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DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING
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DESIGN OF AN EARTH RETAINING WALL WORKING ALSO AS A FLOOR SLAB FOR A BUILDING: CASE STUDY OF A RETAINING WALL IN BASTOS YAOUNDE

A thesis submitted in partial fulfilment of the requirement for the degree of Master of Engineering (MEng) in Civil Engineering

Curriculum: Geotechnical engineering

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DEDICATION

I dedicate this work to my precious family which has supported me throughout the journey.

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LIST OF SYMBOLS AND ABBREVIATIONS

A	Area of the cross section
A_c	Area of the concrete cross section
As,	Minimum section area
A_s	Area of the steel reinforcement section
A_{sw}	Cross sectional area of the shear reinforcement
$A_{s,\ sup}$	Area of steel reinforcement for section in compression
$A_{S,\ provided}$	Area of steel section provided
C_{min}	Minimum concrete cover
$C_{min,b}$	Minimum cover due to bond requirement
$C_{min,dur}$	Minimum cover due to environmental conditions
M_{Ed}	Maximum Bending moment
M_{Rd}	Resisting moment
c	Concrete cover
d	Effective height of the section
f_{cd}	Design resisting strength of the concrete
f_{ctm}	Tensile strength of the concrete
f_{yd}	Design yielding strength of the steel
f_{yk}	Characteristic yield strength
f_{ywd}	Design yield strength of the shear reinforcement
h	Structure height or the effective structure height
$s_{l,max}$	Maximum longitudinal spacing
$s_{t,max}$	Maximum transversal spacing
γ_c	Partial factor for concrete
γs	Partial safety factor for steel
ν	Poisson’s ratio
ν_1	Reduction factor for concrete cracked on shear
δ	Friction angle at the soil-pile contact surface
θ	Angle of concrete compression struts to the beam axis
ρ_w	Shear reinforcement ratio

σ_c	Stress in the concrete
σ_s	Stress in the reinforcement
EC	Eurocode
MC	Mohr-Coulomb
V_{ED}	Maximum shear force
N_{ED}	Maximum axial force
$M_{RD, lim}$	Limit bending moment with tension reinforcement
FEM	Finite element method
ULS	Ultimate Limit State
SLS	Serviceability Limit State

ABSTRACT

The main objective of this work is to design a retaining structure that also works as a floor slab for a building. A new storey building has to be constructed in Golf Bastos located in Yaounde the capital of Cameroon. The area to be built is surrounded by three deep excavations with a slope on the rear side besides which there is an existing building (R+12) whose property line is 2.5 m from the slope's surface. The wall has to be designed so as to support the huge loads and ensure slope stability. To achieve this objective, a comprehensive review of different types of retaining walls and the types of failures they exhibit were clearly presented. In addition, the forces acting on retaining walls were studied and the methods by which retaining walls could be analysed and designed were described. Then, a site recognition through documentary research was done in order to have information related to the project. This was followed by a collection of data necessary for the study from project documents. The data collected permitted to perform a numerical analysis using the finite element software PLAXIS 2D to check for the stability of the wall and predesign is made by verifying the stability of arbitrary dimensions on PLAXIS. The analysis was done using mainly the Mohr-Coulomb material model. Furthermore, the axial forces, shear forces and bending moments on the wall were equally obtained from this software to perform the structural design. Formulae from Eurocode were used to determine the steel reinforcement necessary for the wall to resist to bending moments and shear under lateral loads and axial loads. The retaining structure was backfilled with gravel laterite and a thickness of 0.4 m was able to resist to these loads with the addition of a micropile of 0.25 cm as a support system below the raft foundation on the backfill. The maximum settlement was 83mm without micropile and 24mm which is below the prescribed limit of 50mm given by the Eurocode. Also, the safety factor gave a value of 1.13 without the micropile and 2.37 with the micropile which is greater than the minimum desirable value of 1.5.

Keywords. Design, Retaining wall, floor slab, deep excavation

RESUME

L'objectif principal de ce travail est de dimensionner un ouvrage de soutènement qui fonctionne également comme un dallage pour un bâtiment. Un nouveau bâtiment à étages doit être construit au Golf Bastos situé à Yaoundé la capitale du Cameroun. La zone à construire est entourée de trois excavations profondes avec un talus sur la face arrière à côté duquel se trouve un bâtiment existant (R+12) dont la limite de propriété est à 2,5 m de la surface du talus. Le mur doit être dimensionné de manière à supporter les charges énormes et assurer la stabilité de la pente. Pour atteindre cet objectif, une étude progressive et objective des différents types de murs de soutènement et des types de défaillances qu'ils présentent a été clairement présentée. De plus, les forces agissant sur les murs de soutènement ont été étudiées et les méthodes d'analyses et de dimensionnement des murs de soutènement ont été décrites. Ensuite, une reconnaissance du site par une recherche documentaire a été faite afin d'avoir des informations liées au projet. Cela a été suivi d'une collecte des données nécessaires à l'étude à partir des documents du projet. Les données recueillies ont permis d'effectuer une analyse numérique à l'aide du logiciel d'éléments finis PLAXIS 2D pour vérifier la stabilité du mur et le pré dimensionnement est fait en vérifiant la stabilité de dimensions arbitraires sur PLAXIS. L'analyse a été effectuée en utilisant principalement le modèle de matériau de Mohr-Coulomb. De plus, les forces axiales, les forces de cisaillement et les moments de flexion sur le mur ont également été obtenus à partir de ce logiciel pour effectuer le dimensionnement structurel. Les formules de l'Eurocode ont été utilisées pour déterminer l'armature en acier nécessaire pour que le mur résiste aux moments de flexion et au cisaillement sous les charges latérales et axiales. Le mur de soutènement a été remblayé avec de la latérite de gravier et une épaisseur de 0,4 m a pu résister à ces charges avec l'ajout d'un micropieu sous le radier posé sur le remblai. Le tassement maximal était de 83 mm sans micropieu et de 24 mm, ce qui est inférieur à la limite prescrite de 50 mm donnée par l'Eurocode. Par ailleurs, le facteur de sécurité obtenu est d'une valeur de 1,13 sans le micropieu et de 2,37 avec le micropieu, ce qui est supérieure à la valeur minimale souhaitable de 1,5.

Mots clés. Dimensionnement, mur de soutènement, dallage, excavation profonde

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GENERAL INTRODUCTION

Nowadays, many constructions in geotechnical and civil engineering have a lot to do with retaining walls, so their analysis and design is of a great importance. A retaining wall is any constructed wall that restrains soil or other material at locations having an abrupt change in elevation. The main purpose of a retaining wall is to flatten the slope so that the soil don't flow downward. Retaining walls are often used in construction of buildings having basements, roads or bridges when necessary to retain embankments or earth in a relatively vertical position nevertheless, they have very nice aesthetic roles.

Obviously, many types of retaining walls exist with different forms and structural characteristics depending on the dimensions and location. Each type is constructed based on a number of reasons which may include the type of retained soil, cost, presence of adjacent structures and aesthetics.

The greatest problem with retaining walls is the problem of overcoming the high active pressure resulting from the retained soil and which can cause the wall to collapse. Many scientists such as Coulomb, Monokobe and Okabe have worked on that field and have been able to come out with reliable and simplified expressions to calculate these external forces which are detrimental to our structure.

Unlike conventional retaining walls, the case we are going to design does not only retain soil but also supports some loads from a building which will be built on top acting thus as a floor slab for the building. The wall will retain a soil with height of 8.2m and is constructed besides an existing building which apply some loads on the wall. The main objective of this thesis will be to design a retaining wall that will overcome these huge loads and ensure the stability of the slope.

In order to fulfil our main objectives, we subdivided this thesis in 3 chapters. Chapter one focuses on the literature review of retaining walls. It presents the classification of retaining walls, the different types of retaining walls, the selection of retaining walls, active and passive earth pressures, forces acting on a retaining wall, external and internal stability of a retaining wall and different methods of design of the retaining wall.

Chapter two presents the methodology of the work from site recognition to data collection from project documents. In addition, a 2D axisymmetric modelling with PLAXIS is established in order to check for the stability of the wall. Finally, the structural design with analytical methods is performed for the reinforcement of the wall.

Chapter three focuses on presenting the results of the verifications done in the numerical analysis and computation of the structural design performed analytically. It also interprets the results obtained from the modelling and a cost analysis of the project is done. Finally, we have the general conclusion which gives a general overview of the design and concludes on the research.

CHAPTER 1. LITERATURE REVIEW

Introduction

A Retaining wall is a structure used to provide stability for earth or other materials at their natural slopes. These walls are essentially characterized by the concept that the lateral earth pressures due to self-weight of the retained fill and accompanied surcharge loads are carried by the structural wall. There are many kinds of retaining walls, retaining walls come in all types, shapes and sizes from simple gravity walls to bored pile walls for basements and reinforced soil walls using geogrids to suit a wide range of project needs, and site conditions. Retaining walls are almost indispensable in areas where the slope is too steep to be stable on its own. In this chapter, we will discuss key concepts of retaining walls and the different methods used for their design.

1.1. Earth retaining Walls

A retaining wall is any constructed wall that restrains soil or other material at locations having an abrupt change in elevation. (Brooks, 2010). The main purpose of a retaining wall is to flatten the slope so that the soil doesn't flow downward. There are many reasons for building these structures, some of which include preventing erosion, aesthetic purposes, and stabilizing slopes.

1.1.1. Classification of retaining walls

For simplicity, the wide variety of earth retaining walls have all been classified into three main groups based on specific characteristics. They may be classified according to: load support mechanism, i.e., externally or internally stabilized walls; construction method, i.e., fill or cut walls; and system rigidity, i.e., rigid or flexible walls. Every retaining wall can now be classified by using these three factors. Further description of these classifications is provided subsequently.

