure, was obtained by requesting the output of the variable for displacement in the y-direction (U2) for the reference node of the rigid caisson. Upon simulation completion, the displacement in the y-direction (U2) of the reference node is exported to Excel using the export tool in ABAQUS/CAE (ABAQUS, 2014).

3.8.2 Total Pull-out Load

One of the primary goals of this thesis was to reproduce the load-displacement relationship for suction caissons under various pull-out rates observed by Vaitkunaite et al. (2016). Vaitkunaite et al. (2016) measured the total load as a function of displacement using a load-actuator that measured the force required to maintain a constant-velocity extraction of the caisson. The same principle was applied to the FE model through the use of the rigid body constraint and reference node. The motion of the caisson is constrained to the motion of the reference node, to which a boundary condition has been applied to lift the caisson at constant velocity in the positive y-
direction. The variable, RF, which stands for reaction force, is an available output variable that tracks the load acting on the reference node of the suction caisson in reaction to the pull-out boundary condition (ABAQUS, 2014). By default, the reaction force (RF) has two components: RF1 is the reaction force in the x-direction and RF2 is the reaction force in the y-direction (ABAQUS, 2014). Since the caisson was fixed from moving in the x-direction, RF2 was the true tensile load required to extract the caisson at the specified constant rate. Upon simulation completion, the reaction force at the reference node in the y-direction (U2) was exported to Microsoft Excel using the export plug-in tool. The load-displacement curve for each pull-out test was generated by plotting the reaction force (RF) of the reference node as a function of the displacement in the y-direction (U2) of the same reference node.

3.8.3 Pore Pressure and Suction Load

Simulation of pore pressure changes was a key feature of this FE model design and the primary focus of the scale-model investigation conducted by Vaitkunaite et al. (2016). To observe changes in pore pressure at various nodes calculated by the ABAQUS solver, the variable for pore pressure (POR) was requested as an output variable for all elements defined as porous. By default, the pore pressure was requested from every node in the model’s domain. However, since the suction caisson was discretized with non-pore pressure coupled elements, no pore pressure information was output for these elements. The output of the pore pressure variable for all nodes in the soil domain was used to generate contour plots of the mobilized pore pressure gradient.

For back-calculation of the suction load, the pore pressure changes at a specific node were requested and exported to Microsoft Excel using the export plug-in tool.
The location of this specific node corresponded to the location of pore pressure transducers PP7 and PP9 used by Vaitkunaite et al. (2016), which were located just under the lid of the caisson. Under the lid of the caisson was observed in both experiment and simulation as experiencing the highest pore pressure drop in response to caisson extraction (Vaitkunaite et al. 2016). The suction pressure and suction force at any time during the simulation was calculated from the measured pore pressure using the following equations:

Suction pressure at any time t (MPa)

\[ s(t) = \gamma_w \cdot H_w - p_{pp\text{node}}(t) \]  \hspace{1cm} (25)

Suction force at any time t (N)

\[ F_s(t) = s(t) \cdot \pi(D_l/2)^2 \] \hspace{1cm} (26)

where \( p_{pp\text{node}}(t) \) is the value of the pore pressure of the lid node at time \( t \), \( H_w \) is the depth of the water above the soil part equal to 20,000 millimeters, \( \gamma_w \) is the unit weight of the water equal to \( 9.81 \times 10^{-6} \) N/mm\(^3\), \( D_l \) is the inner diameter of the caisson equal to 500 millimeters.
In this chapter, the results of the finite element modelling campaign described in the previous chapter are presented. Back-calculations of the scale model tests done by Vaitkunaite et al. (2016) were carried out for the purpose of assessing the validity and accuracy of the numerical formulation used.

4.1 Geostatic Step

Before pull-out deformations were simulated, a calculated geostatic step was necessary in order to establish the initial stress distribution within the soil in response to the loads and boundary conditions. A separate geostatic model was built for each of the 9 tests listed in Table 3.5 in order to match the reported installation depth ($d_{inst}$), density ratio ($D_R$), and coefficient of lateral pressure ($K_0$) of each test. Figure 4.1 shows the vertical effective stress and the pore pressure distribution calculated at the end of the geostatic step for test number v0.05, which was reported as having an installation depth of 241 millimeters and a density ratio of 85%.

The converged solution for vertical stress and pore pressure increases linearly with depth into the soil base, which agrees with the predicted stress distribution formulated using fundamental geostatic theory (Roberts & Britto, 2012). In addition, the stress and pore pressure distributions have contour lines that are perpendicular to the y-axis, with the exception of the soil interacting with the caisson skirts. For example, in the soil two millimeters from the caisson tip, shown in higher magnification in Figure 4.1(a), there are pockets of low stress on either side of the tip separated by smaller, high stress concentrations. Since the total stress is the sum of the
effective stress and the pore pressure, the low effective stress pockets near the caisson results in slightly higher pore pressure at the tip, shown in the magnified Figure 4.1(b).

Figure 4.1: Output results for test v0.05 which was installed \(d_{\text{inst}}\) 241 millimeters into \(D_R=85\%\) sand (a) the effective stress in the vertical direction, where negative indicates downward, expressed in megapascals \(10^6\) N/m\(^2\) and (b) pore fluid pressure, also expressed in MPa.
The stress and pore pressure distributions calculated by the geostatic step were based on user-provided initial predictions. The vertical displacements (Figure 4.2) are due to differences between the predicted stresses and the actual converged stresses reached by the analysis. Smaller displacements indicate fewer errors in the initial prediction of the stress state. The smallest displacements shown in Figure 4.1(c) are on the order of $10^{-6}$ millimeters and located near the bottom of the model, where the stress and pore pressure distribution are nearly unaffected by the presence of the caisson. Near the caisson tip, however, where the largest differences between predicted and converged stress states exists, the maximum displacement is on the order of $10^{-4}$ millimeters. This deformation is negligible in comparison to the displacements induced during the pull-out step, which are on the order of 10 to 150 millimeters. As a result, the displacements shown above are sufficiently small to ensure convergence of the next analysis step (Roberts & Britto, 2012).

Figure 4.2: Geostatic analysis output results for test v0.05 which was installed ($d_{\text{inst}}$) 241 millimeters into $D_R=85\%$ sand showing vertical displacement in mm.
4.2 Simulation of Pull-out at Nine Different Rates

The pull-out simulation of the suction caisson was carried out using the solution of the geostatic step as the initial conditions for the pull-out step. The results of each simulation were organized by pull-out speed and each test was compared against the results of Vaitkunaite et al. (2016). For each simulation, the deformed geometry and pore pressure distribution after pull-out were examined and presented in the form of screenshots and plots of the pull-out resistance and pore pressure as a function of the vertical displacement achieved. A total of six model permutations are presented for each pull-out speed for purposes of investigating the influence of the Young’s modulus ($E$) of the “water” elements and the initial void ratio (VR) on the load vs. displacement relationship and the formation of negative pore pressures. To investigate the influence of initial void ratio, Models 1 - 4 varied the initial void ratio between 0.549 and 0.858, whilst the water elements were defined with a very low stiffness of $E = 0.0001$ MPa. The influence of the Young’s modulus ($E$) was observed by comparing the results of Model 2, 5, and 6, which were conducted using fixed void ratios of 0.600 whilst increasing the stiffness of the water elements by one order of magnitude, respectively.

4.2.1 Test v0.05 and Test v0.1

The results of Test v0.05 and Test v0.1, which are the slowest pull-out simulations, are presented simultaneously as their behavior was similar. Table 4.1 and 4.2 show the duration and total pull-out achieved by six model versions simulated at 0.05 mm/s and 0.1 mm/s pull-out rate. At such slow rates of pull-out, longer simulations were required.
Table 4.1: A list of all model versions for Test v0.05.

<table>
<thead>
<tr>
<th>Model</th>
<th>Initial Void Ratio (unitless)</th>
<th>Young's modulus, $E_w$ (MPa)</th>
<th>Pull-out step time (s)</th>
<th>Number of increments</th>
<th>Caisson uplift (mm)</th>
<th>Soil uplift (mm)</th>
<th>Soil uplift (% of caisson uplift)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.549</td>
<td>0.0001</td>
<td>142.700</td>
<td>207</td>
<td>7.135</td>
<td>0.361</td>
<td>5.1%</td>
</tr>
<tr>
<td>2</td>
<td>0.600</td>
<td>0.0001</td>
<td>142.671</td>
<td>205</td>
<td>7.134</td>
<td>0.358</td>
<td>5.9%</td>
</tr>
<tr>
<td>3</td>
<td>0.704</td>
<td>0.0001</td>
<td>130.000</td>
<td>156</td>
<td>6.500</td>
<td>0.340</td>
<td>5.2%</td>
</tr>
<tr>
<td>4</td>
<td>0.858</td>
<td>0.0001</td>
<td>142.400</td>
<td>187</td>
<td>7.120</td>
<td>0.350</td>
<td>4.9%</td>
</tr>
<tr>
<td>5</td>
<td>0.600</td>
<td>0.0100</td>
<td>190.500</td>
<td>231</td>
<td>9.525</td>
<td>9.459</td>
<td>99.3%</td>
</tr>
<tr>
<td>6</td>
<td>0.600</td>
<td>0.0010</td>
<td>190.200</td>
<td>228</td>
<td>9.510</td>
<td>8.611</td>
<td>90.6%</td>
</tr>
</tbody>
</table>

Table 4.2: A list of all model versions for Test v0.1.

