CHALMERS

Alternative foundation techniques in urban environment

A study of European piling techniques and methods for retaining structures

Master of Science Thesis in the Master's Programme Geo and Water Engineering

THOMAS BORG
ERIK ULVÅS

Department of Civil and Environmental Engineering
Division of Geo Engineering

Geotechnical Engineering Research Group

CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2012

Master's Thesis 2012:91

MASTER'S THESIS 2012:91

A study of European piling techniques and methods for retaining structures

Master of Science Thesis in the Master's Programme Geo and Water Engineering

THOMAS BORG

ERIK ULVÅS

Department of Civil and Environmental Engineering
Division of Geo Engineering
Geotechnical Engineering Research Group
CHALMERS UNIVERSITY OF TECHNOLOGY

Göteborg, Sweden 2012

Alternative foundation techniques in urban environment

A study of European piling techniques and methods for retaining structures

Master of Science Thesis in the Master's Programme Geo and Water Engineering
THOMAS BORG
ERIK ULVÅS

Examensarbete / Institutionen för bygg- och miljöteknik,
Chalmers tekniska högskola 2012:91

Department of Civil and Environmental Engineering
Division of Geo Engineering
Geotechnical Engineering Research Group
Chalmers University of Technology
SE-412 96 Göteborg
Sweden
Telephone: + 46 (0)31-772 1000

Cover:
Construction of the *Götatunneln* in Gothenburg (NCC, 2012)

Chalmers Reproservice / Department of Civil and Environmental Engineering
Göteborg, Sweden 2012

Alternative foundation techniques in urban environment

A study of European piling techniques and methods for retaining structures
Master of Science Thesis in the Master's Programme Geo and Water Engineering

THOMAS BORG
ERIK ULVÅS
Department of Civil and Environmental Engineering
Division of Geo Engineering
Geotechnical Engineering Research Group
Chalmers University of Technology

ABSTRACT

Construction or maintenance of civil infrastructure in an urban area will always to some extent affect the urban environment with negative impacts. Conventional methods used for piling and retaining structures in Sweden are commonly associated with noise emitting activities, causing vibrations, soil displacement and settlements for surrounding buildings. Within an urban environment, these impacts may decrease the convenience and welfare of people present in that environment. Regulations regarding the allowable degree of these negative impacts vary between the European countries. The aim of this project is to present an inventory of foundation methods used in selected European countries and examine the degree of negative impacts caused by these methods.

To develop and implement innovative solutions for construction and maintenance of civil infrastructure in an urban environment, the European Commission initiated the research-project *Pantura*. The Swedish construction and property development company NCC AB is the project leader of a work-package within *Pantura* covering strategies for construction with a zero negative impact. In line with these strategies, this project has resulted in a linear additive MCA (*Multi-criteria analysis*) model for the foundation methods found during the inventory.

The countries included in this study were Sweden, Norway, Denmark, Germany, Poland, the Netherlands, U.K. and Spain. From the inventory, it was found that the foundation market in the Scandinavian countries was dominated by pre-fabricated, soil displacing foundation elements, installed by driving methods that causes impulse noises. The piling market are in the Scandinavian countries dominated by pre-fabricated concrete piles and the market for retaining structures are dominated by steel sheet piling. Foundation methods involving cast-in-place, such as CFA-piles and diaphragm walls, are more frequently used in Germany, the Netherlands, U.K. and Spain. It was also found that the regulations regarding the allowable degree of negative impacts on the urban environment are stricter in those countries where cast-in-place methods dominate the foundation market.

From the linear additive MCA model performed in this study, it was shown that the most preferable piling technique in an urban environment is bored steel tube piles for a heavy structure nearby a sensitive structure, and jacked precast concrete piles for a light structure at a distance from a sensitive structure. The most preferable method of retaining structure is diaphragm wall if the retaining structure will be constructed close to a sensitive structure and king post/berliner wall if it is constructed at a distance from the sensitive structure.

Key words: Negative impacts on the urban environment, foundation methods, piling, retaining structures, PANTURA

Contents

Preface

In this study, an inventory of foundation methods used in selected European countries has been performed. The study was conducted from January 2012 to June 2012. This master thesis is a part of a European research project concerning development and implementation of innovative solutions for construction and maintenance of civil infrastructure in an urban environment. This study was carried out at the Department of Civil and Environmental Engineering, Chalmers University of Technology, Sweden, and at NCC AB.

This master thesis was performed with Mats H. Olsson from NCC AB as supervisor and Claes Alén from Chalmers University of Technology as examiner. The hospitality showed by NCC Teknik and Hercules Grundläggning has been absolutely crucial for our work during the spring of 2012. We would especially like to thank Jonas Magnusson and Christina Claesson-Jonsson at NCC Teknik for guidance during the introduction of our work, Tara Wood at NCC Teknik for guiding us through foundation methods used in U.K., Mats Olsson, Peter Alheid, Johan Blumfalk and Maria Kristensson at Hercules Grundläggning for guidance when certain foundation methods was not fully understood and Mats H. Olsson at NCC Teknik for giving us the motivation to finish the report in time. We also want to thank Anders Salomonson and Ingrid Gårlin at SGI library in Linköping for helping us with the literature survey.

Finally, we would like to wish all the other students that wrote their master thesis at NCC Teknik during the spring of 2012 good luck in the future!

Göteborg, June 2012

Thomas Borg

Erik Ulvås

1. INTRODUCTION

1.1 Background

As urbanization and economic growth strives for increasing the efficiency in our European cities, the demand on the civil infrastructure grows constantly. Studies show that transports of both freights and people are increasing in Swedish cities, as well as for the majority of the European cities, due to economic growth (Vinnova, 2012). A vital part of the civil infrastructure network in many European cities is bridges, which are often located in densely populated urban areas. According to Rizal Sebastian (2011), Senior Research Scientist at TNO, the Netherlands organization for applied scientific research, about 30 % of all steel and composite bridges in Europe are more than 100 years old and more than 70 % are older than 50 years. Together with the previously mentioned increased demands, the need for upgrades of the European civil infrastructure network in general, and the urban bridges in specific, will constantly increase in the future.

However, constructions within an urban environment are often associated with negative impacts for those living and working in that environment, disturbing the daily urban life and affecting the convenience of those present in the urban environment. A construction site may reduce the mobility of pedestrians in the urban environment, traffic diversions may cause traffic delays and the activities at the construction site may cause high levels of noise and vibrations. Conventional foundation methods in Sweden, such as drop-hammer piling, are a common source of inconvenience regarding noise and vibrations in an urban environment. Infrastructure project are in general being performed to increase the ability for the region to increase its welfare, but as these infrastructure projects often are a long-termed activity, the negative impacts will affect the local welfare in a negative way.

To reduce these negative effects and to promote a more efficient resource usage when constructing infrastructure in an urban environment the research-project *Pantura* has been created, with co-financing by the European Commission. *Pantura* aims at develop and implement innovative solutions for design and coordination when maintaining and constructing bridges.

Since the start of the *Pantura*-project in 2010, NCC AB has been project leader for work-package 2 (*Flexible Construction Processes*), where the ambition is to create strategies for construction with a zero negative impact, construction techniques for new and existing bridges and sustainable construction regarding energy usage, environment and socio-economic issues.

In line with the strategy for construction with zero negative impact, NCC AB now intends to examine how different foundation techniques for infrastructure projects affect the urban environment.

1.2 Aim

This study will present different techniques used for piling and retaining structures in selected countries in Europe, considering disturbances to the urban environment regarding noise, vibrations, soil displacement and settlements.

1.3 Method

To obtain information regarding various foundation methods and how the urban environment is affected by the direct negative impacts, a literature study was performed. Interviews and questionnaires were performed to obtain primary data regarding the extent of negative impacts for each category of foundation method. The geological and geotechnical description of each ingoing country was performed via a literature study and GIS-simulations of geological databases. The inventory of used foundation methods in selected countries in Europe was conducted through a literature study. The foundation methods found during the inventory was then analyzed using a linear additive MCA (*Multi-criteria analysis*)-model with the negative impacts and relative costs as criterions.

1.4 Delimitations

The selected countries that were examined in this study were;

- Sweden
- Norway
- Denmark
- Germany
- Poland
- the Netherlands
- United Kingdom
- Spain

All of these countries are included in the *Pantura* project except Germany. Due to lack of correspondence, Italy were excluded, which is a country included in the *Pantura* project. To more thoroughly study specific foundation methods, 1 – 4 construction projects including either a retaining structure and/or a piling method in an urban area were selected in 15 cities within the countries listed above. The negative impacts studied were limited to noise, vibrations, soil displacement (piling) and settlements (retaining structures).

1.5 Orientation

Chapter 2 – *Negative impacts on the urban environment*, will present the direct negative impacts caused by foundation works in urban areas, which later will be used to compare the various foundation methods.

At chapter 3 – *Foundation methods*, various types of foundation methods, both piles and retaining structures, is presented. The degree of negative impact from each type of foundation method is presented for each foundation method.

The purpose of chapter 4 – *Theorem of welfare*, is to present a broader perspective of costs related to infrastructure projects. Conclusions drawn from this chapter is primary used as basis of discussion.

The result from the study is presented in chapter 5 – *Geological and geotechnical conditions*, and chapter 6 – *Inventory of used foundation methods*.

Foundations methods presented in chapter 3– *Foundation methods*, and also found in chapter 6 - *Inventory of used foundation methods*, are analyzed regarding negative impact on the urban environment and its relative costs using a linear additive MCA (*Multi-criteria analysis*) -model in chapter 7 – *Analysis*.

The report is ended with chapter 8 – *Discussion*, and chapter 9 – *Conclusions*, where the report is summarized and the outcome of the MCA is discussed.

2. NEGATIVE IMPACTS ON THE URBAN ENVIRONMENT

Construction of infrastructure in urban areas is commonly associated with negative impacts for the people present in that environment. The object being built, e.g. a bridge or a tunnel, might in the long run be profitable for the city and its inhabitants, but will during its construction cause various problem. Physical barriers such as construction sites might intrude on the accessibility in the city, leading to traffic delays as well as hinder the mobility for pedestrians. Power-full machines and equipment for the construction might increase the amount of pollution in the city, resulting in decreased air quality. Also, due to heavy lifts and the vast number of heavy transportations performed during a construction, the risk for accidents is increased both for the construction workers and for the people present in the urban environment. However, the more direct negative impacts caused by foundation works that will be examined in this thesis are noise, vibrations, soil displacement and settlements for adjacent buildings. The following chapter will present these four negative impacts caused by piling and/or constructing a retaining structure.

2.1 Noise

Noise from piling and constructing retaining structures may disturb the environment during construction works in urban areas. The disturbances of the noise depend on the time during the day and may cause a large impact on the living, work and business environment. Moreover the vibrations enlarge the disturbance effect. The vibrations which have frequencies within the audible spectra causes disturbances in form of structure borne sound. There are special issues about noise from piling and other ground work. (Hintze et al, 1997).

- Construction work is performed in the “free air”
- The duration of the construction work is limited
- Different equipment have very different noise properties and these properties may vary during construction work
- It is not possible to move the construction work to areas which is not sensible to noise
- Resonance problems may occur

Noise from piling and ground work commonly contains impulse sound. The impulse frequency varies between the piling methods used. This kind of noise disturbs more than an even noise e. g. traffic noise due to the impulse sound, see Figure 1. Noise from piling and ground work is also a problem for personnel who are working at construction sites (Hintze et al, 1997).

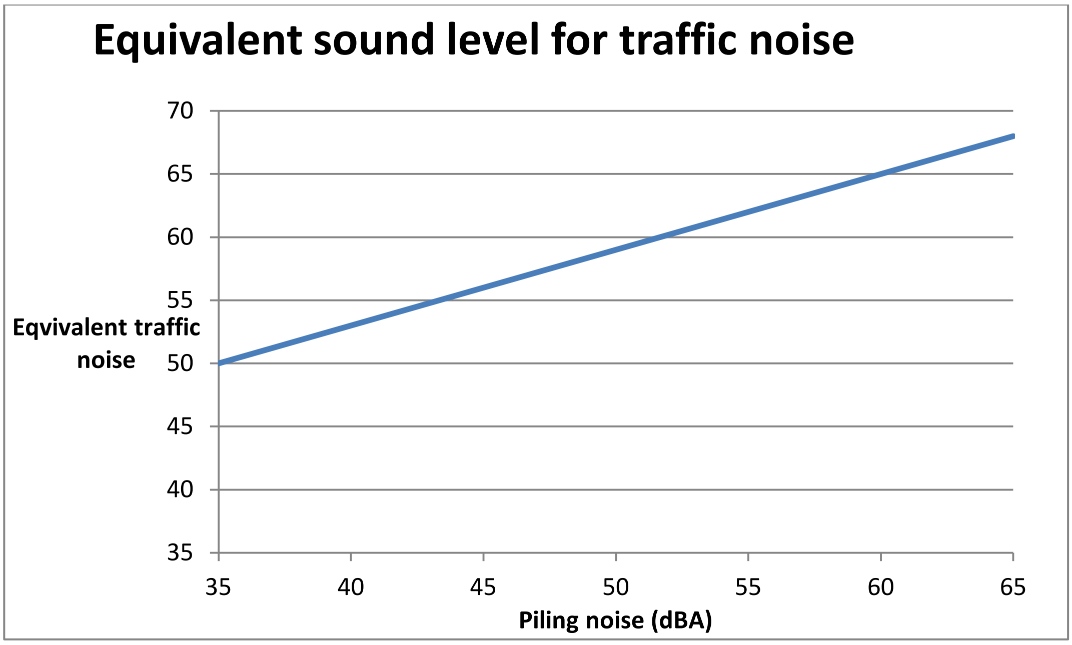


Figure 1 The equivalent traffic noise generated from piling noise. (Hintze, 1997)

Guidelines regarding equivalent noise levels (L_{Aeq}) has in Sweden been set by Naturvårdsverket (Swedish Environmental Protection Agency), and is included in miljöbalken (Swedish Environmental Code). The Swedish guidelines for equivalent noise levels consider type of area and at what time and day it is. *Day* is referred to between 07:00 – 19:00, *Evening* between 19:00 – 22:00 and *Night* between 22:00 – 07:00. In table 1, the Swedish guidelines for equivalent noise levels (L_{Aeq}) and highest allowed noise level at night for residential area (L_{AFmax}) is presented. The figures in Table 1 is gathered from 2 chapter 3 § and 26 chapter 19 § in miljöbalken.

Table 1 Guidelines for equivalent noise levels and highest allowed noise level

Type of area	Weekday (Monday - Friday)		Saturday, Sunday and public holiday		All days	
	Day	Evening	Day	Evening	Nigth	Night
	L_{Aeq}	L_{Aeq}	L_{Aeq}	L_{Aeq}	L_{Aeq}	L_{AFmax}
Residential						
outdoors	60 dBA	50 dBA	50 dBA	45 dBA	45 dBA	70 dBA
indoors	45 dBA	35 dBA	35 dBA	30 dBA	30 dBA	45 dBA
Health care						
outdoors	60 dBA	50 dBA	50 dBA	45 dBA	45 dBA	-
indoors	45 dBA	35 dBA	35 dBA	30 dBA	30 dBA	45 dBA
Education						
outdoors	60 dBA	-	-	-	-	-
indoors	40 dBA	-	-	-	-	-
Office						
outdoors	70 dBA	-	-	-	-	-
indoors	45 dBA	-	-	-	-	-

The values presented in Table 1 are not seen as limit-values, rather than guidelines. In those cases the noise-emitting activity is limited in time (shorter than two months), an extra 5 dBA may be added to the values in table 1. When the noise-emitting activity is of short-term nature (less than five minutes per hour), an extra 10 dBA might be added to the values in table 1, except for the values expressing the guideline for evenings and nights. If the outdoor noise levels is technically impossible or economically unreasonably to achieve, the ambition should be to at least fulfil the guideline-values for indoor noise levels. (Naturvårdsverket, 2004)

Similar guidelines regarding noise levels from construction-activities have been found for Denmark, U.K., Germany and Norway. For the guidelines in Denmark it is stated that residents in close adjacent to the construction site has to have the opportunity to sleep completely undisturbed for at least 8 hours (Hintze et al, 1997). The Danish guidelines consider only the outdoor noise level, and are over all categories stricter than the Swedish guidelines. In U.K., the contractor has to apply for consents from local authorities before starting a construction involving noise-emitting activities. In this application, the contractor has to describe which actions they will make to minimize the noise levels. Similar to the Danish guidelines, the guidelines in Germany only considers outdoor noise levels. Compared to the Swedish guidelines, the German guidelines are over all categories stricter than the Swedish. Also, according to Vägverket publikation 1997:130, Germany is the only country of these that quantifies noise in monetary terms. The guideline-values for Norway also only implies for the outdoor noise level, and are stricter than the Swedish guidelines regarding noise level adjacent to health care facilities (Hintze et al, 1997). For the remaining categories, the Norwegian guideline-values are less strict than the Swedish guidelines.

2.2 Vibrations

The energy propagates in the soil as elastic waves where three main types of waves exist. The fastest wave is the pressure wave (P-wave) where the particles are moving in a forward and returning action along the propagation of wave. The shear wave (S-wave) is the slower wave where the particles are moving perpendicular to the propagation of the wave. The third form of wave is the Rayleigh wave (R-wave) where the particles are moving in an elliptical orbit and the amplitude are decreasing with the depth. In stratified ground, another type of surface waves also exists: Love-waves (L-waves). The P- wave propagates in the soil skeleton and in the pore water; the S-wave however, propagates only in the soil skeleton since the water cannot transfer shear stresses. The R-waves propagates both in the soil skeleton and in the pore water since the R-wave consists of special type of reflected S- and P- waves. Within a wave length of the vibration source, the S- and P- wave dominates. Outside this zone, the surface waves dominate. Resonance in soil is affected by the relation between wave length and the thickness of the soil where the wave propagates. The wave length of compression and surface waves are between 5 and 15 m which often correlates with the thickness of the soil layer placed on solid bottom. The ground water level also influences the wave propagation. (Hintze et al, 1997)

During piling, vibrations will be produced. These vibrations alone will normally not damage structures but during unfavourable conditions, these vibrations together with other factors (stresses, differential settlements) may create cracks and other damages. A lot of factors affect piling vibrations. Generally said these factors are:

- Energy transfer hammer-pile
- Energy transfer pile-soil
- Wave propagation in soil
- Interaction structure-subgrade
- Vibration amplification in structure

Problems with vibrations can be divided into two different parts; active and passive vibration problems. The active vibration problems deal with the circumstances at the vibrations source and in the vicinity of it i.e. the interaction between the pile and soil. The passive vibration problems deal with the wave propagation in soil and the impact of the wave energy on structures in soil and buildings on soil. (Hintze et al, 1997)

2.3 Soil displacements

In piling operations soil displacements may not be allowed or the displacement has to be reduced. In the following chapter, the soil displacements in clay, silt and friction material will be described.

2.3.1 Soil displacement in clay and silt

When displacing piles are used in clay, it is generally said that the displaced soil volume is equal to the volume of piles reduced with the soil volume which is removed by “plug removal”. The ambient soil is affected by the soil displacement induced by the pile driving in different ways. A consequence of this is ground heave, horizontal displacements and increased horizontal stress. These consequences are determined by soil type, stress situation and geometrical properties within and outside the foundation area. Based on studies in low sensitive clay by Dugan and Freed, the following factors affect the ground heave: (Hintze et al, 1997)

- The ground heave is proportional to the volume of displaced soil.
- The area of ground heave is estimated by a line with 45 degrees inclination from the pile top.
- “Plug removal” reduces the effect of ground heave. The hole can be stabilized by bentonite but the diameter of the hole must be large enough so the slurry can escape during pile driving, to reduce the risk of slurry penetrating into the soil.
- Ground heave of adjacent ground are inversed proportional to the in situ stress.
- The ground heave is dependent of the direction of piling. It will become greater in the direction of piling.
- In low sensitive clays greater ground heave will occur. However, in sensitive clays the degraded particle structure and to some extent consolidation will reduce the ground heave.
- If excavation is performed, the ground heave will be reduced outside the piling area but increased within the area.
- During piling in homogenous clay, the heave will be greater compared to if the piles will be installed in clays composed of clay with intrusions of coarser material.

Piling in quick clay crates completely stirred and liquid acting clay that comes up to the surface between the mantle of the pile and adjacent soil. This crates small amount of heave and displacements. To reduce the ground heave and horizontal displacements “plug removal” is performed. It is important that the “plug removal” depth is not larger than the critical depth of the hole. Otherwise the hole will collapse. Piles that are located close to the piling operation can be heaved due to the global ground heave and it can correspond to the geotechnical capacity of the pile. Especially problems with spliced piles can occur when a pile is heaved.(Hintze et al, 1997).

2.3.2 Soil displacement in frictional material

For clay and silt it is assumed that the displaced soil volume is equal to the volume of piles. This more correct for cohesive soils than frictional soil, since frictional soil will change in volume when it is displaced. In firm friction material an increase of volume will occur and in loose fiction material a decrease in volume will be obtained. The volume increase is difficult to calculate but a rule of thumb is that it corresponds 1.0 to 1.5 the piling/sheet piling volume if the soil are mean firm to very firm friction soil. The following factors create horizontal displacements and ground heave in frictional soil. (Hintze et al, 1997).

- Soil type
- Volume of piles/sheet piles
- Ground water level
- Relative compaction density
- Type of structure
- Mass of structure
- Foundation type
- Slope of ground

2.4 Settlements outside the excavation

Settlements outside the excavation may cause damages on the surrounding structures. As described below these settlements may be caused of groundwater lowering and/or deformation of the retaining structure.

2.4.1 Lowering of groundwater table and settlement in soil

A consequence of lowering of the groundwater table is increased effective stress in the soil due to lower pore water pressure. This will cause settlements in the soil profile. In a granular soil profile, the effective stress is transmitted between the particles by contacts between the mineral grains. Since the surfaces of the large granular particles are rough the force from the difference between total stress and pore pressure is great per particle. The contact area is very small which makes the contact stress high and the water is fast drained out. In fine grained profile, containing clay minerals, the water is not fast drained out. In this kind of soil the force from the difference between total stress and pore pressure is small and the contact area between the particles is large. The change in effective stress will cause a change in void volume, an increase of effective stress result in a compression and a decrease of effective stress in swelling. Equilibrium of the forces acting between the particles is not obtained before an effective stress change stops. The readjustments and redistributions last during long time and the results in further change of void volume. This change is known as secondary compression if the effective stress is increased and secondary swelling if the effective stress decreases. (Terzaghi et al, 1996)

If groundwater observation pipe is placed below the groundwater table the water level will rise to the same level as outside the pipe. If the soil around the pipe is loaded the void ratio will decrease and the water will rise in the observation pipe. The water rise is a consequence of the pore water pressure increase caused by the void volume reduction. This creates excess free water that will drain depending on the permeability of the soil. As the water drains the level in the observation pipe will decrease to the same level as in the soil around. (Bowels, 1996).

In areas where artesian groundwater is present, drainage of the aquifer can be caused through the confining layer along the mantle of the piles. Due to the increased pore pressure the effective stress is reduced and also the shear strength, therefore the stability is reduced. (Hintze et al, 1997).

2.4.2 Deformation in retaining wall

To reduce the horizontal deformations in the retaining wall, the wall is supported by a strut which is connected to a slab or the bedrock. Moreover the wall can be supported by props between the retaining structures if a trench is excavated or the ground in front of the retaining structure if one retaining structure is constructed. This support creates a point load on the wall which reduces the horizontal displacements (Sällfors, 2001). Since the retaining wall deflects, the ground behind the wall will settle and possible damaging deformations will be obtained. To reduce this deformations stiff retaining walls and stiff struts can be used. (Kullingsjö, 2011)

3. FOUNDATION METHODS

Various foundation methods will cause various negative impacts on the urban environment to various extents. Therefore, it is of greatest importance to choose a suitable foundation method that has the capacity required, has an acceptable degree of negative impacts on the urban environment and can be performed within an economical reasonable budget. In the following chapter, a presentation of different piles, piling methods and retaining structures will be presented. Pile types are divided into the degree of soil displacement. At chapter 3.2, *Retaining structures*, the degree of the negative impact *Settlements* is explained for each specific method. The degree of the negative impacts *Noise* and *Vibrations* will be presented for the piling methods and methods of retaining structures in subchapter 3.3.

3.1 Piling

There are many ways to categorize piling methods. The categorization can be made based on the two basic methods of piling installation; driving them into the ground, or filling an excavated void with concrete (Fleming, K. et. al. 2009). However, using that criterion for categorization might cause confusion as new piling methods might be a combination of these two installation method. Another common way to categorize piles is by the three major pile materials; concrete, steel and timber. As combinations of concrete and steel often occur, e.g. reinforced concrete, this categorization is regarded as ineffective for this report. Soil displacement is an important factor for piling in an urban environment. Therefore, the categorization that includes the degree of soil displacement will be used in this report. The categories will therefore be as following; large-displacement piles, low-displacement piles and non-displacement piles, Figure 2 provides an overview of the categorization and main categories of piles that will be presented in this chapter. As the installation of pre-formed piles is the major source of noise and vibration, methods for pile driving will be presented in sub-chapter 3.1.4.

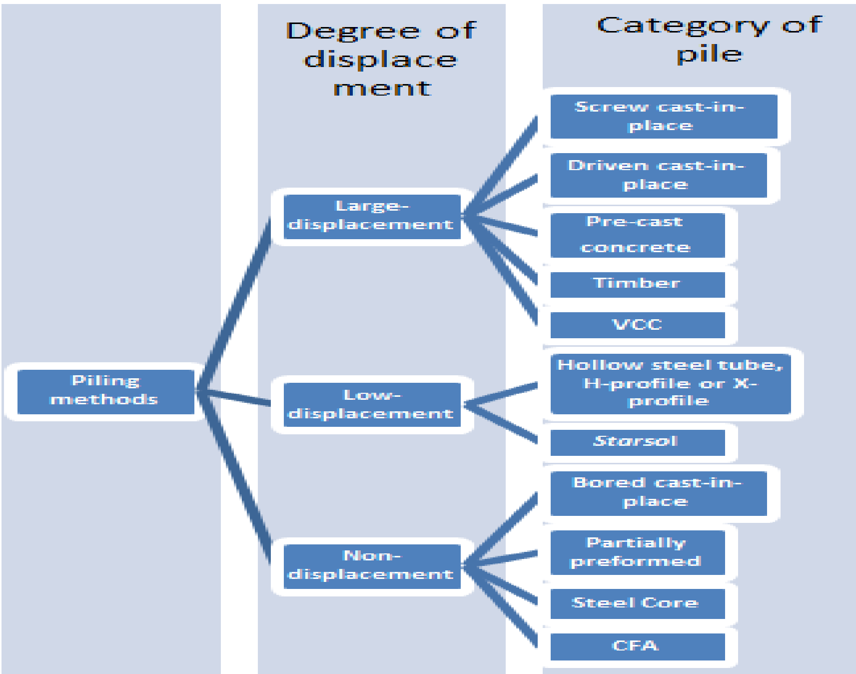


Figure 2 Overview of the classification used in this report

3.1.1 Displacement piles

For displacement piles, the soil is mainly displaced radially as the pile or auger[1] head penetrates the ground (Fleming, K. et. al. 2009). A vertical soil displacement may also occur in some extent. The negative effect on the surrounding urban environment due to soil displacement is mostly depending on the soil condition as granular soils get compacted and clay soils may heave, causing possible negative impact for surrounding urban environment.

Screw cast-in-place displacement piles

Installation of these types of piles involves screwing an auger head with a short flight into the ground to a required depth, resulting in a compaction and displacement of the surrounding soil (Flemming, K. et. al. 2009). A hollow stem transmits the required torque and compressive force to the auger head, which penetrates the soil without any extraction of soil material at the ground surface. The compressive force, or "crowd" pressure, together with the torque can be monitored for each pile, making it possible to analyze each specific pile bearing capacity (GeoForum, 2012). For reinforcement, a steel cage is inserted into the hollow stem after the completion of the installation process. Plastic concrete is then inserted from a funnel-shaped hopper to the auger head, which during unscrewing and withdrawing toward the ground surface forms a cast-in-place pile. A schematic drawing of the installation process for a screw cast-in-place displacement pile is shown in Figure 3. According to GeoForum, this particular type of pile installation is suitable for deep foundations in urban areas as the execution is performed without vibration and excessive noise. The allowable design load, P_a, for all un-prestressed concrete piles depend on the area of the concrete (A_c) and possible steel shell (A_s) multiplied with the respective allowable material stresses (f_c and f_s respectively) as seen in Equation 1 (Bowles, J.E., 1996);

$$P_a = (A_c \times f_c) + (A_s \times f_s) \quad (1)$$

[1] Helix-shaped tool for boring holes

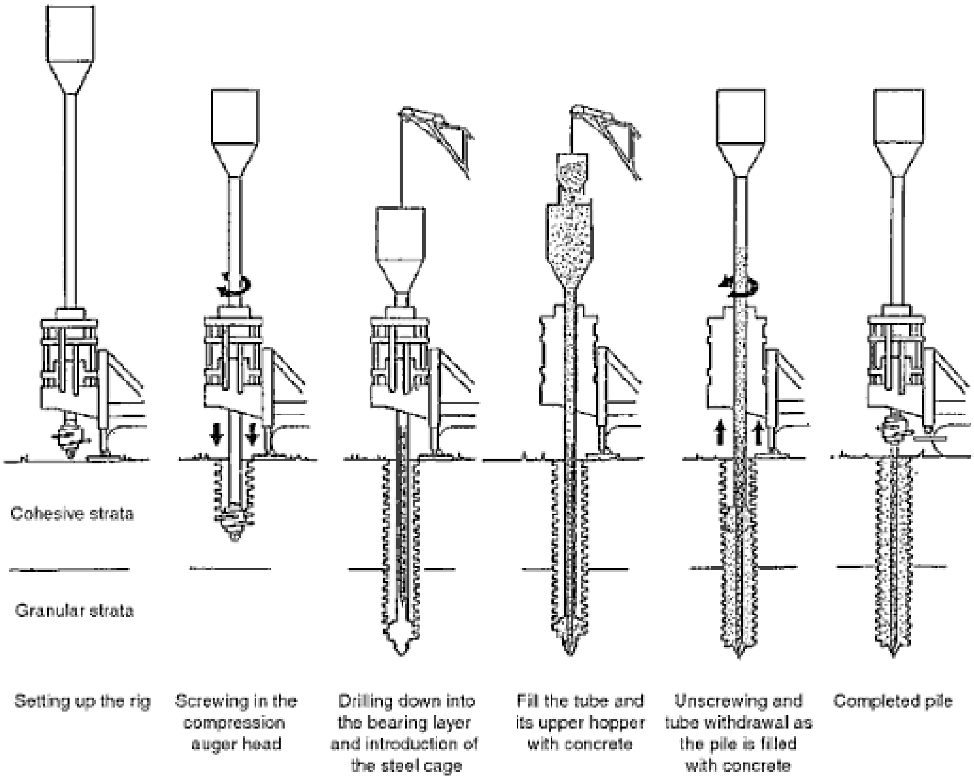


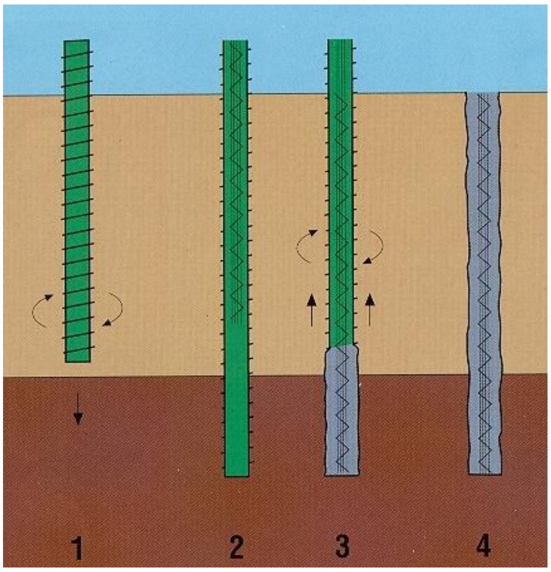
Figure 3 Schematic drawing of the installation process of the Atlas piling system (Flemming, K. et. al. 2009)

There are some variations of the screw cast-in-place displacement piles. If the unscrewing of the screw auger head is performed using a reverse direction of rotation compared to when screwing the screw auger head into the ground, a helix formed pile is formed (GeoForum, 2012). The helix formed cast-in-place displacement pile, known as the *Atlas Piling System*, has a higher pile bearing capacity than traditionally constructed bored piles for a given pile size and concrete volume due to its jagged outer jacker with increased skin friction (Flemming, K. et. al. 2009). In the third edition of *Piling Engineering*, Flemming et. al. also state that the *Atlas Pile System* "combine many of the advantages of a displacement pile with the low noise and vibration characteristics of a bored pile". A pile diameter of 360 – 560 mm is obtained with the *Atlas Pile System* and can reach a maximum depth of 22 m. The *Atlas Pile System* is appropriate for most cohesive and granular soil conditions.

If the unscrewing of the screw auger head is performed using the same direction of rotation as when screwing the screw auger head into the ground, a straight-shafted pile is formed (GeoForum, 2012). This method is named the *Omega Pile*. Small amount of spoil will be produced using this method, which has to be taken care of at the ground surface. To increase the skin friction of a straight-shafted screw cast-in-place displacement pile, an extended fin on the screw auger head will create a helically threaded pile, named the *Screwsol Pile* (Flemming, K. et. al. 2009). To protect the concrete shaft in aggressive ground condition, a steel casing can be installed during the placement of the concrete. If this steel casing is left in place, a *Fundex Pile* is formed (Flemming, K. et. al. 2009). To increase the bearing capacity, a cement-based grout can be injected through the auger head into the surrounding soil as the casing is installed (GeoFroum, 2012). This sort of injected piles, called *Tubex* *grout-injected piles*, will have a greater corrosion protection as the grout shell will protect the pile shaft.

A further development of the screw cast-in-place displacement pile is the *continuous helical displacement* (CHD), where a reinforcement cage is inserted into the fresh concrete as the screw auger has finished constructing a helical-shaped cast-in-place concrete pile (Flemming, K. et. al. 2009). The CHD method is according to Flemming, K. et. al. 2009 a cost-effective method at contaminated urban sites as no spoil is produced and noise and vibration is minimized.

Two screw cast-in-place displacement piles have been developed by the German company Jebens GmbH; *SVB-pile* and *SVV-pile*. The *SVB-pile* (Schnecken-Verdrängungsbohrpfahl) is installed by rotating and pushing a helix-shaped steel casing into the ground, transporting some soil to the surface and laterally displacing some soil. Before raising the casing and pump cement into the formed void in the soil, reinforcement is installed. During extraction the casing is rotated in the same direction as during installation, forming a straight-shafted pile. The bottom plate which seals the casing during installation is left in the ground when extracting the casing. Another method, named the *Presso-drill pile*, follows the same procedure as the *SVB*-pile. Installation of the *SVV-pile* (STRABAG Vollverdrängungsbohrpfahl) also follows the same procedure as the *SVB-pile*, except this method uses another type of casing with a drill head where no spoil is produced. Both the *SVB-pile* and the *SVV-pile* can take a load up to 1 500 kN, with a pile diameter of 0.4 – 0.67 m and a maximum length of 24 m. The sequence of *SVB-pile* and *SVV-pile* construction is shown in Figure 4. (GeoForum, 2012).