1.1.1.1. Classification by load support mechanism

The stability component of walls can be organized according to two principal categories: externally and internally stabilized systems (O'Rourke and Jones, 1990). An externally stabilized system uses an external structural wall against which stabilizing forces are mobilized. Some examples of this category are gravity walls, reinforced concrete cantilever and reinforced concrete counterfort walls. These walls are essentially characterized by the concept that the lateral earth pressures due to self-weight of the retained fill and accompanied surcharge loads are carried by the structural wall. An internally stabilized system comprises of horizontally laid reinforcements which carry most or all of the lateral earth pressure via soil-reinforcement interaction or via passive resistance from the anchor block. This reduces the volume of concrete and steel reinforcement in the wall significantly; thus, its construction is relatively fast speed. Some examples include metal strip walls, geotextile reinforced walls and anchored earth walls. Hybrid systems combine elements of both internally and externally supported walls. An example of a hybrid wall would be a concrete wall built forming an L or T, with pile foundations (Samtani and Nowatzki, 2012).

1.1.1.2. Classification by construction method

Earth Retaining Structures (ERS) can also be classified according to the method required for their construction, i.e., fill construction or cut construction.

Fill wall construction refers to a wall system in which the wall is constructed from the base of the wall up to the top, i.e., “bottom-up” construction.

Cut wall construction refers to a wall system in which the wall is constructed from the top of the wall down to the base concurrent with excavation operations, i.e., “top-down” construction. It is important to recognize that the “cut” and “fill” designations refer to how the wall is constructed, not necessarily the nature of the earthwork associated with the project. For example, a prefabricated modular gravity wall, which may be used to retain earth for a major highway cut, is considered a fill wall because its construction is not complete until the backfill has been placed from the “bottom-up” after the excavation for the cut has reached its final grade (Samtani and Nowatzki, 2012).

1.1.1.3. Classification by system rigidity

The rigidity or flexibility of a wall system is fundamental to the understanding of the development of earth pressures. In simple terms, a wall is considered to be rigid if it moves as a unit in rigid body rotation and/or translation and does not experience bending deformations. Most gravity walls can be considered rigid walls.

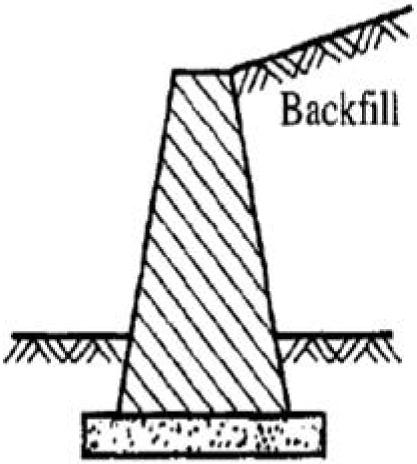
Flexible walls are those that undergo bending deformations in addition to rigid body motion. Such deformations result in a redistribution of lateral pressures from the more flexible to the stiffer portions of the system. Virtually all wall systems, except gravity walls, may be considered to be flexible. Every retaining wall can be classified by using these three factors. For example, a sheet-pile wall would be classified as an externally-stabilized cut wall that is relatively flexible. A Mechanically Stabilized Earth (MSE) wall is an internally stabilized fill wall that is relatively flexible(Samtani and Nowatzki, 2012)

1.1.2. Types of retaining walls

There are several types of retaining walls, some of the popular ones include gravity walls, gabion walls, cantilever walls, counterfort walls, bored pile walls, sheet pile walls, anchored walls and reinforced soil retaining walls which are discussed below.

1.1.2.1. Gravity retaining walls

They depend mostly on their own weight and any soil resting on the wall for stability. Stone, bricks, mass concrete, and precast concrete blocks are the most common materials used in this type of wall construction. To maintain stability, the mass and friction of the interlocking wall materials must be greater than the force of the material being retained. Gravity retaining walls are suitable for height up to 2 to 3 m. A key benefit of gravity walls is their rugged construction, but they are not economical for walls higher than 3 m. Figure 1.1 shows typical gravity walls.



(a) **(b)**

Figure 1.1. Gravity walls (a) Example (Ibrahim, 2018) (b) Plan view (Erizal and Magr, 2017)

1.1.2.2. Gabion retaining walls

Gabions are multi-celled wire or rectangular wire mesh boxes, which are then rock-filled, and used for the construction of erosion control structures and to stabilize steep slopes. The main advantage of this system is its flexibility but it has additional advantages when constructing in remote areas. Only the mesh needs to be transported to the site, and local labour and materials can be used to complete the structure. Gabion walls are particularly good at absorbing impact energy and are often used as rock fall barriers. Gabion walls (Figure 1.2) can be particularly attractive and blend in extremely well with a mountainous natural environment. They are simple to maintain and repair if damaged, and particularly easy to reuse or recycle.

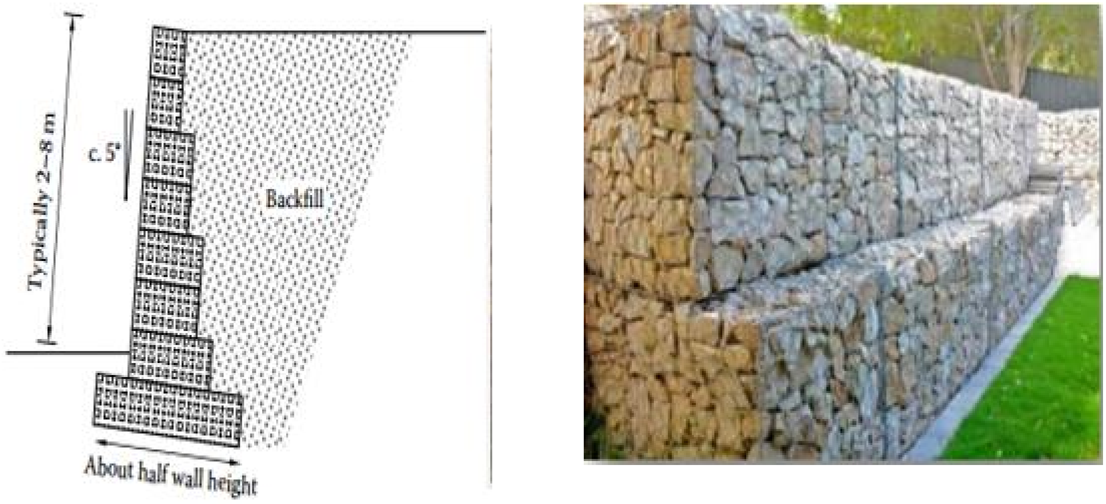


Figure 1.2. Gabion retaining wall. (a) Cross-section through a gabion wall. (Clayton et al., 2013) (b) Detail of gabions and fill (Ibrahim, 2018).

1.1.2.3. Cantilever retaining walls

They are the most common type of retaining walls. Cantilever walls (Figure 1.3) rest on a slab foundation which allows horizontal pressures from behind the wall to be converted to vertical pressures on the ground below. This slab foundation is also loaded by back-fill and thus the weight of the back-fill and surcharge also stabilizes the wall against overturning and sliding its stability is a function of strength of its individual parts. The very simple form of L or inverted T are suitable for low walls (less than 6 m), but for higher walls, it is necessary to introduce counterforts or buttresses.

(a) **(b)**

Figure 1. 3. Cantilever walls (a) Typical cantilever wall (Ibrahim, 2018) (b) Cross sections through typical (inverted) T-shaped and L-shaped reinforced concrete cantilever walls. (Clayton, et al, 2013)

1.1.2.4. Counterfort retaining walls

Counterfort walls (Figure 1.4) are similar to cantilever retaining walls, at regular intervals, however, they incorporate wing walls projecting upward from the heel of the footing into the stem. The counter-forts act as tension stiffeners and connect the wall slab and the base to reduce the bending and shearing stresses. They are economical when the wall height exceeds 8m. Whereas, if bracing is in front of the wall and is in compression instead of tension, the wall is called Buttress retaining wall.

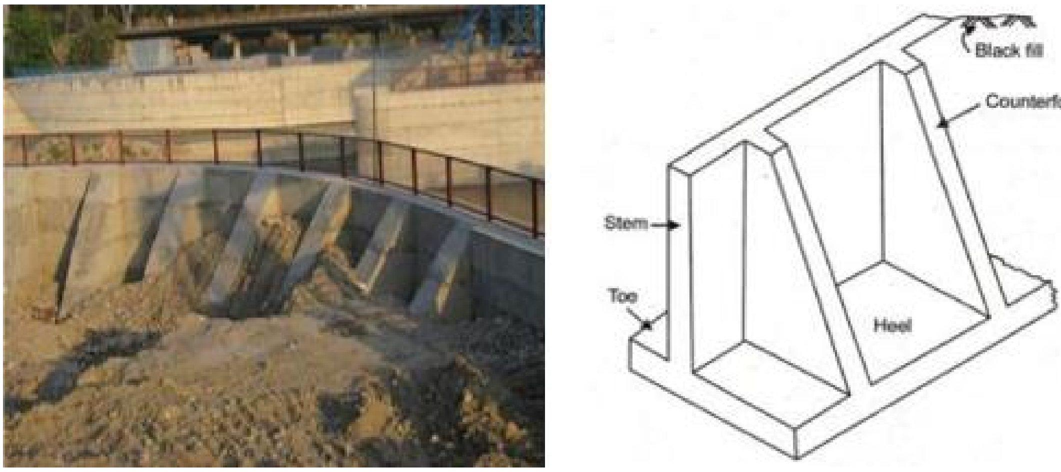
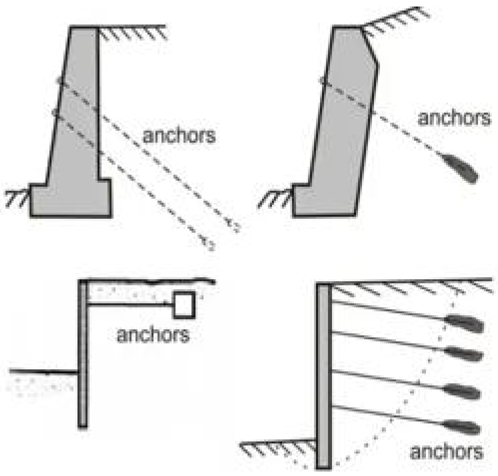


Figure 1.4. Counterfort wall(Ibrahim, 2018.)