<table>
<thead>
<tr>
<th>Model</th>
<th>Initial Void Ratio (unitless)</th>
<th>Young's modulus, $E_w$ (MPa)</th>
<th>Pull-out step time (s)</th>
<th>Number of increments</th>
<th>Pull-out achieved (mm)</th>
<th>Soil Uplift (mm)</th>
<th>Soil uplift (% of caisson uplift)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.549</td>
<td>0.0001</td>
<td>71.320</td>
<td>139</td>
<td>7.132</td>
<td>0.370</td>
<td>5.2%</td>
</tr>
<tr>
<td>2</td>
<td>0.600</td>
<td>0.0001</td>
<td>65.187</td>
<td>94</td>
<td>6.519</td>
<td>0.351</td>
<td>5.4%</td>
</tr>
<tr>
<td>3</td>
<td>0.704</td>
<td>0.0001</td>
<td>71.380</td>
<td>137</td>
<td>7.138</td>
<td>0.354</td>
<td>5.0%</td>
</tr>
<tr>
<td>4</td>
<td>0.858</td>
<td>0.0001</td>
<td>71.620</td>
<td>110</td>
<td>7.162</td>
<td>0.348</td>
<td>4.9%</td>
</tr>
<tr>
<td>5</td>
<td>0.600</td>
<td>0.0100</td>
<td>95.360</td>
<td>194</td>
<td>9.536</td>
<td>9.470</td>
<td>99.3%</td>
</tr>
<tr>
<td>6</td>
<td>0.600</td>
<td>0.0010</td>
<td>95.200</td>
<td>187</td>
<td>9.520</td>
<td>8.620</td>
<td>90.5%</td>
</tr>
</tbody>
</table>

As the two slowest pull-out simulations, the behavior of Test v0.05 and Test v0.1 was expected to be fully-drained such that little to no suction should be generated and formation of a gap between the lid and the soil should be immediate (Vaitkunaite et al. 2016). For Models 1-4 of both tests, which had the same stiffness ($E_w = 0.0001$ MPa) but varied initial the void ratio, the uplift of the soil plug was small (less than 5% of total pull-out) in comparison to the amount that the caisson was pulled out (Table 4.1 and 4.2). For example, Model 1 of Test v0.05 simulated 0.05 mm/s pullout over 142.7 seconds, which lifted the caisson itself a total of 7.134 millimeters in that time period. In comparison, the soil inside the caisson closest to the lid was lifted only 0.38 millimeters by the end of the simulation. The gap between the soil (which was
only lifted 0.38 mm) and the lid (which was lifted 7.134 mm) was maintained by the extension of the low stiffness water elements as shown in Figure 4.3a and 4.3b. During extension, the ratio of volume of water to volume of soils (void ratio) of the low stiffness water elements increased more than 20-fold from the initial value of 0.600 to a final value of 11.441. As Table 4.1 and 4.2 show, increasing the void ratio from Model 1 to Model 4 resulted in slightly less soil uplift. It was observed that increasing the stiffness of the water elements in Model 5 and 6 of both Test v0.05 and Test v0.1 prevented the expansion of the water elements within the gap between soil and caisson lid and caused the soil to lift as much as 99.3% of the total pull-out distance (Table 4.1 and 4.2).

Figure 4.3: Screenshot of the output of Model 1 of (a) Test v0.05 and (b) Test v0.1 highlighting the maximum soil uplift achieved (VR = 0.549, $E_w = 0.0001$).
The load vs. displacement plots for both Test v0.05 and Test v0.1 are shown in Figure 4.4a and 4.4b. Validation of the results of Test v0.05 and Test v0.1 were limited as they are tests for which Vaitkunaite et al. presented only a single data point which was plotted on the load vs displacement curves shown in Figure 4.4a and 4.4b. No results were given for the pore-pressure development because measured changes in pore pressure were small enough to be obscured by sensor drift, which was reported over a range of 0.5 to -1.7 kPa (Vaitkunaite et al. 2016). The unrealistic lifting of the soil elements observed in Models 5 and 6 of both Test v0.05 and Test v0.1 resulted in more than double the pull-out resistance found for Models 1-4. The load-displacement plots for Models 1-4 of Test v0.05 and Test v0.1 are similarly shaped; there is an initial, steep linear increase in pull-out resistance followed by an approximately parabolic curve with gradually decreasing slope (Figure 4.3b). The initial void ratio, which is varied between 0.549 and 0.858 across Models 1 through 4, offset the load vs displacement curve such that, the lower the initial void ratio of the soil, the higher the total load that was required to achieve pull-out at a constant rate.

![Figure 4.4: Total Load vs Displacement results for caisson undergoing (a) 0.05 mm/s and (b) 0.1 mm/s pull-out for all six models with initial conditions listed in Table 4.1 and Table 4.2.](image-url)
The difference in total resistance between Models 1 – 4 of Test v0.05 and Test v0.1 was the result of differences in pore pressure generated beneath the lid of the caisson, which also showed dependence on initial void ratio. Contrary to suction caisson load-bearing theory, which states that a suction caisson undergoing uplift will create a pocket of negative pore pressure within the interior, all six models simulating 0.05 mm/s pull-out predicted that the pore pressure within the caisson actually increased, implying that pore water flowed out of the caisson instead of in (Houlsby et al. 2005b; Senders, 2009). As Figure 4.5a shows, the higher pore pressure region beneath the caisson appeared to move into the caisson interior during pull-out. In this manner, instead of resisting pull-out, the excess pore pressure generated actually assists the pull-out mechanism depending on the void ratio. Figure 4.5b shows a plot of the simulated pore pressure during Test v0.05 in which all model versions displayed counter-intuitive development of excess pore pressure instead of negative suction pressure. It was also observed that the greater the void ratio, the greater the increase in pore pressure in response to pull-out.

Simulations of Test v0.1 resulted in identical pore pressure changes to Test v0.05, except that Models 1 and 2 of Test v0.1, which were defined with the lowest values for initial void ratio, showed expected negative pore pressure mobilization, as shown in Figure 4.5c and 4.5d. Instead of positive excess pore pressure, Models 1 and 2 of Test v0.1 predicted a decrease in pore pressure which corresponds to suction, which is shown by the dark blue area in Figure 4.5c. The mobilization of suction for these two tests is why the peak tensile load is greatest for Models 1 and 2 on Figure 4.4b.
Figure 4.5: Screenshot of the output of Model 1 for (a) Test v0.05 and (c) Test v0.1 highlighting the distribution of pore pressure at the end of the simulated pull step and Lid Pore Pressure vs Displacement results for caisson undergoing (b) 0.05 mm/s pull-out and (d) 0.1 mm/s pull-out with initial conditions listed in Table 4.1 and 4.2.

When plotted against the findings of Models 1-4 of Test v0.05 and Test v0.1 (Figure 4.4a and 4.4b), it is clear that all model versions fail to reproduce the single
data point observed by Vaitkunaite et al. (2016). The failure of the models in this case was most likely due the exclusion of interface fluid flow and friction, which dominate the tensile resistance for slow pull-out tests (Iskander et al. 2002; Senders, 2008). The numerical formulation chosen here emphasizes the contribution of pore pressure changes to total resistance rather than friction development. In similar modeling campaigns in which an identical transient u - p formulation was used, an inexplicable increase in pull-out resistance was also observed when simulating pull-out at an extremely low rate of $10^{-3}$ mm/s (Sorensen et al., 2016).

4.2.2 Test v1

Test v1 was the first series of models simulated at a moderate rate of pullout of 1 mm/s, which is 10 times faster than Test v0.1 and 20 times faster than Test v0.05. Table 4.3 below shows the six model versions of Test v1. Models 1-4 were conducted over an average of 7.471 seconds, which is an equivalent 7.471 mm average pull-out. Unlike Tests v0.05 and v0.1, for which the number of increments was on the order of 100 to 200 over approximately 70 to 150 seconds, Models 1-4 of Test v1 were conducted in less than 50 increments on average. Models 5 and 6, which are defined with 10x and 100x stiffer water elements under the caisson lid, the average time and displacement reached by simulation termination was 40% less than Models 1-4, but required 2000% (20x) more increments.
Table 4.3: A list of all model versions for Test v1.

<table>
<thead>
<tr>
<th>Model</th>
<th>Density Ratio</th>
<th>Installation depth</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ratio (unitless)</td>
<td>= 90%</td>
</tr>
<tr>
<td>1</td>
<td>0.549</td>
<td>0.0001</td>
</tr>
<tr>
<td>2</td>
<td>0.600</td>
<td>0.0001</td>
</tr>
<tr>
<td>3</td>
<td>0.704</td>
<td>0.0001</td>
</tr>
<tr>
<td>4</td>
<td>0.858</td>
<td>0.0001</td>
</tr>
<tr>
<td>5</td>
<td>0.600</td>
<td>0.0100</td>
</tr>
<tr>
<td>6</td>
<td>0.600</td>
<td>0.0010</td>
</tr>
</tbody>
</table>

For Models 1-4 of Test v1, which were defined with the same lowest stiffness of 0.0001 MPa, the amount of soil uplift in relation to the caisson was inversely correlated to void ratio (Table 4.3). In other words, the percentage of soil uplift relative to the total caisson extraction decreased when the void ratio was increased. For example, Figure 4.6a illustrates how the soil plug within the caisson was lifted 0.569 mm, while the caisson was pulled 7.548 mm from the soil. The water elements between the soil plug and the caisson lid expanded to accommodate the uplift of the caisson lid, increasing the void ratio of those elements from an initial 0.600 to 11.76. In comparison against Test v0.05 and v0.1, the soil plug was lifted on average 2% higher during all versions of Test v1.
Figure 4.6: (a) Screenshot of the output of Test v1: Model 2 (VR = 0.600, Ew = 0.0001) highlighting the maximum soil uplift achieved and (b) Total Load vs Displacement results for caisson undergoing 1 mm/s pull-out for all six models with initial conditions listed in Table 4.3.

Over the course of the simulation, the load applied to maintain the constant pull-out velocity increased sharply at first and leveled off until simulation termination, as shown in Figure 4.6b. A strong trend was observed between the initial void ratio and the peak tensile load such that: if soil elements were initialized with a smaller void ratio, the load required to pull the caisson at 1 mm/s was greater, as shown on Figure 4.6b. Similar to Tests v0.05 and v0.1, increasing the stiffness of the water elements (i.e., Model 5 and 6) strongly influenced the load bearing behavior of the simulation. Instead of the water elements expanding, the soil elements directly beneath the water
elements and near the tips of the caisson expanded, which required a higher load, as shown in Figure 4.6b.

In direct comparison against the load-displacement curve observed by Vaitkunaite et al. (2016), none of the simulated model responses reproduced the post-peak softening behavior. In other words, the load-displacement curves obtained from simulation are “too stiff”, and over-predict the tensile resistance to extraction. However, the simulated load-displacement curves show strong agreement with the initial portion of the load-displacement curve observed by Vaitkunaite et al. (2016).

