LATERAL EARTH PRESSURE, SLOPE STABILITRY AND PILE SHEETING

Sheet piled constructions.

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Preface

This individual master thesis is worked out as a conclusion of the master studies program at the institute of engineering at the university of Agder. This project was taken as a concluding assignment for the subject BYG508 in the fourth and final semester, in spring 2023. The main purpose of this study is to attain a deeper understanding on lateral earth pressures, slope stability and sheet pile walls. This project deeply studied on how the constructions on slope areas can be kept stable and durable. The effect of water pressure on the shear strength of soils and reasons behind landslides and rock falls are investigated and analyzed. Finally, I would like to thank my supervisor from UIA professor Songxiong Ding for his kind and professional supervision, without his help this project would not be achieved.

Summary

This master thesis studies about how the sheet pile walls resist the lateral earth pressures on slope area. Sheet pile walls are one type of construction elements that are built by a continuous interlocking of pile segments side by side to each other. They are embedded in the soils to provide lateral supports by resisting the horizontal pressures released from sea waters, or any other earth masses. Sheet pile walls are flexible retaining systems therefore they can tolerate larger horizontal deformations compared to other types of walls such as stone and concrete. Sheet piles are made up of different types of materials in different forms and sizes. Although steel sheet pile has a higher rate of corrosion property, they are the most used due to its availability in the markets all over the world and its super resistance while being installed by hydraulic pressures. PVC can substitute steel sheet piles concerning environmental impact, sustainability, and maintenance requirements against corrosion. The thesis question is (How can one keep the stability and durability of a sheet piled wall construction on slope areas?) and some sub questions that widen the thesis question are listed as

1. What is the concept of pile sheeting?
2. How can one ensure the stability of sheet piled constructions?
3. How can one select the size and material type of a sheet pile for a certain construction?
4. How can one possibly keep the durability of sheet piled wall constructions?

The maximum moment induced by the latera earth pressures, the section of modulus of sheet piles are the key expression for selecting a sheet pile size. However, the environmental conditions, strength sustainability and maintenance requirements have a great role in selecting material type of the sheet piles.

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1. Introduction

Environment conservation and reducing of deaths due to natural and man-made accidents should be mandatory responsibilities for every scholar or layman. Overflooding, land sliding, and erosion are some examples of destructive occasions that lead to a considerable material loos or death of many people all over the world every year. Here the question is how can one stabilize the environment and reduce these accidents? The solution to such problems is that the soil mass on slopes should be retained, the flood speed should be slowed down or infiltrated into the ground to enrich the environment with vegetation. Rock falls from slopes to public roads are also other challenges.

Figure 1.1 Rock falls at Larvik in Norway 13 December 2019

However, how can such types of instabilities be secured before their actual occurrence?
Reducing the steepness of slopes of the mountainous areas adjacent to the public roads is helpful to avoid a sudden fall of racks. In addition, another effective technique of retaining the slope's earth mass is by installing supportive vertical walls. These walls are thin sheets made up of either steel, concrete, plastic composites, or wood depending on their application areas [1]. In addition to the natural accident like rockfalls, land sliding and over flooding, there are also other manmade challenges such as deep excavations on construction and mining sites that displace huge soil mass. This displaced soil around the deep excavation especially if it is a soft soil, it can slide down to the work site and cause life loss, material damage, and economic and time loss for reworking. Therefore, what measures must be taken to avoid such consequences?

While constructing bridges which require supporting columns whose bases are built under the water level, temporary watertight enclosure structures are built in which the water is then pumped out to expose a dry working surface. Sometimes it is required to build a construction in the middle of a water body. To do this, a dry surface must be exposed to start the foundation.
However, how can the lateral pressure forces from the nearby waters be protected to give a safe working condition for the staff?

Figure 1.2 a building bound by water body [2].

Seawater and polluted water are aggressive to metals like steel when they are permanently installed underwater. What alternatives can be done to avoid frequent maintenance of weather and physiochemical related challenges?

2. Social perspective

Steel sheet piles are the most used for temporary and permanent later earth pressure supports. Steel sheet piles are a sustainable alterative specially when they are used as temporary solutions. They can be extracted at any appropriate period and again be used on another construction sites. Most importantly, the steel sheet piles can completely be recycled after their final lifespan.

Steel sheet piles are installed in some terrains to protect earth mass loss due to erosion. They are installed by the waterfront sides where the waves and tidal splash forces strongly destruct the backshore sides. A conventional way of protection in these types of areas is performed by installing seawalls of steel sheet piles. Steel sheet pile walls are generally satisfactory regarding their mechanical strength. However, at a critical stage of corrosion they cause serious problems. The rate of corrosion depends on loading and location specificity. Regarding the environment factor dry and wet soil or fresh natural water or salty water influence the rate of corrosion differently. I addition the physiochemical factors like PH value, electrical resistivity, redox potential difference, and electrical potential at the earth to metallic connections and the length of exposure time affect the amount of mass loss due to corrosion [3].

The common way of ensuring durability sheet piled walls is performed by applying a secondary corrosion protection, for instance by coating protective films, cathodic protection, or allowance of corrosion loss. Out of these three protecting methods the corrosion loss allowance is considered as the most effective measure to extend the lifespan of the sheet piled constructions. Nevertheless, this technique requires enlarging of the thickness of the steel material greater than its design or its theoretical size. This results to the increase of the steel material consumed that make its unsustainable solution [3].

Concerning the problems related to use of steel sheet pile walls in aggressive zones, the polyvinyl chloride (PVC) is analyzed to be the substitute material for steel structures in wet environment, waterfront, or marine areas. PVC provides a significant mechanical strength and are highly resistant to corrosion, UV radiation and similarly other environment related challenges.

2.1 Sheet piling and UN's sustainability goals:

One of the most effective objectives of the sheet pile walls is to protect the natural mass and particularly the soil. This may be done by installing sheet piles to reduce the speed of the floods on valleys or by completely blocking and then infiltrating the running water again into the soil. Infiltrating the running water reserves the nature and enriches the earth's ability to grow vegetation. Vegetations are further used as food to animals and people whose lives are dependent on agriculture. Thus, installing sheet pile walls reduces the number of deaths due to hunger and draught. Another advantage of vegetation is that it infiltrates much water and avoids overflooding that can destruct slum houses and farms and thus reduces the number of people affected by water related disasters.

Sometimes sheet pile walls are installed under suspected landslides or rack falls to safeguard the lives of people during the occasion of actual accident. In construction and mining sites where there is a deep excavation, huge amount of earth mass is displaced. This displaced mass is required to be supported by installing sheet pile walls to avoid the sliding of the displaced soil to the excavated depths where it

can harm staffs' lives and machineries. These priorly installed sheet piles save people's lives and economic loses and time for reworking.
The above-mentioned objectives of sheet pile walls fulfill one of the UN's 2030 sustainability development sub-goals. UN's sustainability sub-goal 11.5 verifies ''By 2030, significantly **reduce the number of deaths** and **the number of people affected and substantially decrease the direct economic losses relative to global gross domestic product caused by disasters, including water-related disasters**, with a focus on protecting the poor and people in vulnerable situations.'' [4].

Currently the necessity of the environmental impact is being much important in addition to the utility features and financial matters for selecting a product. PVC sheet pile has a lower density compared to steel sheet piles. Thus, when PVC is used as a substitute of steel sheet pile for supporting the same lateral earth pressure on waterfront walls, a smaller amount of PVC mass is required to cover the same project. This smaller amount of PVC material contributes to minimizing the environmental impacts of steel sheet piles during both the production stages of steel sheet piles and transportation of the final products to the construction sites [3].
Unlike the steel sheet piles, the PVC sheet piles do not need corrosion allowances which significantly reduces the total amount of material required, more specifically in regard with the corrosive marine and waterfront conditions. This further reduces the environmental impact that occurs in production of steel sheet piles. When steel seawalls are deeply corroded, they must be replaced. The replacement old seawalls by new steel sheet piles leads to additional environmental impact through all the life cycle phases of the steel products [3].

3. Theoretical background:

3.1 Pile sheeting theory:

The rapid increase of populations in the urban areas is leading to an increase in the demand for residential areas and transportation systems in general. However, the flat and useable areas are limited. Therefore, an effective use of the existing areas in cities would become necessary. The new residence areas are often being practical by excavating dangerous slopes that may result in large settlements and slope slips while constructing a foundation. To prevent and avoid such deformation problems, lateral support is used. One of the most common lateral supports is the sheet pile wall (SPW). SPWs are preferred not only because they are economical but also, they are waterproof, flexible structures, easy to mantle and take little space [5].

The origin of sheet piling was started about one hundred ago. Mr. Larssen who was a state chief Engineer in Bremen, Germany was the first person ever to develop a U-shaped steel sheet piles having riveted interlocking system in 1902. 1914 the riveted interlocking system was modified to interlocking adjacent sides of the sheet piles as it is available and used now. The U section profiles are water-tight nevertheless, because of their structural potential concerns, the Z-shaped profiles were developed.

3.2 Sheet pile wall:

Sheet pile walls are one type of construction elements that are built by a continuous interlocking of pile segments side by side to each other. They are embedded in the soils to provide lateral supports by resisting the horizontal pressures released from sea waters, or any other earth masses. Sheet pile walls are flexible retaining systems therefore they can tolerate larger horizontal deformations compared to other types of walls such as stone and concrete. Sheet piles are widely used for protecting waterfront structures for erosion protection. In addition, sheet piles are mainly used for stabilizing slope areas against flood and other physical forces that can wash out the soil and other masses from the area. Other important role of sheet piles is to retain loose soils around a foundation either temporarily or permanently [1].

3.2.1 Advantages of sheet pile walls:

Sheet pile walls have several advantages and some of them are described as follows.

Sheet piles are not a use and throw material, they can be recycled and reused many times. Pile walls can be designed in a variety of lengths, thickness sizes and different steel type options. Their functions can be both for permanent and temporary structures. Regarding the installation systems, it can be performed by silent and vibration-free technics if the soil they penetrate is not rocky. Sheet pilling work does not take long time and it is cost effective compared to other types of walls such as secant walls. Cofferdams can have different shapes by the help of sheet piles although the circular and rectangular are among the most cost effective and most used. Sheet pile walls provide watertight joints, and they are durable even in all marine and coastal areas. Anchored sheet piles give a larger flexibility with regard installation purposes [6].

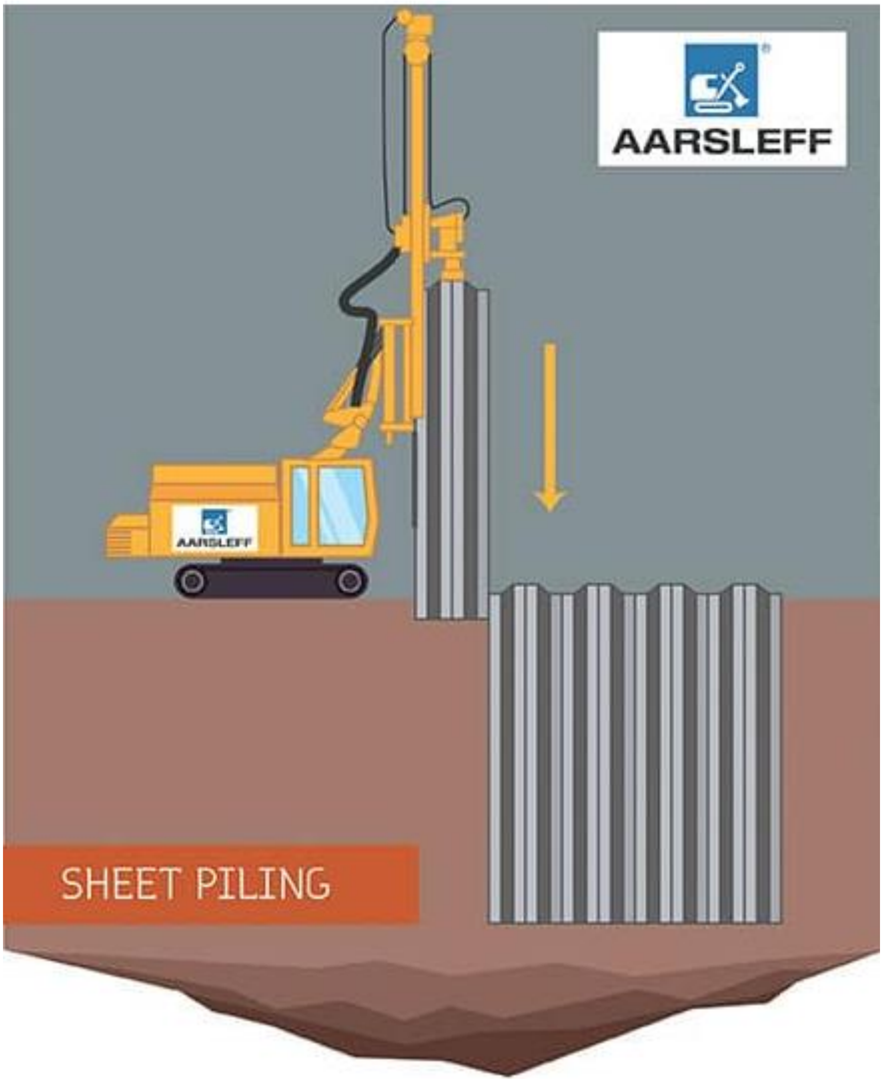


Figure 3.1 sheet piling [6].

3.2.2 Methods of construction using sheet pile walls:

While using sheet pile walls there are two methods. The first method is that the sheet pile is driven into the ground on the construction site and then the backfill is placed on the land side. On the second method, the sheet pile is first be driven into the ground and then the soil on the waterside (in front of the sheet pile) is dredged. The soil type that is used as a back-fill behind the sheet pile wall is always granular. However, the type of soil of the surface of the dredge line can be clayey or sandy. Dredge line sometimes called mudline is the upper surface of the soil on the water side. According to (Tsinker,1983) construction methods are generally classified into two types as backfilled structure and dredge structure [7].

3.2.3 Sequence of construction methods:

For a backfilled structure the sequence of construction is shown in the figure below. The order is listed as:

1. First dredge or excavate the in-situ soil back and in front of the structure.
2. Next drive in the sheet piles.
3. Third step is to backfill up to the anchor level and fix the anchor system, otherwise skip over this step if the system is cantilevering type.
4. Finally backfill up to the maximum height of the wall [7].

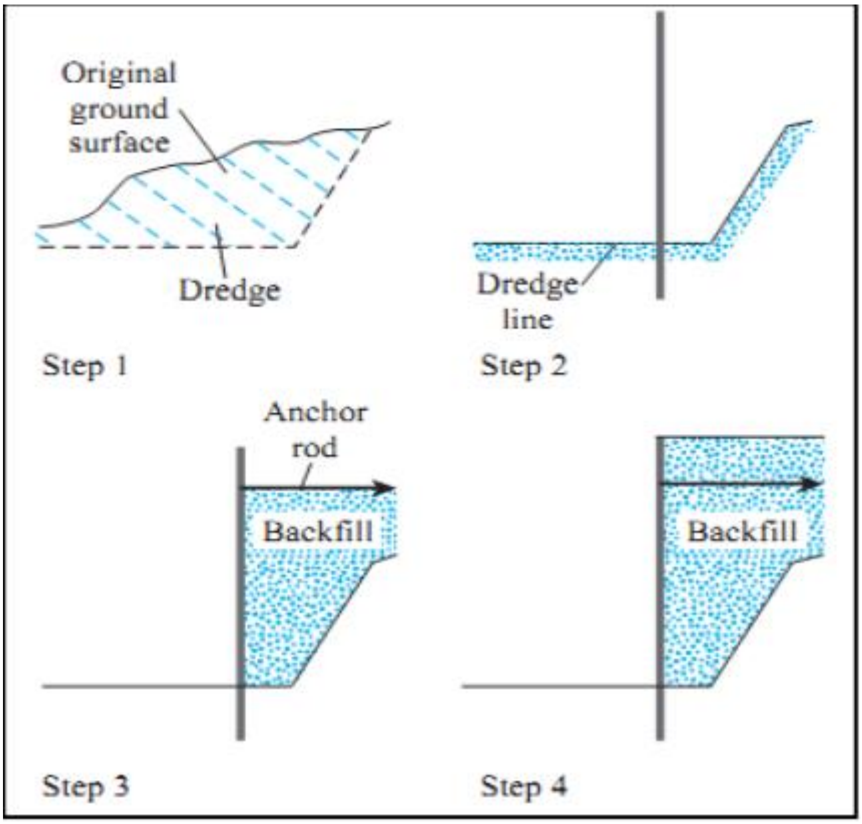


Figure 3.2 Sequence of construction for backfilled method [7]

For a dredge structure the sequence of construction as listed follows and are diagrammatically shown in the figure below.

1. Firstly, drive in the sheet piles.
2. And then backfill up to the level of the anchor and adjust the anchor system if it shall be the anchor system, otherwise skip over this step.
3. Next fill up to the level of the wall.
4. Dredge or excavate the front side the sheet pile (the water side) [7].

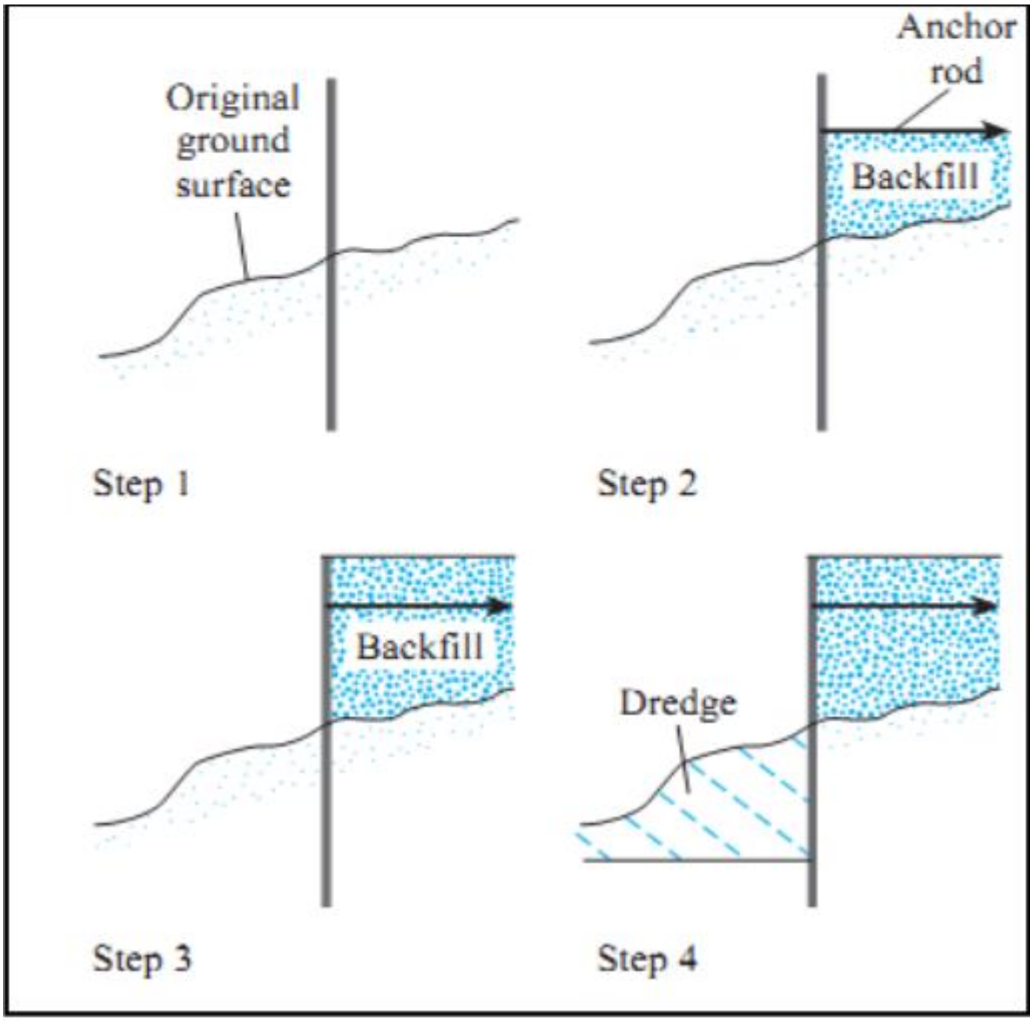


Figure 3.3 Sequence of construction for a dredge structure [7]

3.3 The earth pressures:

All structural elements in the soil are subjected to earth pressures. In 1984, Aarhaug described and classified these pressures as normal and shear stresses. These two types of stresses interact with in the soil and the structural element. When the soil exerts a pressure in the horizontal direction, this pressure is known as a lateral pressure. When a sheet pile wall is driven into the ground and if the soil on one side is excavated, the pressure exerted to the wall from the excavated side will dramatically be reduced.

When a wall in the form a sheet pile penetrates a soil or supports a backfill, it experiences a force, and this force is called the earth pressure. Depending on their magnitude and direction there are three types of earth pressure forces as listed below:

3.3.1 At rest earth pressure:

At-rest earth pressure is the force exerted by a backfill soil on a wall when the wall is rigidly fixed and shows no movement to the left or to the right. The resultant movement of the soil mass is at equilibrium state. At rest pressure is denoted by P and is expressed as:

$P = \frac{1}{2} * \gamma * H^2 * K_0$ (3.1)

Where:

H= height of the wall,

γ= unit weight of the backfill soil,

K_0= coefficient of the earth pressure at rest.

From the theory of elasticity, the at rest earth pressure coefficient can be calculated as:

$K_0 = \frac{v}{1-v}$ (3.2)

Where v is the Poisson ratio of the backfill soil.

Alternatively, Jack in 1944 demonstrated a good approximation of K_0.

According to Jacky:

$K_0 = 1 - sin\phi$ (3.3)

Where ϕ is the internal friction angle of the backfill soil [8].

The following table shows the Poisson's ratio value for different soils and their respective calculated K_0 values.

Table 3.1 poisson's ratio value of different backfill soils [8]

Backfill soil type	Poisson's ratio (v)	$K_0 = \frac{v}{1-v}$
Loose sand	0.2-0.35	0.25-0.54
Dense sand	0.3-0.4	0.43-0.67
Sandy soil	0.15-0.25	0.18-0.33
silt	0.3-0.35	0.43-0.54
unsaturated clay	0.35-0.4	0.54-0.67
Saturated clay	0.5	1
Clay with sand and silt	0.3-0.42	0.43-0.72

3.3.2 Active earth pressure

This pressure is the earth pressure force the backfill soil exerts on the wall as the wall moves away from the backfill. Active earth pressure is denoted by P_a.

According to Rankine and Coulomb Pa is obtained by:

$P_a = \frac{1}{2} * \gamma * H^2 * K_a$ (3.4)

Where Ka is the coefficient of the active lateral active pressure which is expressed differently in different approaches.

3.3.3 Passive earth pressure:

This type of earth pressure on the other hand is the force the soil exerts on the wall when the wall moves toward the soil. As the wall moves closer and closer towards the soil the passive earth pressure increases and at it moves far away from the soil the active earth pressure decreases. However, in the case of at-rest earth pressure there is not any movement of the wall and as a result there is no difference in earth pressure [9].

According to Rankine and Coulomb theory of lateral earth pressure, the passive earth pressure is obtained by:

$$P_p = \frac{1}{2} * \gamma * H^2 * K_p \quad (3.5)$$

Where Kp is the coefficient of the passive earth pressure and is obtained differently from different theories.

The following figure shows the fundamental earth pressures sketch of wall movement.

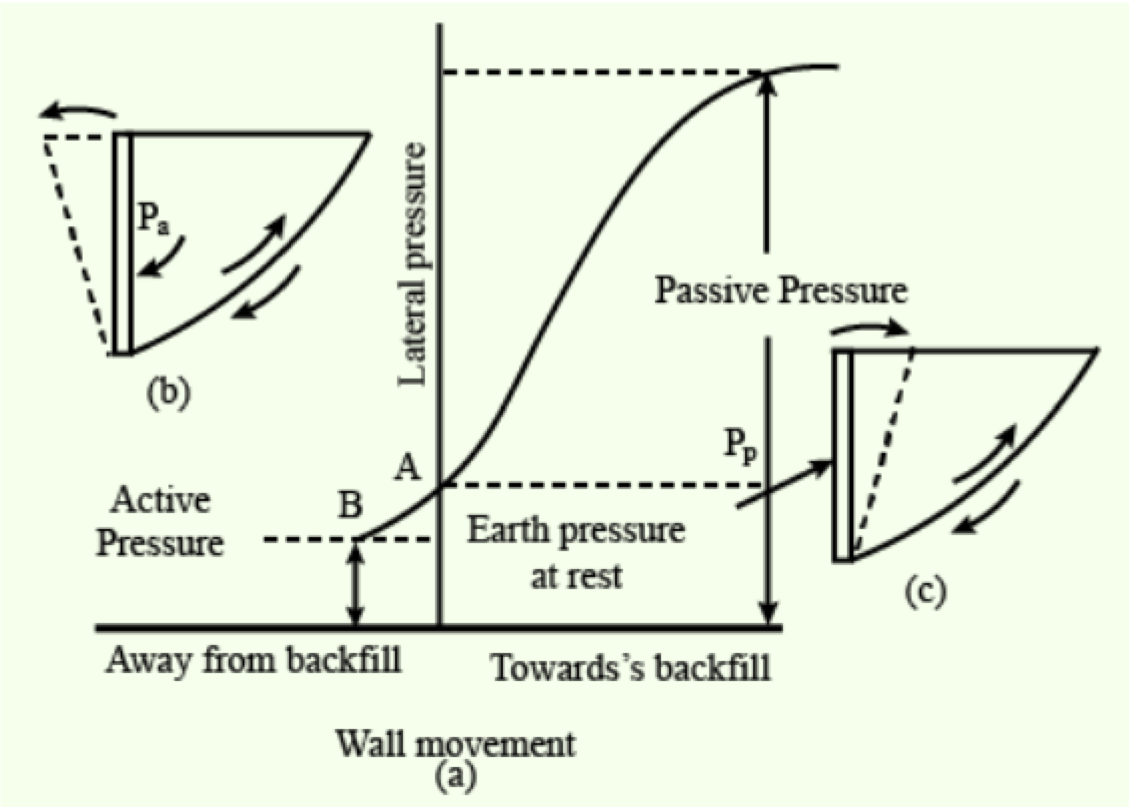


Figure 3.4 pressure direction vs wall movement sketch

N.B: **Passive earth pressure is always greater than the active earth pressure and the earth pressure at rest because in passive state the structure becomes the actuating element, and the soil becomes the resisting element to maintain the stability of the wall.**

In both active and passive earth pressures there is a deformation while in the case of at- rest earth pressure the deformation is negligible. The following figure [10] shows the relationship of earth pressure and deformation in all three earth pressures. The vertical axis represents the earth pressure, and the horizontal axis represents deformation.

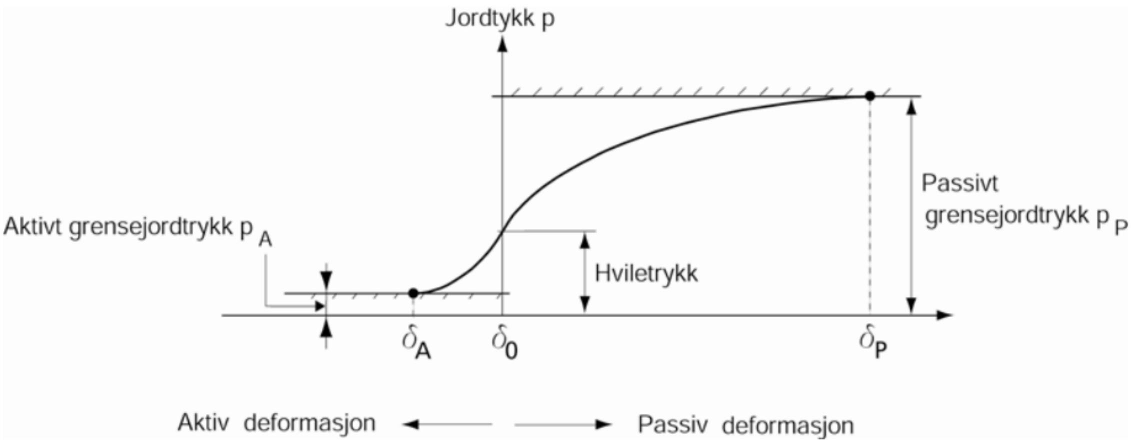


Figure 3.5 earth pressure vs deformation [10]

The table below shows the minimum wall movement magnitudes for different backfill soil types required to reach the maximum passive earth pressure and the minimum active earth pressure conditions. The result was found from the experimental data and finite element method investigated by Clough and Duncan in 1991 [9].

Table 3.2 approximate wall movement to height ratio [9].

Type of backfill soil	Value of Δ/H	
	Minimum Active	Maximum passive
Dense sand	0.001	0.01
Medium dense sand	0.002	0.02
Loose sand	0.004	0.04
Compacted silt	0.002	0.02
Compacted lean clay	0.01	0.05
Compacted fat clay	0.01	0.05

Where Δ, is the movement of the top of wall to be minimum active or maximum passive pressure. The movement can be lateral translation or tilting. H is the height of the wall.

3.4 Lateral earth pressure theory for sheet pile wall (SPW)

To determine the effective lateral pressure forces that are acting on the sheet pile walls, four different classical theory approaches are used depending on their respective assumptions.

3.4.1 The Rankine theory:

The Rankine theory describes the earth pressure as a function of stresses, and it assumes that the resultant force is parallel to the slope of the backfill.

However, sheet pile walls are flexible, as a result they produce a large horizontal displacement. This leads to the occurrence of slip between the soil and the wall. Because of the soil slip, a frictional resistance force is developed to reduce the active lateral pressure coefficient (Ka). The Rankine theory assumes a smooth or frictionless wall where friction ratio (r=0) as well as a homogeneous and cohesionless soil (c=0). Consequently, the Rankine method is not recommended for such cases unless a conservative solution is desired. For the passive failure case this yields a failure surface at an angle of (45-ϕ/2). The Rankine passive and active pressure coefficients are calculated as shown in equations 3.6 and 3.7 [11] [12].

$$K_a = tan^2\left(45 - \frac{\phi}{2}\right) \tag{3.6}$$

$$K_p = tan^2\left(45 + \frac{\phi}{2}\right) \tag{3.7}$$

The passive earth pressure is determined as a function of Ka and Kp.

$$\delta_p = \delta_v * K_p + 2c\sqrt{K_p} \tag{3.8}$$

However, c is assumed to be zero in the Rankine method therefore the passive earth pressure according to this theory will be:

$$\delta_p = \delta_v K_p \tag{3.9}$$

Rankine theory is a special case of the coulomb theory with an exclusive assumption of some factors that are accounted in the coulomb method.

3.4.2 The Coulomb theory:

Coulomb method is one of the earliest theories for estimating earth pressures against walls. This theory assumes that the soil failure occurs as a wedge experiencing translation like a firm body in the same direction with a shear plane. The coulomb method threats the soil as an isotropic and considers both the internal friction angle (ϕ) and the friction that occur due to the soil slip against the (wall friction angle δ). As this theory accounts these factors it is obvious that the coulomb method is a recommended method compared to the Rankine method. According to coulomb theory the resultant passive pressure force (P_p) is determined as a function of K_p, ɣ, and H [13].

$$P_p = \frac{1}{2} * K_p * \gamma * H^2 \tag{3.10}$$

Where:

Pp = the passive pressure force,

ɣ = the unit weight of the backfill soil,

H = the height of the wall

Kp = the passive pressure coefficient and is determined as

$$K_p = \frac{sin^2(\alpha - \phi')}{sin^2\alpha * sin(\alpha+\delta) * \left[1 + \sqrt{\frac{sin(\phi'+\delta) * sin(\phi'+\beta)}{sin(\alpha+\delta) * sin(\alpha+\beta)}}\right]^2} \tag{3.11}$$

Where:

β= angle between the backfill surface lines and a horizontal line

α= the angle between the horizontal line and the back face of the wall

δ= angle of wall friction

ϕ'= drained friction angle of the backfill soil.

The Kp value in the case of coulomb method will be equal to the Kp value in the Rankine theory under special conditions when (α=90, and β=δ=0) [13].

The following figure show the relationship between the internal friction angle (ϕ') of the backfill soil, the wall-friction angle (δ) and the variation of the passive earth pressure coefficient Kp.

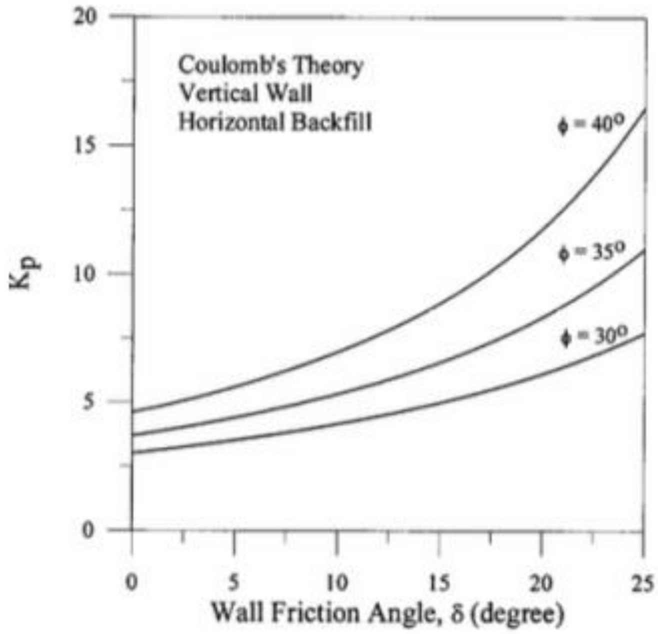


Figure 3.6 Kp versus internal friction angle (ϕ') and soil-wall friction angle (δ) (taken from Master thesis NTNU, Mari Melhus Romstad)

The table below shows the wall friction angle (δ) values for different backfill soils.

Table 3.3 soil type versus wall-soil friction angle [14]

Type of backfill soil	Wall-soil friction angle (δ) in degrees
Coarse sand	20-28
Fine sand	15-25
Silty clay	12-16
Stiff clay	15-20
gravel	27-30

The following figure [13] describes the above different angles in schematic form, where ρ is the rapture surface angle.

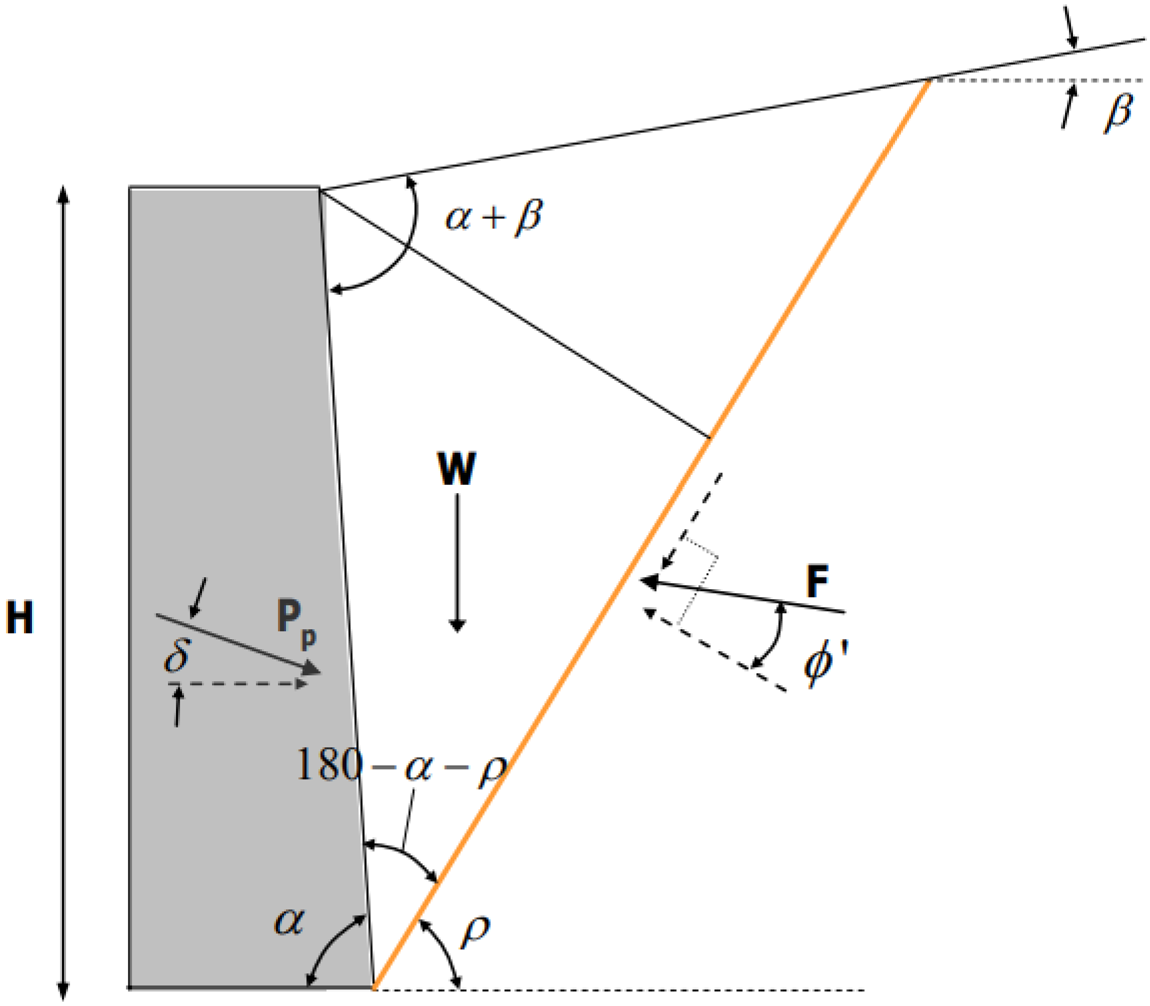


Figure 3.7 schematic forces acting on a retaining wall [13].

3.4.3 The Logarithmic spiral theory:

In 1943, Terzaghi developed the coulomb earth pressure theory to describe a failure surface geometry which consists of a logarithmic spiral and straight-line segments as shown in the figure below [13]. Unlike Rankine and Coulomb, Terzaghi assumed that the real failure surface has a curved shape at the lower section and a straight path at the upper as shown in the figure below. According to Terzaghi it is the wall-friction angle (δ) that causes the curvature of the failure around the wall.