1.1.2.5. Anchored wall

They are often used for higher walls where a cantilevered wall may not be economical. Restraint is achieved by drilling holes and grouting inclined steel rods as anchors into the zone of earth behind the wall beyond the theoretical failure plane in the backfill. The anchors can be placed at several tiers for higher walls, and can be post-tensioned rods grouted into drilled holes, or non-tensioned rods grouted into the drilled holes. The latter are also known as soil nails. Figure 1.5 illustrates typical anchored wall.



(a)

(b)

Figure 1.5. Anchored walls (a) Plan view (b) Example (theconstructor.org).

1.1.2.6. Bored piles

They are low-deformation retaining walls which are capable of transferring large vertical loads into the ground but also of supporting large horizontal loads due to their high flexural stiffness. Bored pile walls have the advantage that they can be constructed in almost any ground conditions. Bored pile walls are shown in Figure 1.6. Bored pile walls may be:

a. Intermittent

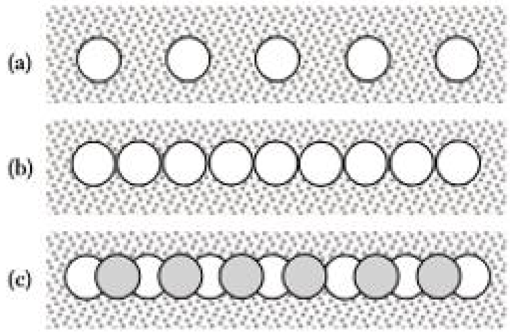
Large gaps between adjacent piles (spacing s > diameter d) are only feasible in over consolidated soils, or soils with some natural cementing, and where groundwater is below excavation level.

b. Contiguous

Because of construction tolerances, contiguous piles may be just in contact along their length or have a small gap between piles. This is not easy to guarantee in practice, so water tightness cannot be ensured. In very low permeability soils, however, seepage will be negligible.

c. Secant

Here spacing is less than the diameter. Primary concrete piles typically have no reinforcement, and in a 'hard-soft' configuration may be constructed of cement bentonite to control groundwater flow. The secondary piles are installed whilst primary pile concrete is still 'green' and not hardened. With this form of construction, seepage will be negligible, and the overall bending stiffness of the wall will be considerably increased. (Clayton et al., 2020)

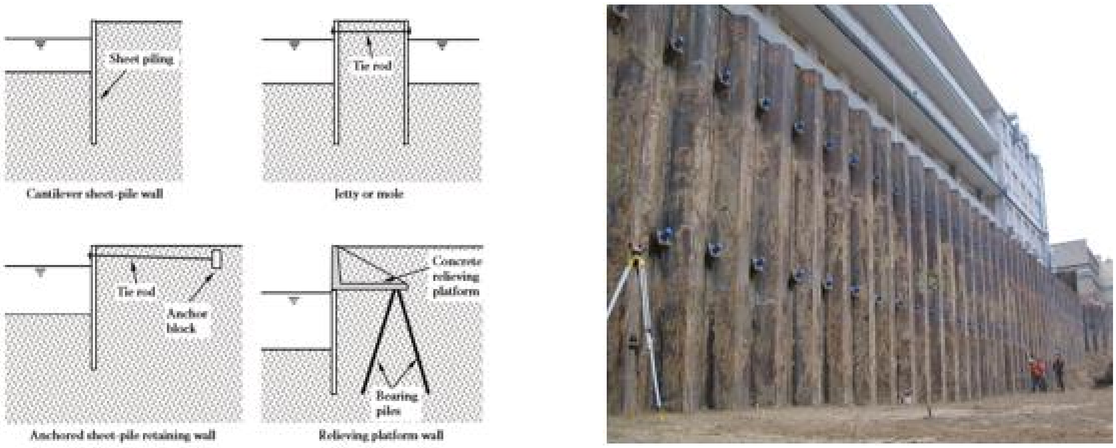


(a) **(b)**

Figure 1.6. Bored pile walls (A) Plan view through typical bored pile wall configurations: (a) intermittent, (b) contiguous, (c) secant. (Clayton et al., 2020) (B) Typical bored pile walls (theconstructor.org)

1.1.2.7. Sheet pile

Sheet-pile walling is widely used to construct flexible support systems, often for both large and small waterfront structures, or in temporary works. It is often used in unfavourable soil conditions (for example, soft clays) because no foundations are needed. Over the years, different materials have been used—steel, timber, and precast reinforced concrete (Clayton et al., 2020). Figure 1.7 shows some typical sheet pile walls.



(a) (b)

Figure 1.7. Sheet pile walls (a) Cross sections through some typical Permanent sheet-pile structures. (Clayton et al., 2020) (b) Typical sheet pile wall (www.constrofacilitator.com)

1.1.2.8. Reinforced earth retaining wall

Construction of a Reinforced Earth wall is straightforward and simple. Merely place a layer of facing panels, bolt on the reinforcing strips then backfill and compact. Repeat this cycle until the appropriate wall height has been reached. Properly compacted to a uniformly high density, the earth combines with the reinforcement to produce a strong, durable structure with predictable performance characteristics. Figure 1.8 illustrates a typical reinforced earth retaining wall.

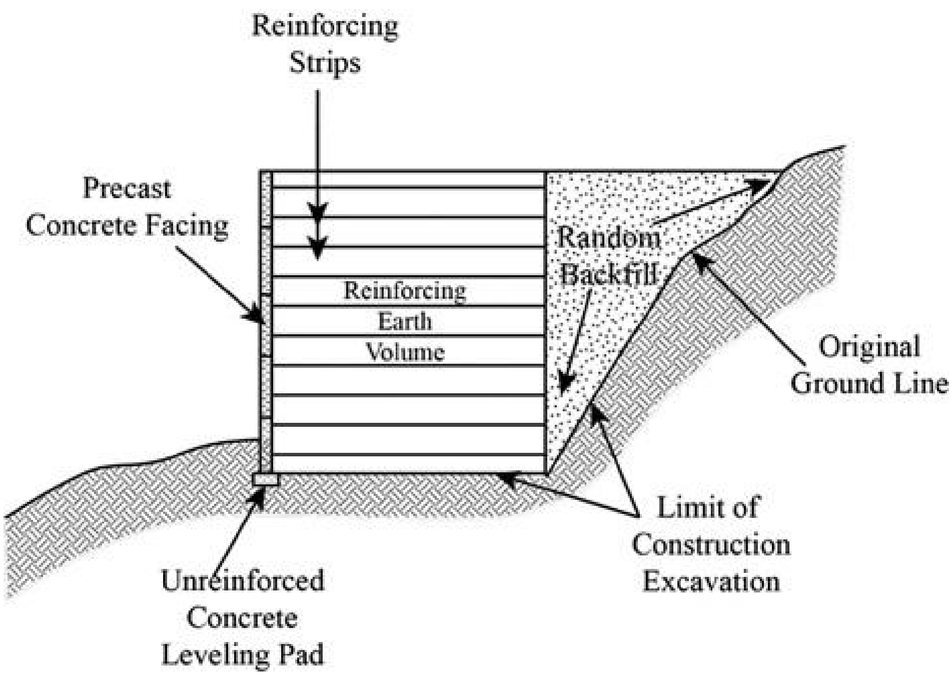


Figure 1. 8. Reinforced earth retaining wall (www.constrofacilitator.com)

1.1.3. Selection of earth retaining walls

Given the wide variety of retaining walls, selecting a type of retaining wall is a complex process, considering also the various geotechnical and non-geotechnical factors involved. It is important to assess the appropriateness of the available types of structure for the given application. The wall selection process should therefore include consideration of various factors which will be described below.

1.1.3.1. The characteristics of the site soils and bedrock

Some walls are better suited to a relatively shallow bedrock surface while others will perform just as well in a thick soil mantle. Soft soils may induce bearing capacity and settlement concerns. Very hard rock layers may make rock excavation (for particular types of walls) impractical. Chemical characteristics of the soil may dictate that specific materials be used to construct the retaining wall to mitigate corrosion concerns (Hoek and Bray, 1981).

1.1.3.2. Physical and spatial site constraints

The size and orientation of a site may dictate the type of wall that will be constructed; some walls require relatively large construction areas while others can be built in a relatively small area. The location of overhead or buried utilities may dictate what type of wall may be constructed relocating utilities can be dangerous, inconvenient, and expensive; it may be more cost-effective to choose a wall that can work around utilities rather than relocating them (Hoek and Bray, 1981).

1.1.3.3. Ground water regime

The presence of ground water makes the choice of walls whose construction is straight forward even where water is present preferable to others. Durability can be an issue in aggressive groundwater or marine environments. Life expectancy can be shortened, even with special preservative treatment especially when used in permanent structures above water level for a type of wall. However, the drainage of the soil retained behind certain types of walls can be improved by using permeable fill.