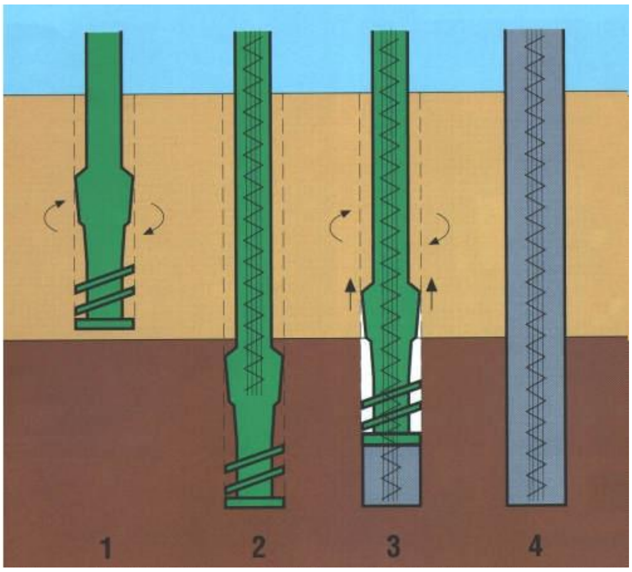


Figure 4 Installation of SVB-pile to the left and installation of SVV-pile to the right (GeoForum, 2012)

Driven cast-in-place displacement piles

This type of piles is installed using either a temporary or permanent tube to form a void in the soil through driving, which is then filled with concrete and eventually reinforcement. The piling technique which uses a permanent tube can be used to support considerable loads in suitable ground conditions. However, even though the tube surrounding the concrete is then left in place, the load-supporting medium is generally the infill, i.e. the concrete and eventually reinforcement. As the tube is not load-supporting, driving methods have been progressed to not over-stress the non-load-bearing forming tube. To minimize the thickness of the forming tube, internal hammers may be used for bottom driving for reducing driving stresses. A schematic drawing of the installation process for a driven cast-in-place displacement pile is shown in Figure 5. (Flemming, K. et. al. 2009)

Figure 5 Schematic drawing of the installation process of a bottom-driven Franki pile (GeoForum, 2012)

The *Franki pile*, which is shown in Figure 5, is a high-capacity pile which can be performed in a variety of methods. However, the common factor for *Franki piles* is that pile has an enlarged base and cylindrical shaft (GeoForum, 2012). The *Franki pile* is executed by compacting a plug of non-fluid concrete mix or gravel at the base of a steel forming tube by an internal hammer with a weight of 2 – 8 tones, making the plug and tube penetrate the soil. The plug becomes relatively water-tight as it gets compacting, preventing groundwater and soil to enter the tube. When the plug and tube has reached the required depth, the tube is raised slightly and maintained in position with the aid of steel cables. The internal hammer continues to compact the plug, which without the support of the tube is expelled radially to form an enlarged pile base. A certain amount of the plug-material should be remained in the tube to prevent any leakage of groundwater or soil material into the pile shaft. If a larger bulb is considered to be favorable, more plug-material can be added during the last stage of compaction. The reinforcement can be installed either before or after the forming of the enlarged base, but before the concreting. If the pile is designed to only withstand compressive forces, the reinforcement may be limited only to the upper section of the shaft (Flemming, K. et. al. 2009). The hammer and tube is retracted simultaneously as the fluid concrete is pumped into the formed void. Pile-dimensions of a typical *Franki Pile* ranges from 0.3 m to 0.7 m, can reach a maximal length of 30 m and has a design load from 35 to 200 tons (GeoForum, 2012). To cope dynamic loads with a horizontal component, *Franki Piles* may be installed raked with a tilt of about 25°. The use of *Franki piles* has declined during the last few decades due the major imposed vibrations caused by the internal hammer, and by its relatively high cost. However, due to its high load-bearing capacity, the *Franki Pile* has an advantage where site conditions are suitable. The method of performing a *Franki pile* can be modified for varying soil conditions, as permanent steel casings when negative friction is encountered, or using pre-cast elements (*Franki composite pile*) when it’s difficult to cast-in-place.

To minimize the negative environmental impacts caused by the *Franki Pile,* a further development of the system aims at combining the advantages of a bored pile with the advantages of a pile with enlarged base. The *Franki VB pile* is installed by pressing and rotating a steel casing into the ground, rather than driving it using a hammer as for ordinary *Franki piles* (GeoForum, 2012). After installing the pile to required depth, a driven base is constructed as described above. The installation process thereafter follows the same sequence as for ordinary *Franki piles*. According to GeoForum (2012), the *Franki VB pile* is vibration and noise free, well suited for use in urban areas.

Pre-cast concrete piles

This is the most commonly used type of pile in many parts of the world, especially in Scandinavia where this pile type is representing about 75 – 80 % of the total piling length installed in Sweden (Olsson, C., Holm, G., 1993). The pile type is usually manufactured with square cross-sectional area ranging from 200×200 mm (400 kN) to 350×350 mm (1200 kN) with reinforcement installed. If the pile is designed for loads up to 3000 kN, a cross-section area up to 600×600 mm may be used (Flemming, K. et. al. 2009). Although the square cross-sectional area is the most commonly used, other cross-sectional areas may include circular, triangular, hexagonal and H-sections. *Pre-cast concrete piles* are usually installed using a drop hammer of 3 – 4 tons (GeoForum, 2012). However, hydraulic hammers have become more common for driving pre-cast concrete piles. Where the ground conditions a suitable, vibratory installing methods may be used. *Pre-cast concrete piles* in Sweden are nowadays exclusively produced at indoor factories, but at-site production is possible. The advantage of pre-cast concrete piles is that the quality of the pile and production process can be monitored in a more efficient way then for cast-in-place concrete piles. One disadvantage for *pre-cast concrete piles* is that it is not suitable for soil conditions containing a significant amount of boulders as the installation process may disfigure the pile (GeoForum, 2012). However, the pile may be installed with a protective pile shoe in such conditions. To reduce the degree of soil displacement when using *pre-cast concrete piles* in clayey soil conditions, plug removal can be performed before the piling procedure (Olsson, C., Holm, G., 1993). A rule of thumbs says that a plug removal can be performed to a depth of about 8 m in clayey soil conditions.

Unless the piling is performed at a constant depth with more or less uniform soil conditions, the most common type of pre-cast concrete piles is the *Jointed Concrete*

Piles (Flemming, K. et. al. 2009). If the pre-cast concrete piles are not of the jointed type, extending the piles is a time-consuming and costly process as it involves breaking down the pile head of the installed pile to join together the steel reinforcement and casting concrete to form a joint. As the joint has to have at least the same bearing capacity as the rest of the pile, execution of pile joints is a critical part of the whole piling process. *Jointed Concrete Piles* are generally of smaller dimensions than non-jointed pre-cast piles, up to 400×400 mm handling loads up to 2500 kN (GeoForum, 2012). Each pile-unit is usually between 5 – 14 m long and joined together by various types of splices. The two most commonly used splicing methods are the bayonet joints (Hercules joint) and wedge joints (Stabilator joint) (GeoForum, 2012). Joints with a low quality may lead to loss of energy during driving and misalignment of the pile which can lead to damage during driving (Flemming, K. et. al. 2009). Each pile unit usually has a main 12 – 20 mm reinforcement bar at each corner and transverse reinforcement by a 5 mm diameter spiral for minimize the risk for longitudinal cracks. *Jointed piles* have been installed to a depth up to 100 m (Flemming, K. et. al. 2009). However, pile length exceeding 30 m is not common.

To more efficiently resist tensile stresses in the pile during driving and to minimize the risk of cracking in the pile from bending stresses during driving, the pre-cast concrete pile may be *pre-stressed* (Flemming, K. et. al. 2009). However, there are some disadvantages with *pre-stressed concrete piles*, such as reduced ultimate strength in axial compression which makes the piles weaker to striking e.g. boulders during driving. Also, *jointed piles* cannot be of *pre-stressed* type, making the *pre-stressed* type of pile only suitable for situation where the pile length is foreseeable and constant. A *pre-stressed pile* element is usually up to 20 m long and can be welded together by steel end plates (GeoForum, 2012).

To cope with heavy axial loads combined with extent bending loads, *hollow concrete tube piles* may be used. *Hollow concrete tube piles* may also be used at sites with thick deposits of sand where great piling depth is required combined with good shaft and end bearing capacity. Typical diameters for this type of pile are 600 – 1500 mm and a piling depth of 80 m has been reached. To penetrate hard strata the hollow tube may be closed-ended, but the pile can also be driven open-ended. Either way, the degree of displacement is large. The sections of the concrete tubes are pre-tensioned, making this type of pile twice as resistant to moment as a conventional pre-cast concrete pile of equivalent weight. (Flemming, K. et. al. 2009)

One version of this type of pile is the *Daido SS Pile*, which due to a special concrete-element production method using autoclave curving and centrifugal processes can achieve a concrete with a compression strength of 78 MPa (GeoForum, 2012).

The *Hollow concrete tube pile* can be used in a combination with steel H-sections or timber piles to form a composite pile. After the concrete tube has been installed at required depth, a steel H-section or timber pile is inserted inside the void of the concrete tube. The advantage of this pile is that the concrete tube will act as protective barrier for the material inside, minimizing the risk of moldering / corrosion. (GeoForum, 2012).

Timber piles

Driving timber piles into the soil is one of the oldest piling methods, and is still commonly used in the Scandinavian countries. Where suitable raw material is present, timber piles may be a cost-effective method for transmitting modest loads (< 500 kN) to a piling depth of about 12 meter. Usually, the cross-sectional area is circular, using the natural shape of the timber. But also square cross-sectional area may be used. The diameter of the pile is dependent on the features of the raw material, but rarely exceeds 0.4 meter. The most common installation method for timber piles is by using drop hammers with a maximum weight equal to the weight of the pile. However, the driving process may cause damage to the pile, as the drop hammer can crush and separate the fibers within the timber. Due to the risk of damaging the pile, this material is not suitable for piling through dense strata. Steel or cast-iron pile-shoe may be used to penetrate relatively dense strata. To minimize the risk of damage during the installation, steel bands around the pile head may be used. The main disadvantage of timber pile is the vast variation in quality due the material anisotropy. Also, there is a risk of deterioration. (Flemming, K. et. al. 2009)

Vibrated Concrete Columns (VCCs)

Where weaker deposits are located above layers of dense gravel, *Vibrated Concrete Columns* (*VCC*) may be a suitable alternative to driven pre-cast or cast-in-situ piles. This sort of displacement pile may be installed so that an enlarged base and / or "mushroom" head are obtained. The installation is performed by using a so called *Vibroflo-tube*, which is a tube that when being vibrated penetrates soft soils. The tube is during installation filled with wet cement, which after reaching required depth is pumped into the void formed when the tube is retracted. To obtain an enlarged base, the tube is after being retracted about 1 meter re-entered into the wet cement, compacting and displacing the concrete. Reinforcement may be installed to the wet cement after that the tube has been fully retracted. Typical shaft diameters range from 400 – 750 mm. The load capacity of a *VCC* pile is similar to those for traditional cast-in-place piles. Although its narrow application, this method is regarded as quiet and causes low levels of vibrations. (Flemming, K. et. al. 2009)

3.1.2 Low-displacement piles

To decrease the displacement from driven piles, piles with relatively small cross-sectional area can be used. This include piles with steel H-section, X-piles or open pipe piles. One method which is not of steel-type has been categorized as "low displacement pile", the *Starsol Pile* as some amount of spoil is produced during installation of this pile. Installation of driven *hollow steel piles* is similar to when installing conventional pre-cast concrete piles, using various sorts of hammers or vibratory equipment. The amount of displaced soil for driven hollow steel piles is therefore equivalent to the pile volume, same as for conventional pre-cast concrete piles. The reason for why driven hollow steel piles is categorized as "low displacement piles" in this report is that an certain volume of pile of hollow steel has a higher bearing-capacity than equivalent volume of conventional pre-cast concrete pile. By using low displacement piles, effects from soil heave and compaction are reduced (Flemming, K. et. al. 2009). The allowable design load, P_a, for steel piles is the cross-sectional area of pile at cap, A_p, multiplied with the allowable steel stress, f_s according to Equation 2 (Bowles, J.E., 1996);

$$P_a = A_p \times f_s \quad (2)$$

Preformed steel piles may be of many different sorts, and is in this report divided into *H-sections*, *X-sections*, *box sections*, and *tubular sections*. According to Flemming, K. et. al. (2009), there has been reluctance from some engineers regarding to use steel piles due to the risk of corrosion. But Flemming, K. et. al. (2009) also states that the corrosion rate of steel is low in natural, uncontaminated ground. At sites where granular soils are present, vibratory hammers is the most effective pile installation method for steel piles (GeoForum,, 2012). Impact hammers are also commonly used, and if the environmental requirements are lower diesel hammers may be used. Steel piles tend to be more expensive compared to equivalent length of concrete piles. However, higher load-carrying capacities is received for a given pile-weight if steel is used.

Slender *H-section* and *X-section* piles have a low displacement volume due to its reduced cross-sectional area. However, the degree of displacement is exclusively dependent on the soil condition as “soil plugs” can be formed at sites with stiff cohesive soils (GeoForum, 2012). Problems with obtaining a required resistance in some soils are current for *H-section* and *X-section* piles (Flemming, K. et. al. 2009). However, problems with low resistance can be solved with added “wings” to increase the surface area of the piles. *H-section* and *X-section* piles have better driving characteristics compared to conventional concrete piles and can therefore be installed to greater depths (GeoForum, 2012). Due to the better driving characteristics, *H-section* and *X-section* piles are more suitable for driving through harder soil layers or other sub-surface obstacles. However, *H-section* and *X-section* piles have a bending tendency with considerable curvature as a result when driving them to great depths (GeoForum, 2012).

To reduce shaft resistance during *H-section* pile installation and to increase the shaft resistance when the pile is in use, grouting can be injected at the toe of the pile (van Paassen, B.P.H., van Dalen, J.H, 2009). This method, named the *MV-pile* (Müller Verpress), comprises an *H-section pile* fitted with a grouting tube connected to the toe of the pile, see Figure 6. The grouting agent (usually cement-based) is injected to the toe of the pile during driving, which can be performed with a hammer or a vibrator (GeoForum, 2012). The injected grouting agent will form a liquid state zone in the area adjacent to the pile shaft, reducing the friction along the pile during installation and increasing the resistance after the grouting agent has hardened. *MV-piles* can be installed to a great depth as the reduced friction will lead to a more efficient energy-transfer from the hammer to the toe of the pile (van Paassen, B.P.H., van Dalen, J.H, 2009). Another advantage for the *MV-pile* is, according to van Paassen, B.P.H., van Dalen, J.H (2009) that its bearing capacity for tension is greater compared to for conventional piles. Post-grouting (injecting grout agents after the pile installation) can also be used to increase the bearing capacity of steel piles.

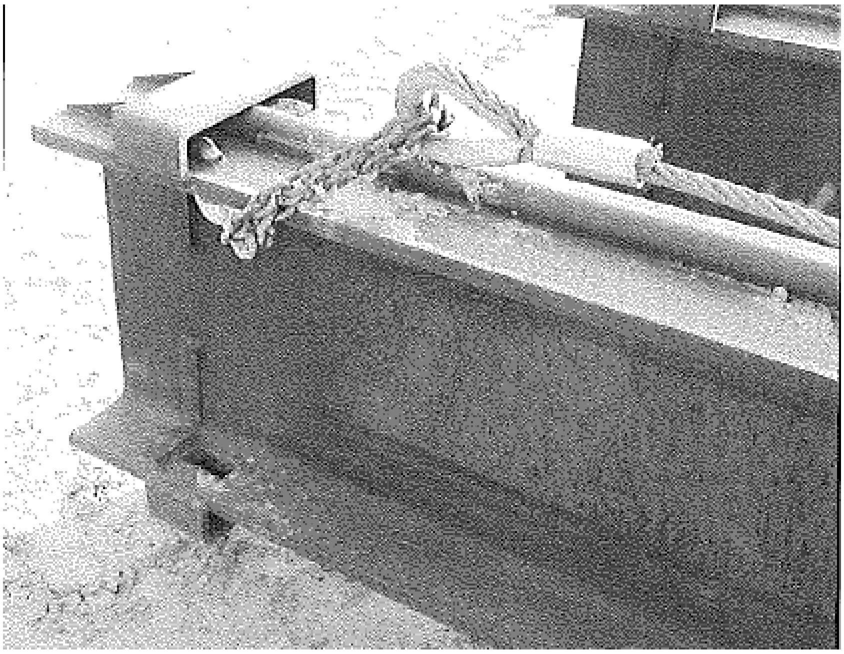


Figure 6 MV-pile (van Paassen, B.P.H., van Dalen, J.H, 2009)

To reduce the earlier mentioned problem with bending during installation, slender *H-section* and *X-section piles* can be replaced with the stiffer *Hollow steel piles*, such at the Ruukki *RR-pile*. The steel tubes used for piling is usually manufactured in seamless spirally welding (Ruukki, 2012c), with an outside diameter of 0.3 – 2.0 meters (GeoForum, 2012), making it suitable for foundation of heavier superstructures or constructions requiring high resistance to bending moments. When driving tubular piles, the end can either be closed or open (GeoForum, 2012). Even though open-ended piles are used, the soil displacement tend to be equivalent to closed ends as plugs usually are formed, especially at sites where cohesive soil is present, if impact hammers are used or if long piles with a small diameter is used. To reduce the risk of plugging in granular soils, vibratory hammers can be used, which also may reduce the total installation time. According to GeoForum (2012), closed-end piles have generally a higher bearing capacity than open-ended piles. Installing hollow steel piles is often performed using top driving, but at difficult ground condition a combination of drilling and driving might have to be used (GeoForum, 2012).

One piling method which are not steel piles has been categorized as "low-displacement piles", the *Starsol Pile*. The *Starsol Pile* is a further development of the *CFA Pile*, which is described at "non-displacement piles". When constructing a *Starsol Pile*, a hydraulic motor will drive an earth-cutting rotation head, which in turn will drive an auger head into the ground (GeoForum, 2012). During the penetration of the soil by the auger head, no spoil is produced. Stiff soil and soft rock can be penetrated with this method. When the auger head has reached required depth, it is raised a few centimeters to reveal two openings at the grouting tube and concrete is injected as the auger head and grouting tube is retracted without rotation. As the auger head is not rotated, spoil will be received at the ground surface. After the auger head is removed, a reinforcement cage can be put in place. The diameter for a *Starsol Pile* is usually 0.4 – 1.0 meter and can be installed to a depth of 20 meters (GeoForum, 2012). The sequence of a *Starsol Pile* construction is shown in Figure 7.

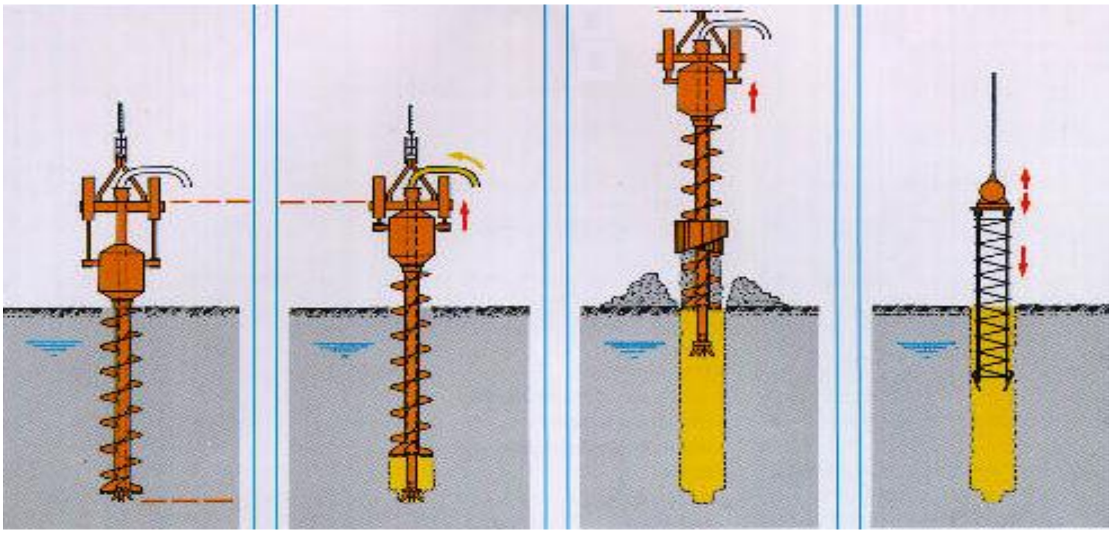


Figure 7 Installation process of a Starsol Pile (GeoForum, 2012)

3.1.3 Non-displacement piles

To reduce the lateral stresses in the ground during piling, the “non-displacement piles” enables some sort of excavation of soil material during pile installation (Flemming, K. et. al. 2009). Similar to the soil displacing *Atlas piling system*, a void is produced in the soil which is later filled with concrete and reinforcement. In contrast with the displacement pile, the casing or auger is bored into the ground rather than screwed or driven. The “non-displacement piles” have been divided into four sub-categories;

- Bored cast-in-place piles
- Partially pre-formed piles
- Steel Core piles
- Continuous flight auger piles (CFA)

The negative effects from soil displacement are therefore eliminated, making this type of piles suitable for urban conditions where surrounding constructions are adjacent to the piling site. However, advantages as compaction of granular soils and no soil spoil production are lost using “non-displacement piles”. Handling of soil spoil can be costly, especially if the site is contaminated.

Bored cast-in-place piles

The specific design load determines the diameter of the pile, where piles with a diameter ≤ 600 mm is referred as small-diameter piles and piles with a diameter > 600 mm is referred as large-diameter piles (Flemming, K. et. al. 2009).

The *Benoto System* is an early method to produce large diameter bored piles (GeoForum, 2012). A large diameter double-walled casing is pressed into the ground using an oscillatory movement. Figure 8 shows the equipment needed to achieve the oscillatory movements. As the casing is open-ended, soil material will be accumulated inside the casing as the penetration proceeds. This soil material is removed using an internal grab, lowered from the surface to bring the soil material from the bottom of the casing to the surface. Where harder strata are to be penetrated, such as weak sedimentary rock, hardened cutting edges can be fitted to the casing (Flemming, K. et. al. 2009). After that the casing has reached the required depth and the internal soil material is removed, the casing is filled with concrete (GeoForum, 2012). During the concreting, the casing is retrieved towards the surface in short intervals in an oscillating motion. After each retrieving interval, the concrete is compacted through pushing down the casing a few centimeters. Also, this compaction will improve the contact between the concrete and the surrounding soil. Instead of pushing down the casing, a further development of the *Benoto System* uses vibratory equipment to compact the concrete, called the *Monierbau Pile* System. Using the *Benoto System*, piles with a diameter of 0.5 – 2.0 m, with a length up to 40 m can be obtained, and tilted up to 14°. The *Benoto System pile* can be used in most soil types, and has therefore been very popular in urban areas in Europe for transmitting larger loads to deeper soil layers. Due to the slow progress rate of the *Benoto System*, pile system using oscillation unit separated from the pushing unit for the casing has been developed. The *Bade System* and the *Hochstrasser-Weise System* emphasizes this type of separated system. However, the installation sequence of the *Hochstrasser-Weise System* also differs from the *Benoto System* as compressed air is used when the casing is lifted after the concreting stage. To obtain an enlarged pile base, similar to the conventional *Franki Pile*, the bored method of *Brechtl System* may be used. In this method, concrete is inserted under pressure. At a specific pressure-level, the detachable bottom plate is released and a concrete bulb is formed at the base of the pile.

Figure 8 A casing oscillator machine used for installing Benoto System piles (Hellotrade, 2012)

Another way to produce large diameter piles is using the *Lind-Calweld Pile System*, which does not emphasize oscillation when installing the casing. Rather, an open-ended tubular casing is driving into the ground using impact hammer or vibratory methods. The soil inside the casing is removed using an auger when the pile has reached the required depth. Cementation takes then place as the casing is simultaneously retrieved and vibrated. (GeoForum, 2012).

The *Franki Pile* with enlarged base described earlier can also be used as a non-displacement pile. Some configuration from the conventional *Franki Pile* has to be done to achieve the “non-displacement”-criteria. This is done by using a grab consisting of two semi-cylindrical jaws, which slides inside the casing and excavate the soil inside the casing during penetration (GeoForum, 2012). When the casing has reached the required depth, the *Franki Pile* then constructed as described above.

To increase the bearing capacity of bored piles, an enlarged base can be created using an underreamer, resulting in a so called *Underreamed Pile*. A picture of an underreamer tool is shown in Figure 9. After the casing has been installed to required depth, the drill rod continues to bore into the soil and the underreamer is pushed outward and expands, producing the enlarged base. The most common shape of the underream cuts at 45 – 60 degree angle and has a maximum diameter of three times the diameter of the casing. The expanded underreamer is then retracted as the drill rod is lifted. This method is primary used at sites where stiff cohesive soils is present, as there is a risk of collapse if the soil is too unstable. (GeoForum, 2012)

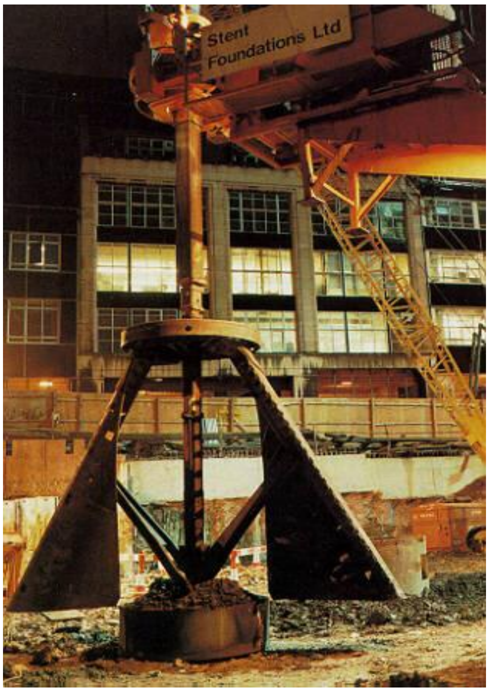


Figure 9 Underreaming tool to perform enlarged pile basis.(Geoforum, 2012)

Small diameter (≤ 600 mm) *bored cast-in-place piles* can be installed by various methods depending on the nature of the soil. This type of piles can carry their loads either by shaft friction, end bearing or a combination of shaft friction and end bearing (GeoForum, 2012). To construct *small diameter bored cast-in-place piles*, tripod rigs were generally used (Flemming, K. et. al. 2009). These tripod rigs are light and easily transported. However, this method is labour-intensive and require manual handling of casing sections. The advantage with the light tripod rigs is that it can be used for installing piles in urban areas and even from inside buildings.

Partially pre-formed piles

In contrast to “bored cast-in-place” piles, this type of piles emphasizes filling a bored void in the soil with a solid pre-formed piling element. The advantage with these types of piles is that the pile material can be closely monitored and tested during production as it is not produced in-situ. The disadvantage is that the shaft friction is lower compared to cast-in-place piles and post-grouting is sometimes needed to achieve required shaft friction. (Flemming, K. et. al. 2009)

A piling method which uses cylindrical pre-cast concrete units is the *Prestcore System*. A conventional boring method is used to install a casing to required pile length, where fluid concrete is placed at the bottom of the excavation. The cylindrical pre-cast concrete units with a central hole and peripheral holes are then lowered down the casing. Reinforcement is threaded through the peripheral holes. The casing is then slowly lifted, forcing the fluid concrete at the bottom to form an enlarged pile base. Grouting is then inserted into the central hole of the cylindrical pre-cast concrete units as the casing is extracted, filling the gap between the cylindrical pre-cast concrete units and the surrounding soil. The usual diameter of the *Prestcore System* is 0.35 – 0.66 m, coping with loads up to 1000 kN. (GeoForum, 2012)

Instead of using pre-formed concrete units as pile material in bored void, hollow steel piles can be used, such as Ruukki´s *RD-pile* (Ruukki Drilled pile). The hollow steel pile is then fitted with a drilling crone at the bottom of the pile, which is given torque via a stem leading from centric percussive drilling equipment (Rukki, 2012c). The connection between bedrock and pile will be solid as the RD-piles may be drilled into the underlying rock. The pile diameter ranges from 90 to 800 mm and each pile section can spliced together by either thread joint or welding. The *RD-pile* can be used for all soil conditions. However, the *RD-pile* has to be installed to solid bedrock as the shaft friction is not sufficient if grouting is not used.

Steel Core Piles

To cope with high loads and still need to avoid excessive noise and vibrations, *Steel Core Piles* may be used. The *Steel Core Pile* has a bearing capacity in the range of 500 – 5000 kN and can be used for both compressive and tensile forces (Bredenberg, H. 2000). The *Steel Core Pile* is installed by drilling a steel tube casing through the soil and into the solid bedrock about 0.3 – 0.4 m (Hercules, 2012). There are various methods for the drilling-procedure, since the drilling unit can either be feed from the top of the casing or lowered into the casing and the drill bit can either be of centric or eccentric design (Bredenberg, H. 2000). After the installation of the casing, a steel core is inserted into the casing. The steel core may be further drilled into the solid bedrock if required (Hercules, 2012). Commonly used diameters for the steel core varies between 80 – 210 mm (Bredenberg, H. 2000). The void formed between the casing and the steel core is then filled by injecting wet concrete from the bottom. This method can be used at sites which contain large amounts of boulders and stones as these obstacles are easily penetrated through drilling. The steel core is well-protected from corrosion as both the casing and the concrete will act as protective barriers. According to Bredenberg (2000), the *Steel Core Pile* is mainly used in the Scandinavian countries. The main disadvantage of the *Steel Core Pile* is according to Berglars (2009) that this piling method is very expensive compared to conventional pre-cast concrete piles.

Continuous flight auger piles (CFA)

Continuous flight auger piles, hereafter named *CFA piles*, are constructed by a rotating continuous-flight auger, drilling it into the ground to the required depth (Flemming, K. et. al. 2009). Some amount of soil is during the drilling removed and deposited as spoil at the ground surface. The torque of drilling equipment has to be powerful enough so soil displacement doesn’t occur. The hollow stem that later will feed concrete to the auger is during installation closed-ended by a disposable temporary plug. During installation of the *CFA pile*, there is no need for temporary casing as the soil-filled auger supports the sides instead of the in-situ soil (GeoForum, 2012). When required depth is reached, the temporary plug of the stem is displaced as the auger is lifted a few centimeters and the workable concrete mix is pumped into the hollow stem (Flemming, K. et. al. 2009). The required depth of a *CFA pile* is restricted to about 30 m. To obtain the concrete shaft, the auger is retrieved as concrete is pumped down the hollow stem of the auger. As the auger is rotating in the same direction during the retraction as during installation when it is retrieved, the in-situ soil is efficiently replaced by the wet concrete. The direction of the rotation during extraction of the auger is also necessary to ensure that the wet concrete mix fills the entire void (GeoForum, 2012). Also the speed of the auger extraction and the flow of wet concrete mix must be in balance as drilling spoil must not enter the wet concrete, which would result in a soil-contaminated pile shaft. Reinforcing cage is then installed by pushing the cage into the cement while it is still fluid (Flemming, K. et. al. 2009). Longer cages (>10 m) will usually require the use of a vibrator to be able to sufficiently sink into the wet concrete. To eliminate the risk of soil-contamination of the pile shaft due to eroded soil walls, the reinforcement cage may be installed inside an over-sized stem before extracting the auger (GeoForum, 2012). The sequence of *CFA pile* construction is shown in Figure 10.

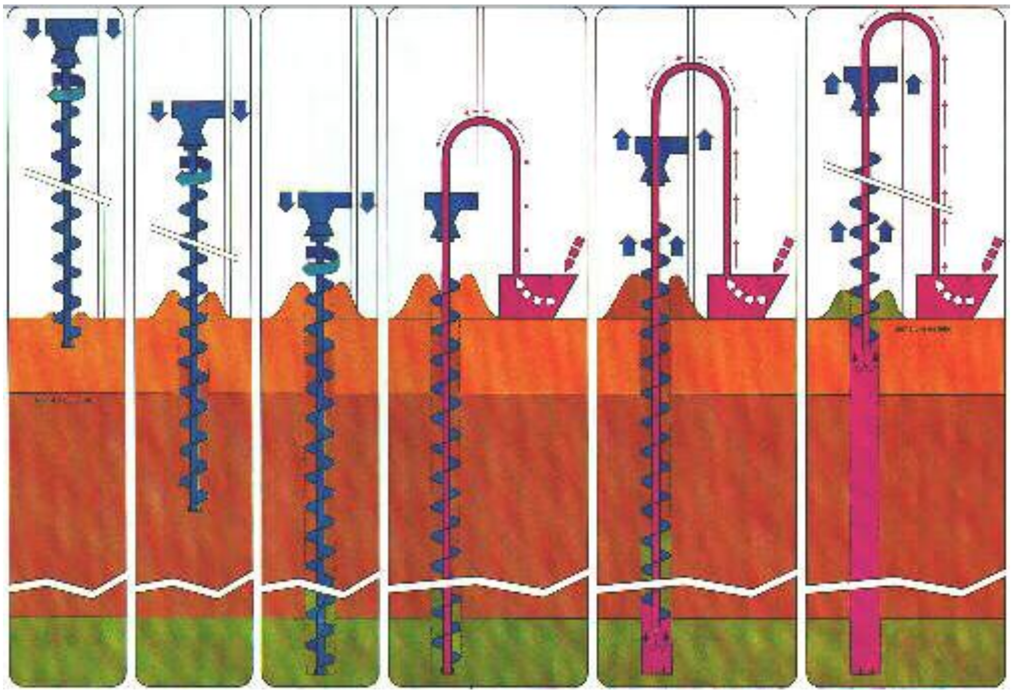


Figure 10 Installation process of a CFA pile (GeoForum, 2012)

Typical pile diameters of *CFA piles* and pile loads are presented in Table 2 below.

Table 2 CFA pile diameters and design loads (Flemming, K. et. al. 2009)

Pile diameter [mm]	Approx. design load [kN]
300	350
400	500
450	500 – 750
500	750 – 1000
750	1000 – 2500

The reason for the relatively large span in design load for the larger pile diameters is because of that the quality of the pile is dependent on the operator performance, which can lead to an over-estimation of the load bearing capacity if inexperienced or un-skilled personnel install the pile (GeoForum, 2012).

3.1.4 Driving methods

Driving pre-fabricated pile-elements is an intrinsically noisy process which also in some cases may be a major source of vibration problems. It ought to be mentioned that the magnitude of the impact to the urban environment regarding noise and vibrations is strongly influenced by the current soil conditions at site and not only by a certain driving method. As environmental effects from piling are becoming more significant, development of "silent" pile driving methods has become more commonly used the last few years (Fleming, K. et. al. 2009). Also monitoring of the hammer input, such as hammer velocity and energy input, may be used to adjust the piling sequence to reduce the negative impacts to the surrounding urban environment. Four major types of pile driving methods with different alternatives to each method will be presented in this chapter. Also typical noise and vibration levels will be presented for each major type of pile driving method.

Drop hammers

The drop hammer is the simplest and most traditional method of driving pre-fabricated pile-elements and is still widely the most frequently used method for installing pre-fabricated concrete piles (Fleming, K. et. al. 2009). To drive the pile into the ground, a weight approximately equal to that of the pile is released from a suitable distance from the pile head and guided via a leader strikes the pile head. The weight of the hammer usually varies between 1000 – 5000 kg and is usually released from a distance of 0.2 – 2 meter from the pile head (GeoForum, 2012). To reduce the risk of eccentric strikes, which may produce peak stress at the pile head, long and narrow hammers are preferable as the blows are more likely to be axial with the slender pile (Fleming, K. et. al. 2009). Drop hammers are mounted on a leader on either mechanical or hydraulic powered piling rigs, hoisting the hammers via a simple winch. A schematic drawing of a piling rig is shown in Figure 11.

