NTNU – Trondheim
Norwegian University of
Science and Technology

New Method of Sinking Caisson Tunnel in Soft Soil

Abda Berisso Bame

Geotechnics and Geohazards
Submission date: June 2013
Supervisor: Arnfinn Emdal, BAT
Co-supervisor: Anders Beitnes, Statens vegvesen (SVV)

Norwegian University of Science and Technology
Department of Civil and Transport Engineering

Norwegian University of Science and Technology

Department of Civil and Transport Engineering

Geotechnical Division

New Method of Sinking Caisson Tunnel in Soft Soil

Abda Berisso Bame

June 2013

Title of the Thesis: *New Method of Sinking Caisson Tunnel in Soft Soil*	**Date:** *10 June 2013* **Number of pages (with appendices):** *70*			
	Master Thesis	x	Semester project	
Name: *Abda Berisso Bame*				
Supervisor: *Ass. Professor Arnfinn Emdal*				
External supervisor: *Anders Beitnes*				

ABSTRACT

Strict environmental regulations and the impact of tunneling on existing structures are among the major challenges in tunnel construction especially in urban areas. The common way of constructing tunnel in soft soil is cut-and-cover method. Sinking a caisson tunnel in soft soil is new idea and this new concept could be an alternative method of tunneling in soft soil. This caisson tunneling method is proposed to reduce the use of temporary works such as propping of sheet pile walls and increase the ease and speed of construction. Besides, it reduces the disturbance on the nearby structures due to vibration.

The aim of this work is to evaluate geotechnical feasibility of sinking the caisson tunnel to the desired depth at the selected soil profile along tunnel alignment. Empirical analysis was carried out to evaluate the penetration depth of the structure due to self-weight. In addition, numerical analysis was also carried out using PLAXIS and the results from different methods of analyses were compared. Some of the geotechnical challenges during sinking, the limitations and suggested future work were pointed out.

Keywords:

1. Caisson Tunnel
2. Sinking
3. Soft Soil
4. PLAXIS

(Signature)

MASTER THESIS

Spring 2013

For

Abda Berisso Bame

New Method of Sinking Caisson Tunnel in Soft Soil

Background

The demand to utilize underground spaces in urban areas is increasing due to environmental factors and traffic congestion. Most of the underground projects are related to urban development, which requires the construction of metro systems, underground parking facilities and different utility lines. Other applications of underground construction include the crossing of natural barriers such as rivers and mountains that are found across the alignment of major road, motorway or railway link projects.

Many of underground structures may be constructed in difficult ground conditions like soft clays, water bearing sands, creepy soils, weathered and swelling soil. Excavation of shallow tunnels in densely populated urban areas needs great care to reduce the risks and effects on nearby structures. Many difficulties may arise due to the occurrence of heterogeneous ground conditions which may require frequent adjustments during excavation and tunneling.

The traffic flow through county road (FV) 32 in Porsgrunn municipality in South Norway, which is flowing along Gimlevegen-Augestad road, was proposed to be re-routed. It was proposed to develop a new road to maintain a safe, eco-friendly and efficient transport system. Among various alternatives proposed to develop the road there is an option of about 350m long environmental tunnel along the east side of the railway. A new concept of caisson tunneling was proposed by Anders Beitnes and the main objective of this study is to evaluate the geotechnical feasibility of the proposed idea.

Task Description

Tunneling in soft soil has various challenges. Strict environmental regulations especially in urban areas and the potential impact of tunneling on existing structures are among challenging issues. The common way of constructing tunnel in soft soil is cut-and-cover method. Sinking a caisson tunnel in soft soil is new idea and this new concept could be an alternative method of tunneling in soft soil. The aim of this work is to investigate geotechnical challenges in lowering the caisson tunnel to the desired depth in the soil. This caisson tunneling method is proposed to reduce the use of temporary works such as propping of sheet pile walls and increase the ease and speed of construction. Besides, it reduces the disturbance on the nearby structures due to vibration.

In order to investigate the feasibility of the concept empirical and numerical analyses will be done. The numerical analysis will be carried out by Finite Element Method using PLAXIS software. The results from numerical and empirical analyses will be compared.

The following are among some of the results from this study:

- Brief literature review
- Investigation of geotechnical challenges during and after tunneling
- Empirical and numerical analyses
- Evaluation of the feasibility of the concept

Arnfinn Emdal (Ass. Professor)
NTNU

Anders Beitnes
Norwegian Public Roads Administration

PREFACE AND ACKNOWLEDGEMENTS

This master thesis was written at Geotechnical division of the Norwegian University of Science and Technology (NTNU). The aim of the study is to evaluate the geotechnical feasibility of a new method of sinking caisson tunnel in soft soil.

I would like to express my deep and sincere gratitude to my supervisor Ass. Professor Arnfinn Emdal. I would also like to thank Anders Beitnes who proposed the new method of sinking caisson tunnel in soft soil.

I am truly indebted and thankful for Norwegian State Educational Loan Fund (Lånekassen) for financing my two years study in Norway.

Finally, I am grateful to my family and parents for their love and encouragement over the years of my study.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1. Background

The demand to utilize underground spaces in urban areas is increasing due to environmental factors and traffic congestion. Most of the underground projects are related to urban development, which requires the construction of metro systems, underground water mains, gas pipes, telecommunication and electric power networks, as well as underground parking facilities. Other applications of underground construction include the crossing of natural barriers such as rivers and mountains that are found across the alignment of major road, motorway or railway link projects.

Many of underground structures have to be constructed in difficult ground conditions, including soft clays and water bearing sands, as well as soft rocks with particular behavioral features such as creep, weathering and swelling. Additional difficulties may arise because of the occurrence of a variety of heterogeneous ground conditions which may require frequent adjustments to be made during tunneling. Very strict environmental regulations especially in urban areas and the potential impact of tunneling on existing structures are among other challenging issues.

1.2. Problem Description

The traffic flow through county road (FV) 32 in Porsgrunn municipality in South Norway, which is now flowing along Gimlevegen-Augestad road, was proposed to be re-routed. It was proposed to develop a new road to maintain a safe, eco-friendly and efficient transport system. The purpose is to allocate sufficient space and to ensure the best possible solution in terms of accessibility for public transport, pedestrians and commercial transport.

Various alternatives were proposed to develop Gimlevegen-Augestad road. Among the options the construction of new route in the east side of the railway was included. This alternative has different options and a 350m environmental tunnel along the east side of the railway was part of one of the alternative.

Tunneling in soft soil has various challenges. Strict environmental regulations especially in urban areas and the potential impact of tunneling on existing structures are among challenging issues. The common way of constructing tunnel in soft soil is cut-and-cover method. Sinking a caisson tunnel in soft soil is new idea and this new concept could be an alternative method of tunneling in soft soil if it is found feasible. This caisson tunneling method is proposed to reduce the use of temporary works such as propping of sheet pile walls and increase the ease and speed of construction; to reduce the disturbance on the nearby structures due to vibration. The tunnel will shield the buildings against noise. It also provides the restoration of the area after the environmental tunnel is completed and if there is sufficient ground.

1.3. Objective of the Study

The aim of this work is to investigate geotechnical challenges in lowering the caisson tunnel to the desired depth in the soil. The main objective of this study is:

- Brief literature review
- Empirical and numerical analysis and comparison of the analyses
- Evaluation of the feasibility of the concept
- Assessment of geotechnical challenges during and after tunneling

2. LITERATURE REVIEW

2.1. Introduction

The tunneling method commonly used to construct shallow tunnels in soil is cut-and-cover method. If there is existing structure close to the alignment of the tunnel, temporary works such as driving of sheet pile wall and propping are important to undertake excavation and construction. The vibration due to temporary works could disturb the nearby structures and reduce the speed of construction. In order to minimize these problems especially in soft soils a new method of sinking caisson tunnel was proposed. As the proposed idea is new there is limitation of literature and therefore, only topics which were found relevant are presented.

2.2. Tunneling Method in Soil

A tunnel is a closed or roofed structure carrying a road through, or under an obstacle such as a mountain, a body of water, a building or a complete development. Tunnels are among most expensive parts of road infrastructure, but they are necessary to overcome circumstances where other solutions cannot be applied. Although tunnel is expensive it may still be the only option for the site for a range of reasons.

Road tunnels are feasible alternatives to cross a water body or transverse through physical barriers such as mountains, existing roadways, railroads, or facilities; or to satisfy environmental or ecological requirements (Hung and Parsons 2009).

Tunnel design is a multi-disciplinary activity requiring interaction of specialist designers from different disciplines such as structural engineering, geotechnical engineering, engineering geology, electrical and mechanical engineering, fire safety engineering and communication engineering. Decision on location and form of tunnels should only be undertaken with the advice of these specialist designers (Hung and Parsons 2009).

Tunnels are constructed in types of materials ranging from soft clay to hard rock. Tunnel construction methods depend on many factors such as ground condition, ground water condition, geometry of the tunnel, depth of the tunnel, tunnel support during excavation and risk management.

The tunnel construction method commonly used in soil at shallow depth is cut-and-cover tunneling. Cut and cover tunneling is a common technique to construct shallow tunnels where trench is excavated and roofed. This method can accommodate changes in tunnel width and non-uniform shapes and is often adopted in construction of stations. Several overlapping works are required to be carried out in using this tunneling method. Trench excavation, tunnel construction and soil covering of excavated tunnels are among major integral parts of the tunneling method.

Most of the works in cut-and-cover tunneling are similar to other road construction except that the excavation levels involved are deeper. Cut-and-cover tunnels are rectangular or horse-shoe type in shape. There are two basic forms of cut-and-cover tunneling methods:

- Bottom-up method: a trench is excavated, with ground support as necessary, and the tunnel is constructed in it. The tunnel may be of in-situ concrete, precast concrete, precast arches or corrugated steel arches. After the construction work is completed the trench is then carefully back filled and the surface is reinstated.
- Top-down method: side support walls and capping beams are constructed from ground level by such methods as slurry walling, or contiguous bored piling. Then a shallow excavation allows making the tunnel roof of precast beams or in-situ concrete. The surface is then reinstated except for access openings. This allows early reinstatement of roadways, services and other surface features. Excavation then takes place under the permanent tunnel roof, and the base slab is constructed.

There are various factors that should be considered for cut-and-cover tunnel: depth of water table, the need to dewater the excavation, soil condition, stability and earth pressure on the side walls and surcharges on the tunnel roof, temporary construction loads and uplift forces (Hung and Parsons 2009).

Groundwater control is very important in tunnel construction. The tunnel will become unstable and will not be safe to work in if there is water leakage inside. One of the most effective methods to control such problem is ground freezing. Pipes are inserted into the ground surrounding the shaft and are cooled until they freeze. This freezes the ground around each pipe until the whole shaft is surrounded frozen soil, keeping water out. The most common method is to install pipes into the ground and to simply pump the water out. This works for tunnels and shafts.

Cut-and-cover construction consists of tunnel construction by deep excavation in trench, construction of the permanent tunnel structure, and subsequent backfill and reinstatement of the ground surface. The method is economical in comparatively shallow tunnel works and is typically applied in urban highway schemes and for urban metro stations and running tunnel construction. This method is used as an alternative to bored tunnel construction for underground railway and river-crossing highway schemes.

Cut-and-cover construction works are frequently constructed in water-bearing soils and in such cases the risk of failure of structure by uplift pressures both during construction and during the design life of the structure should be considered. The total downward self-weight of the structure together with the frictional resistance of the external walls is required to exceed the upward hydrostatic force by an acceptable factor of safety at each stage (Puller 2003).

2.3. Caisson Types and Sinking Techniques

Caissons are hollow box or cylinder mostly constructed above ground level and then sunk to some desired depth by excavating or dredging soil from within the caisson. There are two principal types of caissons: open caissons and pneumatic caissons. The open caisson could be well type (open at bottom and top) or floating type (open at the top and closed at the bottom).

2.3.1. Open Caisson

Historically, well caissons have been used for bridge foundations and were sunk by hand-excavation by divers. The working depth was limited to about 6m below water level and in late nineteenth century excavation by grab and sand pump was introduced to allow well foundations to greater depth (Puller 2003).

The oldest type of caisson is the open caisson (Fig. 2-1) and it sinks due to self-weight as the soil at the bottom of the caisson is excavated. The excavation could be done by hand if the bottom is located above the water table. However, if it is below water table the soil is removed by dredging and the bottom of the caisson is sealed with underwater concrete when grade is reached (Terzaghi, Peck et al. 1996).

Open caissons are used for many geotechnical engineering applications such as for deep foundation elements bypassing weak soils to tip in firm deeper strata, in rivers and maritime construction to reduce the risk of scour, for collecting sewage water through gravity sewer pipe networks or from sewer force mains. The design and construction of open caissons require a detailed soil investigation. The engineering and construction techniques are key factors to achieve functional caissons (Abdrabbo and Gaaver 2012).

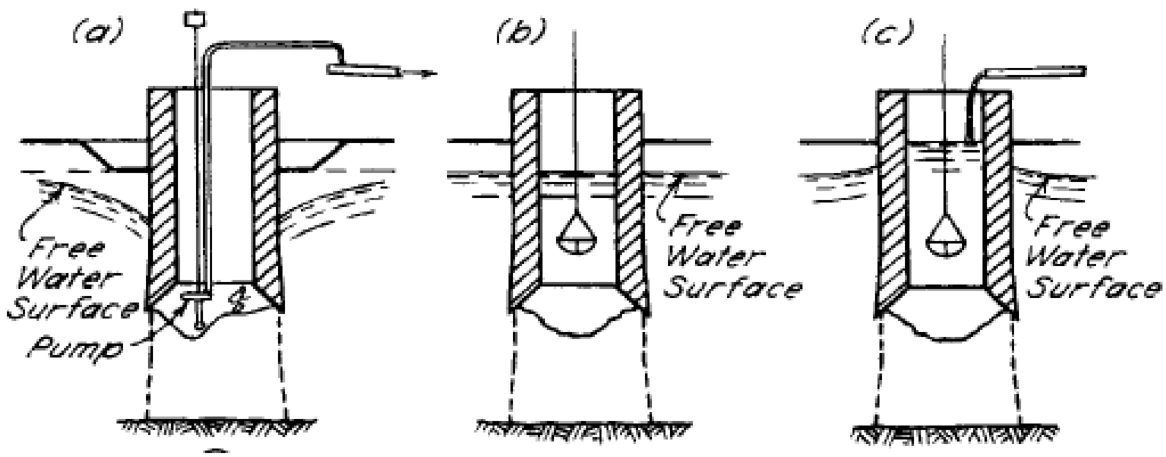


Figure 2-1: Open Caisson (Terzaghi, Peck et al. 1996)

Open caisson is used as foundation of bridges and/or other heavy structures when the bearing beds are below water or deep below surface. It is reinforced concrete structure and its dimensions correspond to the desired foundation area. It may be casted on the ground at the location of future foundation area on scaffold or floated to the site of sinking if it is to be constructed in a river or at sea. Its lower part consists of cutting edge usually strengthened with steel (Nonveiller 1987).

Sinking of open caissons in dense or very dense sands is risky. Incorrect sinking of open caissons may cause extra cost, construction delay, and harm to nearby structures. To drive an open caisson downward air or water jetting may be used near the cutting edge of an open caisson and inside open trench. Unsymmetrical work around an open caisson may lead to tilting of the caisson. If this occurs, the tilt should be immediately corrected before resuming the sinking process (Abdrabbo and Gaaver 2012).

Open caisson-sinking techniques permit a shaft structure to be progressively sunk, either due to self-weight or with the help of caisson jacks, to a predetermined depth from the surface in a controlled manner. Open caissons in tunneling and public engineering works are circular in cross-section, whereas those used in harbor works are commonly square or rectangular in plan (Solhjell, Sparrevik et al. 1998).

2.3.2. Pneumatic Caisson

The pneumatic caisson is a four-sided box in steel and concrete with air deck and the bottom of the caisson is like a diving bell. Water from the bottom of the caisson is removed by compressed air and this assists to work under air to carry out the excavation in the dry. The air pressure balances the pore-water pressure at the cutting edge. The maximum depth to which pneumatic caissons can be sunk is controlled by the maximum air pressure at which work can proceed. Working in the caisson for long hours under high pressure increased costs, and objections increased from health and safety point of view. Thus, the use of pneumatic caissons has declined now a day (Puller 2003).

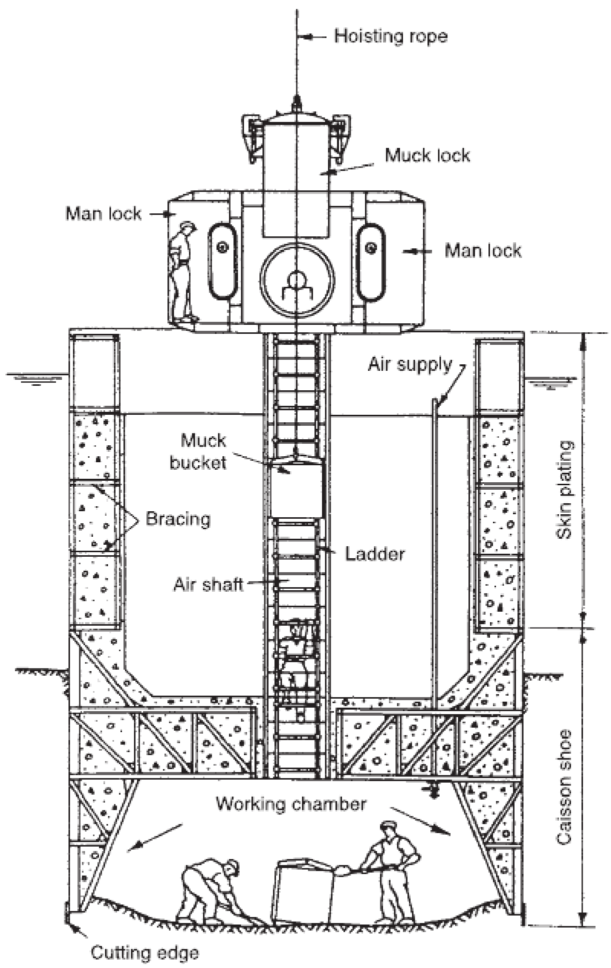


Figure 2-2: Typical arrangement of a pneumatic caisson(Puller 2003)

2.3.3. Estimation of Skin Friction during Sinking of Caisson

The downward movement of caisson is controlled by tip resistance and skin friction along the wall of the caisson. To overcome the skin friction dead loads might be loaded on light weight shafts. But heavy caissons may sink due to their own weight.