1.1.3.4. Height of ground to be supported

Certain walls are only viable for small retained heights. They can probably be designed for greater heights, but as the height increases, other types of wall become more economical.

1.1.3.5. Economic factors

Financial concerns often impose the majority of restrictions on a project not only does the owner wish to preserve the budget, but he or she would also like the best product they can have for the lowest price possible.

1.1.3.6. Adjacent structures

Considerations of the magnitude of the external loads and allowable movements are necessary. Flexible walls tolerate large wall movements without damage, follow complex plan shapes with ease, and cause minimal soil displacement. Where construction noise and vibration are relatively low, installation close to existing structures is favourable.

1.1.3.7. Aesthetic considerations

The look of a wall is especially important when located in an area where it will frequently be seen, for example, a wall supporting a hillside behind a building, above a road, near a home, or in a scenic area. In recent years, concrete has been used for many aesthetic touches walls can be made of coloured and/or stamped concrete, mimicking a stone wall or rock-cut.

The wall selection process may also include the consideration of other factors such as, speed of construction, environmental concerns, durability, maintenance, availability of construction techniques and equipment, availability of standards and codes of practice.

1.2. Design of retaining walls

The design of a retaining wall must account for all applied loads. The resistive force acting on a wall consists of a net vertical force acting on the wall (sum of the self-weight of the wall W, the weight of the backfill soil, and the surcharge load, minus the uplift pressures acting below the wall). The driving force acting on the wall is calculated as the summation of the net lateral earth pressures (active pressure minus passive pressure) and lateral pressures due to the surcharge load.

1.2.1. Lateral earth pressures

Earth pressures develop primarily as a result of loads induced by the weight of the backfill and/or retained in-situ soil, earthquake ground motions, and various surcharge loads. The magnitude and direction of these pressures as well as their distribution depend upon many variables; such as height of the wall, the slope of the ground surface (β), type of backfill used, draining of the backfill, level of the water table, added loads applied on the backfill (surcharges either live or dead loads), degree of soil compaction, and movement of the wall caused by the action of the backfill.

1.2.1.1. Lateral earth pressures from backfill

The design of earth retaining structures such as retaining walls requires a thorough knowledge of the lateral pressures that act between the retaining structures and the soil masses being retained. The purpose of a retaining wall is to retain soil and to resist the lateral pressure of the soil against the wall. For purposes of earth retaining system design, three different types of lateral earth pressure are usually considered: at-rest earth pressure, active earth pressure and passive earth pressure.

a. At-rest earth pressure

It is defined as the lateral earth pressure that exists in level ground for a condition of no lateral deformation. The at-rest earth pressure represents the lateral effective stress that exists in a natural soil in its undisturbed state as shown in Figure 1.9.

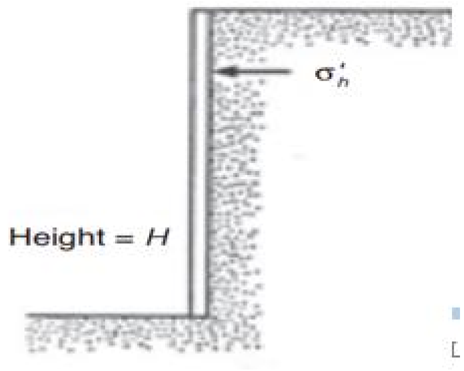


Figure 1. 9. At-rest earth pressure (Braja M., 2011)

- For walls constructed in near normally consolidated soils, the at rest earth pressure coefficient, Ko, can be approximated by the equation (Jaky, 1944) :

$$K_o = 1 - \sin\varphi' \tag{1. 1}$$

Where φ′ is the effective (drained) friction angle of the soil.

- In over consolidated soils, Ko can be estimated as (Schmidt, 1966) :

$$K_o = (1 - \sin\varphi')(OCR)^{\Omega} \tag{1. 2}$$

Where Ω is a dimensionless coefficient, which, for most soils, can be taken as sin φ′ (Mayne and Kulhawy, 1982) and OCR is the over consolidation ratio.

b. Active earth pressure

It is developed as the wall moves away from the backfill or the retained soil. This movement results in a decrease in lateral pressure relative to the at-rest condition. A relatively small amount of lateral movement is necessary to reach the active condition as shown in Figure 1. 10.

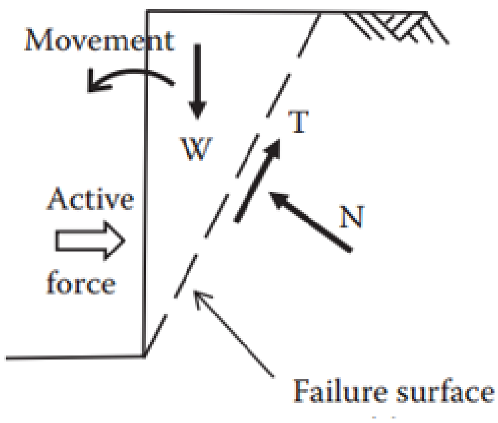


Figure 1. 10. Active earth pressure (Clayton, et al., 2013)

c. Passive earth pressure

It is developed as the wall moves towards the backfill or the retained soil. This movement results in an increase in lateral pressure relative to the at-rest condition. The movements required to reach the passive condition are approximately ten times greater than those required to develop active earth pressure as shown in Figure 1.11.

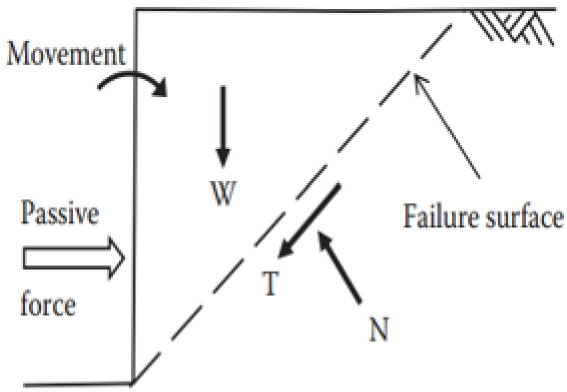


Figure 1. 11. Passive earth pressure (Clayton et al,. 2013)

1.2.1.2. Lateral earth pressure theories

The lateral earth pressure acting on the back of a wall is the driving force that can cause instability, such as sliding and rotation, of the wall. Thus, determination of the lateral earth pressures acting on a wall is important. There are two classical earth pressure theories namely Coulomb's (1776) earth pressure theory and Rankine's (1857) earth pressure theory. Both theories propose to estimate the magnitudes of two lateral earth pressures: active earth pressure and passive earth pressure.

a. Coulomb's Earth Pressure Theory

Coulomb observed that a wedge of soil formed behind the retaining structure when the lateral force became a minimum. He made these observations on walls with a planar back face. These conditions infer that the active state of stress develops behind the wall and the soil within the failure wedge is in a state of plastic equilibrium. The failure wedge is bounded by the back of the wall and a rupture surface through the backfill.

To obtain the magnitude of the lateral force, Coulomb assumed: The soil is homogeneous and isotropic; the rupture surface is a plane; the shear resistance is uniformly distributed along the rupture surface; the failure wedge acts as a rigid body; Friction is developed between the wall and the failure wedge and Plane strain applies.

The principal deficiencies in the Coulomb theory are the assumptions that the soil is ideal and that the rupture zone is a plane. Real soils are neither homogeneous nor isotropic. A plane rupture surface was assumed to simplify computations. This is considered to have a minor effect for the active case but can lead to large errors for the passive case. Cantilever retaining walls do not have the planar back face which is a basic condition in the derivation of Coulomb's theory.

The active earth pressure P_a acting on a wall is illustrated in Figure 1.12 and is given by:

$$P_a = \frac{1}{2}\gamma H^2 K_a \tag{1.3}$$

c'= 0 for cohesionless soils

Where K_a = the active earth pressure coefficient and is given by

$$K_a = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cos(\alpha+\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-i)}{\cos(\alpha+\delta)\cos(\alpha-i)}}\right]^2} \tag{1.4}$$

Where α = inclination (with respect to the vertical axis) of the back face of the wall, δ= friction between the wall and the backfill soil, and i = slope of the backfill soil.

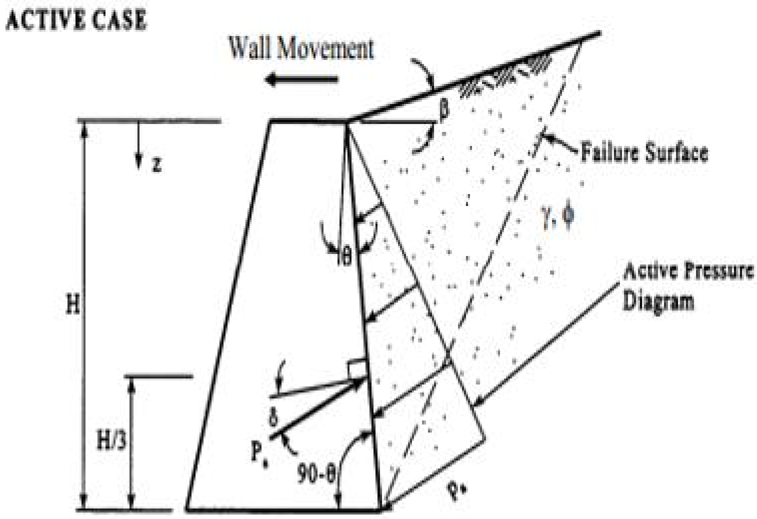


Figure 1. 12. Coulomb active pressure (Samtani and Nowatzki, 2012).