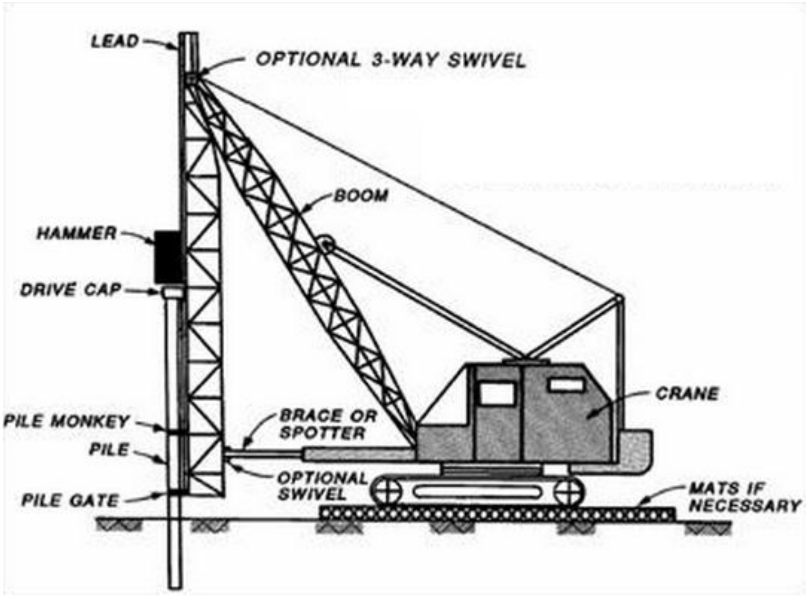


Figure 11 Typical pile driving rig using drop hammer for pile driving (Foundation Engineering, 2012)

More energy is required to install raked piles, as approximately 10 % of the driving energy is lost for a pile with an inclination of 1:3. Excessive stress at the pile head is more likely to occur using a hammer that is too light, dropped from an excessive height, then using a hammer that is too heavy, dropped from a lower height. Increasing the ratio of hammer weight / pile weight will lead to successive increase of pile driving efficiency, which is shown in Figure 12. The pile represented in Figure 12 weights 4.7 ton, and the weight of the hammer varies from 1.5 ton to 5 ton. One can see that when the ratio of hammer weight / pile weight is 0.32 only about 5 % of the total energy from the hammer is useful for pile driving, remaining energy is lost due to temporary compression and inertia loss. When the ratio of hammer weight / pile weight is raised to 1.06, almost 50 % of the total energy from the hammer is useful for pile driving. (Fleming, K. et. al. 2009)

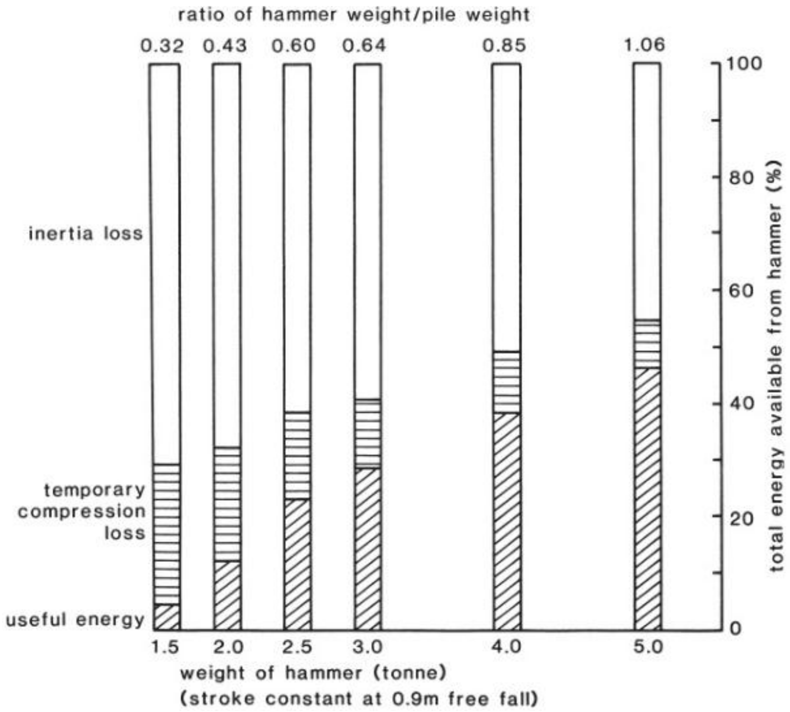


Figure 12 Energy distribution during pile driving for increasing hammer weights (Fleming, K. et. al. 2009)

The temporary compression showed in Figure 12 is strongly influenced by current ground conditions as the temporary compression will be reduced for driving in easy conditions (GeoForum, 2012).

Drop hammers typically produce rather high noise levels. From measurements of 45 piling projects, the typical noise level, L_{eq}, has been determined for various piling methods. These measurements show that the typical noise level at a distance of 10 meters from the piling operation for unscreened drop hammers is about 101 dBA. However, drop hammers can be fitted with noise abatement devices, such as sound-reducing boxes around the hammer equipment, resulting in an equivalent noise level of 75 dBA. (Langley, M.S. 1980)

There are some variants of the traditional drop hammer. These variants include raising the hammer by steam, compressed air or hydraulic machinery (Fleming, K. et. al. 2009). The hammer may be free falling from the required height (*single-acting hammer*) or forced by pressure on the down-ward motion (*double-acting hammer*). *Single-acting hammers* use a cylinder of a weight varying from 2500 kg to 20000 kg for pile driving, making it more powerful than traditional drop-hammers. The cylinder is raised by steam, compressed air or hydraulic machinery, eliminating the need for winch, which has a limited capacity (GeoForum, 2012). The pneumatically or hydraulically pressure is adjusted so suitable height is received by the cylinder, which is limited to about 1.5 m. *Single-acting hammers* are commonly used for driving large-diameter steel tube piles in marine structures. However, its usage in urban areas has declined due the vast environmental effects such as noise and vibrations. Another disadvantage with *single-acting hammers* is the relatively slow rate of operation with a driving rate of about 60 strokes per minute. *Double-acting hammers* is powered by air or steam in the up-ward motion as well for the down-ward motion (GeoForum, 2012). This type of hammer is generally not suitable for concrete pile-driving and is mainly used for driving sheet piles (Fleming, K. et. al. 2009). The advantage with *double-acting hammers* is the rapid succession of strokes to the pile, with a driving rate of 100 – 300 strokes per minute (GeoForum, 2012). The disadvantage with *double-acting hammers* is that it require extent maintenance and lubrication. Schematic sketches of *single-acting* and *double-acting hammers* are shown in Figure 13.

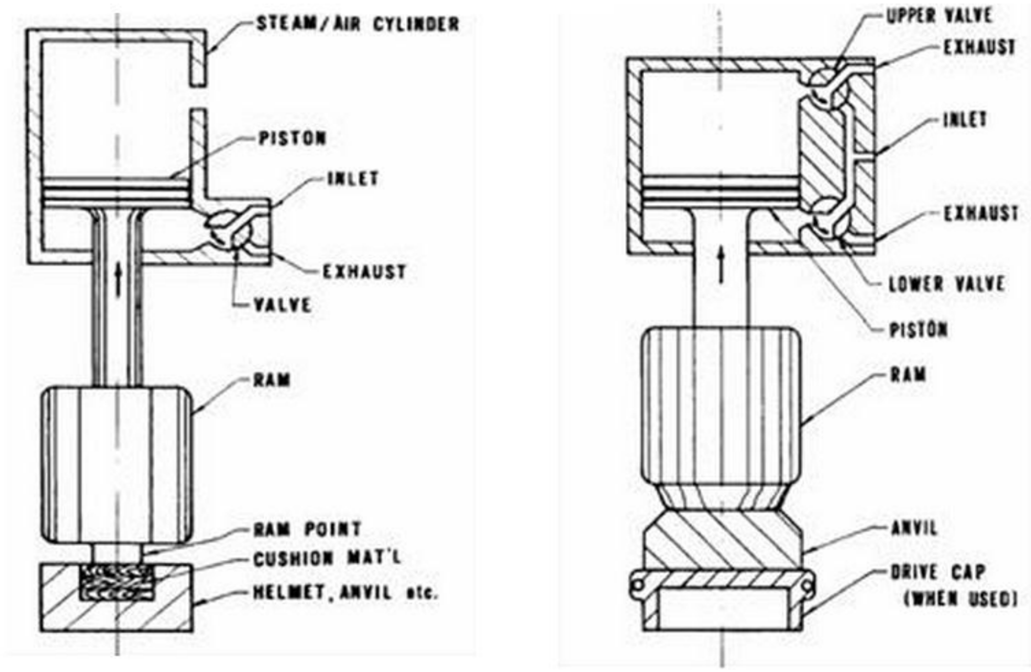


Figure 13 Schematic sketch of single-acting hammer (to the left) and double-acting hammer (to the right) (Foundation Engineering, 2012)

A further development of the *drop hammer* is the *hydraulic hammer*, where the hammer is lifted hydraulically to required height and then either released or accelerated down-ward. The advantage with this system is that the drop height can be set automatically with high accuracy. Another advantage of *hydraulic hammers* is that the stroke-efficiency is higher than the driving systems described above. Due to the hydraulic uplift, this system emits less noise and vibrations during this stage of the process. However, the dominant noise is caused by the impact of the hammer on the pile, resulting in similar noise emissions as the other forms of drop hammers. *Hydraulic hammers* are in general more expensive than other forms of drop hammers and have a relatively low driving speed. (GeoForum, 2012)

Diesel hammer

For pile driving in hard conditions, *diesel pile hammers* may be an efficient method. The *diesel pile hammer* is executed by raising a piston by a controlled explosion at the base of a closed cylinder (Fleming, K. et. al. 2009). During the down-ward motion of the piston, the air within the cylinder is compressed (GeoForum, 2012). By inserting a certain amount of diesel oil at the concave-shaped end of the cylinder, the compressed air will be ignited, which will give down-ward driving energy to the pile, which in turn already is moving downward by the impact from the piston. The piston will after the impact with the pile cap be raised by the explosion, and the process with the falling piston begins all over again. The working-sequence of a single-acting *diesel hammer* is shown in Figure 14.

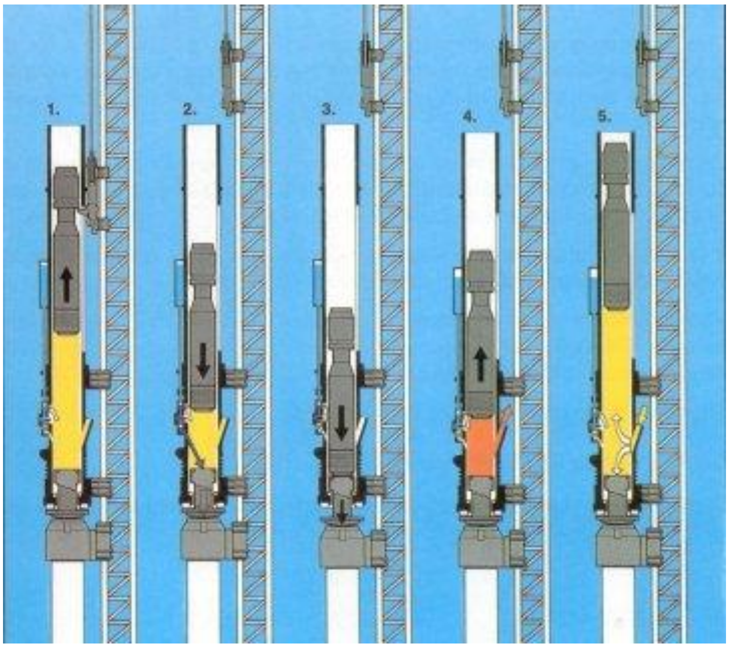


Figure 14 Working-sequence of a single-acting diesel hammer (Ape Holland, 2012)

Diesel hammers can also be double-acting, where vacuum is created in the cylinder as the piston is raised. This vacuum will accelerate the piston on its down-ward motion, giving almost twice the amount of output rate compared to single-acting *diesel hammers*. *Diesel hammers* are however ineffective in soft soil as the impact of the piston is not strong enough to atomize the fuel. The disadvantages with *diesel hammers* are that it produces fumes from the diesel oil ignition and is also a very noisy method. (Fleming, K. et. al. 2009)

Vibratory methods

Pile driving by *vibratory hammers* can be an efficient method for certain ground conditions. This type of hammers is powered either by electricity or hydraulic pressure and involves a vibrating driving unit attached to the pile head. The driving unit is powered by mobile generators or hydraulic power packs. The vibration is caused by rotation of eccentric masses, as can be seen in Figure 15. Connecting the vibrator to the pile head is done by using hydraulic clamping devices. (Fleming, K. et. al. 2009)

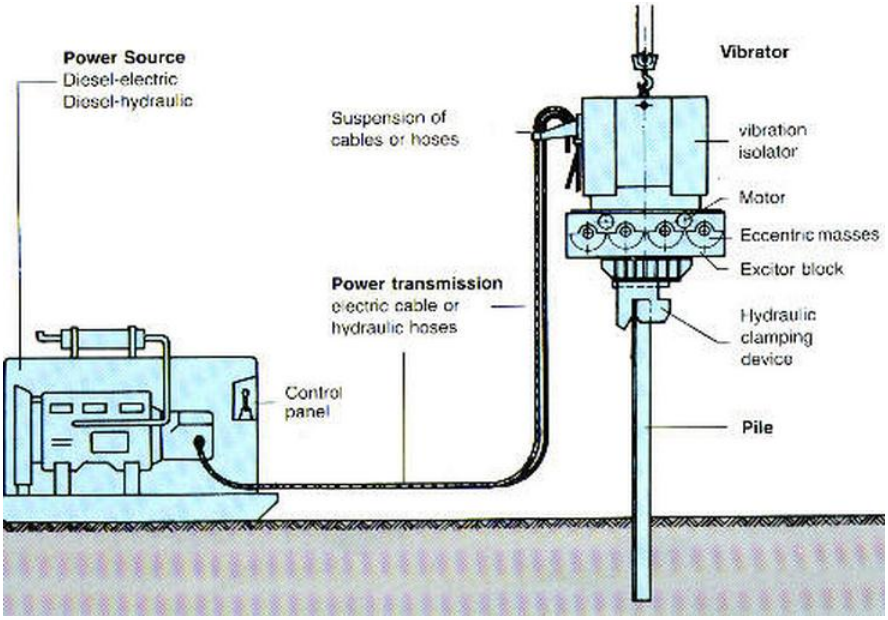


Figure 15 Schematic drawing of vibratory pile driving equipment (GeoForum, 2012)

The vibrations used for pile driving are usually of rather low frequency, typically 20 – 40 Hz. Resonance within the pile or in the soil will not be take place for these low frequencies. The typical amplitudes for the wave-motion within the slender pile are about 5 – 30 mm. Using *vibratory hammers* for pile driving in frictional soils, rapid rate of progress can be achieved. The granular soil in close adjacent to the pile is fluidized by the vibration and shaft friction is reduced, making it possible for the pile to penetrate the ground. Fluidization will not take place at sites with cohesive soils, making this method rather ineffective for this type of ground conditions. For soils with alternating properties, vibrators with variable frequency may be used to adjust the driving (GeoForum, 2012). Noise emission is usually low for this type of pile driving method and the vibrations usually does not cause damage to surrounding constructions. However, if the frequency of the vibrator is increased to about 100 Hz, the pile will resonate longitudinally (Fleming, K. et. al. 2009). This will increase the penetration rate, reaching about 20 m / minute for granular soils as the pile friction will almost be eliminated. The resonance created by the higher frequency might result in vast settlement problems, damage to surrounding constructions and increased noise emissions.

Jacking methods

For certain soil conditions, pile driving by *hydraulic jacking* is an efficient method. This method may be a suitable alternative to dynamic pile installation methods if e.g. using *drop hammers* is not possible (GeoForum, 2012). Adjacent piles or surrounding constructions can be used as a counterweight for the jacking. This type of installation method is regarded as very quiet and vibration-free (Fleming, K. et. Al. 2009). However, its usage is limited to soft and medium dense soils and only piles with reduced tip resistance can be installed.

3.2 Retaining structures

In this section different retaining structures will be presented. The retaining structures consists of cast-in-place piles, sheet piles, tubular piles, diaphragm wall panels or H-section piles.

3.2.1 CFA pile wall and Secant wall

Secant and CFA pile walls are constructed of CFA piles. A continuous CFA pile wall, see Figure 17, consists of only CFA piles with no overlap between. Secant pile wall, see Figure 16, consists of CFA piles with interlocking piles of CFA piles. (Korff et al, 2007). The stiffness and bearing capacity of the diaphragm wall is large. However, the diaphragm wall is very costly and large excavations are needed for economical reasons. One solution for smaller excavations is the continuous CFA or the secant walls. The secant wall is first constructed with two primary piles that is not reinforced and after the secondary pile is installed between and reinforced with a reinforcement cage or a steel profile.

One problem with all retaining walls are the water tightness, no retaining structure are totally impermeable. In case of secant walls the imperfections during casting of the CFA piles performs the leakages. The lack of verticality of the piles will also cause possible leakage through the wall since the overlap that is needed between will not be enough. Furthermore the structural behavior of the wall can perform cracks; this is a consequence of that the primary piles are not reinforced and the difference in stiffness between primary and secondary piles will create vertical shear cracks. Good concrete quality and enough overlap are the best remediation to this structural problem. Also the bending moment in the unreinforced piles will be greater than the breaking moment of the concrete and horizontal cracking occurs. Since the primary pile contains no reinforcement, to distribute the cracks over the pile, the width of the cracks will be large and with too little overlap it may reach the groundwater. The leakages can transport soil particles, especially in sandy soil, and sinkholes are obtained inside the retaining wall. This is mainly a problem in friction soil, since the cohesive soil will conduct less water than frictional soil. Also, the leakages will cause settlements in the buildings close to the excavation. In secant wall construction the water leakages can be solved with injection when the excavation is constructed with underwater concrete floor. (Korff et al, 2007).

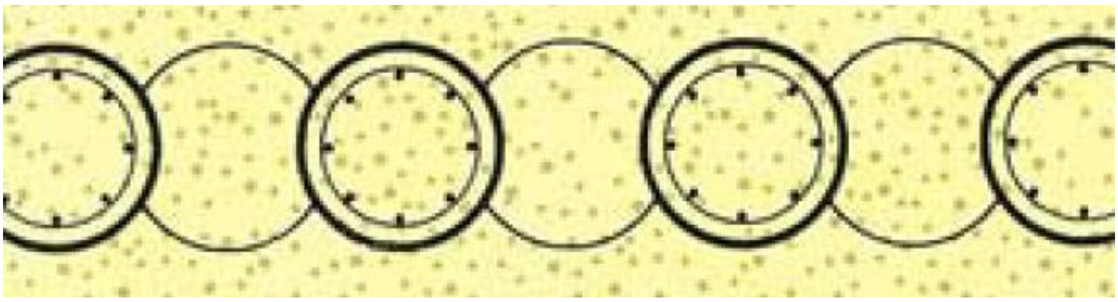


Figure 16 Secant pile wall. (Franki piling, 2008)

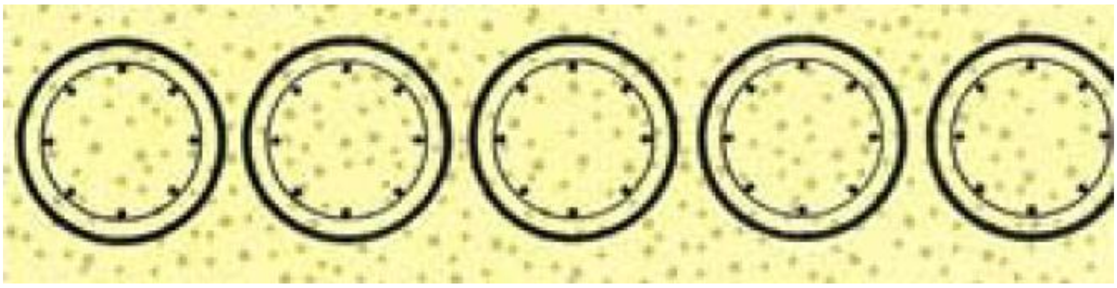


Figure 17 Continious pile wall, CFA-pile wall (Franki piling, 2008)

The advantages with continuous walls are (Puller, 2006)

- Continuous walls are suitable for granular soils with almost no exceptions. Cohesive soils, where excavation depth is lower than 8m in hard clay, are suitable. Also intermediate soils are suitable. Not suitable soils are clays with lower c_u than 10 kPa, weak organic soils and hard rocks. Soft rocks as chalk and marl are suitable.
- The high speed and low cost to construct a continuous wall for temporary and permanent purposes where the drilling conditions are suitable.
- When small excavations are performed, the distance between the surrounding buildings and the excavation can be minimized.
- Low vibrations and noise levels during the installation of the piles

The continuous wall method got however drawbacks where the main drawbacks are:

- The risk of groundwater leakage between the piles
- The 18 m depth limit of installation reinforcement cages without special measures, however it is possible to install steel beams for deeper piles, which is more costly.
- Low bending capacity of circular cross section
- To obtain an acceptable wall surface, extra work may be needed; a reinforced concrete wall or shotcrete can be used.
- For deeper walls the minimum distance to other structures may be large
- It is only possible to use the continuous wall where the ground water is not a hazard or where grouting or jet grouting can be performed to prevent leakage. When installing the CFA piles it is important to obtain verticality during installation, if not there will be too large spacing between the piles.

Many of the disadvantages of the continuous wall are overcome with the secant wall and interlocking wall. As mentioned before the secant wall are casted with primary piles and secondary piles. The importance with this construction process is that the concrete in the primary piles have not obtained the full strength before the secondary piles are casted. If this occurs, the wear on the auger is increased. Therefore the concrete quality and the differences in strength are of great importance. The secant wall as for the continuous wall has poor bending capacity in circular cross section. Often only the secondary pile is reinforced because the reinforcement in a primary pile would be damaged when the secondary pile is casted. In a few secant walls in London, the depth has been down to 40m and steel beam sections have been used for reinforcement of the secant wall. Also rectangular reinforcement cages or steel beam sections have been used in the primary piles in deeper walls where large flexural strength is needed. Weaker concrete, a cement bentonite mix, can be used in the primary piles when not large flexural strength is of concern. The advantage with this method is higher output of rigs and lower torque when boring the primary piles. This method is called hard-soft secant piling. The disadvantage is the lower durability of the soft concrete for permanent structures. The use of a reinforced concrete lining to obtain water resistance may be necessary. The use of hard-firm method for secant wall construction the objective is to obtain a permanent, watertight and durable structure. In hard-firm construction a concrete of characteristic strength of 10-20kPa (56 days) with retarder added is used. This because it is easier to cut secants in the hard-firm secant wall compared to hard-hard secant wall. CFA rigs are used for installation of the piles. To produce hard-hard secant walls, high torque rigs are used for deep installations and CFA rigs are used for less depth. In the hard-hard secant wall the primary piles may be reinforced by I-beams or rectangular/small square reinforcement cages. Secant walls can be installed with a small inclination to the vertical plane when basement excavations are performed.Guide walls are always needed to construct secant walls. (Puller, 2006)

3.2.2 Diaphragm wall

Diaphragm walls are concrete walls constructed in soil by hydraulic grabs or hydromills. The benefits of the diaphragm wall are that it can be used as both temporary and permanent support, better efficiency of bending in the rectangular shaped wall section compared to circular walls previously used, reduction of noise and vibrations when installation compared to installation of sheet piling where percussive drilling are used, easier installation of propping, anchoring and strutting, better ability to handle vertical loads and the possibility to construct deeper excavations than other methods of constructing walls. Since the diaphragm wall can be used both for temporary support during construction work of a basement and as permanent support during the lifespan, the diaphragm wall becomes sufficiently economical. However, the drawbacks of the diaphragm wall are the risk of losing and spill of bentonite slurry, the high cost of cleaning and disposal of slurry, the large space needed to store the reinforcement cages and the big cranes that are used to handle the cages. The continuity needed through the construction process; from excavation, concreting to removing of temporary stop is a drawback of the method. The diaphragm wall is the preferred method in construction of deep excavations, only concrete pile walls, especially secant walls, can compete with diaphragm walls at medium depth. Diaphragm walls may be constructed by prefabricated concrete elements. The benefit with this kind of solution is that there will be a clear and nice surface, better placing of reinforcement and better concrete quality, a consequence of this is thinner wall structure, better tolerances during installation and more watertight wall in the joints compared to conventional diaphragm wall. One drawback with this method is the heavy lifting equipment that is needed. This reduces the width and length of the concrete elements. Often the concrete elements are cast on site, thus some area for this is needed. The slurry that stabilizes the cut for the diaphragm is used to support the concrete element on the rear side of the wall and be easily removed on the front side. Another system is to use the slurry as support to the diaphragm wall cut and after grout cement grout before the concrete element is placed in the cut. According to the manufacturer the benefit with the method is that the wall can take larger vertical load. The precast diaphragm wall may be supported by anchors to reduce the horizontal deformations. Post tension of the concrete in the diaphragm wall can be used. The reason for using pre stressing is to reduce the amount of reinforcement, therefore lower reinforcement cost are obtained. However, the costs of tendons are high and therefore pre stressing is not very often used. (Puller, 2006)

3.2.3 Hydrofraise, Hydromill

The Hydrofraise was developed by Soletanche for construction of diaphragm walls. Similar methods of Hydrofraise have been developed by competing companies. It consists of a panel with two cutting drums rotating in counterclockwise and clockwise directions respectively. The alignment of the Hydrofraise can be adjusted in two ways; first the cutter drums are rotated in different speeds to adjust the direction to the right or left. The cutter drums can be tilted on order to get an adjustment perpendicular to the adjustment that the cutter drums crate. Powerful Hydrofraise can operate to a depth to 70m and the most powerful can bore into 100m. (Puller, 2006)

3.2.4 Steel sheet pile

In sheet pile wall construction different factors influences the installation. One of these factors is the ability to withdraw the sheet piles when using expires. Because of limited space the withdrawal will be complicated. The depth of the excavation will demand long sheet piles that will create a logistical problem in the inner parts of the town. The sheet pile to be used in a project is determined by the flexural strength and the strength for resisting the driving of the sheet pile. Different profiles of sheet pile sections exists; the Z-section and the U-section. Noise levels from piling are often restricted in urban areas despite the use of acoustic hammers. To reduce the noise levels during installation of the high frequency pile vibrators are used in cohesionless soil and in soft to firm clay. Hydraulic pile presses are used to for same reason in clays. However, increase of vibrations and reduced production result when obstructions, natural and manmade, occur in the ground. Dense sand and hard clays will demand jetting to improve the installation. Obstructions in the ground may demand pre-boring since driving through these obstructions may damage the clutches. (Puller, 2006)

3.2.5 King post wall, Berliner wall

The king post wall is one kind of a Berlin wall, see Figure 18. The reasons to this method are popular are the cheaper timber, cheaper boring with power auger in some soils and the better possibility of working in the excavation pit with anchors. In the king post method vertical soldier piles (H-beams) are bored or driven into the ground in a c/c space to 2-5m. The soldier beams are concreted at the end of the structure. During excavation timber laggings are placed between the soldier piles. Steel wailings are placed to transfer the load from the vertical soldier piles to ground anchors along the waling. An alternative is to place ground anchors in the soldier pile. When shallow excavations are performed the timber laggings may be replaced by reinforced concrete skin wall. Where excavation is performed in stiff clays the concrete can be arched in plan to distribute the load to the soldier piles. The use of king post wall is limited to rather dry ground or dewatered soils that are self-supporting during each lagging is constructed. The king post method is most used as temporary support of cutting and conventional methods are needed to construct permanent soil support. As mentioned before one of the advantages of the method is that the working space in the excavation pit is not reduced by using anchors or soil berms. However the method occupies thickness outside the permanent structure without contribute to the permanent structure. In areas the tolerance may not allow the use of the soldier wall as support to the permanent works. If poor support between soil and laggings are obtained the risk of settlements behind the wall becomes evident, which cause vast damages on structures in the proximity of the excavation. Some projects in Germany have used shotcrete instead of timber laggings or concrete between the soldier piles. (Puller, 2006)

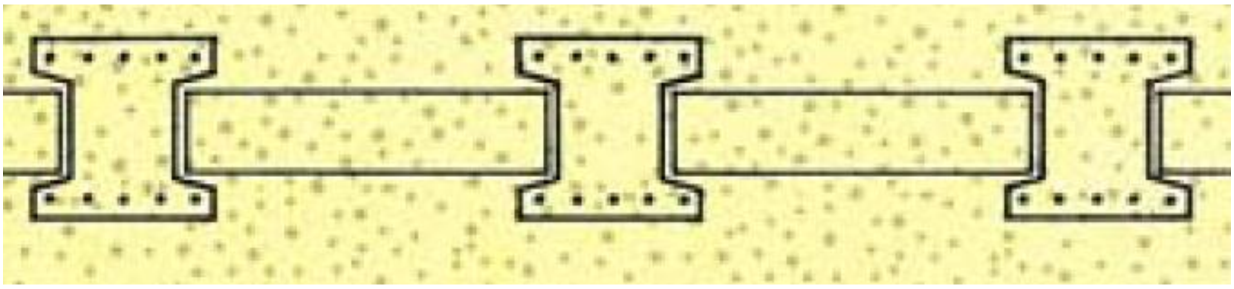


Figure 18 Soldier pile wall of concrete soldiers. (Franki piling, 2008)

3.2.6 SPTC (soldier pile tremie concrete)

In the soldier pile termie concrete wall the horizontal lagging is replaced with reinforced termie concrete. The space between the soldier piles are excavated and filled with Bentonite. After that, the termie concrete is filled into the excavated volume between the soldier piles. The concrete can be replaced with mesh reinforced shotcrete between the soldier piles. To use the shotcrete method absence of ground water is necessary.

3.2.7 Soil nailing

In order to stabilize natural or excavated slopes, as well as wall support, soil nails can be used. Small-diameter rods (25 – 30 mm) are either driven into the earth or holes with a diameter of 150 – 200 mm are drilled and a small-diameter rod (25 – 30 mm) is inserted within it, and the remaining of the hole is filled with cement based grout (Bowles, J.E. 1996). Soil nails are usually used as temporary support, however, the method is sometimes used for permanent support, see Figure 19. In urban environment, soil nailing is often used as support for temporary retaining walls (Avén, S. et. al. 1984). At excavations, nails are installed gradually from the upper part and downwards. The length of spikes is usually about 60 % of the excavation depth. Soil nailing may often be used in combination of other soil improvement methods, such as soil reinforcement to stabilize the soil between the nails in slopes. The fundamental principle of soil nailing is similar to soil reinforcement, i.e. mobilizing friction between the soil material and reinforcing material. Soil nails mainly mobilize tension forces, which will reduce the pressure against a retaining structure (SGF Jordförstärkningskommitté, 2003).

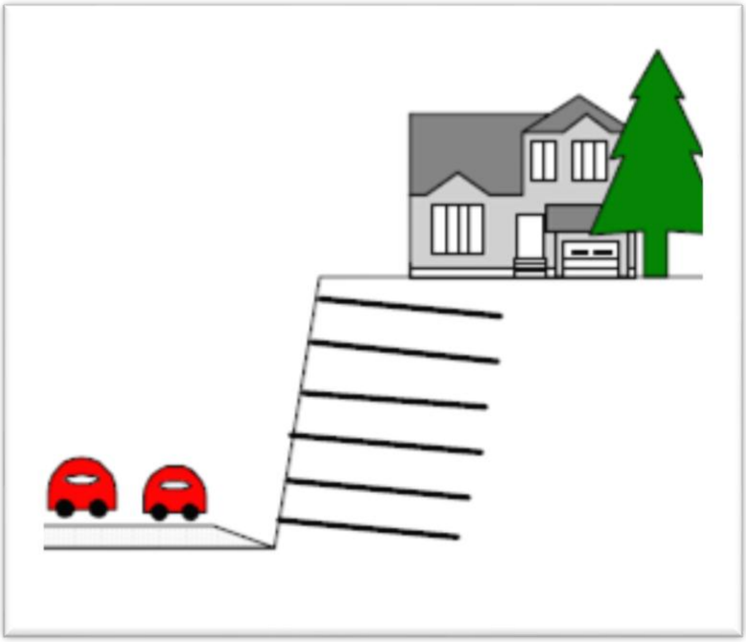


Figure 19 Soil nailing in slope. (SGF Jordförstärkningskommitté, 2003).

The first soil nailed wall was constructed in Vesailles in France 1973 and the method is most suitable at granular soil conditions. The method needs relative dry ground conditions. (Puller, 2006)

The benefits of this method are according to SGF Jordförstärkningskommitté;

- Can be used as an alternative to sheet pile walls at sites with limited space
- Easily fitted for various geometrical features and unpredicted soil conditions
- The front can be adapted to fit the surrounding

The drawbacks of this method are according to SGF Jordförstärkningskommitté:

- A minor deformation is required to mobilize the tension forces in the nails (about 0.1 % of the excavation depth)
- Ground water pressure can cause problem during construction
- Frozen ground has to be considered

3.2.8 Ruukki-Retaining walls -RR, RD and combi walls

Steel piles can be used for construction of retaining walls and in this section the steel pile walls by the Finnish manufacturer Ruukki will be presented.

Steel tube pile wall

The RD wall consists of drilled steel pipe piles, helix welded RD-piles with diameter of 400 – 800 mm and interlocks on each side, see Figure 20. The RD-walls are used as retaining walls to construct underground basements, harbor structures, road and railway structures and bridges. When horizontal and vertical forces are acting on the structure and the ground conditions are difficult this kind of retaining wall is suitable. The connection between bedrock and pile will be very reliable with RD-piles since it will be drilled into the rock with very small differences compared to the prescribed position. The retaining wall is installed by an oversized drilling crone instead of a normal sized drilling crone. The drilling is performed by centric percussive drilling with the DTH hammer technique. The RD-piles can be delivered in different steel qualities S355J2H, S440J2H, S550J2H, X60 and X70. The RD-piles are merged with welding. The RD-wall is designed according to Eurocodes or national regulations. The structure can be designed as a steel structure or as a steel concrete composite structure if the piles are filled with concrete. To secure the bearing capacity of the pile during its life length, larger material thickness is needed to compensate for the corrosion of the pile. (Rukki, 2012a) If the RD-piles are drilled into the rock the structure can resist large vertical forces without failure. The RD-wall can also be used as permanent retaining structure where it in common practice would be used as temporary retaining wall. (Ruukki, 2011b)

Figure 20 The RD- wall by Ruukki, (Ruukki, 2012a)

The combi wall

Ruukki´s combi walls consist of steel tube piles and sheet piles, see Figure 21. The RD-pile in the retaining wall has an interlocking “section” on each side of the pipe which can be connected to a row of sheet piles. This structure is often used in harbor construction since it has large bending stiffness, bearing capacity and bending resistance compared to usual sheet pile walls. The steel pipe piles in the combi wall are driven RR-piles.(Ruukki,2011a) Other areas where the combi wall is used are in permanent and temporary retaining structures, wharfs and jetties, building foundations, road and railway structures and bridge foundations. In this structure the piles are resists the load on the structure and the sheet piles makes the wall continuous. Steel qualities and design regulations are the same for combi walls and Rd-walls. (Ruukki, 2012b)

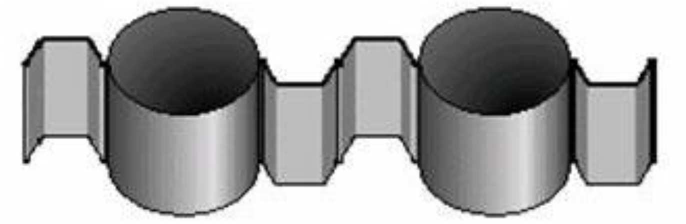


Figure 21 The combi wall by Ruukki. (Ruukki, 2012b)

3.3 Noise and vibrations caused by foundation work

In the following section tables and figures describing the noise and vibrations caused by the installation methods are presented. These figures and tables has varying sources, from literature to expert judgments.

3.3.1 Noise

In Table3 below, a summary of various noise levels (L_{eq}) from different pile installation method is presented (Langley, M.S. 1980). The different driving methods are presented in chapter 3.2.4. In this survey the measurements have been made at 45 different piling projects at a distance of 10 meters. The noise levels are presented in dBA and therefore the noise levels are A-weighted.

Table 3 Noise level for driving methods

Pile installtion method	**Typical *Leq* noise levels at 10 m in dBA**
Diesel hammer	96 - 108
Air hammer	85 - 98
Drop hammer	82 - 101
Screened drop hammer	75 - 88
Jacked system	65 - 78
Auger (crane and lorry mounted)	68 - 90
Tripod winch	76 - 90

The data from Table 4 below is taken from the report "Omgivingspåverkan vid pål- och spontslagning"(Hintze et al, 1997) which is written by the Swedish commission on pile research. The data is based on more than 230 measurements found during their literature survey. In table 4 the noise levels are presented which is a selection of the installation methods listed in the report from the commission of pile research. As seen the noisiest method is the double acting diesel hammer and the least noisiest the hydraulic jacking technique. The noise levels are presented in A-weighted equivalent sound effect mean value when more measurements are made. The notation for this is L_w, and it has the unit dBA.

Table 4 Noise level for different installation techniques for sheet piles, diaphragm wall and cast-in- place piles. (Hintze et al, 1997)

Piletype	Installation method	Noise level,Lw, dBA, mean
Sheet piles	Diesel hammer	128
Sheet piles	Diesel hammer, double acting	138
Sheet piles	Hydraulic jacking	97
Sheet piles	Vibrating	116
Diaphragm wall panels	Grab	115
Sheet piles	Drop hammer	125
Cast in place	Impact bored	108
Cast in place	Auger	114

3.3.2 Vibrations

In this section the vibrations from a number of the methods described in the thesis are rated by Fanny Deckner from NCC Stockholm. First the soil where the methods are installed in are divided into cohesive and frictional soil, see Table 5. After the methods and installation technique are presented in the column to the left. The methods are ranked from 1 which means little risk of vibration problems, 2 means moderate risk of vibration problem and 3 means great risk of vibration problems.