Adding weights on top of a caisson increase the cost of construction and thus concrete caissons are designed in a way that their weight exceeds the skin friction at every stage of construction. The skin friction depends on various factors: type of soil, shape of the lower part of the caisson, method of excavation, and diameter of the caisson.

The skin friction is likely to increase with time in clays. The friction between fine grained soils and the concrete caisson can be reduced by using smooth coating on the caisson during sinking. Such coating could reduce the friction between the concrete and fairly stiff clay by about 40% (Terzaghi, Peck et al. 1996).

Table 2-1: Values of Skin Friction during Sinking (Terzaghi, Peck et al. 1996)

Type of Soil	Skin Friction f_s [kPa]
Silt and soft clay	8-30
Very stiff clay	50-100
Loose sand	13-35
Dense sand	35-70
Dense gravel	50-100

Sinking of open caissons is appropriate where the prevailing soil consists of soft to medium clays, silty sands, or loose sands. These soils can be easily excavated using grab buckets within the open caisson and do not offer high skin friction along caisson-soil interface. The soil resistance along the cutting edge should be uniform to achieve controlled sinking of the caisson without excessive tilting during excavation.

The other controlling factor for smooth sinking of the open caisson is the ratio between total weight, W, and the friction on the surface of the caisson in contact with the surrounding soil. This ratio must be greater than 2 and measurement should be taken to keep in the range, otherwise smooth sinking could be difficult (Nonveiller 1987).

2.4. Suction Caisson Installation

A suction caisson is a large cylindrical structure, which is usually made of steel, open-ended at the bottom and closed at the top. The use of suction caisson in off shore industry is increasing now a day. It might be used either as shallow foundation or as suction anchor. The shallow foundation option is more common at sandy soil sites, whereas the anchor application is common in clay or layered soils (Houlsby and Byrne 2005).

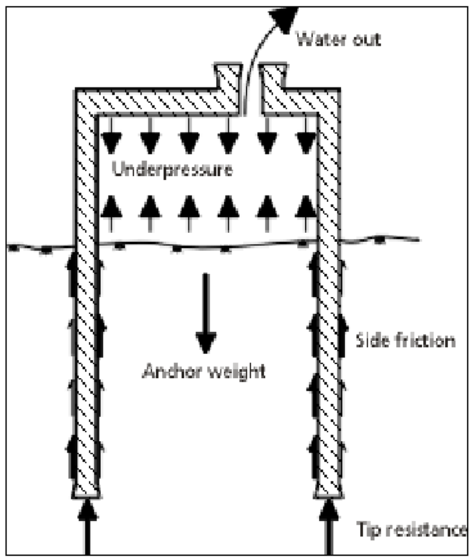


Figure 2-3: Suction caisson with forces acting during penetration (Andersen and Jostad 1999).

The suction caisson can be installed by first allowing it to penetrate under its self-weight, and then pumping out the entrapped water to cause further penetration of caisson to the desired depth. The resistance to penetration is calculated as the sum of adhesion on the outside and inside of the caisson, and the end bearing of the annular rim. The adhesion terms are calculated, following the usual practices in pile design, by applying a factor α to the value of the undrained shear strength. The end bearing is calculated following the standard bearing capacity analyses.

$$V' = h\alpha_o\, s_{u1}(\pi D_o) + \; h\alpha_o\, s_{u1}(\pi D_o) + (\,\gamma' h N_q + s_{u2} N_c)(\pi D_t) \tag{1}$$

Adhesionon the outside + Adhesion on inside + End bearing on the tip

Where D_o, D_i and D are the outside, inside and mean diameters respectively, s_{u1} is the average undrained shear strength, s_{u2} is undrained shear strength at depth h, α_o, α_i are adhesion factors on the outside and inside of the caisson (as used in undrained pile design), and N_c is bearing capacity factor for deep strip footing in clay (Houlsby and Byrne 2005).

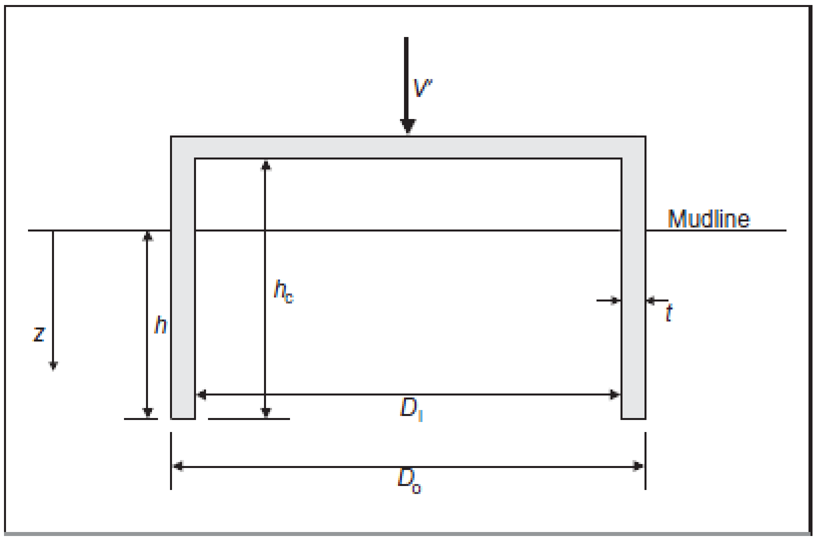


Figure 2-4: Outline of suction caisson (Houlsby and Byrne 2005)

2.5. Bearing Capacity

The bearing capacity of the soil is developed in a footing due to three properties of the soil: cohesion, friction and density of the soil. The ultimate bearing capacity (q_u) of a foundation is a load beyond which a foundation would fail and no longer be useful (Rajapakse 2011).

Failure of a foundation could occur due to inadequate bearing capacity of the soil beneath the foundation (leading to shear failure), overturning or sliding of the foundation.

The ultimate bearing capacity of saturated cohesive soils (clay and silt) with low permeability is most critical immediately after construction, before excess pore water pressure has had time to dissipate i.e. undrained conditions. As time proceeds, consolidation occurs; the soil becomes stiffer and has more strength. Therefore, design on fine grained soils should be in terms of undrained or total stress. Skempton (1955) suggested for undrained saturated clay (φ=0), the basic Terzaghi equation should be used. The general bearing capacity equation according to Prandtl (1920) and Terzaghi (1925):

$$q_u = \frac{Q_u}{A} = cN_c + pN_q + 0.5\gamma BN_\gamma \tag{2}$$

Where c is soil cohesion, γ is unit weight of the soil, p is soil surcharge, A is the cross-sectional area of the footing and N_c, N_q and N_γ are bearing capacity factors. The bearing capacity factor, N_c, can be determined from (Skempton 1951) Chart depending on the ratio of depth to width of footing and shape of footing.

The ultimate bearing capacity of strip footing on homogeneous clay in an undrained condition (s_u-analysis) can be derived from general bearing capacity equation and is reduced to:

$$q_u = s_u N_c + p \tag{3}$$

Where s_u is undrained shear strength; N_c is undrained bearing capacity factor found from chart (Skempton, 1951); p is the total overburden pressure (=γz).

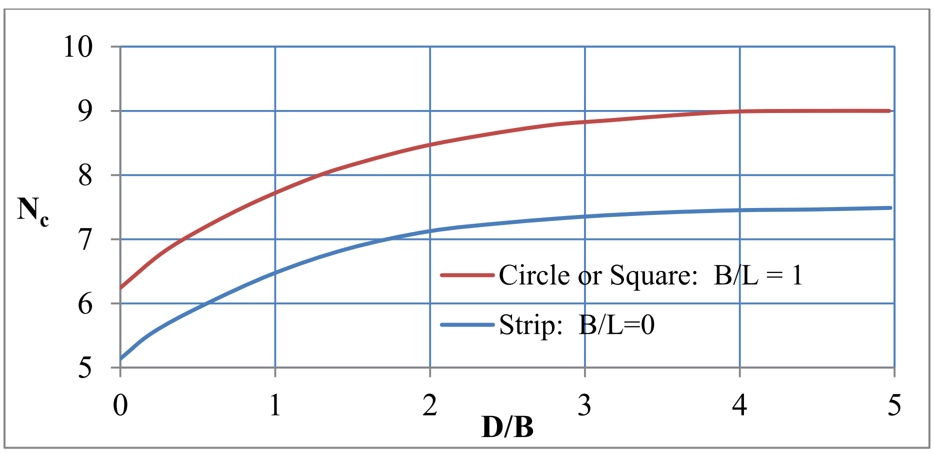


Figure 2-5: The bearing capacity factor Nc value chart (Skempton 1951)

Depending on the stiffness of soil and depth of foundation, there are three modes bearing capacity failure experienced by the foundation soil.

a) General shear failure:
b) Local shear failure:
c) Punching shear failure:

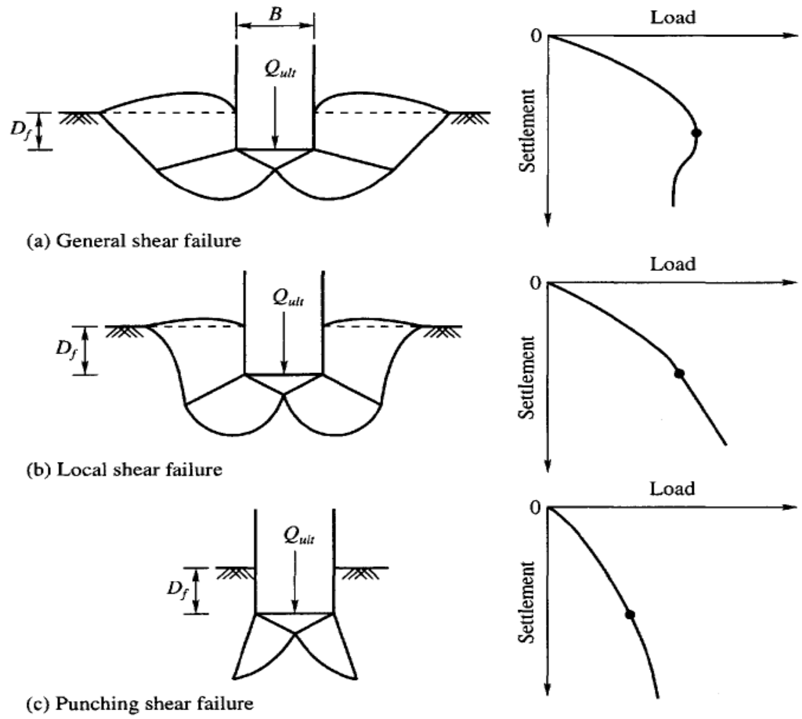


Figure 2-6: Modes of bearing capacity failure (Vesic 1963).

A footing founded on pre-compressed clays or saturated normally consolidated clays will fail in general shear if it is loaded so that no volume change can take place and fails by punching shear if the footing is founded on soft clays (Murthy 2002).

Driven Piles

When a pile is driven into granular soil the soil equal to the volume of the driven pile is displaced and the soil around the sides is compacted since the displaced soil particles enter the soil spaces of the adjacent mass which leads to densification of the mass. The compaction of the soil mass around a pile increases its bearing capacity.

If a pile is driven into saturated silt or cohesive soil, the soil around the pile cannot be densified because of its poor drainage qualities. The displaced soil particles cannot enter the void space unless the water in the pores is pushed out. The stresses developed in the soil mass adjacent to the pile due to the driving of the pile have to be borne by the pore water only. This results in the development of pore water pressure and a consequent decrease in the bearing capacity of the soil. The soil adjacent to the piles is remolded and loses to a certain extent its structural strength. The immediate effect of driving a pile in a soil with poor drainage qualities is, therefore, to decrease its bearing strength. However, with the passage of time, the remolded soil regains part of its lost strength due to the reorientation of the disturbed particles and due to consolidation of the mass (Murthy 2002).

2.6. Retaining Structures and Deep Excavation

Retaining structures provide permanent lateral support to vertical or near-vertical slopes of natural ground or fill. These structures prevent earth from moving. The moving earth is either a cut or a fill. The structural system usually includes a wall, which may be supported by other structural members such as props, floor slabs, ground anchors or reinforcing strips (Potts and Zdravković 2001).

The forces imposed on the retaining structures and other structural members, the predicted movement of the structure and the retained soil should be assessed. Besides, estimates of the magnitude and extent of ground movements in the short term and long term are very important.

In underground construction deep excavation and tunneling are commonly used. Both types of constructions affect the structures directly adjacent to them. Deep excavation in soft soils cause large displacements due to the compressibility of the material and are usually combined with high groundwater tables (Korf 2009).

For deep excavations in urban areas reliable prediction of the magnitudes of movements in the surrounding soil and estimation of the effects of these movements on adjacent structures are very important.

There are two methods for estimating wall deflections due to excavation: empirical and numerical analyses. Compared to numerical analysis, empirical methods provide less accurate but straightforward estimations. Empirical methods did not account for some important factors such as soil-structure interaction, soil properties, duration of excavation, types of struts and etc. It is difficult for empirical methods to give reliable prediction under complex construction conditions (Lu, Tan et al. 2012).

These predictions can be achieved using powerful numerical methods such as finite element analyses. Most of the case studies documented in literature are back analyses of comparisons between computed soil deformations and measured data. The widespread use of back analyses reflects the difficulties in achieving reliable analytical predictions of soil deformations (Whittle, Hashash et al. 1993).

In using numerical methods such as finite element methods to predict the magnitude of soil movements and its effects, the accuracy of soil parameters such as soil stiffness, shear strength, permeability, location of ground water table are very important.

Excavations are usually not perfectly plane strain and so three-dimensional effects can influence the amount and shape of settlement curves. Three-dimensional effects usually focus on corners in the construction which can be either outward facing or inward. The shape of excavation will affect the magnitude and distribution of ground movements around it. Corners in the excavation restrict movement if they are outside corners or increase movements if they are inside corners (Korf 2009).

2.7. Bottom Heave due to Excavations

During excavation of supported trenches and larger holes in the ground it is possible for failure to occur due to soil heaves up into the base of the excavation. The weight of the soil besides the excavation tends to push the underlying soil into the excavation. The risk of base failure during excavation applies for soft and very soft clays. It is caused by relief of load and similar to bearing capacity failure in reverse direction (Puller 2003).

The stress field and displacement field change in saturated soft soil due to unloading of soil during excavation would cause significant soil upheaval. Thus, it is very important to manage the deformation of the retaining structure and settlement of the surrounding ground.

For soils with constant undrained shear strength s_u and zero friction angle ($\emptyset = 0$) the factor of safety against base failure where stiffer clay exist at depth and sheeting terminates at base of the cut is given by (Puller 2003):

$$Factor\ of\ Safety = \frac{Bearing\ Capacity}{Stress\ causing\ failure} = \frac{s_u\, N_c}{\gamma D + q} \qquad (4)$$

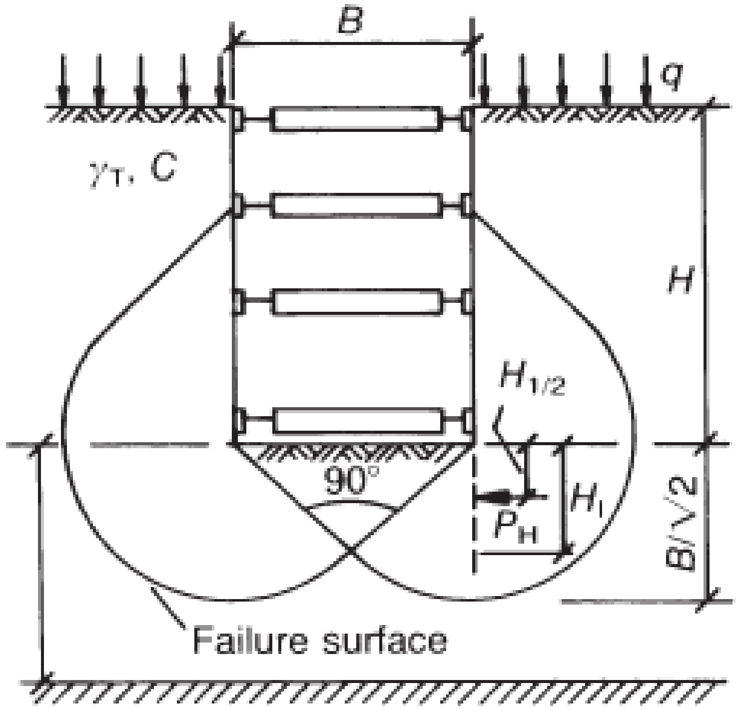


Figure 2-7: Basal heave in cuts in clay of considerable depth (Puller 2003).

The factor of safety against basal failure is required to be greater than 1.5; if it is less than 1.5 the wall should penetrate below the base of cut to insure stability. For shallow excavation the restraining effects of the soil around the excavation has the effect of slightly increasing the factor of safety (Puller 2003).