The passive pressure acting on a wall with cohesionless backfill is shown in Figure 1.13 and is given by:

$$P_p = \frac{1}{2}\gamma H^2 K_p \quad (1.\ 5)$$

Where Kp = the passive earth pressure coefficient, given by

$$K_p = \frac{\cos^2(\phi+\alpha)}{\cos^2 \alpha \cos(\alpha-\delta)\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+i)}{\cos(\alpha-\delta)\cos(\alpha-i)}}\right]^2} \quad (1.\ 6)$$

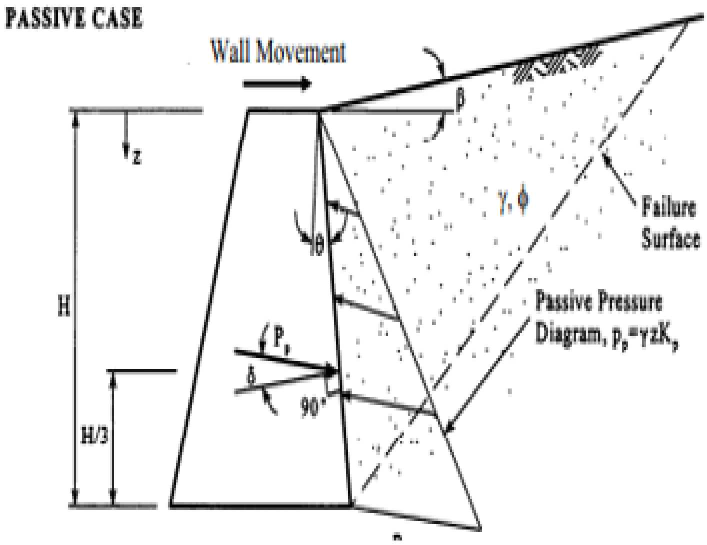


Figure 1. 13. Coulomb passive pressure (Samtani and Nowatzki, 2012)

For simple cases involving vertical walls retaining homogeneous soil with a level ground surface, without friction between the soil and the wall face, and without the presence of groundwater, the formulas for computing the earth pressure coefficients can be simplified considerably by substituting, $\delta = \theta = \beta = 0$ in Coulomb's equations. For such simplified cases, K_a and K_p can be expressed by Equations 1.7 and 1.8, respectively:

$$K_a = \frac{1-\sin\phi'}{1+\sin\phi'} \tag{1.7}$$

$$K_p = \frac{1+\sin\phi'}{1-\sin\phi'} \tag{1.8}$$

b. Rankine's Earth Pressure Theory

Rankine considered an infinitely long and deep cohesionless soil deposit with no external forces and examined the effects of laterally expanding or compressing the soil mass. Rankine's active state of stress occurs when the soil within the failure wedge is in a state of plastic equilibrium and the lateral earth pressure developed is a minimum.

The following assumptions are made in the Rankine theory: The soils homogeneous and isotropic, in a state of plastic equilibrium and possesses internal friction; The failure surface within the backfill is planar; The shear strength is mobilized uniformly on all planes throughout the backfill; The presence of the wall does not influence the state of stress in the backfill; The failure is a two-dimensional problem; The resultant P_a. is inclined at angles to the wall.

The main deficiency in Rankine's theory concerns the ideal soil which was assumed and is inconsistent with real soil deposits. The planar rupture surfaces are assumed to simplify computations.

According to Rankine,

The active earth pressure is illustrated in Figure 1.14 and is given by

$$P_a = \frac{1}{2}\gamma H^2 K_a \tag{1.9}$$

Where K_a is the active earth pressure coefficient, given by

$$K_a = \cos i \frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}} \tag{1.10}$$

The passive earth pressure is illustrated in Figure 1.15 and is given by

$$P_p = \frac{1}{2}\gamma H^2 K_p \tag{1.11}$$

Where K_p is the passive earth pressure coefficient, expressed as:

$$K_p = \cos i \frac{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}} \tag{1.12}$$

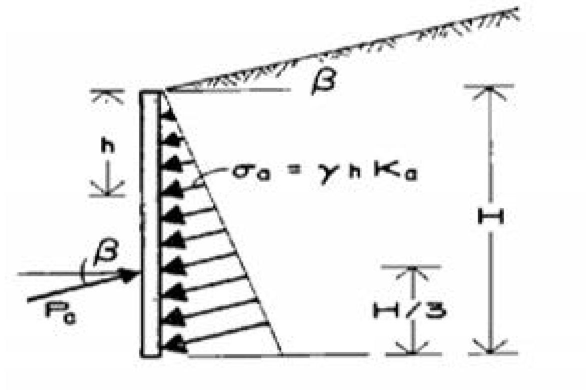


Figure 1. 14. Rankine active pressure (Samtani and Nowatzki, 2012)

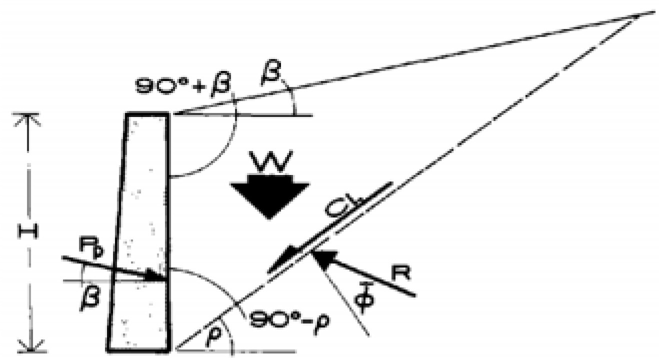


Figure 1. 15. Rankine passive pressure (Samtani and Nowatzki, 2012)

c. Earth Pressure Theory for Clayey Soil

The active earth pressure for a clayey soil is given by

$$P_a = \frac{1}{2}\gamma H^2 \frac{1}{N\phi} - 2c\frac{H}{\sqrt{N\phi}} \tag{1.13}$$

Where N_ϕ is given by $N_\phi = \tan^2(45 + \frac{\phi}{2})$ and c = cohesion of the soil. For soft soil, ϕ= 0 and $N\phi$ = 1. Therefore,

$$P_a = \frac{1}{2}\gamma H^2 - 2cH \tag{1.14}$$

The expression for the passive earth pressure in clayey soil is given by

$$P_p = \gamma H\, N_\phi + 2c\sqrt{N\phi} \tag{1.15}$$

1.2.1.3. Lateral earth pressures from surcharge loads

Surcharge loads on the backfill surface near a earth retaining structure also cause lateral pressures on the structure. Typical surcharge loadings may result from railroads, highways, sign/light structures, electric/telecommunications towers, buildings, construction equipment, and material stockpiles. The loading cases of particular interest in the determination of lateral pressures are: uniform surcharge, point loads, line loads parallel to the wall and strip loads parallel to the wall.

a. Uniform Surcharge Loads

Surcharge loads are vertical loads applied at the ground surface, which are assumed to result in a uniform increase in lateral pressure over the entire height of the wall. The increase in lateral pressure for a uniform surcharge loading can be written as:

$$\Delta p_h = K q_s \tag{1.16}$$

Where Δp_h is the increase in lateral earth pressure due to the vertical surcharge load, q_s applied at the ground surface, and K is an appropriate earth pressure coefficient.

b. Concentrated loads

Concentrated loads include Point loads, line loads, and strip loads. They are vertical surface loadings that are applied over limited areas as compared to surcharge loads. As a result, the increase in lateral earth pressure used for wall system design is not constant with depth as is the case for uniform surcharge loadings. These loadings are typically calculated by using equations based on elasticity theory for lateral stress distribution with depth (Figure 1.16.). Examples of such loads include heavy cranes (temporary) or walls (permanent). Lateral pressures resulting from these surcharges should be added explicitly to other lateral pressures.