Table 5 Vibration estimation for different methods and two soil types cohesive and frictional soil. Estimation made by Fanny Deckner.

	Degree of generated vibrations	
Category of pile	**Cohesive soils (e.g. clay)**	**Frictional soils (e.g. sand)**
Pre-cast concrete	-	-
single-acting drop hammer	1,5	2,5
double-acting drop hammer	1,5	2,5
single-acting diesel hammer	1,5	2,5
double-acting diesel hammer	1,5	2,5
vibratory hammer	3	2
hydraulic jacking	1	1
Steel tube or H-profile	-	-
single-acting drop hammer	1	2
double-acting drop hammer	1	2
single-acting diesel hammer	1	2
double-acting diesel hammer	1	2
vibratory hammer	3	2
hydraulic jacking	1	1
Timber	-	-
single-acting drop hammer	1,5	2,5
double-acting drop hammer	1,5	2,5
single-acting diesel hammer	1,5	2,5
double-acting diesel hammer	1,5	2,5
vibratory hammer	3	2
hydraulic jacking	1	1

According to the Swedish standard (SS 025211) the vibrations can be calculated with Equation 3;

$$V = V_o * F_b * F_m * F_g \qquad (3)$$

V= The guiding value

V_o= Uncorrected frequency of oscillation in mm/s. This factor depends on the source of disturbance and the ground conditions.

F_b= Building factor

F_m= Material factor

F_g= Foundation factor

The uncorrected frequency of oscillation depends on the kind of activity, (Piling, Sheet piling, excavation or compaction), where least allowable frequency of oscillation is for clay, silt and gravel, higher for till and greatest for rock. The building factor depends on the vibration sensitivity of the building. Old historical buildings have the lowest factor and heavy structures e.g. bridges, quays and defense facilities have the highest value. The material in the structure determines the material factor, where the lowest factor belongs to limestone and the highest to reinforced concrete, steel and wood. The foundation factor is determined by the type of foundation and soil type that the structure is founded on. Structures founded on end bearing piles obtain the highest value, friction piles the intermediate and slabs the lowest. These values are valid for foundation on clay, silt, sand or gravel. If the ground conditions are stiffer same value is used as for the end bearing piles. (SS 025211)

4. THEOREM OF WELFARE

When examining the cost of various foundation methods, it is of greatest importance to not only focus on the relative cost for installing the pile or retaining structure, but to also see the costs from a broader perspective, i.e. from a national economy point of view. Therefore, in order to describe how the negative impacts can be included within a decision-process regarding foundation methods in an urban environment, this chapter presents the basic outlines for the theorem of welfare. The action taken regarding infrastructure projects is determined by potential gain of welfare (Jones, C. 2005). National road authorities and city councils strive for maximizing the value for the whole society from a certain infrastructure project. Obvious incomes taken into account are decreased travelling time, increased safety and increased capacity. However, during an infrastructure project some disturbance will occur, greatly affecting the environment where people live or work. These disturbances can be regarded as costs for the society. Therefore, decisions regarding infrastructure projects have to consider both the incomes and the costs for the society. To include both the expected incomes and the expected costs for a certain infrastructure project, a *Cost-Benefit-Analysis* (CBA) may be performed to examine the total consequences for the society. Before presenting the basic outlines of a CBA, some fundamental parameters in the theorem of welfare will be stated.

4.1 Pareto optimality

In infrastructure projects, as in all projects were reallocation of resources occur, there will be some individuals that are gained more than others. One criteria for decisions regarding the theorem of welfare is named after the Italian economic Alfredo Pareto, who in the early 20th century stated that "there is a Pareto improvement from a reallocation of resources if it makes at least one consumer better off and no others worse off" (Jones, C. 2005). At a Pareto optimum it is not possible to make one consumer better off without making at least one or more worse off. Figure 22 below gives a schematic graph over the Pareto improvement.

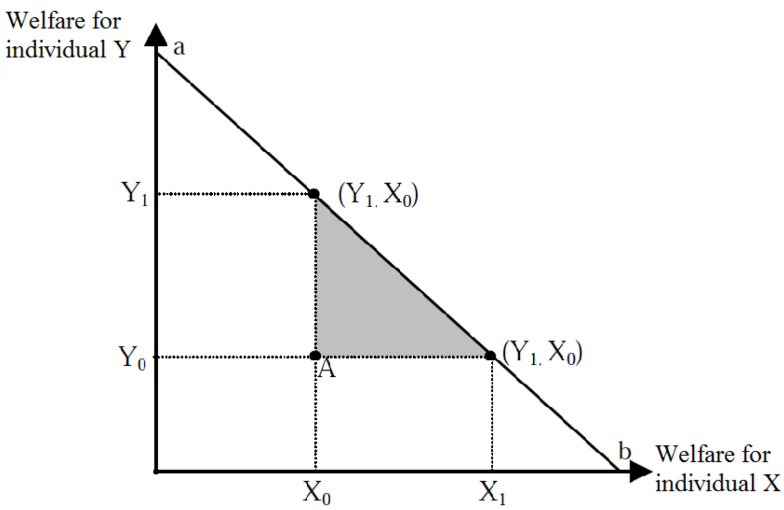


Figure 22 Schematic drawing of the Pareto criteria (Matsson, B. 1979)

Point A in Figure 22 can be regarded as the starting point, i.e. before any resources are reallocated. The welfare for individual X and Y is represented by the X and Y axes respectively. Pareto optimum is represented by the straight line between point a and b, which is a maximized societal economical state. According to the statement on Pareto improvement above, the only actions resulting in a welfare change within the grey area in Figure 22 is allowed as it will lead to a higher welfare for the society.

For an infrastructure project this could mean that individual X will receive a shorter commuting time if a certain infrastructure project is conducted whereas individual Y will not. For individual X, who will gain a decrease time spent in traffic, this certain infrastructure project will improve the welfare of the society. The project will fulfill the Pareto criteria only if individual Y will not receive a decreased welfare.

The Pareto criteria has however some limitations when it comes to decisions regarding infrastructure in an urban environment. It does not consider the fact that a project may result in a decreased welfare for a limited amount of individuals and an increased welfare for a larger group. In infrastructure projects in an urban environment there will always be some individuals that will be disturbed, both during construction and when the specific infrastructure is in use. In a market economy with unlimited competition, every single action will result in higher welfare for the society, which means that the economy can be regarded to fulfill the Pareto criteria. However, in the complex reality, market economy with unlimited competition does not exist as we have monopolies and shared gains for the society. Therefore, the Pareto efficiency had to be evolved to be more applicable for societal welfare, which was made by the two economists John Hicks and Nicholas Kaldor in the late 1930´s. (Jones, C. 2005)

4.2 Kaldor-Hicks criterion

As mentioned above, the Pareto criteria can be seen as too strict for evaluating infrastructure projects which will have a positive effect on a larger group of people. One major factor for economic growth in a region is due to the accumulation of means of production, such as infrastructure, which will make the workers more productive. Infrastructure will therefore contribute directly to the production in a certain country or region. (Burda M,. Wyplosz C. 2005)

John Hicks and Nicholas Kaldor therefore evolved the Pareto criteria, hereafter named Hicks/Kaldor-criteria. The formulation differs between the two economists, but the broad outline of the Hicks/Kaldor-criteria implies that if the monetary value of the goods exceeds the monetary value of the costs, those who profit from the action can theoretically compensate those who do not profit from the action and still have a surplus (Lindahl, H. 2001). However, the Hicks/Kaldor-criteria do not actually imply that a compensation take place, only that the monetary value of the goods is so large that compensation can occur. Similar to Figure 22, Figure 23 gives a schematic drawing of the Hicks/Kaldor-criteria.

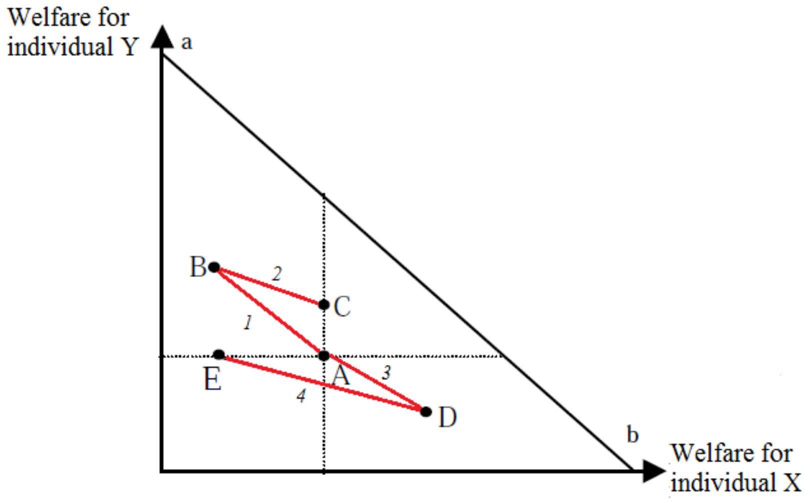


Figure 23 Schematic drawing of the Hicks/Kaldor-criteria (Matsson, B. 1979)

As for the schematic drawing in Figure 22, the welfare of individual X and Y is represented by the X-axis and Y-axis respectively. The straight line between a and b is the maximized societal economical state. The starting point is as in Figure 22 the point A. Action 1 will result in a welfare change to point B. This reallocation of resources will lead to an increased welfare for individual Y and a decreased welfare for individual X, i.e. this action will not fulfill the Pareto-criteria. Assume that individual Y theoretically can compensate individual X´s welfare loss and still have a monetary surplus. This compensation is marked with line 2, resulting at theoretical point C. As the theoretical point C is preferable to point A, this action fulfills the Pareto criteria. Therefore, the real point B is preferable over point A, so action 1 fulfills the Hicks/Kaldor-criteria as it will gain the national welfare. However, actions can also not fulfill the Hicks/Kaldor-criteria. This is represented by action 3, which will lead to a welfare change to point D. In this case individual X will receive an increased welfare and individual Y will receive a decreased welfare compare to the starting point. If individual X theoretically compensate individual Y (line 4) so individual Y will not receive a decreased welfare, point E will be reached. As the theoretical point E will result in a decreased welfare for individual X, action 3 does not fulfill the Hicks/Kaldor-criteria. Action 1 is therefore preferable to the starting point, and the starting point is preferable to action 3.

4.3 Partial analysis; infrastructure and society

An analysis of the national economy has to, due to the cost of information, be limited to those objects that is to be analyzed (Vägverket 1997). Those objects that will remain unaffected or will be negligible affected from a certain action may be removed from the analysis. Instead of performing a total analysis, which considers all factors in the national economy, a partial analysis may be performed. Suitable limitations vary from project to project and also depending on the purpose of the analysis. Many of the goods (e.g. reduced travel time) and costs (e.g. increased noise levels close to residents) in an infrastructure project lack a market price but do absolutely have a value (Mattson, B. 1979). For these situations, the value of the goods and costs are estimated on hypothetical markets.

In a market economy with unlimited competition, every resource reallocation will fulfill the Pareto criteria, i.e. a change in the market economy will occur if someone will receive an increased welfare and no one will receive a decreased welfare from that change (Vägverket 1997). However, the market economy is only a theoretical system and does not exist in the real world due to market failures. For the market of infrastructure, there are three market failures that prohibit market economy; External effects, collective goods and monopolies.

Finally, before presenting the basic outlines of a CBA, it is important to define the term *society*. A common misconception is that *society* is equivalent to the public sector or the state. However, *society* is an expression for all the citizens within a nation, both now and those in the future. To include future citizens, an infrastructure project is designed for to last for a certain period of time. The term can also be used for a limited fraction of the nation, often geographical limitations. (Lindahl, H. 2001)

4.4 Cost-Benefit analysis

To quantify and estimate the value of goods and costs for an infrastructure project, a Cost-Benefit Analysis (CBA) may be used (Lindahl, H., 2001). CBA is mainly used to prioritize and evaluate the usage of resources with regards to the consequences for the society (Vägverket 1997). Using CBA, the decision makers can determine what the goods and cost are and how these should be expressed in monetary terms. The ultimate goal for a change in the national economy is to obtain a highest possible welfare. To estimate if the welfare of a country or region will increase or decrease due to a certain action, the Hicks/Kaldor-criteria described above may be used.

The first step of a CBA is to specify the limitation, i.e. deciding which parameters should be included in the partial analysis (Mattson, B. 1979). To be able to compare different options or solutions, different alternatives has to be evolved. The Swedish Traffic Administration, Trafikverket, uses the current situation as reference when comparing the goods and costs for a certain project (Vägverket 1997). When different alternatives have been evolved, the effects for the citizens from each alternative have to be identified. The effects should then be measured in monetary terms. When the alternatives and effects are identified, the decision makers have to calculate them as goods and costs at different point of time. As an infrastructure project may take years to complete and has a relatively long life length, it is important to analyze the effects over several years. The difference between the goods and costs (expressed in present monetary terms) is called the *Net Present Value* (NPV). To prioritize different alternatives, the Swedish Road Authorities uses the *Net Present Value Quota*, as Equation 4;

$$Net\ Present\ Value\ Quota = \frac{P_vB - P_vC}{P_vC} \qquad (4)$$

Where P_vB is the expected net present value of the benefits and P_vC is the expected net present value of the costs.

To determine if a certain investment will be profitable for the society when the investment spans over a long period of time and with alternating streams of benefits and costs it is possible to show the future streams of benefits and costs as NPV. Figure 24 shows a simplified model of an investment over a certain time period, with the initial investment (C_o), the cost for maintenance (C_t) and the benefits (B_t).

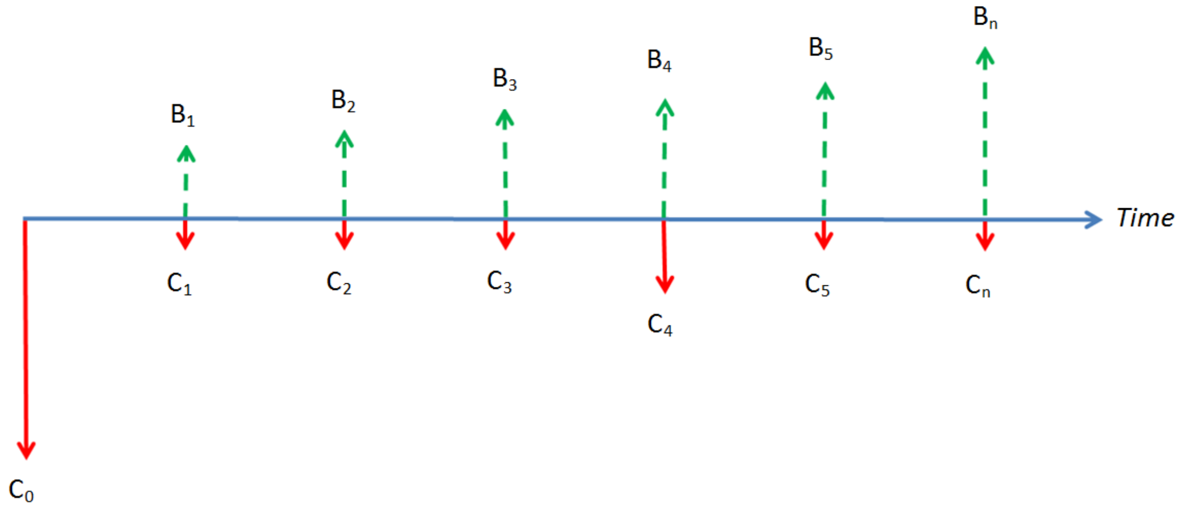


Figure 24 Costs and benefits for an investment

To determine if a certain investment will be profitable, Equation 5 may be used;

$$NPV = \sum_{t=0}^{n} \frac{B_t - C_t}{(1+r)^t} > 0 \qquad (5)$$

An investment is only profitable if the NPV exceeds 0. The discount rate, *r*, varies depending on the alternative costs and the insecurity of the project, but is usually around 4-5 % (Vägverket 1997).

Environmental effects can in some cases be impossible to value in monetary terms as its impact varies depending on each individual's conception and sensitivity. The environmental effects are therefore in Sweden mostly covered in a so called *Environmental Consequence Description*, which is mandatory for all larger infrastructure projects in Sweden. Only those impacts that can be valued in monetary terms are included in the CBA, which in Sweden are noise, vibrations and emissions. The way to calculate the cost of emissions will not be presented in this report. (Vägverket 1997)

The monetary values for noise disturbance are gathered from *Vägverkets samhällsekonomiska kalkylvärden* (publikation 2008:67). The monetary valuation of noise disturbance is presented as SEK (price level of 2006) for one person each year. Noise from construction or traffic will disturb people differently if the individual receives the noise while he/she is outdoors or indoors. Therefore, data for monetary value of noise disturbance will be presented for both outdoor conditions and indoor conditions. Using these two series of data, a combination of these to conditions is used to present a total monetary valuation of noise disturbance.

The monetary values for noise disturbance outdoors, indoors and a total valuation are presented in appendix 1-3.

Figure 25 shows a graphical representation of the values presented in appendix 1. In Figure 25 one can see that a reduction from 75 dBA to 65 dBA is worth almost six times more than a reduction from 65 dBA to 55 dBA.

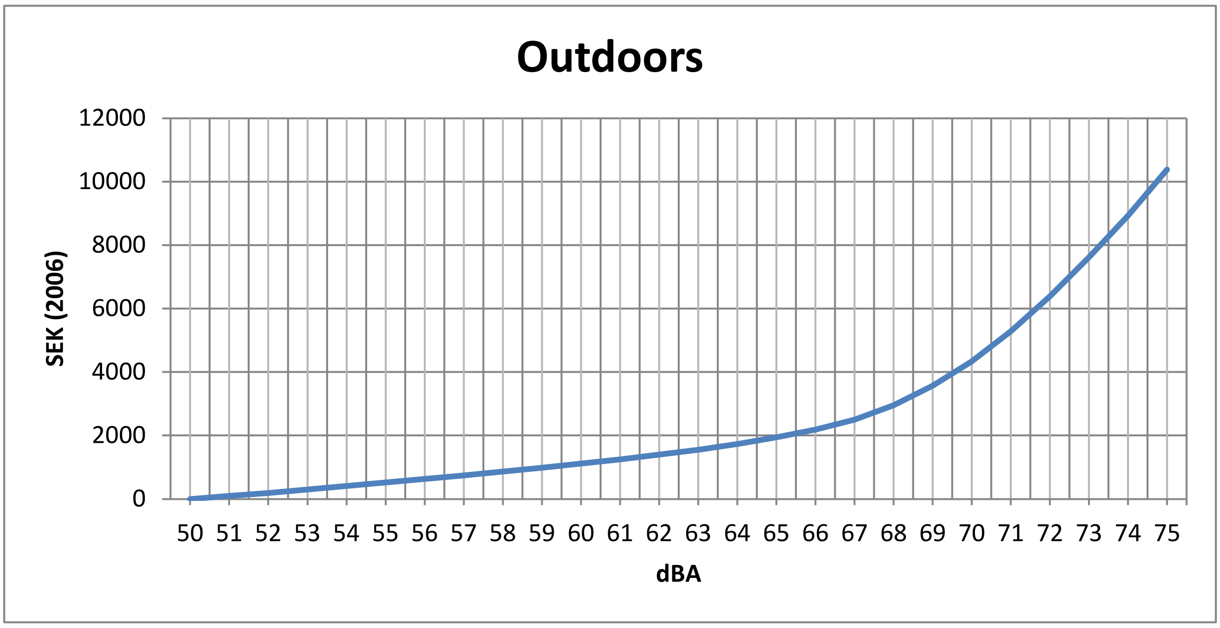


Figure 25 Graphical representation of the monetary values of noise disturbance outdoors

Figure 26 shows a graphical representation of the values presented in appendix 2. In figure 26 one can see that a reduction from 50 dBA to 40 dBA is worth almost six times more than a reduction from 40 dBA to 30 dBA.

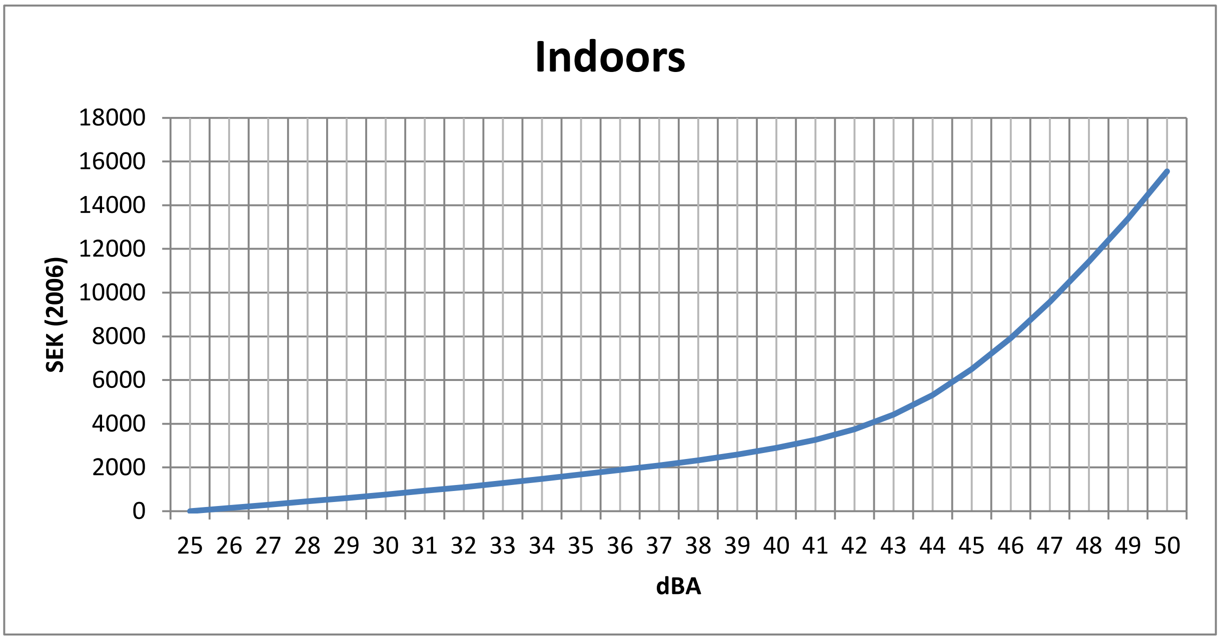


Figure 26 Graphical representation of the monetary values of noise disturbance indoors.

Figure 27 shows a graphical representation of the values presented in appendix 3. In figure 27 one can see that a reduction from 75 dBA to 65 dBA is worth almost six times more than a reduction from 65 dBA to 55 dBA.

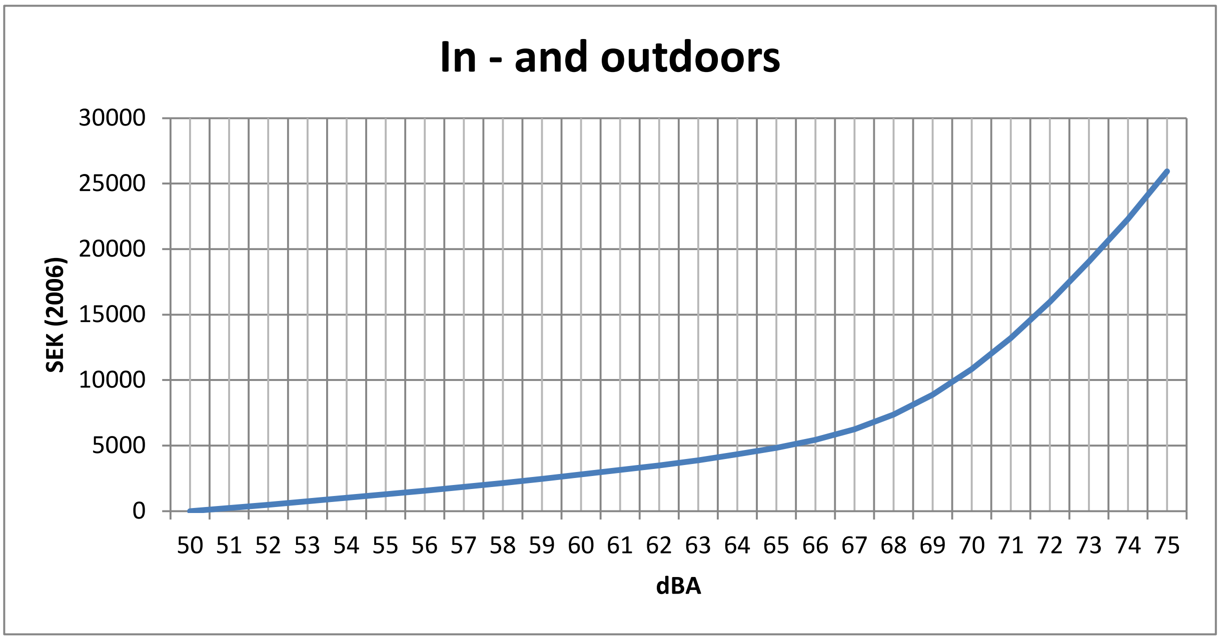


Figure 27. Graphical representation of the monetary values of a combination of noise disturbance indoors and outdoors

The way to calculate the monetary value for vibrations differs greatly compared to how the monetary value for noise is calculated. In general, the calculation implies estimating how many people will be affected by vibrations from a certain project. Each of these persons is given an absolute monetary value based on the willingness to pay. Expressed in the price level of 1993, the willingness to pay for not be affected by vibrations is 8000 SEK (Vägverket 1997)

5. GEOLOGICAL AND GEOTECHNICAL CONDITIONS

Although one certain foundation technique might be preferable regarding reduced negative impacts on the urban environment, the geological and geotechnical conditions at the site sets the limitations regarding if that certain foundation technique can be used at that site. Europe shows a great variety of geological and geotechnical conditions, as some regions are greatly affected by previous glacial periods, some regions by tectonically movements, some by massive erosion and sedimentation and some by its close adjacent to coastal shores. For that reason, the following chapter will present the main features of the geological and geotechnical conditions of eight European countries, namely; Sweden, Norway, Denmark, Germany, Poland, the Netherlands, U.K. and Spain. As the geological and geotechnical conditions usually varies greatly also within regions, the geological and geotechnical conditions of 15 cities within these countries are presented, namely; Gothenburg, Stockholm, Oslo, Trondheim, Copenhagen, Berlin, Hamburg, Warszawa, Gdansk, Wroclaw, Amsterdam, Rotterdam, London, Southampton and Barcelona. The geological and geotechnical conditions for the mentioned cities presented in this chapter will be referred to in chapter 6, *Inventory of used foundation methods*, as all projects presented in that chapter have taken place in those cities.

5.1 Geological and geotechnical conditions of Sweden

The soil geology of Sweden is determined by the last ice age. In the landscape formations such as terminal moraines, end moraines, glaciofluvial deltas and ridges, clay deposits, beach deposits etc. occurs. The highest coastline is a very important boarder since the material that is situated below this boarder has been influenced by the sea. Therefore fine material has been washed away during the wave actions and fine material has settled some distance away from the moraine and glaciofluvial deposits. Beach deposits of sand and gravel is situated where the glaciofluvial deposits and moraines have been washed by the wave actions. Peat deposits occur where old lakes have been overgrown or water-logging occurred. (Sveriges Nationalatlas, 2009)

5.1.1 Gothenburg

The soil of the western part of Sweden consists of a hilly landscape with alternating valleys and mountains, where the mountains have thin soil cover. The valleys are filled with clays and silts. The clays are stiff glacial clay and in the near coastal areas the glacial clay is overlain by postglacial clay of similar properties. These sediments in the valleys are of 50 to 100 meters thickness. Till is present along the boundary of the hills and in terminal moraines. Glaciofluvial deposits occur, mainly in form of deltas, in the valleys and are perpendicular to the terminal moraines. In the glaciofluvial deposits there may be enclosed folded clay layers. (Sveriges Nationalatlas, 2009). The marine clay in western Sweden is deposited in salt environment and has normally a shear strength of 10 kPa to 15 kPa. Most of the clays in Sweden has friction angle of 30 degrees (Sällfors, 2001). A geological map of the area is presented in Appendix 4.

5.1.2 Stockholm

In the Stockholm- Mälarregion the rock is almost always covered with soil, except for the area near the coast. The soils in this region are mainly made of glacial deposits, but post glacial and peat deposits occur. Since the sea level has been 150 meters above today´s sea level, the glacial material has been affected by the sea waves. A consequence of this is that fine sediments settled in the valleys and creates post glacial clay. Till covers about 75 to 80 percent of the Stockholm-Mälarregion. One of the most significant formations of these region are the boulder ridges with glaciofluvial material. (Nationalatlas, nd). The normal consolidated clays in the Mälarregion have normally a shear strength of 5 kPa to 10 kPa (Sällfors, 2001). A geological map of the area is presented in Appendix 4.

5.2 Geological and geotechnical conditions of Norway

The last ice age has influenced the soil cover in Norway, as it has in the rest of the Nordic region. The thickness of the glacier has been 2 to 3 km thick during its maximum. The consequence of this is that the bedrock was pressed down several hundreds of meters. When the glacier had melted the bedrock was uplifted slowly and the water from the glacier performed a sea level rise. The consequence is that there are marine deposits above today´s sea level. The rock in Norway is to a considerable extent of Precambrian age, and consists of gneisses and granites. However, in the mountain range the rocks are of sedimentary origin. The glacial ice has milled the rock and the material has been transported in tunnels in the ice or has been consolidated to hard till below the ice. The material from the tunnels in the ice has deposited in eskers with sand and gravel at the river mouth and the fine sediment, clay and silt, has sedimented beyond the esker. The clay in Norway has been deposited in a marine environment and therefore salt has been leached out from the clay structure. This process has created quick clay that is very sensitive to new loads. The thickness of the soil cover in Norway is in between of 0 to 50 meters, but in some areas in Norway the soil thickness exceeds 100m. Frequently there are soft, hard and coarse layers in the soil profile that causes irregularity in the soil profile. The depth to rock may be changed rapidly within small areas. (Holm, 1992)

5.2.1 Geology of Oslo

To construct a building, where a deep excavation is needed, in Oslo the geotechnical investigation presented the following soil profile. First there is 2 to 3 meters of dry crust clay with silt content. This dry crust clay is underlain by middle firm to wet clay with varying content of sand and silt. The 10 to 12 meters from surface is over-consolidated and the clay below is normal-consolidated. The depth to rock varies about 40-50 meters with in the construction area and the groundwater level is located 2-2,5 meters below the surface. (Bergersen and Bye, 2000). A geological map of the area is presented in Appendix 5.

5.2.2 Geology of Trondheim

When constructing a retaining wall for the hospital in Trondheim, the geotechnical investigation presented a soil profile with silt, sand, gravel and clay the first 3 to 4 meters. This was underlain by a sand layer of approximately 5 meters thickness. From about 8 to 10 meters the soil is more homogenous with silt as the main component and some embedments of sand. The bedrock is assumed to be on large depth at least 100 m. A part of the clay and silt in the upper layer are probably marine deposits and has its origin from landslides from adjacent areas or from the river side. The upper layer is also firm due to the landslide activity and differences in Nidelvas placement. The lower silt layer has probably its origin from the delta of Nidelva. The groundwater pressure is not hydrostatic and the groundwater level is located at 8 to 10 meters below surface. (Rösand, 2000). A geological map of the area is presented in Appendix 5.

5.3 Geological and geotechnical conditions of Denmark

The geology of Denmark is influenced by the glacial period when the bedrock was covered by a continental glacier 2 to 3 km thick. The maximum extent of the glacier ice was in Southern Denmark and Northern Germany. The glacial ice pressed the bedrock down and a consequence of this is the land heave in Scandinavia. However, the Danish bedrock is sinking. The bedrock in Denmark is to a big extent limestone and other sedimentary rocks. (Holm, 1992). A geological map of the area is presented in Appendix 6.

5.3.1 Geology of Copenhagen

The geology of Copenhagen consists of pre quaternary bedrock which is chalk and limestone. The thicknesses of these layers are about 1.5 km and their origin is from Maastrichtan and Danian time respectively. This area is tectonically called the Danish basin where the bottom of the basin is crystalline rock. The bedrock, which is overlain by glacial and post glacial deposits, is from Tertiary time more exactly from Danian and Selandian time. The bedrock of Copenhagen can be divided into three units, see Figure 28; the Lellinge Greensand, the Copenhagen limestone and the Bryozoan limestone, where the first two are formal units and the last is informal unit. The Bryozoan limestone is about 60 m thick and rests on the Maastrichtan white chalk formation.

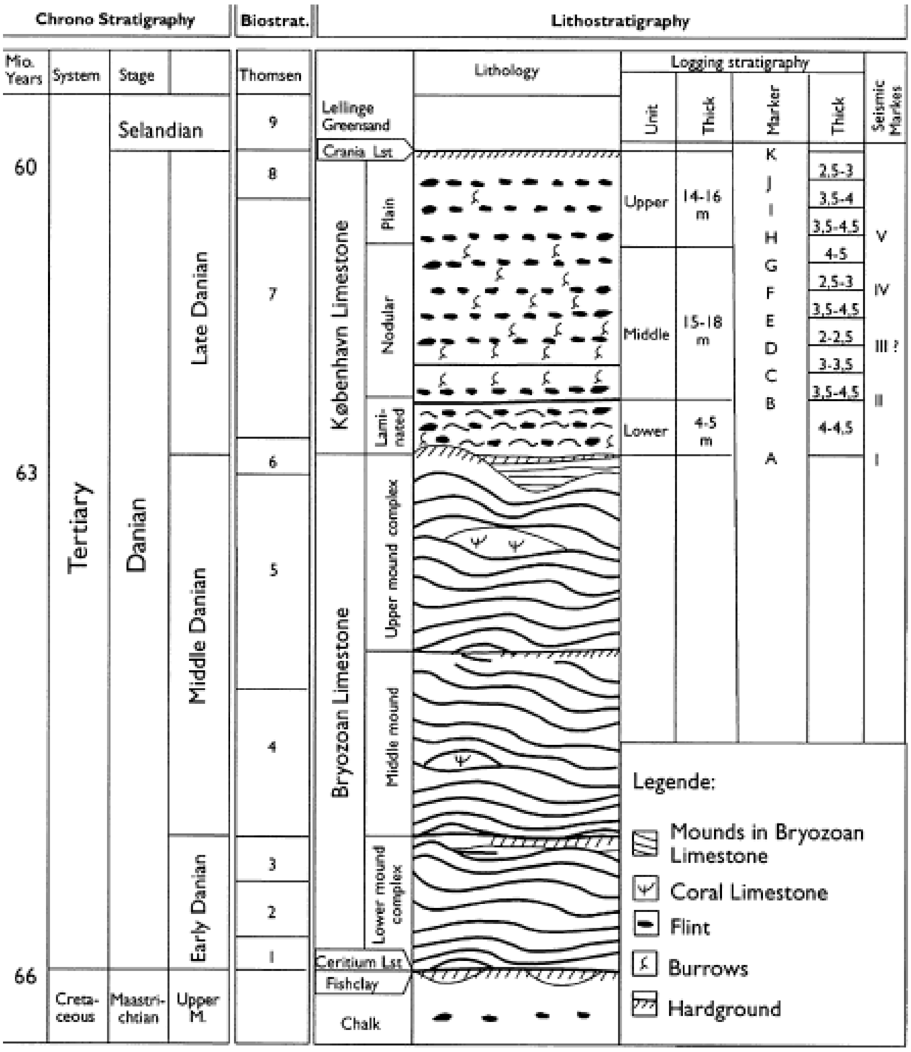


Figure 28 The rock profile of Copenhagen (Fredrikssen et al, 2003)

Quaternary deposits cover limestone and locally the Selandian Greensand over the Copenhagen area. The quaternary deposits are composed of transitional deposits which are covered by real glacial deposits. The deposits that overlie the quaternary deposits are late- and postglacial deposits. In the central Copenhagen these are covered with fill material. The uppermost layer of the limestone is fractured due to the continental ice movements. This influences the hydraulic conductivity of the limestone. Over large parts of Copenhagen there is a 1 to 2 meter thick layer of white till. The till consists of crushed limestone and granite that has been brought in by glaciers. Two different till layers occurs in the Copenhagen area, these are the till from the latest two ice ages. The lower till contains a mixture of clay till, brecciated melt water clay and parts of melt water sand. Since the lower till is harder and has high soil strength than the upper, the upper till has been deposited during the last ice age. The lower till can have shear strength of 1 MPa and be too hard to perform vane test in, therefore triaxial tests have to be performed to test this clay. It has been assumed that this hard till has been created by lime precipitation in the pores of the till.