3. PROJECT DESCRIPTION

3.1. Introduction

Prefeasibility investigations were conducted to re-route the road network along county road 32 (FV-32) in Porsgrunn in Southern Norway. One of the alternative options proposed is construction of about 350m long environmental tunnel. The new idea proposed by Anders Beitnes is to build a caisson tunnel with net internal dimension of 4.80mx10.50m. The idea is to erect complete 25m long sections of concrete tunnel (with roof and walls) above ground, where the invert is left open. Beneath the invert slab level there will be transverse beams firmly bound to the base of each side wall. These ribs and the skirt represent an exposed vertical area of such size that when it is sunk in the soil and subject to activated shear failure, the friction/shear will balance the size order of the gravity of the structure.

Interface between each section must have sliding locks and provisions for watertight packers. At the front of each new section is a provisional, watertight shutter or steel wall. When each new section is sunk to its position, the invert slab is cast in place to form a monolithic structure together with the beams and the walls. In case of too easy lowering, the inside can be provided with hydraulically operated “flaps” above the rib level, which can be used to control a bearing area. On the outside of the walls, it can be installed vertical drains to avoid accidental rise of hydraulic pressure. And there can be nozzles to provide for lubrication along the outside and/or lime-cement to enhance the friction under movement and for consolidation in final position.

The caisson tunnel will be casted at the site. The structure is expected to sink to the desired depth primarily due to its self-weight and certain controlled static load. The side walls will be kept firmly stable with transverse beams and the roof slab. In the front end of each new item is a temporary shut, inside the caisson tunnel are one or two excavators and the roof has a central slot for a belt or hoists to remove the excavated soil. Hydraulic flaps could be used to assist the driving and braking control during sinking. In each joint interlocking sections mounted between the walls and the prefabricated flap around the profile is included at the end of each new section. The presence of soft and sensitive silt and clay soil with low undrained shear strength along the tunnel alignment will make the caisson construction and installation difficulty.

3.2. Location

The study area, Porsgrunn, is located in South Norway at about 150 km south of Oslo. It is part of cluster of municipalities in southern Telemark that constitute the Grenland area of Norway. The city is situated near Gunneklevfjord, and at the mouth of Porsgrunn River. Porsgrunn is a well-known industrial town.

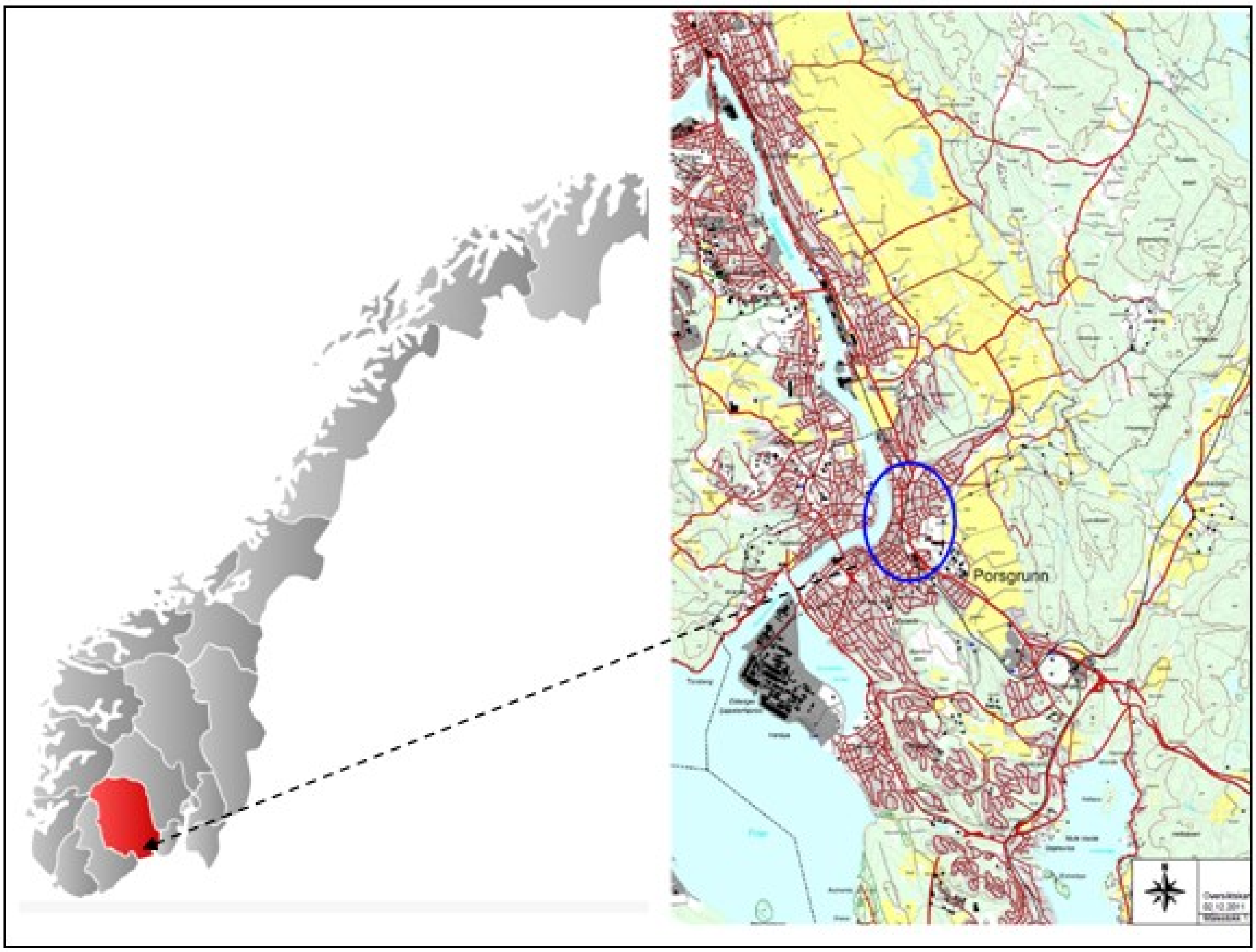


Figure 3-1: An overview of the study area

3.3. Soil Condition and Geology of the Area

The soil in the study area is predominantly marine deposit from the last ice age which has normally consolidated thick layer. Earlier studies in the area show that the whole area is dominated by soft and sensitive silt and clay soils with thin sand or gravel layer and clay pockets. The terrain in the study area is mostly flat.

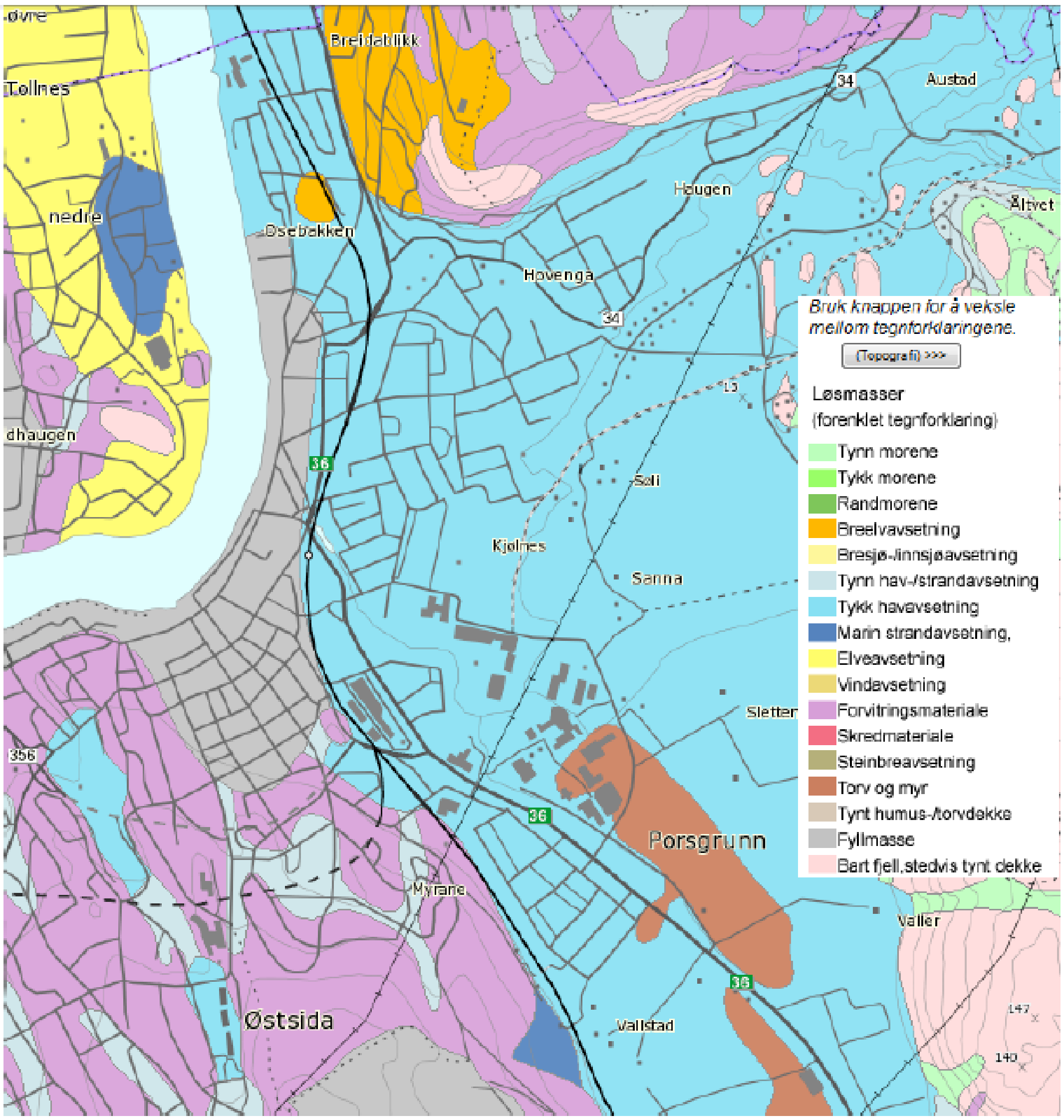


Figure 3-2: Geologic Map of the Study Area (Statens vegvesen Region sør 2011) .

Previous soil investigations were conducted under the auspices of Norwegian Public Roads Administration (NPRA) at different adjacent locations in the study area. The soil investigation data from previous investigations was considered adequate at the feasibility stage and more investigations were proposed after the appropriate alternative route is selected.

The geotechnical parameters found on the basis of NPRA (1978) along the route option 3 include rotary sounding, ground water investigations and undisturbed sampling (with NGI 54mm sampler). The parameters considered for this study are the parameters along the profile 1080 (with profile designation in NPRA, 1978 - see appendix 1) along the tunnel alignment.

One of the soil parameters used for different analyses is undrained shear strength of the soil. The undrained shear strength is the measure for the shear strength of fine grained soils. The undrained shear strength of the soil along the tunnel alignment from falling cone test and the unconfined compression test is shown on the table below.

Table 3-1: Undrained shear strength of the selected profile along tunnel alignment

Depth	**Soil Type**	**Unit Weight**	**Undrained Shear Strength, s_u [kPa]**		**Water Content**
[m]	[-]	[kN/m^3]	Falling Cone Test	Unconfined Compression Test	w [%]
0-1	Block				
1-2	Silt, sandy clay	18.7	34	26-39	35-39
2-7	Silt, sandy clay	19.1-19.7	15-32	18-31	24-33

A representative profile along the tunnel alignment is selected. This profile is selected for two reasons: it is one of the critical locations due to the presence of existing building close to the tunnel alignment in the east side of railway; and the undrained shear strength values from the two tests (Falling Cone Test and Unconfined Compression Test) are almost the same.

NPRA has performed triaxial test, odometer test and other measurements in 1994 along profile 1080 (NPRA, 1994-2). The result from odometer test is summarized in table 2. The modulus of elasticity is calculated from empirical relation in equation 1 below.

Table 3-2: Results of odometer tests conducted by NPRA (1994, 2)

Depth	**Soil type**	**Consolidation Coefficient**	**Coef. of Permeability**	**Preconsol. Stress**	**Elastic Modulus**
z, [m]	[-]	c_v, [m^2/yr]	**k**, [m/yr]	**p'**$_c$, [kPa]	**E**, [kPa]
3.7	Clayey silt	60	0.18		3333
4.3	Clayey silt		0.04		
5.3	Clayey silt	16	0.07	58	2286
6.6	Clayey silt	30	0.10		3000
7.6	Sandy silt	160	0.23	85	6957
12.3	Silty clay	22	0.03	115	7333
16.4	Clayey silt	24	0.03	166	8000
18.4	Silty clay	41	0.07	170	5857

The modulus of elasticity of the soil in the above table is derived from the following relation:

$$k = \frac{c_v \, \gamma_w}{E} \Rightarrow E = \frac{c_v \, \gamma_w}{k} \qquad (5)$$

The over consolidation ratio (OCR) is the ratio of effective preconsolidation stress (p'_c) to effective vertical stress (σ'_v). The preconsolidation at different depth in the above table shows that the soil is normally consolidated since the OCR is nearly unity.

The coefficient of permeability, k, given in the above table shows the soil in the study area has very low permeability. The soil along the selected profile has high water content. The sandy clay layer has water content of 35-39% whereas the silt clay layer has water content of 24-33%. The remolded undrained shear strength of the lower silty clay layer is between 2kPa and 6kPa.

The ground water level measured along the tunnel alignment using electric and hydraulic pore pressure measurements shows 0.55m to 1.88m below ground surface (NPRA, 1994). This measurement is almost similar to the measurement taken from August to December 1978 (NPRA, 1978) which shows 0.6m to 1.5m below ground surface. Along the selected route of the tunnel there is no bed rock found up to 28m depth (NPRA, 1978).

The friction angle for the silty clay layer was 32^0 and the attraction value was 2-4 kPa (NPRA, 1994-2).

The soil along the selected profile has three layers. The top crust and the sandy clay layers are 1m thick each. The top crust of the soil has high undrained shear strength and there is no available data for this layer. The sandy clay layer has average undrained shear strength of 34kPa. The lower layer has relatively lower undrained shear strength which increases linearly. The most probable undrained shear strength of this lower thick layer is 17+2.5z.

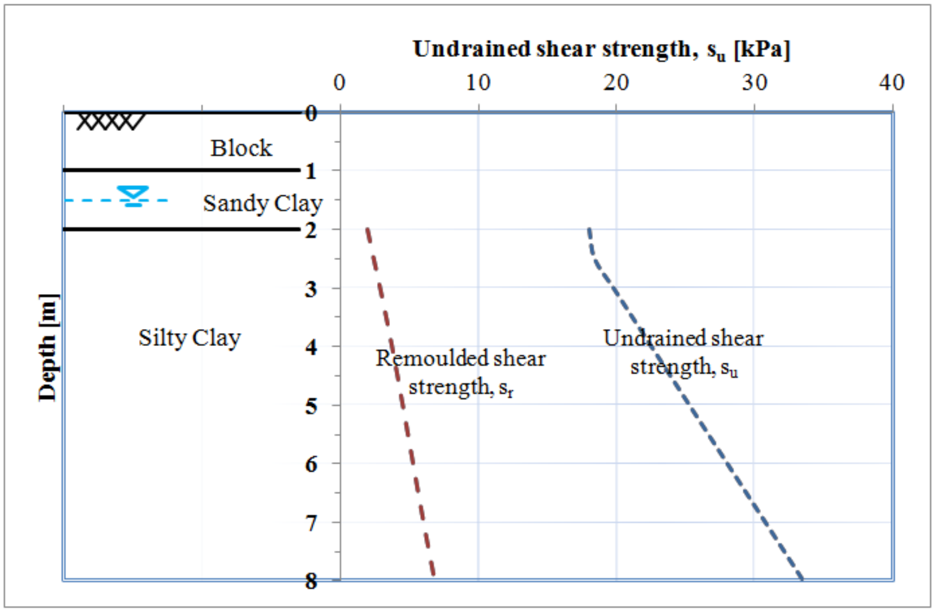


Figure 3-3: The most probable remolded and undraind shear strength of the selected profile

3.4. Alignment of the Environmental Tunnel

Various alternatives were proposed to develop the county road (FV) 32 along Gimlevegen-Augestadvegen in Porsgrunn town. This study focuses on alternative 3 (see figure below-blue line). Alternative 3 starts at the junction with Gimlevegen and runs along the River to a new intersection with Dr Munksgate. From there, the road runs along the eastern side of the railway until it crosses the bridge over Hovenggata as the railroad and connects to the roundabout with Augestadvegen.