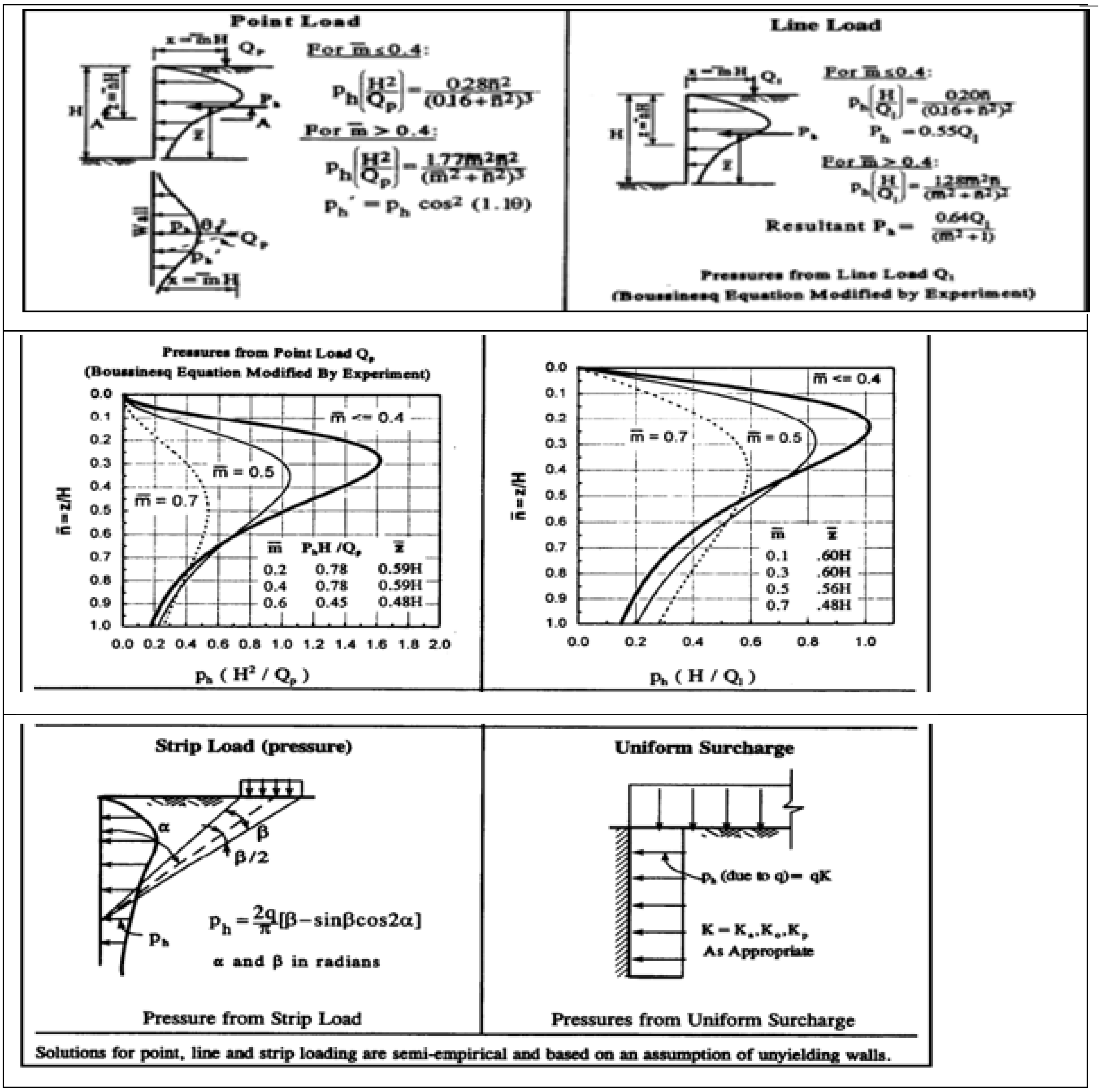


Figure 1.16. Point, line and strip loads (Samtani and Nowatzki, 2012)

1.2.1.4. Lateral pressures due to water

In retaining wall design, it is general practice to provide drainage paths, commonly known as "weep holes," through the earth retaining structure, or use other methods to drain groundwater that may otherwise collect behind the structure. The purpose of these drainage features is to prevent the development of water pressure on the structure. Occasionally, however, it may not be feasible or desirable to drain the water from behind the structure. For example, maintenance of existing ground water levels may be desirable to safeguard against potential settlement of adjacent structures or to prevent contaminated groundwater from entering the excavation. In such instances, the earth retaining structure must be designed for both lateral earth pressure and water pressure.

In this case, the water pressure represents a hydrostatic condition since there is no seepage or flow of water through the soil. The lateral earth pressure below the water level is based on the effective vertical stress, p'_o, times the active lateral earth pressure coefficient. The lateral pressure due to the water is added to the active lateral earth pressure to obtain the total lateral pressure on the wall. By analogy to lateral earth pressure coefficients, the lateral water pressure coefficient = 1.0. The lateral pressure computations should consider the greatest unbalanced water head anticipated to act on the wall, since this generally results in the largest total lateral load.

1.2.2. External Stability of retaining walls

A retaining wall must be stable as a whole, and it must have sufficient strength to resist the forces acting on it. There are four basic modes of instability from a geotechnical view relating to the soil/ structure interaction and the overall stability of the wall system. These are limiting eccentricity or overturning, sliding, bearing capacity and global stability. Since these modes of instability assume that the wall is intact, the evaluation of these modes is commonly referred to as the "external stability" analysis.

1.2.2.1. Overturning check

The primary force causing overturning is the lateral earth pressure against the wall. It occurs because of unbalanced moment (Figure 1.17), when overturning moment about toe to due lateral pressure is greater than the resisting moment of self-weight of wall and weight of stabilising soil.

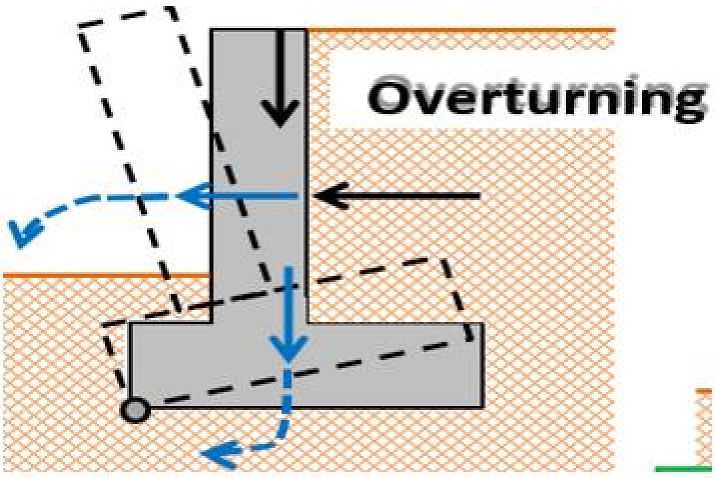


Figure 1.17. Overturning failure (Ibrahim, 2018)

The critical condition occurs when maximum horizontal force acts with minimum vertical load. The factor of safety against overturning is the ratio of the moments which resist overturning to the moments which cause overturning.

Check for Overturning about Toe (point O)

$$SF_{overturning} = \frac{Resisting\ moments}{Overturning\ moments} = \frac{\sum M_R}{\sum M_o} \tag{1. 17}$$

≥1.5 for cohesion less soils or ≥2.0 for cohesive soils

1.2.2.2. Sliding check

A sliding failure can occur along the contact surface of the bottom of the footing and the foundation soil, or by shear along a surface through the soil beneath the footing.

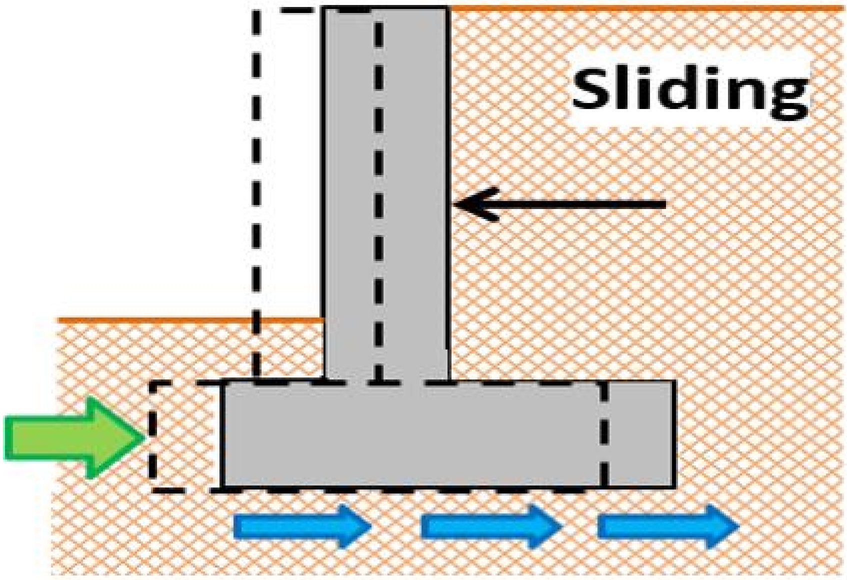


Figure 1. 18. Sliding Failure (Ibrahim, 2018)

The lateral pressures tend to push the retaining wall away from the higher backfill (Figure 1.18). Resistance against sliding is provided by friction between the bottom surface of the base slab and soil beneath. The resistance provided by passive earth pressure on the front face of the base gives some contribution which is often ignored because the backfill may be placed behind the wall before the fill is placed in front of the wall. The resisting force is a function of the shear strength of the foundation soil. If the resistance to sliding is not sufficient to offset the driving forces, the footing size may be increased, or a key is added to the footing. This may increase the length of the shear surface and passive resistance and therefore, the magnitude of the resisting force.

Check for Sliding along the Base of the Wall:

$$\mathrm{SF}_{\mathrm{sliding}} = \frac{Resisting\ Forces}{Sliding\ Forces} = \frac{\sum F_R}{F_S} \tag{1. 18}$$

≥ 1.5 for Cohesion less soils or ≥ 2.0 for cohesive soils.

1.2.2.3. Bearing capacity

The footing should be sized to ensure that the maximum contact pressure does not exceed the bearing capacity of the soil.

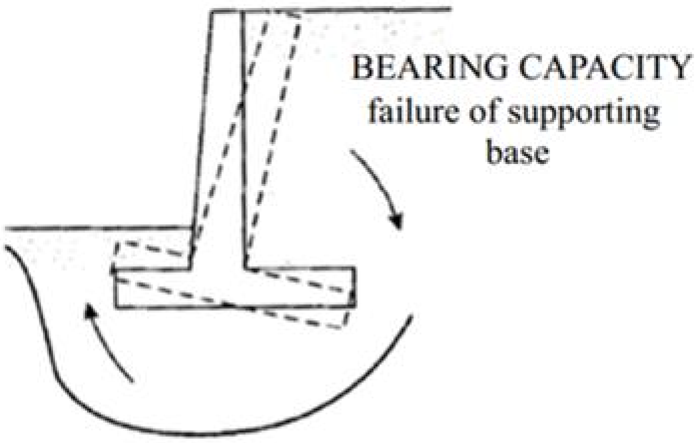


Figure 1.19. Bearing capacity failure (Erizal and Magr, 2017)

The width of the base slab must be adequate to distribute the vertical force to the foundation soil. To determine the required size of base, bearing pressure underneath is assessed on the basis of the ultimate limit state (GEO). Since the base slab of the wall is subjected to the combined effects of an eccentric vertical coupled with an overturning moment, the analysis is similar to that of foundation design. If these settlement problems cannot be overcome by adjusting the size of the footing, pile supports may be necessary. Figure 1.19 illustrate a bearing capacity failure.