The discrepancies between the load vs displacement curves for Models 1-4 were due to differences in pore pressure mobilization. Previously, Test v0.05 and v0.1 produced counter-intuitive pore pressure behavior with some of the model versions predicting that pull-out would not generate suction at all. However, for all versions of Test v1, negative pore pressure and therefore suction was successfully generated. Figure 4.7a shows a contour plot of the pore pressure distribution in the soil after the suction caisson had been extracted at 1 mm/s for 7.548 seconds. The original pore pressure distribution, which had parallel contour lines (Figure 4.1b), was distorted by the concentration of low pore pressure at the lid of the caisson. Since seepage into the caisson occurs perpendicular to the pore pressure contour lines, near the caisson skirt tip, the water was drawn in uniformly from the surrounding soil, while the interior of the caisson directed seepage vertically into the expanding gap (Helwany, 2007). The pore pressure distribution for Models 1-4 of Test v1 was identical in appearance to Figure 4.7a, but differed in magnitude, as shown in Figure 4.7b. Models 1-4, which examined the effect of varying the void ratio whilst keeping the stiffness of the water elements low, displayed the same relationship found in Model 1 and 2 of Test v0.1:
the lower the void ratio, the lower the drop in pore pressure, which generated more suction force to resist pull-out.

Unlike varying the void ratio, increasing the Young’s modulus of the water elements altered the time-development in addition to the magnitude of pore pressure changes. Models 5 and 6, which had the same initial void ratio as Model 2 but were defined with water elements that are 10x and 100x stiffer, respectively, displayed a less severe pore pressure decrease than observed in Model 2. In addition, the negative pore pressure was not maintained and reached a peak before dissipating to a lower, steady value that is 78% lower than the negative pore pressure maintained by Model 2.

Figure 4.7: (a) Screenshot of the output of Test v1: Model 2 (VR = 0.600, Ew = 0.0001) highlighting the distribution of pore pressure at the end of the simulated pull step and (b) Lid Pore Pressure vs Displacement results for caisson undergoing 1 mm/s pull-out with initial conditions listed in Table 4.3.

Test v1 was one of the experiments for which Vaitkunaite et al. (2016) provided a full load vs displacement curve (Figure 4.7b) in addition to an estimation
for the suction force generated, calculated using pore pressure observations. As shown in Figure 4.7b, the scale model experiment achieved pullouts in excess of 120 mm, which far exceeds the maximum pull-out that any version of the numerical formulation could sustain. Figure 4.6b shows there was an overlap in the first 0.1 mm of displacement on the load vs. displacement curve that suggests the behavior at the very start of pull-out was well predicted by any of the six numerical model versions. However, for the remainder of the pull-out test, the model failed to produce the softening behavior observed: instead of reaching a peak tensile load after 3.6 mm of pull-out, the numerical prediction of tensile load increases until simulation termination.

The simulated pore pressure mobilization also differed from that observed by Vaitkunaite et al. in both magnitude and duration. Vaitkunaite et al. calculated that 57%, or 4,600 Newtons, of the total maximum tensile load observed is due to suction generated by negative pore pressure. As Figure 4.8 below shows, the peak back-calculated suction pressure fell between the maximum suction forces in Model 3 and Model 4. For these two models the suction pressure made up 37.6% and 28.2% of the total maximum tensile load, which falls short of the suction contribution observed by Vaitkunaite et al. For Models 5 and 6, which had order of magnitude stiffer water elements beneath the lid of the caisson, the shape of the curve due to dissipation of suction pressure vaguely resembled that of the experiment for the simple fact that there was peak behavior. However, since the total load for Model 5 and 6 were the largest predicted, the peak suction force makes up just 11.9% and 21.8% of the total tensile load. Likewise, the peak suction for Models 5 and 6 was mobilized after just 1
millimeter of displacement, whereas Vaitkunaite et al observed peak suction after 3.6 millimeters of uplift.

Figure 4.8: Suction force vs. displacement response for Test v1, Models 1-6 compared against the back-calculated suction observed by Vaitkunaite et al. (2016).

4.2.3 Test v10

At such a high rate of pull-out, ABAQUS/Standard solver required an average of 304 increments to simulate pull-out over 0.956 seconds, which is equivalent to 9.56 millimeters of extraction. Models 1 - 4 of Test v10 required fewer increments than Models 5 and 6 to simulate the same amount of deformation.
Table 4.4: A list of all model versions for Test v10.

<table>
<thead>
<tr>
<th>Model</th>
<th>Initial Void Ratio (unitless)</th>
<th>Young’s modulus, Es (Mpa)</th>
<th>Pull-out step time (s)</th>
<th>Number of increments</th>
<th>Pull-out achieved (mm)</th>
<th>Soil Uplift (mm)</th>
<th>Soil uplift (% of caisson uplift)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.549</td>
<td>0.0001</td>
<td>0.967</td>
<td>250</td>
<td>9.669</td>
<td>7.738</td>
<td>80.0%</td>
</tr>
<tr>
<td>2</td>
<td>0.600</td>
<td>0.0001</td>
<td>0.963</td>
<td>296</td>
<td>9.625</td>
<td>7.254</td>
<td>75.4%</td>
</tr>
<tr>
<td>3</td>
<td>0.704</td>
<td>0.0001</td>
<td>0.954</td>
<td>284</td>
<td>9.543</td>
<td>6.039</td>
<td>63.3%</td>
</tr>
<tr>
<td>4</td>
<td>0.858</td>
<td>0.0001</td>
<td>0.951</td>
<td>246</td>
<td>9.513</td>
<td>4.726</td>
<td>49.7%</td>
</tr>
<tr>
<td>5</td>
<td>0.600</td>
<td>0.0100</td>
<td>0.950</td>
<td>428</td>
<td>9.501</td>
<td>9.425</td>
<td>99.2%</td>
</tr>
<tr>
<td>6</td>
<td>0.600</td>
<td>0.0010</td>
<td>0.951</td>
<td>321</td>
<td>9.514</td>
<td>8.685</td>
<td>91.3%</td>
</tr>
</tbody>
</table>

In previous, slower simulations of Test v1, v0.1 and v0.05, the pull-out of the caisson was accompanied by relatively small vertical soil uplift of less than 10% of the total caisson extraction (e.g., in Test v1, the Model 2 caisson was extracted a distance 7.548 mm, but the soil was only lifted 0.569 mm at the highest point). During simulation of Test v10, however, the soil within the caisson was uplifted 75.8% of the height achieved by the caisson lid, as shown in Figure 4.9a below. In other words, 9.625 millimeters of caisson uplift also lifted the soil as high as 7.3 millimeters, which was significant in comparison to the relative displacements achieved by slower simulations. Since the soil was lifted a higher percentage of the total pull-out amount, the gap formed between the caisson lid and the soil was smaller than for the slower simulations. The results of Model 2 of the slower Test v1, for example, showed that the void ratio of the expanding water elements increases from 0.600 to 11.76, which was a 1960% increase in volume. In comparison, Figure 4.9c shows that the water elements under the lid during Test v10, Model 2 experienced an 730% increase in volume, which was less than half of that observed for Test v1. Smaller gap formation and the subsequent lifting of the soil within the caisson was accompanied by as much as 2.192 millimeters of lateral displacement of the soil directly beneath the skirt tips, pulling surrounding soil into the caisson (Figure 4.9b). In addition, since expansion of
the gap beneath the caisson lid was limited, additional volume expansion was observed in the elements near the tip of the caisson, shown in Figure 4.9c. It was observed that the magnitude of soil displacement in both horizontal and vertical directions and volume expansion was the same for Models 1-4, which all had the same Young’s modulus of 0.0001 MPa.

![Images of results](image)

Figure 4.9: Screenshots of results of Test v10, Model 2 (VR=0.600, Ew=0.0001) displaying (a) vertical displacement in mm, (b) horizontal displacement in millimeters, where negative indicates motion in the direction of the axis of symmetry, and (c) final void ratio of the two regions undergoing expansion.

Results obtained from Models 5 and 6 of Test v10, which were defined with high stiffness water elements, displayed different uplift behavior than Models 1-4: the gap between soil and caisson lid only expanded 16% by volume and extension occurred primarily in the soil elements near the caisson tip. The result of minimal gap
formation was that the soil within the caisson was lifted 90 - 99% of the total uplift, which is 10 - 40% more than the soil uplift observed in Models 1-4. However, since most of the deformation was sustained by the soil elements near the tip, Model 5 and 6 required 20 - 60% more increments to simulate the same pull-out distance as Models 1-4.

The load vs. displacement curve of Models 5 and 6 of Test v10 were remarkably similar to Models 1-4. In previous, slower simulations, the load vs. displacement curves for Models 5 and 6 were nearly double (or more) the magnitude of the other model versions (see Figure 4.4a, Figure 4.4b, and Figure 4.6b). Instead, as Figure 4.10 shows, load vs. displacement curves for Models 5 and 6 of Test v10 nearly overlapped with Model 2, which was initialized with the same void ratio of 0.600.

In comparison against the load-displacement curve observed by Vaitkunaite et al. (2016), none of the model variations adequately simulate the post-peak softening behavior. In other words, the load-displacement curves obtained from simulation are “too stiff”, and over-predict the tensile resistance to extraction. Unlike the findings for Test v1, the initial slope of the load-displacement curves are also too stiff in comparison to experimental observations.
Similar to the results for Tests v1, v0.1, and v0.05, there was an inverse relationship between maximum vertical load and initial void ratio such that, the lower the initial void ratio, the higher the maximum resistance to pull-out. The Young’s modulus of the water elements did not affect the load vs. displacement curve as drastically as in previous tests and instead of increasing the pull-out resistance, the load required to remove the caisson by the end of the simulation actually decreased for Models 5 and 6 of Test v10 (Figure 4.11b). This decrease in pull-out resistance (in
comparison to Model 2) can be explained by examining the development of pore pressure within the caisson during pull-out.

Figure 4.11: (a) Screenshot of the output of Test v10: Model 2 (VR = 0.600, Ew = 0.0001) highlighting the distribution of pore pressure at the end of the simulated pull step and (b) Lid Pore Pressure vs Displacement results for caisson undergoing 10 mm/s pull-out with initial conditions listed in Table 4.4.

Figure 4.11a is a contour plot of the pore pressure distribution at the very end of the simulated pull-out of Model 2. The area of lowest pore pressure for all model versions was closest to the lid. Figure 4.11b shows the pore pressure vs. displacement curves for the water elements closest to the caisson lid for all model versions of Test v10. All model versions exhibited a steep, rapid decline in pore pressure at the start of simulation before reaching a relative minimum. The magnitude of the initial pressure
drop was greatest for Model 1, which had the lowest initial void ratio and therefore the lowest hydraulic conductivity. After reaching a minimum pore pressure, which corresponds to suction acting against pull-out, all model versions exhibited dissipation of said pore pressure for the remainder of the simulation. For Models 1-4, the dissipation of pore pressure occurs gradually, such that a strong suction force was maintained. However, for Models 5 and 6, with high stiffness water elements, the negative pore pressure was dissipated more rapidly. In the case of Model 5, the dissipation of pore pressure is most rapid and reaches a steady, lower pore pressure value after just 3.934 millimeters of pull-out (Figure 4.11b).