The late and post glacial deposits in Copenhagen are peat deposits that has been flooded by the sea and covered with sediments, therefore peat can be found some meters below today´s sea level. The post glacial sediments locally occur in

Copenhagen area up to 3 meters above the sea level and consist of thin sand, which can contain some organic mud. (Fredrikssen et al, 2003)

For construction of the Tunnel Syhavn station, the following geological profile is interpreted: The first layer of fill material that varies between 1 and 8 meters in thickness. This is underlain by boulder clay with a thickness of about 5 meters. The boulder clay rests on Copenhagen limestone. (Hess, 1997)

5.4 Geological and geotechnical conditions of Germany

The southern part of Germany foothills from the Alps are extending at a level of 500 meters above sea level. In this landscape moraine formations are common, with lakes and moors. This landscape comes to an end at the Rhine graben in the West and the Danube graben area. The area north of the Danube, in Southern Germany, is called the *Mittelgebirge* which consist of hills and massifs up to the North German basin. The typical features of this landscape are wide sediment filled valleys and soft rounded hills. The North German basin continues up to the Baltic Sea and North Sea coast and covers over 50 % of the German land area. Along the coast it is covered with terminal moraines and the rest with glacial deposits. The valley along the river Rhine and the erosion channels of the other large rivers in Germany contains considerable quaternary gravel and silt deposits. The district of *Mittelgebirge* is mining district and this causes subsidence due to mining and challenges for foundation works.(Arz, 1992)

The North German basin is part of the Central European Basin System. In the German Basin, coarse sedimentary rocks have been deposited from Perm to Cretaceous (Grassmann et al, 2005).

A geological map of the area is presented in Appendix 7.

5.4.1 Geology of Berlin

For a construction of a retaining wall in the Landwehr canal in Berlin, the soil profile begins with a layer of fill material that consists of debris and low strength sand. Under the fill material, fine to medium sand with low strength in the upper part and medium strength in the lower part is present. This section has a depth of 8,4m and rests on glacial till. The groundwater level is situated 0.8 m below the water level in the canal. (Schuppener, 1997)

Prior to construction of a basement at Potsdamer platz in Berlin a site investigation was performed that presented a typical soil profile for the Center of Berlin. The first 3 meters consist of fill and this is underlain by a 6 meter thick layer of medium dense sand, medium to fine grained. Under this sand layer boulder clay of 2 -12 meter thickness is found containing independent boulders. The boulder clay is underlain of a medium dense to dense sand with medium grains of a thickness of 40 m. The ground water table is situated about 3 to 4 meters below the ground. (Triantafyllidis et al, 1997)

5.4.2 Geology of Hamburg

The geology in the Hamburg harbor area can be described as one layer of fill material about 5m thick. This layer is underlain by a layer of clay varying from some maters to 10m thick. Under the clay layer a layer of sand is present which are some meters thick. The sand layer is underlain by a varying gravel layer which varies between 0 and about 5 m thickness. Under this gravel layer a sand layer with intercalated gravel and silt is present; this layer varies between some meters and 20 m in thickness. (Gatterman et al, 2001)

5.5 Geological and geotechnical conditions of Poland

Through Poland the Tornqvist tectonic zone goes in a North West South East direction. This zone separates the East European craton and the Palerozoic Europe. Along the Tornqvist zone, during Perman and Mezosoic time, a sedimentary basin was created in the Polish lowlands, as the south eastern part of the Danish- Polish basin. The thicknesses of these sediments are approximately 7000 m. (Malolepszy, 2005)

The soil types in Poland are determined by the origin of the parent material and the terrain. From north to south the soil zones are the following: terminal moraines with poor sorted sand and clays, glacial fluvial sands, flat Periglacial zone which is interrupted by valleys created by melt water, eroded mountains covered by loess and glacial deposits and the mountain zone of Carpaty and Sudety. (European soils bureau network, 2005)

In Poland cohesive soils are tertiary and quaternary clays, boulder clays and varved clays. The present Polish soils have its configurations from the four quaternary glaciations that were reaching the Karpathian Mountains, Southern Poland, Central Poland and the Baltic Sea shore. The influences of the glaciations are detected down to about 100 m. The clays cover most of the surface area of Poland or occur in the upper most layers. The most significant is Pliocene clay in central Poland, which has a thickness of 100-160m. North of Poznan this clay has problematic swelling properties. (Stepkowska, 1979)

Loess soil occurs in a belt in the southern part of Poland and the loess covers 6% of the surface of Poland. The thickness of the loess deposit varies between 2 to 30m depending on the level of bedrock. The loess deposits are mainly created by eolian transport. (Grabrowska-Olszewska, 1988)

A geological map of the area is presented in Appendix 8.

5.5.1 Geology of Warszawa

The city of Warszawa is situated on two different geological formations. The Visula river valley (right bank part) and the post glacial upland (left bank part). At the left bank part tills are the main foundation soil when constructing. The tills were created during two glaciations (Odra and Warta). In borehole loggings there is not seen any limit between the older (Odra) and the younger (Warta) till. The Warta till is, in general, more sandy and contains embedments of clayey sand. It varies between 3-5m in thickness, but it overlies Odra till and forms a 10m thick continuous layer together. Since the Odra till is more consolidated, it has a lager shear strength and modulus compared to the Warta till. A difficulty when foundation works are performed on Warta tills is the embedded sand that conduits groundwater. This necessitates dewatering. (Dundulis, 2008)

To construct the foundation for a tall building in the center of Warsaw, the investigation presented the following soil profile was observed. The first meters are composed of fill material, sand and cohesive material from lacustrine origin. Under, two stiff to very stiff boulder clay layers are present. These boulder clay layers are underlain by very dense interglacial sand and gravel. This formation rests on firm to stiff Tertiary clays. The top of the Tertiary clay are located at a depth between 38 and 47 meters. This soil profile is what can be expected in the area of central Warsaw. (Leszczynski, 2009)

5.5.2 Geology of Gdansk

When constructing a pedestrian underpass in Gdansk the soil conditions were examined. First a layer, 1.6 to 3 meters thick, of fill material was observed. The fill consists of sand, gravel, brick, concrete rubble and humus. The fill is underlain by a layer of wet peat 0.7 to 1.2 meters thick. Under the peat an organic clay layer is present, 3.3 to 5 meters thick. This organic clay layer is mixed up with silt, peat and sand, and together with the peat creating a compressible soil unit. Below the organic clay layer a layer of fine and medium sand is present. The groundwater level varies between 0.8 and 2.8 meters below the ground surface due to the season. (Topolnicki, 1997)

5.5.3 Geology of Wroclaw

For construction of a new hotel in Wroclaw, a deep exaction was performed. The stratification of this site is as the following: First a layer of fill is present about 5 meters thick. Under the fill layer a medium to dense sand layer of 10 meter thickness occurs. The sand layer is underlain by clay. (Kitching, 2009)

5.6 Geological and geotechnical conditions of the Netherlands

The geology in the Netherlands consists of bedrock that is overlain by clays, sand and gravel sediments with a thickness up to hundreds of meters. This region of Western Europe has been very influenced by the shifts between cold ice periods and warm interglacial periods in the tertiary and quaternary periods. During warmer periods the sea raise, due to melting of ice, over the Netherlands and north of Belgium. The warmer climate allowed vegetation which prevented erosion. During the colder periods the coastline was situated far away as far as North of England. The ice created moraines and clay layers. Due to poor vegetation during cold climate, the erosion was significant. The rivers Rheine and Meuse were moved to the north several times. The sediments from the rivers; sand, clay and gravel cover parts of Holland. During other time periods the rivers from Eastern Europe and the Baltic Sea was transporting coarse material i.e. gravel to the east and north of the Netherlands. Peat formations arose in lakes created by the glacial tongue and behind ridges and costal sand dunes. (Silence, 1992)

Piling in most locations in the Netherlands will face a complicated geological pattern with a combination of extremely different environments; marine, costal, tidal flat river, lake, glacial etc. in variable cycles. (Silence, 1992)

A geological map of the area is presented in Appendix 9.

5.6.1 The geology in Amsterdam

The geology of Amsterdam consists of sediments to a depth of 800 to 1000 meters below ground level. The different sediments like sands, silts, clays and peats, come from the following environments; marine, glacial, eolinan and river. The uppermost sediment layer which is 350 meter thick two geological parts can be distinguished, the

Holocene and Pleistocene deposits. The lower and oldest Pleistocene deposits are fine grained sands and marine clays that stretch down to 250-350 meters below ground level. When the Saalian ice age had its largest extension the ice reached Amsterdam and the glacier ice dug a deep basin in the unconsolidated Pleistocene layers. In this period mostly the formation of glacial and melt water deposits occurred in the basin. When the ice melt the sea flooded the Amsterdam basin and partly filled it with marine sands and Eemclay. When the last ice age was in progress, the Netherlands had a tundra climate and the Amsterdam basin was filled with sand. The sands that were deposited during the tundra climate are very important for foundation engineering in the Amsterdam area and emphasize the end for the Pleistocene period. The Holocene peat and clay deposits were mostly formed by the sea, see Figure 29 (Herbschleb, 2004).

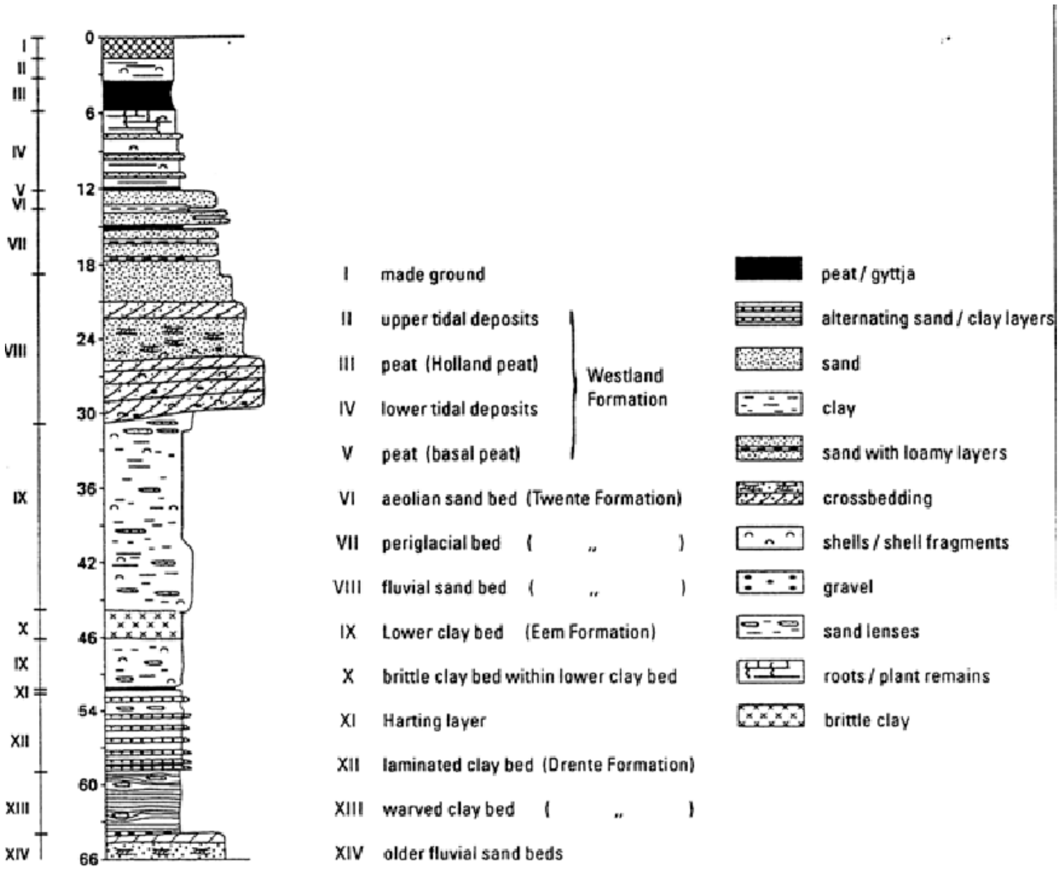


Figure 29. The soils profile the 66m upper layer in Amsterdam (Herbschleb, 2004).

5.6.2 The geology in Rotterdam

The geological conditions in Rotterdam can be described as follows: the uppermost layer is anthropogene sand, which in underlain of a soft Holocene clay and peat layers to the depth of 16 below surface level. Below this layer the Pleistocene sand layer starts and continues down to 35 meters. Under the sand layer the Kedichem formation is present. (Berkelaar and Bosch, 2004). The Kedichem formation consists of early Pleistocene fluvial clay and sand with local peat layers in the middle and south Netherlands (Huisman et al, 2000).The ground water level is situated at 2.5 meters below surface (Berkelaar and Bosch, 2004).

5.7 Geological and geotechnical conditions of the United Kingdom

The geology of the British islands has a very varying geology despite its small area. The rock geology of United Kingdom can be described as the rocks becomes younger from the North west to the South East. In the North western parts of Scotland and some other locations, Pre Cambrian and metamorphic rocks occurs. After the Cambrian, Ordovician and Silurian rocks become present at the hills and mountains of Western England and North Wales. However, the foundation problems usually start with the weaker rocks of next geological period. These rocks include Coal measures and are usual in the center of England and Scotland and the South of Wales. Furthermore, the surface rocks cross the other rocks in a South West-North East band until the Tertiary and Quaternary deposits become present in the South East of England. The soil that the foundations are constructed on varies between very strong rocks to week weathered rocks. The most common in foundation practice are the fissured clays that were created during Jurassic, Cretaceous and Tertiary time. These are the Lias, Gault and London clays. Over these rocks and old clays, since the larger part of UK has been glaciated, glacial drifts occur. These are mainly tills in the lowlands with properties that vary locally. In the North and West of United kingdom the till is granular and contains boulders, which cause problems for the piling works. In the South and East the till becomes better graded and the boulder content decreases. In some area the stone content are negligible but lenses and layers containing granular material occur. When the ice advanced during cold climate and when it retreated during warm climate, the ice created interglacial deposits that consist of laminated clays and silts. These layers are not present in a massive scale everywhere over the UK but can surprise the foundation engineer with weak layers where the soil is expected to be hard. In the south of England where the ice did not reach, the periglacial deposits may cause problems. The rivers in the UK have deposited alluvial material which contains the different rocks that have been eroded. The estuaries and bays along the cost contain weak soil which comes from different sources since the sea level has altered. This material consists of thick layer of peat from vegetation that grew on estuary sediments. Later the peat was submerged due to sea level rise. (Thornton, 1992). A geological map of the area is presented in Appendix 10.

5.7.1 Geology of London

From a borehole investigation in eastern London the following soil profile has been obtained to a depth of 57meters. The first 3 meters consist of made ground with fill material with bricks and hydrocarbon pollution. Under this formation a layer of alluvial silt 2.5 meters thick is located. This is underlain by sand and gravel (river terrace deposits) with a thickness of about 4.5 meters. Below the sand and gravel formation the London clay is located that has a thickness of 13 meters. The London clay is slightly silty, stiff, slightly fissured and thinly laminated. Under the London clay two layers of upper mottled beds are present. These beds consist of dense silty fine to medium sands. The sand beds have a total thickness of about 3 meters. Next layer in the soil profile is very stiff, fissured and silty clay (lower shelly beds) of about 8 meter thickness. Under an almost 3 meter formation of thin layers of silt, sand and silt respectively. These layers forms the lower mottled beds and the silts are very stiff and the sand very dense. Below a thin layer, less than one meter, of gravel is located and then a 2 meter layer of sand. These layers are very dense and called the Upnor foundation. The Upnor formation is underlain by the Thanet sand formation which consist of very dense, slightly silt fine sand of about 12 meter thickness. Before the chalk rock is encountered a thin layer of very dense, sandy, silty gravel is observed. (Ian farmer associates, 2010)

5.7.2 Geology of Southampton

The geology of the central area of Southampton has been created by the River Test. The quaternary deposits consist of soft silts and clays with some peat content. Thickness of these quaternary deposits varies between 3 and 12 meters and in some places plateau gravels from more recent time is present. The quaternary deposits including gravel are underlain by firm to very stiff silty clay. Under the silty clay Bracklesham beds are located. These beds consist of sands and clays to a depth of 70 to 120 meters. (Wood, 1998a)

5.8 Geological and geotechnical conditions of Spain

The geology of the Iberian peninsula can be divided into different regions: The siliceous region, the calcareous region and the clayey region. The Siliceous region consists of rocks that silica is dominating and these region is situated in the north western part of the country. In the calcareous region limestone, marl and sandstone are the dominating rocks. The clayey region consists of depressions where rivers such as Ebro, Guadalquivir and Tajo are present and here are clay and loam the dominating material. (Delgado, 1992). A geological map of the area is presented in Appendix 11.

5.8.1 Geology of Barcelona

In the Barcelona area the topography is very diverse. This is a consequence of actions during the geological time in this part of the Mediterranean. During the end of the tertiary period the sea raise over the ground that today is the plains of the Besos, Llobregat rivers and the city center. The Montjuich Mountain rose from this sea during Miocene period. The deltas from Besos and Llobregat deposited in this sea and these deltas are now called the Besos and Llobregat plains. Diluvian rainfalls made the plain that today is the ground that Barcelona is built on. Three different formations can be distinguished: The Littoral Mountain range, the Littoral platform and the deltoid formations of Llobregat and Besos. (Delgado, 1992)

The Littoral mountain range is made of granitic ground in the valley and covered by slate schists and limestones of Silur. (Delgado, 1992)

The Littoral platform; A platform that constitutes the plain of Barcelona is attached to the Sierra Collcerola, this platform is descending to a lower plain and has been submerged. This platform is composed of Pleistocene clays which are covered by Quaternary detritic (sand, gravel and silt) material. In the center of Barcelona the buildings are founded on this Quaternary material and the Pleistocene clay that appears in some places in the city. Water has formed gullies in the rock in a NW-SE direction. These gullies are filled with mainly gravel and sands but also finer material. The diluvian material is product of weathering of the rocks in the Littoral mountain range and is transported by the water to the foot of the Sierra where it accumulates. This deposit constitutes of clay with embedments of silt and fine sand. (Delgado, 1992)

The deltoid formations of Llobregat and Besos are composed of fine sands and silts. Peat is embedded between the Tertiary and the deltoid formation. Also gravel pockets exist in the deposit. The Besos delta has a thickness of 50-60m and is much smaller than the Llobregat delta. (Delgado, 1992)

A consequence of the conditions mentioned above is that two areas can be determined related to geotechnical conditions. The foot of the Littoral Mountain and the clay areas in the upper part of Barcelona are of good soil properties for shallow foundations. In the lower part of Barcelona and in the delta formations the soil properties are bad and deep foundation is needed. (Delgado, 1992)

For constructing a diaphragm wall in the deltaic soils of Barcelona, the following soil profile with five levels was interpreted: Fill material of the upper two meters and this is underlain by two meters of clay. Under the clay, a sand layer of 11 meters is present, this is underlain by 33 meters of silt. Last a gravel of undefined thickness was observed. (Garitte, 2010)

6. INVENTORY OF USED FOUNDATION METHODS

Even though some regions within the eight countries presented in the foregoing chapter might share some similar geological and geotechnical features, the type of foundation methods used may differ greatly. There may be many reasons for the great variation of foundation methods used, such as abundance or scarcity of raw material, regional and national regulations as well as accustomedness or positive experience of a certain type of foundation method. The following chapter presents the composition of the piling methods and retaining structures most frequently used in seven of the eight countries (no description of the most frequently used methods in Poland were found), and 1 – 4 more specific cases of foundation projects in each of the 15 previously mentioned cities.

6.1 Sweden

Driven pre-cast concrete piles represent the majority of the total length of piles installed in Sweden (Holm, G. 1992). In year 2010, about 63 % of the total length of piles installed were driven pre-cast concrete piles (Pålkommissionen, 2011). Driven steel piles (all sorts of cross-sectional areas) represent about 25 % of the total length of piles installed, timber piles about 8 % and drilled steel tube piles about 4 %. Both in year 2009 and 2010, no cast-in-place piles were reported to the Commission on Pile Research, indicating the absence of this technique in Sweden. During the 1980's, about 1 % of all piles installed in Sweden were cast-in-situ piles (Holm, G. 1992). The development of the total length of piles and the percentage of pile material used in Sweden is presented in Figure 30. During 2010, about 32 % of the total length of piles was for infrastructure projects, about 52 % was for housing projects and about 16 % was for industry projects (Pålkommissionen, 2011).

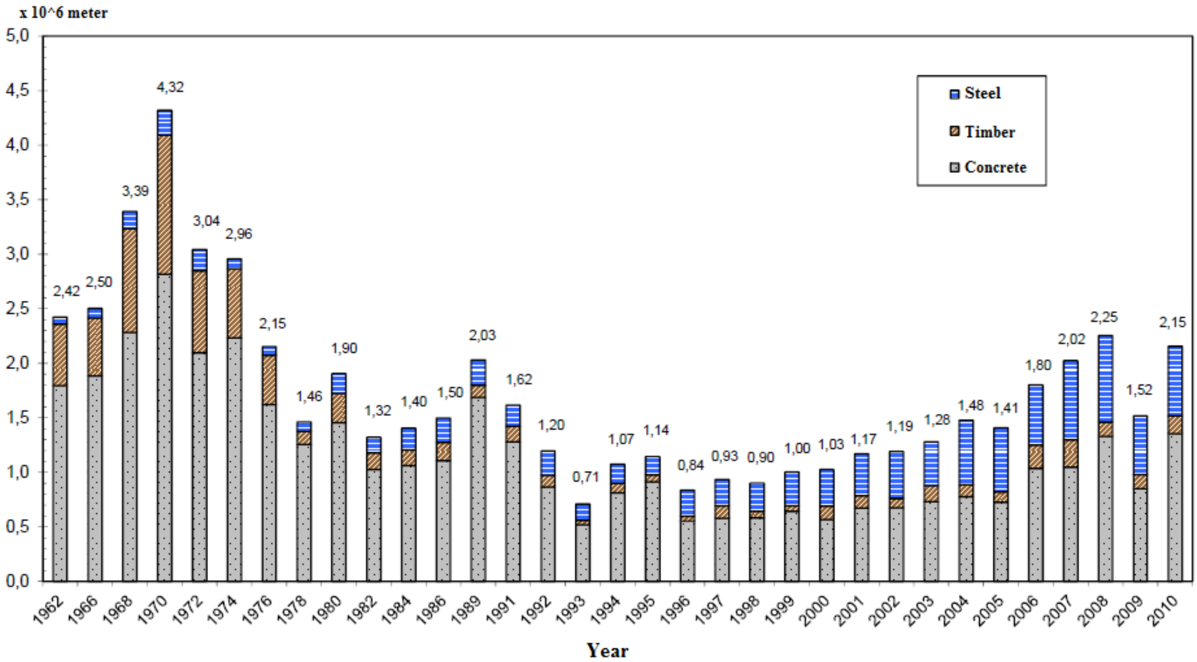


Figure 30 Total length of piles installed in Sweden 1962 – 2010 (Pålkommissionen, 2011. Edited by Ulvås, E., 2012)

The driven pre-cast concrete piles used in Sweden are produced in controlled pile factories and driven mainly by free falling drop hammers. During production, about 25 % of all piles are quality tested by stress wave measurements. The same stress wave measurement is conducted for about 29 % of all pile projects, and analyzed using software such as *CAPWAP*. This type of active design method can result in shorter pile lengths and reduced driving times. During the last few decades, the usage of hydraulic free falling drop hammer has increased, replacing the wire-guided drop hammer. The hydraulic free falling drop hammers are more efficient regarding driving, as these produce more reproducible strokes. The use of steel tube piles, both small and large diameter, has increased during the last two decades. Where special requirements regarding capacities are needed, the use of steel core piles has increased during the last decades. (Holm, G. 1992)

6.1.1 Gothenburg

In the Götatunnel project in Gothenburg, the western part of the tunnel was constructed in soil. The soil in this area consisted of clay and a friction material of varying thickness of 0 and 2 meters on top of bedrock, see chapter 5.1.1. The depth of the excavation is on this part of the project 12 meters and sheet piling is used as retaining structure. The excavation was excavated in sequences and wales and anchors were installed continuously. Three levels of wales and anchors were used and these anchors were anchored in the bedrock. The sheet pile wall was driven into bedrock and the end of the wall was sealed so groundwater could not seep into the excavation. The groundwater pressure was decreased since the safety against hydraulic uplift had to be improved. One of the challenges in this project was to predict the deformation in the sheet pile wall, since the deformations create settlements in the soil around and subsequently settlements in the surrounding structures. (Kullingsjö, 2004)

6.1.2 Stockholm

In Stockholm, the road project northern link is during construction where one part of the project will be a double concrete tunnel. The biggest challenges with this project are to not disturb the traffic too much and reduce groundwater drawdown. In the first design the tunnels should be constructed with the cut and cover method, but the contractor proposed a solution with secant pile wall, performed with the top down method. The secant pile wall would reduce the risk of groundwater drawdown and reduce the traffic disturbances. For the two tunnels, four secant walls was needed and the number of anchor levels varied between two and one depending on the height of the secant wall. The secant wall was also anchored with a foot beam between the rock and wall. The wall was constructed to the groundwater level so the groundwater should not be influenced. When constructing, different complications occurred. One of these was that one casing was left by accident during casting of a primary pile. The consequence of this was that the secondary pile could not be casted, and therefore the overlap and water-tightness could not be secured. The solution to this problem was grouting in the soil outside the secant wall and chemical grouting and sprayed concrete inside the wall. As a permanent solution, a concrete wall was casted inside the pile wall. Another observation during construction was that the bedrock level deviated a lot from the expected and four piles were installed on top of a boulder. To handle this problem grouting and casting of a concrete plate was used, since the boulder was situated on a significant depth. An anchor was also failed during tensioning and another anchor was installed below the failed one. (Söderberg, 2012)

6.2 Norway

As for all Scandinavian countries, piling in Norway is mainly performed using driven pre-cast concrete piles. The most commonly used driving method is using free falling drop hammers. However, as in Sweden, there is a growing usage of steel pipe piles, both with small diameters (60 – 70 mm) and larger diameters (500 – 800 mm). The total length of piles installed in Norway is about a quarter of the length installed in Sweden, resulting in about 0.4 million meters per year. The market share for pre-cast driven piles is about 75 % in Norway. (Holm, G. 1992)

However, Norway is a country that shows extremely varying geological conditions due to its topography, where sites with thick layers of clay (> 50 meter) can be in close adjacent to sites with outcrops of firm bedrock. Therefore, the usage of steel-core piles that are drilled into the firm bedrock has increased during the last decades where special requirements are demanded. Two projects from Norway will be presented in this chapter; One from Oslo and one from Trondheim. The similarity between these two projects is that they are both situated in close adjacent to existing buildings and requires a soil retaining structure.

6.2.1 Oslo

At the spring of 2000, the construction of a new 18-storey office building started in central Oslo. The building site was about 22 000 m^2 and contain two basement floors, requiring an excavation of 5.5 – 7 meter for the whole building site. As the building site was in close adjacent to the transit route Sørkedalsveien and to the Colosseum Cinema, the largest cinema in northern Europe, high demands was put on the retaining structures. (Bergersen, N., Bye, A. 2000).

A geological description of the area is presented in chapter 5.2. To design the retaining structures with sufficient over-all stability in the central Oslo area, with about 40 – 50 meter of clay down to firm rock, the contractor NCC had to come up with innovative solutions. Installing a sheet pile wall down to the firm rock was rejected as it would require too large dimensions and too many braces. A so called floating sheet pile wall was selected, which was nailed to the firm rock with tie-back anchors to bedrock, see Figure 31. To obtain sufficient over-all stability, lime-cement columns were used in a reinforcement-pattern to stabilize the otherwise weak clay within the excavation. As the site earlier was occupied by a high-rise building, existing piles was included in reinforcement-pattern. The excavation was also executed in sections. (Bergersen, N., Bye, A. 2000)

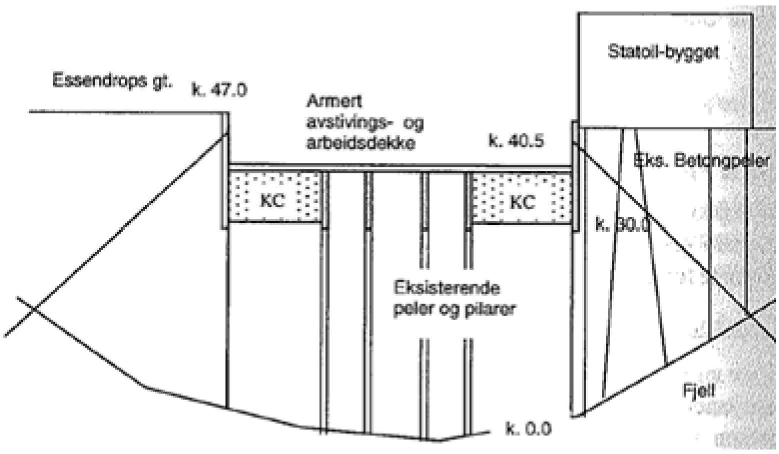


Figure 31. Schematic drawing of the excavation (Bergersen, N., Bye, A. 2000)

To minimize the deformations in the sheet pile wall, some actions were taken. One of the most critical stages in the construction regarding deformation was the installation of the lime-cement columns, so those were installed in good time before excavation took place as the strength in the columns increase over time. The distance from the sheet pile wall and the lime-cement columns were minimized as the zone between these two were expected to deform extensively. Towards the Colosseum Cinema, tensioned tie-back anchors were installed from the sheet pile wall to the firm bedrock before the lime-cement columns were installed. Deformations in the sheet pile wall were measured at four points. These measurements showed that a large part of the deformation was caused by the installation of the lime-cement columns (about 20-30 mm), while the actual excavation caused deformation in the range of about 40 mm. The largest deformation of the sheet pile wall was measured to 150 mm. Measurement of the surrounding settlement was also performed, showing that only minor settlements occurred (less than 5-10 mm). (Bergersen, N., Bye, A. 2000)

The superstructure of the 18-storey office building was constructed on 131 steel-core piles with a diameter of 120 – 150 mm, with a total piling-length of 5350 meter. The steel-core piles were installed into the bedrock. Horizontal loads, such as wind, are to be handled by raked piles and the friction beneath the lowest basement floor and the soil. Only the steel-core was assumed to transfer the load, as the casing will suffer from corrosion during the life-length of the building. Negative skin friction was assumed to only affect the casing and not the steel-core. (Bergersen, N., Bye, A. 2000)

6.2.2 Trondheim

One of the largest on-going construction projects in Norway is the development of the *RiT*-hospital in Trondheim, which has been under construction for about 12 years. Site conditions are presented at chapter 5.2. The construction has been taken place within the hospital area, adjacent to existing buildings, see Figure 32. As the surrounding buildings are in use, with hospital-equipment, patients and hospital-personnel, the requirements regarding noise and vibrations were strict for the contractors. (Rösand, R.H. 2000)

The first stage in the development was to construct a new garage. In order to construct one parking-level below existing ground level, retaining structures were needed. A silent and vibration-free installation method was needed. The technique selected for this project was jacking of sheet piles, a technique never used before in Norway. Equipment used was the hydraulic-push-unit Silent Piler ZP-150 and sheet piles with Z-sections. The Silent Piler ZP-150 can produce 150 ton in downward force. However, according to Rösand (2000), only 50 – 60 ton was used during normal production. The excavation depth was 2 – 2.5 meter for wall1 (Spunt 1) and 4 – 4.5 meter for wall 2 (Spunt 2). The sheet piles had a length of 9 – 10.5 meters. About 1500 m^2 of sheet pile walls were needed to retain the surrounding soil. (Rösand, R.H. 2000)

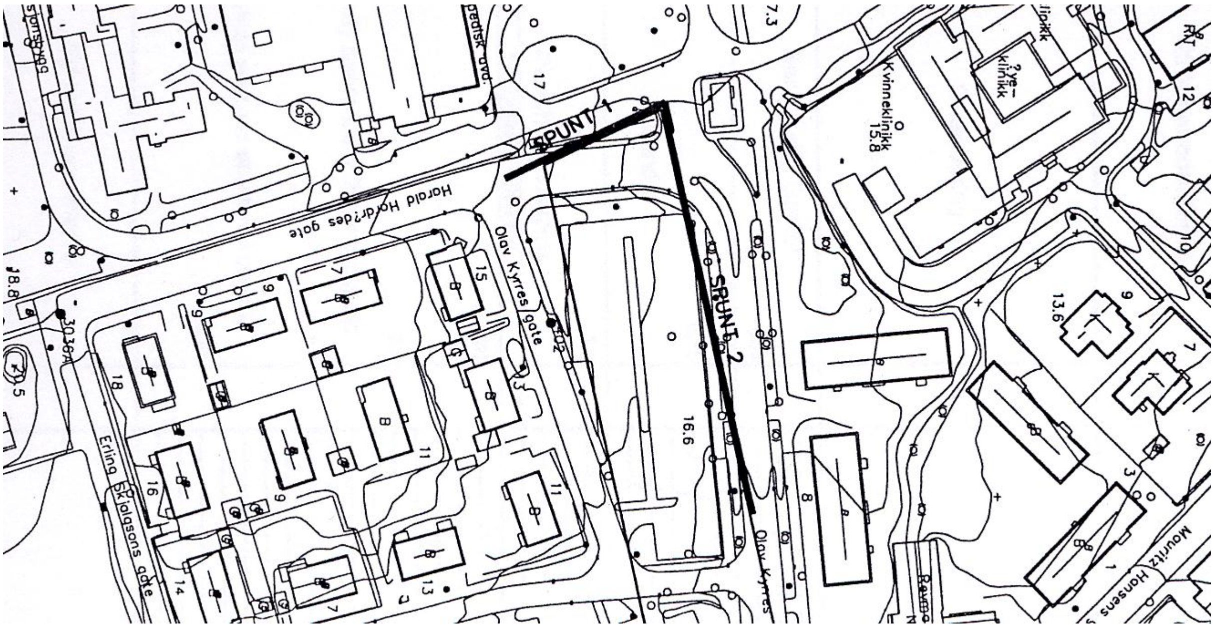


Figure 32. Overview of the hospital area with wall 1 (SPUNT 1) and wall 2 (SPUNT 2) (Rösand, R.H. 2000).

The installation of wall 1 (Spunt 1), which were about 500 m^2, was complicated by boulders occurring in the soil. These boulders were expected as the adjacent river *Nidelva* historically was present at the site where the hospital today is located. This resulted in reduced production rate. Wall 2 (Spunt 2) was however installed without any major complication, and a production rate of 150 m^2/day was achieved. Positive experiences from this project are according to Rösand, R.H. (2000) that a minimum noise level was achieved, no vibrations were produced, good precision during driving and the opportunity to retrieve sheet piles if needed. This method was however more expensive than traditional sheet piling with drop hammers, approximately 20 % more expensive. (Rösand, R.H. 2000)

6.3 Denmark

In Denmark precast reinforced concrete piles are the dominating pile type. The market share varies between 75 to 95 percent. Annually, a total pile length of about 0.5 million meters is installed. In Denmark, as in the other Nordic countries, the interest for steel pipe piles is increasing. Cast-in-situ piles are rarely used in the Scandinavian countries. (Holm, 1992)

6.3.1 Copenhagen

For construction of the "Tunnel sydhavn station", a ramp structure of sheet pile walls was used. The sheet piles are PU 32 piles and the sheet pile wall is supported with ground anchors in two levels, where the upper are of 18 meters length and the lower of 12 meters length. The upper ground anchor was pre-stressed to 550 kN and the lower anchor level to 450 kN. The pre-stressing was about 125% of the design load to minimize the deflection and fatigue load of the sheet pile wall. (Hess et al, 1997)

The trains had to overpass the construction area and therefore a temporary cover had to be constructed. This consisted of steel plates resting on Berliner walls with profiles every 1.2 meter. To create a wall structure between the profiles, steel plates were used the upper 3 meters. The steel plates were pressed down in connection with the steel profile driving. The rest of the wall was constructed by shotcrete arches during excavation. To handle the noise problem, holes were bored for the steel profiles to the excavation level. (Hess et al, 1997)

For reducing the settlements behind the wall, the structure was performed in a way that no voids came up behind the wall. Calculation had to prove that the deflection on the 3 meter upper part of the wall not exceeded 10 mm and not exceeded 50 mm for the lower part. For the railway yard a settlement of 30 mm were obtained from the design. The settlements were also monitored at the rail yard, since settlements was upcoming due to the driving of steel profiles and deformation in retaining wall. This was because the rails could not withstand too large differential settlements. For a point on a track, about 2 meters from the edge, the settlement was 55 mm. (Hess et al, 1997)

Also the noise and vibration levels had to be controlled in this project. The noise limits were set according to Table 6.