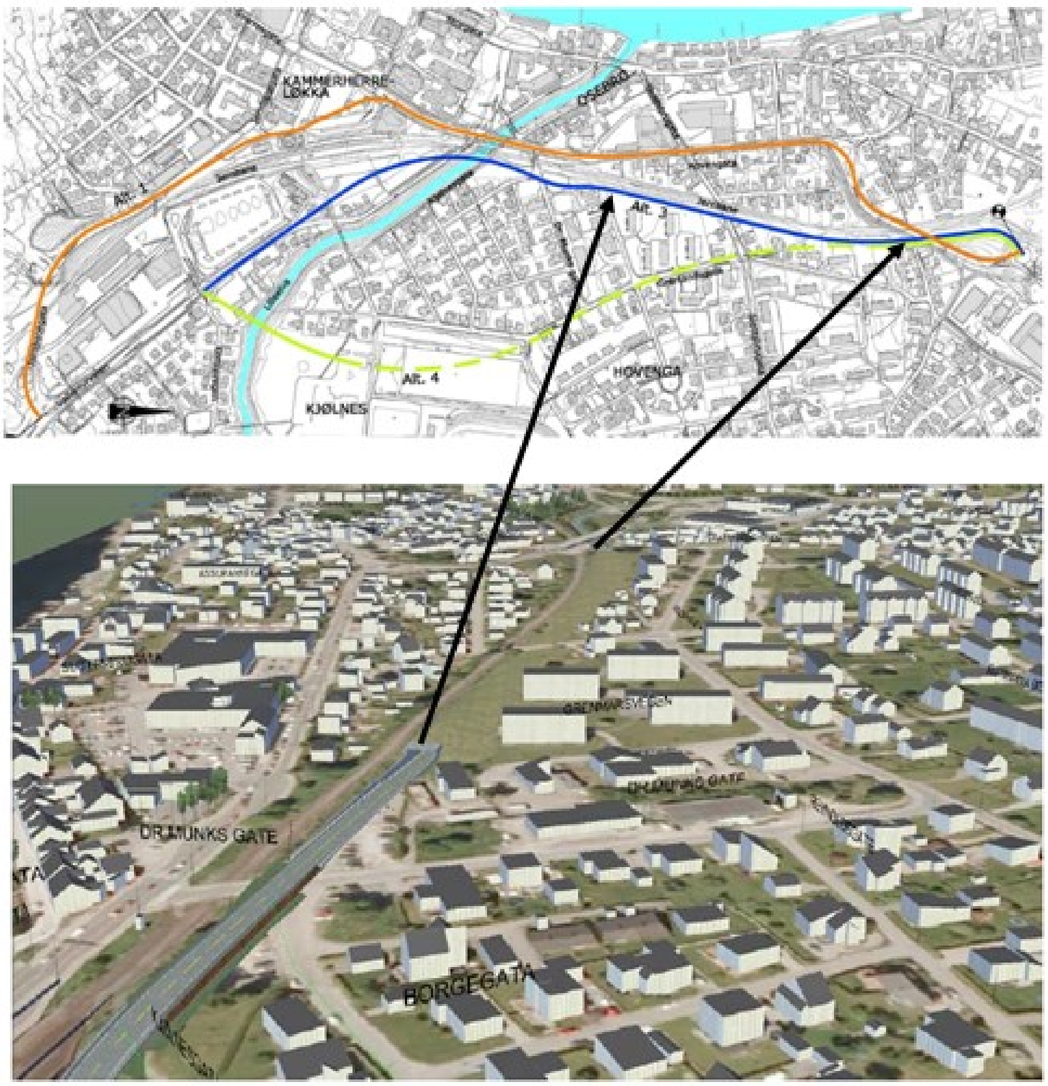


Figure 3-4: Tunnel location in the proposed alternatives (modified) (Statens vegvesen Region sør 2011).

The stretch of the road between Dr Munksgate and Hovenggata bridge is proposed to be an environmental tunnel with excess hold of terrain elevation. The total length of the road along alternative 3 is about 1300m, from which about 350m is an environmental tunnel. The tunnel will shield the buildings against noise. It also provides the restoration of the area of the environmental tunnel if there is sufficient ground.

The total length of the environmental tunnel is about 350m and from profile 760 to 930 the tunnel is below the terrain level.

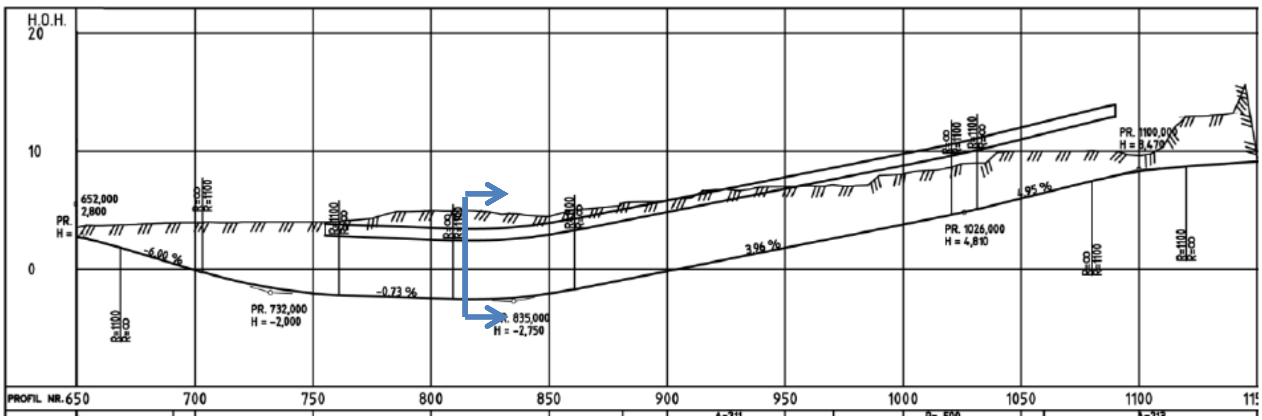


Figure 3-5: Longitudinal profile of the tunnel (Statens vegvesen Region sør 2011).

3.5. Geometry of the Caisson Tunnel

The inverted rectangular tunnel has width of 10.5m for two lane road and has clear height of 4.8m. The length of single concrete structure is 25m. The total length of the environmental tunnel is about 350m.

The walls serve to prevent the movement of the soil and to shield sound. The thickness of the tunnel wall should fulfill all the structural and geotechnical requirements and need to be carefully studied and designed to meet loading, waterproofing and other related requirements. The elongation of the walls below the beam is essential to increase the stability of the structure and to prevent the seepage into the tunnel serving as cut-off wall. It also increases the bearing capacity.

The bottom beams keep the structure stable during and after installation. During sinking of the caisson tunnel the total load of the structure rests on the wall tip and the beams. Increasing the number of beams will increase the bottom area and this increases the penetration resistance.

Therefore the number and bottom area of the beams should be kept as minimum as possible. As the beams are subjected to lateral earth pressure axial and flexural rigidity should be adequate. Struts which could be manipulated by special hydraulic jacks should be used instead of beams.

The total weight of the structure depends on the dimension of the structural elements. The self-weight of 25m long single concrete element with wall and slab 0.5m thick is about 7000kN. The total vertical stress due to the load of the structure during installation/sinking depends on the area on which the total load rests. The vertical stress due to self-weight of the caisson tunnel is between 185kN/m^2 to 280kN/m^2 depending on which structural element the load rests on: the walls, the beams or both.

The advantage of elongating the wall below the beam:

- To reduce the penetration resistance from the soil during installation/penetration
- Serves as cut-off wall and thus prevent the inflow of water to the inside
- For stability of the structure
- To reduce the basal heave

Table 3-3: Load and dimensions of single caisson tunnel

Caisson tunnel element	Quantity	Dimension [m]			Unit Weight [kN/m^3]	Weight [kN]
		Length	Width	Depth		
Beam	3	10.5	0.4	0.5	25	158
Walls	2	25	0.5	6	25	3750
Roof	1	25	10,5	0.5	25	3281
Total						7189

The opening at the middle of the top slab will be closed after the caisson reaches the desired depth. The excavated soil will be removed by a conveyor system through the opening at the middle of the top slab. Two excavators might be used inside the caisson tunnel to excavate the soil.

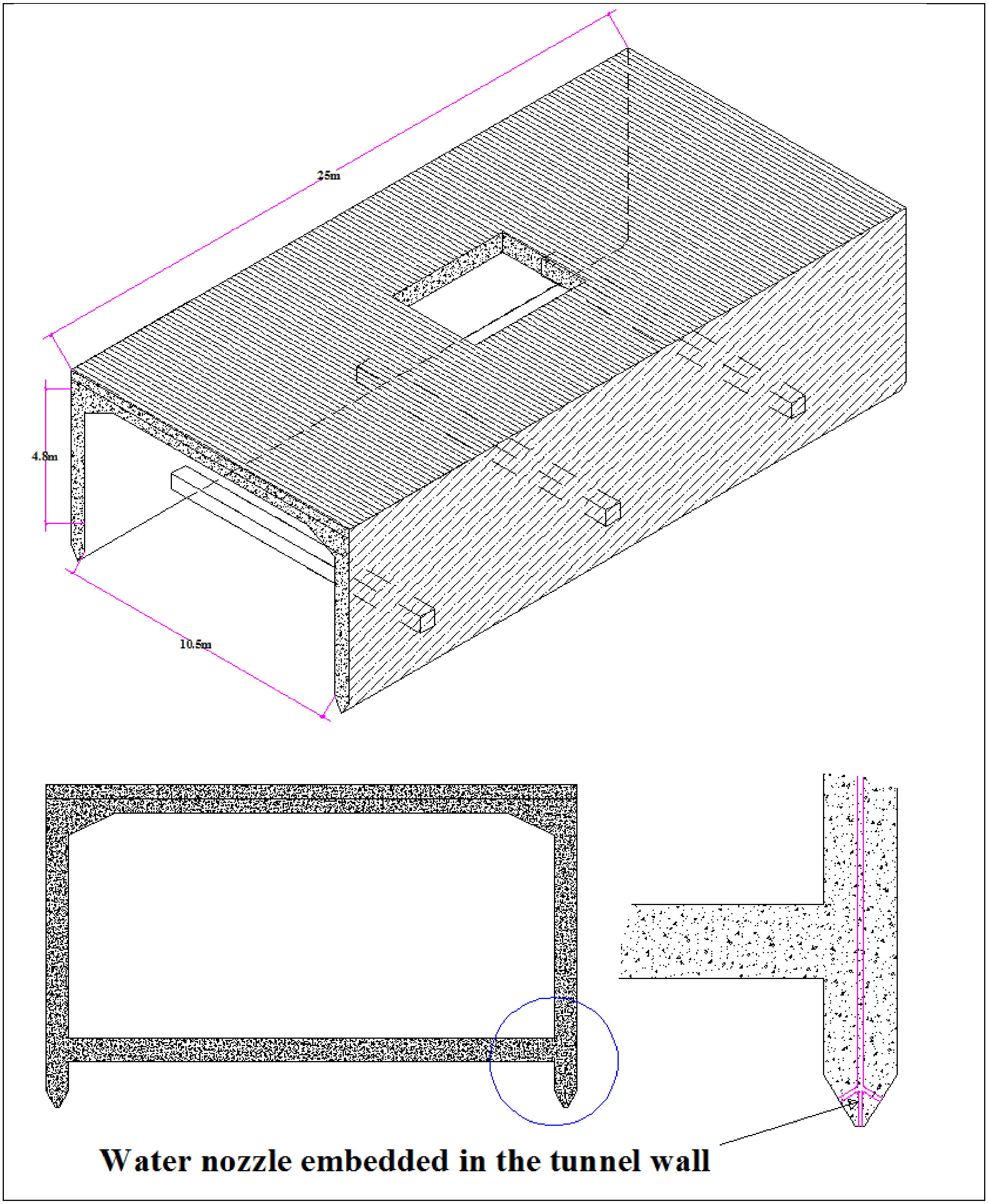


Figure 3-6: Geometry and dimension of a single section of caisson tunnel

4. EMPIRICAL ANALYSIS

4.1. Introduction

In the empirical analysis the bearing capacity failure, the penetration resistance at the tip of the structure and basal heave due to excavation were analyzed. Bearing capacity failure and settlement under the tip of the structure could be an indication of the sinking of the structure. The ultimate bearing capacity of the soil and the vertical stress due to self-weight of the structure at different depths were compared to predict whether the caisson tunnel will sink or not. In addition, the depth where the total penetration force which is the sum of the tip resistance and skin friction exceeds the down ward force and prevent the sinking of the caisson tunnel was evaluated.

4.2. Bearing Capacity

For the caisson tunnel to sink to the desired depth due to self-weight of the structure there should be bearing capacity failure. The structure is considered as strip footing and the ultimate bearing capacity is compared with the pressure due to self-weight of the structure. If the pressure due to self-weight of the structure is less than the ultimate bearing capacity, q_u, the factor of safety will be less than one and thus there will be bearing capacity failure. ($FS=q_{ult}/q_{allow}$).

The total self-weight of the caisson tunnel is about 7000kN. The tip area of the caisson wall is 25m^2. The bottom area of the beams depends on the number and width of the beams. If the number and width of the beams increase, the weight of the structure rests on larger area and bearing capacity failure will not happen. Thus, for single caisson tunnel of 25m three beams with 0.4m width are recommended to be used during installation. If the load of the structure rests on beam and wall tip the vertical stress due to self-weight of the structure is 185kN/m^2. If the load of the structure rests only on the wall the vertical stress due to self-weight is 280kN/m^2.

The caisson tunnel should be casted at the site. The top crust of the soil may not be level and hard to penetrate, therefore it should be removed. The beam of the caisson tunnel will be casted on top of the sandy clay layer. The clay layer has relatively higher undrained shear strength. If the walls of the caisson tunnel will be casted on top of this hard layer the structure may not be able to penetrate and sink down. Therefore the walls should be casted in trench in the sandy clay layer. The maximum penetration depth required is 8m below ground level.

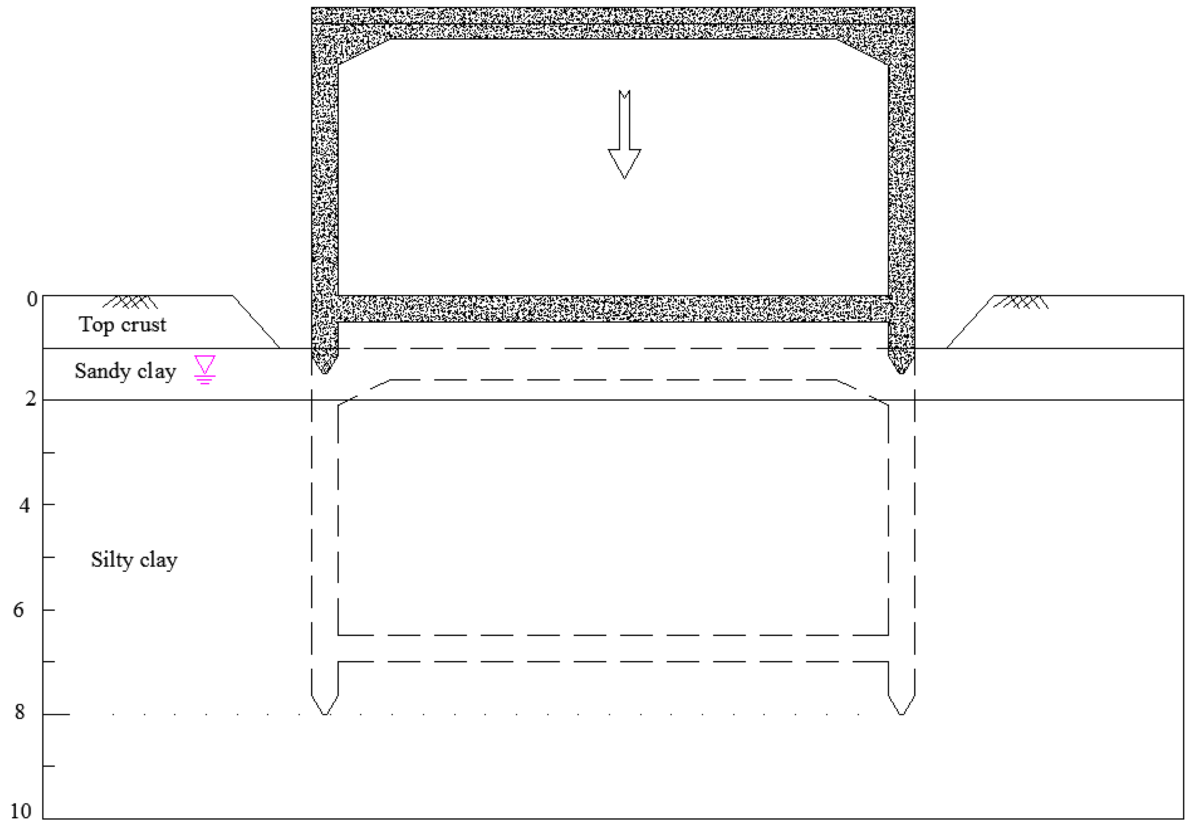


Figure 4-1: Penetration of Caisson Tunnel into the soil layers

The vertical stress due to self-weight of the caisson tunnel when the load rests only on the walls is about 280kPa (=7000kN/25m^2) whereas when the load rests on walls and the beams is about 185kPa (=7000kN/37.6m^2). The comparison of vertical stress due to self-weight of the structure and the ultimate bearing capacity of the soil without using safety factor for material property is shown in the figure below.

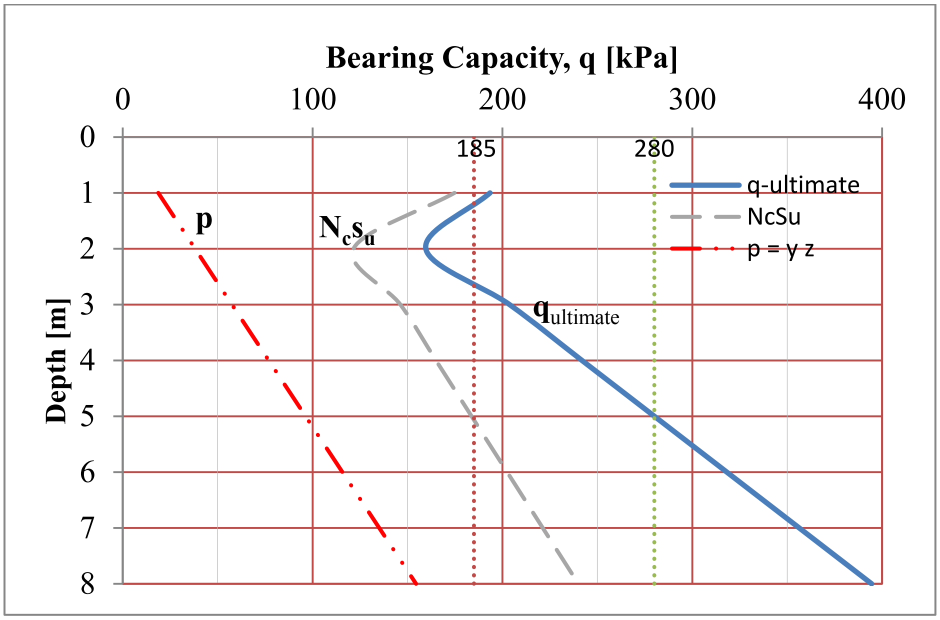


Figure 4-2: Ultimate bearing capacity and self-weight

When the weight of the structure rests on beam and the wall tips there will a bearing capacity failure due to self-weight of the caisson tunnel between 1.2m to 2.6m depth. If the internal part of the caisson tunnel is excavated heaving will happen and this increase bearing capacity failure. In this case there is high tendency of bearing capacity failure until the structure sinks up to 5m depth.