The observed decrease in pore pressure caused by pull-out resulted in a suction load that acted opposite to the direction of pull-out. Figure 4.12 displays the back-calculated suction force generated during the simulated pull-out for all model versions, including the results obtained by Vaitkunaite et al. (2016). As Figure 4.12 shows, the lower the initial void ratio, the larger the suction force that was generated. The effect of increasing the Young’s modulus of the water elements is observable by comparing Models 2, 5 and 6 on Figure 4.12. As mentioned previously, the higher stiffness elements of Models 5 and 6 not only lowered the peak suction force, but also caused suction to decrease rapidly over the course of the pull-out.
Comparison against the findings of Vaitkunaite et al. (2016) was limited for Test v10, for none of the model versions were able to achieve the same amount of deformation as Vaitkunaite et al. (2016). The highest uplift achieved using this numerical formulation was just 9.669 millimeters, whereas the experiment carried out by Vaitkunaite et al. (2016) uplifted the caisson 150 millimeters and reported a peak tensile load of 30.79 kN after 16 millimeters of uplift (Figure 4.10). Back-calculation of the suction load by Vaitkunaite et al. (2016) determined that 80% (or 24.64 kN) of the reported maximum tensile load was generated by suction.

In comparison against these findings, which are shown as the dark blue line on Figures 4.10 and 4.12, all numerical versions failed to reproduce the delayed development of peak tensile load and peak suction observed by Vaitkunaite et al. (2016). In other words, instead of reaching a peak tensile load or suction after 16
millimeters of uplift, the peak tensile load for Models 1-6 was achieved within the first 5 millimeters of pull-out. Furthermore, Figure 4.10 shows that the shape of the load vs. displacement curve observed by Vaitkunaite et al. (2016) also had a second, smaller maximum tensile load that occurred after 75 millimeters of pull-out.

While the time-development of both tensile load and suction load could not be reproduced by any version of the numerical model, the magnitude of the peak tensile and suction loads was reasonably predicted by Model 4. The soil of Model 4 was initialized with the highest void ratio of 0.858 and predicted a maximum tensile load of 30.343 kN, which was just 1.6% smaller in magnitude than the maximum tensile load observed by Vaitkunaite et al. (2016). In addition, back-calculation of the suction load using pore pressure results from Model 4 predicted a peak suction load of 26.964 kN, which was 9.4% smaller than the maximum suction load calculated by Vaitkunaite et al. (2016).

4.2.4 Test v22 and Test v27

The findings of Test v22 and v27 are presented simultaneously as the relatively small difference in pull-out velocity between the two resulted in identical load-bearing behavior that differed only in magnitude. Table 4.5 and 4.6 present all model versions for both Test v22 and Test v27. It should be noted that an extra model version (listed as Model 3 below) was conducted for both Test v22 and Test v27 to investigate the behavior of the model when defined with an intermediate void ratio of 0.650. On average, the model versions of Test v27 achieved 6% higher amounts of pull-out than those of Test v27, using 5% fewer increments.
The results of Test v22 and v27 agree with the trend observed previously: as the pull-out rate increased, the gap formation under the lid of the caisson was reduced as a result of higher soil uplift. Figure 4.13a shows the displacement of the soil within the caisson for Model 2 of Test v22. The results of the slower simulation of Test v10 discussed in the previous section showed that the soil was uplifted 75.8% of the total distance that the caisson was pulled out. At nearly double the rate, Test v22 exhibited a soil uplift of 8.5 mm, which is 84.8% of the total uplift achieved. The gap between the soil and lid of the caisson expanded to 480% of its original volume, which was significantly less than the 730% gap expansion observed during Test v10. Figure 4.13c
and 4.13d show how the smaller gap formed under the lid of the caisson was compensated for by expansion of soil elements adjacent to the caisson tip interior. The void ratio of the expanding elements increased from an initial value of 0.600 to a maximum of 2.122.

The results for Test v27, which simulated pull-out at 5 mm/s (or 22%) faster than Test v22, also displayed reduced gap formation and relative soil uplift as great as 91.3% of the total pull-out distance. Horizontal displacement for Test v27 increased 20.7% from Test v22, reaching a maximum of 2.757 millimeters and displacing a larger volume of soil beneath the caisson skirt tip. The water-filled gap between the lid of the caisson and the soil increased in void ratio from an initial value of 0.600 to 2.415, which is a 402.5% increase and 80% less than the gap expansion noted in Test v22. Since the gap between the caisson lid and soil expanded a smaller amount, the soil elements near the caisson tip increased in volume by a higher percentage to accommodate pull-out deformation. For example, during the slightly slower Test v22, the soil near the skirt tip expanded 354%. During Test v27 the skirt tip soil expanded as much as 384%.

It was observed that when the Young’s modulus of the water elements is increased to 0.001 and 0.01, as is the case for Models 6 and 7, the gap underneath the caisson lid expanded less than 3% of the amount observed for Models 1-5, which corresponds to the soil within the caisson being lifted as much as 99.4% of the total pull-out.
Figure 4.13: Screenshots of results of Test v22, Model 2 (VR=0.600, Ew=0.0001) displaying (a) vertical displacement in mm, (b) horizontal displacement in millimeters, where negative indicates motion in the direction of the axis of symmetry, and (c) final void ratio of the lid elements and (d) caisson tip region undergoing expansion.

Figure 4.14a and Figure 4.15a show an example of the pore pressure distributions observed at the end of the pull-step for Test v22 and v27. It should be noted that the area of lowest pore pressure was located immediately beneath the caisson lid and that lower pressure was generated during the faster of the two pull-out velocities. The magnitude of the pore pressure drop is greatly influenced by the value of initial void ratio. As Figure 4.14b and Figure 4.15b show, the magnitude of the initial void ratio determined the minimum pore pressure such that, simulations with the lowest void ratio generated the lowest pore pressures within the caisson. The stiffness, or Young’s modulus of the water elements beneath the caisson, determined
the rate of pore pressure dissipation. It should be noted that the pore pressure of Models 6 and 7 of both Test v22 and Test v27 failed to reach the same minimum pressure as Model 2 despite having the same void ratio. In addition, Model 6 and 7 showed an increase in pore pressure with increasing pull-out distance that indicates dissipation of suction. Models 1-5 of Test v22 and Test v27, which had the lowest Young’s modulus, were not only able to reach lower pore pressures, but they also maintained said minimal pressure for a longer portion of the total pull-out step.

Figure 4.14: (a) Screenshot of the output of Test v22: Model 2 (VR = 0.600, Ew = 0.0001) highlighting the distribution of pore pressure at the end of the simulated pull step and (b) Lid Pore Pressure vs Displacement results for caisson undergoing 22 mm/s pull-out with initial conditions listed in Table 4.4.
Figure 4.15: (a) Screenshot of the output of Test v27: Model 2 (VR = 0.600, Ew = 0.0001) highlighting the distribution of pore pressure at the end of the simulated pull step and (b) Lid Pore Pressure vs Displacement results for caisson undergoing 27 mm/s pull-out with initial conditions listed in Table 4.4.

For direct comparison against the findings of Vaitkunaite et al. (2016), the total load applied to the caisson during pull-out was plotted against the displacement and the suction force was back-calculated using pore pressure results as shown in Figure 4.16. Vaitkunaite et al. (2016) experimental results for Test v22 were presented as single data points: the maximum load applied to the caisson to maintain a constant pull-out of 22 mm/s was 44,100 Newtons after 14.7 mm of extraction, which corresponds to a back-calculated maximum suction load of 35,370 Newtons. Plotted as a horizontal line on Figures 4.16(a) and 4.16(b), the maximum loads observed by Vaitkunaite et al. (2016) occurred after displacements of 14.7 mm, which exceeded the amount of uplift successfully simulated by the FE model by 3 - 4 millimeters. In contrast to the findings of Vaitkunaite et al. (2016), the FE models of Test v22 reached
maximum load after 6 to 10 millimeters of pull-out. While none of the numerical models shown here adequately resembled the delayed peak load observed during experiment, Model 3 and 4 yielded reasonable predictions for the peak total tensile load and suction load, respectively. Model 3, with a void ratio of 0.650, predicted a peak tensile load of 43,805 Newtons after 7.56 millimeters of uplift, which was 1.8% lower than the experimental value observed by Vaitkunaite et al. (2016). Likewise, Model 4, which was defined with an initial void ratio of 0.7035, predicted peak suction of 37,591 Newtons after 3.59 millimeters of extraction, which was 5.9% higher than the back-calculated findings of Vaitkunaite et al. (2016).

Experimental results for Test v27 were not limited to single data points, as shown in Figure 4.16(c) and 4.16(d), which plots the tensile load and back-calculated suction force up to 25 millimeters of pull-out (the experimental data continues until 75 millimeters but is not shown). Both experimental datasets provided by Vaitkunaite et al. (2016) exhibited a peak at 14.3 mm of uplift that corresponds to a maximum tensile load of 48,800 Newtons and a peak suction force of 37,710 Newtons. Model 3 of Test v22, which was initialized with a void ratio of 0.650, most accurately predicted the magnitude of the maximum tensile load with a maximum value of 47,946 Newtons (1.7% lower than the target maximum tensile load). However, the maximum load achieved by Vaitkunaite et al. (2016) occurred after 14.3 millimeters, whereas the maximum load achieved by Model 3 occurred after just 10 mm of extraction. In terms of predicted suction, Model 6 of Test v27, which is the model that is defined with the highest water element stiffness of 0.01 MPa, appears to mimic the post-peak shape of the suction curve shown in Figure 4.16(d). Model 6 of Test v27 predicts a peak suction of 37,354 Newtons, which is 0.9% lower than the reported value, after 1.68
millimeters of extraction. In addition, Model 6 also reasonably predicts the post-peak suction force, which Vaitkunaite et al. (2016) observed as reaching a relatively steady 39,950 Newtons after 23 millimeters of uplift.