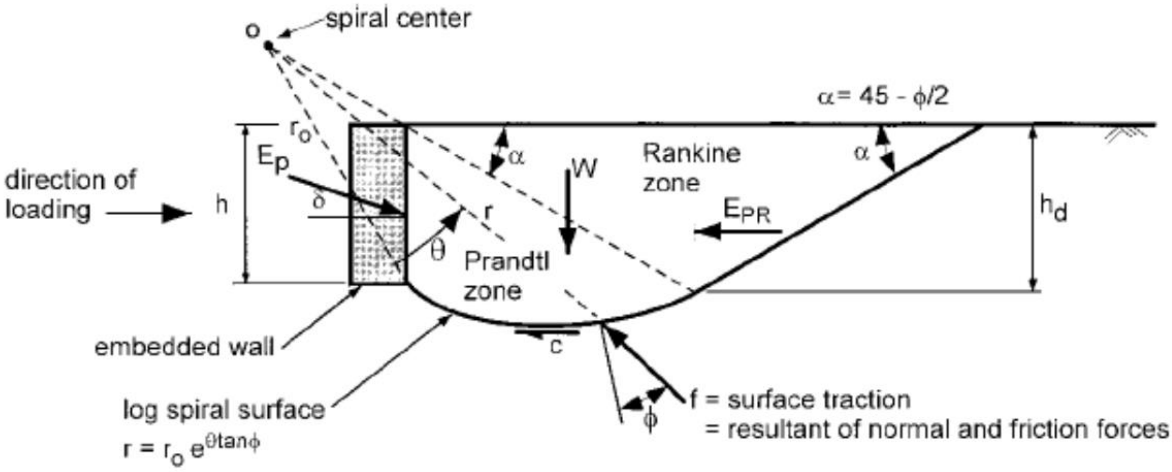
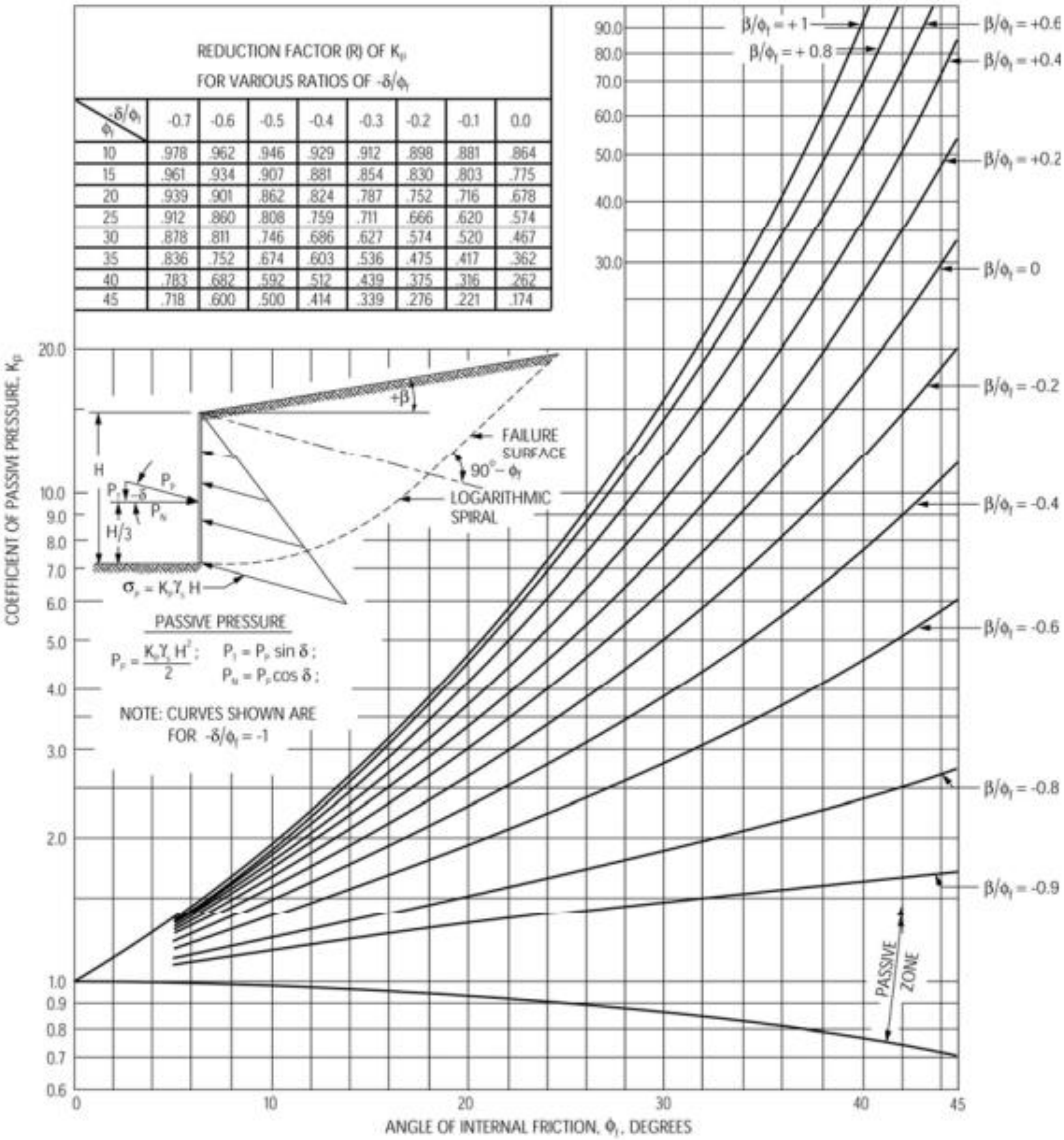


Figure 3.8 log-spiral failure mechanism after Terzaghi (Duncan and Mokwa,2001) [13]

The same as in the Coulomb theory the geometry at critical failure surface is created reducing the passive earth pressure needed to balance the equilibrium with the soil at the failure through the shear surface. The location of the passive earth pressure force at the surface of the wall is priorly selected based on the type of the backfill. By first obtaining the Kp values from the figure [13] or chart below, the resultant passive earth pressure force can be determined by using equation 3.5. This Terzaghi or logarithmic spiral solution gives a lower Kp values compared to the Coulomb method because the wall-soil interface frictional value (δ) is greater than one third of the ϕ'. The logarithmic spiral shape of the failure plane is accompanied by experiments and therefore the Terzaghi solution is preferred over the Coulomb method [13]. The coefficient of both the actine and passive earth pressures obtained from the graph is further multiplied by its respective reduction factor as given in the table form on the upper left corner of the chart. For example, for ϕ=30 and choosing -δ/ϕ = -0.1, β/ϕ =0, then the corresponding K_p value from the graph is read 6.5. The reduction factor obtained from the table for -δ/ϕ =-0.1 is 0.520.

K_p= (the R value for -δ/ϕ =-0.1) multiplied by (the K_p value for β/ϕ =0).

K_p =0.520*6.5= 3.4. According to the graphs below, to determine the coefficient of the passive earth pressure, this method needs the three angles (ϕ, δ and β).



ϕ_f \ $-\delta/\phi_f$	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Figure 3.9 Coefficients for passive earth pressure using log- spiral method [13].

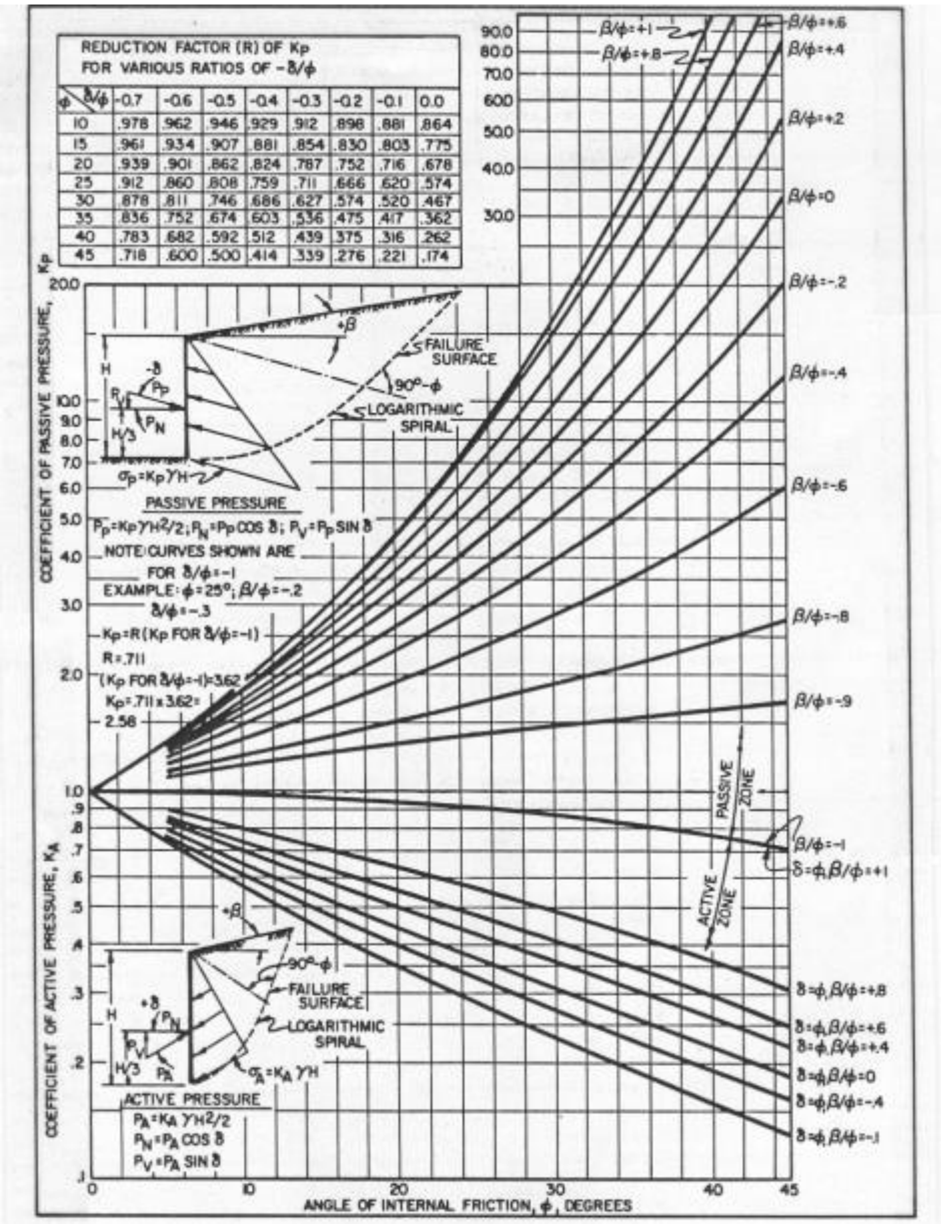


Figure 3.10 coefficients of active and passive earth pressure with wall friction angle [15].

3.4.4 The Janbu's theory:

Unlike all the above-mentioned theories, Janbu's theory considers the mobilized friction angle (ρ) and the ratio of roughness (r) parameters. However, similarly to Coulomb's theory this theory assumes a slightly curved failure surface. This theory helps to determine the coefficients of the active and passive earth pressures from a graph. Compared to the Rankine, Coulomb and log-spiral methods, this theory gives the least conservative solution, and it is the most used in Norway.

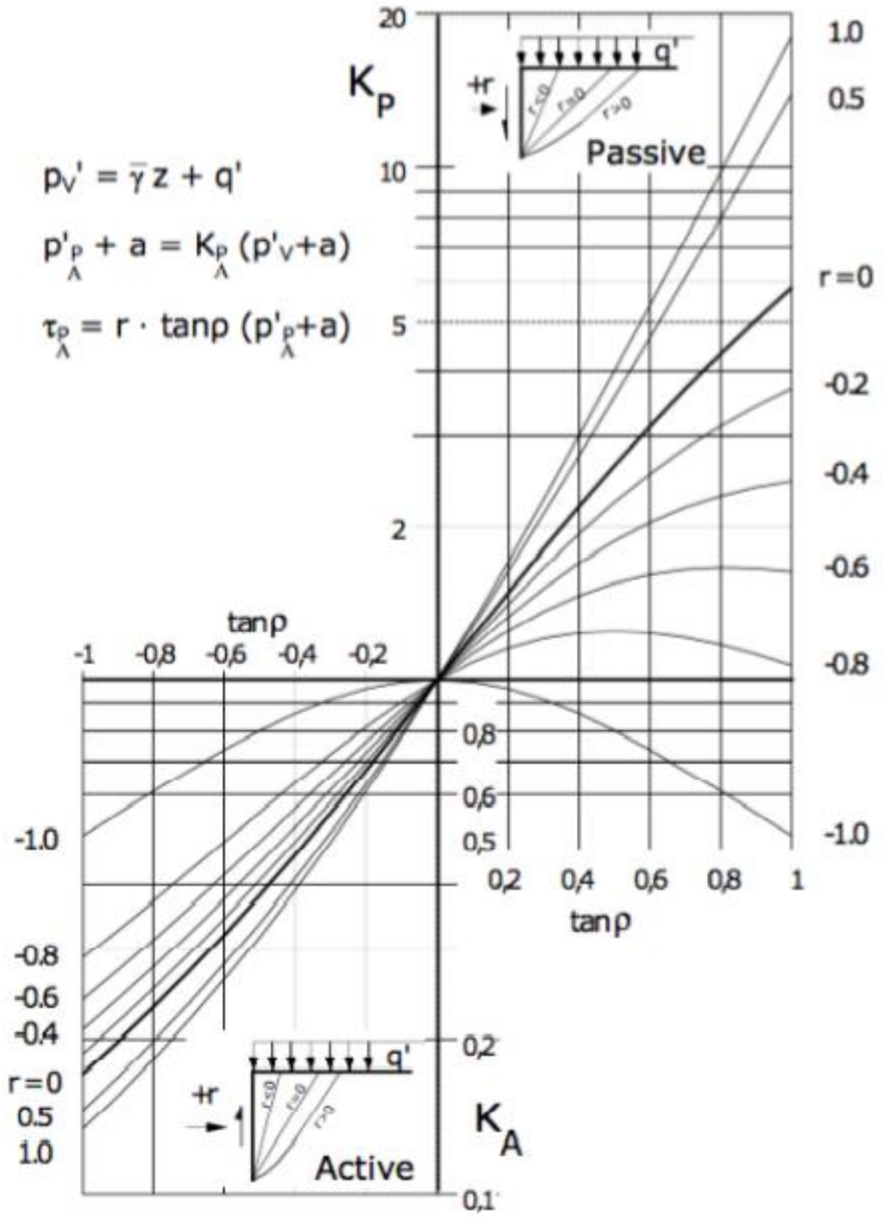


Figure 3.11 Janbu's passive and active earth pressure coefficients. (Taken from master thesis NTNU, Mari Melhus Romstad).

3.5 The roughness and the roughness ratio (r)

In the boundary between a construction element and underlying earth or the backfill soil there exists a friction. In geotechnics this friction is termed as a roughness.

The use of the roughness ratio is popular in Norway. It is expressed as the ratio of the mobilized shear stress on the given wall to the critical shear strength of the soil.

$$r = \frac{\tau_v}{\tau_c} = \frac{\tan\delta}{\tan\rho} \qquad (3.12)$$

Where r is the roughness ratio, τ_v and τ_c are the mobilized shear stress on the wall and the critical shear strength of the soil respectively.

If the roughness ratio is assumed to be zero (r=0) according to the above equation, it indicates that the numerator mobilized shear stress is equal to zero. When the mobilized shear stress on the wall (τ_v) is zero it implies that the wall is smooth and has no friction effect. In such cases it is assumed that no vertical or horizontal forces are transferred from the soil to the wall.

3.5.1 Positive and negative roughness

In the active state the wall is pushed outward far away from the backfill soil. Consequently, the soil slides down toward the wall. As a result of the movement of the soil, a shear stress is formed and pulls the wall downward. This is movement is called a positive roughness in the active state. In this active state if the wall is forced to move downward more than the downward movement of the soil due to the inclination of braces or other reasons, the force of friction will act in the opposite direction and this phenomenon is termed as a negative roughness in the active state.
On the other hand, in the passive state the wall comes closer to the backfill soil. Consequently, the soil is pushed inward and upward with respect the wall. The upward movement of the soil creates a shear stress that tries to push the wall upward. This phenomenon is defined as a positive roughness in the passive state [16]. In this case if the wall is forced to move upward more than the vertical movement of the soil it is termed as a negative roughness in the passive state. The figure below shows positive roughness in both the active and passive states.

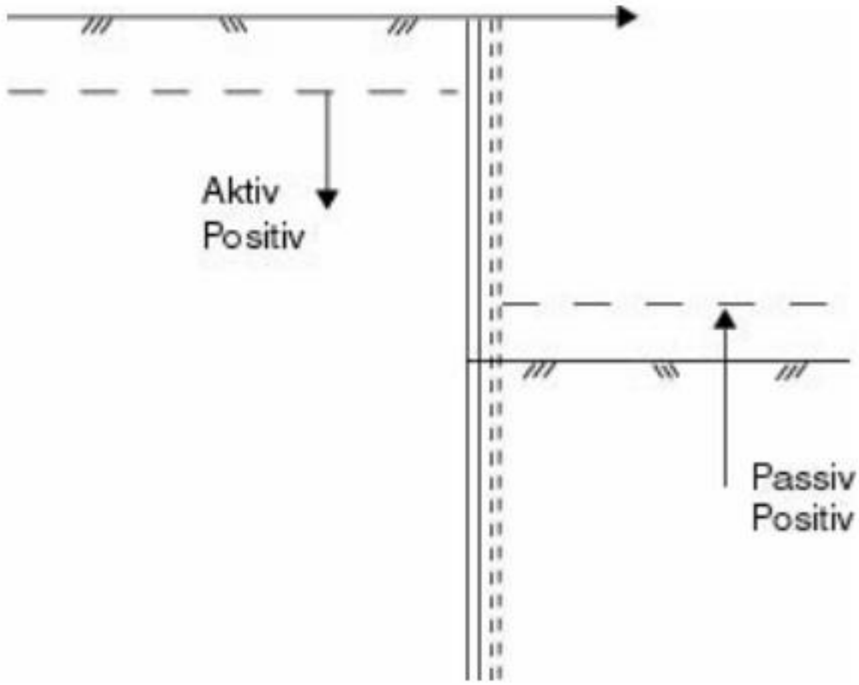


Figure 3.12 positive roughness in the active and passive state. taken from vegvesen handbook 016 page 10 (7-8)

3.6 Degree of mobilization and material coefficient

Degree of mobilization is expressed by symbol f and describes how much of the earth's shear strength is mobilized in a certain construction task. When f=1 it implies that 100% of the shear strength is mobilized.

$$f = \frac{\tan \rho}{\tan \varphi} \qquad (3.13)$$

The degree of mobilization f is bused in the serviceability limit state and helps for earth pressure calculations in a stiff construction.

Material coefficient is used in the ultimate limit state, and it describes the characteristic of a construction element made up of steel, wood, concrete, and others. The minimum acceptable value is 1.3. The material coefficient is mathematically expressed as follows.

$$\gamma_m = \frac{\tau_k}{\tau_d} \tag{3.14}$$

Where, τk is the characteristic shear strength and τd is the actual strength. Alternatively, it can be expressed as:

$$\gamma_m = \frac{\tan\varphi}{\tan\rho} = \frac{1}{f} \tag{3.15}$$

Thus, the material coefficient $\boldsymbol{\gamma_m}$ and the degree of mobilization **f** are reciprocals to each other.

3.7 Types of sheet pile walls and sheet pillings:

When sheet piles are used as retaining walls, they are basically divided into two classes, the cantilevered and the anchored. According to the shape or profile of their cross-section sheet piles are commonly divided as ‘Z’ type, ‘U’ type, combined, and straight sheet piles. Depending on the material type they are made up, sheet piles are categorized into sex types. These sex type materials are steel, precast Concrete, timber, viny composite and plastic, fiber glass, light gauge aluminum sheet pile walls, out of which steel piles are the most dominantly used. Aluminum and fiber glass are least used [17] [18] [19].

Sheet piles join with different interlocking design types. The following figure shows the different interlocking designs.

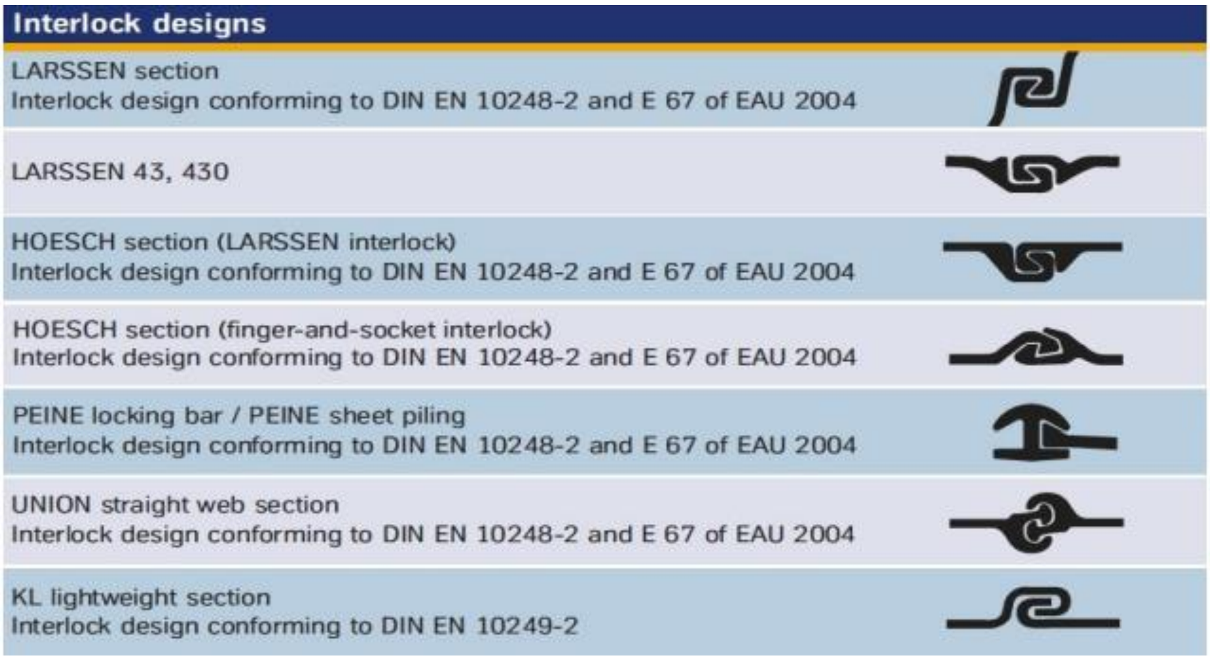


Figure 3.13 different interlock designs [19]

3.7.1 The anchored sheet pile wall:

This kind of construction is needed as it is more economical when the height of the backfill material in the case of a cantilever sheet pile is higher than six meters. Anchoring a sheet pile wall helps to reduce the required penetration depth, cross-sectional area, weight of the sheet pile material and the moment encountered to the sheet pile wall. As a result, this type of wall undergoes a lower lateral deflection compared to the braced sheet walls. It needs a small excavation for allowing instrument access during installation [20].

Figure 3.14 anchored sheet pile taken from handbook016 page 10-3

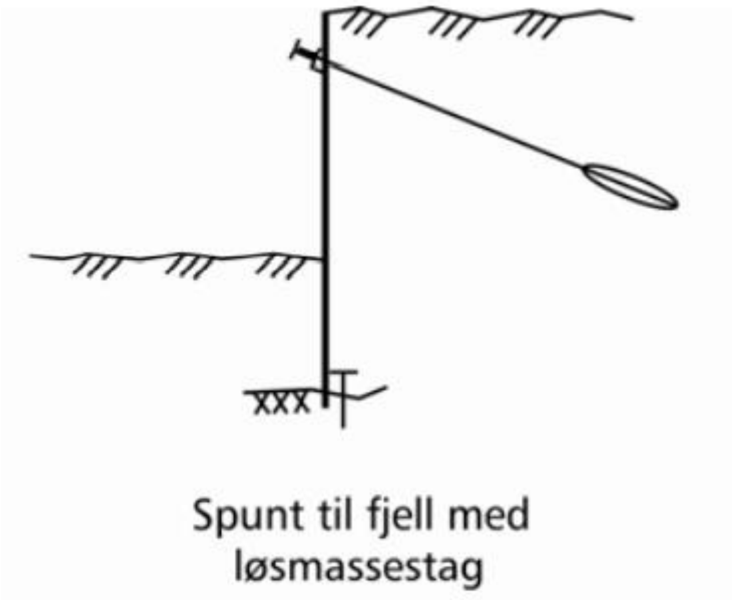


Figure 3.15 sheet pile anchored to mountain with soft soil scruts. taken from handbook 016 page 10-3

Anchored sheet piles can be designed based on two methods. These two possible methods are the free earth support and the fixed earth support. The figures below show the two methods of anchored sheet pile designs and the nature of their respective deflections [21]. In the case of the free earth support method, it involves a minimum depth of penetration, and no pivot exists below the dredge line.

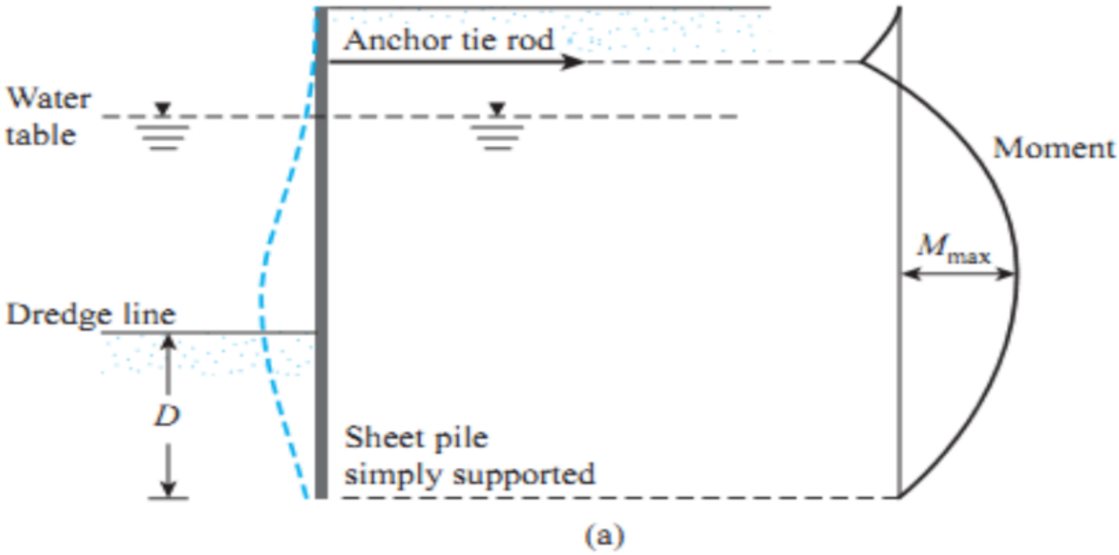


Figure 3.16 free earth support method [21].

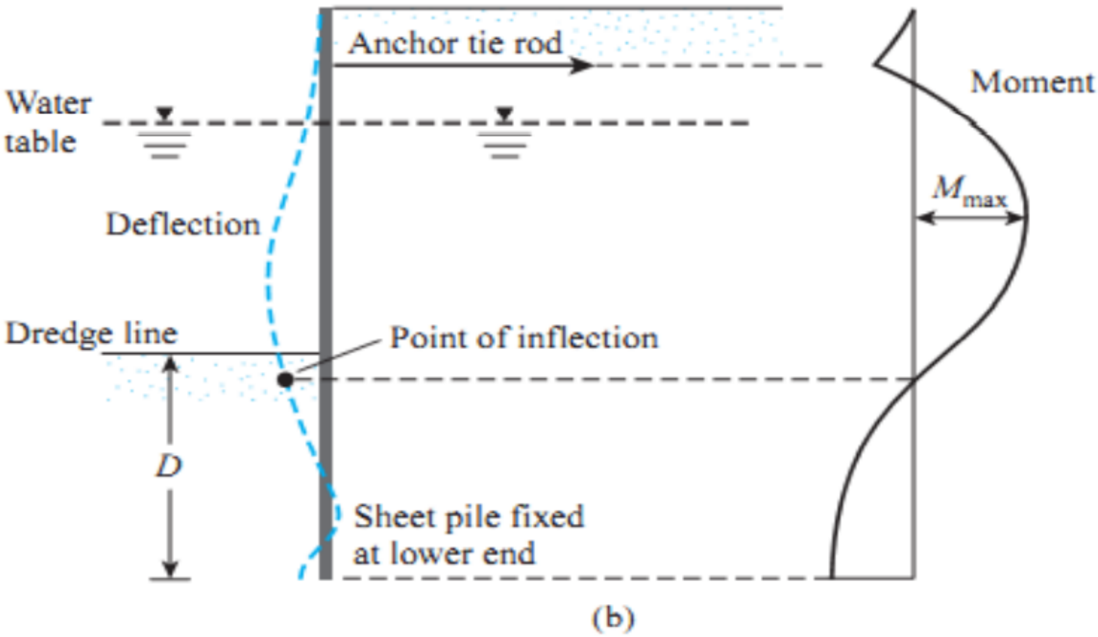
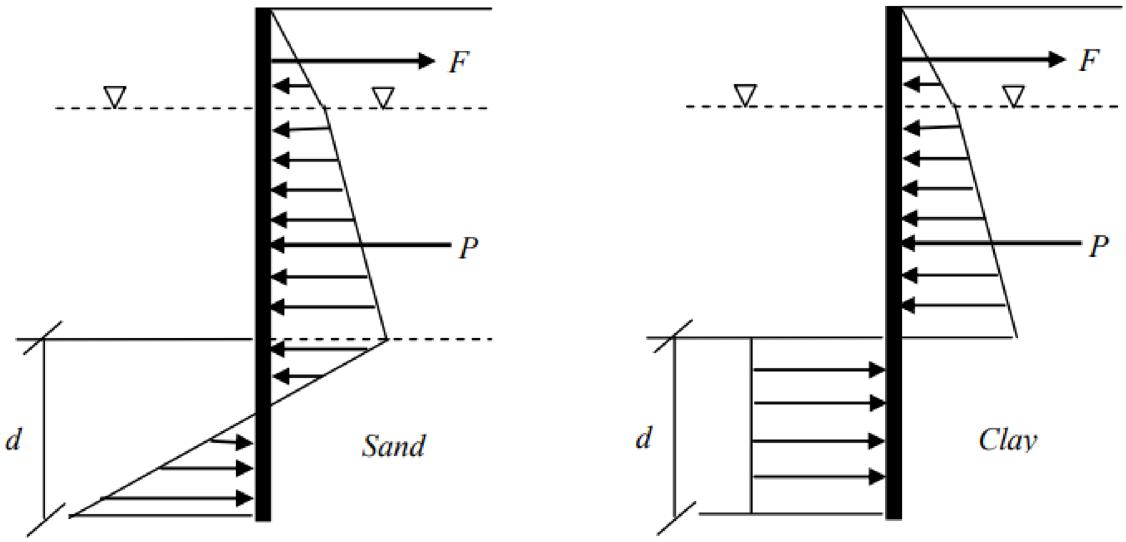


Figure 3.17 fixed earth support method [21].



(a) Anchored sheet pile wall penetrating sand.

(b) Anchored sheet pile wall penetrating clay.

Figure 3.18 net pressure diagram of Anchored sheet pile penetrating sand (a) and clay (b) soil [22].

let us consider the following figure [23] which shows the pressure distribution diagram of anchored sheet pile wall penetrating sand, where P is the area of the pressure diagram ACDE. By considering a moment about the point O' we can get the following degree for equation in terms of L4 where L4 can be solved by trial-and-error method.

$$L4^2 + 1.5L4^2(l2 + L2 + L3) - \frac{3P[(L1+L2+L3)-(\overline{z}+l1)]}{\gamma'(K_p-K_a)} = 0 \tag{3.16}$$

The penetration depth from the dredge line to the bottom end of the sheet pile wall (point B) marked by D is the theoretical depth of the construction. How ever in practical facts it is increase by 30 to 40 percent of its theoretical size.

$$D_{theoretical} = L3 + L4 \tag{3.17}$$

$$D_{acual} = 1.3\ to\ 1.4D_{theoretical} \tag{3.18}$$

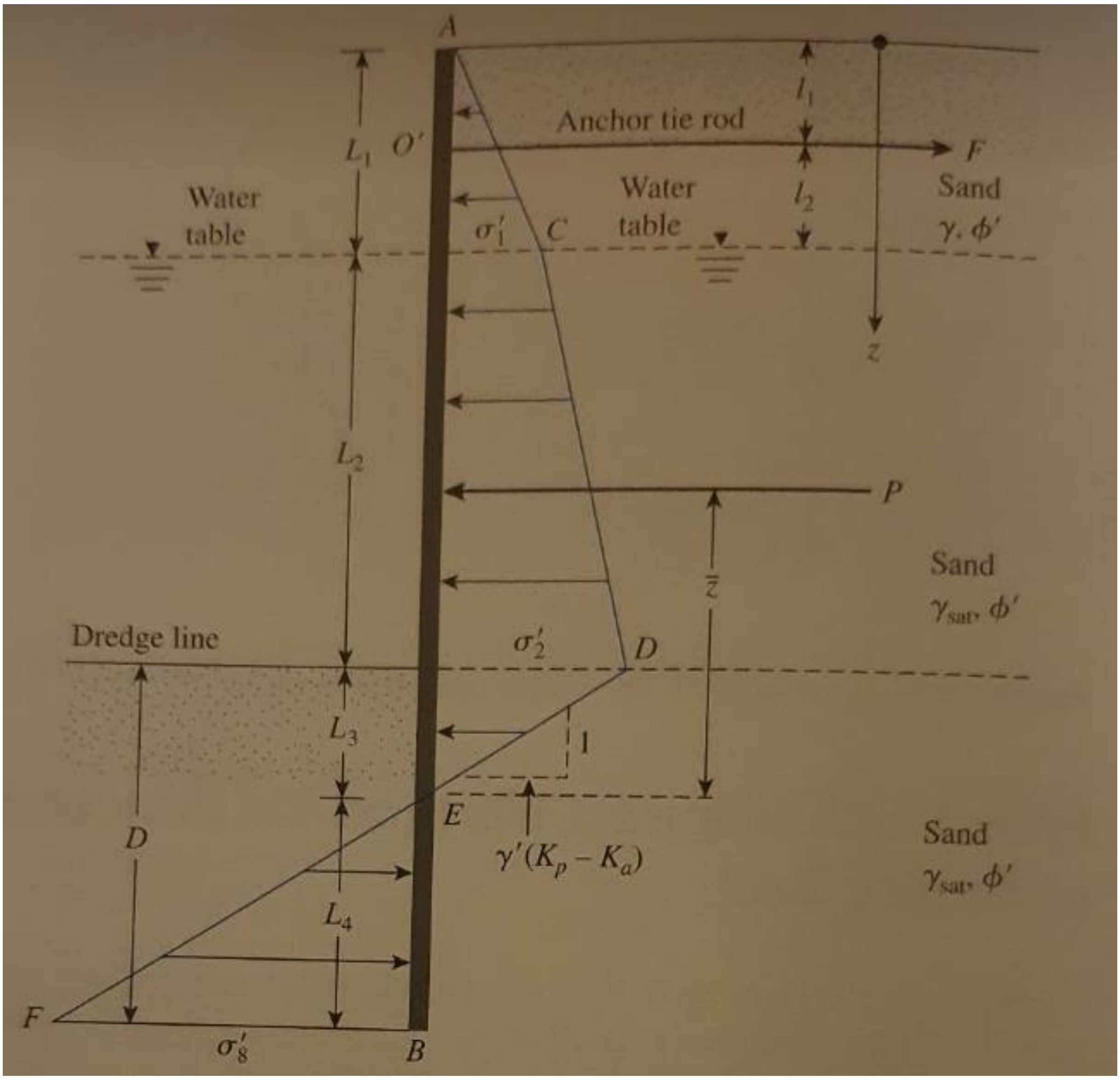


Figure 3.19 Anchored sheet pile penetrating sand [23].

3.7.2 The cantilevered sheet pile wall:

Cantilevered sheet pile walls are mass retaining structures mainly used for protecting both permanent and temporary excavations. They are also used for safeguarding during high-way constructions and landslide accidents.

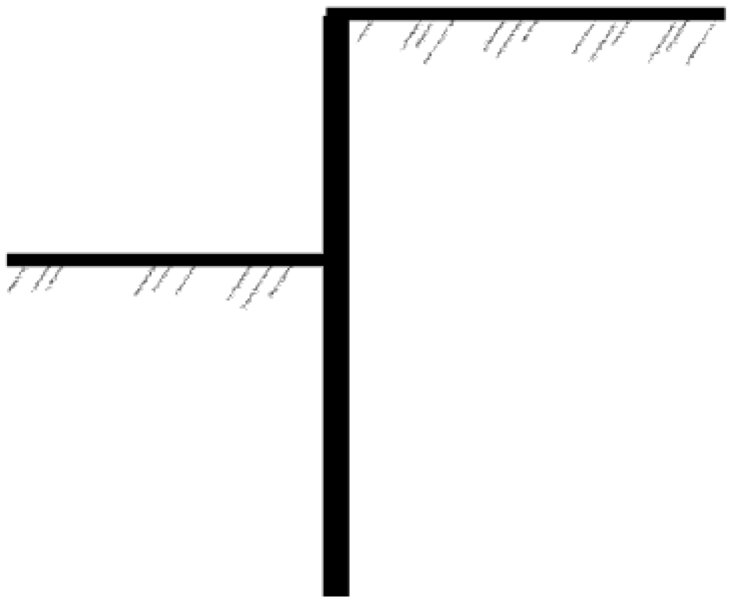


Figure 3.20 a cantilever sheet pile taken from handbook 016 page 10-3. illustrated by Yonas Tsighe

Cantilevered sheet piles are mostly recommended where the height of the wall above dredge line is less or equal six meters. The fundamental principles for determining or expecting the net lateral pressures for cantilevered sheet pile wall is described in the below diagrammatical representation. This diagram shows the behavior of lateral yielding pressure of a cantilever sheet pile wall passing through a sandy layer soil just under the mudline. It is supposed that the wall rotates about the pivot point O. Due to the hydrostatic pressure, equal pressures at any given depth in both lateral sides of the wall oppose and cancel each other. As a result, only effective lateral pressure is taken into consideration. The layer of soil along the length of the sheet pile on the land side is divided into three zones (zone A, Zone, B and Zone C) [19].

Zone A is the area or layer of soil above the dredge line where the lateral pressure is only the active pressure exerted from the land side. Zone B is the layer of the soil between the dredge line and the horizontal line passing through the point of rotation of the wall. In zone B there is a passive pressure from the water side and active pressure from the opposite side (land side). The situation in Zone C is the opposite of Zone B where there is a passive pressure right under the land side and there is active pressure on the front side. The net effective lateral pressure distribution exerted on the wall is shown on the figure with parabolic diagram however for the sake of design simplicity, the far-right version of diagram drawn with straight lines (figure (C)) is considered [19].