Check for Bearing Capacity Failure of the Base Soil:

$$SF_{Bearing\ capacity} = \frac{Net\ ultimate\ bearing\ capacity}{Maximum\ bearing\ pressure} = \frac{q_{lim}}{q_{actual}} \geq 2 \tag{1. 19}$$

The net ultimate bearing capacity of the base soil can be calculated from Hansen's equation,

$$q_{lim} = \frac{1}{2}\gamma' B' N_\gamma\, s_\gamma\, i_\gamma\, b_\gamma\, g_\gamma + c' N_c\, s_c\, d_c\, i_c\, b_c\, g_c + q_0 N_q\, s_q\, d_q\, i_q\, b_q\, g_q \tag{1. 20}$$

Where: s coefficients: shape of the foundation d coefficients: contribution of the soil above the foundation surface i coefficients: inclination of the load b coefficients: inclination of the foundation base g coefficients: inclination of the ground surface

c = cohesion of the base soil

q′ = surcharge load or overburden pressure for shallow side γ = unit weight of the base soil

B′ = retaining wall effective base width

1.2.2.4. Global Stability check

Where retaining walls are underlain by inadequate foundation materials, the overall stability of the soil mass must be checked with respect to the most critical failure surface. As shown in Figure 1.20, both circular and non-circular slip surfaces must be considered. A minimum factor of safety of 1.5 is desirable. Slope failures can be due either to a sudden or gradual loss of soil strength, or to an increase in stress (overloading, removal of the toe stop, deforestation, earthquake), or to a change in the mechanical (loss of resistance through reshaping) or hydraulic characteristics (appearance of runoff, snowmelt) of the field (Khemissa, 2006). With this type of failure the walls remain intact but the soil mass slips and rotates as a bowl shaped mass. If global stability is found to be a problem, deep foundations or the use of lightweight backfill may be considered. Alternatively, measures can be taken to improve the shear strength of the weak soil stratum. Other wall types, such as an anchored soldier pile and lagging wall or tangent or secant pile wall, should also be considered in this case.

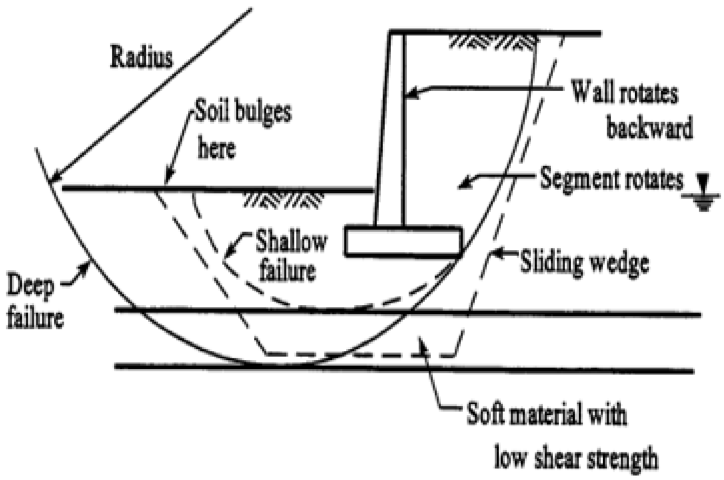


Figure 1. 20. Overall stability (Samtani and Nowatzki, 2006)

Several methods of analysis of slope have been developed including limit equilibrium method and finite element method. All the different methods of slope analysis using limit equilibrium (LE) are based on an arbitrary selection of a series of sliding surfaces and the definition of the one that gives the minimum value of the coefficient safety. The method of slices is a versatile and powerful tool under the category of limit equilibrium analysis method for dealing with slopes with an irregular slip surface and in non-homogeneous soils.

1.2.3. Internal stability of retaining walls

The internal stability of a retaining structure refers to the integrity of the structural elements of the wall. The wall has to be designed such that it resists to bending moments and shear through the use of steel reinforcement. The design of bending and shear reinforcement is based on an analysis of the loads for the ultimate limit state (STR), with the corresponding bearing pressures. Gravity walls will seldom require bending or shear steel, while the walls in counterfort and cantilever construction will be designed as slabs. The design of counterforts will generally be similar to that of a cantilever beam unless they are massive.

1.2.4. Methods of design of retaining walls

The constructions designed by civil engineers must satisfy a number of requisites or conditions. Such requisites have been condensed into stability (safety against failure or structural collapse), serviceability (the capacity to fulfil the design purpose without significant constraints for the users) and durability. The methods employed to satisfy the above-mentioned criteria are described in this section.

1.2.4.1. Limit equilibrium method (LEM)

This method compares the overall resistance of a structure, with respect to a possible failure mechanism, with the resultant of actions inducing the failure mechanism (i.e., wall design): the ratio between the two resultants is the global safety factor. Limit equilibrium method was first introduced by Coulomb to calculate the force of a fill on a retaining wall. Later the theory was extended to infinite body by Rankine and earth pressure theories were developed. Subsequent developments were made by Fellenius, Terzaghi and others that results in making the limit equilibrium method a well utilized tool for stability calculations by practicing engineers. An arbitrary mechanism of collapse can be constructed with limit equilibrium method. Each element of mechanism should be in equilibrium, the whole mechanism is in equilibrium is the main assumption of the limit equilibrium approach.

1.2.4.2. Limit state design (LSD)

A structure designed by LSD is proportioned to sustain all actions likely to occur during its design life, and to remain fit for use, with an appropriate level of reliability for each limit state.

a. General considerations

The structural codes of practice for concrete, steel, and other structural materials started to adopt limit state design (LSD) as the basic philosophy from the 1960s and 1970s. This method uses partial safety factors and is based on considerations of a probabilistic type. More recently, especially since the 1990s, geotechnical design codes have been progressively adapted to the limit state design philosophy. This occurred in Europe, with the approval in 1994 of Eurocode 7 as a European Pre-norm (ENV 1997-1:1994), but also in the United States, Canada and Japan (Ovesen and Orr, 1991; Becker, 1996ab; Coduto, 2001; Honjo and Kusakabe, 2002).

This evolution of the codes applied in geotechnics results from the obvious advantage inherent in the adoption of the "same language" and coherent design concepts by both structural and geotechnical engineers in the design teams of civil engineering structures, preventing misunderstandings that may cause more or less serious errors. Moreover, a common background philosophy for the design of structures, whichever they may be, will also facilitate the teaching of civil engineering. A limit state can be defined as a state beyond which the structure no longer satisfies, to some manner, the functions for which it was designed. Within the limit states, there are ultimate limit states and serviceability limit states, (Fernandes, Manuel Matos, 2020).

b. Ultimate limit state

It is a state at which a failure mechanism can form in the ground or in the retaining wall, or severe structural damage (e.g., yielding or rupture) occurs in principle structural elements. For each possible ULS the relation E_d (Design Value for the action effect) $\leq R_d$ (Design Value for the resistance) must be verified with different possible combinations (sets) of partial safety.

c. Serviceability limit state

Serviceability limit states include strains or movements in a retaining wall which would render the wall unsightly, result in unforeseen maintenance or shorten its expected life. They also include deformations in the ground which are of concern, or which affect the serviceability of any adjacent structures or services. The permissible movements to be specified in a design should be based on a consideration of the tolerable movement of nearby structures, services and land. More stringent requirements may be necessary where structures or services are present behind the wall. To satisfy the serviceability limit state, the effect of the applied actions, E_A (also called an **action effect**) must be less than or equal to a limiting value of the action effect, C_A (i.e., a limiting settlement). This may be expressed as $E_A < C_A$.

In verifications of serviceability limit states, partial factors are normally set to 1.

d. EUROCODE

The European Union started the set-up of European Codes (EN or Eurocodes) in 1975 and after several steps, Eurocodes are published in 2002-2005.

i. Basic features of Eurocodes

Eurocodes introduced a new approach to design defining:

- ❖ **Categories of geotechnical structures** with a different level of prescription according with the level of importance
- **Category 1**: Civil structure relatively simple and small sizes. Their design can be performed on the base of experience and qualitative geotechnical survey because their collapse complains small risk for people.
- **Category 2**: Ordinary structures
- **Category 3**: Special structures for size or effects related to their collapse. Their design requires special care and more restrictive limitations.
- ❖ **Ultimate Limit State ULS** indicated according with the structure type.
- ❖ **Serviceability Limit States SLS** indicated the limit for the deformation or settlements of a structure in order to not lose its serviceability.
- ❖ **Criteria for selection of geotechnical parameters for design** according with semi-probabilistic approach.
- ❖ **Partial safety factors** which can be different in various countries and varied in accord to the considered SLU and the approach adopted for the analysis.

ii. List of EUROCODES

The Eurocodes for Structural Design are 10 (labelled as EC0, EC1… EC9). Table 1.1 presents the complete list of Structural Eurocodes currently applicable.