Figure 4.16: (a) Total Load vs Displacement and (b) Suction force vs displacement results for caisson undergoing 22 mm/s compared against the load-displacement results and back-calculated suction observed by Vaitkunaite et al. (2016). (c) Total Load vs Displacement and (d) Suction force vs displacement results for caisson undergoing 22 mm/s compared against the load-displacement results and back-calculated suction observed by Vaitkunaite et al. (2016).
4.2.5 Test v47

Six simulations of pull-out at a rate of 47 mm/s were carried out for Test v47, of which Model 1 achieved an unprecedented pull-out of over 130.8 millimeters. Requiring 1,292 increments over a simulated 2.783 seconds, Model 1 of Test v47 was the largest output data base of the modeling campaign in both size of the data file and number of incrementations. It should be noted that such a load, which extracts the caisson a significant distance at a constant rate over several seconds, is not realistic nor frequently encountered in real world scenarios. An extreme load scenario such as this represents a massive, worst-case scenario ship or storm wave impact. Even though this particular situation is unrealistic at full size, the mechanics of extracting at such a velocity are relevant to understanding the overall behavior of suction caissons, even in worse case scenarios, and useful when comparing against numerical simulations such as this one (Achmus & Thieken, 2014; Vaitkunaite et al., 2016). Previously, the highest displacements achieved was 11.918 millimeters of extraction by Model 1 of Test v27. It was observed that Models 2, 3 and 4 of Test v47 were unable to simulate the same amount of deformation as Model 1 due to numerical instability.

Table 4.7: A list of all model versions for Test v47.

<table>
<thead>
<tr>
<th>Model</th>
<th>Initial Void Ratio (unitless)</th>
<th>Young’s modulus, E (Mpa)</th>
<th>Pull-out step time (s)</th>
<th>Number of increments</th>
<th>Pull-out achieved (mm)</th>
<th>Soil Uplift (mm)</th>
<th>Soil uplift (% of caisson uplift)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.549</td>
<td>0.0001</td>
<td>2.783</td>
<td>1292</td>
<td>130.801</td>
<td>126.434</td>
<td>96.7%</td>
</tr>
<tr>
<td>2</td>
<td>0.600</td>
<td>0.0001</td>
<td>1.526</td>
<td>660</td>
<td>71.745</td>
<td>68.025</td>
<td>94.8%</td>
</tr>
<tr>
<td>3</td>
<td>0.704</td>
<td>0.0001</td>
<td>0.249</td>
<td>197</td>
<td>11.722</td>
<td>10.730</td>
<td>91.5%</td>
</tr>
<tr>
<td>4</td>
<td>0.058</td>
<td>0.0000</td>
<td>0.214</td>
<td>160</td>
<td>10.039</td>
<td>8.865</td>
<td>88.3%</td>
</tr>
<tr>
<td>5</td>
<td>0.600</td>
<td>0.0100</td>
<td>0.307</td>
<td>1036</td>
<td>14.413</td>
<td>14.306</td>
<td>99.3%</td>
</tr>
<tr>
<td>6</td>
<td>0.600</td>
<td>0.0010</td>
<td>1.455</td>
<td>728</td>
<td>68.387</td>
<td>67.530</td>
<td>98.7%</td>
</tr>
</tbody>
</table>
Suction caisson theory suggests that the higher the pull-out rate, the smaller the gap that is allowed to form between the caisson lid and the soil. The results of previous, slower simulations showed that smaller gap formation leads to higher uplift of the soil within the caisson and increased deformation of the soil near the caisson tip. Models 1-4 of Test v47 agreed within this trend. Figure 4.17(a) shows that during the simulation of Model 2, the caisson itself was lifted a full 71.745 millimeters from its original position, while the soil was lifted 68.025 millimeters, which is a relative uplift of 94.8%. In comparison, the previous pull-out speed, which was Test v27, predicted a relative soil uplift that was 88.7% of the total extraction.

To accommodate the higher soil uplift, greater lateral soil displacements were also observed for Test v47, as shown in Figure 4.17(b). By the end of simulation, Model 2 of Test v47 exhibited significant lateral displacements towards the centerline of the model as high as 22.5 millimeters to compensate for the displacement of the soil immediately beneath the caisson. It was observed that the horizontal displacement of the soil was 19.6% greater for Test v47 than for Test v27 with the same amount of caisson extraction. Furthermore, since the gap between the caisson lid and soil did not expand a significant percentage of the total extraction, higher deformations and volume expansion were observed within the soil body itself, specifically, near the tips of the caisson. As Figure 4.17(d) shows, the soil elements were distorted and displaced from their original positions and, as indicated by the void ratio, expanded as much as 862% after 71.7 millimeters of extraction. The high distortion of the soil continuum generated excessive distortion warnings during analysis, but only in the dozen local elements at the tip of the caisson. These warnings could be mitigated or removed
entirely with adaptive remeshing techniques, but this was not investigated in this study.

The negative pore pressure mobilized by pull-out had a similar distribution as previous, slower simulations: the area of lowest pore pressure is at the caisson lid (Figure 4.17c). The shape of the pore pressure contour lines indicated seepage along the gradient and perpendicular to the contour lines such that, water was drawn from the surrounding soil, including along the outer edge of the caisson skirt, into the interior of the caisson and the gap formed at the lid.

Figure 4.17: Screenshots of results of Test v47, Model 2 (VR=0.600, Ew=0.0001) displaying (a) vertical displacement in millimeters, where the black areas indicate downward displacements, (b) horizontal displacement in millimeters, where negative indicates motion in the direction of the axis of symmetry, (c) distribution of pore pressure at the end of the simulated pull step, and (d) final void ratio of the caisson tip region undergoing expansion.
The influence of increasing the Young’s modulus of the water elements was observed by comparing the pore pressure changes of Model 2 with Models 5 and 6, which all have the same initial void ratio of 0.600. Similar to the results for Test v22 and Test v27, increasing the Young’s modulus of the water elements prevented Models 5 and 6 of Test v47 from reaching the same minimum pore pressure as Model 2 (Figure 4.18).

Initializing the model with a lower initial void ratio resulted in lower minimum pore pressures, which was also noted in Test v1 through Test v27. However, Test v47 was the first simulation for which some of the model versions predicted a pore pressure drop below the level at which the pore water would theoretically cavitate, which is around -0.1 MPa. Reaching the point of cavitation most frequently results in a sudden loss of suction and, in real world experiments, acts as a lower boundary for possible pore pressures (Senders, 2008; Houlsby et al. 2005b; Iskander et al. 2002). The numerical formulation presented here did not account for cavitation in any aspect of the model and as a result, the pore pressure drops predicted by Models 1 and 2 of Test v47 exceeded this theoretical limit, shown on Figure 4.18.
Figure 4.18: Lid Pore Pressure vs Displacement results for caisson undergoing 47 mm/s pull-out with initial conditions listed in Table 4.7.

Figure 4.19(a) and 4.19(b) below show the load-displacement and suction force-displacement results for all model versions of Test v47 compared against the findings of the scale model experiment conducted by Vaitkunaite et al. (2016). For unknown reasons, the reference data provided by Vaitkunaite et al. (2016) for Test v47 only described the load-displacement behavior of the caisson for 7.12 mm of pull-out. The maximum load achieved by Vaitkunaite et al. (2016) at a rate of 47 mm/s pull-out was reported as 65,360 Newtons after 46.8 millimeters of extraction (Figure 4.19a). The back-calculated suction force results for the FE model and Vaitkunaite et al.’s (2016) findings were compared across the full range of extraction (Figure 4.19b). Peak suction of 48,870 Newtons after 46.8 millimeters was noted, which corresponds to 74.6% of the total tensile load.
Figure 4.19: (a) Total Load vs. Displacement and (b) Suction force vs. displacement results for caisson undergoing 47 mm/s pull-out compared against the load-displacement results and back-calculated suction observed by Vaitkunaite et al. (2016).

Based on the plots above, all model versions of Test v47, regardless of variation of void ratio and Young’s modulus, did not accurately predict the delayed peak in tensile load nor suction force observed by Vaitkunaite et al. (2016). In other words, all model versions of Test v47 over-predicted the slope at the very beginning of the pull-out such that, the peak tensile load and peak suction force were achieved with less extraction than observed experimentally. While none of the numerical models shown here adequately reproduced the delayed peak load observed during experiment, Model 5 of Test v47 yielded the most accurate predictions for the magnitude of the peak total tensile load and peak suction load. Model 5 predicted a peak tensile load that was 0.4% greater than the observed max load and peak suction that was 3.5% less than the maximum reported by Vaitkunaite et al. (2016). Overall, Model 5 predicted
that the suction force made up 71.9% of the total pull-out load. Other model versions, such as Model 2, provided a reasonably accurate prediction for the total tensile load (within 4.5% of target peak load), but massively over-predicted the peak suction load by 36.2%. The opposite was true for Model 4, which under-predicted the tensile load by 27.4%, but was within 7.6% of target suction force.

4.2.6 Test v98 and Test v152

The results of Test v98 and Test v152 are presented simultaneously as their behavior under very high pull-out rates was similar. Table 4.8 and 4.9 list the six model versions executed at each of the pull-out rates of 98 and 152 mm/s. Models 1-6 of Test v98 achieved the highest amount of extraction on average for the entire modeling campaign, but also required the highest number of increments on average. Model 4 for both Test v98 and Test v152, which was defined with the highest initial void ratio of 0.858, was unable to carry out simulation of pull-out greater than 14.7 mm and 17.7 mm of extraction, respectively, indicating numerical instability at higher void ratios, which have a greater chance of encountering rapid pore pressure changes.

Table 4.8: A list of all model versions for Test v98.

<table>
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<th>Test v98</th>
<th>Density Ratio = 82%</th>
<th>Installation depth = 239 mm</th>
</tr>
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<td>Young’s modulus, $E_w$ (Mpa)</td>
</tr>
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<td>0.0001</td>
</tr>
<tr>
<td>2</td>
<td>0.600</td>
<td>0.0001</td>
</tr>
<tr>
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<td>0.0001</td>
</tr>
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<td>0.0001</td>
</tr>
<tr>
<td>5</td>
<td>0.600</td>
<td>0.0100</td>
</tr>
<tr>
<td>6</td>
<td>0.600</td>
<td>0.0010</td>
</tr>
</tbody>
</table>
Table 4.9: A list of all model versions for Test v152.