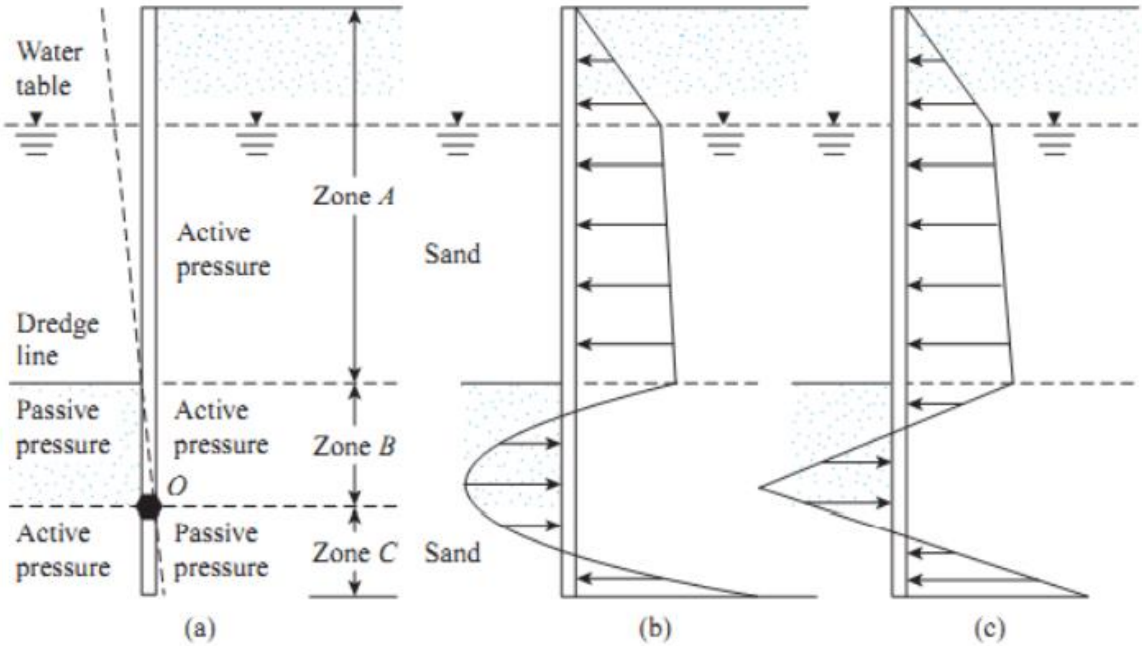


Figure 3.21 Cantilever sheet pile penetrating sand [19].

The diagrams that show the variation of net pressure and induced moment for a cantilever sheet pile penetrating sandy layer soil is given below. The diagram on the left side (figure a) represents the net pressure variation for a cantilever sheet pile penetrating a sandy layer while the parabolic diagram on the right side (figure b) shows for the moment induced on the wall [24]. AB is the sheet pile length penetrating the sand where L1 is the length of the wall over the water level. L2 is the depth of the water or the length of the wall between the dredge line and the water level and D is the depth of the wall below the dredge line.

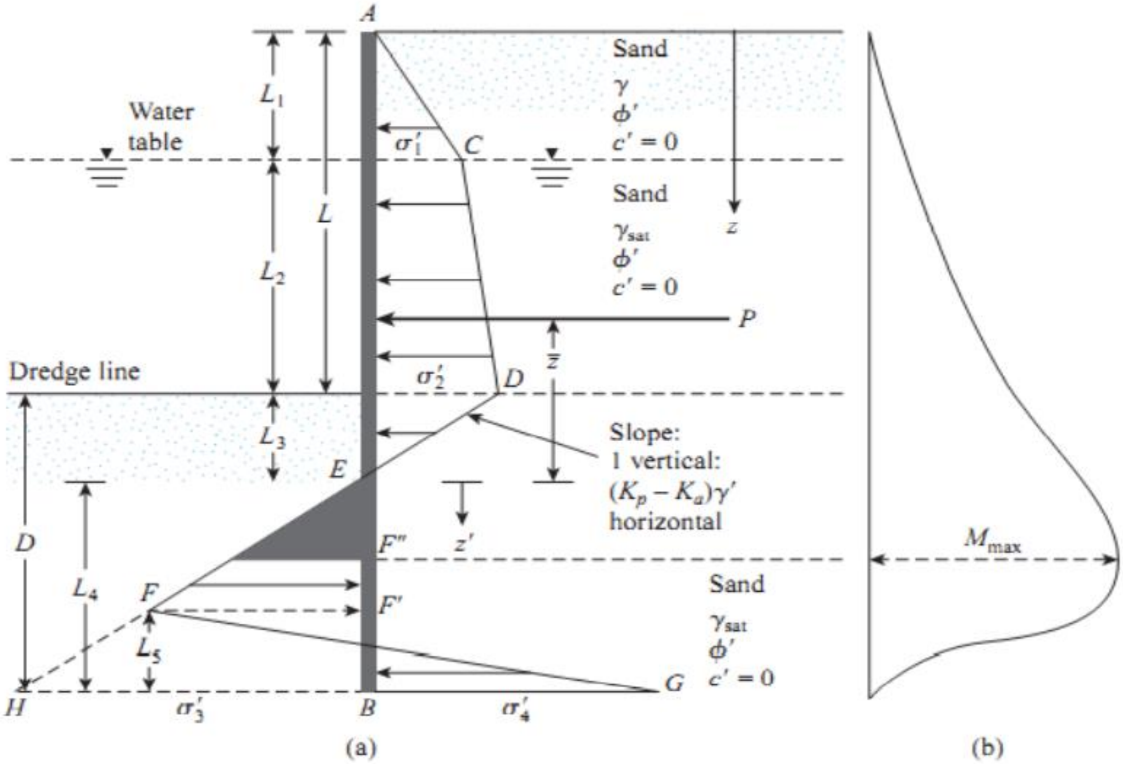


Figure 3.22 cantilever sheet pile penetrating sand: variation of net pressure diagram (a) and the moment induced diagram (b) [24].

There is another special case of pressure distribution of a cantilever sheet pile penetrating a sandy soil without ground water level as shown in the figure below.

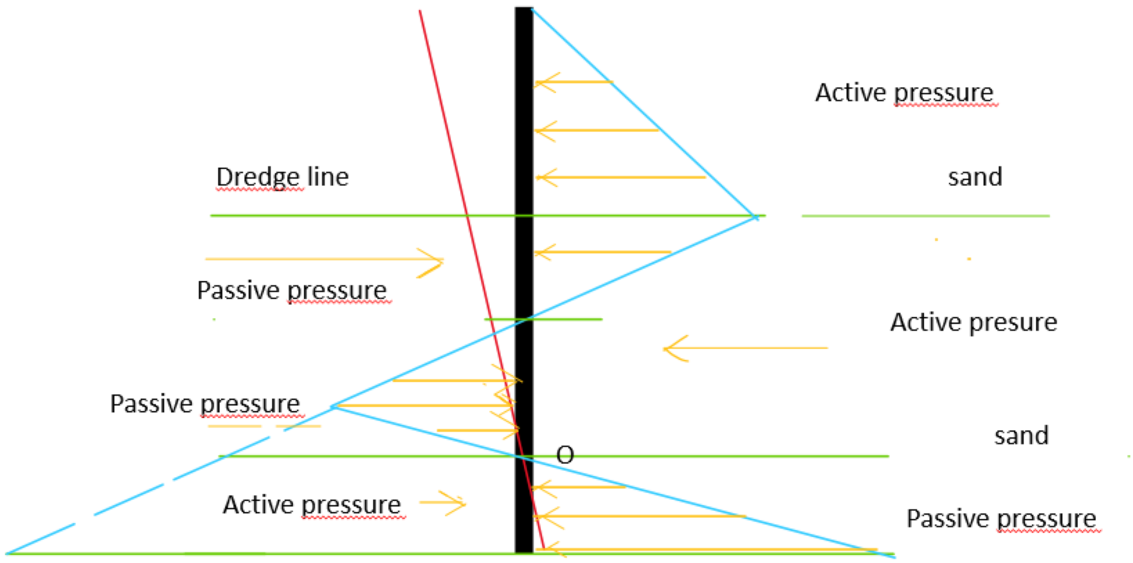


Figure 3.23a cantilever sheet pile wall pressure distribution penetrating a sandy soil without ground water level. Illustrated by Yonas Tsighe.

The figure below shows the variation of net pressure diagram for a cantilever sheet piler penetrating a clay soil.

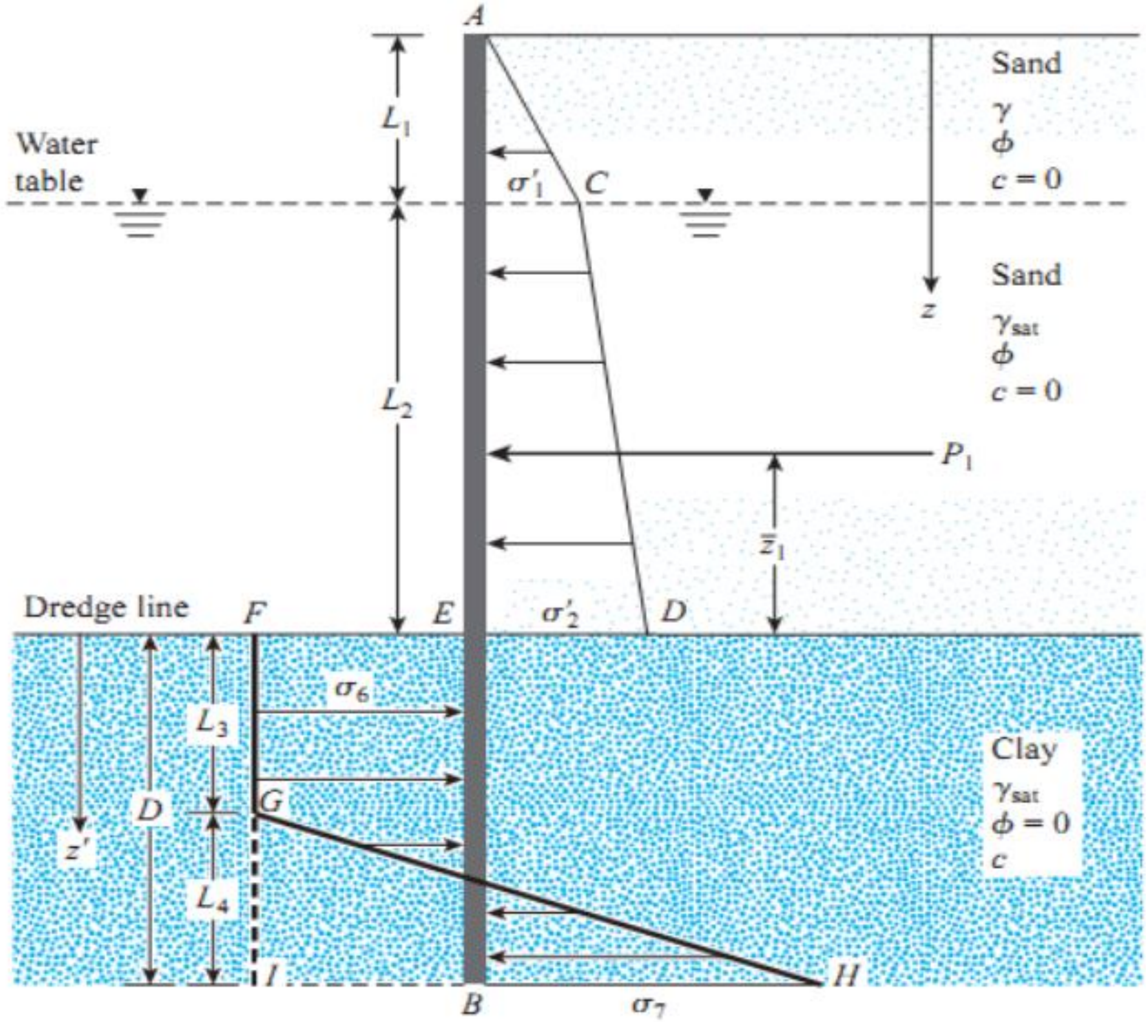


Figure 3.24 cantilever sheet pile penetrating clay soil [24]

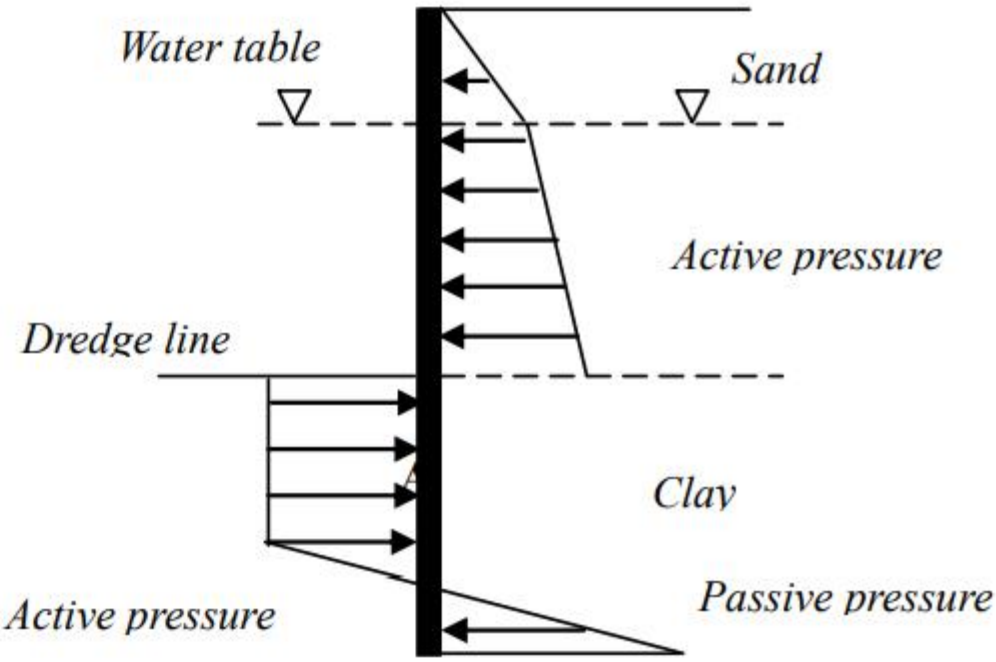


Figure 3.25 net earth pressure diagram of a cantilever sheet pile penetrating clay soil [25].

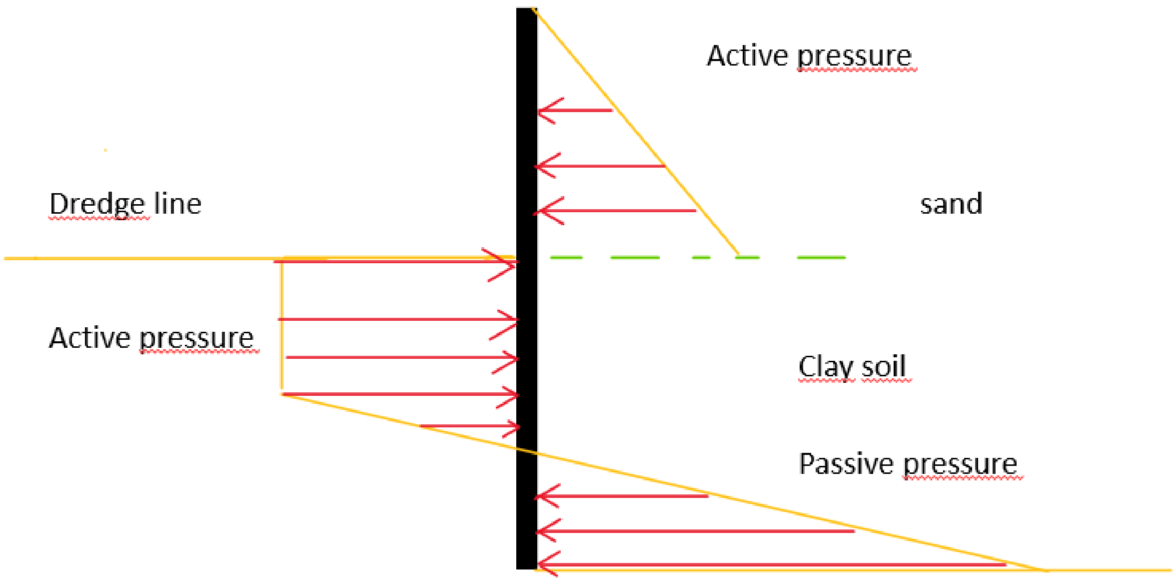


Figure 3.26 a cantilever sheet pile wall earth pressure distribution penetrating clay soil without ground water level. Illustrated by Yonas Tsighe

3.7.3 Steps for design of cantilever sheet pile wall penetrating a cohesionless soil:

The main aim is the to determine or design the length of the sheet pile and the lateral acting pressure forces.

After determining the values of the internal friction angle (ɸ'), the cohesive force (c) and the unit density weight (ɣ) of soil by the help of laboratory tests, it is possible design the length of the sheet pile from different approaches. In this case the Rankine method is chosen.

For cohesionless soil c is assumed to be zero and the length of the sheet pile over the dredge line is provided in terms of L_1 and L_2. Where L1 is the sheet pile length over the ground water table and L2 is the sheet pile length between the dredge line and the water table. The steps below show the sequence of calculating each term given in the diagram of net variation pressure. Let us consider the variation of net pressure diagram as shown in above figure 3.8a.

Step 1: determine lateral earth pressure coefficients K_a and K_p where K_a is the active pressure coefficient and K_p is the passive pressure coefficient.

$$K_a = \tan^2(45 - 0.5\phi') \quad (3.19)$$

$$K_p = \tan^2(45 + 0.5\phi') \quad (3.20)$$

Step 2: calculate δ'_1 and δ'_2(The active pressure intensity at depth Z= L_1 and Z = L_2). ɣ' is the saturated unit weight of the soil minus the water density unit weight. When calculating the lateral earth pressure, we use just the effective lateral pressure not the total latera pressure. where ɣ'= ($ɣsat$ - $ɣ_{water}$).

$$\delta'_1 = L_1 . K_a . ɣ \quad (3.21)$$

$$\delta'_2 = (L_1 . ɣ + L_2 ɣ')K_a \quad (3.22)$$

Step 3: calculate L3 (the sheet pile length where the net lateral pressure (δ'_{net}) equals zero below the dredge line). Below the dredge line the sheet pile is subjected to passive pressure on the excavation side and active pressure on the earth side. The passive earth pressure gets larger and larger and the pressure on the earth side (the active earth pressure) gets smaller and smaller. Consequently, it becomes zero at a certain depth Z. where L3= Z-(L1+L2).

$$L_3 = \frac{\delta'_2}{ɣ'(K_p - K_a)} \quad (3.23)$$

The above equation shows that the slope of the net pressure line DEF is 1 vertical magnitude divided by the product of (Kp-Ka) and ɣ'. Thus, considering the right-angle triangle EHB, tangent of the line EH is equal to the ratio L4 to δ'3. This leads us to the fact that:

$$\overline{HB} = \delta'_3 = L_4\left(K_p - K_a\right)ɣ' \quad (3.24)$$

Step 4: calculate P (area of the depth-pressure diagram ACDE) as show in the variation of net pressure diagram (a) above.

$$P = 0.5 * L1\delta'1 + \delta'1 * L2 + 0.5(\delta'2 - \delta'1)'L2 + 0.5 * \delta'2 * L3 \quad (3.25)$$

Alternatively, P can be calculated as follows provided that the maximum moment occurs at a point between E and F. Calculating the maximum moment (M_{max}) divided by unit length of the wall needs obtaining the point of zero shear. Creating a new axis Z' while the origin is at E for zero shear:

$$P = 0.5 * Z'^2(Kp - Ka)ɣ' \quad (3.26)$$

Step 5: calculate the depth z that is the center of pressure for the area ACDE by considering the moment about the point E.

$$\overline{Z} = \frac{\sum M_E}{P} \quad (3.27)$$

Where:

$$\sum M_E = (A_1 * a_1 + A_2 * a_2 + A_3 * a_3 + A_4 * a_4) - \left(P * \overline{Z}\right) = 0 \quad (3.28)$$

A1, A2, A3 and A4 are areas of the three triangles and one rectangle in the ACDE region and a1, a2, a3 and a4 are their respective moment arm with respect the point E.

Or

$$Z' = \sqrt{\frac{2P}{(Kp-Ka)*\gamma'}} \tag{3.29}$$

Once the depth or the point of zero shear pressure force is obtained as shown in the above figure the size of the maximum bending moment can be calculated as follows.

$$Mmax = P(\overline{Z} + Z') - \left[\frac{1}{2} * \gamma' * Z'^2 * (Kp - Ka)\right] * \frac{1}{3} * Z' \tag{3.31}$$

Step 6: calculate the net lateral pressure at the bottom of the sheet pile (δ'p-δ'a).

$$\delta'p - \delta'a = \delta'_5 + \gamma' L4(Kp - Ka) \tag{3.32}$$

Where:

$$\delta'_5 = (\gamma L_1 + \gamma' L_2) * K_p + \gamma' L_3(K_p - K_a) \tag{3.33}$$

Step 7: calculating A, A2, A3 and A4 constants: where P is the area of the pressure diagram ACDE: Adding the moment of all the forces about point B gives:

$$P(L_4 + \overline{Z}) - \left(\frac{1}{2} L_4 \delta'_3\right)\left(\frac{L_4}{3}\right) + \frac{1}{2} L_5(\delta'_3 + \delta'_4)\left(\frac{L_5}{3}\right) = 0 \tag{3.34}$$

Where:

$$L_5 = \frac{\delta'_3 L_4 - 2P}{\delta'_3 + \delta'_4} \tag{3.35}$$

Combining equations all equations (6), (12), (14) and (15) and simplifying it a four-degree equation in terms of L_4.

$$L_4{}^4 + A_1 L_4{}^3 - A_2 L_4{}^2 - A_3 L_4 - A_4 = 0 \tag{3.36}$$

Where:

$$A_1 = \frac{\delta'_5}{\gamma'(K_p - K_a)} \tag{3.37}$$

$$A_2 = \frac{8P}{\gamma'(K_p - K_a)} \tag{3.38}$$

$$A_3 = \frac{6P[2\overline{Z}\gamma'(K_p - K_a) + \delta'_5]}{\gamma'^2(K_p - K_a)^2} \tag{3.39}$$

$$A_4 = \frac{P(6\overline{Z}\delta'_5 + 4P)}{\gamma'^2(K_p - K_2)^2} \tag{3.40}$$

After calculating all the above expressions, the necessary profile of the sheet pile can then be sized based on the allowable flexural stress (δ_{all}) of the sheet pile material. The section modulus of the sheet pile material required per unit length of the structure is denoted by the symbol S.

Where:

$$S = \frac{M_{max}}{\delta_{all}} \tag{3.41}$$

3.7.4 designing sheet pile length and selecting profile section:

Given: L1=2m, L2=3m, soil type is sand where cohesive force C =0, γ=

S.N0	Material description	Given data (Type)
1	L1 (wall height over water level)	2m
2	L2 (wall height between dredge line and water level)	3m
3	Effective friction angle ϕ'	30^0
4	Soil type sand (cohesionless) granular	C=0
5	Unit weight density (γ)	γ=17KN/m^3
6	Saturated unit weight γ_{sat}	γ_{sat}=20KN/m^3
7	Allowable flexural stress δ_{all}	δ_{all} =170*MN/m^2

Aim: To calculate total sheet pile length and select profile section for different L1 and L2 provided that the other factors are kept constant.

Designing of sheet pile for different excavation depths L1 and L2.

no								
1	L1 (m)	1	2	2	3	3	6	8
2	L2 (m)	2	2	3	3	4	9	12
3	ϕ' (degrees)	30	30	30	30	30	30	30
4	γ (KN/m^3)	17	17	17	17	17	17	17
5	γ_{water} (KN/m^3)	10	10	10	10	10	10	10
6	γ_{sat} KN/m^3)	20	20	20	20	20	20	20
7	$\delta_{allowable}$ KN/m^2	170000	170000	170000	170000	170000	170000	170000
8	γ'	10	10	10	10	10	10	10
9	Ka	0.333	0.333	0.333	0.333	0.333	0.333	0.333
10	Kp	3	3	3	3	3	3	3
11	δ'1	5.667	11.3	11.33	17	17	34	45.33
12	δ'2	12.33	18	21.33	27	30.33	64	85.33
13	δ'3	88	117	140	168	192	394	521
14	δ'4	211	297	353	438	495	1034	1374
15	δ'5 (net.p)	123	180	213	270	303	640	853
16	L3 (m)	0.46	0.68	0.8	1	1.14	2.4	3.2
17	L4 (m)	3.3	4.4	5.25	6.3	7.2	14.8	19.5
18	L total	6.76	9.1	11.05	13.3	15.34	32.2	42.7
19	Mmax (KNm)	65.5	153	265	458	680	5855	14479
20	S (section modulus) m^3/m	0.000385	0,0009	0.001557	0.002696	0.003997	0.03444	0.085170
21	Section modulus (cm^3/m)	385	900	1557	2696	3997	34440	85170
22	Section type							

Designing a sheet pile for a sandy soil having different internal friction angles (30 -36) and keeping all other values constant:

No								
1	L1 (m)	2	2	2	2	2	2	2
2	L2 (m)	3	3	3	3	3	3	3
3	ϕ' in degrees	30	31	32	33	34	35	36
4	ɣ (KN/m^3)	17	17	17	17	17	17	17
5	$ɣ_{sat}$ (KN/m^3)	20	20	20	20	20	20	20
6	$ɣ'_{water}$ (KN/m^3)	10	10	10	10	10	10	10
7	$δ_{allowable}$ (KN/m^2)	170000	170000	170000	170000	170000	170000	170000
8	ɣ’	10	10	10	10	10	10	10
9	Ka	0.333	0.32	0.307	0.295	0.283	0.271	0.26
10	Kp	3.0	3.124	3.255	3.392	3.537	3.69	3.85
11	δ’1	11.3	10.9	10.45	10	9.6	9.2	8.8
12	δ’2	21.3	20.5	19.7	18.9	18.1	17.3	16.6
13	δ’3	140	141	142	143	144	145	146
14	δ’4	354	361	370	379	388	398	409
15	δ’5	213	220	228	236	244	253	263
16	L3 (m)	0.8	0.73	0.67	0.61	0.56	0.5	0.46
17	L4 (m)	5.25	5	4.8	4.6	4.42	4.2	4
18	L total	11.1	10.73	10.5	10.2	10	10.7	
19	M_{max} KNm	265	247	231	216	202	189	177
20	(S) Section modulus (m^3/m)	0.001557	0.0014534	0.001358	0,0012702	0,001189	0,001114	0,001044
21	Section modulus (cm^3/m)	1557	1453	1358	1270	1189	1114	1044
22	Section type							

A special case of earth pressure distribution for a cantilever sheet pile wall penetrating a sandy soil without ground water level. In all cases the sum of all the passive pressures is greater than the sum of all the active earth pressures. For safety reasons the embedment length (D=L3 +L4) is increased by 30% to 40% for a sandy soil and 40 to 60% for clay soil [26].

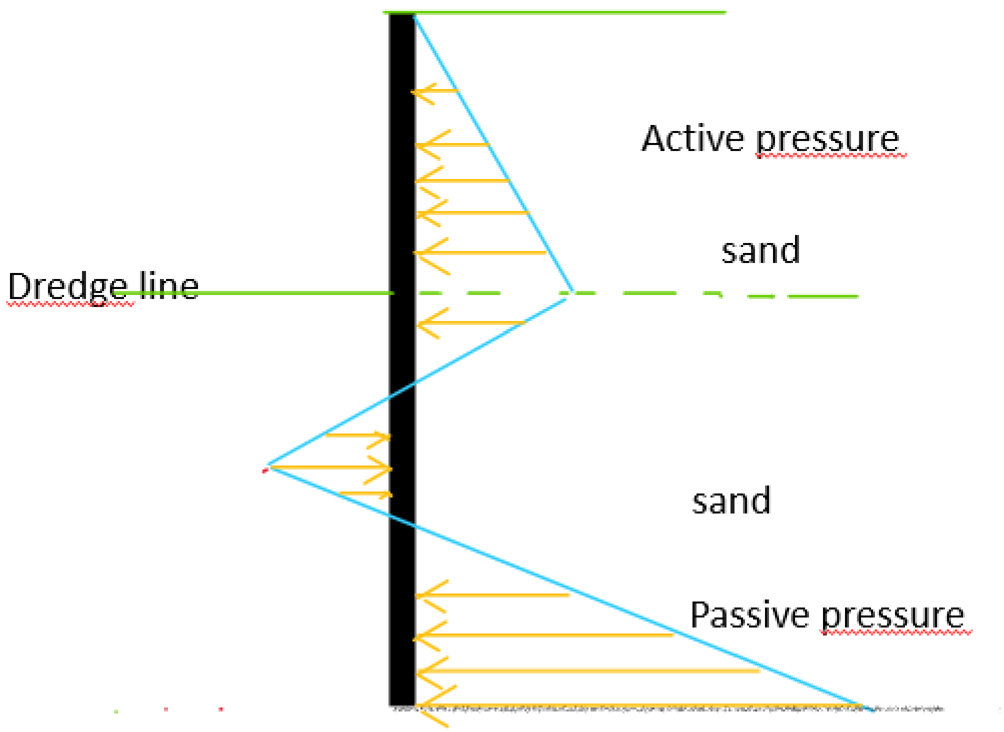


Figure 3.27 earth pressure distribution in sand soil without ground water level. Illustrated by Yonas Tsighe.

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3.8 Material:

Comparing based on the material they are made up of, steel piles are the most dominant structures used.

3.8.1 Steel sheet piles:

Due to its wide range of product availability and its higher strength, steel sheet pile wall is the most used. Steel easily corrodes in wet conditions especially in salty water however if it is protected against corrosion, it has higher durability than all familiar metals. In addition, steel can be utilized almost in all civil engineering constructions [27].

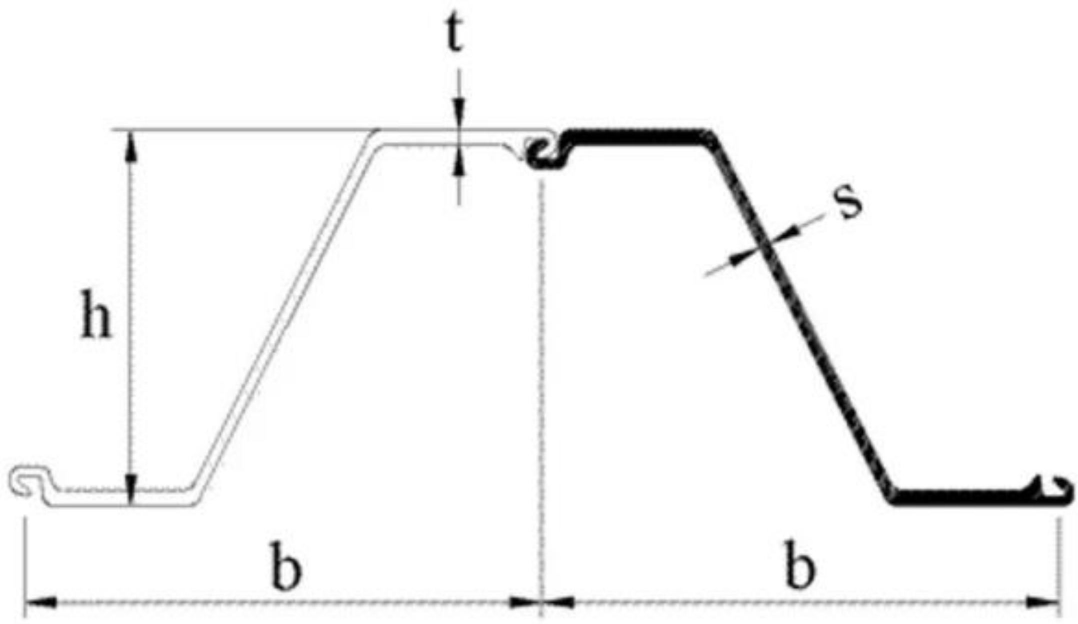


Figure 3.28 Z- type steel sheet pile [27]

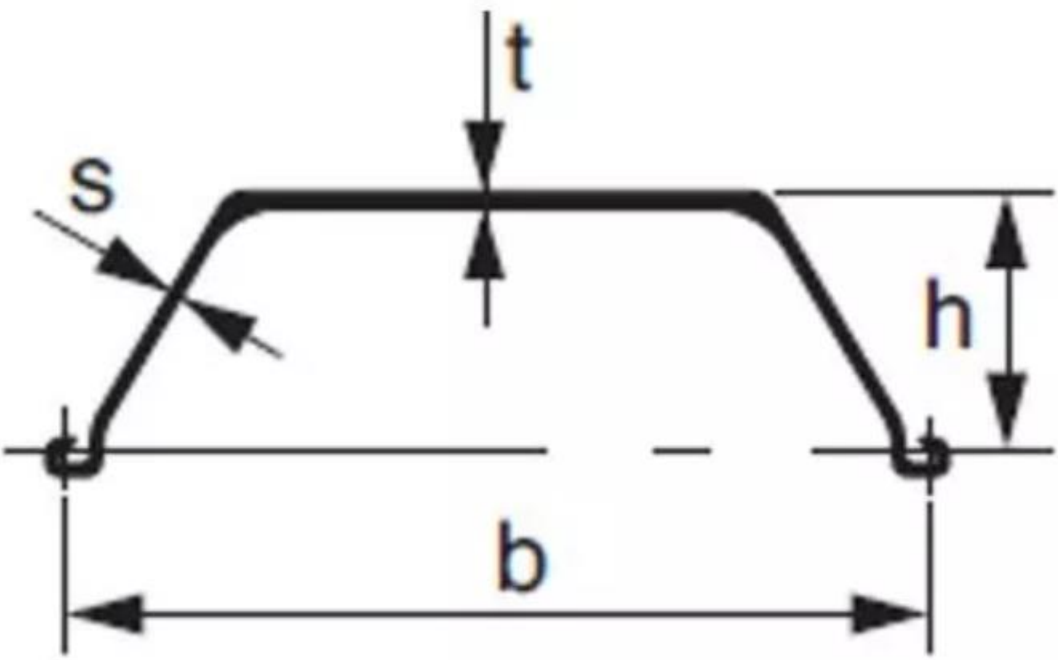


Figure 3.29 U-type steel sheet pile [27]

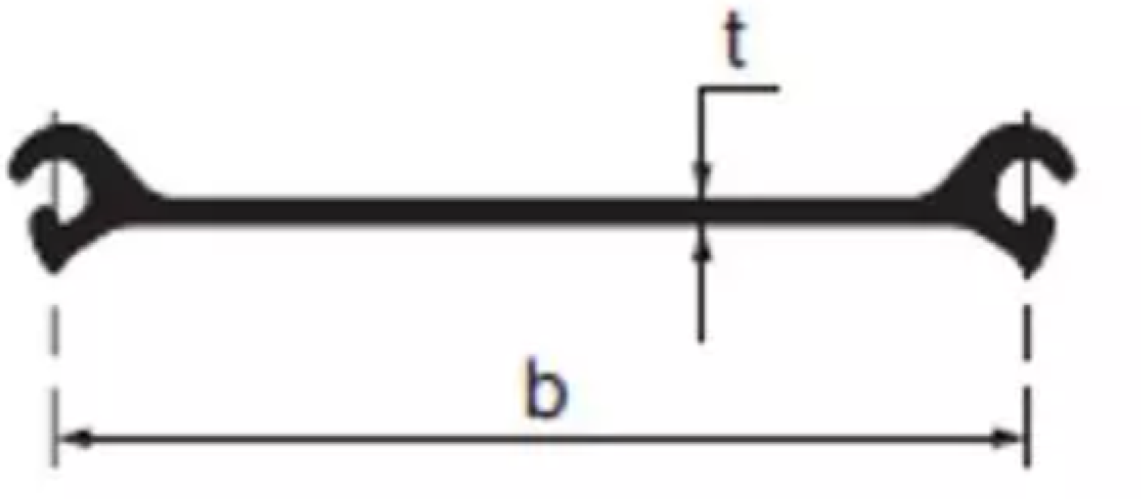


Figure 3.30 straight steel sheet pile [27]

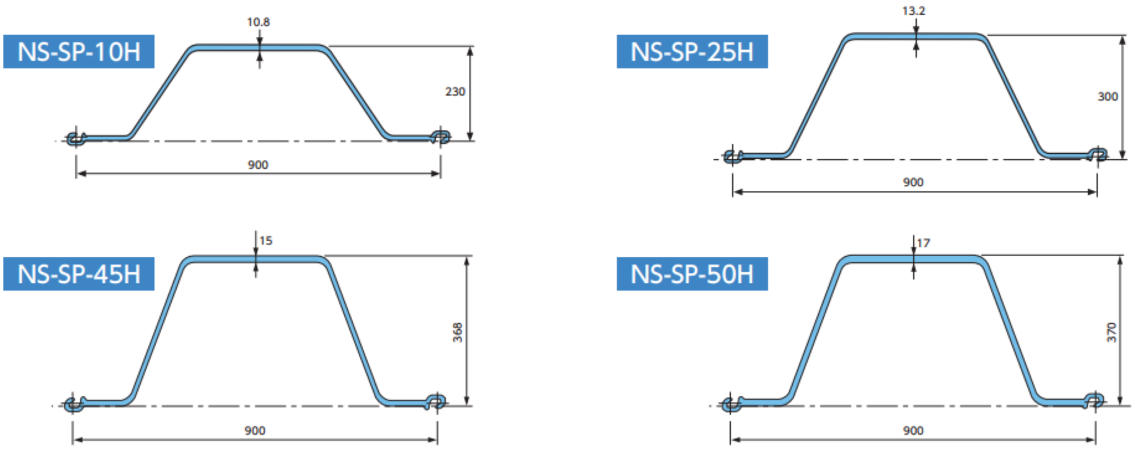
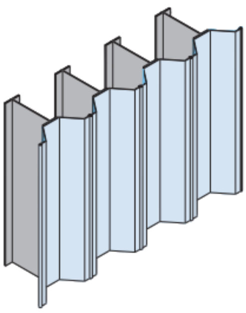


Figure 3.31 Hat type steel sheet pile [28].

Combined steel sheet pile (H+hat) type



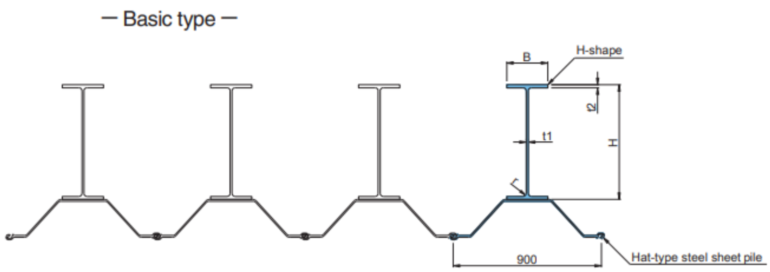


Figure 3.32 combined type (H beam +hat type steel sheet pile

3.8.2 Aluminum sheet pile:

Aluminum sheet piles work in a similar way as steel sheet piles and their strengths vary from 8ksi (kilopound per square inch) to 45ksi which corresponds 55MPa to 310MPa. Aluminum sheet piles have about 30% strength of steel sheet piles however, they are more flexible, lighter and corrosion resistant compared to steel. As a results of this corrosive resistant advantage, they can be used in salty or marine environments. The weak side of aluminum sheet piles is that they can be corroded in acidic soils where the soil is abundantly rich in copper, chrome, and other salts [27].