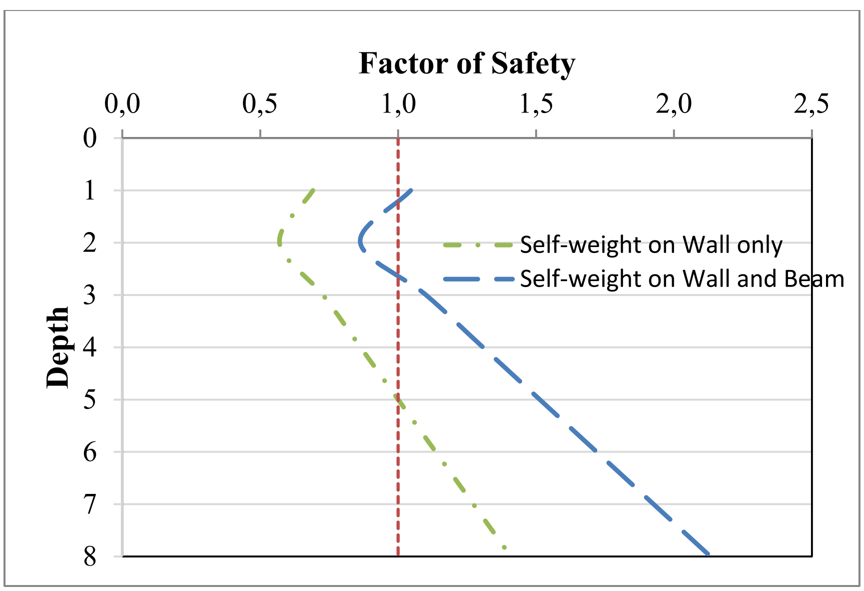


Figure 4-3: Factor of safety against bearing capacity failure

There is bearing capacity failure doesn't mean that the structure can easily sink to the desired depth. But it can give some hint about the resistance of the soil during installation. Ultimate bearing capacity increases with increasing overburden pressure and thus excavating the inner side of the wall will create basal heave and decrease the overburden and results in bearing capacity failure.

4.3. Penetration Resistance against Self-weight

The caisson tunnel under study is expected to sink to the desired depth due to self-weight and external additional static load. For the structure to penetrate and sink into the soil, the pressure due to self-weight of the structure should exceed the penetration resistance from the soil.

It is very difficult to determine the penetration resistance against the whole caisson tunnel at once. It is preferable to consider the caisson tunnel wall as a rectangular pile. To determine the penetration resistance of caisson tunnel under study the theoretical concepts of suction caisson and pile are used. The penetration resistance can be determined in a manner similar to that used for vertically loaded piles in clay by considering the caisson tunnel wall as a rectangular pile. Using limit equilibrium theory, the vertical resistance on the caisson tunnel wall (Q_{tot}) can be calculated as the sum of the side shear along the caisson tunnel walls and the end bearing on the wall tip.

$$Q_{tot} = Q_{side} + Q_{tip} = \alpha . s_{\mathrm{u,ave}} . A_{\mathrm{wall}} + \left(N_{\mathrm{c}} . s_{\mathrm{u,tip}} + \gamma \mathrm{z}\right) A_{\mathrm{tip}} \qquad (6)$$

The side shear (Q_{side}) is calculated following the usual practice in pile design, by applying a factor α to the value of the undrained shear strength. The end bearing resistance (Q_{tip}) is calculated following bearing capacity analysis as the sum of an N_c and an N_q (1 for undrained analysis) term.

Where α is an empirical constant (friction coefficient) which depends on sensitivity of the soil. For pile α value of 0.3 to 1.0 is recommended depending on s_u and the material in the structure. A_{wall} is the total surface area along the caisson tunnel's wall over which shearing resistance is generated and $s_{u,ave}$ is the average undrained shear strength. N_c is a bearing capacity factor, $s_{u,tip}$ is the undrained shear strength at the tip of the wall, γ is the unit weight of the soil, z is the penetration depth and A_{tip} is the base area of the caisson tunnel wall (Solhjell, Sparrevik et al. 1998).

The construction of the concrete structure should be started at a depth where the sum of the tip resistance and the skin friction exceeds the total load of the structure to prevent down ward movement of the structure during construction. Otherwise it is preferable to increase the area on which the self-weight of the caisson tunnel rests so that the vertical stress will be reduced.

The self-weight load per meter length is 140kN/m. For depth up to 3m the total penetration resistance is less than self-weight of the structure and this indicates the caisson tunnel may sink due to its weight up to 3m depth. For depth below 3m to sink the structure either additional static force is required or the penetration resistance should be reduced.

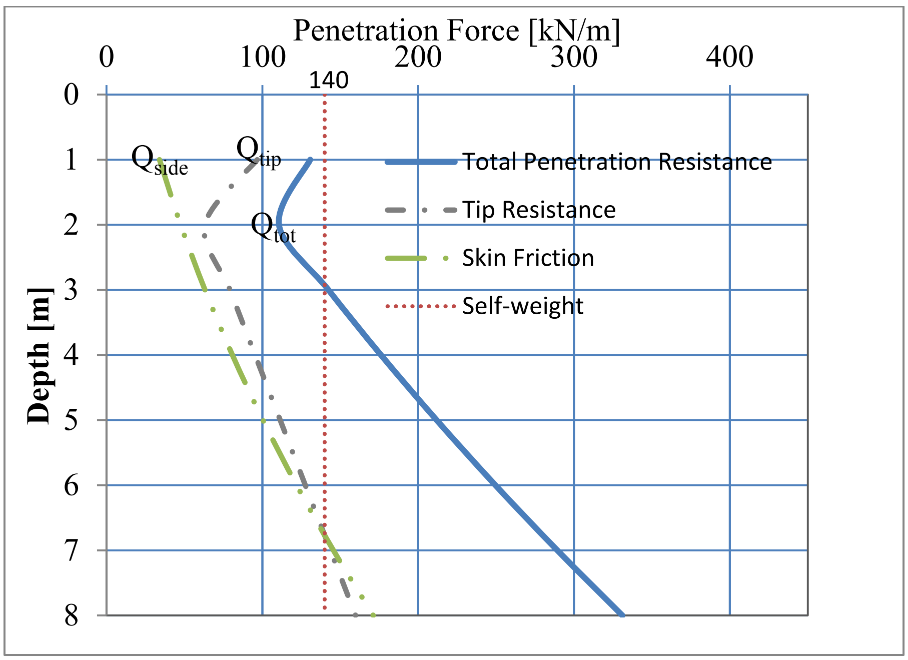


Figure 4-4: Penetration resistance versus depth

According to Ervin Nonveiller, (1987) one of the controlling factor for smooth sinking of the open caisson is the ratio between total weight, W, and the friction, Q_s, on the surface of the caisson in contact with the surrounding soil. This ratio must be greater than 2. However, after depth of 3.4m the ratio W/Qs is less than 2 and thus smooth sinking could be difficult. Therefore, the skin friction should be reduced.

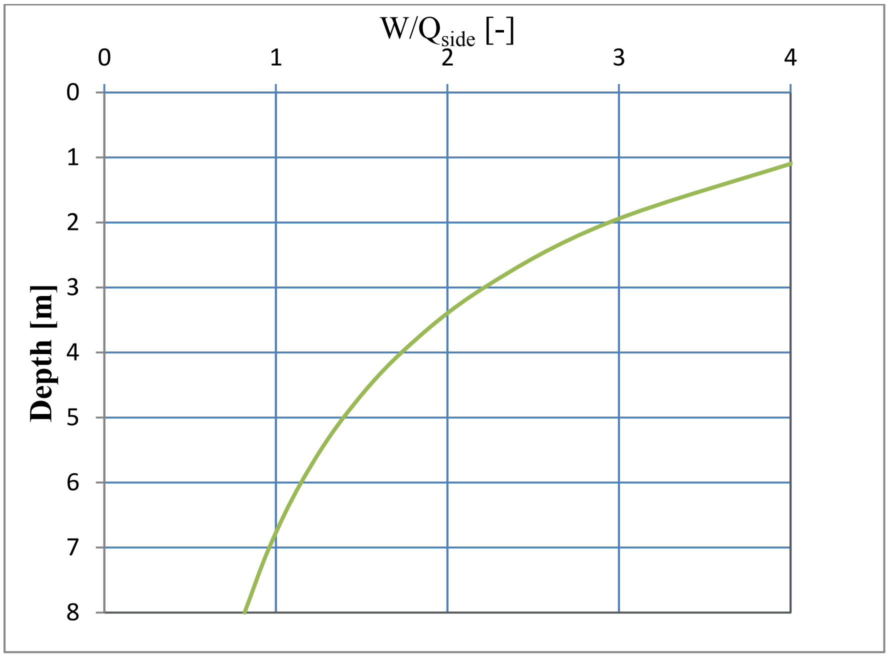


Figure 4-5: Ratio of self-weight to skin friction of caisson tunnel

Some mechanisms to reduce the penetration resistance could be:

- Excavating the inner side of the caisson tunnel so that the skin friction will be reduced.
- Decreasing the shear strength of the soil with jetting water under the tip of the wall
- The tip of the caisson tunnel wall should be not flat. If the wall tip is inclined by 45^0 to 60^0 on both sides of the wall the penetration resistance is reduced. Inclining the tip can reduce the penetration force by about 10%.

Reducing the penetration resistance by certain factor using these mechanisms may help to sink the structure up to 5m depth.

4.4. Heave due to Excavation

One of the problems during deep excavation is heaving of soil. The factor of safety against basal failure is required to be greater than 1.5; if it is less than 1.5 the wall should penetrate below the base of cut to insure stability. As shown in the figure below there is no serious problem of basal heave, but still it is important to elongate the wall below the base of the cut.

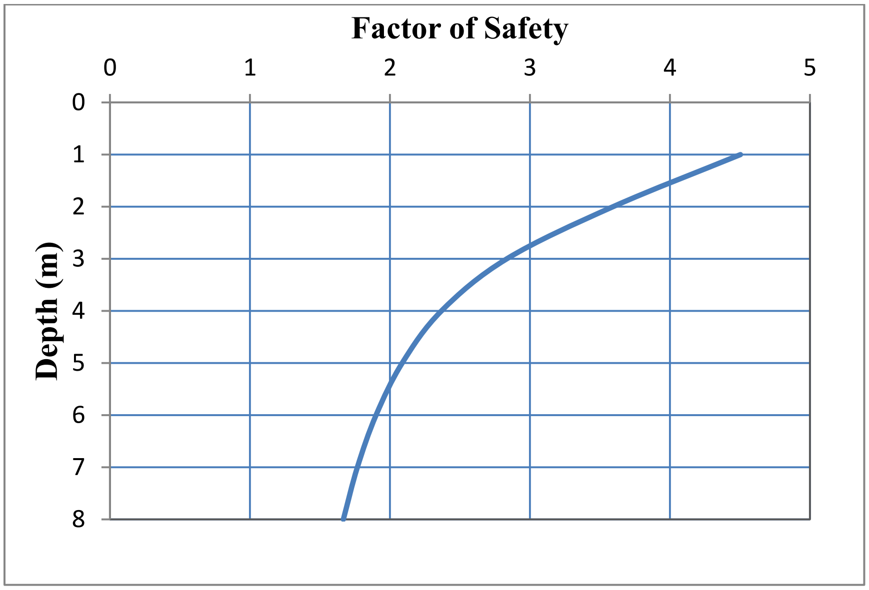


Figure 4-6: Factor of safety against basal heave

5. NUMERICAL ANALYSIS

5.1. Introduction

In geotechnical engineering design and analyses, traditional empirical and semi-empirical methods are commonly used. These empirical and semi-empirical methods are simple and straightforward. But sometimes the results from these methods have shortcomings. These methods do not take into account the soil-structure interaction, pore water pressure, duration and etc. Thus, they may not provide accurate results under complex construction conditions. The numerical analysis models such as PLAXIS could give more reliable predictions.

It is difficult to simulate the penetration of a structure in to soil due to self-weight or by applying external load. One of the options to simulate this is by ‘‘wished-in-place’’ condition. In this chapter PLAXIS was used to simulate the penetration of the caisson tunnel by placing the structure at different depths and evaluate different scenarios.

5.2. FEM PLAXIS 2D

For numerical analysis PLAXIS is used. PLAXIS is a general purpose geotechnical finite element program suitable for wide range of geotechnical processes. It provides several material models to model the behavior of the soil.

The Mohr-Coulomb model was used for this analysis. This model assumes the material exhibits linear elastic perfectly-plastic behavior. The drainage type used for the analysis is undrained B which uses the undrained strength parameters (c’=cu φ’=0). This is a short-term material behavior in which stiffness is defined in terms of effective properties and strength is defined as undrained shear strength. In undrained condition no water movement takes place and excess pore pressure is developed. Undrained analysis is appropriate when rate of loading is high and to assess short term behavior.

Plane strain model with 15-node element and fine mesh is used where the mesh along the tunnel wall is refined to increase the accuracy.

5.3. Material Properties used in PLAXIS

The soil in the selected profile has three layers. The top crust has relatively higher undrained shear strength. Below the crust a sandy clay layer of 1 meter thick has average undrained shear strength of 34 kPa. The thick silty clay layer which is located below 2 meter depth has linearly increasing undrained shear strength of 17+2.5z kPa. The caisson tunnel is modeled as non-porous material with concrete properties. To model the soil-concrete interaction an interface ($R_{interface}$) value of 0.7 and 0.6 are used for sandy clay and silty clay layers respectively.

Table 5-1: Material properties of the soil

Parameter	Unit	Sandy clay	Silty clay	Concrete wall
Material model	-	Mohr coulomb	Mohr coulomb	
Drainage type	-	Undrained B	Undrained B	Non-porous
Unit weight, γ	kN/m^3	18.7	19.4	25
Young's modulus, E	kN/m^2	5000	3500	$30x10^6$
Poisson's ratio, υ	-	0.33	0.33	0.15
Undrained shear strength	kN/m^2	34	17+2.5z	
Friction angle, φ	^{0}	0	0	
Dalitancy angle, ψ	^{0}	0	0	
Interface, $R_{interface}$		0.7	0.6	

5.4. Modeling of the caisson tunnel penetration

It is impossible to simulate the penetration of the caisson tunnel into the soil in PLAXIS. In order to investigate the penetration of the structure only due to its self-weight, the structure is placed at different depths and the soil in the internal side of the caisson tunnel is excavated by phase up to the tip of the wall. If the soil beneath the tip of the wall collapses there might be possibility of the sinking of the structure.

The finite element analyses were performed by considering the tunnel wall as strip footing with a plane strain model. The top crust soil which has relatively higher undrained shear strength will be removed and the tunnel wall will be casted in the sandy clay layer at a depth of 1.5m below ground level. The soil inside the tunnel will be excavated by phase up to the tip of the wall.

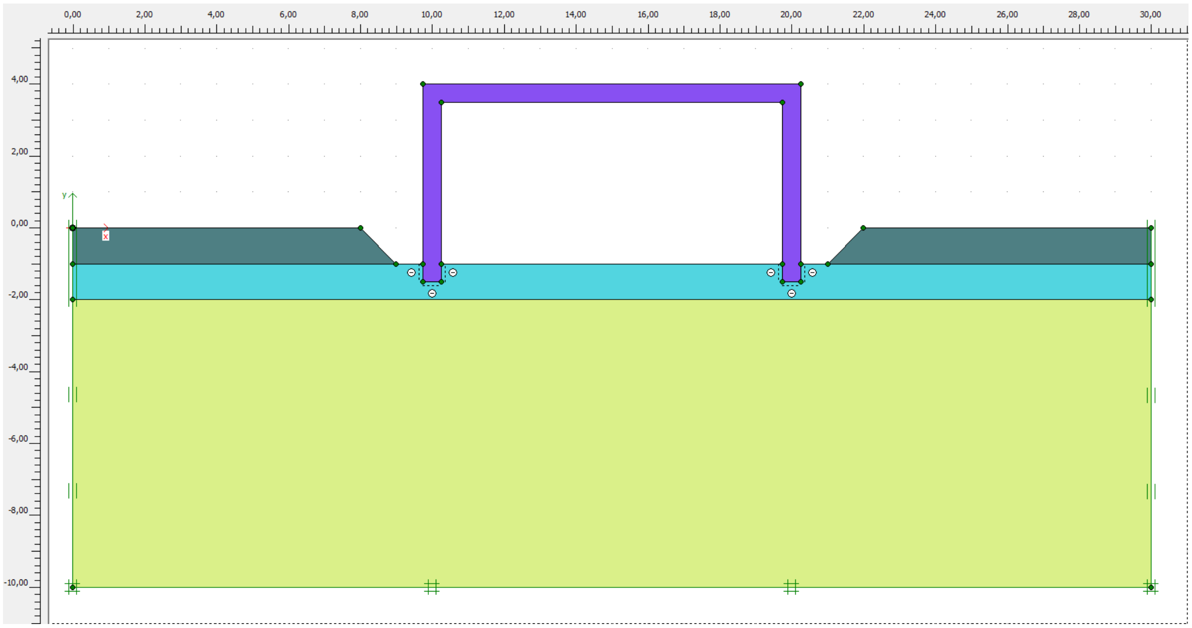


Figure 5-1: Finite element model of the caisson tunnel

The excavation is simulated by a staged construction phase, where the corresponding cluster that is excavated is deactivated and the pore pressures are defined by defining the cluster of the excavated soil to be dry.