Table 6 The limit values for noise in the "Tunnel sydhavn station" (Hess et al, 1997)

Limit values	***Near housing***	***Near offices***
Monday-Friday 7-18	70 dB(A)	75 dB(A)
Other time	40 dB(A)	75 dB(A)
Max values during night	55 dB(A)	

The noise regulations are prescribed as equivalent and adjusted noise levels. For the daytime Monday -Friday period the noisiest eight hour period should be complied. For the evening the noisiest hour should be applied and for the night the noisiest half hour. The vibration speed should not be greater than 3 mm/s on the surrounding premises. During piling in the construction area the noise levels were measured to the following: (Hess et al, 1997)

Hammer without silencer	95 dB
Hammer with silencer	90 dB
Effect from noise screen	-8 dB

For housing about 100 meters away the noise was calculated to 64 dB, which is higher than 40 dB prescribed in Table 6.

6.4 Germany

Due to its size and its varying geological features, Germany shows different main piling techniques depending on which region is studied. In the very south part of Germany, adjacent to the border to Switzerland and Austria, many lakes are present, such as the Lake Constance. In the area around these lakes, bored piles using rotary boring rigs are employed in the majority of cases. These bored piles are usually cased piles with the casing removed during concreting of the pile shaft. As the lacustrine clay in the adjacent to the lakes has very poor bearing capacity, pile lengths up to 70 meters are needed. Similar types of piles are also frequently used in the quaternary deposits along the Rhine Graben, although with shorter pile lengths as the bearing capacity is higher in the gravel deposits. Occasionally, driven cast-in-situ piles are used in areas where residential housing is not present. CFA-piles are being used in this area more frequently. The Mittelgebirge area, where coal and iron mining has been performed in the subsoil for the last century, piled foundations produced by rotary boring rigs are used to handle the shearing forces produced by the ongoing subsidence in the area. (Arz, 1992)

Driven concrete piles, both pre-cast and cast-in-situ, are frequently used in the northern coastal areas. However, due to tougher environmental regulations concerning noise and vibrations, the usage of driven piles in close adjacent to residential areas has decreased during the last few decades. Driven piles are although used where polluted soils are expected as these types of piles displace the soil. As the displacement does not generate any spoil, there is no need to handle the polluted soil. (Arz, 1992)

Another effect of the tougher regulations regarding noise and vibrations is the increased usage of large-diameter bored piles. The large-diameter bored piles also have better bending moment, bearing capacity, making them suitable for superstructures with dynamic horizontal loads. The large-diameter bored piles are usually installed using casing oscillators and hammer grab excavation. The steel casing is in general only used as temporary support for the soil and is extracted during concreting. About half of the total pile-length of bored piles is produced for pile walls, such as secant pile walls. Support by bentonite slurry is rarely used. Also the small diameter bore pile and CFA-pile has gained increased demand as environmental regulations have become stricter. Small-diameter ductile iron piles are sporadically used in Germany, although with great success. Other developments within piling in Germany include piles with prestressed steel core, skin or base grouted piles and Vibro concrete columns.(Arz, 1992)

6.4.1 Berlin

In a canal in central Berlin, a wall that stabilizes the slope needs extra support due to increased traffic in the canal. To support the canal slope, a sheet pile retaining wall was vibrated into the canal bed ahead of the existing retaining wall. Before the sheet pile wall was performed, different driving methods were evaluated: Pressing the sheet piles, vibrate the sheet piles and ramming the sheet piles. The cost of pressing the sheet piles were the largest and ramming is almost unexceptional forbidden in central parts of Berlin. The sheet piles used for a test wall were Larsen 24 with 9 meters length that was vibrated from a pontoon. The first sheet piles were placed with a PE 2001 vibrator which then followed by five double sheet piles placed with ICE 1423 vibrator. After the test wall was vibrated, deformations were recorded and the result was that the deformations were in the acceptable limit. Therefore the 720 meter sheet pile wall along the canal slope was performed with vibrating. (Schuppener, 1997)

At Potsdamer Platz in Berlin, Daimler Bentz constructed a high raise building with basement. The wall used for the basement excavation was a diaphragm wall, thickness 1.2 meters, with support of double ground anchors and the basement slab. Ground anchors are resisting a tension force of 1000 kN and the spacing varies between 0.6-0.9 meters. The diaphragm wall had a depth of 30 meters and the excavation depth varied between 18 -22 meters. Since permeable soil layers are present in the soil profile and the groundwater needs protection, a U-shaped structure was proposed. After excavation, an underwater concrete slab was casted and the slab is supported by tension piles to prevent uplift. The tension piles used were vibrated grouted H-beams with length between 19- 24 meters. (Triantafyllidis, 1997)

6.4.2 Hamburg

In the port of Hamburg a new quay was constructed around the turn of the millennium. The first piles were sheet piles that was placed in a slurry trench at level -25.3 meters With subsequent driving, the piles are driven down to -29.70 meters and the king piles obtain more loading capacity. The king piles were a system of two HZ 975B beams with a length of 32.6 meter. Around the king piles infill sheets (2x AZ $18^{+0.5}$) were installed as earth retaining and load transfer and these are shorter than the king piles. The solution of a HZ-AZ combined wall was proved to be an economical solution. The superstructure is supported by bored piles and raking piles are used as anchors. Raking piles are composed of HTM 600/136 beams that are driven in an angle of 1:1.3. The quay is ended by a sheet pile wall of PU 12. Fender piles of 1219.2 mm diameter X 16 steel piles, were installed in front of the king piles to prevent erosion of the sea bottom.(Gatterman et al, 2001). Figure 33 shows the quay structure in Hamburg.

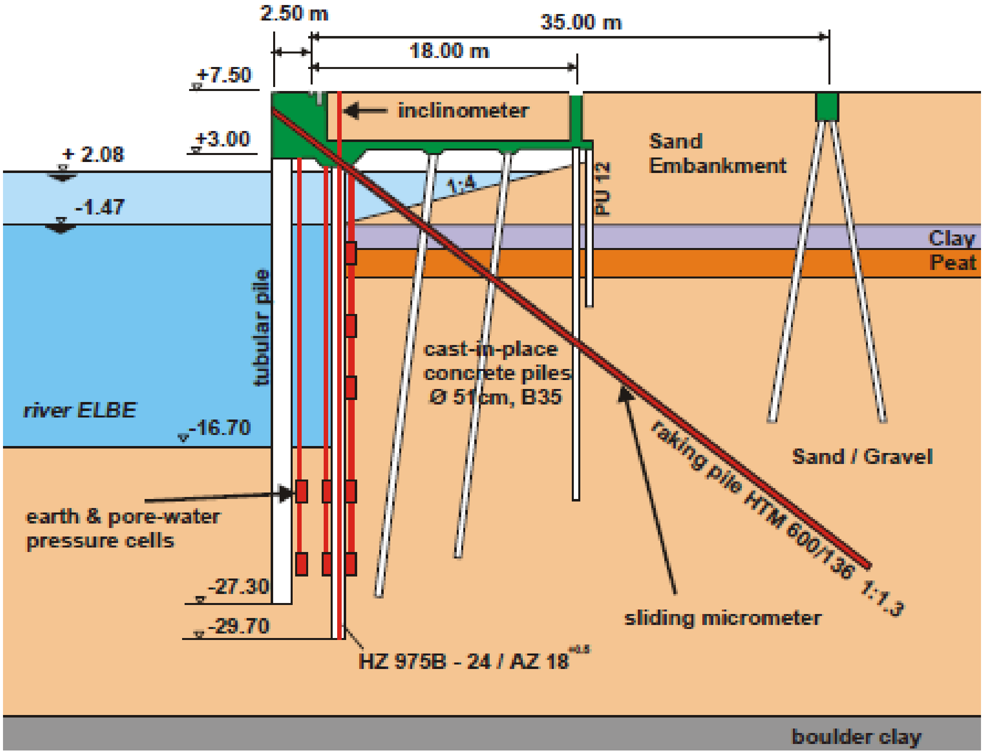


Figure 33. The quay structure in port of Hamburg with the different pile types (Gatterman et al, 2001)

6.5 Poland

During our research, general information about the most used foundation techniques in Poland have not been found. Therefore, the following examples will describe what kind of foundation methods that have been used in specific projects. These projects are retaining wall for two stations on the Warsaw underground, vibrated concrete columns for supporting an underpass in Gdansk and king post wall and diaphragm wall for constructing a basement to a hotel in Wroclaw.

6.5.1 Warszawa

When two stations on the Warsaw underground where constructed diaphragm walls were used at both places. The stations are called A13 (Centrum) and A14

(Swietokrzyka) were constructed in the city center of Warsaw with difficult geotechnical conditions. Since the excavation works influences the surrounding buildings, the movements in the diaphragm wall were constantly monitored. (Sieminska-Lewandowska, 2005).

A13 Metro station

The excavation depth for the A13 station was 17.5 meters. The upper 4 meters of the retaining wall for the A13 station consisted of a soldier wall and the lower part was composed of 22 meter high and 0.80 meter thick diaphragm wall. The soldier wall was anchored with one ground anchor and the diaphragm wall with four ground anchors. The anchor forces varied between 500 kN and 600 kN and the anchor length between 25 and 15 meters. The groundwater level was lowered during the excavation to secure the temporary stability of the bottom of excavation pit. (Sieminska-Lewandowska, 2005)

A14 Metro station

The excavation depth for the station A14 was about 14.5 meters. The retaining wall at the A14 station consisted entirely of a diaphragm wall with a height of 20.7 meters and a thickness of 0.80 meters. Two ground anchors and one steel strut were used to obtain stability of the diaphragm wall. The anchor forces varied between 500kN and 600kN and the length between 20 and 19 meters. Two groundwater levels were observed. One that is located 4 meters below ground level and the other, which is an artesian water level, where the head is located 7 meters above the excavation bottom. The groundwater level was lowered to 0.5 meter below the excavation bottom by pumping. (Sieminska-Lewandowska, 2005)

6.5.2 Gdansk

In Gdansk, an underpass of three lane highway and tramway has been constructed during the mid- 90's. At the same place, the highway and tramway will need support since it rests on compressible soils, in this case peat and clays. The support employed for the highway/tramway was a geogrid reinforced embankment where the loads is transferred to Vibro concrete columns (VCC) and further down to the sand layer that underlain the peat and clay layers (Topolniki, 1997)

The underpass is made of a concrete tunnel 46 meters long and 7.2 meters wide. The tunnel was first designed to be supported by two rows of cast in place concrete piles (Wolfsholz pile), one under each side of the tunnel. In the original design, excavation in open pit and dewatering were prescribed. The dewatering scheme was planned to be performed with four 500 diameter deep wells. The drainage would depress the groundwater level with 4 meters at the most central part of the excavation and a depression radius of 96 meters would be the result. These consequences would compose a risk to neighboring buildings that is supported on wooden piles and beams. The solution in the original design was to recharge the dewatered water with an reinfiltration system. To overcome the high costs the dewatering cause, a method with Vibro concrete columns as vertical pulled anchors to resist uplift were applied. The Vibrated concrete columns were constructed in the soil before excavation and finished at future maximum excavation level. The columns that were installed along the excavation pit side were casted to a level of 2 meters above the maximum excavation level, to perform a retaining wall. The organic clay layer, where the columns were installed, was assumed to have a low hydraulic conductivity and therefore reduce the groundwater inflow into the excavation. At each end of the tunnel the vibrated concrete columns were reinforced to resist the pulling force from the water uplift. Under the tunnel three rows were installed with a length of 3.5 meters and the spacing of 1.6 meters to support the tunnel slab. At the sides of the tunnel two rows of longer columns were installed to take a larger part of the load. These columns were installed to a varying length in the bearing sand layer but at least 1 meter. The last two rows of columns were used as retaining walls. During construction the tunnel bottom was stable, however at two locations the water inflow was large, therefore surface pumps was installed. (Topolniki, 1997)

6.5.3 Wroclaw

To construct a new hotel in Wroclaw, a retaining structure consisting of both king post wall and diaphragm wall were used. The upper part of the retaining wall was the king post wall and the lower part was the diaphragm part. An L-shaped reinforced concrete beam was casted between the king post wall and diaphragm wall to perform interaction between the walls. Due to the archeological investigation the diaphragm wall could not be built within the site boundary and therefore the king post wall was used. The king post wall is made of 10 meters long steel columns which are inserted 5 meters in the made ground, and 900 mm behind the site boundary. Between the steel beams timber waling are inserted. Ground anchors are installed to create stability and steel elements to create stiffness. The groundwater level at this site was assumed to be at 5 meters below surface, but when the 5 meter fill layer was removed, the groundwater level was observed to be much lower. The 600 mm wide reinforced diaphragm wall is about 17 meters deep. It passes through medium to dense sand layer of 10 meter thickness underlain by the fill and then continuous into the clay layer below the sand. One of the purposes with the diaphragm wall is to stop the water flow in the sand layer. The diaphragm wall has been constructed by conventional technique with bentonite support and it will be used as basement wall. To perform stability the diaphragm wall will be anchored with ground anchors and be water tight since the basement is designed to be water tight. (Kitching, 2009)

6.6 The Netherlands

In the Benelux area, a larger number of piling techniques exists due to the varying soil conditions existing in this region. Driven pre-cast piles have lost market shares to augered and bored piles. This because of new stricter noise and vibration regulations in urban areas and more efficient machinery for installing bored piles. However, the production technique for the pre-cast piles was improved and they could reclaim some of the market share. Practically all pile types are available in the Benelux region. Among these pile systems are driven piles, jacked piles, screwed piles, CFA-piles or bored piles with auger or bucket (Silence, 1992). CFA-pile walls are used in the Netherlands to construct walls for deep excavation. In the paper *Risks related to CFA-walls* (Korff et al, 2007) 22 projects are listed which were made during the period of 1992-2006. Most of these projects are performed in sand or gravel soil. The main problems with these projects were leakage trough the wall. The leakage starts due to bad verticality of the piles which cause too little overlap between the secant piles. Another problem with too little overlap is that cracks in the concrete between the piles may occur and these will conduct water. (Korff et al, 2007).

6.6.1 Amsterdam

Two restrictions were determined before the construction of a new subway-line in central Amsterdam began; there should be no notable damage to historic buildings and the disturbance to the city life had to be limited. To construct some of the stations for the North/South Metro line in Amsterdam, diaphragm walls are used. These diaphragm walls are constructed to a depth of 40 meters below street level. The excavation was performed to a depth of 30 meters in soft soil and high groundwater levels. The handle of the impact on historical buildings was a great challenge during the design phase since the distance between the stations and the buildings are less than 3 meters locally. (Herbschleb, 2004)

Ceintuurbaan station

To construct the Ceintuurbaan station in Amsterdam, see Figure 34, a 45 meters deep diaphragm wall was constructed. Since strict requirements of settlements were determined, therefore the deformations in the excavation pit had to be minimized. The historic buildings in the center of Amsterdam are founded on wooden piles with questionable capacity. These two problems resulted in four permanent strut levels and two temporary strut levels. The struts consist of pre-stressed struts, a floor slab and a jet grout strut.(Buykx et al, 2009)

Description	γ_{sat} [kN/m^3]	φ' [°]	c' [kPa]	$E'_{50;ref}$ [kPa]	OCR [-]
Holocene layers, soft PEAT / CLAY / SAND	15	28	3	8000	n/a
1st & 2nd sand layer, (medium) dense SAND	19	34	0	34000	n/a
Eem layer, overconsol. marine firm CLAY	18	32	15	13000	2,0
Intermediate layer, medium SAND	19.5	33	0	25000	n/a
Drente layer, overcons. glacial firm CLAY	19.3	33	11	15000	1,5
3rd sand layer, dense SAND	19.5	35	0	35000	n/a

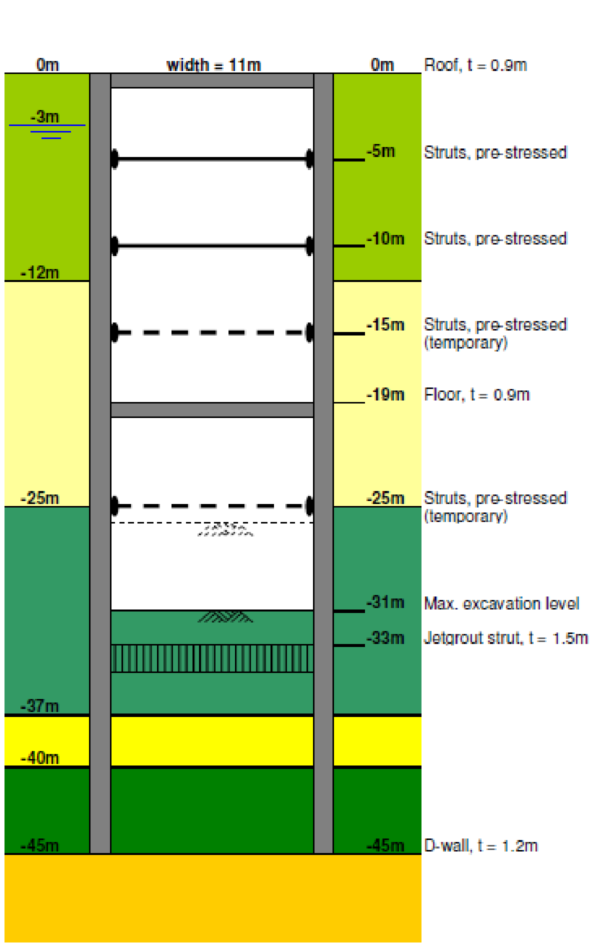


Figure 34 Geological profile and layout of Ceintuurbaan station in Amsterdam (Buykx et al, 2009)

Europaplein station

The extension of the station Europaplein on the subway North/South line in Amsterdam were constructed by sheet pile retaining wall and underwater cast concrete slab with VM-piles as compression and tension piles. The station floor is placed at the depth of 11 meters below surrounding ground. Design pile loads varied between a compression load of 1100 kN and a tension load of 700 kN. The reason to choose the VM-pile was that a stiff load settlement specification was demanded in both tension and compression. Also the influence on the nearby RAI congress center should be as little as possible. Therefore the VM-pile, that is unusual in the Netherlands but common in parts of Germany, were chosen by the client. As mentioned before the VM-pile (also known as MV-pile) consists of an H-beam with grouting along the pile. (van Kempen, 2010)

6.6.2 Rotterdam

Four projects from the Rotterdam area is included in this study and is presented in this chapter. The first involves bridge-constructions on the rail link between The Hauge, Rotterdam and Utrecht. Then, two metro stations in Rotterdam will be described and last the foundation for a power plant in Massvlakte in Rotterdam is presented.

Railway link between The Hauge, Rotterdam and Utrecht

For expansion of the railway system between the cities The Hauge, Rotterdam and Utrecht a large number of bridges were constructed. The original design proposed a design with thick concrete slab with a large number of pre-cast concrete piles as support. On the slab, 1.2 meter diameter concrete columns are placed as support for the bridge. Another design proposal was made with 1.65 meter diameter bored casing piles with one pile below every column, see Figure 35. This kind of piling method has never been used earlier in the Netherlands. With this new design, the slab could be omitted. The bearing capacity of the piles should be 12 000 kN and have a deformation of less than 10 mm when a train passes. Since the load displacement properties of the bored piles are worse, the pile has to be injected after hardening of the concrete by a grouting device. This grouting performed a pre-stressed pile tip and consequently the whole pile was pre stressed. During boring of the piles, stabilizing fluid i.e. bentonite was not used. To perform a stable borehole during drilling in the casing the groundwater level in the bore had to be kept on a higher level than in the pile tip. (Brinkman et al, 2009).

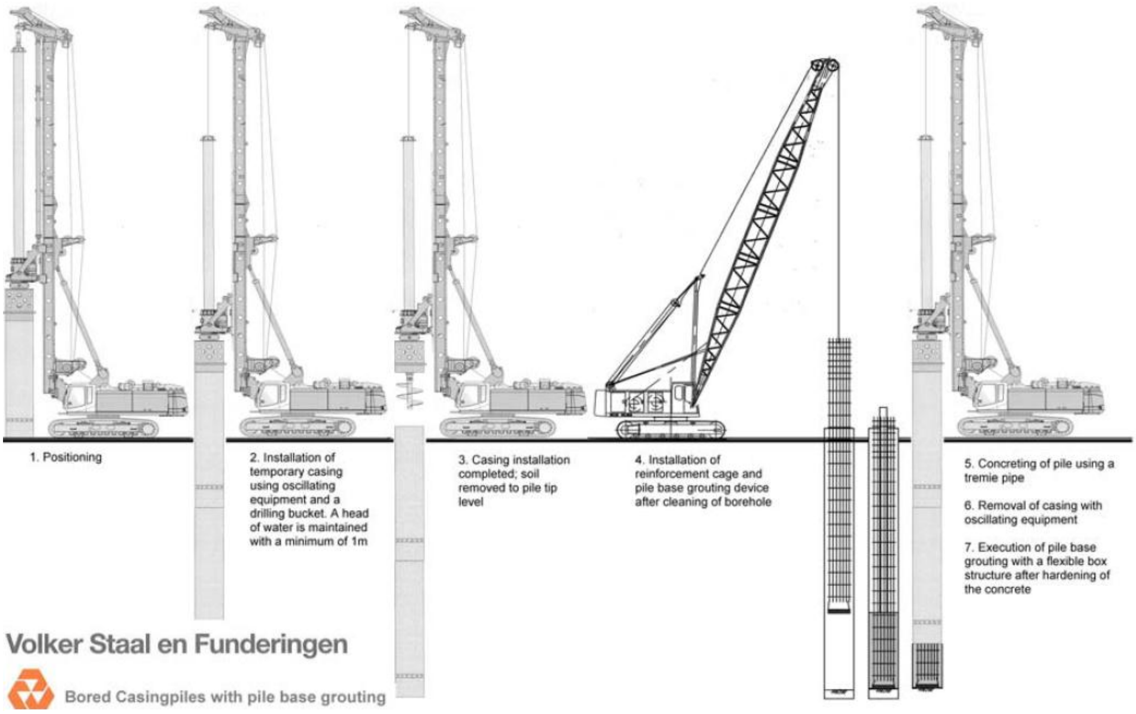


Figure 35 Installation of bored casing piles (de Jong et al, 2010)

Rotterdam metro station

In Rotterdam, reconstruction of a metro station is performed with diaphragm walls and a small part of ground freezing. The diaphragm wall is reaching about 38 meters deep and will stop the water inflow from water carrying Pleistocene sand layers. Groundwater control is very important during construction in Western Netherlands due to damages of structures occur if the groundwater level is fluctuating. During the excavation, a leak in the diaphragm wall occurred at 14 meters depth. A lot of sand and water flushed into the excavation pit through the diaphragm wall joint from the sand layer at app. 16 meters depth. The water was flowing through a clay and peat layer and took a short-cut through the joint. To reduce the leak PU foam was injected close to the leak and sand bags were placed in the excavation pit to decrease leakage. Later it was observed that the leakage continued and washed away the PU foam, installation of wells outside the diaphragm wall reduced the inflow and new PU foam was injected that stopped the leak. Finally, the leak was closed with a combination of sheet piles outside the excavation and jet grout piles to connect the sheet piles to the diaphragm wall. Since the sheet piles could not be installed to 38 meters depth, jet grout piles had to be installed around the sheet piles. The sheet piles and jet grout piles were not watertight but the PU foam made the structure watertight. Since a lot of sand was flowed into the excavation pit, settlements occurred around the excavation pit. Near the leak, the settlements were about 2 meters. (Thumann et al, 2009).

Metro station Wilhelminaplein

In Rotterdam the station Wilhelminaplein was constructed during the mid-90´s as a new station on the Metro. To construct the building pit, sheet pile wall and combi wall were used. The reason to choose a combi wall was that it was intended to be used as foundation element and the height of the excavation pit. The open tubular piles used in this project varied between 1219 and 1420 mm in diameter. The material thickness of the piles was 16 to 22 mm and the length of the piles 31.5 meters. The intermediate sheet piles were Larsen IVS with a length of 21.5 meters. The water tightness of the combi wall was almost total since Holocene clays and peats, with low hydraulic conductivity, are present outside the retaining wall. (Brassinga et al, 1997)

To prevent groundwater from flowing into the tunnel from the ends, over the existing tunnels, jet grouting columns are performed. Also groundwater recharge was performed to reduce the groundwater drawdown. The station will float up if it is not secured with tension piles. The tension load prescribed was 600 kN/m for every pile row. In the original plan the station was planned to be secured with Tubex tension piles with grout injection, but the contractor proposed Franki piles with injection of cement water mix. With Franki piles, the injection takes place at the pile toe compared to the Tubex pile where the injection takes place during penetration. The diameters of the Franki piles were 406 mm at the depth of 21 meters. The Franki piles toe is present at 33 meters. When the pile was installed, it was injected with 500 liters grout, with an increase of the diameter of 30 mm. The maximum load performed on the pile was 3500 kN. The ultimate load was not reached but it was estimated to a span between 5 MN and 7 MN. (Brassinga et al, 1997)

For the narrower parts in the excavation pit, micro piles were used as tension piles. To the depth of 26.5 meters a 178 mm tube is penetrated by water injection. When the tube is cleaned a 114 mm tube is installed inside the other tube. After, the outer tube is pulled out during injection of grout with a maximum pressure of 20 bars. During testing of the piles the shaft friction was interpreted to 240 kPa. (Brassinga et al, 1997)

Power plant foundation in Massvlakte

To construct a boiler vessel for a power plant in Massvlakte near Rotterdam, large diameter bored piles were used. The diameters of the piles were 1500 mm and needed to be mainly end bearing, in a dense sand layer, since a compressible clay layer occurs in the upper part of the soil profile. Strict settlement requirements were prescribed, torsion resistance and high stress levels in the end of the piles occurred. The contractor, Bauer, grouted the pile bases with their grout cells. Grouting of the pile base will significantly increase the stiffness of the pile. Therefore the working load can be higher on the piles and the displacements at the pile tip reduced, without increased ultimate bearing capacity. For staircase towers, constructed at the power plant, the foundation was also made of bored piles with subsequent grouting between the base slab and first clay layer. (Balian et al, 2009)

6.7 United Kingdom

CFA piles are one of the most used bored piling techniques in the United Kingdom. The CFA- piling technique was introduced into the United Kingdom during the early 80´s and it has now 50 % of the bored pile market in the UK. The other most used bored piling technique is the cast in-situ piling method. (Clifford, 2001) Other piling systems used in the UK are Atlas piles that were introduced from Belgium to the UK during early 90's, precast concrete piles of different cross section and geometry, secant and interlocking pile walls, bored piles, bored piles with grout injection and cemcol. Cemcol can be described as a stone column that is used as foundation for light structures. Keller introduced compacted concrete columns around 1990 in the UK. (Thornton, 1992)

6.7.1 London

To transfer heavy loads, underreamed piles are an economical solution. Underreamed piles of large diameter have been used for 40 years in London. For the first piles, the shaft diameters were 1.22 meters and the diameter of expanded end 3.66 meters. When the buildings became large, the loads increased the diameter of the piles and the underreaming. It also took days to complete the piles since the underreaming first was created by hand. However, the machines improved and the hand excavation became less common. The underreams performed nowdays can be up to 6.33 meters in diameter. The most costly part of large diameter bored piles is the concrete, which corresponds to more than 50 % of the cost. If straight shafted piles is compared with underreamed piles, both with a capacity of 10 000 kN to 25 000 kN, the straight shafted piles will consume more concrete. Since the time of construction for both pile types are the same, the underreamed piles will be cheaper. A disadvantage with the underreamed piles is that the settlement under working load will be greater than for straight shafted piles during comparable situations. This is a consequence of a larger amount of load is transferred through the end of the pile compared to straight shafted piles. To construct underreamed piles, the bored shaft and the underreamed end need to be stable a reasonable time period. This is needed because the pile base has to be cleaned and inspected before concreting. (Clifford, 2001)

6.7.2 Southampton

For the retail development project West quay development project in Southampton, CFA piles and pre-cast piles were used. The number of piles in this project was over 6000 pre-cast piles and almost 800 CFA piles. In this project, a continuous CFA pile wall was used with a retained height of 10 meters. The continuous pile wall was temporary propped to take lateral forces. The requirement on the pre-cast piles were a settlement of not more than 8 mm at working load (900 kN) and 15 mm at working load plus fifty percent. For the CFA piles the requirement was a settlement of not more than 5 mm at working load and 10 mm for working load plus fifty percent. (Wood, 1998b)

As mentioned before, both CFA and pre-cast concrete piles were used. The pre-cast piles were driven down to a stiff marine clay layer at a depth between 18-24 meters depending of the loading of the piles. CFA-piles were designed to a depth of 19 meters. The requirements determined that the piles should not penetrate the Bracklesham beds due to the risk of upward groundwater seepage. In this project, CFA piles were used adjacent to buildings to reduce noise and vibration. In all other areas in this project, two section driven piles were used, thus obtaining a cheaper foundation solution. Silenced hammers were also used on the piling rigs to reduce the noise and vibration. (Wood, 1998a)

6.8 Spain

In Spain pre-cast piles are used to some extent in the southern parts of the county, in non urban areas. CFA piles are used for small and medium diameter piles and accounted during the first part of the 90's to the larger part of the pile meters installed. Other pile systems used in Spain are large diameter bored piles with casing or bentonite sludge and bored piles without casing or bentonite where the soil is stable enough. (Delgado, 1992)

6.8.1 Barcelona

Excavation with diaphragm wall support has been made in the soft deltaic soils in Barcelona. The diaphragm wall excavation creates movements in the ground around. When the concrete is filled in the diaphragm wall excavation, the settlements stops and some heave begins. Later, when the concrete hardening has come to its end, the settlements have to some extent compensated for the heave. (Caritte et al, 2010)

According to Caritte et al (2010) the measures to be performed to reduce settlements are to keep a constant high level of bentonite during excavation and reduce the length of the diaphragm wall panel.

To perform an excavation for a hospital in Barcelona, a prestressed diaphragm wall was used with 17.72 meter depth, see Figure 36, the upper part in Figure 36 constitutes an existing retaining wall. The prestressing of the diaphragm wall was performed to prevent cracking in the concrete and to stabilize the diaphragm wall with the vertical and eccentric force that is anchored in the bedrock below. To create the prestressing force five tendons are installed in each T-panel with a force of 900 kN. The excavation depth was between 14-16 meters. Since the diaphragm wall was constructed close to a subway tunnel, it was not allowed to use ground anchors near the tunnel. Also, the vibrations from the subway and movements had to be reduced. Therefore the diaphragm wall was constructed with T-shaped panels and linked together with short panels, where the T-shape is pointing out from the wall. The interlocking panels, also called the secondary panels, are 9.8 meters high and 2.2 meters wide. These panels transfer the pressures from the soil to the T-shaped panels by bending and shear. Therefore the T-shaped panels perform the stability of the diaphragm wall. The ground between the T-shaped panels was left to create a berm for increasing the global stability. (Molins et al, 2006)

All of the diaphragm wall panels were excavated with hydromill and bentonite was used to stabilize the excavation of the diaphragm wall. Then a reinforcing cage with seven ducts for prestressing was lowered into the diaphragm excavation and after concreting was performed. The groundwater level was located far below the excavation bottom. (Molins et al, 2006)

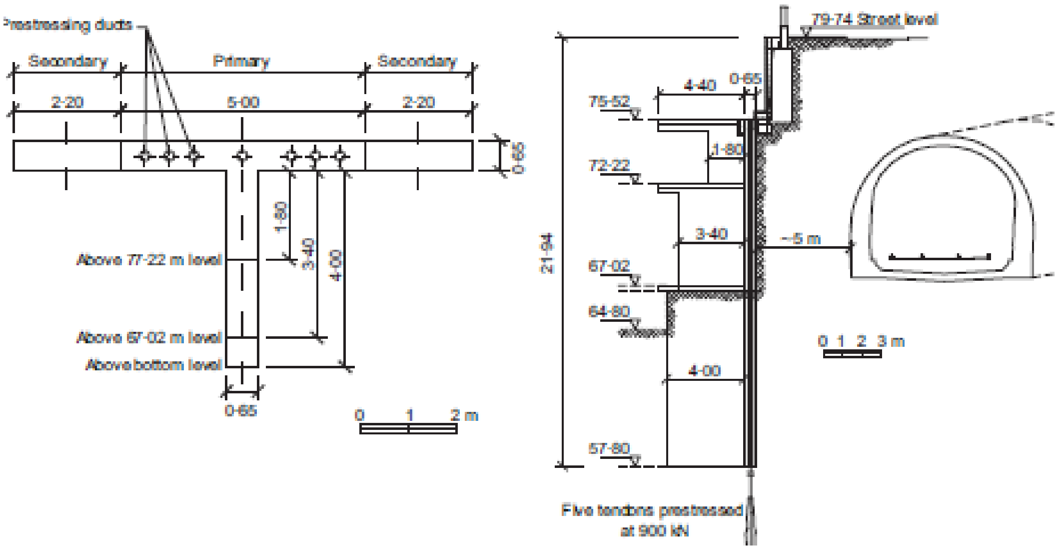


Figure 36 Cross section and vertical cut of the prestressed T-shaped diaphragm wall (Molins et al, 2006)

7. ANALYSIS

Multi criteria analyses (MCA) are performed to support decision makers to handle a large amount of complex information in a consistent manner. In MCA, the different alternatives are evaluated from different objectives determined by the decision maker. To fulfill these objectives, measurable criteria is used. In MCA analyses, a performance matrix is used where each alternative are described in each row and each column represents the score for each criteria. The scoring is the performance of each criteria to the different alternatives. The weighting for each criterion determines how much the criteria will contribute to the summed result and therefore the importance of the criteria. (Communities and local government, 2009)

The multi criteria analysis in this master thesis is performed as a linear additive multi criteria analysis. In the linear additive model, the score of each parameter and weighting of each criteria are multiplied and the sum of all these will be the result. The criteria to be evaluated are noise, vibrations, capacity, cost, displacement for piles and settlement due to ground water drawdown and settlements in the ground due to deformation in retaining structures. The foundation methods to be used in the multi criteria analysis are from the piling area: screw cast-in-place displacement piles, driven cast-in-place displacement piles, pre-cast concrete piles, steel tube piles and H-profile piles, timber piles, bored cast-in-place piles, continuous flight auger piles (CFA), and steel core piles. Different driving techniques for precast piles, steel tube piles, H-profile piles and timber piles will be evaluated in the multi criteria analysis. These are the single-acting drop hammer, double-acting drop hammer, single acting diesel hammer, double acting diesel hammer, vibratory hammer and hydraulic jacking. The following retaining structures will be evaluated: CFA pile wall (continuous and secant), sheet pile wall, diaphragm wall, berliner wall / king post wall, combi wall and steel tube pile wall. Different installation techniques for sheet piling and these are impact driving, vibratory driving and jacking. In the multi criteria analysis four different cases are used. These cases for piling methods are the following; a heavy superstructure close to surrounding constructions, a light superstructure close to surrounding constructions, a light superstructure at a certain distance from surrounding constructions and a heavy superstructure at a certain distance from surrounding constructions. For the retaining structures two cases are used retaining structure close to surrounding constructions and retaining structure at a certain distance from surrounding constructions.

The MCA is made in an Excel sheet where the different piling and retaining structures are scored from the criteria noise, vibrations, cost, capacity, soil displacement for piles and settlements for retaining structures. The criteria are then weighted between 0 and 100 depending on the different cases mentioned above. The score and weight are multiplied and this added will be the result. The result of the multi criteria analysis is presented in a table where the highest value corresponds to the method that performs best in the multi criteria analysis. The multi criteria analysis will be performed from a Swedish point of view. Therefore the cost score will be lower for methods were machinery is not commonly present in Sweden.

7.1 Scoring of parameters

The piling methods and methods for retaining structures are scored from information presented in chapter 3 - *Foundation methods* and chapter 6 - *Inventory of used foundation methods*. In specific, chapter 3.3 is used for scoring of the parameters *noise* and *vibrations.* The scoring is made between 1 and 3. Score 1 corresponds to extensive impact, score 2 corresponds to intermediate impact and score 3 corresponds to low impact. As mentioned before, the score is multiplied by the weighting and then the result is obtained. The method with the highest value is considered as the most preferable method.