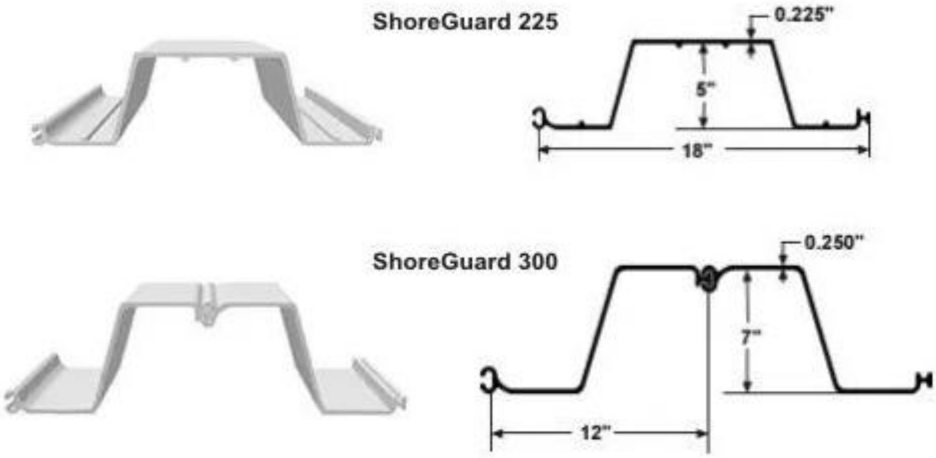


Figure 3.33 U and Z type aluminum sheet piles [27]

3.8.3 Timber sheet pile:

Nowadays timber sheet piles are commonly used in shallow excavation foundations where their soil is dry or where there is no ground water. Timber sheet piles are composed of planks that are interconnected with different types of joints. Some timber sheet piles include a tongue and a groove joint, others are Wakefield timber sheet piles. Wakefield timber piles are several planks nailed and glued together so that they can form one timber sheet pile. Planks joined by tongue and groove joints are other types of timber sheet piles. In addition to the above-mentioned types of timber sheet piles there are splined timber sheet piles [27].

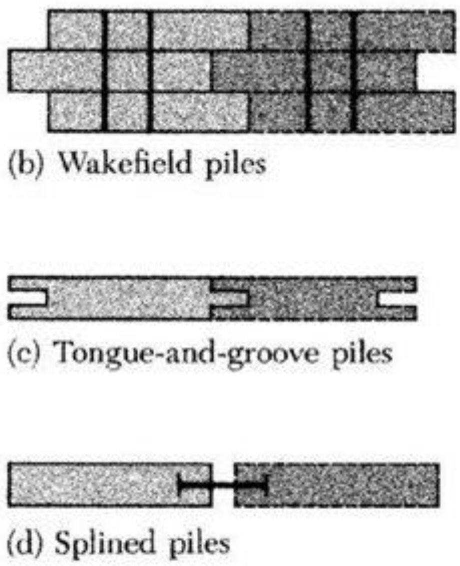


Figure 3.34 different joints of timber sheet piles [27]

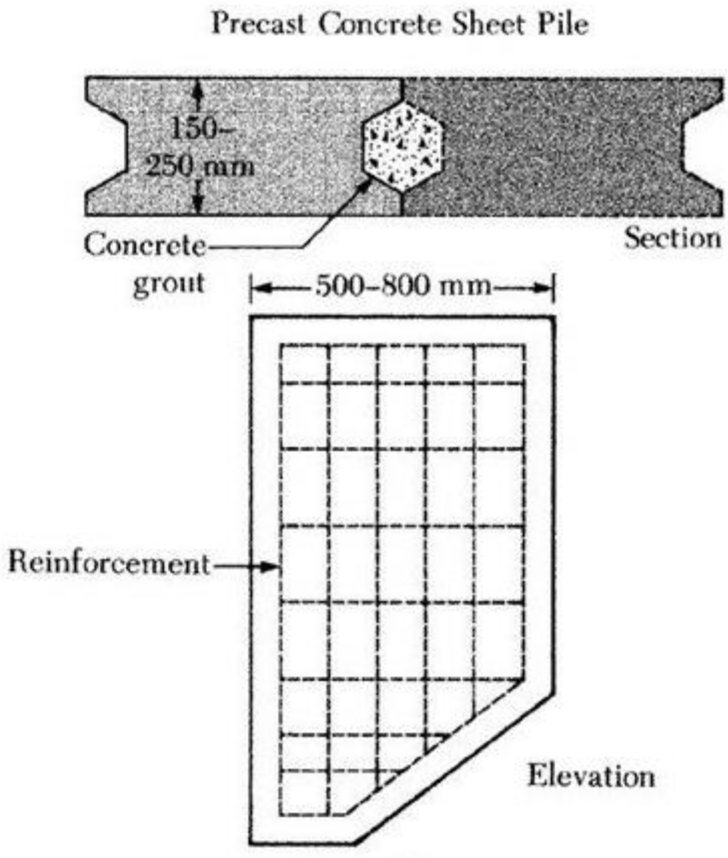
3.8.4 Vinyl Sheet Pile and polymeric sheet pile

These types of sheet piles are like steel sheet piles in design however, they are made up of synthetic material. They have smaller yield strength and are less stiff compared to steel sheet piles. Vinyl sheet piles are commonly applicable as barriers to protect erosion. They are also ideal for small environmental and sea-front projects. Based on the selection of the synthetic type they can be used as long-term barriers against sun-light and chemical exposures. Their drawback is that they cannot be used as the best solution for harder ground and deeper depths. For such types of projects steel sheet piles are preferred [27].

Figure 3.35 U-Type vinyl sheet piles

3.8.5 Precast Concrete Sheet Piles

These types of piles are utilized by casting reinforced concrete section panels. They are connected to each other by tongue and groove joints at the ends of the sheet piles. These type piles are less common as they demand expensive work to fabricate, transport, handle and mantle it. They are used where steel sheet piles are not available or where there is special condition. Precast concrete sheet piles are stiff and have a high yield strength, however it is difficult to drive them in due to their big size (thickness) and they experience tensile cracks as the driving stress tensions exist throughout the whole length section [27].



3.9 Selection of sheet pile type:

There are many types of sheet piles, and selection of a sheet pile type for a certain project further depends on many factors some of which are listed below.

Identifying and analyzing the **kind of work** if it is temporary or permanent.

Site conditions have a great role in selecting the type of a sheet pile. Seawater areas are more aggressive than natural water and thus need special sheet piles that can fit that challenge. Polluted and industrial areas are more corrosive compared to natural and disturbed soils. The soil in some site conditions is dense and hard while in others may be soft. Sheet piles in hard and dense site conditions need to be strong and stiff materials that can tolerate the driving force of the hydraulic machine and the resisting forces from the soil.

Type of protection required is another factor for selecting a type of sheet pile. Some types of sheet piles need to be protected against corrosion however others like aluminum and concretes and plastic do not need to be protected against corrosion.

How deep the sheet pile shall be installed is another factor for selecting a suitable sheet pile. Steel sheet piles are preferable to wood and aluminum sheet piles when the required depth is so long.

How strong the **bending moment** induced is the most important factor in selecting a durable sheet pile wall because all sheet piles whether they are temporary or permanent must tolerate the induced moment.

3.10 Sheet pile failures:

Both cantilevered and anchored sheet piles experience three different types of failures. Each failure has its own cause. The three common failures are the flexural failures, the rotational failure, and the deep-seated failure.

3.10.1 Flexural failure:

This type of failure occurs in each of the cantilever and anchored sheet pile walls. The failure occurs in the span of the wall where the maximum moment is induced. The point in the length of the wall where maximum moment occurs, acts as a hinge or as a location of failure. Therefore, when designing a sheet pile wall, it is important to focus on the point where the maximum moment occurs so that a sheet pile which has a sufficient section of modulus would be selected to tolerate that much moment. Flexural failures are common if the sheet piles are not stiff enough to resist the moment induced. The figure below shows the flexural failure mode in both the cantilever and the anchored sheet pile walls [29].

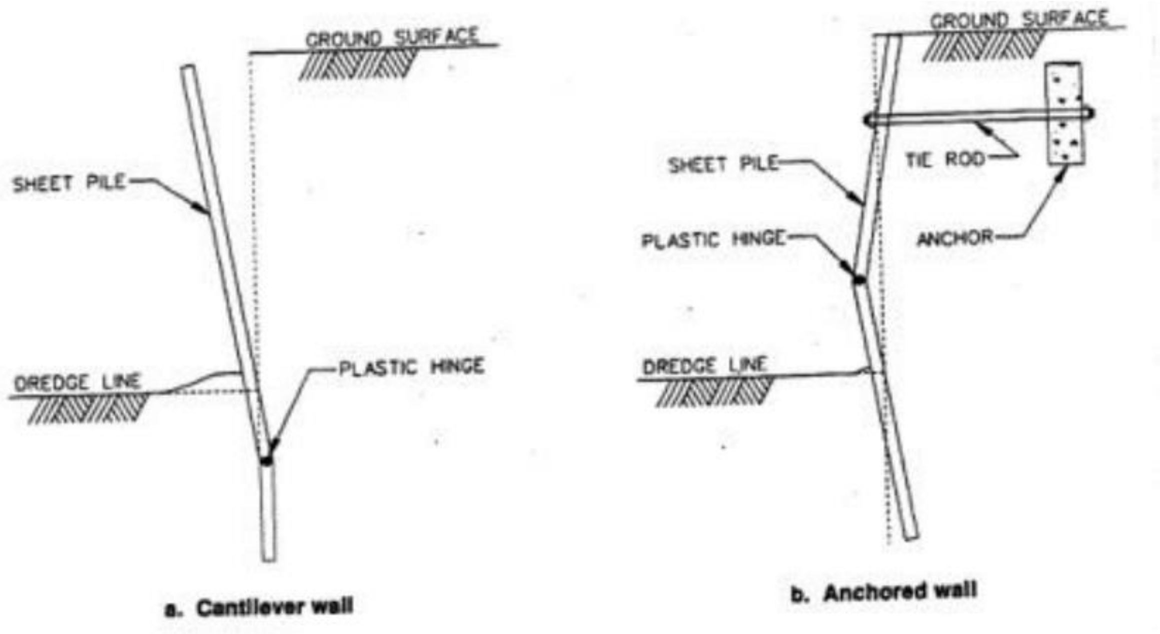


Figure 3.36 flexural failure in both cantilever and anchored sheet piles [29].

3.10.2 Rotational failure:

This type of failure occurs when the sheet piles are stiff however, the passive pressure force is not sufficient to resist the moment induced. As a result, the sheet pile wall rotates. To avoid this rotation failure the penetration depth, need to be increased because as the penetration depth goes deeper and deeper, the region of passive pressure becomes larger and larger. The greater the passive earth pressure the sheet pile wall has, the more stable it will be provided that the sheet pile wall is stiff [29]. The figure below demonstrates the rotational failure of both the cantilever and the anchored sheet pile walls.

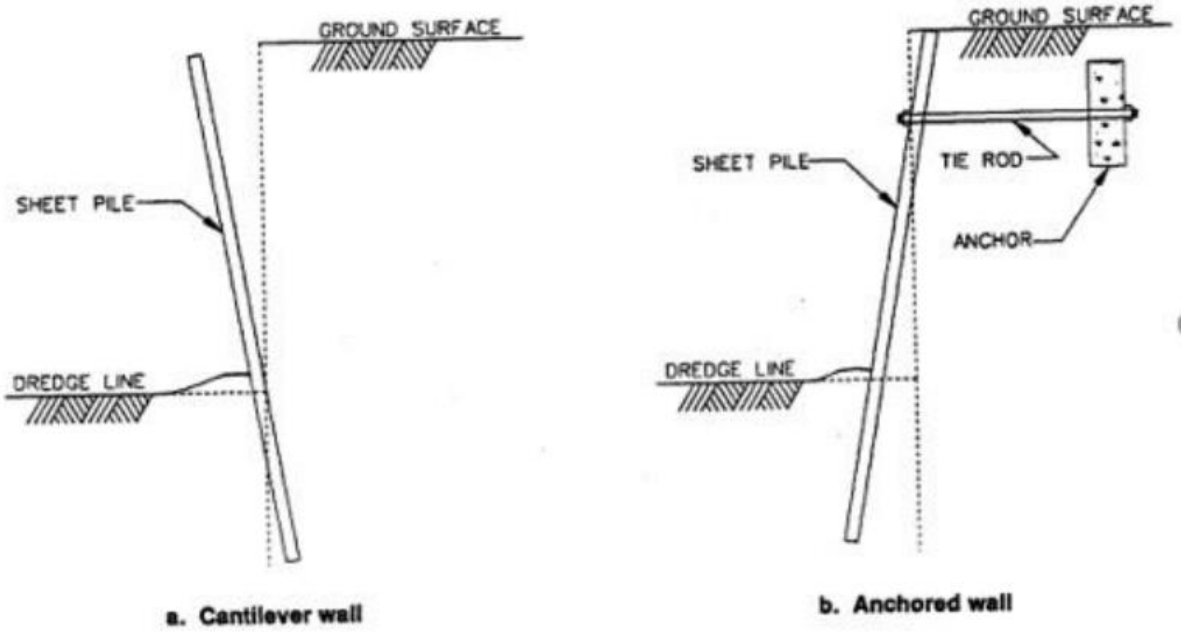


Figure 3.37 Rotational failure of cantilever and anchored sheet pile wall [29].

3.10.3 The deep-seated failure:

The deep-seated failure does not occur because of the structural design. However, it happens within the whole soil material that holds the sheet pile wall. The stability of the slope needs to be analyzed, and thus dewatering of the slope area and addition of structural fill are required to be performed to avoid such failures [29].

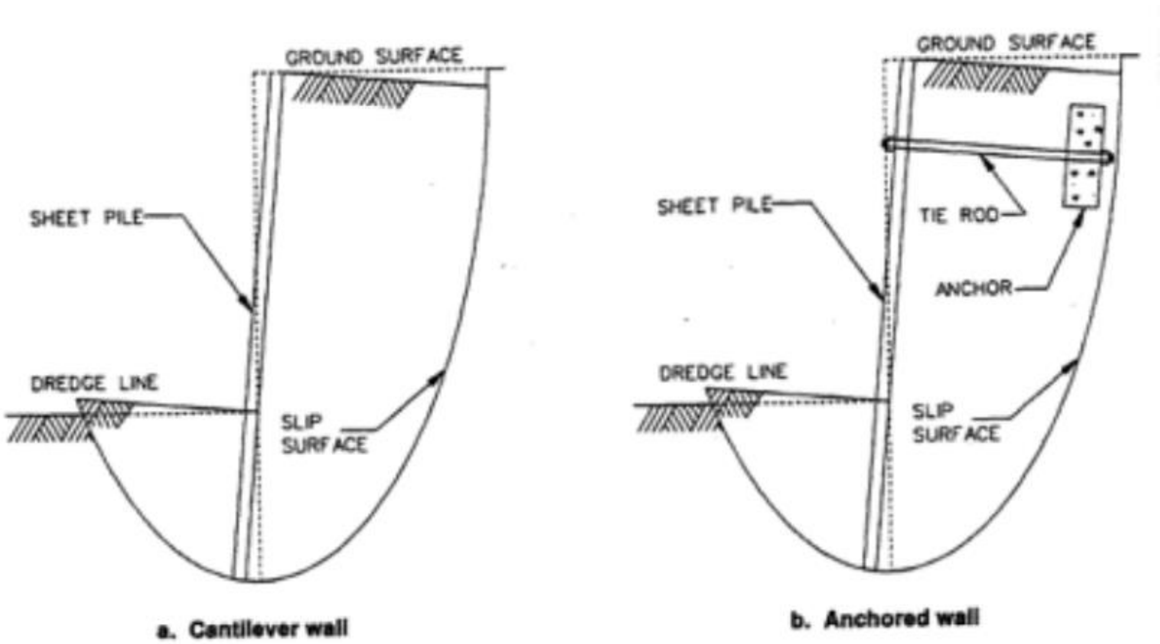


Figure 3.38 the deep-seated failure of cantilever and anchored sheet piles [29].

This type of failure occurs due to the weak cohesion force between the layers of the soils on the slope areas. Thus, the behaviors of the soil where the sheet piles are to be installed must be well studied and analyzed at the start of the project. Soils are generally complex and heterogenous having different layers. Stability problems occur due to the complexity of determining the strength of all layers of the soil [30].

3.11 The soil properties:

A soil may be strong or weak depending on its resistance to shear stress. This shear strength can be expressed in terms of its effective internal force of attraction (cohesion force C') and its internal friction angle. As soil is a complex material, its shear strength depends on the interaction of all its particles that contribute to its make-up, water content and its degree of compaction [31]. Table below shows the cohesion force according to soil type.

3.11.1 Cohesion force:

Cohesion is the force of attraction between the particles of the soil that can keep the soil's form in a limited time before it collapses. Cohesion force of soil depends on how well compacted the soil is and its amount of water content. The cohesion or attraction force of a soil varies as the amount of its water content in percent changes. For example, for a clay and clayey silt soils the cohesion force extends in the range 0 to 30KPa as the water content varies from dry to bleeding [32].

Table 3.4 cohesion vs water content for clay and clayey silt [32]

Soil type (clay)	Cohesion in KPa
dry	20<a<30
medium	0<a<20
Bleeding	a = c =0

The following figure demonstrates the cohesion force of sand, taken from Texas SandFest.

Figure 3.39 Texas SandFest: [33]

In Norwegian system cohesion force is called attraction (attraksjon) and is denoted by the symbol 'a'. The figure below is an NTH plot which is the most frequently used by the Norwegian road authority (vegvesen) [34].

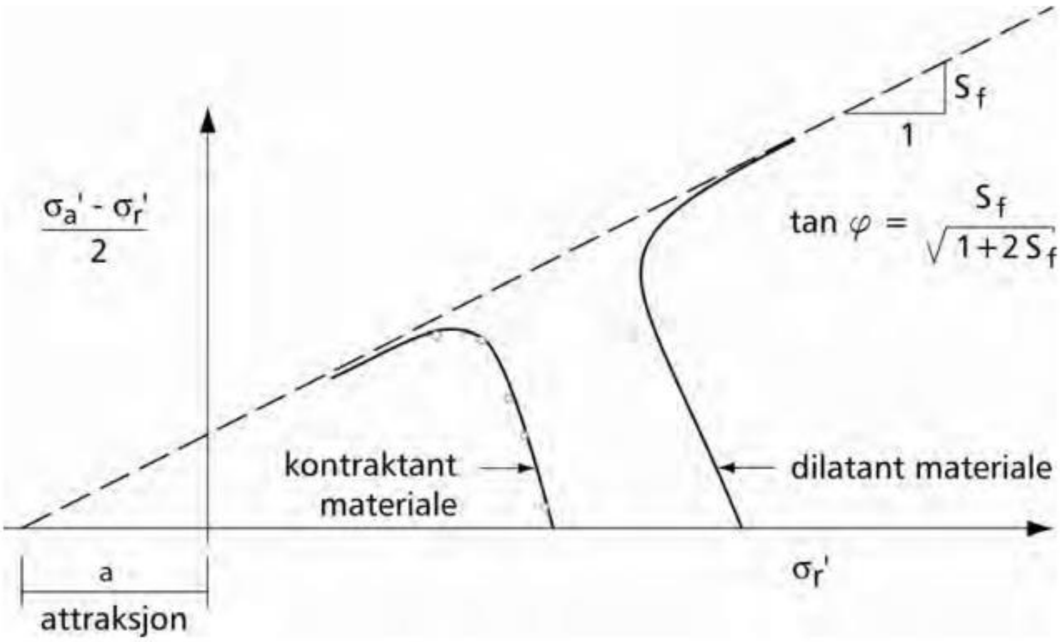


Figure 3.40 NTH plot taken from handbook 016 p 2-19 [34].

In the above NTH plot, attraction is the distance represented from the intersection point of the Y-axis (shear stress) and X-axis (normal stress) to the origin [34].

3.11.2 the internal friction angle (ϕ')

Friction angle is the angle one sliding soil material corn from the top of the collected mass makes with the horizontal flat surface where the soil material rests. In addition, friction angle can technically be defined as the angle between the axis of normal stress and the tangent line to Mohr's' circle at a point where it represents the failure stress condition for a certain solid material [35].

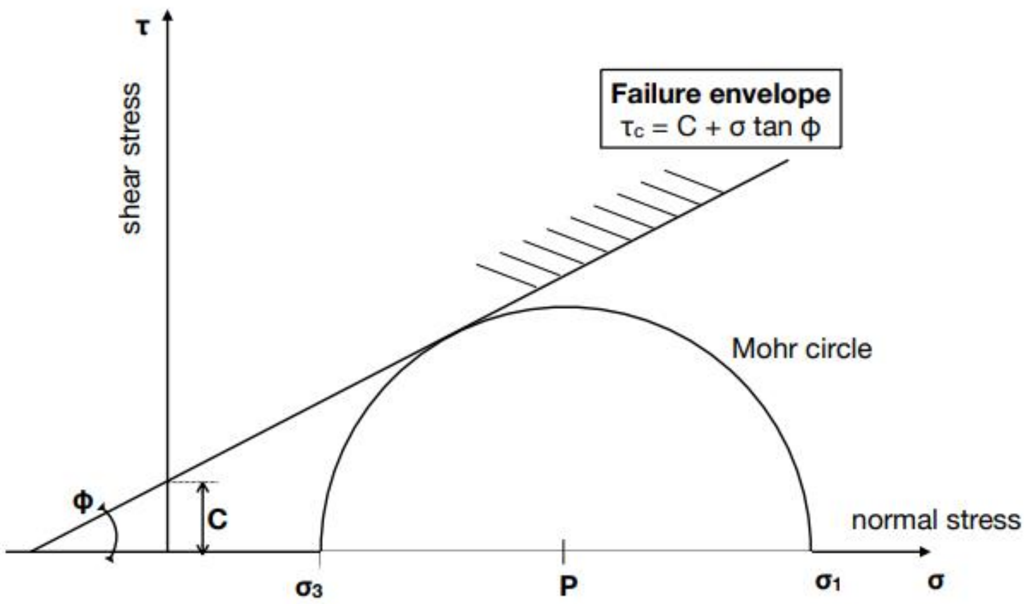


Figure 3.41 internal friction angle ϕ' [35]

The friction angle value of clay and silt soils varies with variation of water content in percent. As the water content ranges between 0 and 10% the internal friction value is about 30 degrees. As the water content increases to the range between 10 and 40% the friction angle reduces to the range between 30 to 20 degrees. If the water content exceeds 40% the internal friction angle is limited to 20 degrees [32].
A table of internal friction angle against its water content in percent for clay and silt soil is given below.

Table 3.5 friction angle vs water content in% [32].

Water content (w) in % of the dry soil	ϕ' in degrees
0-10%	30^0
10-40%	30^0-20^0
>40%	20^0

Table 3.6 typical soil shear strength based on soil type [31].

Soil type	Effective friction angle ϕ'		Effective cohesion c' (KPa)
	peak	residual	
Gravel	34	32	-
Gravel, sandy with few fines	35	32	-
Gravel, sandy with silty or clayey fines	35	32	1.0
Gravel and sandy mixture, with fines	28	22	3.0
Sand, uniform, fine grained	32	30	-
Sand, uniform, coarse grained	34	30	-
Sand well grained	33	32	-
Silt, low plasticity	28	25	2.0
Silt, medium to high plasticity	25	22	3.0
Clay, low plasticity	24	20	6.0
Clay, medium plasticity	20	10	8.0
Clay, high plasticity	17	6	10.0
Organic silt or clay	20	15	7..0

3.11.3 water content, degree of saturation and unit weight density:

In geotechnics it is useful to understand the mathematical relationship of the water content, the degree of saturation and the density unit weight of the soil in a construction site [36].

$$\gamma_n = \gamma_w * \frac{1+w}{\frac{1}{\rho_s}+\frac{w}{S_r}} \tag{3.42}$$

Where:

γ_n= is the natural unit density of soil (KN/m^3)

$\gamma_{w=}$ the unit density of water (10KN/m^3)

w= the water content of a soil

$\rho_{s=}$ dry unit weight of the soil

S_r= degree of saturation

Where degree of saturation is the ratio of the volume of water to the total volume of the pores in the soil [36].

$$S_r = \frac{volume\ of\ water}{total\ volume\ of\ pores} \tag{3.43}$$

The following chart helps to determine the water content, degree of saturation, porosity, and unit weight density of a soil provided that two of the above-mentioned quantities are given.

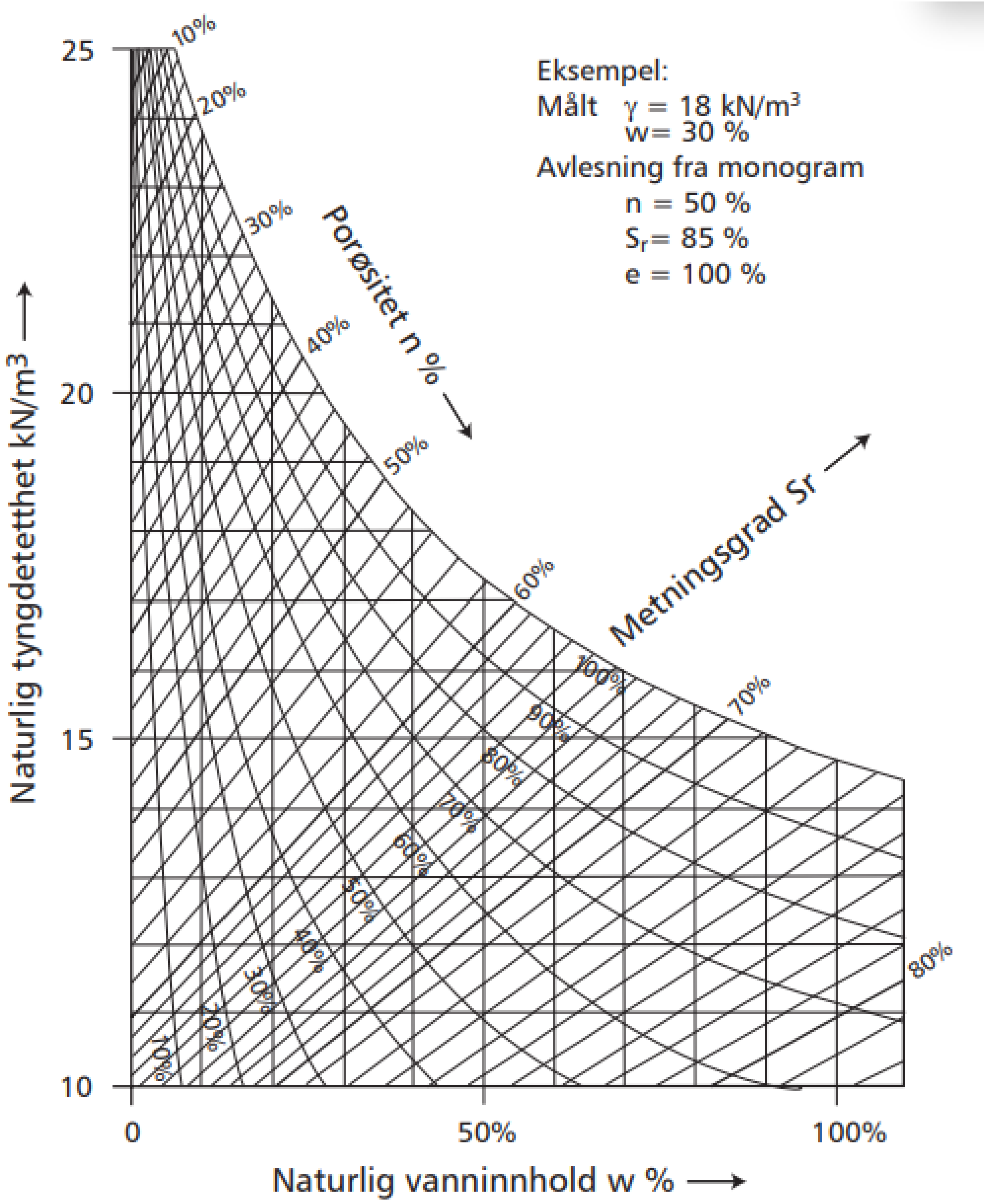


Figure 3.42 unit density as a function of water content and degree of saturation [36].

3.12 The water:

Water is one of the most essential materials however if it is not well controlled it is the cause of uncountable destructions in the construction industry.

3.12.1 Pore-pressure and frost:

An infiltration of rainwater into sediments and rock fractures leads to an increase of water pressures in the materials. These water pressures induce a failure. As the water in the cracks of the rocks gets frozen, its volume is increased by almost 9% that is sufficient and enough to induce huge tensile forces on the fractures. The figure below shows the average number of rock falls per month versus temperature and precipitation. The study was performed in Fraser canyon in the British Columbia region in Canada over the period of 1933 to 1977 [31].

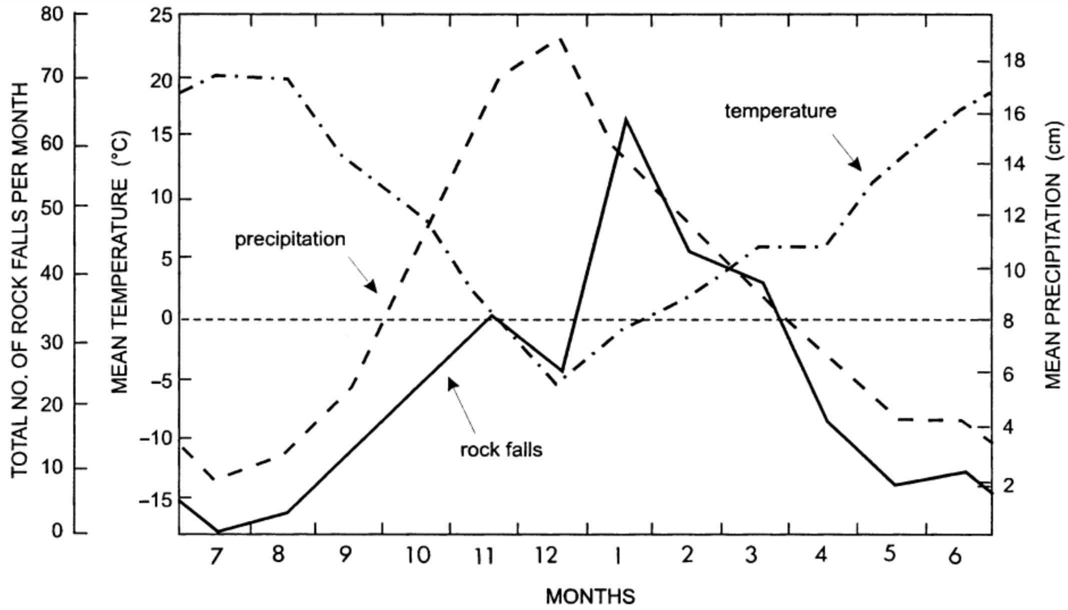


Figure 3.43 Average monthly rockfall frequency taken from page 599 [37].

As huge amount of water rains it increases the mass of the material on the earth that is located on unstable place around the construction site, and this further causes the gravitational driving force to be higher. The penetration of water into the joints of the permeable earth materials also elevates the pore-water pressure to reach its threshold of failure [37].

3.12.2 the drainage conditions

Due to the loading and unloading conditions there exists a change in stress in the soil. This change in stress further forces the pore-water pressure to change. Consequently, two types of drainage states arise. The two drainage classes are the drained and the undrained conditions [38].

3.12.2.1 Drained condition

After a time, soil undergoes some changes in stress and the water in the pores freely moves in and out of the pores without making any change in the pore-water pressure. The permeability characteristic of the soil is the main factor for how quick the water moves in and out of the pores [38].

3.12.2.2 Undrained condition

In the undrained condition, water is unable to flow in and out of the pores when the soil is subjected a change in stress. However, water is naturally incompressible and when there exists a change in stress in the soil, this will cause the water-pore pressure to undergo a huge change [38].

3.13 Basic concepts of slope stability:

When a surface of a soil creates a certain degree of inclination or depression with respect a horizontal plane is termed as a slope. Slopes can either be man-made or created because of natural

phenomena forces. Slope stability is mainly affected by the effect of gravity and water infiltration. A slope becomes unstable mainly due to two reasons. The first factor is a sudden or a long-term increase of stress because of an increase of the unit weight of the soil due to water infiltration in addition to the surcharge load which will further increase the driving force. The second factor is that the soil misses it strength due to pore-water pressure and soil cohesion reduction [39].
The effect of pore-water pressure, friction, and suction are the main factors that play a role on the fundamental concepts of slope stability [39].

3.13.1 Friction:

As the driving force (T) as seen in the figure below increases, the frictional resistance force (F) also increases. The values of these two opposite forces keep on increasing until these two bodies (the lower and upper) start to slide against each other. The figure below shows the concept of friction on the role of the slope stability [39].

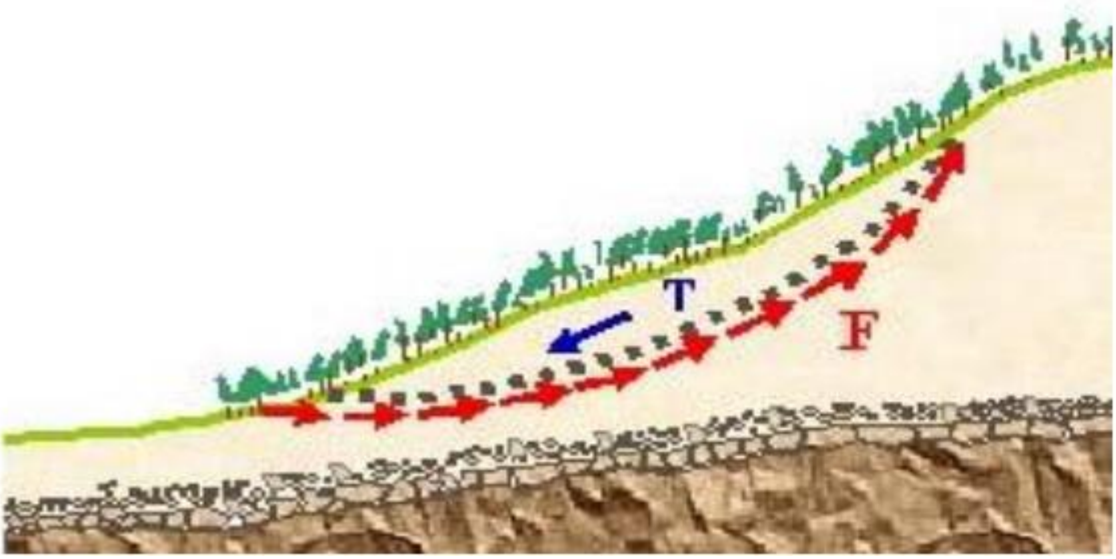


Figure 3.44 Friction concept (Gue and Fong 2003) [39].

3.13.2 Pore-water pressure:

When a layer of a soil is located below the level of the ground water table then the soil is considered as a saturated soil. The pore-water pressure is the pressure of the groundwater held between the soil particles that are found below the level of the ground water table. This water pressure causes to reduce the normal pressure that is available between the soil particles that are in contact. This further lowers the shear stress and its respective shear resistance of the slope. The effect of the pore water pressure force is associated with the seepage force. Seepage forces reduce the shear resisting force by relatively increasing the unit weight of the soil. The weight of the slope increases when water infiltrates into the empty pores and fractures in the soil. As the empty pores in the soil are filled by water, the water table raises up and causes the water pore pressure to increase. An increase of the water pore pressure generates the potential for the fall of the slope. Type and intensity of vegetation, geology and topography affect the amount of water that can be infiltrated into the slope [40].

3.13.3 Suction:

Suction is usually defined as an occurrence or a process when water is drawn out from a partially saturated soil due to evaporation. When the water is evacuated from a soil, the soil particles contract and pull-in toward each other because of the vacuum established. Such situations cause the normal pressure to increase, consequently the shear resistance pressure force increases [40].

3.14 Potential Slip surface analysis:

A potential slip surface is an assumption on a slope cross-section to investigate the stability of the slope under study as it is shown in the figure below. The term, the safety of factor (SOF) is used to determine if the slope is in the state of a stable or critical condition. The slip surface is expressed and analyzed in terms of the driving and resisting forces that occur at the cross-section where the surfaces slip. In determining the analysis of a slope stability, the main soil properties that are involved are the unit weight of the soil (ɣ), the internal friction angle (ϕ') and the cohesion of the soil (c') [41].

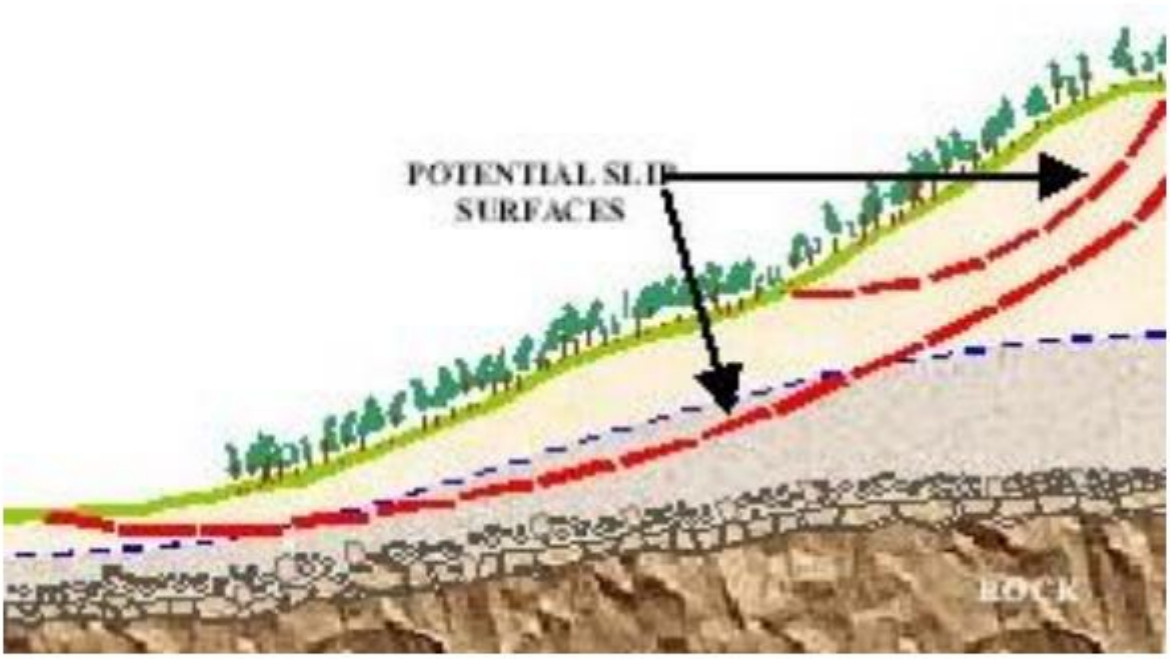


Figure 3.45 potential slip surface (Gue and Fong 2003) [41]

Driving force is the force that acts for the slope to fall. Gravity is the main driving force in most cases of land movement. The effect of gravity is favored by the slope angle, slope material, climate, and water. Steep slopes are more exposed than shallow slopes for mass movement. Water in the form of rivers or waves wash out and erode the base support of the slopes and facilitate the fall or moment of land. In addition, the surcharge load over the slope causes the driving force to increase.
Resisting force is the force that acts in opposite direction of the driving force. Resisting force is dependent on the shear strength of the slope material. Shear strength is dependent on the cohesion force and the internal friction angle of the soil [41].