In order to sink the caisson tunnel to the desired depth the soil under the tip of the wall should collapse due to self-weight of the structure. The maximum target depth of penetration is 8m. At the depth of 1.5m where the structure is initially modeled the soil under the tip of the wall doesn't collapse. However, when the soil inside the tunnel is excavated up to the tip of the wall the soil collapsed.

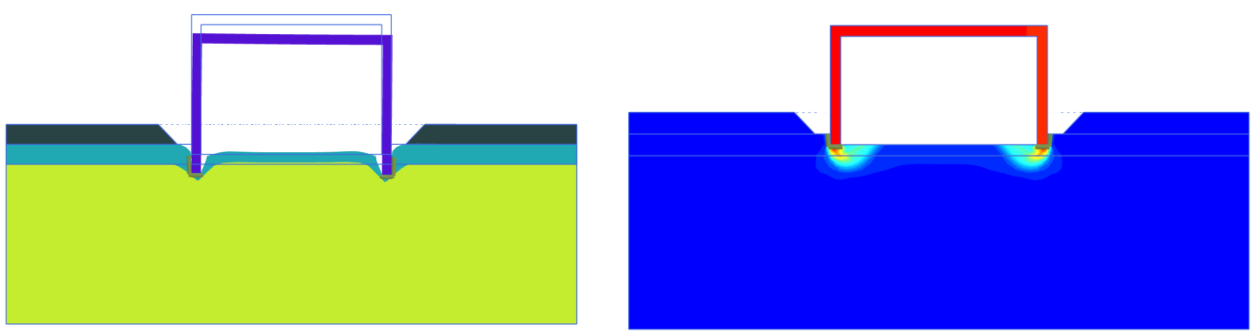


Figure 5-2: Deformed mesh after the inner side is excavated up to wall tip at 1.5m

The penetration resistance is mainly due to the skin friction against the walls of the caisson tunnel and tip resistance at the wall tip. The presence of the beam elements at the rim of the caisson tunnel will increase the penetration resistance. Therefore, limiting the number and bottom area of beams is important to reduce the penetration resistance. Three beams with 40cm width are assumed to be used 1m above the tip of the wall. Struts should be used at certain spacing. The soil below the beams will be excavated as the structure sinks down. In addition to keeping the structure stable the beams also prevent the movement of the structure if there will be differential sinking.

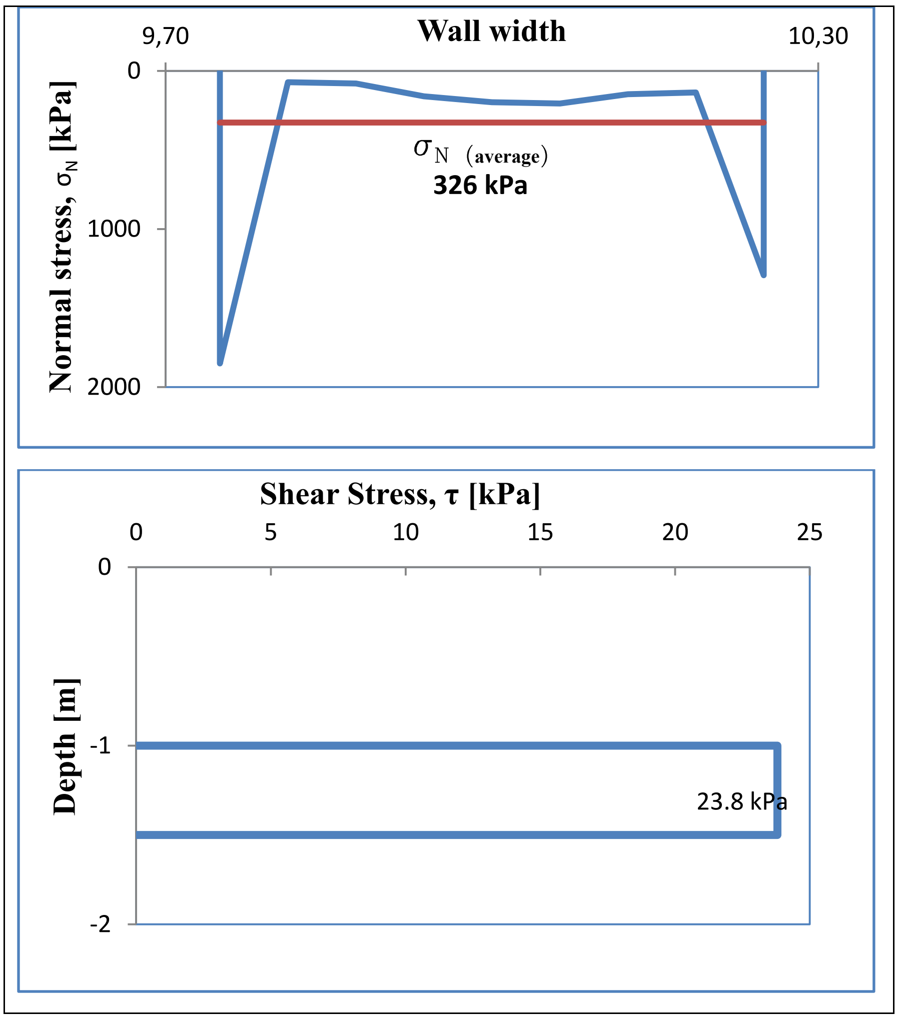


Figure 5-3: Normal stress and shear stress distribution below the tip of the wall at 1.5m before excavation

The total penetration resistance is the summation of skin friction and tip resistance. At 1.5m depth before the interior side of the tunnel is excavated the skin friction and tip resistance are 163kN/m and 12kN/m respectively. The total penetration resistance is greater than the self-weight of the caisson tunnel and the soil at the tip of the wall doesn't collapse. However, when the interior side of the tunnel is excavated the soil started to collapse.

The Finite Element analyses were performed by putting the caisson tunnel at different depth; and excavating the inner side of the caisson tunnel by phase. The analyses result at all depths showed that the structure will not sink down before the inner soil will be excavated. But up to depth of 5.5m the structure will sink when the soil is excavated up to the tip of the tunnel wall. However, after 5.5m depth the soil at the tip of the wall will not collapse even if the whole soil in interior side of the wall is excavated and removed.

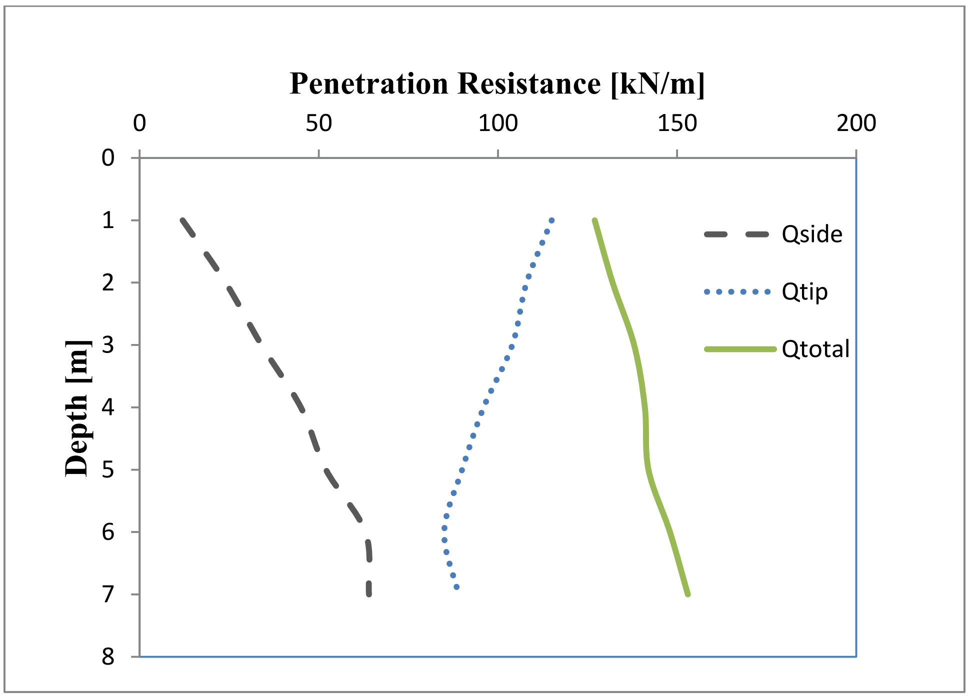


Figure 5-4: Penetration resistance after excavation

As shown from the figure above the major factors that prevent the penetration of the structure are the tip resistance and the skin friction. Some mechanisms should be used to reduce these resistances.

For the caisson wall at 2m depth before the internal part of the tunnel is excavated the soil doesn't fail and the factor of safety due to self-weight of the structure was found to be 1.32. However when the inner side of the tunnel is excavated 1m below ground level the soil body below the tip of the wall collapse and the factor of safety is less than 1.

5.5. Wall deformation and Earth Pressure

During tunneling and deep excavations the structures in the proximity might be damaged due to vertical and horizontal movement of the soil. The movement of the soil and retaining structures should to be controlled within the acceptable levels. Thus, prediction of soil subsidence and wall deflection before excavation is very important to evaluate the effects on the adjacent structures and then adopt appropriate countermeasures.

According to Ye Lu et al. (2012) most excavation analyses by numerical methods are achieved by back-analysis and there are difficulties in giving reliable prediction before excavation. Especially for excavations in sensitive soils, soil strength and stiffness degrade if the soil is disturbed and few models are capable of taking accounts of degradation effects.

The tunnel walls and slab are modeled as a plate element having stiffness property of concrete and interface elements around the walls. The beam is modeled as node-to-node anchor. In this analysis Mohr-Coulomb model is used due to lack of soil parameters to be used in other advanced models. The advanced models give more reliable results.

From the analysis result the horizontal deflection of a 0.5m thick caisson tunnel wall immediately after excavation is 12mm and when the thickness of the wall is reduced to 0.4m it is increased to about 14mm.

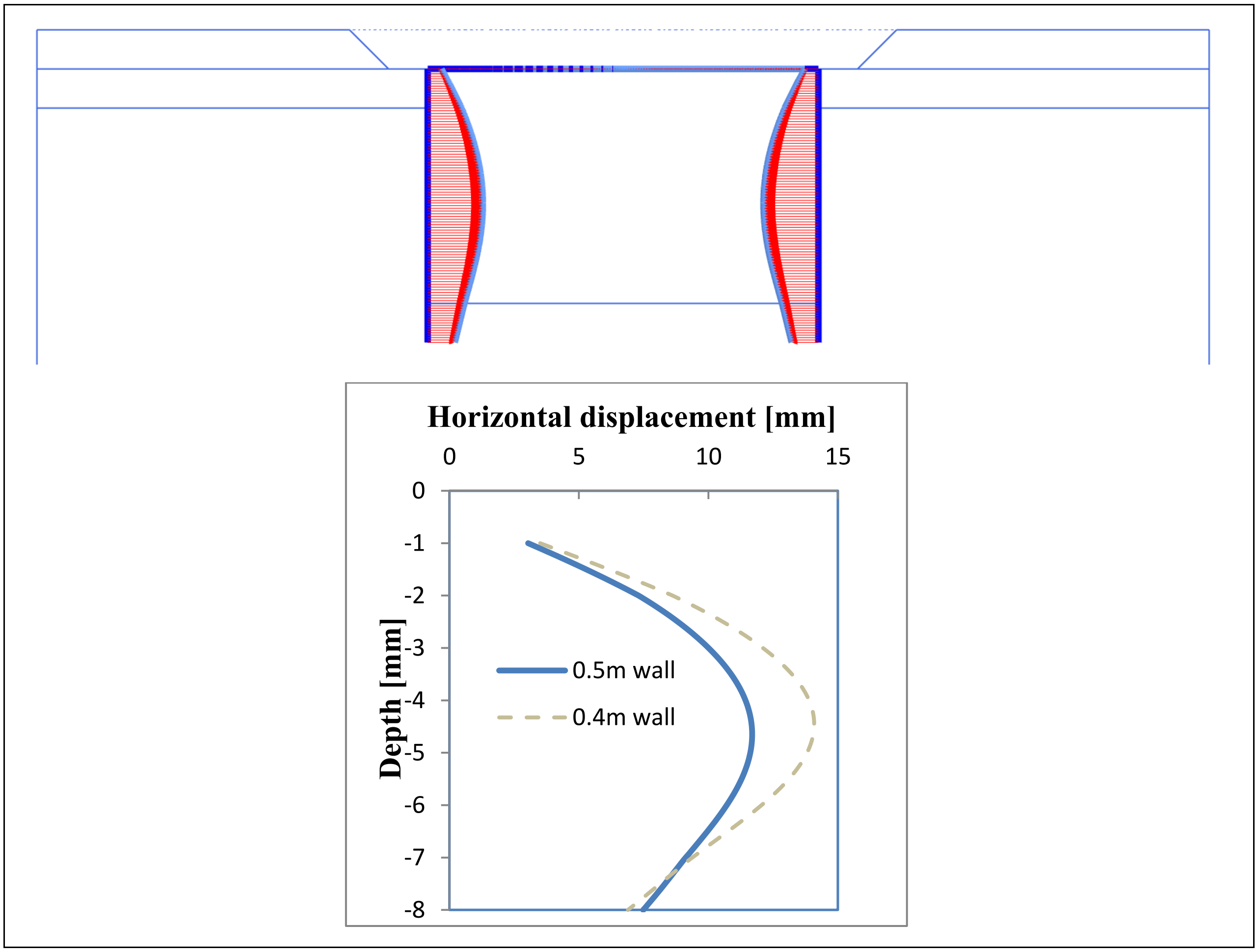


Figure 5-5: Horizontal deformation of the caisson tunnel wall after excavation

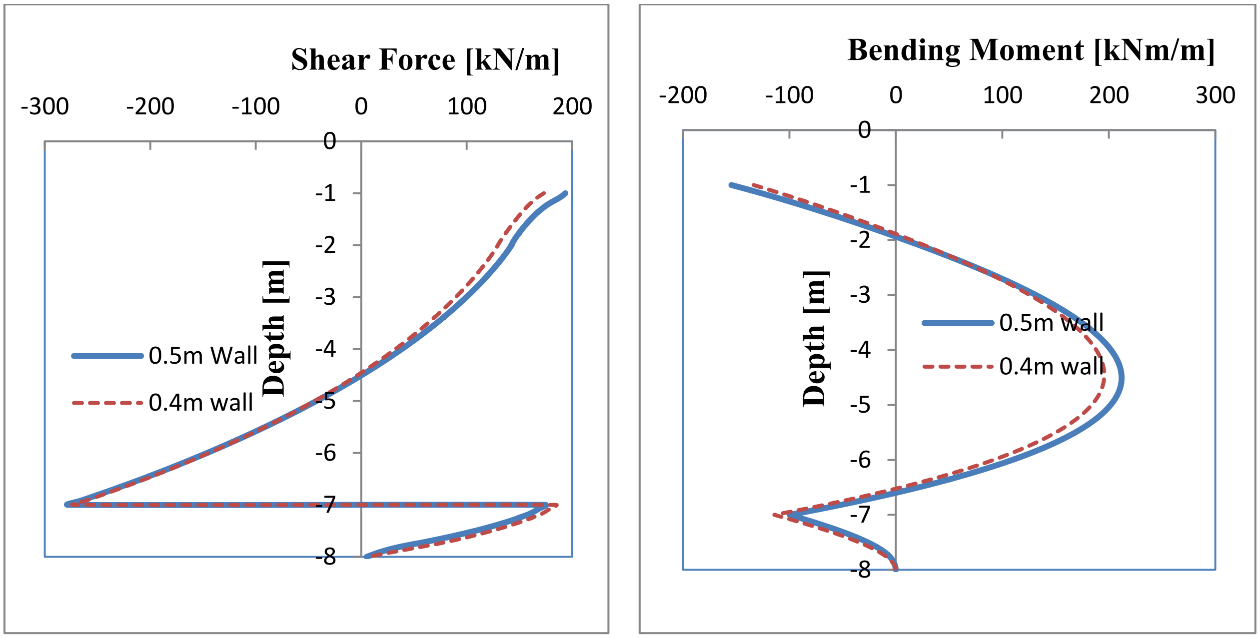


Figure 5-6: Shear force and bending moment diagrams of the tunnel wall after excavation

6. SUMMARY AND CONCLUSION

6.1. Summary of Results

The main objective of this study is to evaluate the feasibility of the new concept of sinking caisson tunnel in soft soil. To evaluate the geotechnical feasibility of the concept empirical and numerical analyses were performed for a selected profile along the proposed tunnel alignment.

In empirical analysis the ultimate bearing capacity failure at different depth, the penetration resistance and the basal heave due to excavation were investigated. The ratio of the self-weight of the caisson tunnel to the skin friction which might have some implication about smooth sinking of the structure was also evaluated and compared with other results from empirical and numerical analyses.

In the bearing capacity analysis the whole caisson tunnel is considered as strip footing. The ultimate bearing capacity of the soil at different depth is compared with the vertical stress due to self-weight of the caisson tunnel. From the analysis it was found that for depth less than 2.6m the ultimate bearing capacity is less than the vertical stress due to self-weight of the caisson tunnel. This indicates that the structure might sink up to this depth if the deformed soil at the tip of the beam and wall is removed using some mechanisms. On the other hand, if the self-weight of the caisson tunnel is made to lie only on the walls by excavating under the beams, the vertical stress will increase and the bearing capacity failure could continue up to 5m depth.