7.1.1 Noise

The degree of noise disturbance from the various piling methods and methods for retaining structures are in Table 7 and Table 8 respectively scored between 1 and 3. This scoring is made by information presented in chapter 3 - *Foundation methods* in general and the tables in chapter 3.3.1 in specific.

Table 7 Scoring of various piling methods regarding noise disturbance

Category of pile	Noise
Screw cast-in-place	3
Driven cast-in-place	1
Pre-cast concrete	
single-acting drop hammer	1
double-acting drop hammer	1
single-acting diesel hammer	1
double-acting diesel hammer	1
vibratory hammer	2
hydraulic jacking	3
Steel tube or H-profile	
single-acting drop hammer	1
double-acting drop hammer	1
single-acting diesel hammer	1
double-acting diesel hammer	1
vibratory hammer	2
hydraulic jacking	3
bored	3
Timber	
single-acting drop hammer	1
double-acting drop hammer	1
single-acting diesel hammer	1
double-acting diesel hammer	1
vibratory hammer	2
hydraulic jacking	3
Bored cast-in-place	3
CFA	3
Steel core	3

Table 8 Scoring of various methods for retaining structures regarding noise disturbance

Category of retaining structure	Noise
Continous wall (CFA)	3
Secant wall (CFA)	3
Sheet pile wall	
Impact driving	1
Vibratory driving	1
Jacking	3
Diaphragm wall	3
Berliner wall / King post wall	2
Combi wall	1
Steel tube pile wall	3

7.1.2 Vibrations

The degree of vibration disturbance that are induced by various piling methods and methods of retaining structures are scored in Table 9 and Table 10 respectively. The scores for the driving methods; *single-acting drop hammer, double-acting drop hammer, single acting diesel hammer, double acting diesel hammer, vibratory hammer and hydraulic jacking* are according chapter 3.3.2. Also the scores for retaining structures, *impact driving and vibratory driving* are according to chapter 3.3.2. In Table 5 in chapter 3.3.2, the degree of vibration disturbance is divided into cohesive and frictional soil. In order to use a linear additive MCA, the scoring of the degree of vibration disturbance for the various methods is compiled by the mean values for the two types of soils is used. The scoring of the other methods is made on the basis of the information in chapter 3 - *Foundation methods* and chapter 6 - *Inventory of used foundation methods*.

Table 9 Scoring of various piling methods regarding vibration disturbance

Category of pile	Vibration
Screw cast-in-place	3
Driven cast-in-place	1
Pre-cast concrete	
single-acting drop hammer	2
double-acting drop hammer	2
single-acting diesel hammer	2
double-acting diesel hammer	2
vibratory hammer	1,5
hydraulic jacking	3
Steel tube or H-profile	
single-acting drop hammer	2,5
double-acting drop hammer	2,5
single-acting diesel hammer	2,5
double-acting diesel hammer	2,5
vibratory hammer	1,5
hydraulic jacking	3
bored	3
Timber	
single-acting drop hammer	2
double-acting drop hammer	2
single-acting diesel hammer	2
double-acting diesel hammer	2
vibratory hammer	1,5
hydraulic jacking	3
Bored cast-in-place	3
CFA	3
Steel core	3

Table 10 Scoring of various methods for retaining structures regarding vibration disturbance

Category of retaining structure	Vibration
Continous wall (CFA)	3
Secant wall (CFA)	3
Sheet pile wall	
Impact driving	2
Vibratory driving	1,5
Jacking	3
Diaphragm wall	3
Berliner wall / King post wall	3
Combi wall	1
Steel tube pile wall	3

7.1.3 Soil displacement

During installation of piles, the soil may be displaced and this may have an impact on the surrounding buildings. The degree of soil displacement for piles is based on the categorization made in chapter 3 – *Foundation methods*, and the scoring is presented in Table 11.

Table 11 Scoring of various piling methods regarding soil displacement

Category of pile	Soil displacement
Screw cast-in-place	1
Driven cast-in-place	1
Pre-cast concrete	
single-acting drop hammer	1
double-acting drop hammer	1
single-acting diesel hammer	1
double-acting diesel hammer	1
vibratory hammer	1
hydraulic jacking	1
Steel tube or H-profile	
single-acting drop hammer	2
double-acting drop hammer	2
single-acting diesel hammer	2
double-acting diesel hammer	2
vibratory hammer	2
hydraulic jacking	2
bored	3
Timber	
single-acting drop hammer	1
double-acting drop hammer	1
single-acting diesel hammer	1
double-acting diesel hammer	1
vibratory hammer	1
hydraulic jacking	1
Bored cast-in-place	3
CFA	3
Steel core	3

7.1.4 Settlements

Deformations in retaining walls may create settlements in the ground outside the retaining wall. Also, lowering of groundwater due to leaking retaining structure will create settlements. The scoring describing the degree of settlements for various methods for retaining structures, presented in Table 12, is based on a combination of both phenomenon. The scoring is based on chapter 3.2 and chapter 6 - *Inventory of used foundation methods*.

Table 12 Scoring of various methods for retaining structures regarding settlements

Category of retaining structure	Settlements
Continous wall (CFA)	1
Secant wall (CFA)	2
Sheet pile wall	
Impact driving	2
Vibratory driving	2
Jacking	2
Diaphragm wall	3
Berliner wall / King post wall	1
Combi wall	2
Steel tube pile wall	2

7.1.5 Relative cost

The scoring of the relative costs for various piling methods and methods for retaining structures is presented in Table 13 and Table 14 respectively. The scoring of relative costs is based on the information presented in chapter 3 – *Foundation methods* and through correspondence with Hercules Grundläggning. Score 1 corresponds to high installation costs, score 2 corresponds to intermediate installation costs and score 3 corresponds to low installation costs. As the relative costs for the various driving methods for installing pre-fabricated piles is largely dependent on the geotechnical conditions at the site, scoring has been performed only for each pile category.

Table 13 Scoring of various piling methods regarding relative costs

Category of pile	Cost
Screw cast-in-place	1
Driven cast-in-place	1
Pre-cast	3
Steel tube or H-profile	2
Timber	3
Bored cast-in-place	1
CFA	1
Steel core	1

Table 14 Scoring of various methods for retaining structures regarding relative cost

Category of retaining structure	Cost
Continuous wall (CFA)	1
Secant wall (CFA)	1
Sheet pile wall	
Impact driving	2
Vibratory driving	2
Jacking	1
Diaphragm wall	1
Berliner wall / King post wall	3
Combi wall	1
Steel tube pile wall	1

7.1.6 Capacity

Since large differences occur in capacities of the various pile types, a scoring regarding the capacity has been made. The scoring is regarding capacity is presented in Table 15, and is based on information presented in chapter 3.1.

Table 15 Scoring of various piling methods regarding capacity

Category of pile	Capacity
Screw cast-in-place	3
Driven cast-in-place	3
Pre-cast	2
Steel tube or H-profile	3
Timber	1
Bored cast-in-place	3
CFA	2
Steel core	3

7.2 Weighting of criteria

How important the various criterions are is specific for each project. In order to simulate that, four cases for piling and two cases for retaining structures has been conducted. Each weight presented in Table 16 – 21 is in chapter 7.3 multiplied with the scores presented in chapter 7.1.

7.2.1 Cases for piling

In table 16 – 19, the weighting of the criteria is presented for the four different cases. The two variables for these cases are the magnitude of the load imposed by the superstructure and the distance to surrounding constructions. *Light superstructure* indicates a construction with lower requirements on capacity (e.g. road embankments or light houses) and *Heavy superstructure* indicates a construction with high requirements on capacity (e.g. bridge abutments or high-rise buildings). *Close to surrounding constructions* indicates that soil displacement might cause major problems for surrounding constructions and *at a certain distance from surrounding constructions* indicates that soil displacement might not be problematic for surrounding constructions. The relative weighting is higher for criterions that are considered more important for each specific case.

Table 16 Case I – Light superstructure close to surrounding constructions

Noise	20
Vibration	30
Soil displacement	30
Cost	10
Capacity	10

Table 17 Case II – Light superstructure at a certain distance from surrounding constructions

Noise	10
Vibration	20
Soil displacement	10
Cost	50
Capacity	10

Table 18 Case III – Heavy superstructure close to surrounding constructions

Noise	15
Vibration	25
Soil displacement	25
Cost	5
Capacity	30

Table 19 Case IV – Heavy superstructure at a certain distance from surrounding constructions

Noise	10
Vibration	20
Soil displacement	10
Cost	30
Capacity	30

7.2.2 Cases for retaining structures

Similar to table 16 – 19, table 20 – 21 presents the relative weightings for criterions important for retaining structures. As retaining structures are rarely used to transmit vertical loads from superstructures to deeper soil levels, the only variable is the distance to surrounding constructions. *Close to surrounding constructions* indicates in these cases that a settlement caused due an excavation is critical and *at a certain distance from surrounding constructions* indicates that a subsidence in close adjacent to the retaining structure may not be problematic to surrounding constructions. The relative weighting is higher for criterions that are considered more important for each specific case.

Table 20 Case I Retaining structure close to surrounding constructions

Noise	20
Vibration	30
Settlements	30
Cost	10

Table 21 Case II Retaining structure at a certain distance from surrounding constructions

Noise	10
Vibration	20
Settlements	10
Cost	50

7.3 Results from MCA

In Table 22 and 23, the result from the linear additive MCA is presented for the various piling methods and methods for retaining structures respectively and for each case. High values indicate that the specific method is suitable for that case.

Table 22 Result from the linear additive MCA for various piling methods

Category of pile	Case I	Case II	Case III	Case IV
Screw cast-in-place	220	180	240	220
Driven cast-in-place	120	120	160	160
Pre-cast concrete				
single-acting drop hammer	160	230	165	210
double-acting drop hammer	160	230	165	210
single-acting diesel hammer	160	230	165	210
double-acting diesel hammer	160	230	165	210
vibratory hammer	165	230	167,5	210
hydraulic jacking	230	270	220	250
Steel tube or H-profile				
single-acting drop hammer	205	210	227,5	230
double-acting drop hammer	205	210	227,5	230
single-acting diesel hammer	205	210	227,5	230
double-acting diesel hammer	205	210	227,5	230
vibratory hammer	195	200	217,5	220
hydraulic jacking	260	240	270	260
bored	290	250	295	270
Timber				
single-acting drop hammer	150	220	135	180
double-acting drop hammer	150	220	135	180
single-acting diesel hammer	150	220	135	180
double-acting diesel hammer	150	220	135	180
vibratory hammer	155	220	137,5	180
hydraulic jacking	220	260	190	220
Bored cast-in-place	280	200	290	240
CFA	270	190	260	210
Steel core	280	200	290	240

Table 23 Result from the linear additive MCA for various retaining structures

Category of retaining structure	Case I	Case II
Continous wall (CFA)	190	150
Secant wall (CFA)	220	160
Sheet pile wall		
Impact driving	160	170
Viratory driving	145	160
Jacking	220	160
Diaphragm wall	250	170
Berliner wall / King post wall	190	240
Combi wall	120	100
Steel tube pile wall	220	160

The most preferable pile type in case I, case III and case IV are the bored steel tube pile. In case II the most preferable pile type is the jacked pre-cast concrete pile. For the retaining structures, the most preferable is the diaphragm wall in case I and the Berliner/king post wall in case II

8. DISCUSSION

Construction and maintenance of civil infrastructure in an urban environment will always affect the surroundings to some extent. When considering negative impacts to the urban environment caused by civil infrastructure projects, one ought to be aware of the fact that these sorts of projects are usually of large scale, long-termed and involve major investments. These vast amounts of reallocated resources will however over the life-length of the object being constructed or maintained increase the welfare for the region as a whole. This maybe because a new subway-line might shorten the commuting-time for people living in the outskirt of the city, as seen in Rotterdam. Or it may be because an extension of a hospital might result in better healthcare, as seen in Trondheim. Or it may be because a road tunnel might make the urban environment more aesthetic, as seen in Gothenburg. It is therefore difficult to apply the *Pareto*-criteria for civil infrastructure projects as some people in Rotterdam might never use the new subway-line, or some people in Trondheim might never need healthcare at the newly constructed hospital or some people in Gothenburg might never use the new urban area created along the river.

More applicable to civil infrastructure project is the *Hicks/Kaldor*-criteria, which state that a project is profitable for the society if the gain in welfare for those who profit from the project is higher than the loss in welfare for those who does not profit from the project. Reducing the negative impacts in an urban environment may in this context lead to that the loss in welfare for those who does not profit from the project is reduced, resulting in higher welfare for the region. Therefore, using a method with less negative impacts on the urban environment might lead to an increased welfare compared to if a conventional method was used, even though that the certain method with reduced negative impacts may be more expensive than the conventional method. Even though the compensation in the *Hicks/Kaldor*-criteria is only of theoretical nature, actual monetary compensation to those affected by negative impacts might increase the tolerance of foundation works, by e.g. reduced rent for residents living in close adjacent to a construction site.

Negative impacts on the urban environment is difficult, and sometimes impossible, to include in a *Cost-Benefit Analysis*. The *Cost-Benefit Analysis* can only process monetary values, which in study is impossible to imply on vibrations, soil displacement and settlements. Vibrations of a certain magnitude and frequency might be unnoticed for some surrounding buildings and devastating for other surrounding buildings as various buildings has different resonance frequencies. Soil displacement can at some sites cause major damage to surrounding buildings and its deep foundation. At other sites, soil displacement might be unnoticeable and sometimes regarded as a positive effect as frictional soil material can in this way become compacted. It is not unusual that the soils within urban or sub-urban areas are contaminated due to previous usage of that site. In those cases, using non-displacement methods were spoil is formed can be regarded as an unsuitable alternative as it is expensive to handle those sorts of soil masses. Addressing a monetary value to settlements is also difficult, as the cost is entirely depending on the state of the surrounding building. However, this study shows that major reductions of losses in welfare can be made regarding noise levels. The parameter C_0 presented in chapter 4 – *Theorem of welfare*, can therefore be affected in a positive way if a less noisy method is used. Figure 27 in the same chapter also shows that a reduction from 75 dBA to 65 dBA will decrease the costs of noise with about 80 %. In an urban environment where thousands of people might be affected by the noise emitting activity, this reduction may generate major cost savings for the project if a method with low noise emissions is used.

As mentioned in the introduction of this report, a construction project in an urban environment can cause negative impacts in many ways. However, the negative impacts considered in this report are limited to noise, vibrations, soil displacement and settlements. The reason for why these negative impacts were chosen is that they are caused directly by the foundation technique used. Secondary impacts, such as traffic jams or reduced access in the city, are parameters that should not be over-locked. Various foundation methods will affect these secondary impacts in different ways. For instance, one foundation method might require a larger construction site than others as it implies using a more extensive set of machines or material (e.g. generators, compressors, storage of piles etc.). An example of this is the diaphragm wall that needs quite a lot of space for production. However, negative impacts like those are not considered in this study.

The noise from the installation techniques are described in the thesis as mean equivalent value. Therefore, the impulse noise levels from the hammer are larger than the mean value presented. The consequence of this is that piling with hammers are more disturbing than what it seems to be with the mean equivalent value because of its impulse properties. This has been taken into consideration when scoring the various foundation methods. The result of the assumption that impulsive noise sources (e.g. drop hammers or diesel hammers) disturbs more than steady or quasi-steady noise sources (e.g. jacking or bored cast-in-place) has been that foundation method generating impulsive noises has received a lower score than those generating steady or quasi-steady noises. This assumption has been made with consideration of figure 1.

Moreover, to what extent people gets disturbed by noise is largely depended on the sensitiveness of the people being exposed to the noise. People growing up in an urban environment tends to be more tolerant to noise emitting activities as they have been exposed to urban noises during their whole life. People growing up in a rural environment have in general been exposed to less noise compared to people growing up in an urban environment. Therefore, if those growing up in a rural environment moves to an urban environment, or that rural area gets developed into a sub-urban environment, relatively low noise levels might cause problem to some people. So using equivalent noise levels and standardized valuations of the monetary value of noise disturbance is problematic as all people do not react standardized on noise emitting activities. However, in order to be able to assign a monetary value to noise disturbance at all, standardized valuations have to be used. Some people might get more disturbed than what the standardized valuation indicate and some people might get less disturbed than what the standardized valuation indicates. But in general, the standardized valuations provide an efficient method to assign a monetary value to noise disturbance.

To estimate the vibrations from piling operation is a complicated task. In this thesis, we have got consultation to perform the scoring of the most common methods and the uncommon methods are based on the literature survey. Based on the literature survey, the methods that are depending on boring (e.g. bored cast-in-place or CFA) are assumed to crate less vibrations than those methods where an element (e.g. pile or casing) is installed by drop hammers. It has also been noticed that people being exposed to both noise and vibrations simultaneous gets more disturbed than if those to negative impacts occurs at separated occasions. However, it has not been possible to put a monetary value on that combination of negative impacts.

It has also been noticed during the work of this thesis that the degree of experienced disturbance may be decreased if people receive adequate information regarding the execution of the construction. If the people know why the construction is carried out and when negative impacts might occur, the tolerance seems to increase. This could be due to that the people realize that the construction might increase the welfare and convenience for them in the future, e.g. better communication or higher attractiveness of that area.

Selecting between various foundation methods should not be made only with consideration of the degree of negative impacts on the urban environment. Some retaining structures are not possible to use when some water tightness is needed. The continuous bored pile wall is an example of this, since it has gaps between the piles in the wall. When good water tightness is required, steel sheet piling, combi wall or steel tube pile wall is a better option, since the groundwater lowering will create settlements in the nearby area.

To reduce soil displacements during piling, H-beam piles, RD-piles, CFA-piles, bored cast in place piles are suitable methods. The H-beam piles create less soil displacements compared to driven piles and the other methods rely on boring out the soil volume used for the pile. If no or almost no displacements will be accepted, CFA-piles or bored cast in place piles should be used, since these techniques are boring a hole with an auger and the soil is removed instead of displaced to the sides. Also, RD-piles can be used since these are bored into the ground with a drilling crone inside the pile. Where requirements about low deformations exist, a stiff structure is needed. In this case a diaphragm wall can be used due to its stiffness. If large bearing capacity is required on the retaining wall, e. g. the wall should be used as base for a building wall, a secant or continuous pile wall, diaphragm wall or a RD-pile wall should have enough bearing capacity.

However, the piling and retaining construction technique will be further developed, and the negative impact on the environment will probably be one of the main development areas. Therefore refined conventional and new foundation methods will be introduced. One example of this development is the *MV-pile*, which is a further development of the *H-pile* with higher shaft resistance when the pile is in use but also with reduced shaft resistance during installation.

It ought to be mentioned that all foundation methods mentioned in this thesis are not suitable for all geological and geotechnical conditions. Some foundation methods require some initial internal strength in the soil, especially foundation methods which imply soil extraction without using a temporary casing. If a foundation methods like that, e.g. the *Starsol pile*, is to be used at a site where clay with a low strength is present, e.g. Gothenburg, the void created in the soil would probably collapse before the wet cement is inserted. Also, if the soil has a very low strength, the void created for the pile shaft could be expanded during cementation as both vertical and horizontal forces would be induced on the un-cased walls. Finally, some foundation methods require that the solid bedrock is within a reachable distance. Foundation method like that, e.g. the *Steel Core pile* or *RD-pile*, would not be economically reasonably to use where the solid bedrock is not within a reachable distance. What the "reachable distance" implies should be determined for each specific project where e.g. *Steel Core piles* are considered. From the construction of the 18-storey building in

Oslo, where the solid bedrock was located about 40-50 meter below ground level, it was found that *Steel Core piles* was used.

Even though the foundation market in Sweden is dominated by pre-fabricated, soil displacing methods, the experiences from the project involving the secant pile wall in Stockholm might open the market for other cast-in-place methods. As mentioned in chapter 6.1, the most common pile driving technique in Sweden involves drop-hammers. If the requirements regarding negative impacts in the urban environment become stricter, jacking methods similar to the one used in Trondheim (chapter 6.2.2) might be a suitable option when possible. The project involving sheet piling in Copenhagen showed that the structure fulfilled the requirements on bearing capacity and settlement. However, as the guideline values for maximum noise levels were exceeded by 16 dBA, it's obvious that the development of more silent but still economical reasonable foundation methods has to take place in the Scandinavian countries, similar to the development showed in Germany and the Netherlands. However, the construction of the metro station in Rotterdam showed that if used at unsuitable location, diaphragm walls may result in vast leakage of groundwater and cause settlements.

In the geological description, different sources have been used to obtain information. These sources describe the geology in the countries in different detail level. One source may describes the geology in one construction project and other in general for the whole country. In the first kind of source, different sections or boreholes are used. In this case, it is important to bear in mind that the geological properties may differ rapidly and therefore complete different geology some hundred meters away will be observed.

During this literature survey, a lot of different geologies have been observed around Europe. A consequence of this is that a foundation method suitable for a country in Western Europe maybe not is suitable for e. g. Northern Europe due to too different geologies. For example the clay in London, which is stiff, and the clay in Sweden which has a shear strength between 5-15 kPa. In these cases boring with casing would be necessary in the Swedish clay, due to its low shear strength and the subsequent cave in if casing would not be used. This makes the method more expensive to use in Sweden compared to England, since the London clay is stiffer and the casing maybe omitted.

To present representative and up-to-date inventory of which foundation method are used in each specific country, a questionnaire was conducted. This questionnaire was adjusted to each country as geological maps using GIS-programs and European soil databases were conducted. The questionnaire was sent to selected stakeholders and partners within the *Pantura*-project, as well as to various technical universities, research organizations, geotechnical associations and civil engineering companies around Europe. In total, about 140 people were invited to participate in this questionnaire. The ambition of this survey was to examine which piling techniques and methods of retaining structures was commonly used in urban environments. However, the outcome of this survey was not satisfying. Through the software used for this questionnaire, it became clear that less than 30 % of the invited participants opened the questionnaire and less than 3 % of the invited participants finished the questionnaire. Due to the lack of correspondence, no conclusions were drawn from that questionnaire. If we would have done the questionnaire again, we would have made it more comprehensive in the hope that more participants would finish it. But the major problem we encountered was that more than 70 % of the invited participant did not even open the questionnaire, which we see as alarming as the majority of the invited participant were stakeholders within the *Pantura*-project. However, some of the invited participants came with useful comments and recommendations regarding literature.

The inventory of different foundation methods was therefore based on a literature study, where conference proceedings have been studied. The literature study corresponds to a little part of all literature available. Therefore the working material can be too small to get a correct picture of the piles and retaining structures used in the selected countries. However, a general picture of what kind of methods used in the countries is presented. In some cases, methods that are new to the countries have been included in the report. Though, the foundation methods have been used in these countries and are therefore included in the report. Some countries in this report are more thoroughly presented than others, especially the Netherlands. The reason for this is that we found many interesting projects and foundation methods in the Netherlands in general, and in Amsterdam and Rotterdam in specific. Also, through the questionnaire mentioned above, good correspondences were established with stakeholders in the Netherlands. Another method would have been to include more countries in the study and describe these more in general. However, as more thorough studies were made at projects that were interesting for this subject, more detail-information were obtained.

Some of the sources in this thesis are quite old. However, the information in these sources is interesting due to overview and trends obtained from these sources. In these old sources, some foundation methods are introduced in one country, and when a newer source is studied, the result was that the foundation method is still used in the country and is further developed.

According to the MCA the most suitable pile in case I, case III, case IV are bored steel tube piles and the most suitable pile in case II hydraulic jacked pre-cast concrete piles. Regarding bored steel tube piles, one limitation occur and this is that bedrock must be located in reachable depth from the ground level. In these cases the second best alternative is bored cast in situ piles in case I and case III, and hydraulic jacking of steel tubes or pre-cast concrete in case IV. For the retaining structures the most suitable retaining structure in case I is the diaphragm wall and the most suitable retaining structure in case II is the berliner/king post wall. However, as mentioned before the diaphragm wall need a lot of space and the berliner needs ground with low groundwater table.

The MCA should not be seen as the final answer to what kind of foundation method to be chosen, since a lot of uncertainties occur such as varying soil conditions, cost of moving equipment, raw material costs and local regulations. The scenarios in the MCA are based heavy or light structures and close to a sensitive building or at a distance from the sensitive building. The distance chosen in the latter case is a distance where the soil displacements/settlements may not be of primary interest, but we still are in the urban environment. Therefore, the displacements/settlements have lower weight in these case compared to the case were the structure is located close to the sensitive structure. Also, noise and vibrations have a lower weight in these cases. Cost has a higher weight in the case where the construction is performed at a certain distance from the sensitive structure. This is because a less costly method can be chosen in this situation due to more alternatives to choose between. Close to a sensitive structure it is more important to limit the noise, vibrations, and soil displacements than the cost of the method. In the case where a heavy superstructure is considered, the weight is higher on capacity.

The scoring performed in the thesis is based on the literature survey and expert judgments. These scores are uncertain since there are a lot of simplifications made in the MCA. Some methods may not be possible to perform in some geologies and the installation cost may vary a lot. Also transportation cost of the equipment varies since the equipment is not present in some parts of Europe. Other parameter that is not taken into consideration in the MCA is the excavation depth for the retaining structures. Since the depth will increase the loads on the retaining structure, resulting in increased deformations if the stiffness of the structure is not increased. Therefore the excavation depth is an important parameter.

Foreign civil contractors and foundation sub-contractors will probably increase their presence in Sweden and therefore introduce new methods. Therefore the contractors may base machinery in Sweden. This will reduce the transportation costs and therefore make the method less costly and more competitive.

Development and implementation of new foundation techniques with low negative impacts on the urban environment is an expensive task for the foundation sub-contractors. This study shows that the regulations and guidelines regarding noise-levels from foundation works in urban environments differs between the selected countries, and are in general less strict in Sweden and Norway compared to the other countries. However, if the regulations regarding negative impacts on the urban environment from foundation works will be stricter in the future for the Scandinavian countries, companies that already have strategies and knowledge about foundation techniques with low negative impacts on the urban environment will gain a sustainable competitive advantage towards those companies that are not prepared for stricter regulations.

9. CONCLUSION

This study shows that the classical *Pareto*-criteria cannot be used when examining various foundation techniques in urban environments, as someone will always to some extent be disturbed by construction of civil infrastructure within an urban environment. Rather, the *Hicks/Kaldor*-criteria may be used as it considers the fact that many people may receive increased welfare due to a certain construction project. Applying monetary values to noise disturbances, a *Cost-Benefit Analysis* may be used to show that a foundation method with higher relative costs may be more economical for the society if it disturbs the urban environment less than foundation methods with lower relative costs.

Stricter regulation regarding using pre-fabricated, soil displacing foundation elements, installed by driving methods that causes impulse noises, in urban environments were found in Germany, the Netherlands and U.K. compared to the Scandinavian countries. As a result of that, foundation techniques causing less severe negative impacts on the urban environment are more frequently used in these countries. Foundation techniques implying cast-in-place are more frequently used in urban environments in Germany, the Netherlands and U.K., such as CFA-piles and diaphragm wall, whereas the Scandinavian market is dominated by pre-cast concrete piles and steel sheet piling.

If stricter regulations regarding negative impact on the urban environment from foundation works were to be introduced in Sweden, companies that have a strategy and knowledge about methods with reduced negative impact on the urban environment will gain a sustainable competitive advantage.

According to the linear additive Multi-Criteria Analysis performed in this study, it was shown that the most preferable piling technique in an urban environment is bored steel tube piles for a heavy structure nearby a sensitive structure, and jacked precast concrete piles for a light structure at a distance from a sensitive structure. The most preferable method of retaining structure is diaphragm wall if the retaining structure will be constructed close to a sensitive structure and king post/berliner wall if it is constructed at a distance from the sensitive structure. However, the results are exclusively dependent on the relative rating of each criterion and should not be regarded as a definitive answer.

References

Ape Holland, *Diesel hammers.* Available: <www.apeholland.com/page/en/diesel-hammers> 2012-03-12

Arz P, *Piling, European practice and worldwide trends- KEYNOTE PAPER:Germany, Austria and Switzerland,* (1992), Bilfinger+Berger Bauaktiengesllschaft, Germany, Proceedings of a conference organized by the Institution of Civil Engineers, and held in London 7-9 April 1992, Thomas Telford

Avén, S. et. al. (1984) *Handboken Bygg – Geoteknik* LiberFörlag, Stockholm 1984. ISBN 91-38-06077-9, (In Swedish)

Balian, Goethert T, Gollub P, Nohl W A, (2009), *End bearing bored piles in sand stiffened by grouted base cells*, Ground Engineering March 2009, Vol 42, Thomas Telford, Great Britain 2009. ISSN 0017-4653

Bergersen N, Bye A, (2000) *Sörkedalsveien 6, Oslo - dyp utgraving*; Fjellsprengningsteknikk / Bergmekanikk / Geoteknikk 2000. Referat fra foredrag fra Fjellsprengningsdagen, Bergmekanikkdagen, Geoteknikkdagen, Oslo Norway, 23-24 November 2000. (In Norwegian)

Berkelaar Robert and Van den Bosch Melinda, (2004) *Obstacle Investigation RandstadRail in Rotterdam, The Netherlands, (Engineering geology for infrastructural planning in Europe - A European perspective),*Springer-Verlag Berlin Heidelberg 2004

Berglars, B. (2009) *Hur man väljer rätt Pålningsmetod*. Available: <www.palplintar.se/bilder/palmetod.pdf> 2012-05-29 (In Swedish)

Bowles, J.E.,(1996), *Foundation analysis and design* Fifth edition, McGraw-Hill, Peoria

Bredenberg, H.,(2000) *Stålkärnepålar – Anvisningar för projektering, dimensionering, utförande och kontroll,* Pålkommissionen rapport 97, Roland Offset AB, Linköping, (In Swedish)

Brassinga, HE, Hannink, G, (1997), *Construction of an underground station around an existing tunnel*, International conference on soil mechanics and foundation engineering, 14, Hamburg, Sept. 1997. Proceedings, vol. 2

Brinkman J, Van De Water J G, de Jong E, (2009), *Large diameter casing piles, design, testing and monitoring*, International conference on soil mechanics and geotechnical engineering, 17: The academia and practice of geotechnical engineering, Alexandria, 5-9 October, 2009. Proceedings, vol. 2

Burda M,. Wyplosz C. (2005). *Macroeconomics – A European text*. 4th edition. Oxford University Press, Oxford.

Buykx S. M., Delfgaauw S, Bosch J. W.,(2009), *Probalistic analysis of soil-Diaphragm wall friction used for value engineering of deep excavation, North/South metro Amsterdam*. Available: <www.ctta.org/FileUpload/ita/2009/papers/P-10/P-10-04.pdf> 2012-03-12

Cernica John N, (1995), *Geotechnical engineering: foundation design*, John Wiley & sons inc, New York

Clifford, J.W., (2001), *Managing geotechnical risk in the piling industry,* Master thesis, The Nottingham Trent University

Communities and local government, (2009) *Multi-Criteria analysis: a manual,* Department for Communities and Local Government: London, January 2009

Deckner, F., NCC Teknik, Stockholm, e-mail conversation, 2012-05-10

De Jong E, Bakker K J, Hoefsloot, (2010), *Statnamic load tests on large diameter casing piles execution and interpretation,* Deep Foundations Institute Europe, Proceedings from conference *Geotechnical challenges in urban regeneration*, London. Available: <www.vwsgeotechniek.nl/bin/ibp.jsp?ibpZone=S78_Publications&ibpDisplay=view&ibpPage=S78_FocusPage&ibpDispWho=S78_Publications&ibpDispWhat=zone&> 2012-04-20

Delgado C, (1992), *Piling, European practice and worldwide trends- KEYNOTE PAPER: Spain and Portugal,* Rodio Cimentaciones Especiales SA, Spain, Proceedings of a conference organized by the Institution of Civil Engineers, London 7-9 April 1992, Thomas Telford, Great Britain

Dundulis K, Kaczynski R, Krauzlis K, Zarzojus G, (2008), *Engineering geological properties of Vilnius and Warsaw region tills as building subsoils,* Geology, Vol 50 Supplement P.S11-S15, Vilnius

European Soils Bureau Network (2005), *Soil atlas of Europe*, European commission, Luxemburg

Fleming, K., Weltman, A., Randolph, M., Elson, K.,(2009) *Piling Engineering* Third edition. Taylor & Francis, Oxon

Foundation Engineering, *Deep Foundation.* Available: <http://osp.mans.edu.eg/deepfoundation/ch3.htm> 2012-02-26

Franki piling, *All Franki piles*. Available: < www.franki.com.au/PDF/All_FrankipilesPDFs.pdf> 2012-05-31

Fredrikssen J.K, Brendsrup J., Eriksen F. S., Gordon M. A., Knudsen C., Jörgenssen M. E., Möller H.M, (2003), *Engineering geology of Copenhagen*, Bulletin of Engineering Geology and the Environment, Volume 62, Number 3, Springer Verlag,

Garitte B., Arroyo M, Gens A., (2010) *Analysis of ground movements induced by diaphragm wall installation,* European conference on numerical methods in geotechnical engineering, 7, Trondheim, Norway, 2-4 June 2010. Proceedings 2010

Gatterman J. Berg T . Rodaz, W, (2001), *Modified instrumentation and results of stress and deformation monitoring at the new quay wall construction-Container terminal Altenwerder, Port of Hamburg,* Swets und Zeitlinger, Germany, ISBN 90 5809 197 X

GeoForum *Pile Classification System.* Available: <http://www.geoforum.com/info/pileinfo/classification.asp.> 2012-03-16

Grabrowska-Olszewska B., (1988) *Engineering geological problems of loess in Poland,* Engineering Geology, Elsevier science publishers B.V., Amsterdam

Grassmann S., Cramer B., Delise G., Messner J., Winsemann J., (2005), *Geological history and petroleum system of the Mittelplane oil field, Northern Germany,* International Journal of Earth Sciences,

Hellotrade, *Casing Oscillator Machine*, Available: <http://www.hellotrade.com/buma-ce/casing-oscillator-machine.html> 2012-03-15

Herbschleb, J., (2004), *Soil Investigation Aspects of a Complex Metro Project in Amsterdam - Engineering geology for infrastructural planning in Europe- A European perspective,*Springer-Verlag Berlin Heidelberg

Hercules, *Stålkärnepålar.* Available: <http://www.hercules.se/templates/GenericPage____5128.aspx> 2012-06-01 (In Swedish)

Hess, U., Pedersen, S. K., Ladefoged L. M., (1997), *Fixed link across the Öresund: Large excavation for tunnel and ramp in a freight railway yard*, International conference on soil mechanics and foundation engineering, 14, Hamburg, Sept. 1997. Proceedings, vol. 2

Hintze, S., Liedberg S., Massarsch R., Hansson M., Elvhammar H., Lundahl B., Rehnman S.E., (1997), *Omgivningspåverkan vid pål- och spontslagning,* Pålkommissionen rapport 95, Roland offset AB, Linköping, (In Swedish)

Holm, G., *Piling, European practice and worldwide trends- KEYNOTE PAPER:Sweden, Denmark, Norway and Finland,* (1992), Swedish Geotechnical institute, Sweden, Proceedings of a conference organized by the Institution of Civil Engineers, and held in London 7-9 April 1992, Thomas Telford

Huisman D.J., Weijers J.P., Dijkshoorn L.,and Veldkamp A., (2000), *Spatial prediction of the variability of the Early Pleistocene subsurface sediments in the Netherlands- Part 1:Heavy mineral,* Geologie en Mijnbouw / Netherlands Journal of Geosciences 79

Ian Farmer Associates, (2010), Borehole BH1A, job number: 20663

Jones, C., (2005), *Applied Welfare Economics,* Oxford University Press, Oxford.

Korff, M., Van Tol, A.F., de Jong, (2007), *Risks related to CFA-pile walls*, Available: <http://www.vwsgeotechniek.nl/bin/ibp.jsp?ibpZone=S78_Publications&ibpDisplay=view&ibpPage=S78_FocusPage&ibpDispWho=S78_Publications&ibpDispWhat=zone&> 2012-05-31

Kitching, R., (2009), *Poland's new pulse*, European Foundations spring 2009

Kullingsjö A., (2011), *Some Retaining structures*, Presentation, Geotechnics course BOM 045, Lecture 2011-04-12, Chalmers University of Technology, Göteborg, Sweden.