<table>
<thead>
<tr>
<th>Model</th>
<th>Initial Void Ratio (unitless)</th>
<th>Young’s modulus, E0 (Mpa)</th>
<th>Pull-out step time (s)</th>
<th>Number of increments</th>
<th>Pull-out achieved (mm)</th>
<th>Soil Uplift (mm)</th>
<th>Soil uplift (% of caisson uplift)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.549</td>
<td>0.0001</td>
<td>0.458</td>
<td>230</td>
<td>69.662</td>
<td>70.045</td>
<td>100.6%</td>
</tr>
<tr>
<td>2</td>
<td>0.600</td>
<td>0.0001</td>
<td>0.512</td>
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</tr>
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<td>0.600</td>
<td>0.0100</td>
<td>0.514</td>
<td>260</td>
<td>78.186</td>
<td>78.069</td>
<td>99.9%</td>
</tr>
<tr>
<td>6</td>
<td>0.600</td>
<td>0.0010</td>
<td>0.524</td>
<td>251</td>
<td>79.584</td>
<td>79.649</td>
<td>100.1%</td>
</tr>
</tbody>
</table>

Unlike slower pull-out simulations, such as Test v0.05, Test v0.1 or Test v1, where soil deformation was limited as a result of rapid expansion of the lid gap between the caisson and soil, rapid extraction of the suction caisson during Test v98 and v152 caused significant soil uplift and negligible lid gap formation. Figure 4.20 below illustrates the high deformation simulated by Model 3 of Test v98, which achieved 138.309 millimeters of caisson extraction and lifted the soil 99.1% of that distance. Inexplicably, in Model 2 of Test v152, which simulated 54 mm/s faster extraction at a slightly lower void ratio, the soil was lifted 100.1% of the extraction distance, which means that the gap under the lid of the caisson actually shrank over the course of pull-out. Increasing the void ratio caused the soil to be uplifted a smaller fraction of the total extraction, corresponding to greater lid gap expansion (Table 4.8 and 4.9).

It was observed that during simulation of Test v98 and Test v152, that a gap was formed between the soil and the caisson skirt. The gap propagated from the skirt tip, shown in Figure 4.20(c), as a result of rapid removal of the caisson. The gap between the caisson wall and soil was widest near the tip of the caisson at 3.2 millimeters in width. Along the interior wall of the caisson, the gap maintained a
constant width of 0.316 mm and extended to the base of the water elements, shown in Figure 4.20(b). During model versions of Test v152, the gap along the interior of the caisson skirt was three times as large at the narrowest point and instead of remaining the same width along the caisson wall, the gap expanded to 14.7 millimeters wide at the caisson tip. Figure 4.20(c) shows hourglassing of elements along the interior caisson-soil boundary, which was unlikely to have affected results due to being limited to a small region of the overall model (Hughes, 1987).

Figure 4.20: Screenshots of results of Test v98, Model 3 (VR=0.704, Ew=0.0001) displaying (a) vertical displacement in millimeters, where the black areas indicate downward displacements, (b) void ratio of water elements directly under caisson lid (c) final void ratio of the caisson tip region undergoing expansion, and (d) horizontal displacement in millimeters, where negative indicates motion in the direction of the axis of symmetry.
Models 1-4 of both Test v98 and Test v152 exhibited a similar trend in pore-pressure mobilization as previous tests: low initial void ratio caused a greater drop in pore pressure. However, the influence of increasing the Young’s modulus of the water elements resulted in slightly different pore pressure changes than observed previously. The effect of altering the stiffness of the water elements was observed by comparing the pore pressure changes of Model 2 with Models 5 and 6, which all had the same initial void ratio of 0.600. Model 5 and 6, with higher Young’s modulus, did not reach the same minimum pore pressure as Model 2 (Figure 4.21a). This trend was also observable in all previous, slower simulations. However, as Figure 4.21(b) shows, increasing the Young’s modulus of the water elements for Test v152 resulted in the opposite trend than observed previously: the higher the stiffness, the greater the total drop in pore pressure.

![Figure 4.21: Lid Pore Pressure vs Displacement results for caisson undergoing (a) 98 mm/s pull-out and (b) 152 mm/s pull-out with initial conditions listed in Table 4.8.](image)
All model versions of Test v98 and v152 experienced the same error noted in the previous section for Test v47: the predicted drop in pore pressure exceeded the theoretical cavitation pressure. In scale model experiments, reaching cavitation pressure reduces suction pressure rapidly and acts as a lower boundary for possible pore pressures (Senders, 2008; Houlsby et al. 2005b; Iskander et al. 2002). The FE model presented here did not account for cavitation in any aspect of the model and, as a result, the higher pull-out rates of Test v98 and Test v152 caused all model versions to exceed this physical limit, as shown in Figure 4.22. The effect of exceeding cavitation pressure on the results for total load and suction force was evident: for both pull-out rates, all model versions of Test v98 and Test v152 significantly over-predicted the observed suction force acting on the caisson lid (Figure 4.22b and 4.22d).

For Test v98, specifically, the peak tensile load observed by Vaitkunaite et al. (2016) was 71,650 Newtons after 60.5 millimeters of extraction. Of this total tensile load, Vaitkunaite et al. (2016) reported that 74.7%, or 53,450 Newtons was due to suction. While Model 4 of Test v98 reasonably predicted a peak total load of 66,794 Newtons, which was 6.8% lower than the target value, the contribution to this total tensile load due to suction was an unjustifiable 97.0%. The over-prediction of suction was clearly a result of exceeding cavitation pressure, which Model 4 of Test v98 did, as shown previously in Figure 4.21(a). In comparison against the observations of Vaitkunaite et al. (2016), the results of Test v152 were similarly inaccurate: over-prediction of suction due to surpassing cavitation resulted in significantly larger total tensile resistance.
Figure 4.22: (a) Total Load vs. Displacement and (b) Suction force vs. displacement results for caisson undergoing 98 mm/s pull-out. (b) Total Load vs. Displacement and (c) Suction force vs. displacement results for caisson undergoing 152 mm/s pull-out compared against the load-displacement results and back-calculated suction observed by Vaitkunaite et al. (2016).
4.3 Summary of Results

The load-bearing response of a miniature suction caisson in dense sand under nine different extraction rates is simulated using a finite element model constructed in ABAQUS/CAE 6.14. The FE model constructed utilizes a pore pressure and stress coupled numerical formulation to apply isotropic permeability in tandem with a Mohr-Coulomb constitutive model for the continuum elements representing dense sand. Specialized water elements were included to allow for the transfer of suction pressure to the lid of the caisson in response to mobilized negative pore pressure. The goal of this thesis was to reproduce the measured resistance to extraction, pore pressure mobilization, and suction load observed by Vaitkunaite et al. (2016) during scale model experiment. The influence of model two parameters, the initial void ratio and the Young’s modulus of the water elements, was investigated to establish an upper and lower boundary on the solution for total tensile resistance and suction load.

4.3.1 Simulated Extraction Distance

In terms of model performance, the ABAQUS/Standard solver invoked by this model performed better than expected. Pore pressure coupled analyses such as a caisson undergoing rapid extraction are notoriously difficult to model using the ABAQUS/Standard solver due to high deformations (Ahmed, 2015; Sorensen, 2016). Despite this, as Table 4.10 below shows, moderate amounts of extraction were successfully simulated. It was observed that the slowest pull-out simulations, which required the longest time steps, achieved the least extraction before analysis abort while the faster simulations achieved more significant extraction.
Table 4.10: Average model performance of all model variations for each pull-out test simulated. The target amount of pull-out is 100 millimeters, which is the distance that Vaitkunaite et al. (2016) extracted each caisson.

<table>
<thead>
<tr>
<th>Pull-out velocity (mm/s)</th>
<th>FE Model Avg. Pull-out achieved (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>6.972</td>
</tr>
<tr>
<td>0.1</td>
<td>6.988</td>
</tr>
<tr>
<td>1</td>
<td>7.471</td>
</tr>
<tr>
<td>10</td>
<td>9.588</td>
</tr>
<tr>
<td>22</td>
<td>9.829</td>
</tr>
<tr>
<td>27</td>
<td>10.261</td>
</tr>
<tr>
<td>47</td>
<td>56.077</td>
</tr>
<tr>
<td>98</td>
<td>96.902</td>
</tr>
<tr>
<td>152</td>
<td>69.116</td>
</tr>
</tbody>
</table>

4.3.2 Prediction of Drained, Partially Drained, and Undrained Behavior

Even though the FE model rarely simulated the same amount of total extraction observed by Vaitkunaite et al. (2016), with the exception of the two fastest pull-out rates, the model results still provided accurate insights into the soil-structure interaction during extraction. For example, during the slowest pull-out tests of v0.05 and v0.1, the soil plug within the caisson was lifted the least amount in relation to the total uplift of the caisson due to gap formation under the lid of the caisson. This behavior corresponds with approximately drained conditions and agrees well with observations of Vaitkunaite et al. (2016), who observed minimal soil plug displacement and little to no suction mobilization during slow speed pull-out tests. On the other hand, simulations of the fastest extractions at 98 and 152 mm/s pull-out rates exhibited roughly undrained behavior: high amounts of soil uplift and significant suction pressures mobilized within the caisson. The intermediate tests of v1, v10, v22, v27, and v47 displayed soil uplift and suction mobilization that was in between the fully drained and undrained responses. In this respect, the FE model was very
accurate, as Vaitkunaite et al. (2016) observed an identical transition between fully drained, partially drained, and fully undrained conditions as a function of pull-out velocity.

### 4.3.3 Prediction of Peak Suction Pressure

One of the primary goals of this FE model construction was to reproduce the magnitude of the mobilized suction during pull-out at various rates. A parametric study was implemented in order to observe the influence of varying initial void ratio between reported minimum and maximum values of 0.549 and 0.858. Results of this parametric study showed that model versions initialized with 0.549 and 0.858 were the upper and lower boundary solutions, respectively, for peak mobilized suction. For most of the pull-out tests, the observed peak tensile load and peak suction fell between the upper and lower boundary solutions of the FE model, as shown in Tables 4.11 below. Test velocities of 0.05 and 0.1 mm/s were not included for the suction load was determined to be negligible during tests exhibiting fully drained conditions (Vaitkunaite et al. 2016).