Mathematically shear strength can be expressed as a function of internal friction angle and cohesion force as follows.

$$S = \delta'_n * \tan \phi' + c' \qquad (3.44)$$

Where S is the shear strength, δ'_n is the effective normal stress at failure, ϕ' the effective friction angle of the soil and c' is the apparent cohesion force of the soil [39].
Water contributes not only an increment of the driving force but also increases the resisting force if the pores are partially filled with water. When the pores are partially filled with water, a thin film is created and acts as a binding cohesive force. Thus, the shear strength increases as the resisting forces increase.
Safety of factor (SOF) is the ratio of the resisting force to the driving force.

$$\boldsymbol{SOF} = \frac{\overline{S}}{\tau} = \frac{\textbf{the resisting force}}{\textbf{the driving force}} \qquad (3.45)$$

Where, $\overline{S}$ is the average shear strength available along the failure surface and $\overline{\tau}$ is the average shear stress developed along the shear surface. A slope with a smaller SOF value has a higher risk to fall and a slope having a greater SOF has a lower potential to fall [41].

3.15 stability of a construction:

There are three factors that affect the stability of a construction at its bottom. These are bottom upheaval, bottom compression, and hydraulic foundation failure. These factors need to be checked to ensure the stability of every construction [42].

3.15.1 Bottom compression (pressure)

The safety for bottom compression is controlled by checking the total stress analysis at the ultimate limit state. The safety is expressed by calculating the material coefficient (γ_m). The bottom compression depends on the bearing capacity factor, the depth to length ratio of a foundation, and width to length ratio of the sheet pile wall [42].

$$\gamma_m = \frac{N_c * S_u}{\gamma * z + q_{d-} p_d} \qquad (3.46)$$

Where:
γ_m= is the material coefficient,
Nc= is the bearing capacity factor,
Su =is the undrained shear strength,
γ = the average unit density of the graved soil
Z= is the depth,
qd = the design terrain load
pd = th design pressure at the bottom of the construction.
The diagram below shows the relationship between the bearing capacity factor (vertical axis) and the depth to width ratio for three different structure shapes. The three different shapes are represented by a, b, and c. where: a represents a square or a cylindrical shape, b represents a rectangular shape where the length is double as big as the size of the width and c represents a shape where the length is many times greater than the width. The ratio of the length of a sheet pile width (thickness) to its

length is almost zero. Therefore, in the diagram below c represents the sheet pile width to length ratio [42].

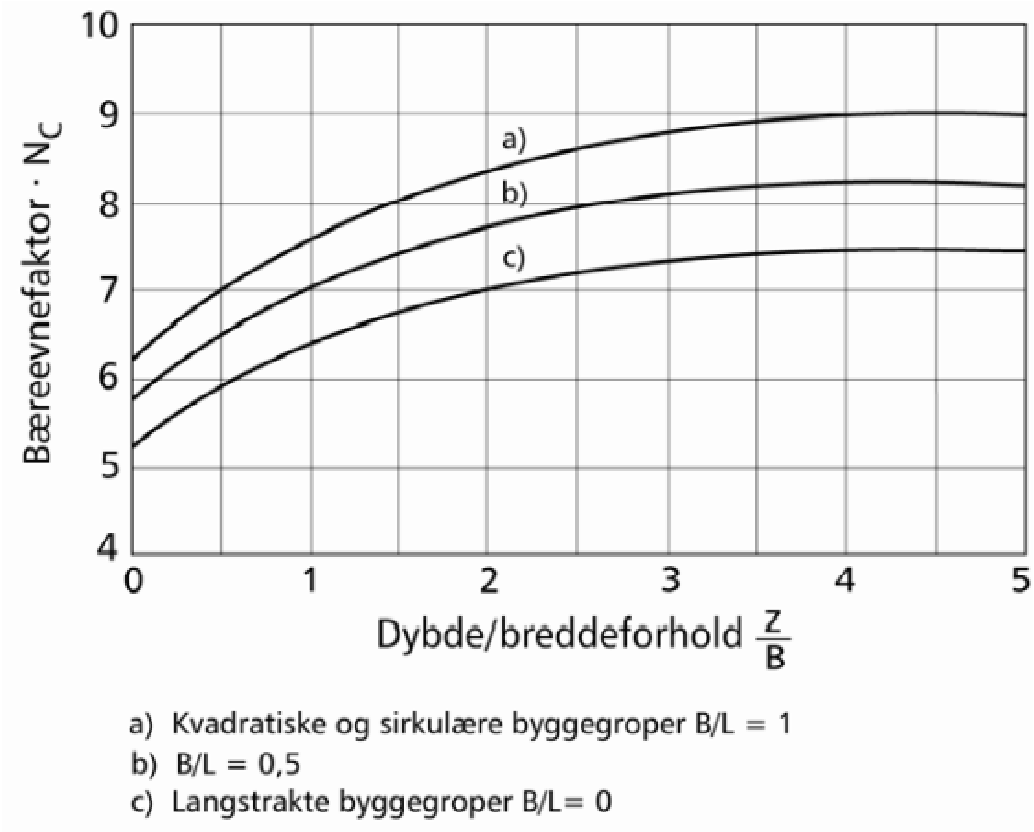


Figure 3.46 bearing capacity factor vs different structure shapes. Taken from handbook 016(page 10-21) [42].

3.13.2 Hydraulic foundation failure:

While excavating a foundation below the ground water level in coarse sized soil, the upward pressures become greater and greater. As a result, the hydraulic pressure foundation failure occurs. This happens when the vertical current pressure ($i*\gamma_w$) is equal to the downward density of the loose masses at the bottom of the foundation (γ') [42].

$$i * \gamma_w = \gamma' \tag{3.47}$$

Hydraulic failure= $i = i_c = \frac{\gamma'}{\gamma_w}$ (3.48)

On the finished excavation the highest upward outlet gradient at the bottom (i_0) is required to be lower or equal to the allowed outlet gradient [42].

$$i_0 \leq \frac{i_c}{\gamma_m} = \frac{\gamma'}{\gamma_m \gamma_w} \tag{3.49}$$

For a temporary excavation depth of a soil with a homogeneous and coarse sand ɣm need to be greater than 1.5 in the ultimate limit state. However. for a homogeneous fine sand ɣm is required to be greater than 2 to 2.5.

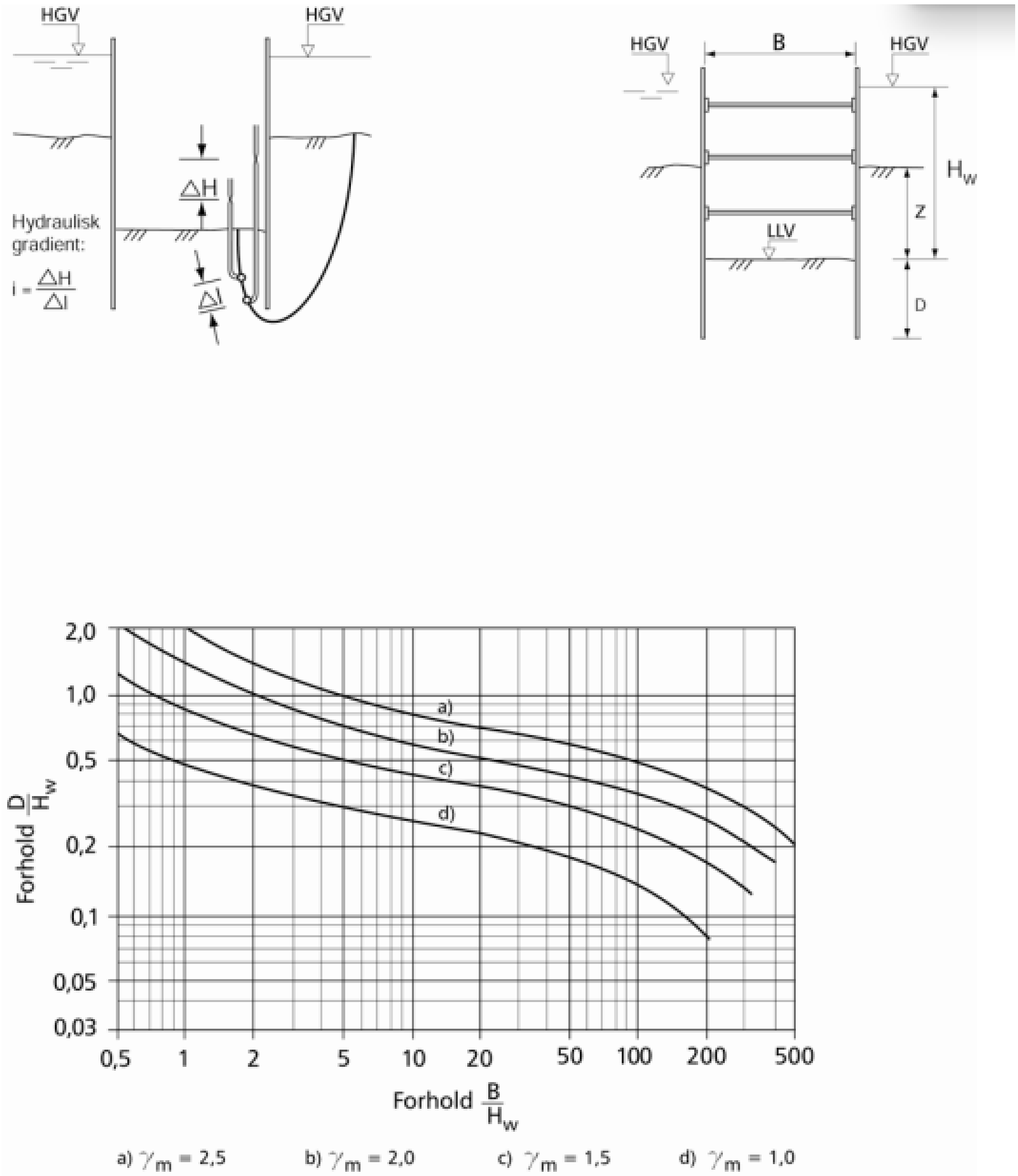


Figure 3.47 safety against hydraulic foundation failure (taken handbook 016 (p 10-22). [42]

3.15.3 upheaval at the bottom

In the case of layered ground if a dense layer is left over a permeable coarse sized soil masses after excavation, one can see upheaval of the bottom part in the form of a single and large block type cracks. Such problems occur because of excessive pore pressures. To avoid such problems a pore pressure monitoring is recommended according to its necessity. Installing of pump wells is one recommended solution to reduce the pore pressures [42].

3.16 Stress analysis:

To determine the horizontal pressures in both the drained and undrained conditions for different soil types a few stress analyses need to be checked. The total stress, the effective stress, and the a-ϕ stress analyses will be described below.

3.16.1 Total stress analysis:

The total stress analysis is used to determine the horizontal pressure in undrained condition where the water in the pores cannot move freely in and out of the pores. Such type of analysis is applied on impermeable or less permeable masses.

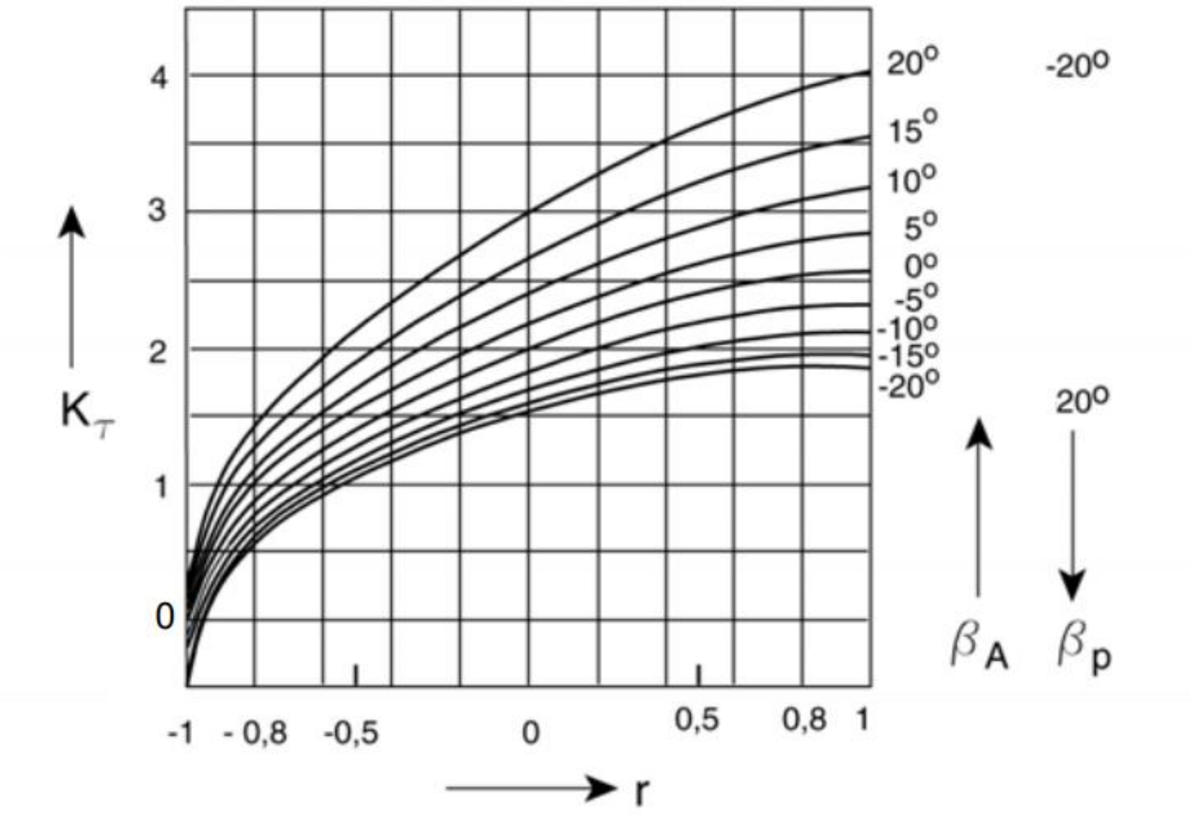


Figure 3.48 total stress analysis, earth pressure coefficients by NTNU. taken from handbook 016 (page 5-9) [43].

The horizontal pressures in the active and passive states are determined by the help of mobilized undrained shear strength and the coefficients of the earth pressure after reading from the figure above [43].

The horizontal pressure on the passive state is calculated as follows:

$$P_p = q + \gamma * z + K_\tau \tau_c \qquad (3.50)$$

Where:

$$\tau_c = \frac{C_u}{\gamma_m} = f * C_u \qquad (3.51)$$

P_p= is the horizontal pressure in the passive state.

q= is the load

ɣ= the unit density of the material

z= the height of the soil material

K_τ= earth pressure coefficient

τ_c= mobilized undrained shear strength

The horizontal pressure on the active state will be:

$$P_A = q + \gamma * z - K_\tau * \tau_c \tag{3.52}$$

3.16.2 Effective stress analysis:

Effective stress analysis is relevant and useful to find the horizontal pressure for fine-grained soils like silt and clay in the long-term for the drained situation. However, a short time and great rate load effects are analyzed based on the total stress analysis. Alternatively, rapid load effects are analyzed based on the effective stress basis for the undrained condition. If we assume a horizontal terrain the earth pressure coefficients Ka and Kp depend on the roughness ratio r and the mobilized friction angle ($\tan\rho$) [44]. The earth pressure coefficients for a horizontal terrain can be found using the figure [44] below.

Effective stress is generally defined as the difference between the earth pressure as a function of its unit density and the water pressure as function of its height.

$$Effective\ pressure\ = (vertical\ earth\ påressure - vertical\ water\ pressure)$$

Where:

Vertical earth pressure= $P_a = \gamma * H$ (3.53)

and

Water pressure = $P_w = \gamma_w * H_w$ (3.54)

Thus. Effective pressure = $P'_a = P_a - P_w$ (3.55)

Where ϕ is the friction angle of the soil and ɣm is the material property and ρ is the mobilized friction angle then:

$$\tan\rho = \frac{\tan\phi}{\gamma_m}$$

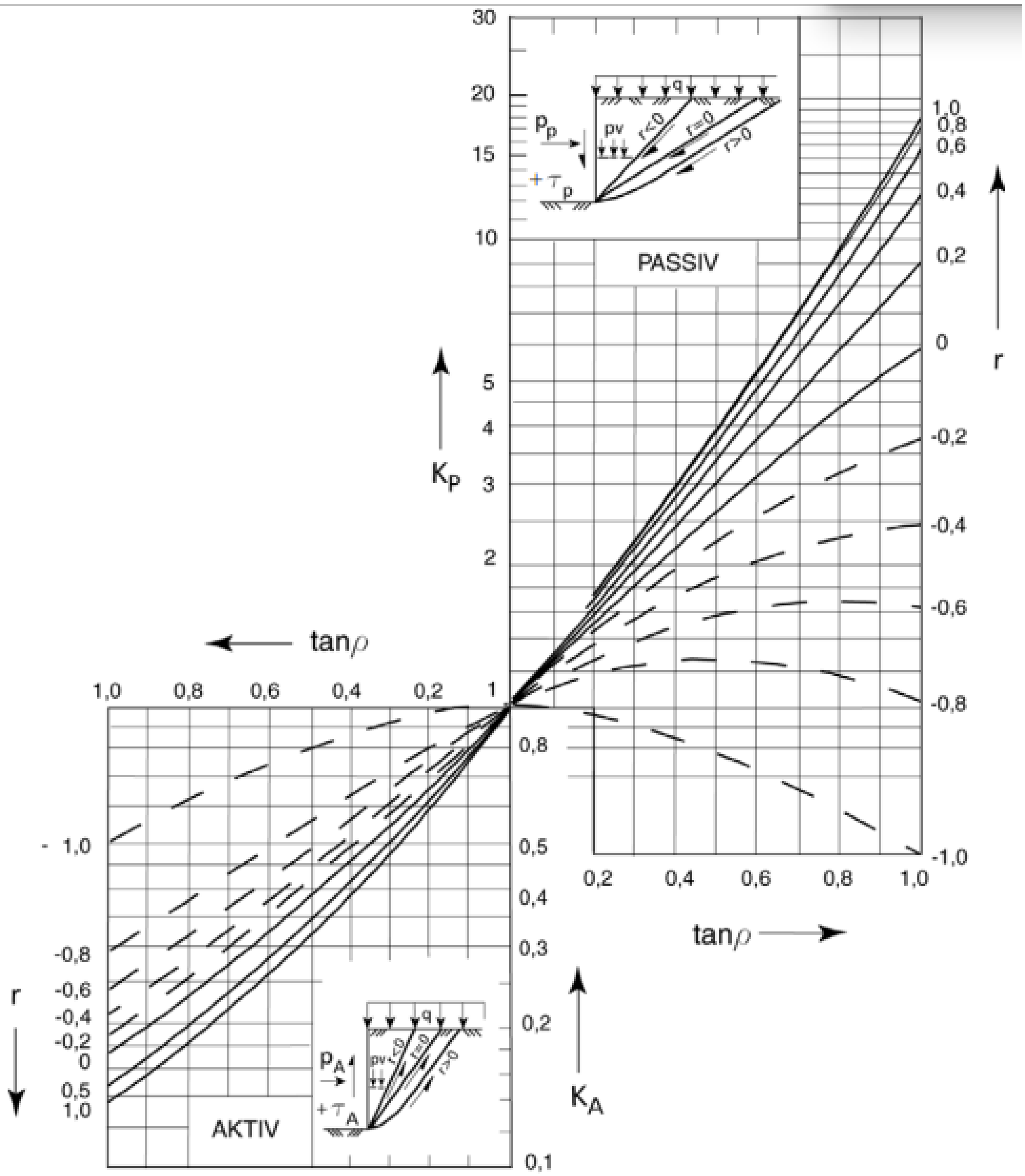


Figure 3.49 effective stress analysis- earth pressure coefficients (by NTNU). Taken from Handbook 016-page 5-5 [44].

3.16.3 The a-ϕ stress analysis:

The a-ϕ analysis refers to the analysis of a shear strength of clay soil within a natural slope as a function of the attraction or cohesion force (a) and the internal friction angle (ϕ). This a-ϕ analysis is used for drained case conditions. As a result, this analysis is used for long-term stability or for open materials. It is obviously clear that a natural slope in clay has relatively a higher safety in drained case than in the undrained condition. Therefore, it is recommended that the analysis in both the drained and undrained conditions need to be taken for a clay soil with a natural slope [45].

Where a is the attraction force pressure, α is the natural slope angle in degrees, γ_m is the partial factor or the material property of the soil, ϕ is the friction angle, then the shear strength is expressed as:

$$\tau_k = \frac{(a+p_0)*\tan\phi}{1+\tan\alpha*\frac{\tan\phi}{\gamma_m}} \tag{3.56}$$

3.17 Corrosion rate of steel sheet piles:

Sheet piles lose their thickness due to corrosion depending on the aggressiveness of the media where the sheet piles are in contact with. As the sheet pile losses its thickness its durability both in the ultimate and serviceability limit states is negatively affected. The corrosion rates of the sheet piles apply on the sides where the sheet piles are in contact with the media. Therefore, while designing a sheet pile the rate of corrosion on each side must be accounted. When the sheet pile is rounded by same media, for example soil or water, then the rate of corrosion is the same on all sides. How ever if some parts of the sheet pile are bound by different media with different aggressiveness properties, then the rate of corrosion on the sheet pile surfaces will naturally be different. For economic reasons the sheet piles need to be designed in that way. The loss of thickness due to normal atmospheric corrosion is taken as 0.01 mm per year. When the structure performance is affected by marine conditions, then the rate of thickness loss due to atmospheric corrosion is 0.02mm per year. When soil is considered as a bounding media then the rate of corrosion depends on the type of soil, the variation of level of the ground water table, the presence of oxygen and contaminants in the soil [46].

The table below shows the recommended value of steel sheet pile's thickness loss [mm] in soils due to corrosion in the presence or absence of water ground table.

Table 3.7 rate of corrosion of steel sheet piles in the absence or presence of ground water table [46]

Required design working life	5 years	25 years	50 years	75 years	100 years
Undisturbed natural soils (sand, clay silt and schist)	0.00	0.3	0.6	0.9	1.2
Polluted natural soils and industrial sites	0.15	0.75	1.5	2.25	3.00
Aggressive natural soils (swamp, marsh, and peat	0.2	1.00	1.75	2.25	3.25
Non-compacted and non- aggressive fills (clay, silt, sand, and schist)	0.18	0.7	1.2	1.7	2.2
Non-compacted and aggressive fills (ashes and slag)	0.5	2.0	3.25	4.5	5.75

NB. The rate of corrosion in non-compacted fills is twice as fast as the one in compacted ones [46].

Sheet piles installed in fresh and seawater, lose their thickness due to corrosion more differently than those installed in soils as described above. The table below shows a recommended thickness value loss in mm for sheet piles due to corrosion in fresh and seawater.

Table 3.8 steel sheet pile corrosion rate in fresh water and seawater [46]

Required design working life	5 years	25 years	50 years	75 years	100 years
Fresh water (river, ship canal) in the high attack zone (water line)	0.15	0.55	0.9	1.15	1.4
Very polluted fresh water (sewage, and industrial effluent) in the zone of high attack or the water lie	0.30	1.30	2.30	3.30	4.30

Sea water in temperate climate in the zone of high attack (low water and splash zones)	0.55	1.90	3.75	5.60	7.50
Sea water in temperate climate in the zone of permanent immersion or in the intertidal zone.	0.25	0.90	1.75	2.60	3.50

NB: The splash zone or the lower water level in the tidal waters is usually the zone with the highest rate of corrosion. However, the highest bending moment stresses are seen in the regions of permanent immersion [46].

3.19 computer programs:

Excel is the software used for calculating the lateral earth pressures, excavation length of sheet pile wall, moment capacity and section modulus for a given angle of friction and sheet pile length over the dredge line. In addition, power point was used to draw figures that represent earth pressure distribution, and straight lines over the stress analysis charts in determining the lateral earth pressure coefficients.

4. Research question:

Sheet pile walls are used to retain water or soil in slope areas. All sheet piles must tolerate the induced moment by the lateral pressure forces exerted from the retained soil or water waves. The lateral bearing capacity of a sheet pile wall depends on many factors. The stability of a construction on slope areas is more sensitive than on flat areas. To avoid such risks and improve slope stability, pile sheeting is needed to support and stabilize a construction. This master thesis will focus on the role of sheet piles over buildings constructed on slope areas having various soil texture profiles.
Thus, the research question is:
How can one keep the stability and durability of sheet piled constructions on slope areas?
Sub questions:

1. What is the concept of pile sheeting?
2. How do the horizontal forces act to stabilize the sheet piling?
3. What are the main factors that affect the stability of sheet piled constructions?
4. How can one select the size and material type of a sheet pile for a certain construction?
6. How can one keep the durability of sheet piled constructions?

4.1 limitations:

This investigation is limited to only cantilever sheet pile walls.
No economic calculations are included in this study.
No laboratory investigations are performed in this master thesis.
Designing of steel quality was not included.
Environmental impact calculations are not included.

5. Case

This master thesis investigates the importance of sheet pile walls and how they are selected for a certain project. The main purpose of this report is to determine and describe all the sectors that possibly enable a sheet piled construction to be a stable and durable. The stability and durability of a construction for retaining the earth materials on slope areas are the main basis for the investigation of this study.

The properties of the soil under the sheet piles and the backfills that exert horizontal forces on the sheet pile wall are studied. The necessity of the sheet pile walls and their application areas as well as the way how they are selected for a certain task are the basis of this master thesis.

Sheet pile walls are used to conservate the nature against erosion. When the slope areas around buildings are eroded, the base of buildings is weakened and thus the stability of the whole building will be at risk. Destruction of a buildings due to lack of control of running and infiltrating water on slope areas around the residential or commercial houses imposes an intention to design and build retaining walls or sheet piles walls and this intention acted as a basis to work with in this master thesis.

Another important role of sheet piles included in this master assignment is that they can be used as a temporary soil retainer in construction sites where there is a deep excavation. Deep excavations displace huge amount of soil. This displaced soil needs to be controlled and contained by these sheet pile walls unless this displaced soil would be a threat for the safety of the workers lives and the machines in the construction site.

Four types of theories are described in this master thesis. Each theory has its own approach regarding on the geometry of the soil movement as the sheet pile wall goes away or closer to it. Sheet piles are made up of different materials out of which steel is the most dominant because of its availability, strength, and durability. Although there many types of sheet pile wall materials, however due to its availability, steel is the most used all over. Thus, steel sheet piles are deeply described in this project.

The whole investigation of this master thesis is dependent on scientific articles, standards, Norwegian and international handbooks, textbooks and Euro codes and hand-written calculations.

6. Method

This section of the master project briefly describes all the way how the project scope as well as the thesis question and the sub-questions are solved. It explains every process that is used in solving all the works done from the start to the final part of the project.

The type of method used to solve this master thesis is a qualitative method. Credible scientific articles, reports, and handbooks both the Norwegian and some other international are the core of the sources used in this master project. More dominantly the Norwegian national authority for road building's handbook 016 is the most used. In addition to these, textbooks, background knowledge acquired from all previous lectures and some handwritten calculations are among the main pillars in completing the project.

6.1 The first phase of the method:

As a first step, the schedule of progress plan and the thesis question for the project were the first two assignments proposed and delivered as an obligatory part of the project. After these assignments were approved the next step was to gather credible sources that can solve the scope of the master project.

6.2 The searching method:

The Web of Science, ResearchGate and ScienceDirect are the key search tools used to gather the necessary scientific articles. It is easier to use these search tools because the student account access is available for signing in. In addition, these search tools enable the user to acquire and analyze the needed data in a title wise arranged and timely manner.

Thes first step of using these search tools is to sign in to and get access over it. After signing into each of these search tools, one can see all the relevant and irrelevant scientific publications are listed down according to the time they are published. However, these publications can be categorized and filtered according to the document type.

To find a relevant article, I selected article out of the many document type alternatives and input some key words from the research question in the refine results part and searched as a first trial. At the first trial the chosen key words were ''sheet pile''. As a result, many articles that include sheet pile in their title were listed down. For example, when the sheet pile was used as a searching key text topic ''for article type document'' the following 646 results were obtained as shown in the figure below taken a screen shot. How ever, it does not mean that all these results are relevant sources for the project. The abstract part of all the articles is available that every user can read them. However, the whole content of the listed articles may be open that can be read and downloaded or may not be open that they need further access keys from the author or the publisher.

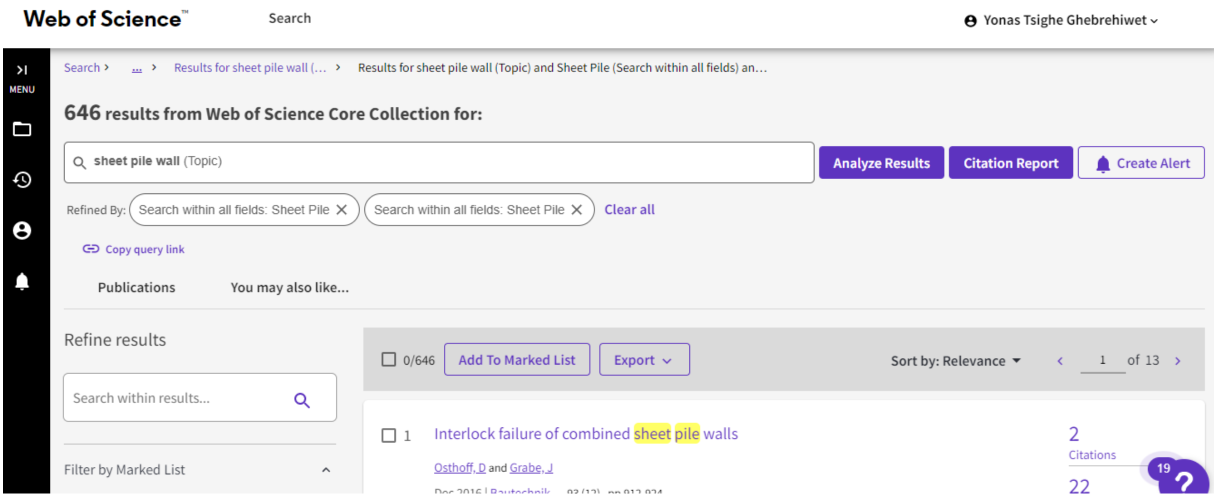


Figure 6.1 way of searching articles in Web of Science (taken from screen shot)

6.3 Selecting the relevant articles:

The articles with complete studies which consist of abstract, theoretical background, results and conclusion were chosen to make a fast overview on them. Out of these chosen articles the abstract part was read, and the relevance of each article was analyzed. These articles with complete investigations which can partially or fully answer the research question were categorized as relevant articles. These content of each of these selected relevant articles were further deeply read and used as part of the source of this master project.

The investigations written other English language were not included as relevant articles. Moreover, the geographical locations where the studies were performed were not limited. In addition, the latest time of publications were prioritized. However, as the latest articles were not enough to cover and to completely answer the research question and the scope of the whole project, limiting the time of publications was avoided. By varying the key words used in the thesis question as searching term the process was repeated until both the main thesis question and the sub questions are answered.

In addition to these scientific articles, it was found that standards and handbooks related to sheet pile walls were interesting. The google search engine was another search tool used to collect standards such as (Eurocode 3 - Design of steel structures - Part 5: Piling) and handbooks for example (the Norwegian road administration handbook 016) that are available for free and do not require any account.

To have a better and deep understanding on sheet pile wall concepts the textbook named as << PRINCIPLES OF FOUNDATION ENGINEERING, 8TH EDITION>> was used as the main reference. After getting a deeper understanding on sheet pile walls, it was helpful to differentiate and analyze which part must be included or excluded in the study, as a result it was a back-and-forth process. Moreover. the thesis question was altered and modified many times. This is the reason that some sources are found early in the start of the project and others are found late in March.

6.4 handwritten calculations:

As for part of the results, a handwritten calculations are included to analyze the differences that occur on maximum moment and section of modulus and the total sheet pile wall length required by changing the excavation depths below and over the ground water table. Another alternative was that the

excavation depth was taken constant, and the internal friction angle was altered from 30 to 36 degrees. The result of each while number angles was obtained. In both cases the type of sheet pile profile was selected depending on the size of section of modulus and maximum moment because flexural bending moment is the weakest axis side of sheet pile walls.
Four different theories on lateral earth pressure are included in this project, out of which the Rankine method is chosen to perform the handwritten calculations.
To have an additional and an easier understanding for the reader in analyzing the consequences of altering the excavation depths and internal friction angle value on section modulus and maximum moment capacity, a cantilevered sheet pile wall in a sandy soil was chosen.

7. Result

This chapter generally consists of two parts. The first part contains calculations obtained in designing a cantilever sheet pile wall into two tables. The first table contains a result that are obtained by changing the sheet pile wall lengths just above and below the water ground level or the sheet pile wall length above the excavation level (L1 and L2) for a cohesionless sandy soil where the cohesion pressure force is assumed to be zero (c=0). The second part consists of results obtained by varying the internal friction angles from 30 to 36 degrees.

The second part of this result chapter is composed of results obtained from the literature studies, EUROCODE 3 standard EN 1993- 5 (2007) English version, the Norwegian handbook 016, and other international handbooks that partially or fully answer the thesis question or deeply describe the subject matter.

7.3. Result from the earth pressures:

The earth pressure theory includes three different pressure expressions, the at rest earth pressure, the active earth pressure, and the passive earth pressure. The active and passive earth pressures act in pashing wall the vertical wall from two opposite directions. The active earth pressure pushes the sheet pile wall away from the backfill, whereas the passive earth pressure pushes the wall towards the backfill.

At rest earth pressure is a horizontal pressure force that acts on a vertical wall. It causes insignificant horizontal deflection or no deflection at all. It depends on the size of the ratio of the horizontal movement of the head of the wall to the height of the wall. It also depends on the internal friction angle and the Poisson's ratio of the backfill soil. Here are the boundary limits of transition from at rest earth pressure to active and passive earth pressures for different backfill soils [9].

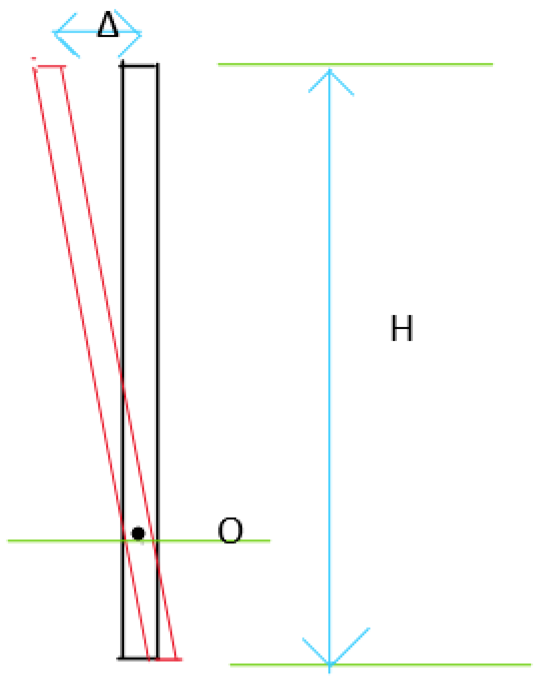


Figure 7.1 demonstrating Δ and H. illustrated by Yonas Tsighe

Table 7.1 the effect of soil strength on stability of wall [9]

Type of backfill soil	Value of Δ/H	
	Minimum Active	Maximum passive
Dense sand	0.001, Δ=0.1% of H	0.01=1% of H
Medium dense sand	0.002, Δ=0.2% of H	0.02= 2% of H
Loose sand	0.004, Δ=0.4% of H	0.04= 4% of H
Compacted silt	0.002, Δ=0.2% of H	0.02=2% of H
Compacted lean clay	0.01, Δ=1% of H	0.05=5% of H
Compacted fat clay	0.01, Δ=1% of H	0.05=5% of

The table below also shows the effect of the internal friction angle and the Poisson's ratio of the backfill soil on the value of the coefficient of at rest earth pressure constant. The magnitude of the coefficient of the at rest earth pressure is directly proportional to the value of at rest earth pressure according to equation (3.1).

Table 7.2 soil strength versus at rest pressure [8].

Backfill soil type	Poisson's ratio (v)	calculated $K_0 = \frac{v}{1-v}$
Loose sand	0.2-0.35	0.25-0.54
Dense sand	0.3-0.4	0.43-0.67
Sandy soil	0.15-0.25	0.18-0.33
silt	0.3-0.35	0.43-0.54
unsaturated clay	0.35-0.4	0.54-0.67
Saturated clay	0.5	1
Clay with sand and silt	0.3-0.42	0.43-0.72

Saturated clay has the highest poisons ratio (v=0.5) and thus the highest coefficient of the at rest earth pressure (K0=1) out of the soil types listed in the table above. However, a loose sand has the least.

Regarding the relationship between the deformation sizes due to the active, passive and the at rest earth pressure are shown on the figure below. From the figure it is shown that the passive earth pressure is greater than both the active and the at rest earth pressures. The deformation in the at rest earth pressure (marked in green color) is a transition to both the active and passive deformations.
δA is a deformation in the active pressure, δ0 in the at rest pressure and δp is in the passive pressure.

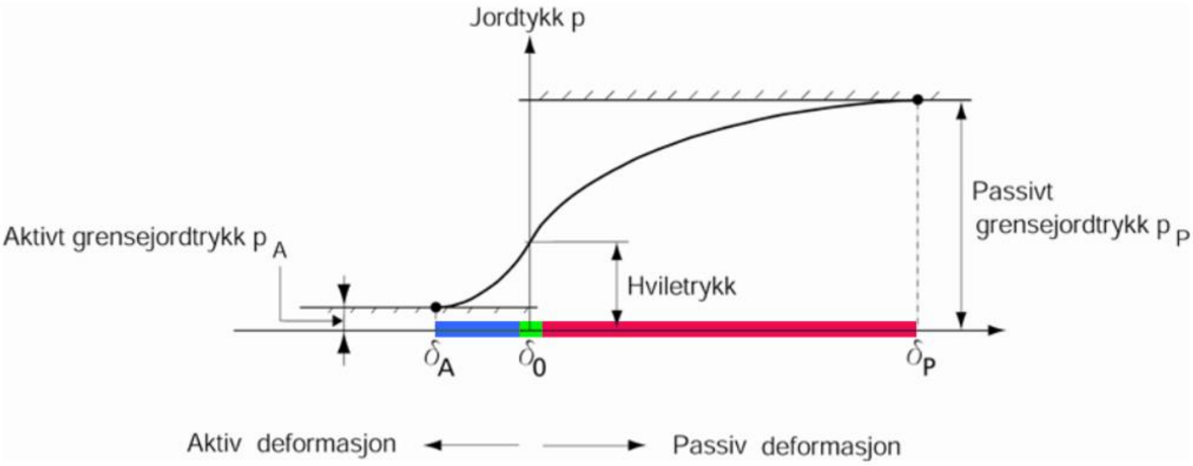


Figure 7.2 deformation vs earth pressures [10].

7.4 Result from the lateral earth pressure theories:

In this part, the lateral earth pressure includes four different theories, namely the Rankine theory, the Coulomb theory, the Terzaghi or the log spiral theory and finally the Janbu's theory.

In the Rankine method, the active and passive lateral coefficients are expressed just as a function of the internal friction angle. This method assumes that the wall is smooth and considers no friction between the wall and the backfill soil. It also assumes that backfill soil is homogeneous and cohesionless. The pressure diagram is a set of straight lines with the coefficient of the lateral passive and active pressure as slopes [11, 12].