Kullingsjö A., (2004), *Deep excavations in soft clay: Numerical and classical calculations compared with field measurements*, NGM 2004. Nordic Geotechnical Meeting, 14, Ystad, Sweden, May, 2004. Proceedings, vol. 1, 2004

Langley, M.S.,(1980), *Noise and vibration from piling operations – Prediction of noise from piling operations* Directorate of Civil Engineering Services, Report PG9, London

Lindahl, H. (2001), *Samhällsekonomisk kostnadsanalys av vägbyggen*, Luleå Tekniska Universitet, ISSN: 1404-5508, (In Swedish)

Leszczynski M., (2009), *Barretted raft design for high-rise building applying 3D numerical tool*, International conference on soil mechanics and geotechnical engineering, 17: The academia and practice of geotechnical engineering, Alexandria, 5-9 October, 2009. Proceedings, vol. 2

Malolepszy Z., (2005), *Three dimensional geological model of Poland and its application to geothermal resource assessment*, Department of Fundamental Geology, University of Silesia

Matsson, B., (1988), *Cost-benefit kalkyler,* Esselte Studium, Akademiförlag, ISBN 91-24-16323-6, (In Swedish)

Matsson, B. (1979) , *Lönsamhet från samhällets synpunkt* Esselte Studium ISBN 91-24-15814-3, (In Swedish)

Molins, C., Ledesma, A. (2006), *Building a deep isolating wall by an existing rail tunnel,* Proceedings of the Institution of Civil Engineers, Geotechnical Engineering 159 July 2006 Issue GE3, Avaliable: <http://upcommons.upc.edu/e-prints/bitstream/2117/2868/1/molins_proceedings_geotechnical_1.pdf> 2012-03-05

Naturvårdsverket, (2004) *Naturvårdsverkets allmänna råd om buller från byggplatser* . NFS 2004:15, ISSN 1403-8234, (In Swedish)

NCC, *Infrastruktur/Referensprojekt/Götatunneln*, Available: <http://www.ncc.se/Infrastruktur/Referensprojekt/gotatunneln/> 2012-06-05

Olsson, C., Holm, G.,(1993) *Pålgrundläggning* AB Svensk Byggtjänst, Solna (In Swedish)

Puller, Malcom, (2006), *Deep excavation a practical manual 2nd edition,* Thomas Telford publishing, London, pp360-370, 67-77,103-106

Pålkommissionen, (2011), *Information 2011:1* Linköping, May 2011, (In Swedish)

Ruukki, (2012c), *Grova stålrörspålar för grundläggningsarbeten,* Product brochure, Rautaruukki Coorporation, Helsinki (In Swedish)

Ruukki, (2011a),*The future will be built on solid foundations,* Product brochure, Rautaruukki Corporation, Helsinki

Ruukki, (2012a), *RD-pålvägg teknisk info,* Product brochure, Rautaruukki Corporation, Helsinki (In Swedish)

Ruukki, (2011b), *RD-pålvägg,* Product brochure, Rautaruukki Corporation, Helsinki (In Swedish),

Ruukki, (2012b), *Kombivägg teknisk info,* Product brochure, Rautaruukki Corporation, Helsinki (In Swedish)

Rösand R.H., (2000), *Lydlös og vibrasjonsfri spuntning- Erfaringerfra regionsykehuset i Trondheim,* Fjellsprengningsteknikk / Bergmekanikk / Geoteknikk 2000. Referat fra foredrag fra Fjellsprengningsdagen, Bergmekanikkdagen, Geoteknikkdagen, Oslo, 23-24 november 2000. (In Norwegian)

Schuppener B.,(1997) *A trial sheet pile vibration at a retaining wall with insufficient stability,* International conference on soil mechanics and foundation engineering, 14, Hamburg, September 1997. Proceedings, vol. 2

Sebastian, R. (2011) *Low Disturbance Urban Projects Through Smart Manufacturing and Building Information Modelling* Proceedings of the 1st International Conference on Sustainable Intelligent Manufacturing, Leria, June 28 – July 1

SGF Jordförstärkningskommitté, *Metodblad – Jordförstärkningsmetoder.* Available: <http://www.sgf.net/web/page.aspx?refid=166> 2012-02-08 (In Swedish)

Sieminska-Lewandowska, A., (2005), *The displacements of anchored diaphragm walls*, Sudia Geotechnica et Mechanica, Vol. XXVII, No. 1-2, 2005. Available: <http://sgem.pwr.wroc.pl/n12-05/art15_no12_2005.pdf> 2012-04-27

Silence, P. G. M., (1992), *Piling, European practice and worldwide trends-KEYNOTE PAPER: Luxemburg, Belgium and Holland*, Delft Geoconsult BVBA, Belgium, Proceedings of a conference organized by the Institution of Civil Engineers, and held in London 7-9 April 1992, Thomas Telford

Stepkowska, E. T., (1979), *Typical cohesive soils in Poland*, Archiwum Hydrotechniki, Poland

Svensk standard, (1999), SS 025211, *Vibration och stöt-Riktvärden och mätmetod för vibrationer i byggnader orsakade av pålning, spontning, schaktning och packning*, standardiseringen i Sverige, Byggstandardiseringen, Stockholm, (In Swedish)

Sveriges Nationalatlas, *Berg och jord*, SNA, Bromma, 2009

Sällfors, G., (2001), *Geoteknik, jordmateriallära – Jordmekanik*, Chalmers University of Technology, Göteborg, (In Swedish)

Söderberg H., (2012), *Berg och betongtunnlar Bellevue- "Top-down" med sekantpålar* (NL 31), Proceedings from the conference *Grundläggningsdagen 2012*: *Geoteknik från stort till smått*, Stockholm, 8 mars 2012 (In Swedish)

Terzaghi, K., Peck, R. B., Mesri, G., (1996), *Soil mechanics in engineering practice*, Third edition, John Wiley & sons inc, New York

Thornton P, (1992), *Piling, European practice and worldwide trends- KEYNOTE PAPER:UK and Eire*, Cementation Piling and foundations, UK, Proceedings of a conference organized by the Institution of Civil Engineers, and held in London 7-9 April 1992, Thomas Telford

Thumann, V. M., Hannink G., de Doelder B. R., (2009), *Ground freezing and groundwater control at underground station CS in Rotterdam*, International conference on soil mechanics and geotechnical engineering, 17: The academia and practice of geotechnical engineering, Alexandria, 5-9 October, 2009. Proceedings, vol. 2

Topolnicki M., (1997), *Stabilization, of excavation bottom against uplift with Vibro Concrete Columns*, International conference on soil mechanics and foundation engineering, 14, Hamburg, Sept. 1997. Proceedings, vol. 2

Triantafyllidis T., Brem G., Vogel U., (1997), *Construction, monitoring and performance of the deep basement excavation at Potsdamer Platz, Berlin*, International conference on soil mechanics and foundation engineering, 14, Hamburg, Sept. 1997. Proceedings, vol. 2

Van Kampen I, de Jong E, (2010), *Vibrated VM-piles (H-beams with grout injections). Design, testing and monitoring*, Deep foundations Institute Europe, Proceedings from the conference *Geotechnical challenges in urban regeneration*. London. Available: <http://www.vwsgeotechniek.nl/bin/ibp.jsp?ibpZone=S78_Publications&ibpDisplay=view&ibpPage=S78_FocusPage&ibpDispWho=S78_Publications&ibpDispWhat=zone&> 2012-04-20

Van Paassen, B.P.H., van Dalen, J.H.,(2009), *High load tests on VM-piles for the quay wall Euromax* Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering, Alexandria, 5-9 October 2009

Vinnova, *Transporter för hållbar utveckling.* Available: <http://www.vinnova.se/upload/EPiStorePDF/vi-10-01.pdf> 2012-05-29 (In Swedish)

Vägverket (1997), *Vägverkets samhällsekonomiska kalkylmodell – Ekonomisk teori och värderingar.* Vägverket publikation 1997:130, Borlänge (In Swedish)

Vägverket (2008), *Samhällsekonomiska kalkylvärden.* Vägverket publikation 2008:67, Borlänge, (In Swedish)

Wood Tara, (1998a), *ICE QUARTERLY REPORT No. 3 April 1998-June 1998*, Stent Foundations

Wood Tara, (1998b), *ICE quarterly report No. 4 July 1998-September 1998*, Stent Foundations

APPENDIX 1

The monetary values for noise disturbance outdoors are presented for decibel-levels between 50 – 75 dBA

Valuation of noise outdoors, SEK / person and year (Vägverket 2008)

dBA	SEK
50	0
51	93
52	187
53	299
54	411
55	523
56	635
57	747
58	859
59	989
60	1120
61	1251
62	1400
63	1549
64	1736
65	1941
66	2184
67	2501
68	2949
69	3565
70	4331
71	5283
72	6384
73	7616
74	8923
75	10379

APPENDIX 2

The monetary values for noise disturbance indoors are presented for decibel-levels between 25 – 50 dBA. A representative wall reduction value of 25 dBA has been used.

Valuation of noise indoors in SEK / person and year (Vägverket 2008)

dBA	SEK
25	0
26	149
27	299
28	448
29	597
30	765
31	933
32	1101
33	1288
34	1475
35	1680
36	1885
37	2091
38	2333
39	2595
40	2893
41	3267
42	3752
43	4424
44	5320
45	6496
46	7915
47	9576
48	11424
49	13385
50	15550

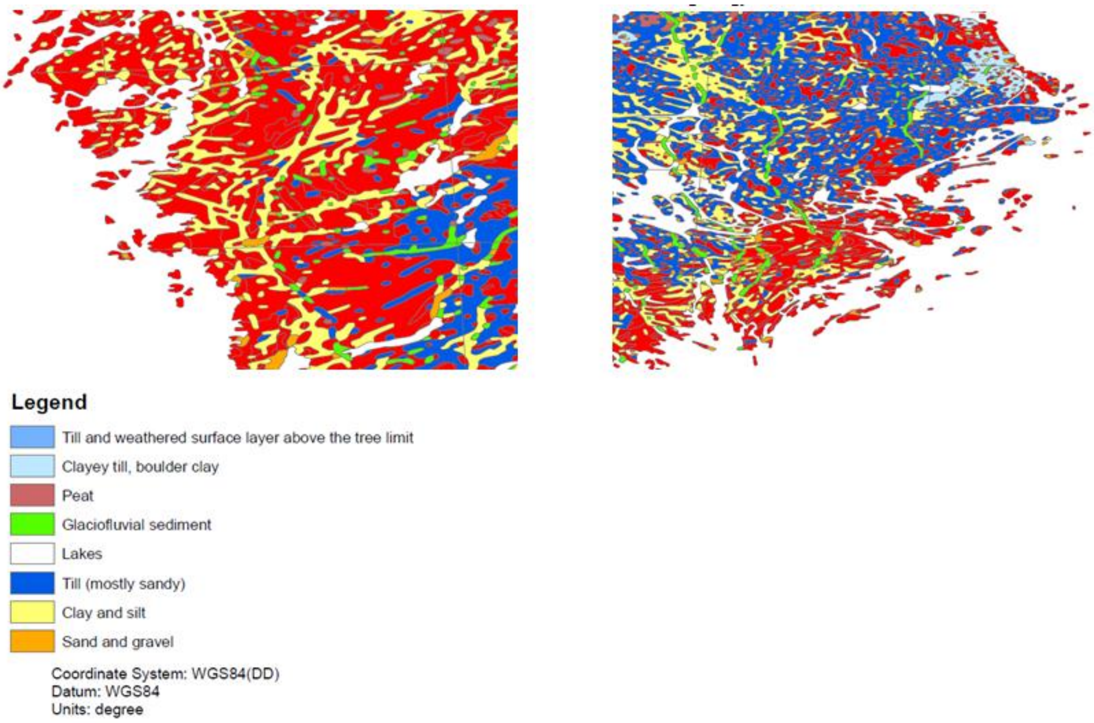
APPENDIX 3

The values presented in Appendix 1 and Appendix 2 can be combined if a representative wall reduction of 25 dBA is used.

Valuation of combined noise indoors and outdoors in SEK / person and year (Vägverket 2008)

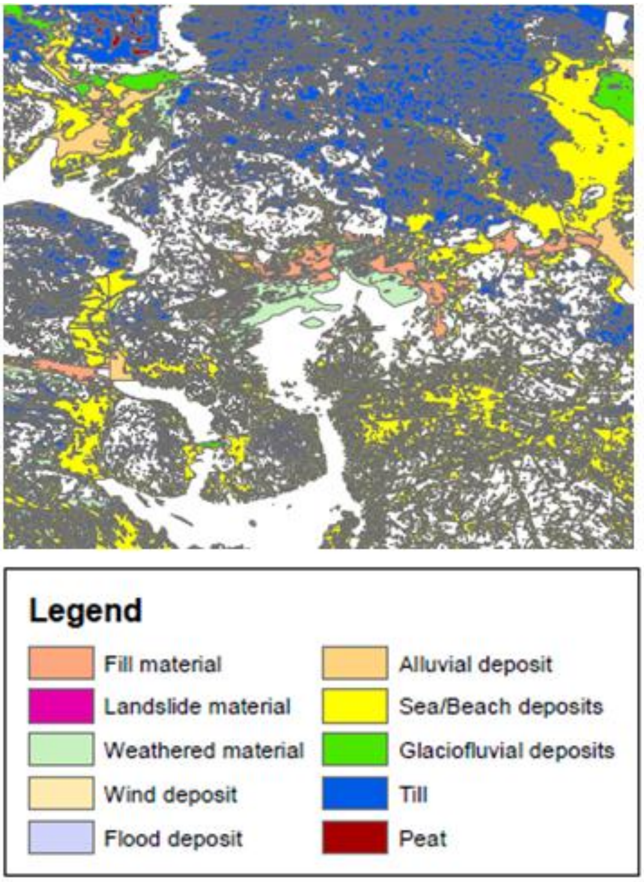
dBA	SEK
50	0
51	243
52	485
53	747
54	1008
55	1288
56	1568
57	1848
58	2147
59	2464
60	2800
61	3136
62	3491
63	3883
64	4331
65	4835
66	5451
67	6254
68	7374
69	8886
70	10827
71	13198
72	15961
73	19041
74	22308
75	25929

APPENDIX 4

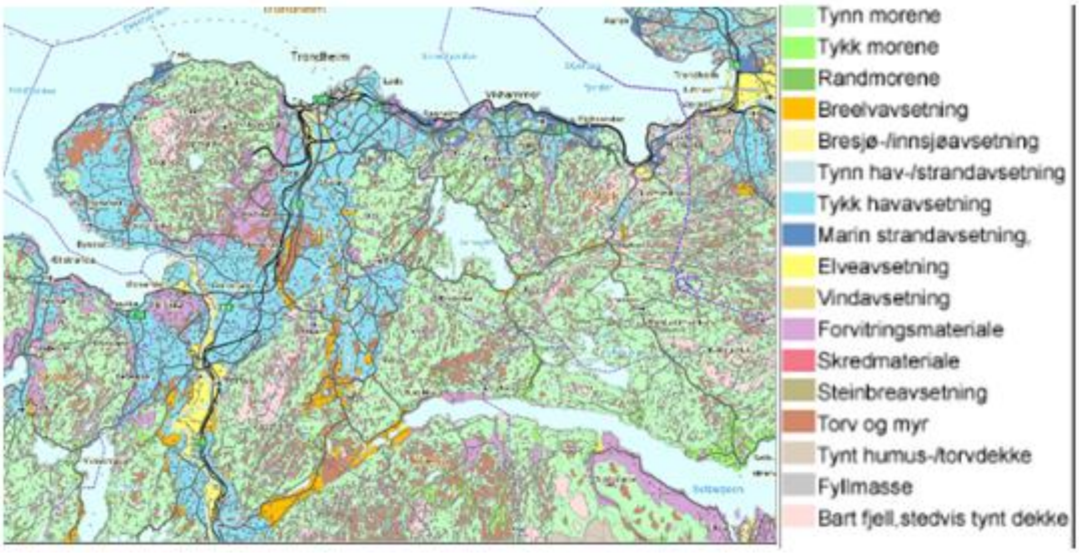


Surface geology of the Gothenburg and Stockholm area. Used and/or reproduced with the permission of the geological survey organisation rightholders who comprised the OneGeology-Europe project. All Rights Reserved.

APPENDIX 5

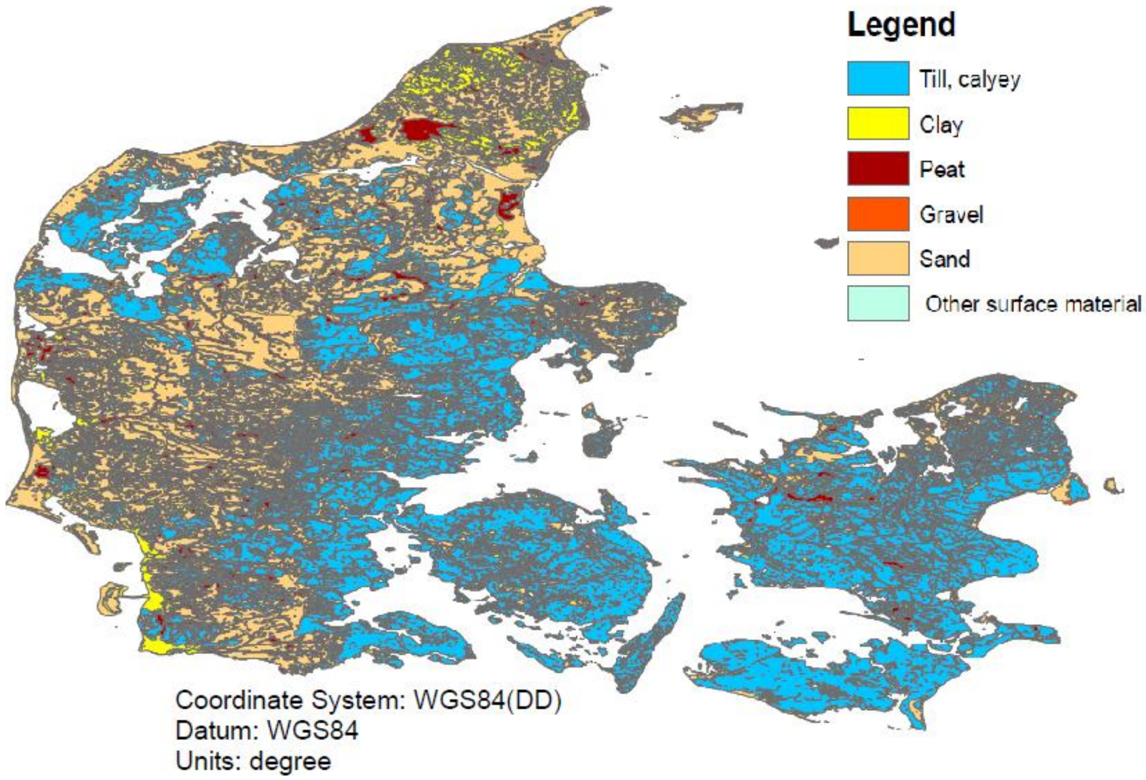


Surface geology of the Oslo area. Used and/or reproduced with the permission of the geological survey organisation rightholders who comprised the OneGeology-Europe project. All Rights Reserved.



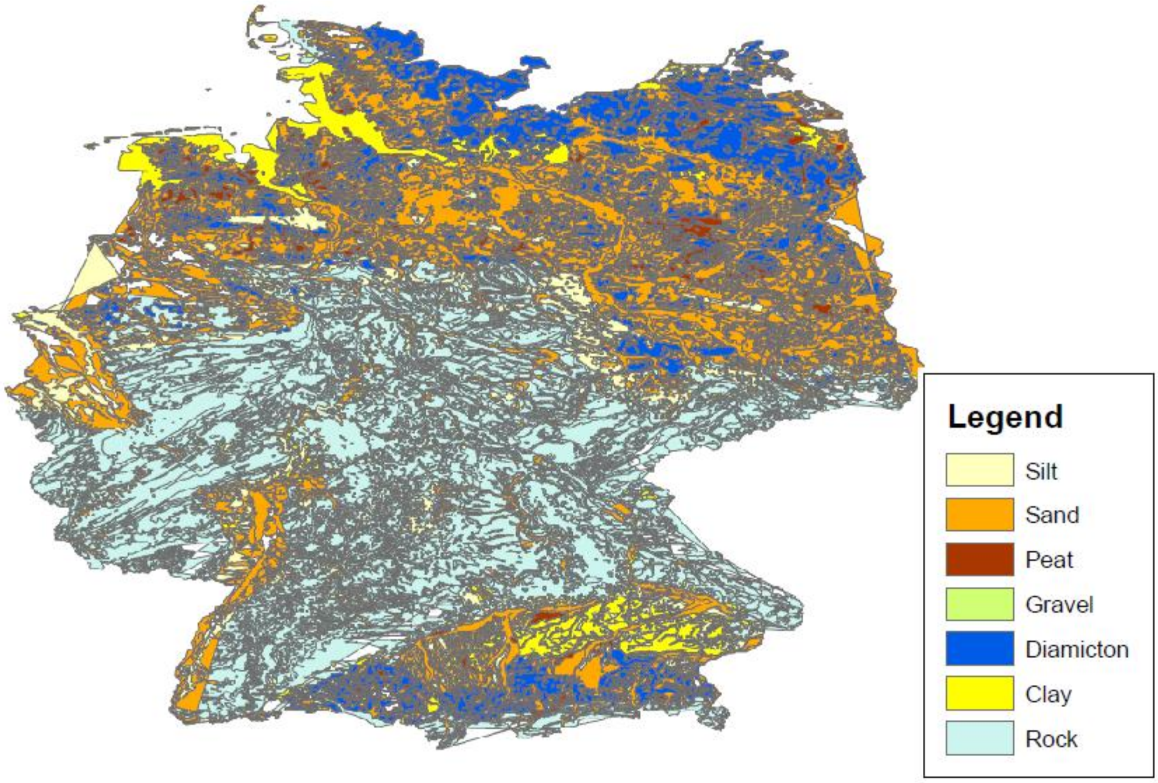
Surface geology of the Trondheim area. Used with the permission of the Geological Survey of Norway. All Rights Reserved.

APPENDIX 6



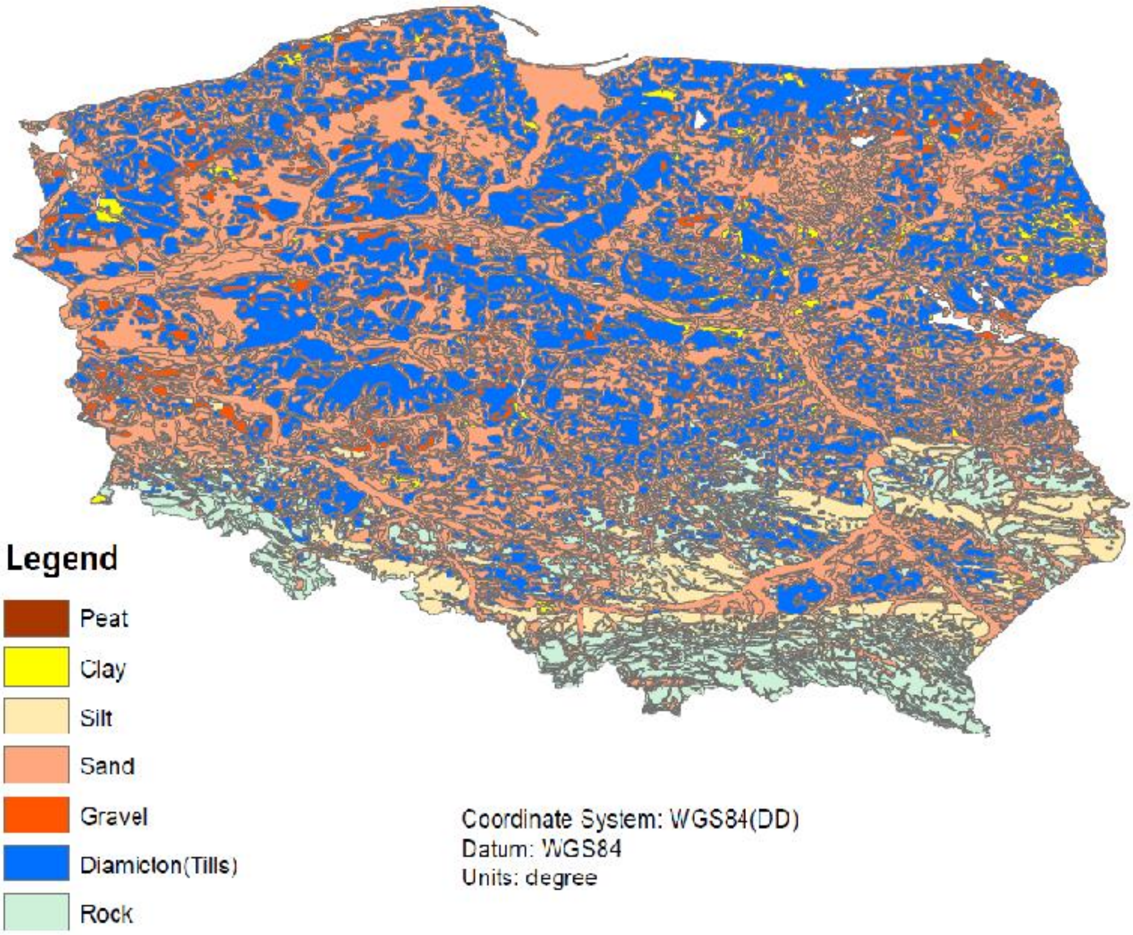
Surface geology of Denmark. Used and/or reproduced with the permission of the geological survey organisation rightholders who comprised the OneGeology-Europe project. All Rights Reserved.

APPENDIX 7



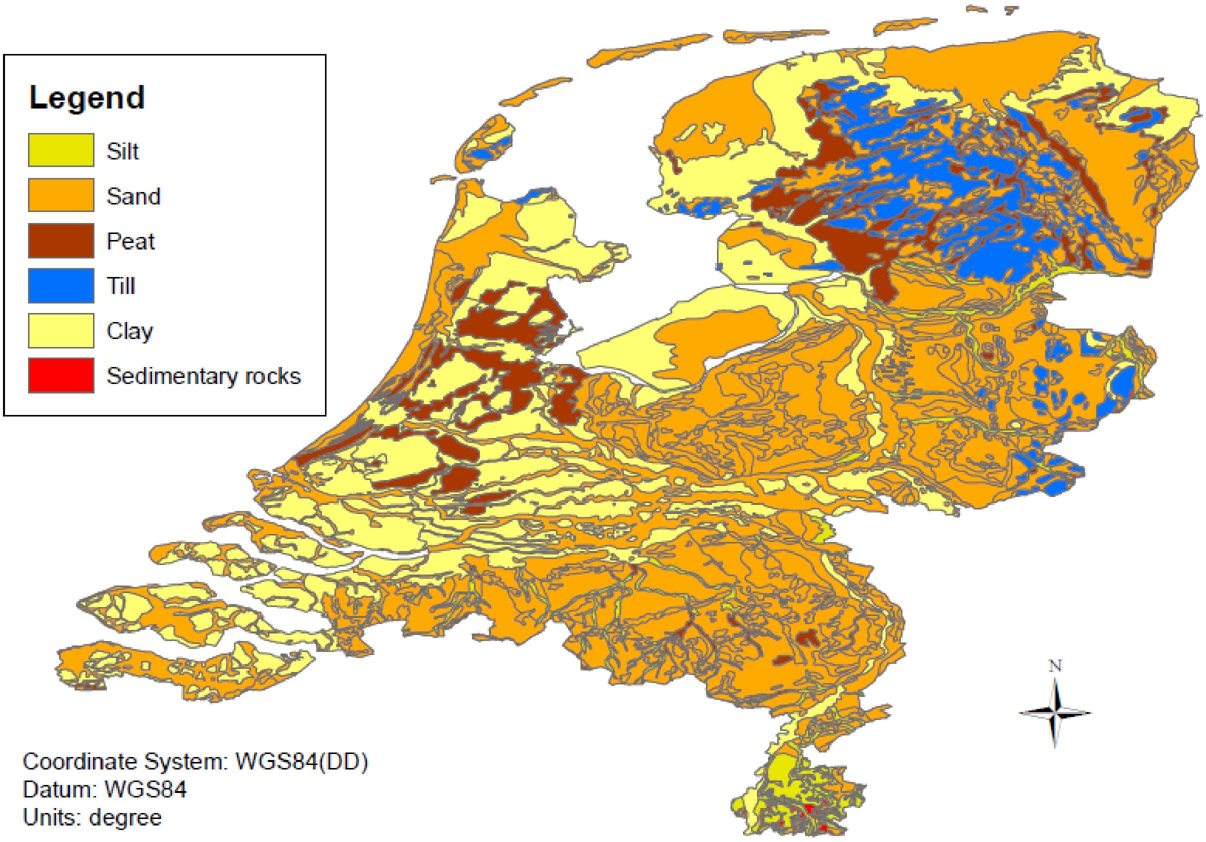
Surface geology of Germany. Used and/or reproduced with the permission of the geological survey organisation rightholders who comprised the OneGeology-Europe project. All Rights Reserved.

APPENDIX 8



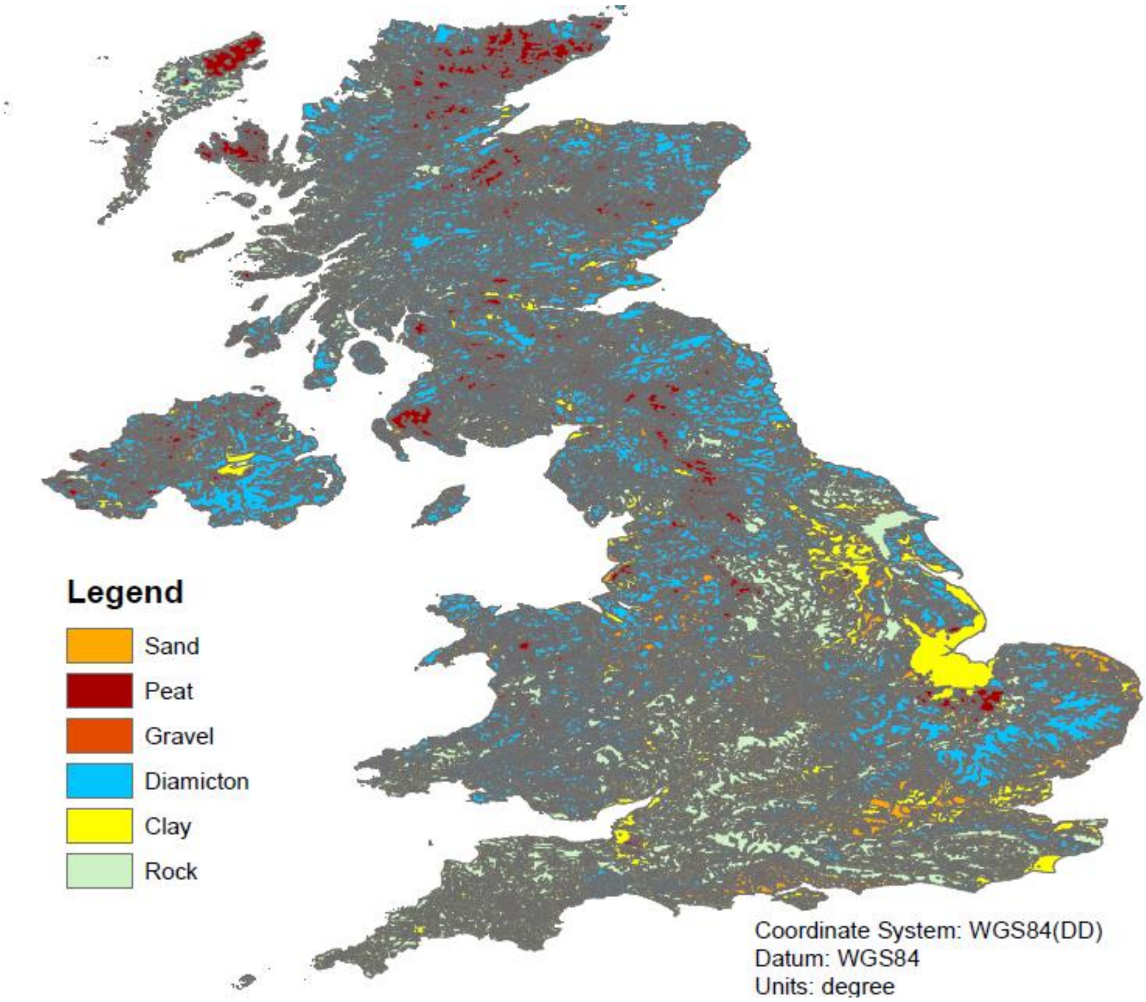
Surface geology of Poland. Used and/or reproduced with the permission of the geological survey organisation rightholders who comprised the OneGeology-Europe project. All Rights Reserved.

APPENDIX 9



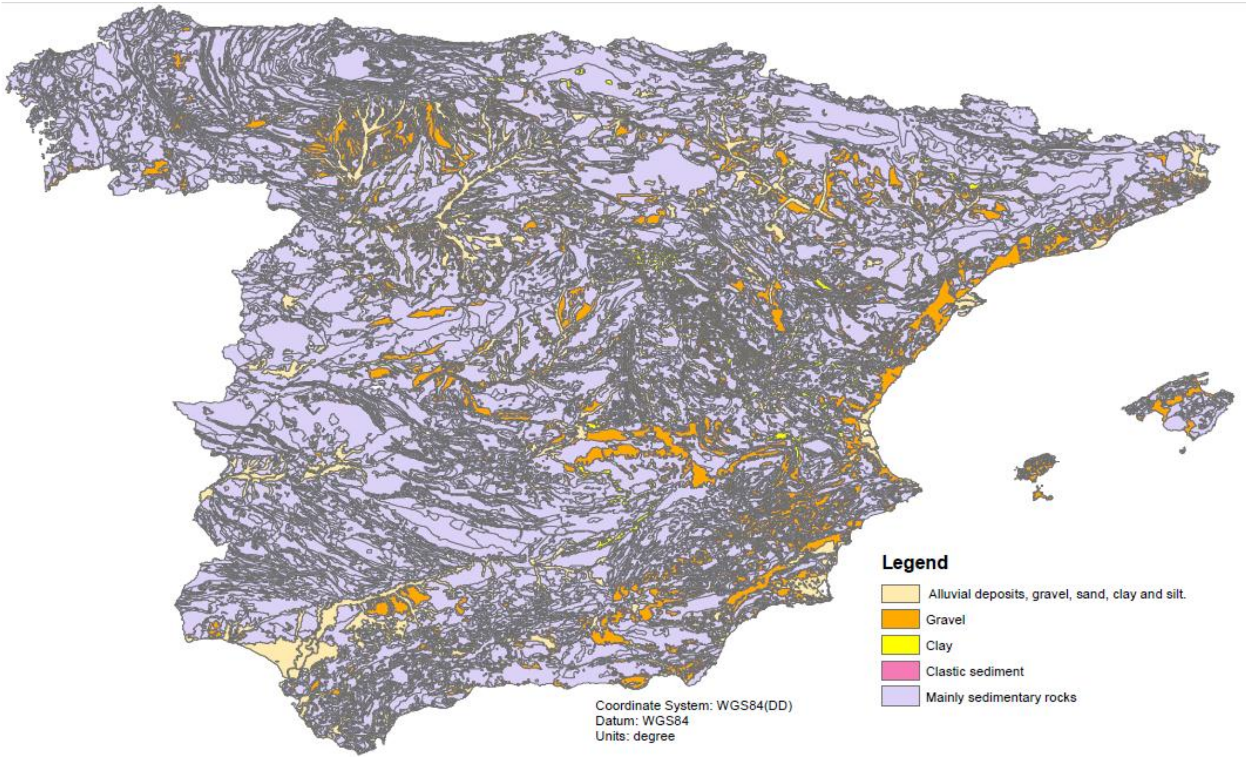
Surface geology of the Netherlands. Used and/or reproduced with the permission of the geological survey organisation rightholders who comprised the OneGeology-Europe project. All Rights Reserved.

APPENDIX 10



Surface geology of the U.K. Used and/or reproduced with the permission of the geological survey organisation rightholders who comprised the OneGeology-Europe project. All Rights Reserved.

APPENDIX 11



Surface geology of Spain. Used and/or reproduced with the permission of the geological survey organisation rightholders who comprised the OneGeology-Europe project. All Rights Reserved.