The results presented in Table 4.11 below highlight the shortcomings of the FE model when predicting the response of the miniature suction caisson under certain pull-out rates. At the slowest extraction velocities of 0.05 and 0.1 mm/s, unexpected pore pressure behavior was observed. Instead of negative pore pressure mobilization creating suction and seepage into the caisson, excess pore pressure was mobilized such that pore fluid flowed out of the caisson, mobilizing a negative suction pressure that assisted in pull-out, shown in Table 4.11. This counterintuitive pore fluid behavior is not readily explicable in the context of this model, but it is assumed to be a result of
the slow extraction rate for, the faster pull-out test velocities exhibited typical suction mobilization with the exception of Test v98 and v152.

Table 4.11 also shows that the suction load predicted by the upper and lower boundary solutions for Test v98 and v152 severely exceeded the peak suction load observed by Vaitkunaite et al. (2016). The overprediction of suction was the result of failure to account for the effects of liquefaction and cavitation. Liquefaction and total loss of effective stress is supposed to occur in dense sands if a critical hydraulic seepage gradient is exceeded, as is the case during rapid extraction of a suction caisson (Houlsby et al. 2005b; Verruijt, 2006; Senders, 2008). In the simulations presented in this thesis, no boundary was applied to the gradient of the seepage directed upwards into the suction caisson and thus the differential suction mobilized far exceeded experimental observations. Likewise, cavitation, which occurs when pore pressure drops beneath a critical limit, was unaccounted for in any model variation. Examination of the pore pressure changes simulated during the two fastest pull-out tests (shown in Figure 4.20), revealed that the negative pore pressure drops far below the theoretical cavitation pressure of water at this depth. Vaitkunaite et al. (2016) reported that cavitation did indeed occur during Test v98 and v152 based on pore pressure transducer data and reasoned that the similarity between the observed peak suction load of these two tests was due to cavitation acting as a limit on peak suction. Based on these results, failure to implement a limit on possible pore pressures, critical seepage gradient, or a material property that approximates cavitating or liquefying soil mechanics results in unrealistic prediction for suction at pull-out rates of 98 and 152 mm/s.
Table 4.11: Comparison of observed peak mobilized suction load and peak total tensile resistance against the upper and lower boundary solutions of the FE model. Upper boundary solution has an initial void ratio of 0.549 and an initial permeability of 0.0617 mm/s. Lower boundary solution has an initial void ratio of 0.858 and an initial permeability of 0.210 mm/s.

<table>
<thead>
<tr>
<th>Pull-out velocity (mm/s)</th>
<th>Vaitkunaite et al. (2016) Peak suction load (N)</th>
<th>FE Model Peak suction load (N)</th>
<th>Vaitkunaite et al. (2016) Peak Total Tensile resistance (N)</th>
<th>FE Model Peak Total Tensile resistance (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>-</td>
<td>-550.0</td>
<td>-276.0</td>
<td>2700</td>
</tr>
<tr>
<td>0.1</td>
<td>-</td>
<td>-373.7</td>
<td>451.7</td>
<td>4100</td>
</tr>
<tr>
<td>1</td>
<td>3980</td>
<td>3309.7</td>
<td>10248.7</td>
<td>8000</td>
</tr>
<tr>
<td>10</td>
<td>24640</td>
<td>25266.1</td>
<td>34086.6</td>
<td>30800</td>
</tr>
<tr>
<td>22</td>
<td>35770</td>
<td>31236.1</td>
<td>47637.4</td>
<td>44070</td>
</tr>
<tr>
<td>27</td>
<td>37710</td>
<td>35534.7</td>
<td>53387.7</td>
<td>48840</td>
</tr>
<tr>
<td>47</td>
<td>48870</td>
<td>42297.2</td>
<td>69776.4</td>
<td>65360</td>
</tr>
<tr>
<td>98</td>
<td>53450</td>
<td>63732.7</td>
<td>103132.5</td>
<td>71650</td>
</tr>
<tr>
<td>152</td>
<td>56170</td>
<td>74648.8</td>
<td>136233.2</td>
<td>75170</td>
</tr>
</tbody>
</table>

4.3.4 Prediction of Peak Total Tensile Resistance

Another goal of this thesis was to reproduce the total tensile resistance to pull-out as a function of extraction velocity observed by Vaitkunaite et al. (2016). Similar to the prediction of suction load, the accuracy of the lower and upper bound solution for total tensile resistance varied based on the load velocity, as shown in Table 4.11 above. The peak tensile resistance observed by Vaitkunaite et al. (2016) across the range of pull-out velocities of 10 mm/s to 98 mm/s fits between the lower and upper boundary solutions of the FE model. For the three slowest pull-out simulations of 0.05, 0.1, and 1 mm/s, and the fastest extraction test at 152 mm/s, the FE model overpredicted the peak tensile resistance to extraction observed by Vaitkunaite et al. (2016).

The poor performance of the FE model at the lower extraction speeds was most likely due to the extension of water and soil elements tied to the underside of the caisson lid, of which the former was defined as linearly elastic and the latter were elastoplastic under tensile strain. Since tangential friction was not applied in this
model and all surfaces were allowed to slide freely along their interfaces, the tensile strain of the soil and water elements and suction generated under the lid were the only sources of resistance against extraction. While the exclusion of friction represents an inconsistency with the physical reality of the situation, as friction is known to comprise a significant portion of the total resistance to extraction especially at slower pull-out velocities (Achmus & Thieken, 2014; Vaitkunaite et al., 2016), friction had to be eliminated from this model for it adversely affected the results of the simulations and in certain scenarios, completely precluded convergence on a solution. For example, when friction was applied to each of the versions of Test v10, the total resistance to extraction was increased over 300%, with frictional resistance comprising an unrealistically high percentage (95%) of the overall resistance to extraction. For the fastest simulations, Test v47, v98 and v152, the simulation failed to converge on a solution adequately when friction was enabled.

However, as discussed in the previous section, suction was actually negative for test v0.05 and test v0.1 and low for test v1, which leaves elastic and plastic strain of water and soil elements as the majority of the total resistance to extraction. Furthermore, the Young’s modulus of the water elements was observed to have the greatest effect on peak resistance for these slow speed tests, as discussed in Sections 4.2.1 - 4.2.3., which further suggests that the elastic strain of the elements beneath the caisson contributed to the unrealistic peak tensile resistance.

The peak tensile resistance reported by Vaitkunaite et al. (2016) for Test v152 did not fall between the upper and lower boundary solutions of the FE model for reasons discussed in the previous section: the FE model did not consider the cavitation limit of water. In both experiment and in simulation, a clear increasing trend was
observed between pull-out velocity and peak tensile resistance. However, the cavitation pressure of water in theory acts as an upper limit to the possible suction pressures, which is why Vaitkunaite et al. (2016) reported only a slight increase in both suction load and total tensile resistance between test v98 and v152, shown in Table 4.11 above. The peak total tensile resistance predicted by the FE model does not take the cavitation limit into consideration.

4.3.5 Comparison Against Analytical Methods of Houlsby et al. (2005b) and Iskander et al. (2002)

4.3.5.1 Peak Suction Pressure

Figure 4.23 compares the predicted of peak suction pressure using equation 8 developed by Houlsby et al. (2005b) against the prediction obtained using the FE model developed in this thesis and the peak suction measured during scale model experiments of Vaitkunaite et al. (2016). It should be noted that Eq. 8 utilized a constant value of 0.074 mm/s for hydraulic conductivity ($k_0$), whereas the FEM shown here varies initial hydraulic conductivity as a function of minimum and maximum void ratio of Aalborg University Sand No.1.
Figure 4.23: Comparison of back-calculated peak suction pressure against measured peak suction pressure. The FE model lower limit is initialized with a void ratio of 0.549 and a corresponding initial hydraulic conductivity of 0.059 mm/s. The upper limit has an initial void ratio of 0.858 and a hydraulic conductivity of 0.207 mm/s.

The analytical method proposed by Houlsby et al. (2005b) clearly overpredicts the suction pressure beneath the caisson lid, especially for higher pull-out rates. The first data point, \( v = 1 \text{ mm/s} \), yields the closest prediction of 27.3 kPa of suction, but this value is still 17% higher than the observed suction of 23.4 kPa. In comparison, the
lower and upper boundary solutions of the FE model more closely resemble the observations of Vaitkunaite et al. (2016), with the exception of the two fastest pull-out rates, where the FEM fails to account for cavitation.

### 4.3.5.2 Peak Tensile Capacity

Figure 4.24 compares the predicted of peak tensile capacity obtained using the method developed by Houlsby et al. (2005b) and Iskander et al. (2002) against the prediction obtained using the FE model developed in this thesis and the peak tensile capacity measured during scale model experiments of Vaitkunaite et al. (2016).

![Figure 4.24: A plot of tensile resistance vs. pull-out rate comparing the predictions of Houlsby et al. (2005b), Iskander et al. (2002), and the FEM developed in this thesis against the measured tensile resistance reported by Vaitkunaite et al. (2016).](image)

The analytical method of Houlsby et al. (2005b) shows strong agreement with observed peak tensile capacity, but only if the suction term of Eq. 10 was adjusted to exactly match the observations of Vaitkunaite et al. (2016). The peak tensile capacity
predicted by Iskander et al. (2002), which also required suction data from Vaitkunaite et al. (2016) to satisfy Eq. 11, under-predicts the peak tensile capacity.
Chapter 5

DISCUSSION AND CONCLUSIONS

5.1 Summary

Suction caissons are gaining popularity as more research is devoted towards their use for offshore wind turbines (Tjelta, 2014). However, under sudden pull-out loads, which are possible during storm wave impacts, suction caissons are much more vulnerable to failure than under compressive loads (Byrne, 2000; Houlsby et al. 2005). Existing research on suction caissons under extraction has identified that under partially-drained and undrained conditions, the suction mobilized within the caisson yields higher resistance to pull-out than frictional resistance alone (Byrne, 2000; Houlsby et al. 2005b; Senders, 2008). However, the mobilization of drained, undrained, or partially drained conditions are very difficult to predict in cohesionless soils which are common to the developing wind farms located on the continental shelf of the U.S. East Coast and the North Sea in Europe (Tjelta, 2014; Vaitkunaite et al. 2016). Current design methodology for suction caissons therefore must include prediction of tensile capacity in order to determine safe operational conditions (Senders, 2008).