The Coulomb theory assumes that the soil fails as solid body and firm body in the same direction as the shear plane. The passive earth pressure coefficient is expressed as a function of the ϕ, α, δ and β. The soil failure is assumed as a linear movement the same as on the Rankine method.
Alternatively, the passive pressure coefficient can be determined as a function of the internal friction angle of the backfill (ϕ) and the wall friction angle (δ) as shown from the graph below. The Kp is determined by extension of a a vertical straight line from the wall friction angle on the x-axis to the required curved graph of the internal friction angle, and then a extending a horizontal line to read the Kp value. Depending on the chart below the following coefficient of passive pressure values can be obtained.

Table 7.3 friction angle, wall friction angle vs Kp

ϕ	δ	Kp
30	15	5
35	17.5	7.5
40	21	12.5

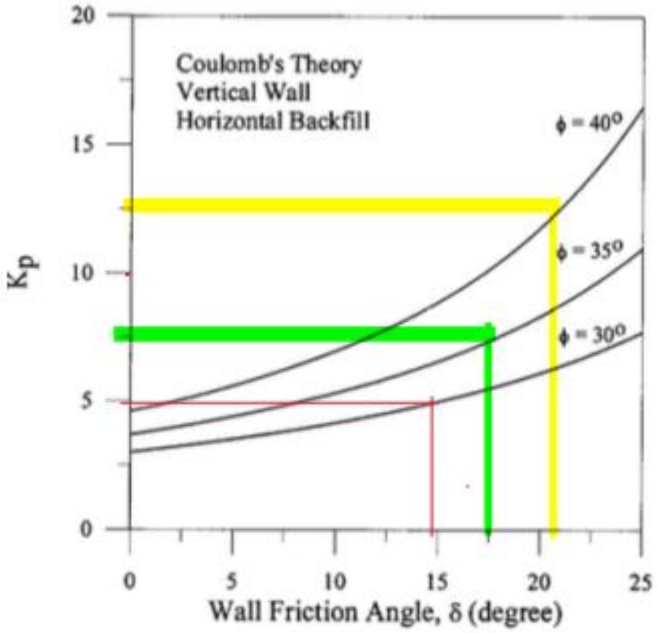


Figure 7.3 Kp versus internal friction angle (ϕ') and soil-wall friction angle (δ) (taken from Master thesis NTNU, Mari Melhus Romstad)

The Terzaghi or the log spiral theory assumes that soil failure occurs as a combination of curved and linear movements. The curved geometry is due to the wall friction angle and occurs at bottom part near the wall and the linear movement occurs at the upper part as shown below [13].

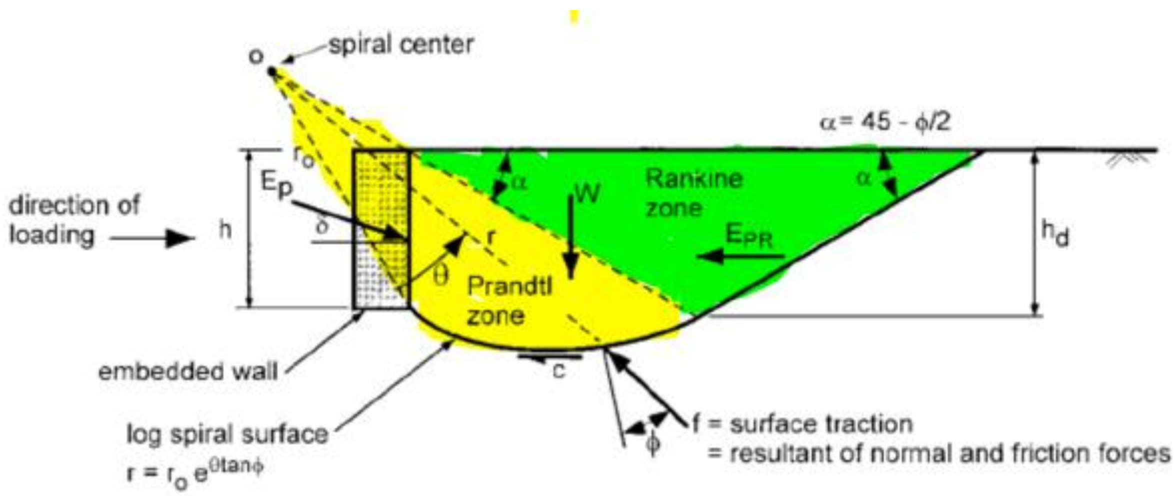


Figure 7.4 failure geometry in log-spiral theory [13].

The log spiral theory considers the wall friction angle δ, the internal friction angle ϕ, and the slope of the backfill β. This theory has two soil shear failure zones. The Rankine zone is a geometry of straight lines and the Prandtl zone is a zone with a curved and spiral geometry.

The Janbu's theory includes a more detailed studies than the Rankine, the Coulomb and the log spiral methods do. To determine the coefficients of the active and passive earth pressure coefficients, it considers the mobilized friction ratio (ρ) and the roughness ratio (r). the chart has two parts. The upper-right part is a solution for the passive earth pressure while the lower-left is for the active earth pressure.

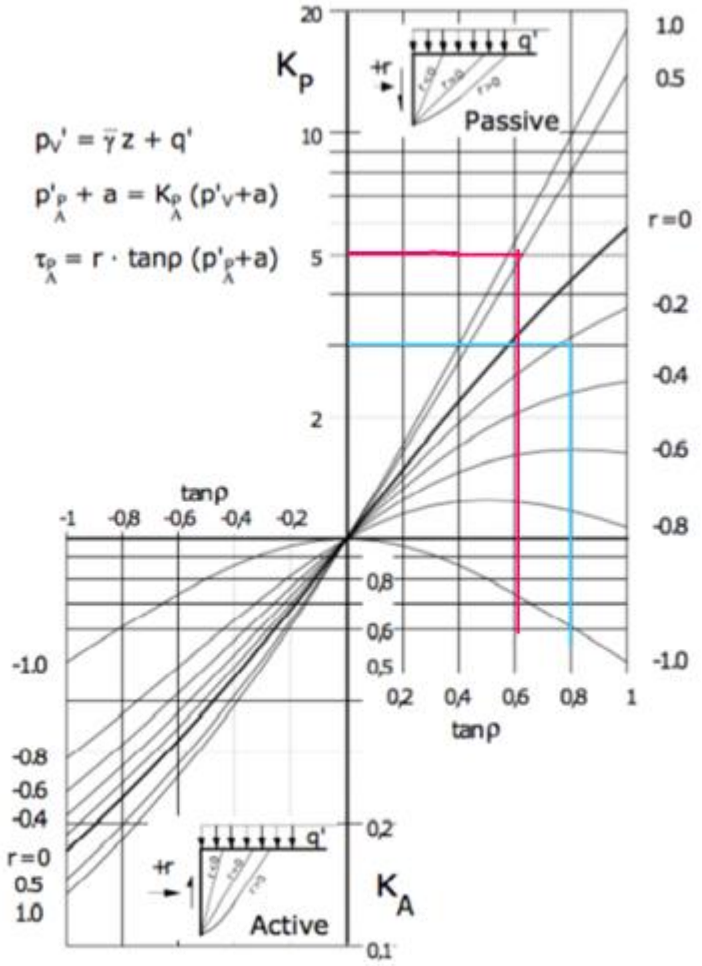


Figure 7.5 Janbu's theory: mobilized friction angle, roughness ratio vs coefficients of earth pressure

For ɣ= 13KN/m^3 and z=3m let us assume q= for simplicity cases, let a=8KN/m^2.
For r=0.5 and tanρ=0.6 the value of Kp can be read from the chart and is equal to 5.

$$p_v = ɣ * z = \frac{13KN}{m3} * 3m = 39KN/m2$$
$$p_P = K_p * (p_v + a) - a = 5 * \left(\frac{39KN}{m2} + \frac{8KN}{m2}\right) - \frac{8KN}{m} = 230KN/m2$$
$$\tau_v = r * \tan\rho \left(p_p + a\right) = 0.5 * 0.6 * (230 + 8) = 71KN/m2$$

7.7 Result from types of sheet pile walls and sheet pilings:

The most used sheet pilings are the cantilevered and the anchored. These sheet piles have different profile sections. The most common sections for simple projects are the U-type, the Z-type and the straight sections. Z-type profiles are most bending moment resistant while straight type sections are least.

7.7.1 result from designing cantilever sheet pile wall penetrating sandy soil:

Here are the results obtained from designing a cantilevered sheet pile penetrating a cohesionless sandy soil. Here it is assumed that the attraction force is zero and thus a=c=0.

no								
1	L1 (m)	1	2	2	3	3	6	8
2	L2 (m)	2	2	3	3	4	9	12
3	ϕ' (degrees)	30	30	30	30	30	30	30
4	γ (KN/m^3)	17	17	17	17	17	17	17
5	γ_{water} (KN/m^3)	10	10	10	10	10	10	10
6	γ_{sat} KN/m^3)	20	20	20	20	20	20	20
7	$\delta_{allowable}$ KN/m^2	170000	170000	170000	170000	17000 0	170000	170000
8	γ'	10	10	10	10	10	10	10
9	Ka	0.333	0.333	0.333	0.333	0.333	0.333	0.333
10	Kp	3	3	3	3	3	3	3
11	δ'1	5.667	11.3	11.33	17	17	34	45.33
12	δ'2	12.33	18	21.33	27	30.33	64	85.33
13	δ'3	88	117	140	168	192	394	521
14	δ'4	211	297	353	438	495	1034	1374
15	δ'5 (net.p)	123	180	213	270	303	640	853
16	L3 (m)	0.46	0.68	0.8	1	1.14	2.4	3.2
17	L4 (m)	3.3	4.4	5.25	6.3	7.2	14.8	19.5
18	L total	6.76	9.1	11.05	13.3	15.34	32.2	42.7
19	Mmax (KNm)	65.5	153	265	458	680	5855	14479
20	S (section modulus) m^3/m	0.000385	0,0009	0.001557	0.002696	0.003997	0.03444	0.085170
21	Section modulus (cm^3/m)	385	900	1557	2696	3997	34440	85170
22	Section type	Hat type W= 900 NS-SP-10H	Hat type W=900 NS-SP-10H	Hat type W=900 NS-SP25H	Hat type W=900 NS-SP-50H	H+Hat type H+NS-SP-10H	Not available Not recomm	Not available

							ended design	Not recomme nded

For safety reasons the embedment length (D=L3 +L4) is increased by 30% to 40% for a sandy soil and 40 to 60% for clay soil [26].

Designing av cantilever sheet pile at different internal friction angles (30-36) degrees.

No								
1	L1 (m)	2	2	2	2	2	2	2
2	L2 (m)	3	3	3	3	3	3	3
3	ϕ' in degrees	30	31	32	33	34	35	36
4	ɣ (KN/m^3)	17	17	17	17	17	17	17
5	$ɣ_{sat}$ (KN/m^3)	20	20	20	20	20	20	20
6	$ɣ'_{water}$ (KN/m^3)	10	10	10	10	10	10	10
7	$\delta_{allowable}$ (KN/m^2)	170000	170000	170000	170000	170000	170000	170000
8	ɣ'	10	10	10	10	10	10	10
9	Ka	0.333	0.32	0.307	0.295	0.283	0.271	0.26
10	Kp	3.0	3.124	3.255	3.392	3.537	3.69	3.85
11	δ'1	11.3	10.9	10.45	10	9.6	9.2	8.8
12	δ'2	21.3	20.5	19.7	18.9	18.1	17.3	16.6
13	δ'3	140	141	142	143	144	145	146
14	δ'4	354	361	370	379	388	398	409
15	δ'5	213	220	228	236	244	253	263
16	L3 (m)	0.8	0.73	0.67	0.61	0.56	0.5	0.46
17	L4 (m)	5.25	5	4.8	4.6	4.4	4.2	4
18	L total	11.1	10.76	10.5	10.2	9.98	9.74	9.52
19	M_{max} KNm	265	247	231	216	202	189	177
20	(S) Section modulus (m^3/m)	0.001557	0.0014534	0.001358	0,0012702	0,001189	0,001114	0,001044
21	Section modulus (cm^3/m)	1557	1453	1358	1270	1189	1114	1044
22	Section type	NS-SP-45H	NS-SP-45H	NS-SP-25H	NS-SP-25H	NS-SP-25H	NS-SP-25H	NS-SP-25H

Here are the table of the section types with their respective value of the moment of inertia and the section of modulus.

Type	Dimension			Per pile				P
	Effective width W mm	Effective height h mm	Thickness t mm	Sectional area cm²	Moment of inertia cm⁴	Section modulus cm³	Unit mass kg/m	Sectiona area cm²/m
NS-SP-10H	900	230	10.8	110.0	9,430	812	86.4	122.2
NS-SP-25H	900	300	13.2	144.4	22,000	1,450	113	160.4
NS-SP-45H	900	368	15.0	187.0	40,500	2,200	147	207.8
NS-SP-50H	900	370	17.0	212.7	46,000	2,490	167	236.3

Here are the different types of Hat type steel sheet pile profiles.

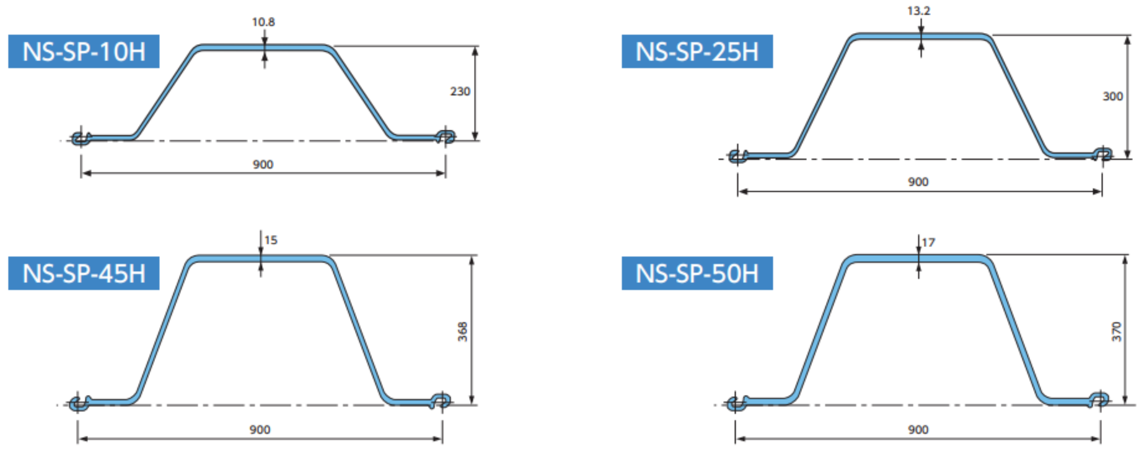


Figure 7.6 hat type steel sheet piles [28]

7.8 result from the material:

Basically, sheet piles are made up of steel, wood, concrete, polymer, and aluminum. However, steel sheet piles are the most used. This is because of their availability, strength to resist high pressure forces while they are being installed. Steel sheet piles are durable if they are protected against corrosion. PVC can substitute steel sheet piles in terms of strength and sustainability and for their advantage over maintenance costs to environment related challenges [3, 28].

7.9 The soil and its properties

The soil or the earth is the one that finally bears or holds every construction. The soil can be strong or weak depending on its cohesive property, internal friction angle and its water content. These properties of soil play an important role in choosing the design and material of sheet piles to be used. The cohesion of clay soil varies depending on its water content as shown below.

Table 7.4 water content vs cohesion force [32]

Soil type (clay)	Cohesion in KPa
dry	20<a<30
medium	0<a<20
Bleeding	a = c =0

The internal friction angle of a soil varies as its water varies. As it is shown in the table below as the water content is between 0 and 10% the internal friction angle is 30 degrees. As it increases towards 40% the friction angle decreases from 30 towards 20 degrees. How ever as it is shown in the table, when the water content exceeds 40% the internal friction angle keeps constant at 20 degrees-

Table 7.5 water content vs friction angle [32]

Water content (w) in % of the dry soil	ϕ' in degrees
0-10%	30^0
10-40%	30^0-20^0
>40%	20^0

Different soils with different effective internal friction angles have various cohesion forces.
The table shows the cohesion properties of different soils having different effective friction angles.

Table 7.6 soil type and the cohesion force [31].

Soil type	Effective friction angle ϕ'		Effective cohesion c' (KPa)
	peak	residual	
Gravel	34	32	-
Gravel, sandy with few fines	35	32	-
Gravel, sandy with silty or clayey fines	35	32	1.0
Gravel and sandy mixture, with fines	28	22	3.0
Sand, uniform, fine grained	32	30	-
Sand, uniform, coarse grained	34	30	-
Sand well grained	33	32	-
Silt, low plasticity	28	25	2.0
Silt, medium to high plasticity	25	22	3.0
Clay, low plasticity	24	20	6.0
Clay, medium plasticity	20	10	8.0
Clay, high plasticity	17	6	10.0
Organic silt or clay	20	15	7..0

The natural unit density, degree of saturation, porosity and the natural water content of a soil are interrelated to each other. From the chart below, once some properties of a soil are determined by laboratory help, the rest can be determined by referring the chart. Knowing the properties of a soil is essential to decide the type of a project to be constructed.
For a soil with a measured unit density (ɣ) 18KN/m3 and a water content (w) 30%, its porosity (n) and its degree of saturation S_r are read from the monogram to be 50% and 85% respectively.

Similarly:
For unit weight density, ɣ=15KN/m3, and water content, w=15% then
Porosity n= 70% and degree of saturation Sr =61% as we can see the two intersection points of three straight lines each on the monogram below.

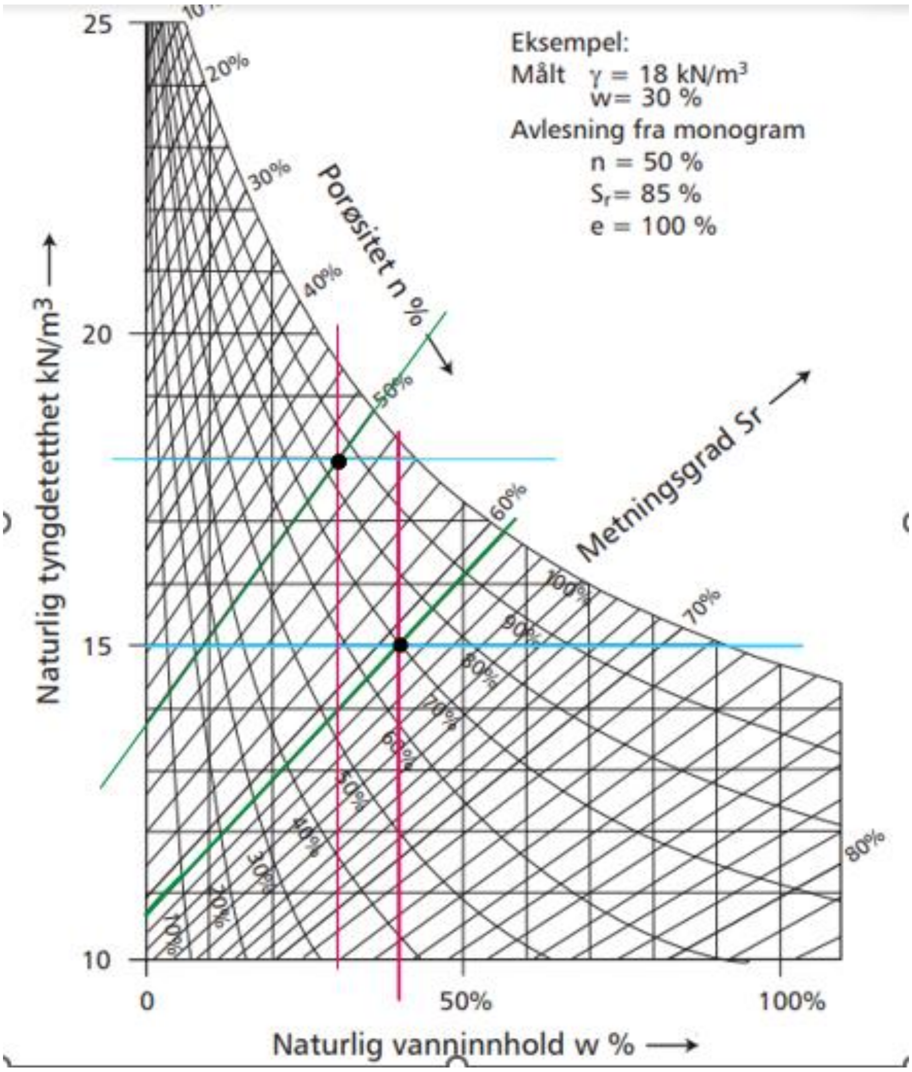


Figure 7.7 relationship of soil properties [36]

7.10 Water and drainage conditions:

When water penetrates sediments and cracks of rocks, it causes to induce a water pore-pressure increment. During cold conditions the water freezes and is volume is increased by 9%. This much volume change creates a tensile force and induce a failure on the earth material. The figure below shows an average number of falling rocks every month due to volume increment of a frozen water in the cracks of rocks that induce a tensile force. The figure verifies the number of falling rocks versus a change in precipitation and temperature in every month of a year.

Due to loading or unloading conditions within a range of time, there occurs a change in stress in the soil. This change in stress further changes the water pore-pressure in the soil depending on the degree of permeability of the soil.

In the case of drained condition, the change in stress in the soil is balanced by the free in and out movement of the water from the pores of the soil. How ever in the case of the undrained condition, an increase in stress due to loading on the soil creates a higher pore-water pressure. In this case the soil is not permeable and thus, the water in the pores cannot move freely in and out of the pores in the soil. Since water is incompressible, the pore-water pressure becomes higher in the soil.

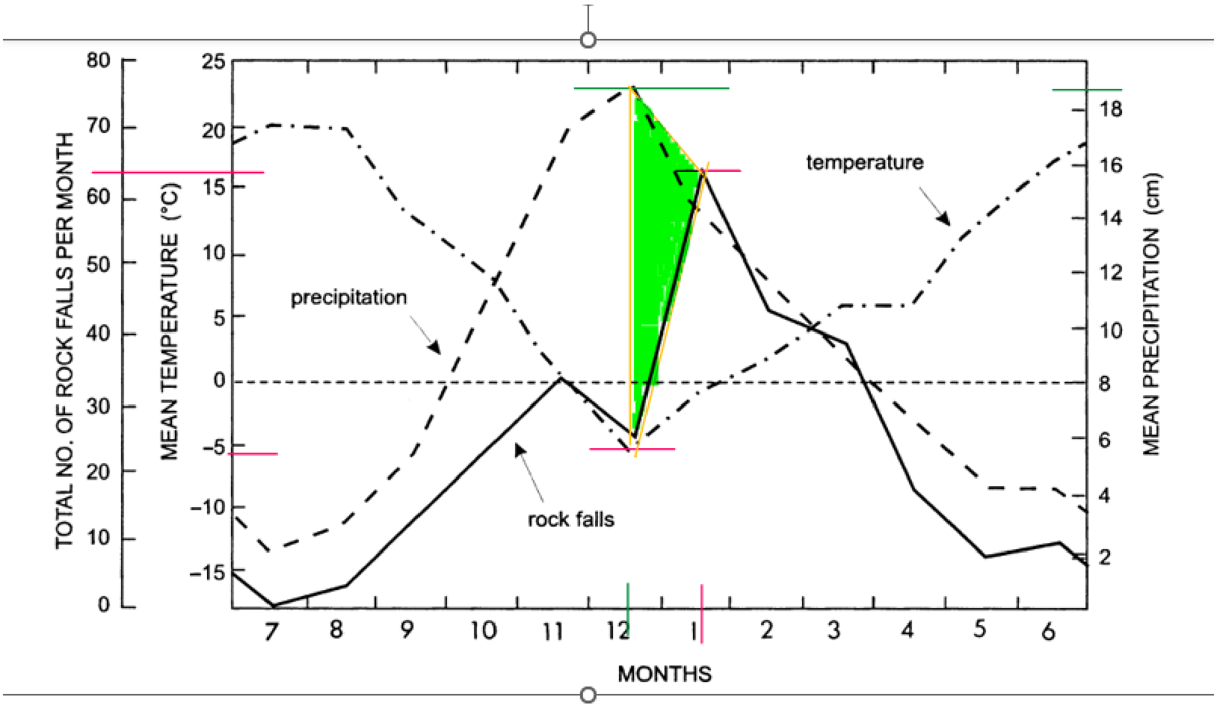


Figure 7.8 rainfall, rockfall, temperature per month [37].

7.10 sheet pile failures:

It is found that both the cantilever and anchored sheet piles experience three types of failures.

7.10.1 flexural failure

In the flexural type, the failure occurs at the point where the maximum moment is induced. The flexural moment is represented as shown in the figure below.

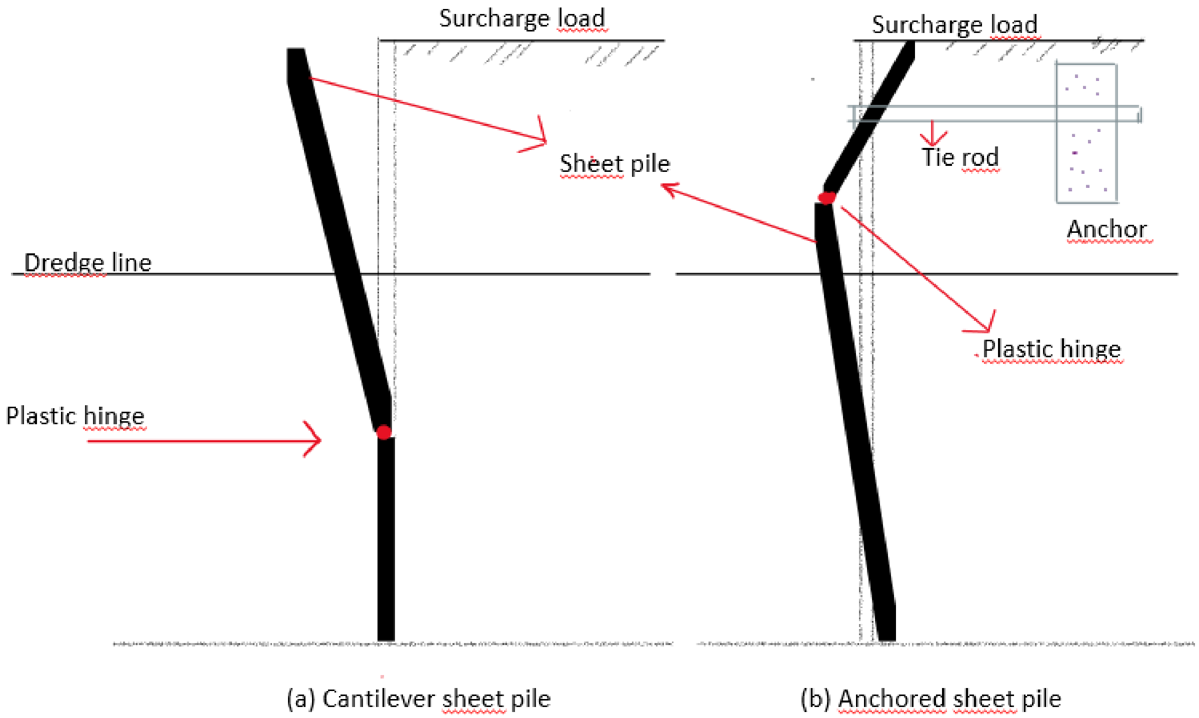


Figure 7.9 flexural failure of cantilever and anchored sheet piles. Illustrated by Yonas Tsighe

Flexural failure occurs when the sheet pile is not stiff enough to resist the moment induced.

7.10.2 Rotational failure

In the case of rotational failure, the sheet pile is stiff however, the penetration depth is shallow, and the passive pressure region or zone is not adequate to resist the moment induced. Consequently, the sheet pile wall rotates. The figure below shows the rotational failure of a sheet pile wall.

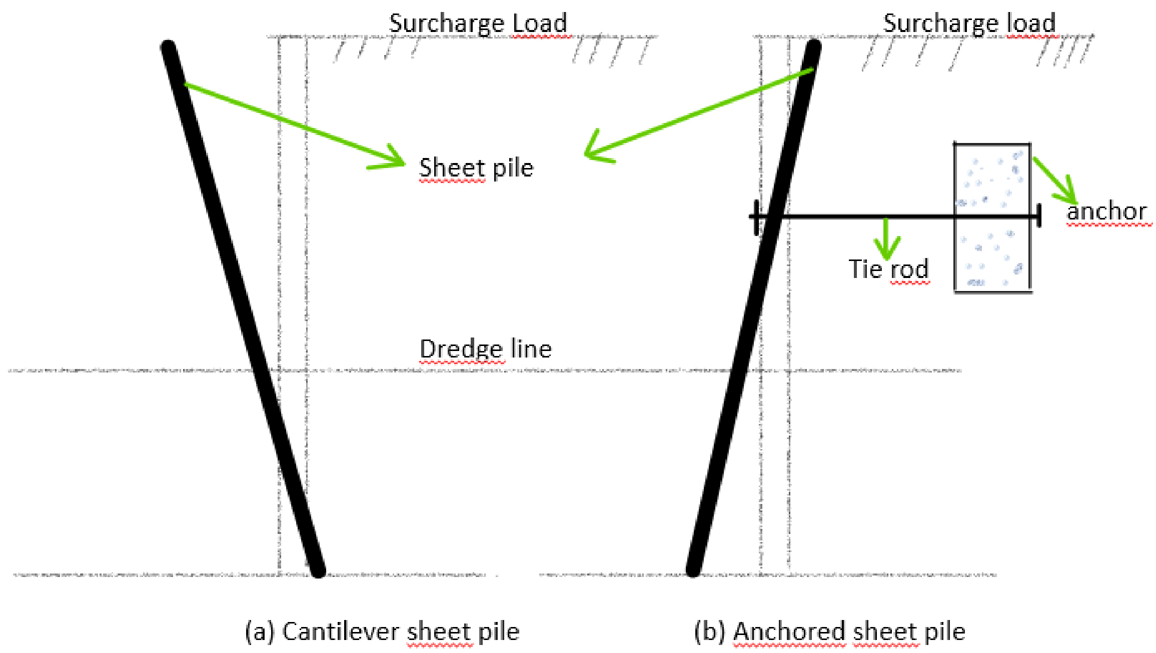


Figure 7.10 Rotational failure of both the cantilever and anchored sheet piles. Illustrated by Yonas Tsighe

7.10.3 Deep-seated failure:

The deep-seated failure occurs due to the failure of the soil mass that holds the steel pile length. The failure arises because of unexpected weak cohesion force of complex soil. This failure occurs due to inadequate study of slope stability and water content or soil property of the site [29].

7.11 Slope stability and stability control:

The friction force between the upper sliding earth and the lower part that is at rest, pore water pressure, and the suction of the soil that rests on a slope area are the main factors that play an essential role for the stability of a slope. When the sum of the driving force and the surcharge load on a slope area is greater or equal to the frictional force between the upper sliding part and the lower part at rest, the stability of the slope becomes weak. Consequently, the slope slides. Pore-water pressure in the soil at a slope area weakens the slope stability. How ever when the water content or the unit density reduces due to evaporation, the soil in the slope area becomes harder and the soil particles contract towards each other [40, 39].

To confirm the stability of a sheet pile wall construction on a slope area, three main stability controls require to be performed. These three controls are compression at the bottom, upheaval at the bottom and hydraulic foundation failure.
The bottom compression control can be described in terms of the bearing capacity factor and the ultimate shear characteristics of soils. The control for bearing capacity factor (NC) is based on the depth to width ratio [42, 41].
For a 6m excavation depth of a sheet pile wall, the total height of the wall is calculated to be 13.3m. if we assume that the width of the wall (B=26m) then

$\frac{Z}{B} = \frac{13.3}{26} = 0.5$
For $\frac{Z}{B} = 0.5 \; then \; NC = 6$
For extended and huge construction projects where the B/L ratio is too small or closer to zero, this relationship corresponds to the line (C) from the chart below.

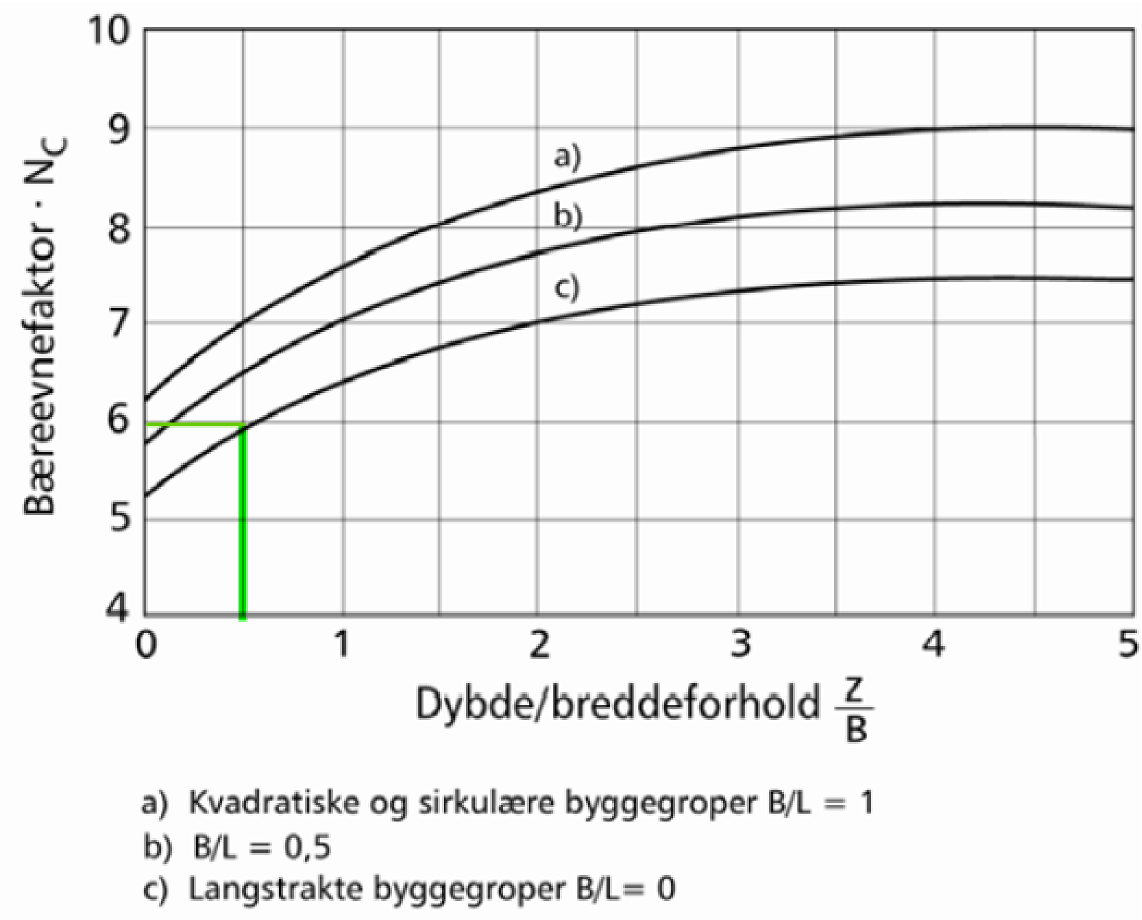


Figure 7.11 bearing capacity factor vs depth to width ratio [42]

For hydraulic foundation failure stability control, it is required to calculate the material factor ɣm. NC= 6 from the above chart, undrained shear strength (Su) is assumed to be 70KN/m2, ɣ is assumed to be 20KN/m3. Z is calculated to be 13.3 meter for the 5m excavation depth. For simplicity cases let us assume that qd be 20KN/m2 and pd be 40KN/m2.