The vertical force that prevents the sinking of the structure is the penetration resistance which is the sum of the skin friction and tip resistance. To compare the penetration resistance with self-weight of the caisson tunnel the structure is considered as rectangular pile. For depth less than 3m the total penetration resistance is lower than the load of the caisson tunnel and there is a possibility of sinking of the structure up to this depth. In order to lower down the structure below this depth the tip resistance and the skin friction should be reduced. If the inner side of the tunnel is excavated the skin friction will be reduced and the structure could sink down.

From the theories of open caissons (Nonveiller 1987) one of the controlling factors for smooth sinking of the caisson tunnel could be the ratio between total weight and the skin friction. This ratio must be greater than 2 for smooth sinking of the structure. The self-weight to skin friction ratio is greater than 2 for depth less than 3.4m and thus the caisson tunnel could sink smoothly. However, after depth of 3.4m the ratio of self-weight to skin friction is less than 2 and thus smooth sinking may be difficult. However, when the inner side of the caisson tunnel is excavated the skin friction could be reduced and sinking of the structure to lower depth may be possible.

One of the problems during deep excavation is heaving of soil. If the factor of safety against basal heave is less than 1.5 the wall should penetrate below the base of cut to insure stability. The factor of safety against basal failure was found to be greater than 1.5. Although there is no serious problem of basal heave still it is important to elongate the wall below the base of the cut.

The Finite Element analyses were performed by putting the caisson tunnel at different depth; and excavating the inner side of the caisson tunnel by phase. The analyses result at all depths showed that the structure will not sink down before the inner soil will be excavated. But up to depth of 5.5m the structure will sink when the soil is excavated up to the tip of the tunnel wall. However, after 5.5m depth the soil at the tip of the wall will not collapse even if the whole soil in interior side of the wall is excavated and removed.

6.2. Conclusion

The results from both empirical and numerical analyses show that the caisson tunnel could sink due to self-weight up to approximately 5m below ground level when the inner side of the tunnel is excavated. In order to smoothly sink the caisson tunnel the resisting forces should be reduced. The major resisting forces that prevent the sinking of the caisson tunnel are the tip resistance and the skin friction along the surface of the wall. Some of the mechanisms to reduce the resisting forces could be:

- Excavating the inner side of the tunnel will reduce the skin friction
- Smooth coating the outer surface of the tunnel with tough coat which could not be rubbed off easily when the caisson tunnel descends
- Tip geometry: inclined tip walls may have lower tip resistance than flat tip
- Bentonite could be used as lubricant to assist penetration of the tunnel
- Water/air jetting through nozzles embedded in the tunnel wall which descends to the wall tip

The water jetting nozzle in the tunnel wall could help the following purposes:

- To reduce the shear strength of the soil at the tip of the wall by jetting water/air so that the caisson tunnel could sink smoothly
- In case of differential sinking during installation of the caisson tunnel the soil at the tip of the tunnel wall could be stabilized intermittently using salt or other appropriate stabilizing material
- When the structure reach the desired depth permanent stabilization of the soil with cement grouting is very important to improvement the strength of the soil with jet grouting underneath the foundation
- To reduce drawdown: the lowering of groundwater table due to excavation in the inner side of the wall may create drawdown of water in the outer side of the tunnel. The drawdown could cause settlement on the nearby structures and this could be minimized by injecting water through the nozzles embedded in the tunnel wall.

The alignment of the caisson tunnel is close to the railway and there are buildings on both sides of the road. To avoid damage on existing structures accurate estimation of ground movement should be predicted. As the ground conditions in the study area are very challenging due to the presence of very thick layers advanced constitutive soil models of PLAXIS have to be used to obtain reliable results. In order to use the advanced soil models adequate soil investigation data is very important.

The new concept of sinking caisson tunnel was primarily proposed as an alternative method of tunneling in soft clays. The soil profile selected in this study has relatively higher undrained shear strength and the results from both empirical and numerical analyses show that it is difficult to sink the caisson tunnel below 5m unless the resisting forces are reduced. The method of sinking caisson tunnel could be feasible for soft soils with lower undrained shear strength than the selected profile.

Major Challenges during Installation of Caisson Tunnel:

- Differential sinking
- Excess pore pressure
- Structural cracks at joints

6.3. Possible future work

In order to evaluate the feasibility of the idea some future works should be conducted. The following future works are suggested:

- Since tunnel design and construction is a team work experts from different discipline could work on the problem to evaluate the feasibility of the concept from different view point. Thus, it is recommended if the feasibility of the concept is evaluated from geotechnical and structural engineering point of view with expertise from both disciplines.
- In addition to evaluate the theoretical feasibility of the concept it is recommended if a prototype of the caisson tunnel with smaller dimensions is casted at the site and tested to evaluate the practical challenges of sinking the structure.

6.4. Limitation

Due to the lack of adequate soil data in the selected study area advanced soil models were not used for numerical analysis. The other limitation of this study is the lack of relevant case study.

REFERENCES

Abdrabbo, F. and K. Gaaver (2012). "Challenges and Uncertainties Relating to Open Caissons." Journal of Deep Foundations, DFI **6**(1): 21-32.

Andersen, K. and H. Jostad (1999). Foundation design of skirted foundations and anchors in clay. Offshore Technology Conference.

Houlsby, G. T. and B. W. Byrne (2005). "Design procedures for installation of suction caissons in clay and other materials." Proceedings of the ICE-Geotechnical Engineering **158**(2): 75-82.

Hung, C. J. and B. Parsons (2009). Technical Manual for Design and Construction of Road Tunnels--civil Elements, US Department of Transportation, Federal Highway Administration, National Highway Institute.

Korf, M. (2009). Deformation and Damage to Buildings adjacent to Deep Excavations in Soft Soils, Deltares**:** 143.

Lu, Y., Y. Tan, et al. (2012). FE Simulation of Deep Excavations in Sensitive Soft Clays. GeoCongress 2012@ sState of the Art and Practice in Geotechnical Engineering, ASCE.

Murthy, V. (2002). Geotechnical engineering: principles and practices of soil mechanics and foundation engineering, CRC Press.

Nonveiller, E. (1987). "Open caissons for deep foundations." Journal of geotechnical engineering **113**(5): 424-439.

Potts, D. M. and L. Zdravković (2001). Finite element analysis in geotechnical engineering: application, Thomas Telford Services Limited.

Puller, M. (2003). Deep excavations: a practical manual, Thomas Telford.

Rajapakse, R. (2011). Geotechnical engineering calculations and rules of thumb, Butterworth-Heinemann.

Skempton, A. (1951). "The bearing capacity of clays."

Solhjell, E., P. Sparrevik, et al. (1998). Comparison and Back Calculation of Penetration Resistance From Subtion Anchor Installation In Soft To Stiff Clay At The Njord And Visund Fields In The North Sea. Offshore Site Investigation and Foundation Behaviour'New Frontiers: Proceedings of an International Conference.

Statens vegvesen Region sør, R. V. s. (2011). Fv.32 Gimlevg-Augestadvg. Geoteknikk. 26070-470**:** 23.

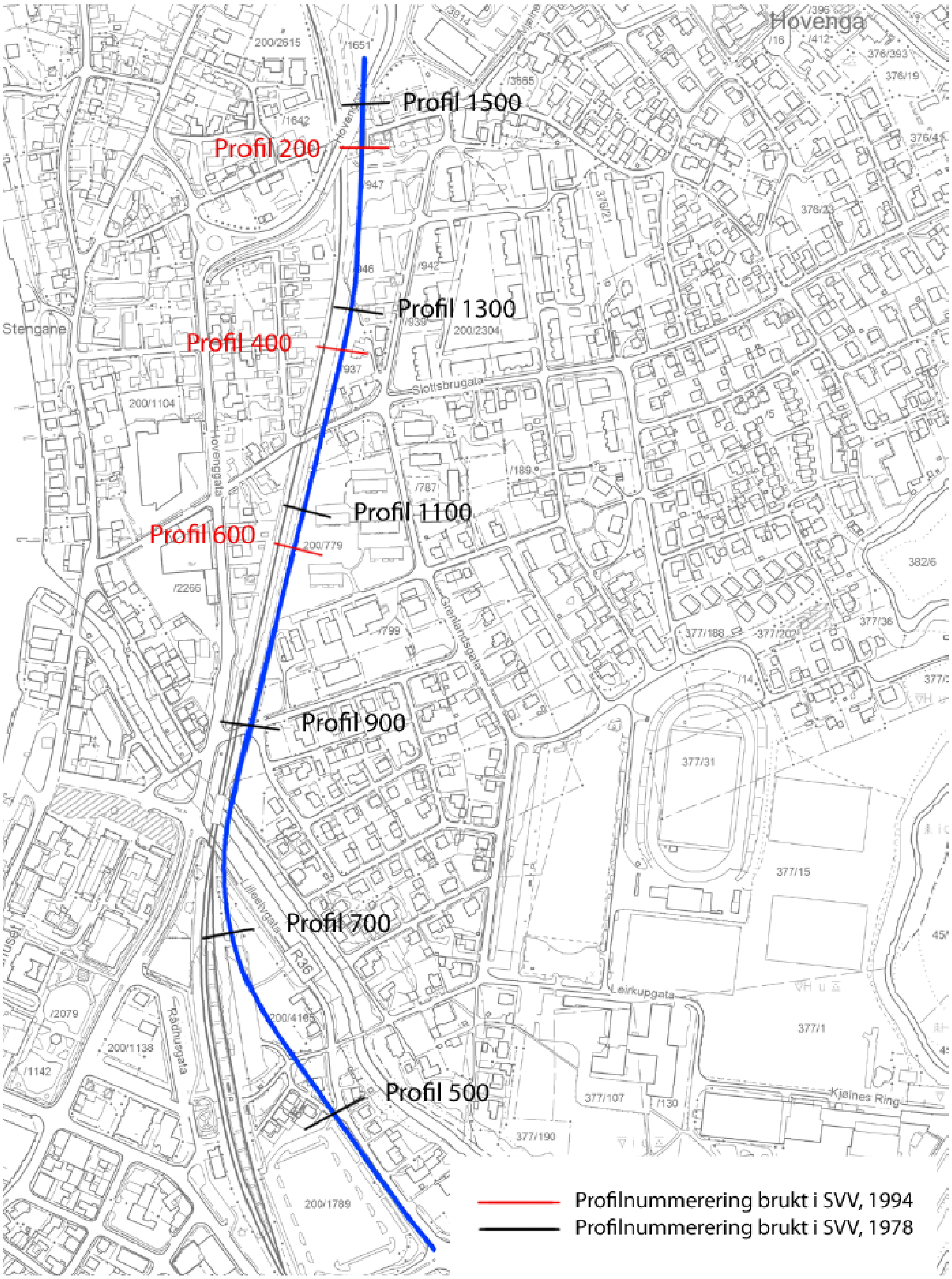
Terzaghi, K., R. B. Peck, et al. (1996). Soil mechanics in engineering practice, Wiley-Interscience.

Vesic, A. (1963). "Bearing capacity of deep foundations in sand." Highway research record.

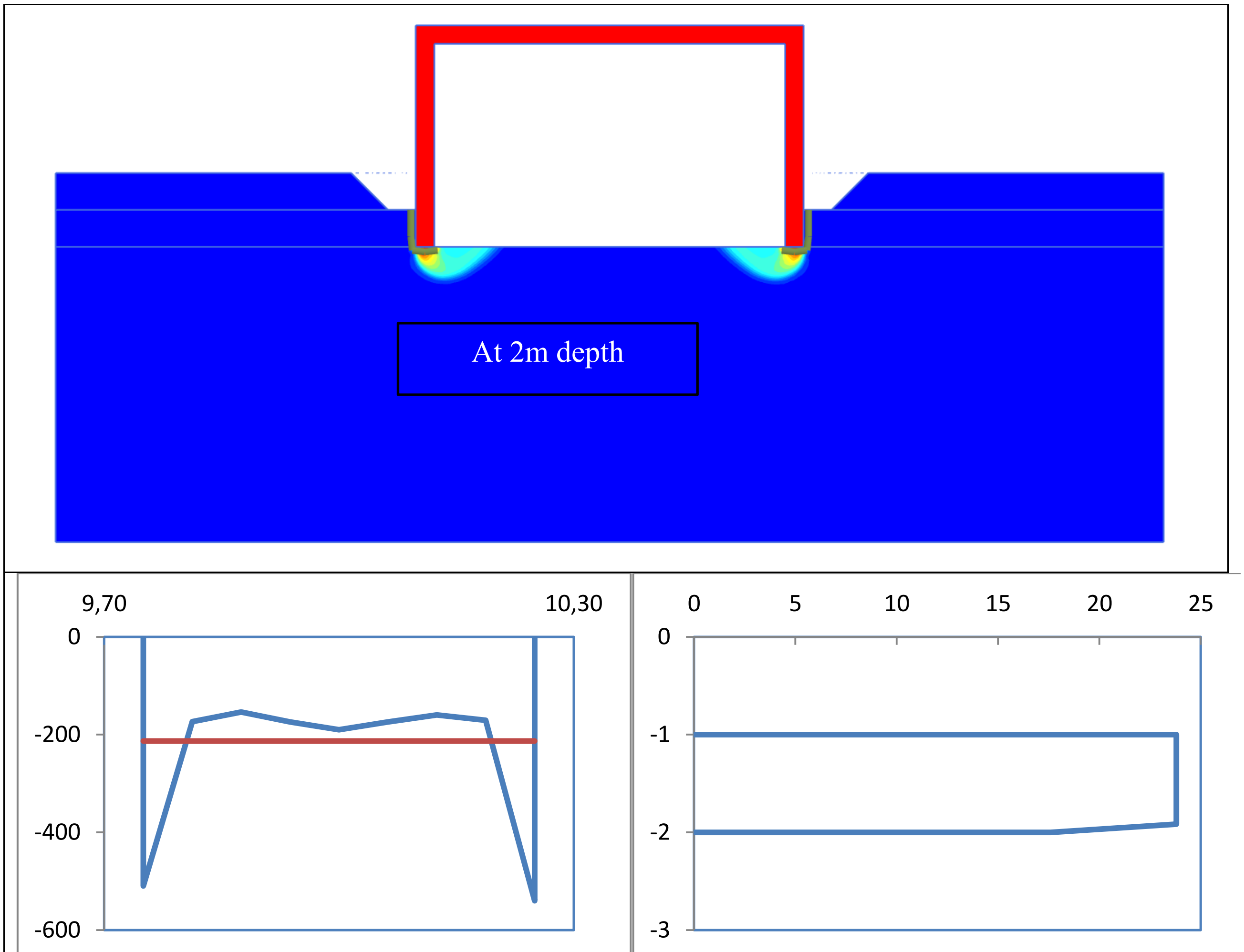
Whittle, A. J., Y. M. Hashash, et al. (1993). "Analysis of deep excavation in Boston." Journal of geotechnical engineering **119**(1): 69-90.

APPENDIX

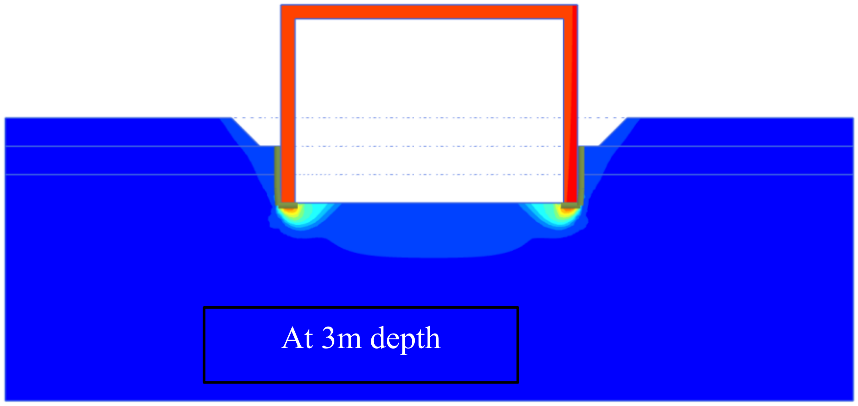
Appendix A: Profile Numbering in Previous Investigations (SVV, 2011)

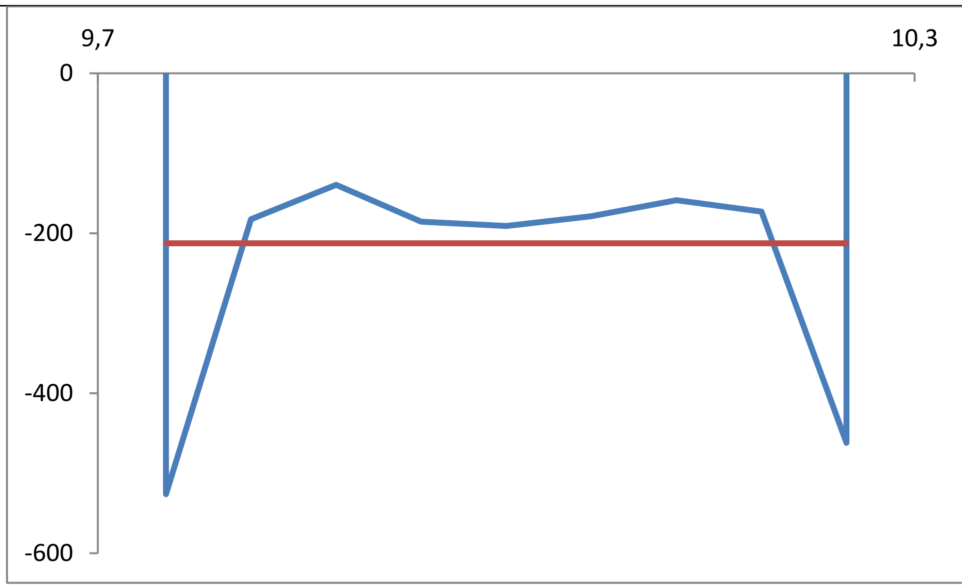


Appendix B: Tip Resistance and Skin Friction at different depth

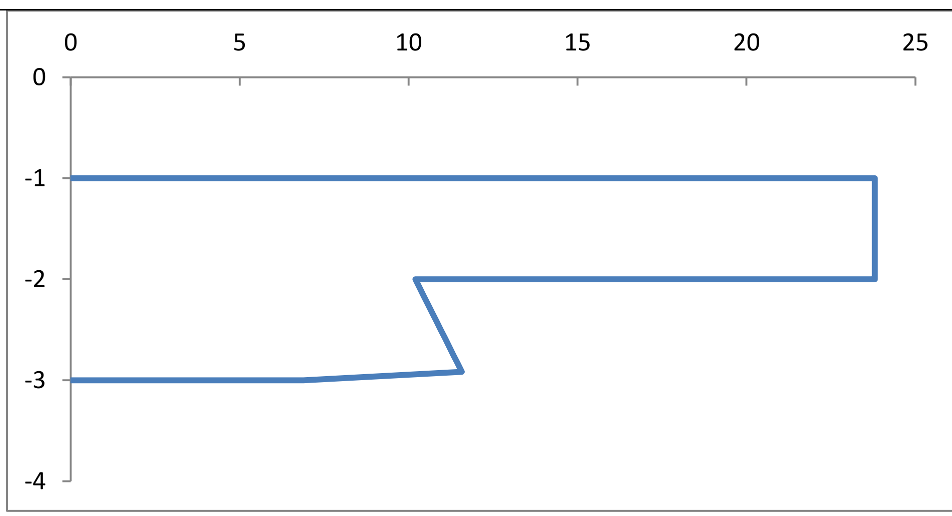


$Q_{tip} = \sigma_N * A_{tip} = 213*0.5 = 106.5$ kN/m	Qside $= \tau * A_{side} = 23.8*1 = 23.8$ kN/m