The high costs of offshore wind turbines necessitate a thorough understanding of suction caissons under tension/pull-out, which is achieved through scale model experiments (Vaitkunaite et al. 2016). The results of scale model experiments are used to develop and validate analytical methods for predicting suction caisson performance (Serbulea, 2013). That said, the challenges associated with predicting the capacity to resist pull-out loads due to the complex, non-linear dynamics of dense, sandy soils has resulted in increased use of finite element (FE) modeling (Cao et al. 2000).
modeling allows for the simulation of complex interaction of saturated soils and structures under complex loading regimes, such as rapid pull-out (Helwany, 2007; Serbulea, 2013).

In this thesis, FE modeling was utilized to develop a model of a single suction caisson undergoing constant-velocity pull-out at nine different rates. The dimensions, soil properties, and loading conditions were selected to match a scale model pull-out testing campaign performed by Vaitkunaite et al. (2016). The goal of this thesis was to reproduce the observed mobilization of pore-pressure and suction during pull-out using a FE model developed in ABAQUS/CAE 6.14. The FE model constructed utilizes a pore pressure and stress coupled numerical formulation to apply isotropic permeability in tandem with a Mohr-Coulomb constitutive routine to the continuum elements representing dense sand. Specialized water elements were included to allow for the transfer of suction pressure to the lid of the caisson in response to mobilized negative pore pressure. The influence of two model parameters, the initial void ratio and the Young’s modulus of the water elements, was investigated to establish an upper and lower boundary on the solution for total tensile resistance and suction load. Based on the results of the numerical simulations, which are discussed in great detail in the previous chapter, the following main conclusions can be drawn:

- The higher the rate of pull-out, the greater the total resistance to pull-out. It was observed that peak tensile resistance is only reached after the caisson has been extracted some distance. The extraction distance required to reach peak resistance also increased with pull-out rate.
- Increasing the rate of pull-out also increases the drop in pore pressure within the interior of the caisson and therefore the suction force resisting extraction.
Pull-out simulations slower than 1 mm/s exhibited approximately drained conditions with negligible suction mobilization, while the fastest simulations at 98 mm/s and 152 mm/s predicted pressure drops of considerable magnitude that exceed theoretical cavitation pressure.

- The higher the rate of pull-out, the smaller the volume expansion of the gap between the caisson lid and the top of the interior soil. Smaller gap formation resulted in increased soil uplift along the interior skirt of the caisson and higher displacements of the surrounding soil.

- Decreasing the initial void ratio, which increases the initial hydraulic conductivity of the soil, increases the drop in pore pressure within the interior of the caisson and therefore the suction force resisting extraction, regardless of extraction speed.

- The value used for the Young’s modulus of the water elements beneath the caisson lid controls the dissipation of pore pressure and the magnitude of mobilized suction. With the exception of Test v152, the lower the Young’s modulus of the water elements, the greater the drop in pore pressure associated with peak suction. Using higher values for the Young’s modulus results in increasingly rapid dissipation of pore pressure and a lower peak suction pressure overall.

5.2 Limitations of the FE Model

By nature, all numerical models approximate real world physics and have inherent limitations (Helwany, 2007). The most relevant limitations of the model constructed in this thesis are as follows:
Installation effects are not considered. In doing so, the soil inside the caisson skirt is modeled as undisturbed with the caisson “wished” into place. Installation usually results in softening of the soil as a by-product of rapid seepage during suction-assisted penetration (Cotter, 2009; Chatzivasileiou, 2014).

Liquefaction is not considered in this numerical formulation. Liquefaction and total loss of effective stress is supposed to occur in dense sands if a critical hydraulic seepage gradient is exceeded, as is the case during rapid extraction of a suction caisson. In the simulations presented in this thesis, no limitation was applied to the gradient of the seepage directed upwards into the suction caisson and thus the differential suction mobilized far exceeded experimental observations. A realistic numerical model that accurately accounts for liquefaction would produce lower peak resistances to extraction as differential suction pressure difference between the interior and exterior of the caisson would be prevented from exceeding cavitation. In other words, this model over-predicts the maximum suction and therefore the maximum tensile resistance to extraction. Achmus & Thieken (2014) presented a method of considering liquefaction, but found that permitting a high hydraulic gradient more accurately reproduced experimental results as an upper bound solution. In future simulations of this problem, a more robust approach and model must be taken to account for liquefaction.

Suction pressure between the lid of the caisson and the porous soil beneath was maintained for the full duration of all pull-out tests as a result of the tie constraint. In reality, loss of suction most likely occurs as a result of failure of
the soil plug through sudden, local losses of effective stress due to liquefaction or cavitation (Mana et al., 2014; Vaitkunaite et al., 2016).

- Defining the model in axisymmetric space precluded the consideration of the effects of horizontal loads and rotations. It has been shown that additional horizontal loading alters the typical vertical load-response of suction caissons through reduction of the effective stress of the soil along the skirt and, in severe cases, formation of a gap between the outer skirt and soil (Achmus & Thieken, 2014; Ahmed, 2015).

- The contact surfaces between the suction caisson and soil were modeled as frictionless such that the effects of interface friction are not considered. Under tension, the behavior of the interface between the soil and caisson is challenging to simulate as there must be friction mobilization along the caisson skirt followed by gap opening (Cerfontaine & Collin, 2016). While the exclusion of friction represents an inconsistency with the physical reality of the situation, as friction is known to comprise a significant portion of the total resistance to extraction especially at slower pull-out velocities (Achmus & Thieken, 2014; Vaitkunaite et al., 2016), friction had to be eliminated from this model for it adversely affected the results of the simulations and in certain scenarios, completely precluded convergence on a solution. While a full explanation of why friction severely influenced this particular model formulation is not available, previous modelers have encountered similar difficulties with accounting for friction while simultaneously using a pore-pressure coupling formulation (Kaliakin, 2018). In particular, friction is notoriously difficult to enable and achieve convergence when using the
axisymmetric space in ABAQUS, which is the case for this model (Kaliakin, 2018). To circumvent this problem, researchers like Cerfontaine (2014) were forced to write custom code in the program LAGAMINE that uses interface elements to account for gap opening as well as tangential friction (Cerfontaine, 2018). Though a significant number of attempts were mounted, creating a custom numerical formulation to account for the complex interaction of friction, seepage, and gap opening within ABAQUS was unfortunately exceeded the scope and time constraints of this project. The numerical formulation shown here represents the best option available to an ABAQUS user that is able to implement built-in functions within the program.

- The soil is always assumed to be 100% saturated. In reality, some air may be present in the form of gas bubbles under the lid of the caisson, which lowers the amount of mobilized suction as a result of the compressibility of air (Achmus & Thieken, 2014).

- The most applicable idealization of this model was the Mohr-Coulomb model, but as the results showed, the simple built-in formulation provided with ABAQUS failed to properly replicate the complex softening behavior observed by Vaitkunaite et al. (2016).

5.3 Conclusions

This thesis presented estimations for the peak tensile resistance and contribution due to pore-pressure mobilized suction of a suction caisson under various extraction rates using a 2D axisymmetric model assigned isotropic permeability and a Mohr-Coulomb elasto-plastic constitutive routine. In industry practice, numerical modeling is used to predict the performance of foundations in soil conditions during
the planning stages of development for offshore wind turbines or any offshore platform (Serbulea, 2013). However, the model developed in this thesis, which was inaccurate in several key aspects, would require reconstruction and reformulation in a FE modeling program that is more adequately suited to simulate the complex soil mechanics of suction caissons under rapid extraction. The purpose of a simplified model which only considers a single caisson under relatively simple loading conditions is to establish a proof-of-concept model and preliminary numerical approximations of laboratory observations (Vaitkunaite et al. 2016, Offshore Wind Accelerator, 2019). Hypothetically, if the model presented in this thesis was able to accurately reproduce all of the observations noted by Vaitkunaite et al. (2016) while accounting for seepage, gap formation, as well as proper friction, then the results (in the form of Force-Displacement and Suction-Displacement plots) would be plugged into a non-finite element model, known as a macro element model, that simulates the entire dynamic structure (from rotating turbine blades to the dragging of the underwater structure by waves) under various loading conditions—not just one caisson alone under pure tension (Offshore Wind Accelerator, 2019). A comprehensive, macro element model would then be used to conduct the design and optimization of the structure, which requires cooperation between the geotechnical engineer in charge of the footings, the mechanical engineer responsible for the rotating turbine, and the structural engineer in charge of the substructure (Offshore Wind Accelerator, 2019). However, because the numerical formulation presented in this thesis had distinct and identifiable errors, the results of this thesis should be used as a stepping stone for a more accurate and more powerful model based on the ultimate finding of this thesis: that the built-in saturated soil modeling ability of ABAQUS 6.14
is not well-suited to simulate the very difficult physical problem of a suction caisson under tensile extraction. In conclusion, the model designed in this thesis can be improved or extended to various applications as follows:

- **Tensile response under cyclic loads** should be investigated as only constant-velocity tensile load are simulated here. Smaller magnitude, but more frequent tensile loads in the form of waves and wind are much more likely than a severe storm wave that generates rapid caisson extraction at a constant rate. Cyclic loads must be studied to obtain a clear picture of the loss of soil stiffness in response to loads that change in magnitude and direction over time.

- **The load-bearing behavior of suction caissons under lateral loads and overturning moments** should be investigated using a three-dimensional continuum model. The loads simulated in this thesis are pure vertical loads, whereas the loading conditions in real life are more likely to be a combination of vertical loads, horizontal loads, and torques generated by wind, waves, and rotor excitation.

- **The contact formulation used here** should be replaced using interface behavior, which can account for fluid flow and gap opening, simultaneously. The effect of friction mobilization and degradation due to fluid flow is not accounted for in this model, but is considered to be of critical importance for dense sands (Cerfontaine & Collin, 2016). The implementation of interface elements, however, is limited to certain analysis types in ABAQUS and requires manipulation of the input file and use of user-defined subroutines (ABAQUS, 2014).
The soil constitutive routine should be updated from the built-in Mohr-Coulomb plasticity. A modified Mohr-Coulomb material law, such as the one used by Ahmed (2015), enables the stress and strain dependent behavior of dense sands while removing the effect of constant dilation. Another option is a hypoplastic material law, such as the one proposed by Von Wolffersdorff, which accounts for barotropy, where the soil becomes more ductile under pressure, and pycnotropy, which improves the behavior of loose soils (Wolffersdorff, 1996; Achmus & Thieken, 2014). In future simulations, more robust models such as the ones described here should thus be used via the user-implemented UMAT option in ABAQUS.
REFERENCES