For the 5m excavation depth sheet pile wall it was assigned that Hw =3m, B=6m (assumed), D was calculated to be 6.05m

Thus,

$$\frac{D}{H_w} = \frac{6.05}{3} = 2$$

$$\frac{B}{H_w} = \frac{6}{3} = 2$$

$$ɣ_m = \frac{N_c * S_u}{ɣ * Z + q_d - P_d} = \frac{6*70KN/m2}{\frac{20KN}{m3} * 13.3m + \frac{20KN}{m2} - 40KN/m2} = 1.7 > 1.5 \text{ that is Ok.}$$

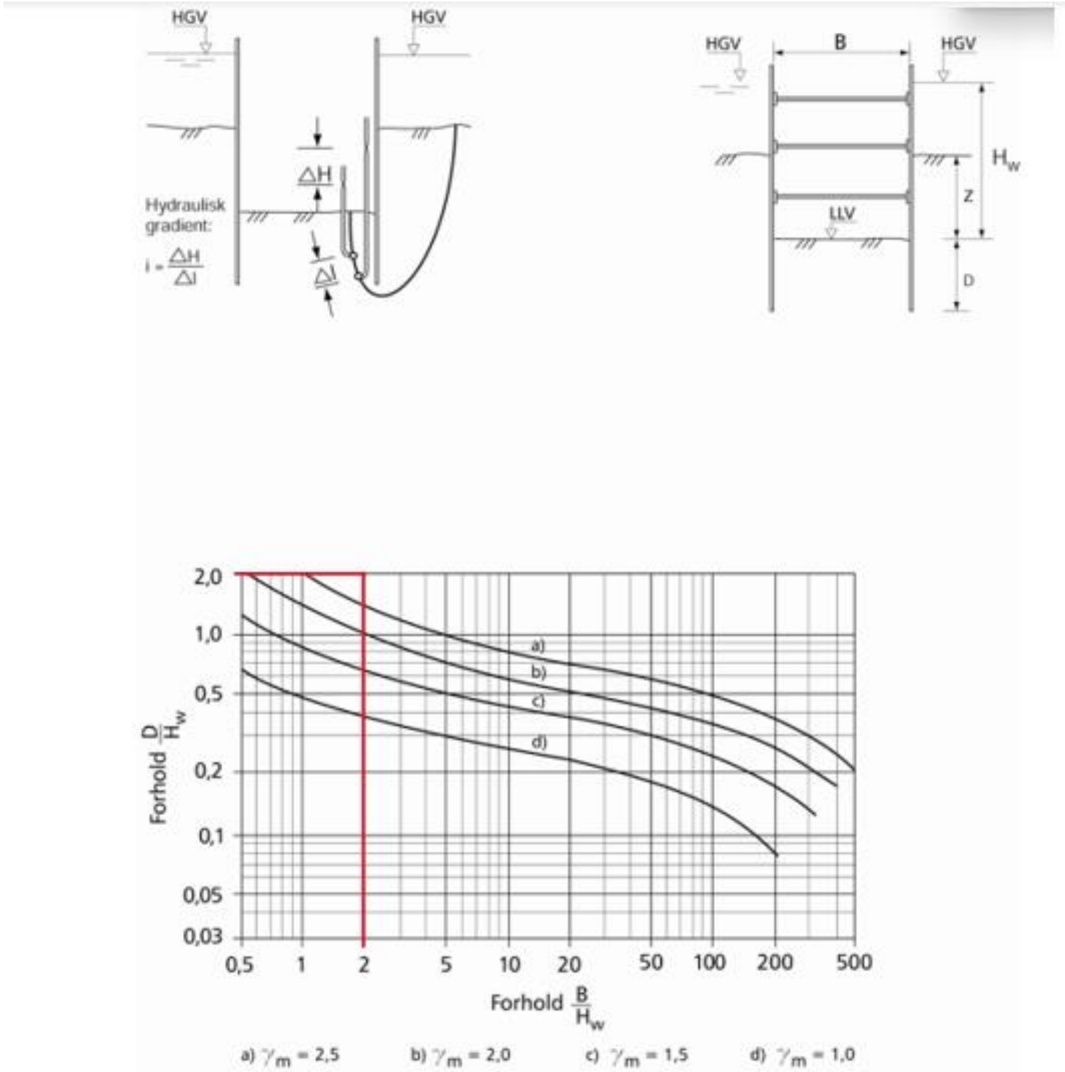


Figure 7.12 material factor vs depth to height ratio [42]

7.12 Result from the stress analysis:

This part includes three different stress type analysis, the total stress, the effective stress and the a-ϕ stress analysis:

Let us first consider the total stress analysis:

Let us assume a clay soil and a flat terrain and thus the inclination angle will be 0 degree. Then the passive coefficient of friction for zero roughness (r=0) will be (Kp=2) as shown from the graph below. Then the mobilized shear strength τ_c is determined by:

$$\tau_c = f * S_u = \frac{S_u}{\gamma_m} = \frac{\frac{70KN}{m2}}{1.7} = \frac{41KN}{m2}$$

$$P_p = q + \gamma * z + K_\tau * \tau_c = \frac{20KN}{m2} + \frac{20KN}{m3} * 3m + 2 * \frac{41KN}{m2} = 162KN/m2$$

$\tau_A = r * \tau_c = 0 * \frac{41KN}{m2} = 0$

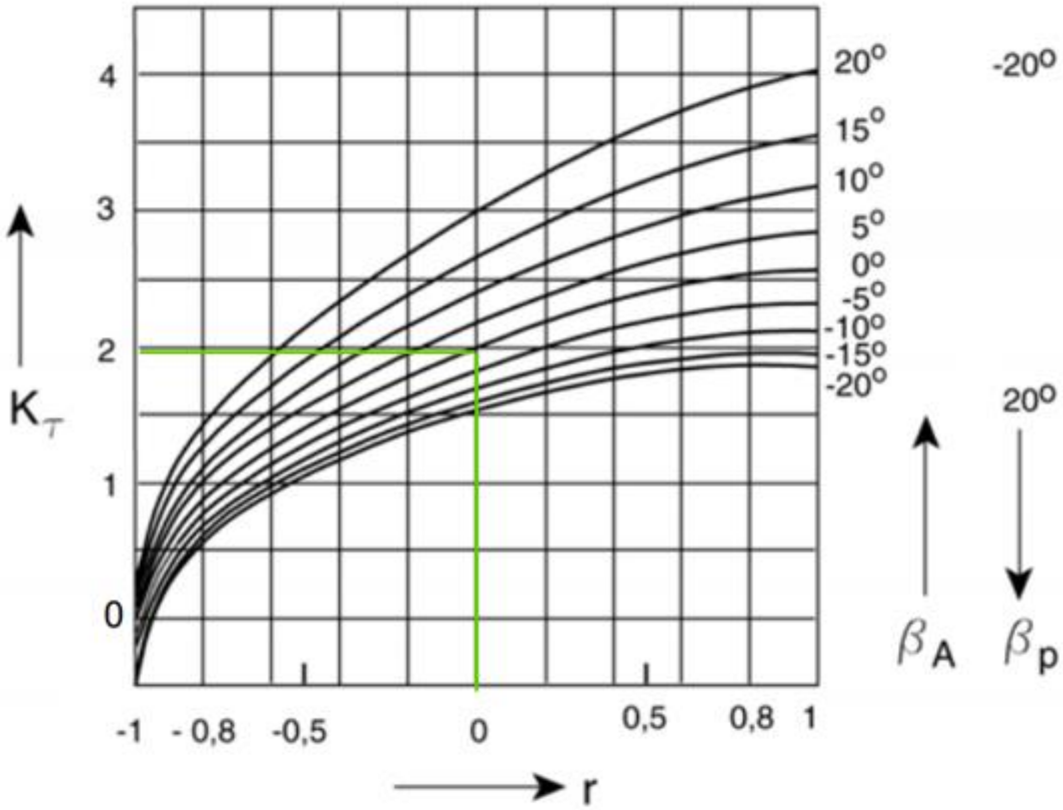


Figure 7.13 roughness vs active and passive pressure coefficients in total stress [43]

Le t us again consider an inclined terrain:

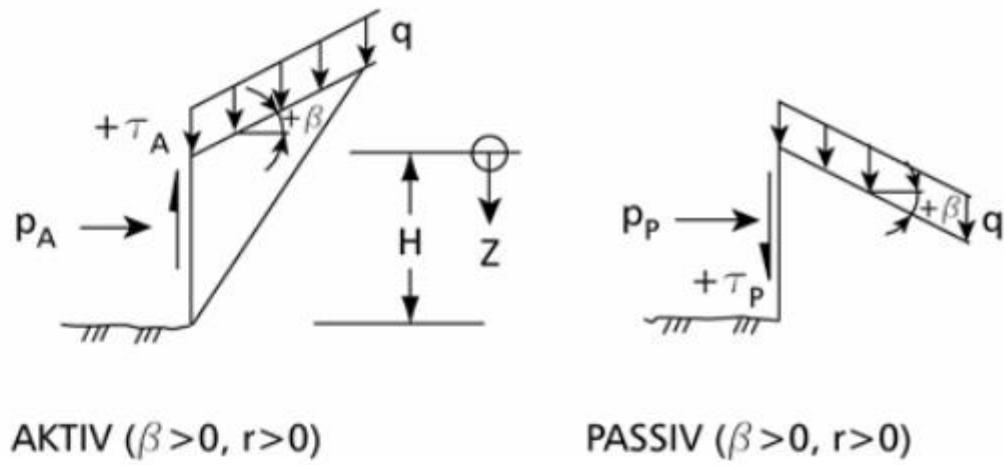
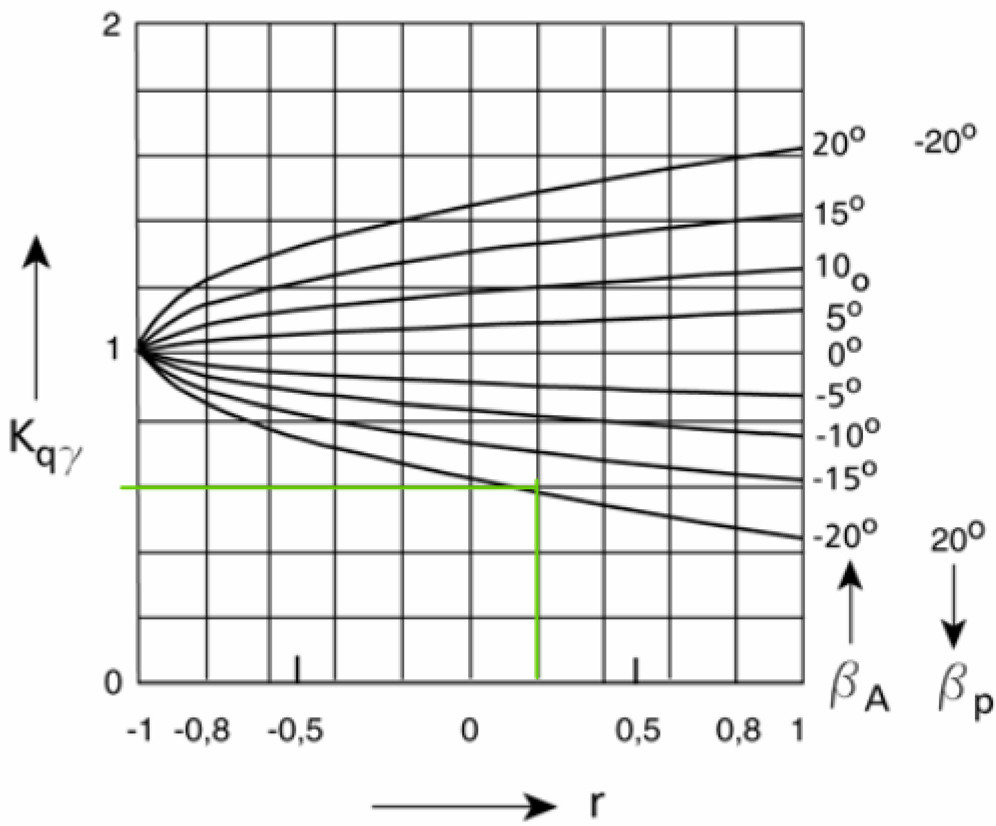


Figure 7.14 positive roughness and positive passive and active inclination

For a fine sized clay soil with 20 degrees of inclination (β=20) having a roughness ratio (r=0.2) then the coefficient of pressure (Kqr) will be 0.6 as it can be shown from the chart below.



$$P_p = K_{qr} * (q + ɣ * Z) + K_\tau * \tau_c = 0.6 * \left(\frac{20KN}{m2} + \frac{20KN}{m3} * 3m\right) + 2 * \frac{41KN}{m2} = 130KN/m2$$

$$P_P = r * \tau_c = 0.2 * 41KN/m2 = 8KN/m2$$

Regarding the effective stress analysis let us consider a soil in a horizontal terrain having a material property ɣm =1.4. let the soils be clay and sand with sand friction angle (ϕ=35) and clay's friction angle (ϕ=20). And let the roughness ratio be (r=0).

Clay: $\tan\rho = \frac{\tan\phi}{ɣ_m} = \frac{\tan 34}{1.4} = 0.5$

Sand: $\tan\rho = \frac{\tan\phi}{ɣ_m} = \frac{\tan 20}{1.4}$=0.26

For r=0 then

Clay: Ka= 0.397=0.4 and Kp= 2.52=2.5

Sand: Ka=0.6 and Kp= 1.67

From the graph below the green lines represent the values of clay soil and the blue lines represent the values for sand.

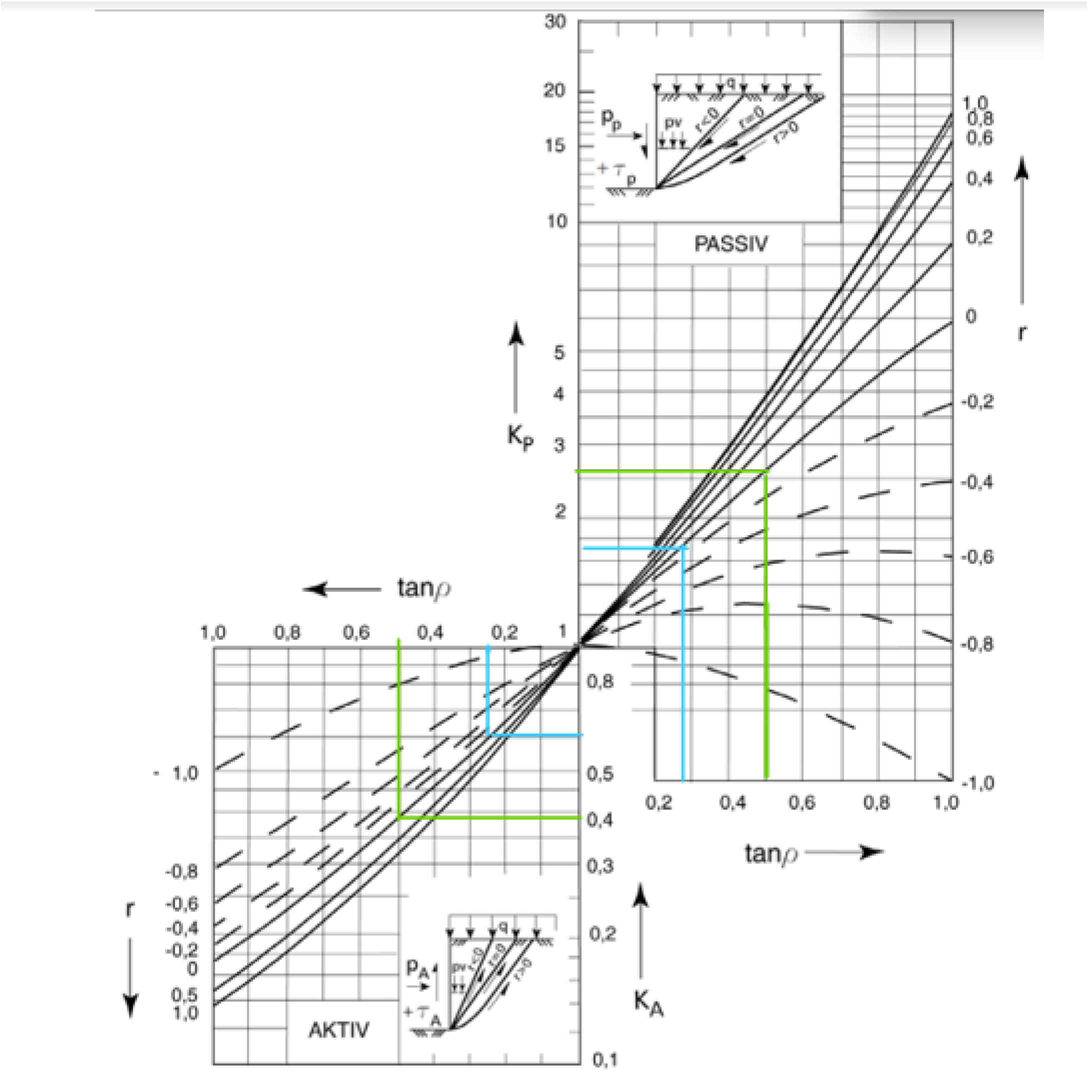


Figure 7.15 roughness, friction wall angle and pressure coefficients in effective stress [44]

For active earth pressure components in the clay soil case, when attraction or cohesion is assumed to be 8KN/m2 and roughness is assumed to be zero, then:

$$P_{v\prime} = q + \gamma * Z = \frac{20KN}{m2} + \frac{20KN}{m3} * 3m = 80KN/m2$$

$$P_{A\prime} = K_A * (P_{v}{}' + a) - a = 0.4 * \left(\frac{80KN}{m2} + \frac{8KN}{m2}\right) - 8KN/m2 = 27KN/m2$$

$$\tau_A = r * \tan\rho\,(P_{A\prime} + a) = 0 * 0.5 * (27 + 8) = 0$$

For the passive earth pressure components of the clay soil:

$P_{v\prime} = \frac{80KN}{m2}$ from above.

$$P_{p\prime} = K_p * (P_{v}{}' + a) - a = 2.5 * (80 + 8) - 8 = 212KN/m2$$

$$\tau_A = r * \tan\rho\left(P_{p}{}' + a\right) = 0 * 0.5(212 + 8) = 0$$

Regarding the sand soil, for the active earth pressure components: where a=8 and r=0:

$$p_{v\prime} = q + \gamma * Z = \frac{20KN}{m2} + \frac{20KN}{m3} * 3m = 80KN/m2$$

$$P_{A\prime} = K_A * (p_{v}\prime + a) - a = 0.6 * \left(\frac{80KN}{m2} + \frac{8KN}{m2}\right) - 8KN/m2 = 45KN/m2$$

$$\tau_p = r * \tan\rho(P_{A\prime} + a) = 0$$

Fot the passive earth pressure components:
$p_{v\prime} = 80KN/m2$

$$P_{p\prime} = K_p * (p_{v}\prime + a) - a = 1.67 * \left(\frac{80KN}{m2} + \frac{8KN}{m2}\right) - \frac{8KN}{m2} = 139KN/m2$$

$$\tau_p = r * \tan\rho * \left(P_{p}\prime + a\right) = 0$$

The a-ϕ stress analysis:
This type of analysis refers to the analysis of a shear strength for a naturally inclined terrain. It describes the shear strength of a clay soil or undrained condition as a function of the cohesion force c and the internal friction angle of the soil. For ϕ =20 and α=45 degrees, a=8KN/m2 and p_0=20KN/m2, and ɣm=1.4 all assumed:
The shear strength τ_k is then:

$$\tau_k = \frac{(a+P_0)*\tan\phi}{1+\tan\alpha*\frac{\tan\phi}{\gamma m}} = \frac{\left(\frac{8KN}{m2}+20KN/m2\right)*\tan 20}{1+\tan 45*\frac{\tan 20}{1.4}} = \frac{8KN}{m2}$$

7.13 Result from the corrosion rate of steel sheet piles:

This sub-chapter verifies the effect of different medias on the durability of sheet piles. Corrosion is one of the most weaknesses of steel sheet piles in reducing its durability. The first table shows the rate of corrosion of steel sheet piles in the presence or absence of ground water table. According to the table, steel sheet piles have the slowest corrosion rate when they are installed in undisturbed natural soils and the fastest in non-compacted and aggressive soils like ashes and slag.

Table 7.7 rate of corrosion of steel sheet piles in the absence or presence of ground water table [46].

Required design working life	5 years	25 years	50 years	75 years	100 years
Undisturbed natural soils (sand, clay silt and schist)	0.00	0.3	0.6	0.9	1.2
Polluted natural soils and industrial sites	0.15	0.75	1.5	2.25	3.00
Aggressive natural soils (swamp, marsh, and peat	0.2	1.00	1.75	2.25	3.25

Non-compacted and non- aggressive fills (clay, silt, sand, and schist)	0.18	0.7	1.2	1.7	2.2
Non-compacted and aggressive fills (ashes and slag)	0.5	2.0	3.25	4.5	5.75

The next table demonstrates the corrosion rate of steel sheet piles installed in fresh and salty water. It shows how the presence or absence of chemical content materials in the fresh water affect the rate of corrosion. Is also shows how the rate of corrosion is affected in different parts of or zones of a sheet pile. According to the table, the rate of corrosion is higher in polluted freshwater than in clean and natural freshwater.

Table 7.8 steel sheet pile corrosion rate in fresh water and seawater [46]

Required design working life	5 years	25 years	50 years	75 years	100 years
Fresh water (river, ship canal) in the high attack zone (water line)	0.15	0.55	0.9	1.15	1.4
Very polluted fresh water (sewage, and industrial effluent) in the zone of high attack or the water line	0.30	1.30	2.30	3.30	4.30
Sea water in temperate climate in the zone of high attack (low water and splash zones)	0.55	1.90	3.75	5.60	7.50
Sea water in temperate climate in the zone of permanent immersion or in the intertidal zone.	0.25	0.90	1.75	2.60	3.50

8. Discussion

The discussion part of this master includes a drafting of the results found. In addition, it includes the candidates understanding for each chapter described in the whole project. Finally, the weaknesses that were occurred in course of the study and the limitations experienced will be explained.

8.1 Discussion on Social perspective:

Steel sheet piles are the most used sheet piles due to their availability and their resistance to driving pressures. Nevertheless, steel products are heavily affected by water and weather conditions. When steel sheet piles are permanently installed in seawater or wet conditions, they corrode and require a frequent maintenance. Although steel products are strong, they result in economic loss for the maintenance cases. In addition, their corrosion products yield a negative impact on the lives in water. PVC can be used as a substitute concerning sustainability and maintenance requirements. PVC are light in weight and a smaller amount can cover for the same project compared to steel.
However, PVC are not available as much as the steel products in the markets.

8.2 Discussion on calculations in designing a cantilever sheet pile:

In designing a cantilever sheet pile two case were considered. Both cases were computed by the help of Microsoft excel program. In the first case a constant internal friction angle was set constant equal to 30 degrees. The soil type was chosen sand with a ground water level. The calculations were performed by varying excavation lengths above and below the ground water table (L1 and L2) where L1 is the excavation length above the ground water table and L2 is the excavation length below the ground water table. The main purpose of the calculation was to find the penetration depth, the maximum moment induced and the section of modulus of the sheet pile wall can tolerate. However, before reaching these final goals of the calculations several expressions were first performed. The active and the passive lateral pressure constants (Ka and Kp), the active and passive earth pressures in every soil layer along the whole sheet pile length, the net earth pressure at the bottom of the sheet pile, and finally the effective density (γ') were calculated. According to the calculations done it was observed that as the excavation depth increases, so does the penetration depth. As the penetration depth increases the maximum moment induced also gets enlarged. Consequently, the thickness of the sheet pile to be selected gets larger to resist the increasing maximum moment. The most important observation is that as the penetration depth goes deeper and deeper, the passive pressure zone also gets larger. The larger the passive pressure zone, the more stable the sheet pile wall becomes.

Another noticeable observation is that when the excavation depth exceeds 6 meters, the maximum moment induced becomes very huge. According to the first table, for the 6m, 7m, 15m and 20 m excavation depths, the maximum moment induced obtained are 458, 680, 5855 and 14479KNm respectively. This much amount of maximum moment requires 2696, 3997, 34440, and 85170cm^3/m section of modulus to resist the moment. Specially the last two are so huge that there is no available section or sheet pile profile in the standards that is suitable for these sizes.
From this table it is observed that the if the excavation depth exceeds 6m it is not recommended to choose and design a cantilever sheet pile wall.
In the case of the second table, the excavation length was kept constant (5m) 2m above the ground water level and 3 me below the ground water table. The internal friction angle (ϕ) was a variable from

30 to 36 degrees. 30 to 36 degrees of friction angle belongs to sandy soil. As the angle of the internal friction increases from 30 to 36 degrees, the required penetration depth decreases. This verifies that dense soils do not require a deep penetration as the soft soils do. However, the difference is not so much compared to the case seen on varying the excavation depths.
Thus, it does not show much difference on the maximum moment induced. Comparing tables 1 and 2 it is observed that a change in excavation length matters more than changes in friction angle of the soil.

8.3 Water and drainage conditions:

Referring to the figure below (figure 7.7 on results) [37] that describes the effect of precipitation, temperature on the number of rockfalls per month for a one-year period, it is clearly shows that the number of rockfalls is directly proportional to the amount of rainfall and inversely proportional to temperature. This means that as precipitation increases the number of rockfalls also increases and however as the temperatures increase the number of falling rocks decreases. The effect of precipitation relates to the changes in water-pore pressure within the cracks of the rocks or with in the soil particles holding the rocks. The effect of temperature is related with the drainage of the water from the pores of the soils or cracks of the rocks, and this further contributes the soil or rock to attain or loss its shear strength depending on the increase or decrease of the temperature.

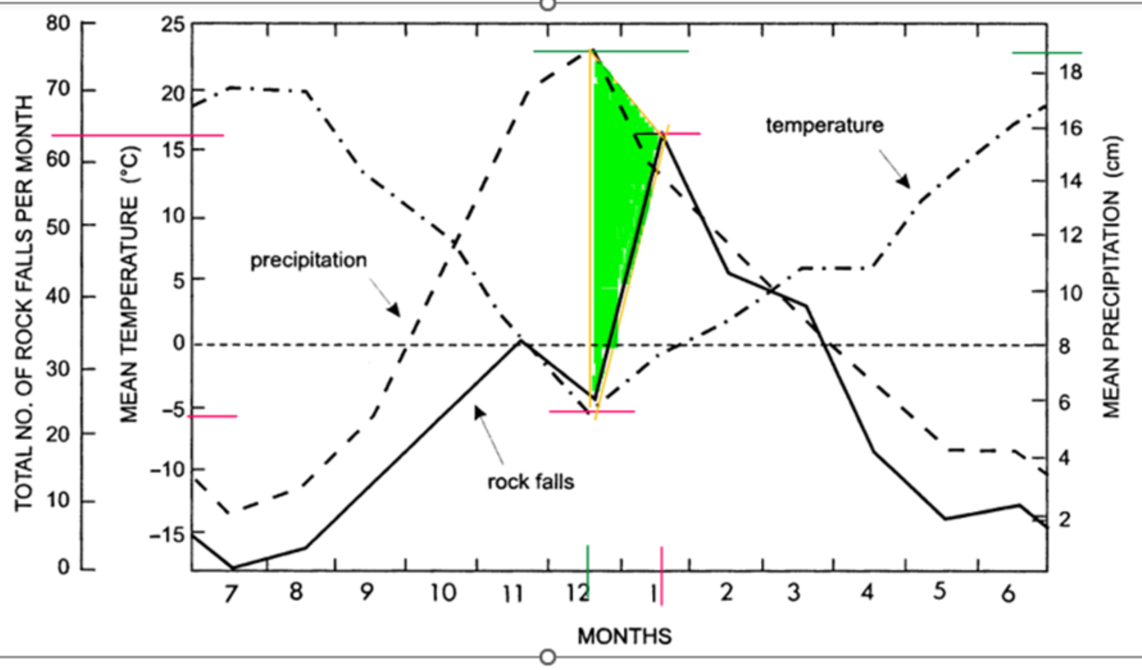


Figure 8.1 rainfall, rockfall, temperature per month in a year [37].

As it is shown on the figure, in July there is maximum temperature and minimum or zero rockfalls. This is mainly because the water is drained out from the cracks of the rocks and thus there is no water pore pressure in the rocks. At this time the rocks attain their maximum shear strength and become more stable on their initial position. In December it reaches a maximum precipitation, and the number of rockfalls shows a landslide increase.

8.4 the earth pressures:

The at rest, active and the passive earth pressures are the three types of pressures that exist on sheet piled walls. The at rest earth pressure is the transition pressure to the active earth and the passive earth pressures depending on the direction of the pressure pushing the sheet pile wall. It has inconsiderable effect on the stability or instability of the sheet pile wall. The at rest pressure changes to active earth pressure or passive earth pressure depending on how far the top of the wall has moved compared to the total height of the wall. Let us consider a dense sand, when the top of the wall moves 0.1% of its height it changes to minimum active pressure. Similarly, when the at rest pressure acts towards the back fill and the head of wall moves 1% of its height, it changes to maximum passive pressure.

The active earth pressure is the lateral pressure from the soil that pushes the sheet pile wall away from the back fill soil. It plays the most dominant role in destabilizing the sheet pile wall. The active earth pressure from the backfill soil pushes the wall towards the excavated side. The active earth pressure is located under the pivot point of rotational moment on the excavation side and over the axis of rotation on the backfill side. The active earth pressure is dominant over the dredge line.

The passive earth pressure is the stabilizing pressure force that pushes the sheet pile wall towards the backfill. It reacts against the active pressure. The passive earth pressure zone is located over the pivot or axis of rotation on the excavation side and below the pivot point on the backfill side. The passive earth pressure zone can be increased by increasing the penetration depth. The reason why the actual depth is increased by 30 to 40 % of the theoretical depth is to enlarge the passive earth pressure zone. The larger the passive zone the more stable the sheet piled construction will be.

8.5 the lateral earth pressure theories:

In this master thesis 4 different lateral earth pressure theories are included. Each theory has its own assumption.

The Rankine theory assumes a homogeneous and cohesionless soil (c=0). Similarly, it assumes a smooth and frictionless wall (r=0). However, there will always be a friction between the wall and the soil. The earth pressure diagrams are a set of simple straight lines whose slopes are the coefficients of the earth pressure. Therefore, the Rankine theory is not expected to provide the exact solution. As the solution is far from the reality, this theory is not a reliable method for a stable sheet piled construction. Therefore, this method is not recommended compared to the other theories. The probability of unexpected construction failure is maximum compared to the rest three theories.

The Coulomb theory is an extension of the Rankine theory. it considers the backfill soil as an isotropic and firm body. However, the behavior of the backfill soil is not actually isotropic and firm body. The positive side of this theory is that it considers the friction between the wall and the backfill soil. Therefore, this method is expected to provide a better solution compared to the Rankine theory. This method provides a chart or diagram as a function of the wall friction angle in the horizontal axis, the coefficient of the passive earth pressure (Kp) in the vertical axis and angles of internal friction as curved and inclined lines. This chart helps to determine the coefficient of earth pressure values for soils with a given internal friction angle and wall friction angle. As this method describes the interaction between the soil and the wall more than the Rankine does, it provides a more correct solution and thus a better stability is expected.

The logarithmic spiral theory describes the soil failure as a combination of two regions. The first region is that is closer to the wall is a zone of curved diagrams. This curved diagram is due to the friction between the backfill soil and the wall. The upper region is a set of straight lines because the soil failure in this zone is linear. This method is an improved version of the Rankine and the Coulomb theory because it includes both the curved and linear failures of the soil. Therefore, it is expected that this method provides a better solution and thus a better stability of a sheet piled construction.

The Janbu's theory considers the mobilized friction angle and the roughness ratio. This method is the most acceptable theory that is practically used in Norway. It provides a chart as function of mobilized friction angle, roughness ratio and the active and passive earth pressure coefficients. As it considers a detailed studies on the interactions between the soil and the wall, it is the most acceptable method to yield a better solution and thus a more stable sheet pile wall. The probability of unexpected failure is minimum compared to all to the above three theories.

8.6 Sheet pile failure:

Sheet piled wall construction experience three type of failures, namely the flexural failure, rotational failure, and the deep-seated failure. The flexural failure occurs at the point where the maximum moment occurs. This is because section of modulus of the selected profile is not sufficient to resist the induced moment. In such cases the type or property of the soil or water content has not any role for the failure to be actual. If the failure is occurred at early stage of its service time, the only possible reason for this failure is that a wrong sheet pile is used for the project. Another possible can be that the sheet pile wall was installed in an aggressive soil or salty water or industrial polluted sewages that can constantly react with the sheet pile and weaken its strength.
However, if the failure is occurred after a long service time the reason behind this can be that the sheet pile was corroded and did not get maintenance services or there was not corrosion allowance in the design stage. Another possible reason for such failure is that its lifespan is over and needs to be replaced.

The second type of failure is the rotational failure. This type of failure occurs due to inadequate penetration depth of a stiff sheet pile wall. When the sheet pile wall is not penetrated deep enough, the sheet pile wall will have insufficient stabilizing passive earth pressure. Consequently, the sheet piled wall rotates and fails. This type of failure occurs at the early stage of the life service. Therefore, the property of soil, the size of the profile selected, and environment conditions have not any role for this type of failure.
The third type of failure of sheet pile walls is the deep-seated failure. This type of failure is associated with the weakness of the base soil property. Either the cohesion of the soil is weak that the sheet pile wall penetrates or settles down due to its huge weight compared to the strength of the soil or the base soil itself is subsided together with the sheet pile wall. In either way the sheet pile wall deep-seated below its original position it had in the early stage of installation.

8.7 slope stability and stability control:

Friction plays a vital role in strongly holding the layers of a soil in a certain slope area. Because of loading or water content difference between the layers of a soil in the slope region, the layers tend to slide to each other. As the upper part tends to slide down a frictional force opposes the tendency of

sliding. When the driving load force of the upper part becomes greater than the force of friction between the layers, then the upper part slides down while the lower layer remains at rest in its initial position. Thus, friction between soil layers or soil particles contributes a great role for a sheet pile wall installed on a slope region.
When water is infiltrated into the soil of an inclined area, then the pores of the soil particles are filled with water. As the infiltration depth increases, the ground water level raises up and the saturated layers of the soil approaches towards the upper surface of the slope. Water pore-pressures weaken the resistance of the saturated soil against shearing. Thus, the layers of the soil on the slope will have different shearing strengths depending on the degree of saturation of the layers of the soil. This makes the slope unstable and can slide down. Cultivating vegetations and trees on the slope areas can contribute to strengthening the shear resistance of the soil by sucking water from the soil. As the vegetations and trees suck water from the soil through their roots, the ground water level becomes lower. Alternatively, the saturated soil may loss its water due to high temperature by evaporation. This further reduces the water pore-pressure in the soil. Consequently, the soil layers loss water and become denser, and harder. This leads the shear strength of the soil to be higher and thus the soil becomes more stable.
To avoid a sudden failure of sheet piled walls on a slope area, it is possible to perform the slope stability control of the area at the basement of the sheet pile walls. One possible way of controlling the slope stability is checking the bottom compression pressure by total stress analysis for the ultimate limit state. This is performed by determining the bearing capacity factor of the sheet pile wall by the help of chart and some calculations. However, this method is more relevant for sheet pile wall that acts as a vertical load bearer. In this study the sheet pile walls are assumed to be lateral earth pressure supporters, however, the weight of the sheet pile walls can be considered as the vertical load. Therefore, the bottom compression pressure control is required to be checked by taking the total weight of sheet pile walls as the ultimate limit state.

To perform a hydraulic foundation failure control stability the material factor plays a key role to decide if the safety factor against the hydraulic foundation failure is sufficient or not. The minimum requirement for a material factor depends on whether the excavation is temporary or permanent and the type of soil on the basement of the sheet pile wall. The minimum requirement of a safety material factor for temporary excavation is 1.5 while for a permanent excavation it is greater than 2.

In the case of upheaval at the bottom failure, it occurs because of excessive pore pressure that forces the wall to be lifted. The failure in such cases can be controlled by constantly checking the size of the pore pressure. Alternatively, it can be secured by installing a fixed pump wells that can quit the excessiveness of the pore pressures.

8.8 Discussion on Stress analysis:

In this master thesis three various stress analysis are considered. The total stress and the effective stress analysis have their respective graphs as a function of roughness ratio and inclination angle. In either way the main goal is to determine the earth pressure coefficients and then to calculate the passive and active pressures and the shear strength of the soil on the slope where sheet piles are installed. Generally, flattening the inclination of slopes increases the stability of that region against all types of failures. If the passive pressure zone is larger than the active pressure zone, then the slope can be accounted as a stable slope.

The a-ϕ stress analysis is a description on the stability of a slope as a function of the internal friction angle ϕ and the attraction pressure force of the soil. This analysis is used to calculate the shear strength of a soil on a slope. Depending on this calculated value one can decide the stability of the slope.

8.9 Discussion on corrosion rates of steel sheet pile walls:

The corrosion rate of steel sheet pile walls varies depending on the media they are installed in. Corrosion is the weakest side of steel product especially when it is exposed to moist air. The strength of steel sheet piles reduces when the steel corrodes deeply. As the strength weakens it is natural that its lifetime will be shortened. Aggressive medias like salty water and polluted industrial zones cause a higher corrosion rate of steel sheet pile walls than natural and undisturbed soil. The higher the corrosion rate of steel products the shorter will be its durability. Sea water in temperate climate in the zone of high attack (low water and splash zones) is the most aggressive or corrosive media for steel sheet piles.

9. Conclusion

The conclusion sector of this master thesis consists of the main task done or will shortly answer the head research question and the sub questions which are asked in chapter 4.
The thesis question says as follows:
How can one keep the stability and durability of a sheet piled wall construction on slope area?

To extend this master thesis question some sub-questions are included. The sub-questions are listed as follows:

1. What is the concept of pile sheeting?
2. How can one ensure the stability of sheet piled constructions?
3. How can one select the size and material type of a sheet pile for a certain construction?
4. How can one keep the durability of sheet piled constructions?

Sheet piling is a process of building a continuous wall by interlocking pile segments side by side to each other to form different watertight shapes. They are embedded in the soils to provide lateral supports by resisting the horizontal pressures released from sea waters, or any other earth masses. Sheet pile walls are flexible retaining systems therefore they can tolerate larger horizontal deformations compared to other types of walls such as stone and concrete [1].

To ensure the stability of a sheet pile walls the actual penetration depth is required to be 1.3 to 1.4 for sandy soil and 1.4 to 1.6 times the theoretical penetration depth. A good penetration depth increases the stability and durability of a sheet pile wall by avoiding the rotational failure provided that all other criteria are fulfilled [29].
D_{actual} =1.3 to 1.4*$D_{theoretical}$ for a sandy soil and D_{actual}=1.4 to 1.6*$D_{theoretical}$ for a clay soil [26].

Corrosion allowance increases the durability of steel sheet pile walls. However, if PVC are used as a substitute, they additionally prolong the durability sheet pile walls and avoid environmental impacts related to corrosion [3].

Good stiffness of sheet piles ensures the stability and durability by avoiding the flexural failure provided that all other criterions are fulfilled [29].

Strong cohesion force of the base soil ensures the stability of sheet pile walls by avoiding the deep-seated failure [29].

Reducing the degree saturation of a saturated soil on slope areas either by cultivating vegetations increases the shear strength of the slope soil and thus, ensures its stability [39, 40].

As steel products have higher corrosion rate when installed in seawater, PVC must be used as a substitute for steel to avoid corrosion problems and thus to increase the durability of sheet piled walls [3, 46].

10. Recommendations

My recommendation for further work is that:

- This project is very wide, and it needs much time, therefore it would be more effective it was a group work than an individual work (although it is the candidate's choice).
- It would be helpful if anchored sheet pile wall was designed the in addition to cantilever sheet pile.
- It would be advantageous and specific if the project was from an actual construction site to get an extra recommendation from external supervisor.
- It would be a good idea if designing of a steel quality was included in the project.

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Supervision documentation:

BYG508 master thesis 2023

Candidate Yonas Tsighe Ghebrehiwet

Supervisor: Songxiong Ding (UIA)

Supervision 1:

Date 31.01 2023

Case1: to request if he can be my supervisor for the master thesis.

Case 2: to ask him which day of a week we can meet.

He replied that he was willing to be my supervisor and said that we can meet every Monday.

Supervision 2:

Date 20. 02. 2023:

Case 1: to request a recommendation about the structure of the research question: The thesis question is << **How does a pile sheeting stabilize buildings and affect the bearing capacity of a foundation on slope areas?>>**

Case 2: to request if it is recommended to include all three terms (bearing capacity, sheet piling, and slope stability) in the thesis.

Case 1: the thesis question is good.

Case 2 it is recommended to include all three terms but better to focus and work in detail on sheet pilling parts.

Supervision 3:

Date: 13.03. 2023

Case 1: is it needed to make some example calculations in such a way that the reader can understand more deeply?

Case 2: is it better to show how to design both cantilever and anchored sheet pile or to choose one of them and design by varying excavation depths?

A recommendation from the supervisor:

Case 1: the supervisor recommended that it is a good idea to include some calculation examples if it is helpful to further clarify the intention needed to be described.

Case 2: the supervisor recommended that it would be better to choose and design one sheet pile type with varying excavation depths than to design both types.

Supervision 4:

Date 20. March. 2023

Case 1: I need your recommendation about the use of sheet pile handbooks. Is it better to refer most the Norwegian road authority handbooks related to sheet piling? Or is it acceptable to refer any other country's sheet pile handbook as long as it is important and relevant?

A recommendation from the supervisor:

The supervisor recommended that any reference can be used however the sources must be cited correctly.

Supervision 5:

Date 03. April 2023

Case 1: do you recommend any idea about some useful software programs to demonstrate the moment-diagram of a cantilever sheet piling?

Case 2: Regarding the lateral earth pressure theory, I have included the Rankine theory, coulomb theory, and the log-spiral theory. Do you recommend including some handwritten example calculations for each?

The supervisor's recommendation:

Case1:

The supervisor recommended that there are many useful programs that can help to demonstrate moment-diagrams. One example is the sap 2000 as we have a license for it.

Case 2:

All these three theories are presented in most textbooks. It is recommended to apply them by giving some example calculations for each theory. Simultaneously it is recommended to get some result based on the Euro-code in the same situation and then to make some comparison among each other.

Supervision 6

Date 10 April 2023

Case 1: I am just focusing on the lateral pressure forces acting on sheet piles. My question is that <<is it important to include some applications of sheet piles where the sheet piles are used to support vertical loads? Sometimes sheet piles are used as vertical load axial supporters after closing their lower end by welding.