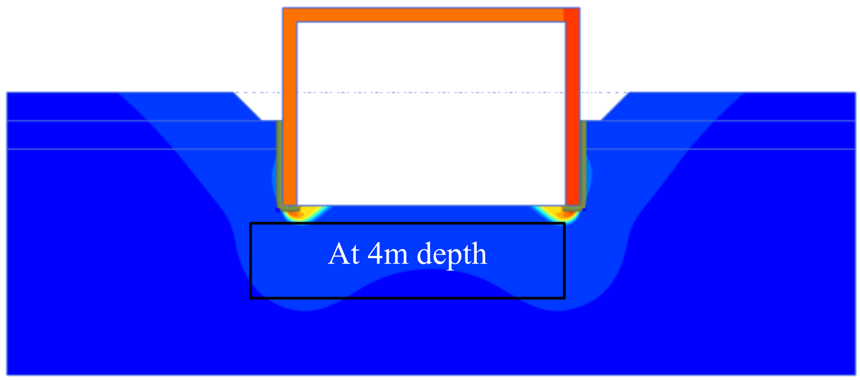


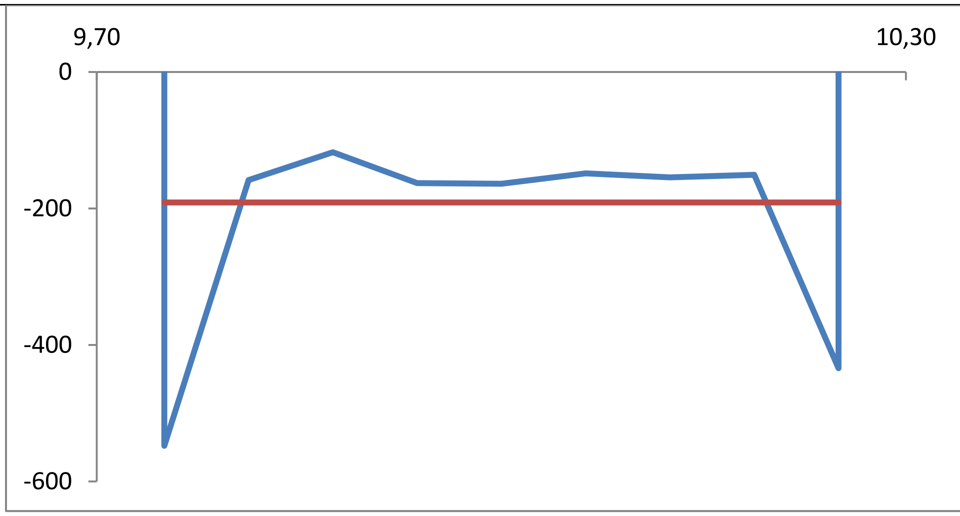


$Q_{tip} = \sigma_N * A_{tip} = 212*0.5 = 106$ kN/m

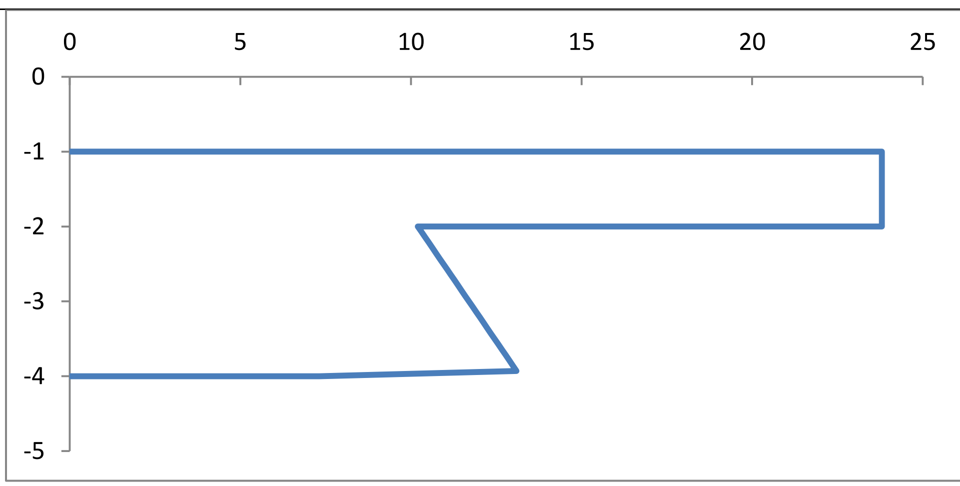


$Qside = \tau * A_{side} = 23.8*1+10.6*1 = 34.4$ kN/m

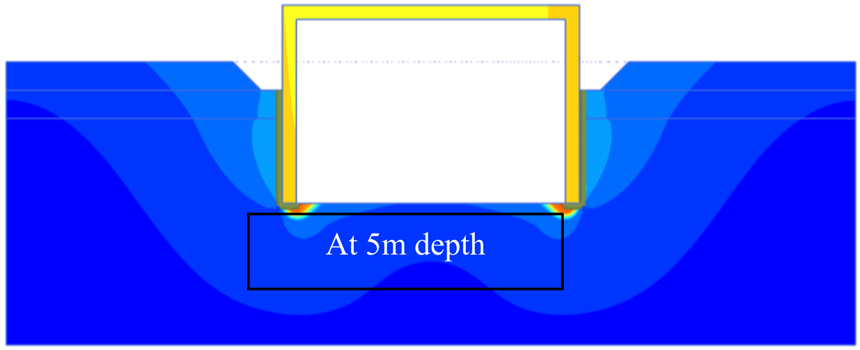


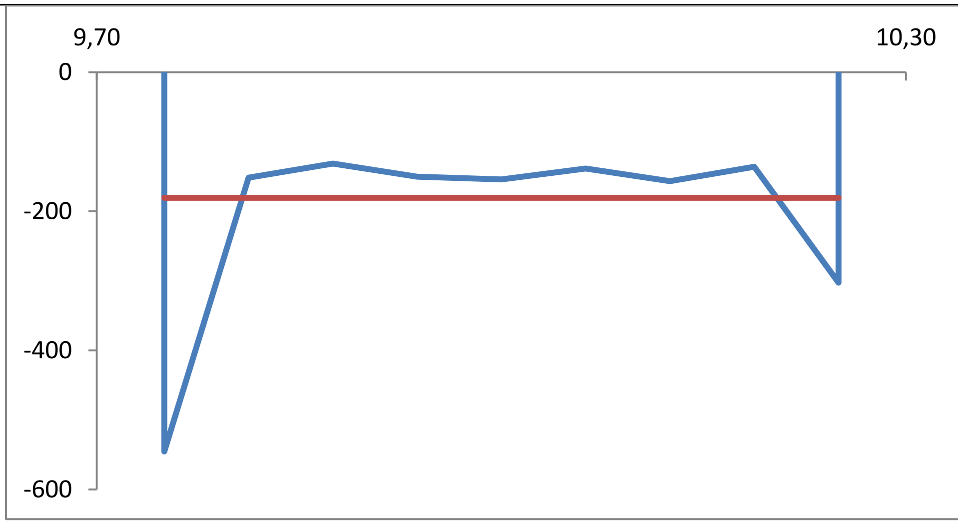


$$Q_{tip} = \sigma_N * A_{tip} = 191.4*0.5 = 95.7 \text{ kN/m}$$

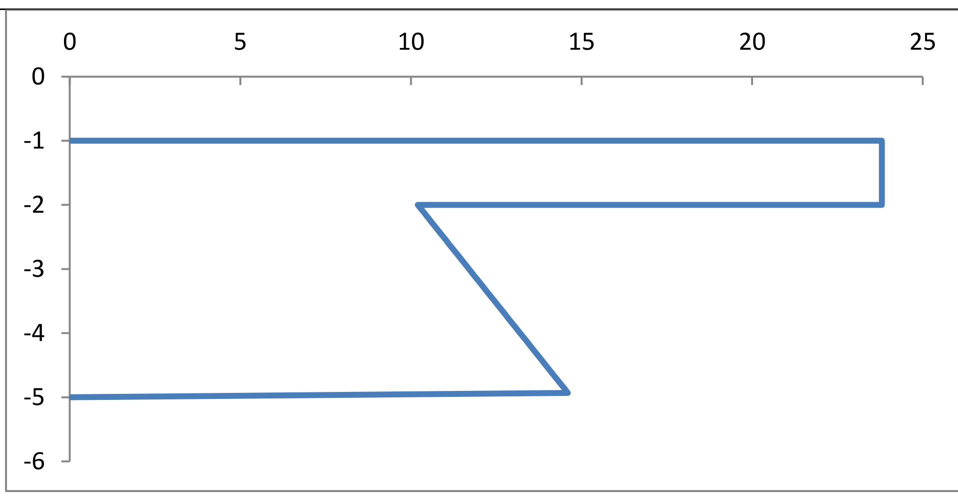


$$Qside = \tau * A_{side} = 23.8*1+11.6*2 = 47 \text{ kN/m}$$

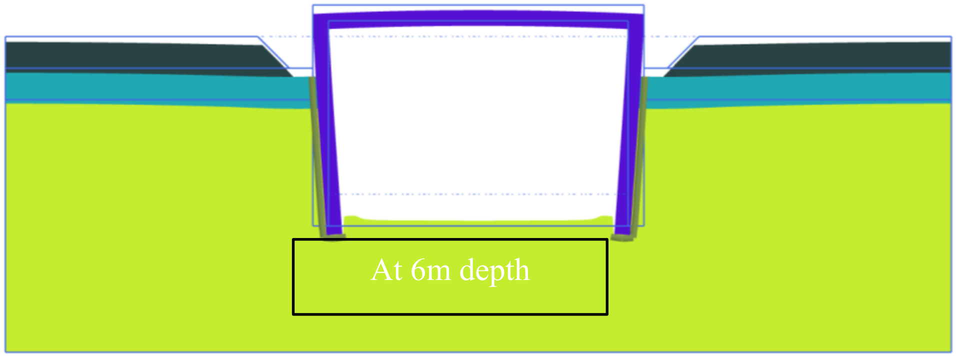


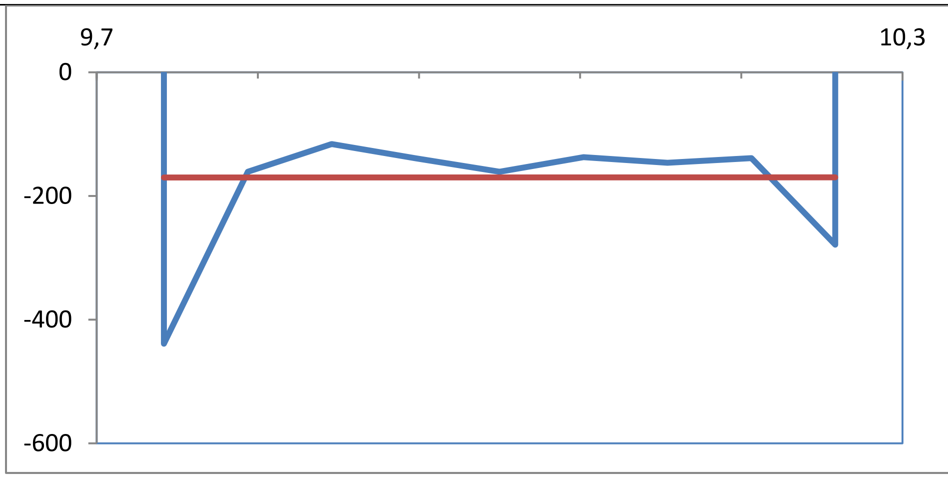


$Q_{tip} = \sigma_N * A_{tip} = 180*0.5 = 90$ kN/m

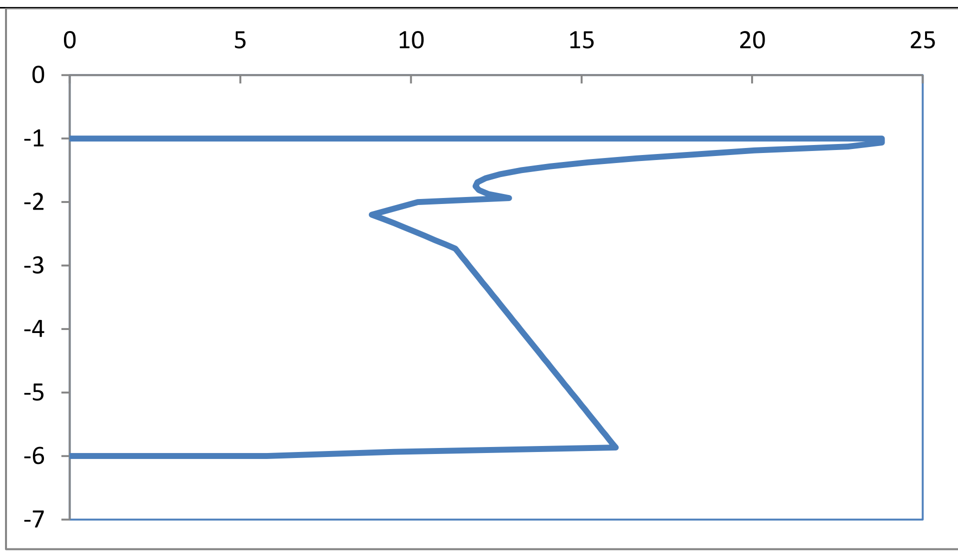


$Q_{side} = \tau * A_{side} = 23.8*1+12.4*3 = 61$ kN/m



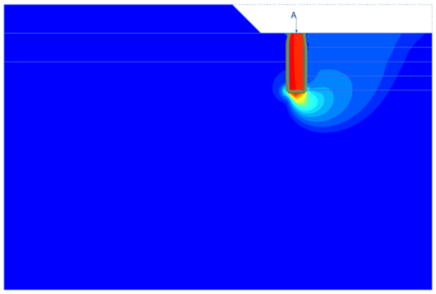
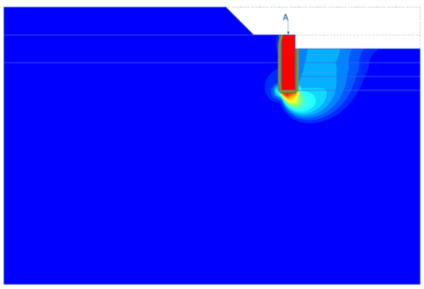
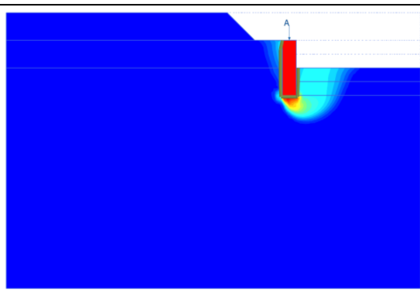
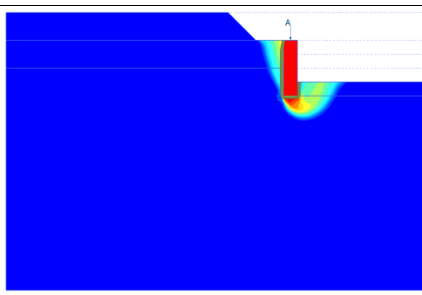
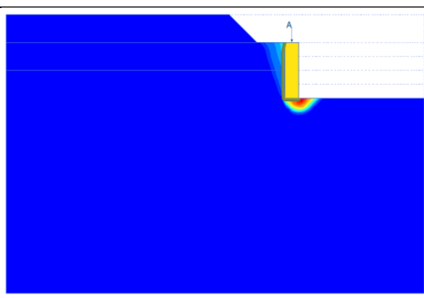


$Q_{tip} = \sigma_N * A_{tip} = 170*0.5 = 85$ kN/m



Qside $= \tau * A_{side} = 15.4*1+12.95*4 = 67$ kN/m

Appendix C: Failure mechanism for different excavation phases

Failure mechanism at 3m depth for different excavation phases	Failure Load [kN/m]	
	Flat tip caisson wall	Inclined tip caisson wall
	176 (FS=1.25)	156
	149 (FS=1.06)	129
	125 (FS<1)	102
	108	74
	78	31