Case 1:

Sheeting is mostly for lateral pressure. So, focusing on the lateral pressure forces acting on the sheet piles would be fine.

Supervision 7

17. April 2023

Case 1: The thesis question is modified to (How does a pile sheeting stabilize building constructions on slope areas?) Do you recommend some improvements on it?

Case 2: the title of the master thesis says << sheet piled constructions>> Does it look good, or do you recommend some changes?

The supervisor's recommendation:

Case 1 and case 2: the supervisor replied that the thesis question is fine. However, regarding the scope of limit can be seen at the next meeting after seeing the overall result.

Supervision 8:

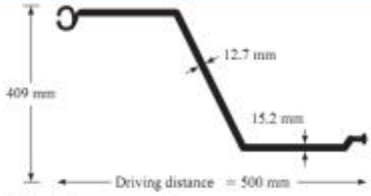
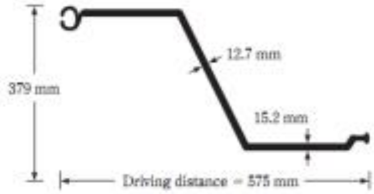
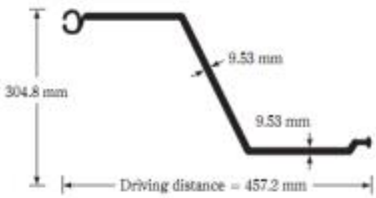
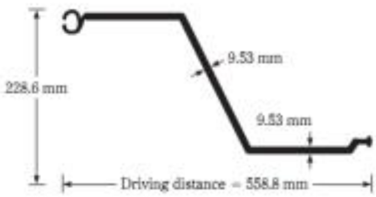
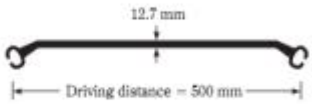
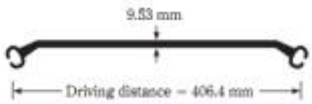
Date 15 May 2023

Case1: to show the result of the report and to get some recommendations from the supervisor.

The supervisor saw the results and recommended to add captions of their sources for all figures on the result.

12. Attachements

12.1 sheet pile profiles

Section designation	Sketch of section	Section modulus m^3/m of wall	Moment of inertia m^4/m of wall
PZ-40		326.4×10^{-5}	670.5×10^{-6}
PZ-35		260.5×10^{-5}	493.4×10^{-6}
PZ-27		162.3×10^{-5}	251.5×10^{-6}
PZ-22		97×10^{-5}	115.2×10^{-6}
PSA-31		10.8×10^{-5}	4.41×10^{-6}
PSA-23		12.8×10^{-5}	5.63×10^{-6}

12.2 excel calculations: attachment 1

L1=1m

L2= 2m

Kolonne2	Kolonne1	γ'	difference	Kolonne3	sum
data	given values	γsat-γwater	45-ϕ'/2	TAN(45-30/2)	tan(45+ϕ'/2)
Fi= ϕ' in degrees	30	10	30	0,577	1,732
ϕ'/2	15				
γsat in KN/m3	20				
γwater in KN/m3	10				
γ in KN/m3	17				
L1 (m)	1				
L2 (m)	2				
standard angle In degree	45				
allowable flexural stress (δall) KN/m2	170000				

Ka (active pressure)	Kp (passive pressure)	δ'1	δ'2	L3	P	Z	δ'5
tansquared(45-ϕ'/2)	tansquared(45+ϕ'/2)	Ka*L1*γ	δ'1+L2*γ'*Ka	δ'2/(γ'*(Ka-Kp))	a1+a2+a3+a4	Me/P	net p at bottom
0,333	3,000	5,667	12,333	0,4625	25,8	1,62	123,33

difference	product	L3
Kp-Ka	γ'*(ka-Kp)	
2,667	26,67	0,46

Areas of ACDE region					total sheet pile length	
a1	a2	a3	a4		D=L3+L4	L=L1+L2+D
2,8	11,33333333	6,666666667	4,9		3,76	6,76

ME (sum of moment at E)	A1	A2	A3	A4
41,67	4,625	7,73	45,567125	47,10

equation to solve							
	x=L4	y	δ'3	Z'	Mmax	S	δ'4
			L4*(Kp-Ka)*γ'	zero shear	max moment	section modulus	δ'5+γ'*L4*(Kp-Ka)
x^4+A1x^3-A2x^2-A3x-A4=0			88,02	1,39	65,55444372	0,000385614	211,353891
x^4+*x^3-F29*x^2-G29*x-H29=0	3,300770913	3,299957755	88,02055767	1,390143877			211,353891

L1=L2=2m, attachement 2

Kolonne2	Kolonne1	γ'	difference	Kolonne3	sum
data	given values	γsat-γwater	45-ϕ'/2	TAN(45-30/2)	tan(45+ϕ'/2)
Fi= ϕ' in degrees	30	10	30	0,577	1,732
ϕ'/2	15				
γsat in KN/m3	20				
γwater in KN/m3	10				
γ in KN/m3	17				
L1 (m)	2				
L2 (m)	2				
standard angle In degree	45				
allowable flexural stress (δall) KN/m2	170000				

Ka (active pressure)	Kp (passive pressure)	δ'1	δ'2	L3	P	Z	δ'5
tansquared(45-ϕ'/2)	tansquared(45+ϕ'/2)	Ka*L1*γ	δ'1+L2*γ'*Ka	δ'2/(γ'*(Ka-Kp))	a1+a2+a3+a4	Me/P	net p at bottom
0,333	3,000	11,333	18,000	0,675	47,9	1,93	180,00

difference	product	L3
Kp-Ka	γ'*(ka-Kp)	
2,667	26,67	0,68

Areas of ACDE region					sheet pile length	
a1	a2	a3	a4		D=L3+L4	L=L1+L2+D
11,3	22,66666667	6,666666667	7,2		5,05	9,05

ME (sum of moment at E)	A1	A2	A3	A4
92,29	6,75	14,36	114,228	153,05

equation to solve							
	x=L4	y	δ'3	Z'	Mmax	S	δ'4
			L4*(Kp-Ka)*γ'	zero shear	max moment	section modulus	δ'5+γ'*L4*(Kp-Ka)
			116,72	1,89	152,7529243	0,000898547	296,7246436
x^4+A1*x^3-A2*x^2-A3*x-A4=0							
x^4+E29*x^3-F29*x^2-G29*x-H29=0	4,377174134	5,000000977	116,7246436	1,894729532			296,7246436

L1=2, L2=3

Attachment 3

Kolonne2	Kolonne1	γ'	difference	Kolonne3	sum		
data	konstant	γsat-γwater	45-ϕ'/2	TAN(45-30/2)	tan(45+ϕ'/2)		
Fi= ϕ'	30	10	30	0,577	1,732		
ϕ'/2	15						
γsat	20						
γwater	10						
γ=	17						
L1=	2						
L2	3						
standard angle	45						
allowable flexural stress (δall)	170000						
Ka	**Kp**	**δ'1**	**δ'2**	**L3**	**P**	**Z**	**δ'5**
tansquared(45-ϕ'/2)	tansquared(45+ϕ'/2)	Ka*L1*γ	δ'1+L2*γ'*Ka	δ'2/(γ'*(Ka-Kp))	a1+a2+a3+a4	Me/P	net p at bottom
0,333	3,000	11,333	21,333	0,8	68,9	2,33	213,33
difference	product	L3					
Kp-Ka	γ'*(ka-Kp)						
2,667	26,67	0,80					
areas of ACDE				spile length			
				test 3	test 3		
a1	a2	a3	a4	D=L3+L4	L=L1+L2+D		
11,3	34	15	8,5	6,05	11,05		
ME (sum of moment at E)	A1	A2	A3	A4			
160,37	8	20,66	196,128	315,35			
equation to solve	x=L4	y	δ'3	Z'	Mmax	S	δ'4
			L4*(Kp-Ka)*γ'	zero shear	max moment	section modulus	δ'5+γ'*L4*(Kp-Ka)
x^4+8x^3-20,66x^2-196,128x-315,35=0	5,25	1,86045E-07	139,91	2,27	264,713846	0,00155714	353,2412574

12.1.4 Attachment 4:

L1=L2=3

Kolonne2	Kolonne1	γ'	difference	Kolonne3	sum		
data	given values	γsat-γwater	45-ϕ'/2	TAN(45-30/2)	tan(45+ϕ'/2)		
Fi= ϕ' in degrees	30	10	30	0,577	1,732		
ϕ'/2	15						
γsat in KN/m3	20						
γwater in KN/m3	10						
γ in KN/m3	17						
L1 (m)	3						
L2 (m)	3						
standard angle In degree	45						
allowable flexural stress (δall) KN/m2	170000						
Ka (active pressure)	**Kp (passive pressure)**	**δ'1**	**δ'2**	**L3**	**P**	**Z**	**δ'5**
tansquared(45-ϕ'/2)	tansquared(45+ϕ'/2)	Ka*L1*γ	δ'1+L2*γ'*Ka	δ'2/(γ'*(Ka-Kp))	a1+a2+a3+a4	Me/P	net p at bottom
0,333	3,000	17,000	27,000	1,0125	102,3	2,63	270,00
difference	product	L3					
Kp-Ka	γ'*(ka-Kp)						
2,667	26,67	1,01					
Areas of ACDE region					sheet pile length	test 4	
a1	a2	a3	a4		D=L3+L4	L=L1+L2+D	
25,5	51	15	10,8		7,33	13,33	
ME (sum of moment at E)	A1	A2	A3	A4			
269,38	10,125	30,69	354,272625	672,55			
equation to solve							
	x=L4	y	δ'3	Z'	Mmax	S	δ'4
			L4*(Kp-Ka)*γ'	zero shear	max moment	section modulus	δ'5+γ'*L4*(Kp-Ka)
			168,41	2,77	458,2878246	0,002695811	438,4084041
x^4+A1*x^3-A2*x^2-A3*x-A4=0							
x^4+E29*x^3-F29*x^2-G29*x-H29=0	6,315315152	7,000000164	168,4084041	2,769927797			438,4084041

L1=3

L2=4

Kolonne2	Kolonne1	γ'	difference	Kolonne3	sum
data	konstant	γsat-γwater	45-ϕ'/2	TAN(45-30/2)	tan(45+ϕ'/2)
Fi= ϕ'	30	10	30	0,577	1,732
ϕ'/2	15				
γsat	20				
γwater	10				
γ=	17				
L1=	3				
L2	4				
standard angle	45				
allowable flexural stress (δall)	170000				

Ka	Kp	δ'1	δ'2	L3	P	Z	δ'5
tansquared(45-ϕ'/2)	tansquared(45+ϕ'/2)	Ka*L1*γ	δ'1+L2*γ'*Ka	δ'2/(γ'*(Ka-Kp))	a1+a2+a3+a4	Me/P	net p at bottom
0,333	3,000	17,000	30,333	1,1375	132,3	3,04	303,33
difference	product	L3					
Kp-Ka	γ'*(ka-Kp)						
2,667	26,67	1,14					
Areas of ACDE region					sheet pile length		
a1	a2	a3	a4		D=L3+L4	L=L1+L2+D	
25,5	68	26,66666667	12,1		8,34	15,34	
ME (sum of moment at E)	A1	A2	A3	A4			
401,77	11,375	39,69	519,400125	1 126,73			
equation to solve							
	x=L4	y	δ'3	Z'	Mmax	S	δ'4
			L4*(Kp-Ka)*γ'	zero shear	max moment	section modulus	δ'5+γ'*L4*(Kp-Ka)
x^4+A1x^3-A2x^2-A3x-A4=0			191,99	3,15	679,5962484	0,003997625	495,319273
C39x^4+D34*x^3-E34*x^2-F34*x-G34=0	7,199472736	8,00000038	191,9859396	3,15	679,6049314	0,003997676	495,319273

L1=6m

L2= 9m

Kolonne2	Kolonne1	γ'	difference	Kolonne3	sum
data	given values	γsat-γwater	45-ϕ'/2	TAN(45-30/2)	tan(45+ϕ'/2)
Fi= ϕ' in degrees	30	10	30	0,577	1,732
ϕ'/2	15				
γsat in KN/m3	20				
γwater in KN/m3	10				
γ in KN/m3	17				
L1 (m)	6				
L2 (m)	9				
standard angle In degree	45				
allowable flexural stress (δall) KN/m2	170000				

Ka (active pressure)	Kp (passive pressure)	δ'1	δ'2	L3	P	Z	δ'5
tansquared(45-ϕ'/2)	tansquared(45+ϕ'/2)	Ka*L1*γ	δ'1+L2*γ'*Ka	δ'2/(γ'*(Ka-Kp))	a1+a2+a3+a4	Me/P	net p at bottom
0,333	3,000	34,000	64,000	2,4	568,6	5,94	640,00
difference	product	L3					
Kp-Ka	γ'*(ka-Kp)						
2,667	26,67	2,40					
Areas of ACDE region					sheet pile length		
a1	a2	a3	a4		D=L3+L4	L=L1+L2+D	
102,0	306	135	25,6		17,18	32,18	
ME (sum of moment at E)	A1	A2	A3	A4			
3379,25	24	170,58	4591,104	20 066,56			
equation to solve							
	x=L4	y	δ'3	Z'	Mmax	S	δ'4
			L4*(Kp-Ka)*γ'	zero shear	max moment	section modulus	δ'5+γ'*L4*(Kp-Ka)
			394,12	6,53	5854,677652	0,03443928	1034,123919
x^4+A1*x^3-A2*x^2-A3*x-A4=0							
x^4+E29*x^3-F29*x^2-G29*x-H29=0	14,77964697	15,00005523	394,1239192	6,530313928			1034,123919

L1=8, L2=12

Kolonne2	Kolonne1	γ'	difference	Kolonne3	sum
data	given values	γsat-γwater	45-ϕ'/2	TAN(45-30/2)	tan(45+ϕ'/2)
Fi= ϕ' in degrees	30	10	30	0,577	1,732
ϕ'/2	15				
γsat in KN/m3	20				
γwater in KN/m3	10				
γ in KN/m3	17				
L1 (m)	8				
L2 (m)	12				
standard angle In degree	45				
allowable flexural stress (δall) KN/m2	170000				

Ka (active pressure)	Kp (passive pressure)	δ'1	δ'2	L3	P	Z	δ'5
tan^2(45-ϕ'/2)	tan^2(45+ϕ'/2)	Ka*L1*γ	δ'1+L2*γ'*Ka	δ'2/(γ'*(Ka-Kp))	a1+a2+a3+a4	Me/P	net p at bottom
0,333	3,000	45,333	85,333	3,2	1101,9	7,08	853,33

difference	product	L3
Kp-Ka	γ'*(ka-Kp)	
2,667	26,67	3,20

Areas of ACDE region					sheet pile length	
a1	a2	a3	a4		D=L3+L4	L=L1+L2+D
181,3	544	240	136,5		23,37	43,37

ME (sum of moment at E)	A1	A2	A3	A4
7801,17	32	330,56	11443,968	62 997,82

equation to solve							
	x=L4	y	δ'3	Z'	Mmax	S	δ'4
			L4*(Kp-Ka)*γ'	zero shear	max moment	section modulus	δ'5+γ'*L4*(Kp-Ka)
			538,00	9,09	14478,96611	0,085170389	1391,331079
x^4+A1*x^3-A2*x^2-A3*x-A4=0							
x^4+E29*x^3-F29*x^2-G29*x-H29=0	20,17491545	21,00005842	537,9977453	9,090654542			1391,331079

12.2 attachment 12.2 varying angle (30-36) with constant depth

L1 =2, L2= 3, ϕ=30

Kolonne2	Kolonne1	γ'	difference	Kolonne3	sum
data	given values	γsat-γwater	45-ϕ'/2	TAN(45-30/2)	tan(45+ϕ'/2)
Fi= ϕ' in degrees	36	10	27	0,510	1,963
ϕ'/2	18				
γsat in KN/m3	20				
γwater in KN/m3	10				
γ in KN/m3	17				
L1 (m)	2				
L2 (m)	3				
standard angle In degree	45				
allowable flexural stress (δall) KN/m2	170000				

Ka (active pressure)	Kp (passive pressure)	δ'1	δ'2	L3	P	Z	δ'5
tan^2(45-ϕ'/2)	tan^2(45+ϕ'/2)	Ka*L1*γ	δ'1+L2*γ'*Ka	δ'2/(γ'*(Ka-Kp))	a1+a2+a3+a4	Me/P	net p at bottom
0,260	3,852	8,827	16,615	0,46253899	50,8	2,37	263,13

difference	product	L3
Kp-Ka	γ'*(ka-Kp)	
3,592	35,92	0,46

Areas of ACDE region					sheet pile length	
a1	a2	a3	a4		D=L3+L4	L=L1+L2+D
8,8	26,48085074	11,68272827	3,8		4,52	9,52

ME (sum of moment at E)	A1	A2	A3	A4
120,43	7,325077981	11,32071393	102,4249256	155,36

equation to solve							
	x=L4	y	δ'3	Z'	Mmax	S	δ'4
			L4*(Kp-Ka)*γ'	zero shear	max moment	section modulus	δ'5+γ'*L4*(Kp-Ka)
			145,92	1,68	177,4441231	0,001043789	409,0500679
x^4+A1*x^3-A2*x^2-A3*x-A4=0							
x^4+E29*x^3-F29*x^2-G29*x-H29=0	4,062020631	5,000000349	145,9168724	1,682313432			409,0500679

12.4 hand calculations:

The following calculations represent the four different lateral earth pressure theories after assuming some values.

ϕ=36, c=a= attraction =0

dimension of sheet pile (width= 0.9m and height=3m), E=

12.4.1 The Rankine theory:

$K_a = \tan\left(45 - \frac{36}{2}\right) = 0.5095$

$K_p = \tan\left(45 + \frac{36}{2}\right) = 1.963$

$p_v = ɣ * Z + q = \frac{13KN}{m3} * 3m + 0 = 39KPa$

$p_p = P_v * K_p + 2 * c * \sqrt{K_p} = 39KPa * 1.963 + 2 * 0 * \sqrt{1.963} = 77KPa$

$P_p = p_p * A = 77KPa * 0.9m * 3m = 208KN$

$\delta = \frac{p_p}{E} * 2 * H = \frac{77KPa*2*3m}{96000KPa} = 5mm$

12.4.2 The Coulomb Theory:

ϕ'= 36, c=7KPa, α=90, β=0, r= R_{int} =0.8. beta needs not to be zero. E=96000KPa

$\tan\delta = R_{int} * \tan ϕ$

$\tan\delta = 0.8 * \tan 36 = 0.58$

$\delta = \tan^{-1} 0.58 = 30$

$$K_p = \frac{sin^2(\alpha-ϕ')}{sin^2(\alpha)*\sin(\alpha+\delta)*\left(1-\sqrt{\frac{\sin(ϕ'+\delta)*\sin(ϕ'+\beta)}{\sin(\alpha+\delta)*\sin(\alpha+\beta)}}\right)^2}$$

$$= \frac{sin^2(90-36)}{sin^2(90)*\sin(90+34.6)*\left(1-\sqrt{\frac{\sin(36+30)*\sin(36+0)}{\sin(90+30)*\sin(90+0)}}\right)^2} = 16.72$$

$p_p = p_v K_p + 2 * c * \sqrt{K_p} = 39KPa * 16.72 + 2 * 7KPa * \sqrt{16.72} = 709KPa$

$P_p = p_p * A = (709KPa * 0.9m * 3m = 1914KN)$

$P_p = \frac{1}{2} * K_p * ɣ * H^2 = 0.5 * 16.72 * 13KN/m^3 * (3m)^2 = 978KN$

$\delta = \frac{p_p}{E} * 2 * H = \frac{709KPa}{96000KPa} * 2 * 3m = 44mm$

12.4.3 The Logarithmic Theory

Let us assume that the β to ϕ ratio is equal to 0.2, and the -δ to ϕ ratio is 0.4. Let ϕ be 35 degrees. Then the reduction factor from the table will be 0.417 and the Kp value from the chart will be 17.5. Then:

$K_p = R(for \frac{-\delta}{\phi} = -0.1) * (Kp\, for \frac{\beta}{\phi} = 0.4)$

$K_p = 0.417 * 17.5 = 7.3$

$\delta_p = K_p * \gamma * H$ = 7.3*13KN/m^3*3m=285KPa.

$P_P = \delta_p * A = 285KPa * 0.9, * 3m = 770KN$

$\delta = \frac{p_P}{E} * 2 * H = \frac{285}{96000} * 2 * 3m = 17.8mm$

$P_p = K_p * \frac{H^2 * \delta_p}{2} = 7.3 * \frac{(3m)^2 * \frac{285KN}{m2}}{2} = 1283KN$

12.4.4 The Janbu's theory:

Let us assume that ϕ= 36, a=c 7KPa, r=0.8, γ_m=1.3, ɣ=13KN/m3, z= 3m (q=0 for simplicity's sake)
$p_v = \gamma * Z + q$ =39KPa

$\tan\rho = \frac{\tan\varphi}{\gamma_m} = \frac{\tan 36}{1.3} = 0.6$

Referring to Janbu's theory and chart for (r=0.8 assumed) and $\tan\rho = 0.6$, gives Kp=4.5.
Thus:
$p_p + c = K_p(p_v + c)$

$p_p = K_p(p_v + c) - c = 4.5 * (39 + 7) - 7 = 200KPa$

$P_p = p_p * A = 200KPa * 0.9m * 3m = 450KN$

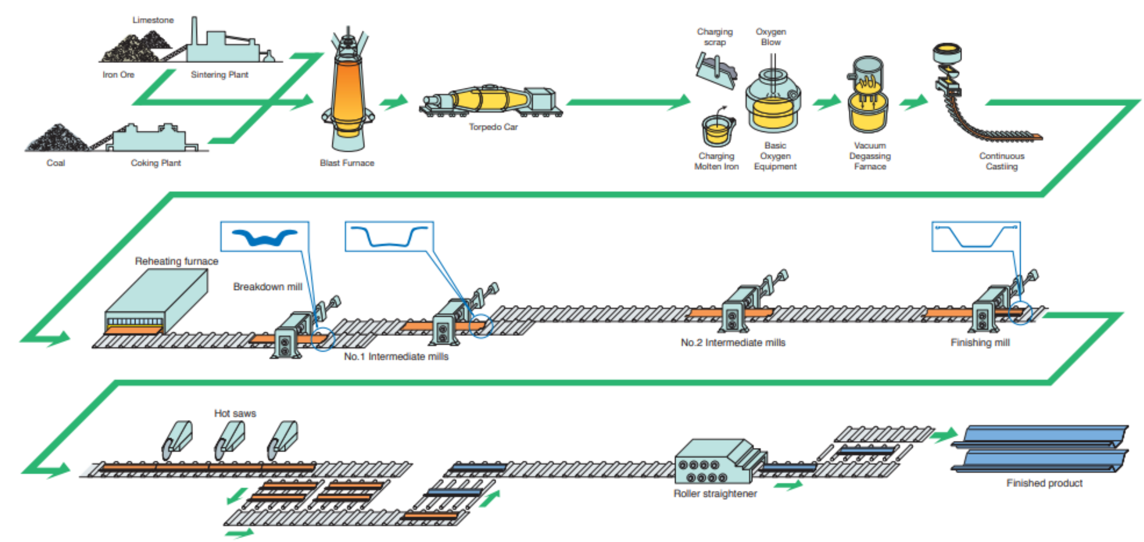
$\delta = \frac{p_p}{E} * 2 * H = \frac{200KPa}{96000KPa} * 2 * 3m = 12.5mm$

Steel sheet piles and their descriptions

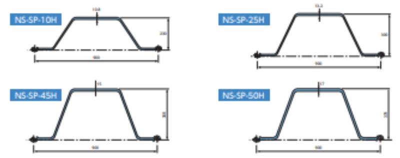
Product lineup

		Shape	Type	Moment of inertia I (cm⁴/m)	Section modulus Z (cm³/m)	Unit mass W (kg/m²)		Maximum length L (m)	Grade: JIS A 5523 SYW295	JIS A 5523 SYW390	JIS A 5523 SYW430	JIS A 5528 SY295	JIS A 5528 SY390	KS F4604 SY300	EN10248 S355GP	EN10248 S430GP	ASTM A572 Gr.50	ASTM A992 Gr.50
Hat-type sheet piles	Hat w = 900 mm		NS-SP-10H	10,500	902	96		30*2	✓	✓	✓*4	–	–	–	–*2	–*4	✓*2	–
			NS-SP-25H	24,400	1,610	126		30*2	✓	✓*1	✓*1	–	–	–	–*2	–*4	✓*2	–
			NS-SP-45H	45,000	2,450	163		24*1*2	✓	✓	✓*1	–	–	–	–*2	–*2	–*4	–
			NS-SP-50H	51,100	2,760	186		23.5*1*2	✓*1	✓*1	✓*1	–	–	–	–*2	–*2	–*4	–
	Hat+H w = 900 mm		NS-SP-10H+HY NS-SP-25H+HY	87,800 to 1,316,000	2,320 to 19,970	169 to 514		30*2	Please refer to "NS-SP-10H"and "NS-SP-25H". H-shapes is also available JIS, ASTM and BS.*2									
	Hat + (Hat+H) w = 900 mm		NS-SP-10H+HY	49,100 to 273,000	1,290 to 4,940	132 to 210		30*2										
Straight web-type sheet piles	Straight web w = 500 mm		NS-SP-FL	396	89	123		38*2	✓	✓	–*4	✓	✓	–	✓*2*3	–*4	✓*2	✓
			NS-SP-FXL	570	121	154		38*2	✓	✓	✓	✓	✓	–	✓*2	✓*2	✓*2	✓
U-type sheet piles	U-type w = 600 mm		NS-SP-Ⅱw	13,000	1,000	103		30*2	✓	✓	–*4	✓	✓	✓	✓*2	–*4	✓*2	–
			NS-SP-Ⅲw	32,400	1,800	136		30*2	✓	✓	–*4	✓	✓	✓	✓*2	–*4	✓*2	–
			NS-SP-Ⅳw	56,700	2,700	177		30*2	✓	✓	–	✓	✓	✓	✓*2	–*4	✓*2	–
	w = 500 mm		NS-SP-VL	63,000	3,150	210		30*2	✓	✓	–	✓	✓	✓	✓*2	–*4	✓*2	–
			NS-SP-VIL	86,000	3,820	240		30*2	✓	✓	–	✓	✓	✓	✓*2	–*4	✓*2	–
	w = 400 mm		NS-SP-Ⅱ	8,740	874	120		25*2	✓	–*4	✓*4	✓	–*4	–	–*4	–*4	–	–
			NS-SP-Ⅲ	16,800	1,340	150		30*2	✓	✓	✓*1	✓	✓	✓	✓*1*2	✓*1*2	✓*2	–
			NS-SP-ⅢA	22,800	1,520	146		30*2	–	✓	–	✓	–	✓	–	–	–	–
			NS-SP-Ⅳ	38,600	2,270	190		30*2	✓	✓	✓	✓	✓	✓	✓*1*2	✓*1*2	✓*2	–
NS-SP-J	NS-SP-J w = 600 mm		NS-SP-J	12,090	1,175	145		30*2	✓	✓	–	✓	✓	✓	✓*4	–*4	✓*2	–

Production process



Shapes



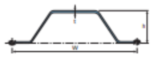
Deviation angle

Each interlock allows for a certain rotation. The minimum angle of coupling mating joint (the interlock swing) for the combination of the identical versions of Hat-type sheet piles is shown in the figure below.

Compatibility

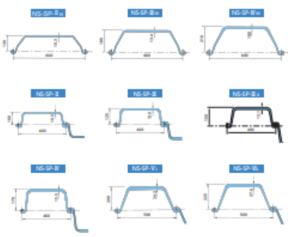
NS-SP-10H — NS-SP-25H — NS-SP-45H — NS-SP-50H

Sectional properties



Type	Dimension			Per pile				Per 1 m of pile wall width			
	Effective width W mm	Effective height h mm	Thickness t mm	Sectional area cm²	Moment of inertia cm⁴	Section modulus cm³	Unit mass kg/m	Sectional area cm²/m	Moment of inertia cm⁴/m	Section modulus cm³/m	Unit mass kg/m²
NS-SP-10H	900	230	10.8	110.0	9,430	812	86.4	122.2	10,500	902	96.0
NS-SP-25H	900	300	13.2	144.4	22,000	1,450	113	160.4	24,400	1,610	126
NS-SP-45H	900	368	15.0	187.0	40,500	2,200	147	207.8	45,000	2,450	163
NS-SP-50H	900	370	17.0	212.7	46,000	2,490	167	236.3	51,100	2,760	186

Shapes

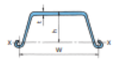


Compatibility

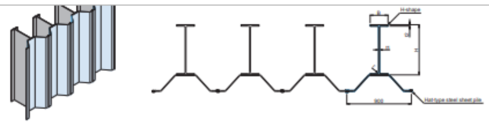
Deviation Angle

Each interlock allows a certain rotation. The minimum angle of deviation (the interlock swing) for the combination of the identical versions of U-type sheet piles is shown in the figure below.

Sectional properties

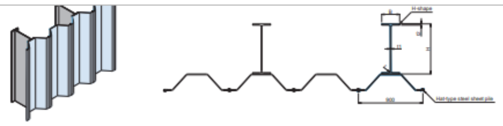


Type	Dimension			Per pile				Per 1 m of pile wall width			
	Effective width W mm	Effective height h mm	Thickness t mm	Sectional area cm²	Moment of inertia cm⁴	Section modulus cm³	Unit mass kg/m	Sectional area cm²/m	Moment of inertia cm⁴/m	Section modulus cm³/m	Unit mass kg/m²
NS-SP-II	400	100	10.5	61.18	1,240	152	48.0	153.0	8,740	874	120
NS-SP-III	400	125	13.0	76.42	2,220	223	60.0	191.0	16,800	1,340	150
NS-SP-IIIA	400	150	13.1	74.40	2,790	250	58.4	186.0	22,800	1,520	146
NS-SP-IV	400	170	15.5	96.99	4,670	362	76.1	242.5	38,600	2,270	190
NS-SP-VL	500	200	24.3	133.8	7,960	520	105	267.6	63,000	3,150	210
NS-SP-VIL	500	225	27.6	153.0	11,400	680	120	306.0	86,000	3,820	240
NS-SP-IIw	600	130	10.3	78.70	2,110	203	61.8	131.2	13,000	1,000	103
NS-SP-IIIw	600	180	13.4	103.9	5,220	376	81.6	173.2	32,400	1,800	136
NS-SP-IVw	600	210	18.0	135.3	8,630	539	106	225.5	56,700	2,700	177



Hat type sheet pile	NSHYPER BEAM ™					Mass per m² of wall	Sectional area	Section modulus	Moment of inertia
	H	B	t1	t2	r	kg/m²	cm²/m	cm³/m	cm⁴/m
NS-SP-10H	400	200	9	22	13	202	257	3,240	113,400
	400	200	12	22	13	211	269	3,410	117,100
	450	200	9	16	13	186	237	2,970	119,100
	450	200	9	19	13	196	250	3,280	128,300
	450	250	9	16	13	200	255	3,420	132,600
	450	250	9	19	13	212	271	3,790	143,500
	450	250	9	22	13	225	287	4,160	153,800
	450	250	12	25	13	248	316	4,700	167,700
	450	250	12	28	13	261	332	5,040	176,500
	500	200	9	16	13	190	242	3,270	142,000
	500	200	9	19	13	200	255	3,600	153,000
	500	200	9	22	13	210	267	3,920	163,400
	500	200	12	19	13	212	270	3,860	160,200
	500	200	12	22	13	222	282	4,170	170,100
	500	200	12	25	13	231	295	4,470	179,500
	500	250	9	16	13	204	260	3,750	158,100
	500	250	9	19	13	216	276	4,160	171,200
	500	250	9	22	13	229	292	4,560	183,600
	500	250	12	22	13	241	307	4,800	189,700
	500	250	12	25	13	253	323	5,180	201,000
	500	250	12	28	13	266	339	5,550	211,800
	600	200	12	16	13	212	271	4,270	208,200
	600	200	12	19	13	222	283	4,640	222,400
	600	200	12	22	13	232	296	5,010	236,000
	600	200	12	25	13	242	308	5,350	248,600
	600	200	12	28	13	252	321	5,720	261,900
	600	250	12	19	13	239	304	5,280	246,600
	600	250	12	22	13	251	320	5,740	262,900
	600	250	12	25	13	264	336	6,190	278,900
	600	250	12	28	13	276	352	6,630	294,500
	600	250	16	28	13	295	376	7,010	304,800
	600	250	16	32	13	312	397	7,570	323,900
	650	250	12	19	13	244	311	5,750	286,300
	650	250	12	22	13	257	327	6,240	305,300
	650	250	12	25	13	269	343	6,720	323,800

Hat type sheet pile	NSHYPER BEAM ™					Mass per m² of wall	Sectional area	Section modulus	Moment of inertia
	H	B	t1	t2	r	kg/m²	cm²/m	cm³/m	cm⁴/m
NS-SP-10H	700	250	14	25	18	287	365	7,580	383,000
	700	250	14	28	18	299	381	8,080	403,300
	750	250	12	19	18	256	326	6,770	378,300
	750	250	12	22	18	268	342	7,330	403,400
	750	250	12	25	18	281	358	7,890	428,100
	750	250	14	22	18	280	357	7,840	414,700
	750	250	14	25	18	293	373	8,200	438,900
	750	250	14	28	18	305	389	8,730	461,700
	750	250	16	28	18	317	404	9,000	470,800
	750	250	16	32	18	334	425	9,710	500,700
	800	250	12	22	18	273	348	7,900	458,700
	800	250	14	22	18	287	365	8,240	471,700
	800	250	14	25	18	299	381	8,820	498,400
	800	250	14	28	18	311	397	9,370	523,600
	800	250	16	22	18	300	382	8,570	484,200
	800	250	16	25	18	312	398	9,140	510,500
	800	250	16	28	18	324	413	9,690	535,400
	800	250	16	32	18	341	434	10,440	568,900
	850	250	14	22	18	293	373	8,840	532,300
	850	250	16	22	18	307	391	9,220	547,600
	850	250	16	25	18	319	406	9,810	576,400
	900	250	14	19	18	286	365	8,810	563,300
	900	250	16	19	18	302	384	9,240	582,000
	900	250	16	22	18	314	400	9,890	615,800
	900	250	16	25	18	326	415	10,520	648,000
	900	250	16	28	18	338	431	11,140	680,000
	1000	250	16	22	18	328	418	11,270	766,300
	1000	250	16	25	18	340	433	11,980	806,700
	1000	250	19	25	18	365	465	12,680	838,900
	1000	250	19	28	18	377	480	13,360	877,300
	1000	250	16	22	18	358	456	12,240	892,400
	1000	250	16	25	18	370	471	12,980	935,900
	1000	250	19	25	18	395	503	13,770	973,700
	1000	250	19	28	18	407	518	14,470	1,014,600
	1000	300	16	22	18	377	480	13,420	962,300



Hat type sheet pile	NSHYPER BEAM ™					Mass per m² of wall	Sectional area	Section modulus	Moment of inertia
	H	B	t1	t2	r	kg/m²	cm²/m	cm³/m	cm⁴/m
NS-SP-10H	400	200	9	22	13	149	190	1,770	61,900
	400	200	12	22	13	154	196	1,860	63,800
	450	200	9	16	13	141	180	1,620	64,800
	450	200	9	19	13	146	186	1,770	69,400
	450	250	9	16	13	148	188	1,840	71,600
	450	250	9	19	13	154	196	2,030	77,000
	450	250	9	22	13	160	204	2,220	82,100
	450	250	12	25	13	172	219	2,500	89,100
	450	250	12	28	13	178	227	2,670	93,500
	500	200	9	16	13	143	182	1,750	76,200
	500	200	9	19	13	148	188	1,920	81,800
	500	200	9	22	13	153	195	2,080	86,900
	500	200	12	19	13	154	196	2,060	85,400
	500	200	12	22	13	159	202	2,210	90,300
	500	200	12	25	13	164	209	2,370	95,000
	500	250	9	16	13	150	191	2,000	84,300
	500	250	9	19	13	156	199	2,210	90,900
	500	250	9	22	13	162	207	2,410	97,100
	500	250	12	22	13	168	215	2,530	100,100
	500	250	12	25	13	175	223	2,720	105,700
	500	250	12	28	13	181	230	2,910	111,100
	550	200	9	19	13	150	191	2,080	95,600
	550	200	12	19	13	157	199	2,240	100,100
	550	200	12	22	13	161	206	2,410	105,900
	550	200	12	25	13	166	212	2,570	111,500
	550	250	12	22	13	171	218	2,750	117,500
	550	250	12	25	13	177	226	2,960	124,300
	600	200	12	16	13	154	196	2,240	109,300
	600	200	12	19	13	159	203	2,430	116,400
	600	200	12	22	13	164	209	2,610	123,200
	600	200	12	25	13	169	215	2,790	129,600
	600	200	12	28	13	174	222	2,970	136,200
	600	250	12	19	13	167	213	2,750	128,500

Hat type sheet pile	NSHYPER BEAM ™					Mass per m² of wall	Sectional area	Section modulus	Moment of inertia
	H	B	t1	t2	r	kg/m²	cm²/m	cm³/m	cm⁴/m
NS-SP-10H	600	250	12	22	13	174	221	2,980	136,700
	600	250	12	25	13	180	229	3,210	144,700
	600	250	12	28	13	186	237	3,430	152,500
	600	250	16	28	13	196	249	3,620	157,600
	600	250	16	32	13	204	260	3,910	167,200
	650	250	12	19	13	170	217	2,980	148,400
	650	250	12	22	13	176	225	3,230	157,900
	650	250	12	25	13	183	233	3,470	167,200
	650	250	12	28	13	189	240	3,710	176,300
	700	200	12	19	18	165	210	2,870	155,100
	700	200	12	22	18	170	216	3,080	164,000
	700	200	12	25	18	175	223	3,280	172,600
	700	250	12	19	18	173	221	3,240	170,900
	700	250	12	22	18	179	229	3,510	182,200
	700	250	12	25	18	186	237	3,760	192,700
	700	250	14	22	18	185	236	3,640	186,400
	700	250	14	25	18	191	244	3,890	196,800
	700	250	14	28	18	198	252	4,150	206,900
	750	250	12	19	18	176	224	3,480	194,400
	750	250	12	22	18	182	232	3,760	207,000
	750	250	12	25	18	188	240	4,040	219,300
	750	250	14	22	18	188	240	3,920	212,600
	750	250	14	25	18	194	248	4,200	224,700
	750	250	14	28	18	201	256	4,460	236,100
	750	250	16	28	18	207	263	4,600	240,600
	750	250	16	32	18	215	274	4,950	255,600
	800	250	12	22	18	185	235	4,040	234,600
	800	250	14	22	18	191	244	4,210	241,100
	800	250	14	25	18	197	252	4,500	254,400
	800	250	14	28	18	204	259	4,780	267,100
	800	250	16	22	18	198	252	4,380	247,400
	800	250	16	25	18	204	260	4,670	260,500
	800	250	16	28	18	210	268	4,940	273,000

Note: Other size except for the regular size mentioned above also can be available. Please contact us.