Table 1. 1. List of Structural Eurocodes – European Norms (EN) (Fernandes, Manuel Matos, 2020)

EN 1990	Eurocode 0: Basis of Structural Design
EN 1991	Eurocode 1:Actions on Structures
EN 1992	Eurocode 2: Design of Concrete Structures
EN 1993	Eurocode 3: Design of Steel Structures
EN 1994	Eurocode 4: Design of Composite Steel and Concrete Structures
EN 1995	Eurocode 5: Design of Timber Structures
EN 1996	Eurocode 6: Design of Masonry Structures
EN 1997	Eurocode 7: Geotechnical Design
EN 1998	Eurocode 8: Design of Structures for Earthquake Resistance
EN 1999	Eurocode 9: Design of Aluminium Structures

For the geotechnical problems three Eurocodes are important:

❖ **Eurocode 0** – Basis of Structural Design

❖ **Eurocode 7** – Geotechnical design:

- EN 1997-1:2004 Part 1: General rules
- EN 1997-1:2004 Part 2: Ground investigation and testing

❖ **Eurocode 8** – Seismic design:

- EN 1998-1 (2004) Part 1: General rules, seismic actions and rule for buildings
- EN 1998-1 (2004) Part 5: Foundations, retaining structures and geotechnical aspects

For the design of steel reinforcement in concrete structures, Eurocode 2 is used.

iii. Partial factor of safety

In order to take into account the uncertainties associated with the determination of parameters involved in the design, partial (safety) factors are used to modify the three terms (actions, material properties, resistance) to give the design. A characteristic value which has been modified by a partial factor is known as design value.

The Design Value for the action effect, E_d at ULS is given by:

$$E_d = E\left\{ \gamma_F F_{rep}, \frac{X_k}{\gamma_M}, a_d \right\} \text{ or } E_d = \gamma_E E F_{rep}\left\{, \frac{X_k}{\gamma_M}, a_d \right\} \quad (1.21)$$

The design value for the resistance, R_d at ULS is given by:

$$R_d = R\left\{ \gamma_F F_{rep}, \frac{X_k}{\gamma_M}, a_d \right\} \text{ or } R_d = \frac{R}{\gamma_R}\left\{ \gamma_F F_{rep}, X_k \ , a_d \right\} \text{ or } R_d = \frac{R}{\gamma_R}\left\{ \gamma_F F_{rep}, \frac{X_k}{\gamma_M} \ , a_d \right\} \quad (1.22)$$

Where:

a_d: design value of element size

F_{rep}: Representative value of an action

X_k: Characteristic value of a material property

γ_E: Partial factor for the action effect

γ_F: Partial factor for a action

γ_M: Partial factor for a material property

γ_R: Partial factor for the system resistance

For serviceability limit state checks, all values of γ_F and γ_M should be set to unity. An exception is that the value of γ_F should be set to zero for those surcharge loads which produce a favourable effect, e.g., surcharge in the area between the wall stem of a R.C. L-shaped retaining wall and its virtual back.

In EC7, three (four) possible design approaches are proposed:

a- Design Approach 1 (DA1)- combination 1: A1+M1+R1

b- Design Approach 1 (DA1)- combination 2: A2+M2+R1

c- Design Approach 2 (DA2): A1+M1+R2

d- Design Approach 3 (DA3): A2+M2+R3

Table 1. 2. Partial factors on actions or effects of actions

Action		Symbol	Set	
			A1	A2
Permanent	Unfavourable	γ_G	1,35	1,0
	Favourable		1,0	1,0
Variable	Unfavourable	γ_Q	1,5	1,3
	Favourable		0	0

Table 1. 3. Partial factors resistance factors

Resistance	Symbol	Set		
		R1	R2	R3
Bearing	γ_{Rv}	1,0	1,4	1,0
Sliding	γ_{Rh}	1,0	1,1	1,0

Table 1. 4. Partial factors for materials

Soil parameter	Symbol	Value	
		M1	M2
Shearing resistance	$\gamma_{\varphi'}$ [1]	1,0	1,25
Effective cohesion	$\gamma_{c'}$	1,0	1,25
Undrained strength	γ_{cu}	1,0	1,4
Unconfined strength	γ_{qu}	1,0	1,4
Effective cohesion	$\gamma_{c'}$	1,0	1,4
Weight density	γ_{γ}	1,0	1,0
[1] This factor is applied to tan φ'			

1.2.4.3. Finite element method (FEM)

The method divides the continuous space subjected to analysis in small parts, i.e. the Finite Elements (the assemble of Finite Elements is also called the mesh), and solves the differential equations governing a particular phenomenon (mechanical problem, flow in a river, seepage, coupled analysis, etc.) by integrating in a numerical way the equations written in a discretized form for all the elements. The various FEM codes differ for:

- The type of phenomena taken into account (deformability only, deformability + failure, deformability + failure + seepage);

- The constitutive model adopted (linear elastic, rigid and perfect plastic, no linear elastic, hardening material, elasto-plastic within Critical state model, constant or variable permeability, partially saturated soil, etc.);

- The dimensions of the studied problems (2D or 3D problems, plane strain or axial-symmetrical strain condition, etc.).

The FEM takes into account the media deformability, requiring the respect of variable continuity at the contact surface among elements.

The FEM procedure is as follows:

- Subdivision of soil continuum in Finite Elements (meshing);
- Define the properties of materials according to perfect-plastic or elasto-plastic model;
- Calculation of initial stress state assuming elastic behaviour and applying a gravitational load to the nodes of elements.

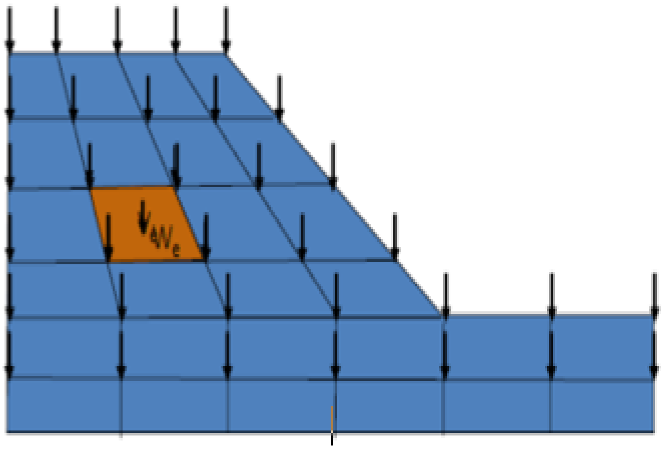


Figure 1. 21. Illustration of meshing in FEM

Conclusion

This chapter focused on explaining how retaining walls can be classified, giving the different common types of retaining walls and describing the selection criteria of retaining walls. It was seen that the characteristics of the site soils and bedrock, physical and spatial site constraints, the ground water regime, the height of ground to be supported, economic factors, adjacent structures and aesthetic considerations affect the selection of a specific type of retaining wall for a given construction. The three groups into which retaining walls can be classified were looked at. Different types of retaining walls including Gravity retaining walls, Counterfort retaining walls, Anchored wall, Bored piles, Sheet pile and cantilever wall were also described in this chapter. Forces acting on the retaining wall which include lateral pressures from backfill and surcharge loads made up part of this chapter. Different methods of the design of retaining wall were explained.

Chapter 2. METHODOLOGY

Introduction

The previous chapter enabled us to understand the types and uses of retaining walls, the failures to which retaining walls are subjected and the different methods for their design. This chapter will therefore unfold chronologically the design process up till the end so as to achieve the set objectives. So we are going to follow the step by step design procedure starting from general site recognition, and overview of the area and site presentation. The next part will present the necessary data collected, both geometric and geotechnical data. Subsequently, the software and simulation procedure used for the numerical simulation of the retaining wall is presented. Finally, the structural design with analytical methods is presented.

2.1. General recognition of the site

The site recognition was done through documentary research to define the physical characteristics of the site (climate, relief, geology, seismicity and hydrology) and the socioeconomic characteristics.

2.2. Data collection

The data collection consists of obtaining the different characteristics of materials through geotechnical tests (in-situ tests or laboratory tests). Geometric data and geotechnical data of the retaining wall will enable its design by numerical methods later described in this chapter.

2.2.1. Geometric data

The geometric data included the height of the retaining wall, slope of embankment and surcharge loads.

2.2.2. Geotechnical data

The geotechnical data were acquired from project documents. These geotechnical data included soil strength parameters like friction angle, cohesion, unit weight, water content and coefficient of lateral earth pressure obtained from the geotechnical investigations including a number of tests (both in-situ and in the laboratory) carried out by the geotechnical consultants. The tests' descriptions are presented in the table 2.1.

Table 2.1. Description of geotechnical tests

Tests	Objectives
Standard tests with a heavy dynamic penetrometer (SPDL) to a depth of fifteen meters or by refusal	Distinguish the different soil horizons; Detect the presence of anomalies; Determine the position of the roof of a resistant layer; Draft of foundation sizing.
Menard pressure meter survey (SPM)carried out to a depth of fifteen (15) meters or by refusal	Deep soil cuts Dimensioning of foundations (admissible constraints and settlements)
Sampling of intact samples on the slope	Collection of intact soil samples for analysis and laboratory testing
Laboratory test: natural water content; granular analysis; specific weight, Atterberg limits; density -straight shear tests	Calculation of the safety coefficients of the slope Pre-dimensioning of the retaining structure

2.3. Numerical analysis

In order to design the retaining wall, arbitrary dimensions are chosen and analyses performed on Plaxis 2d v20 to verify if the wall remains stable under the loads applied to it. The main checks are the verification of Global stability at ULS and checks of the vertical and horizontal displacements of the wall subjected to lateral pressures and surcharge loads at SLS.

2.3.1. Presentation of the modelling software: PLAXIS 2D CONNECT 20

PLAXIS is a finite element program developed for the analysis of deformations, stability and ground water flow in geotechnical engineering. We are going to use PLAXIS 2D which is a high-capacity version and can analyse a large range of geotechnical problems. It is possible to use extensive 2D finite element mesh. The software helps to simulate the soil behaviour and its accuracy depends on the expertise of the users regarding the modelling of the problem, understanding and limitations of the soil models, model parameters and the ability to judge the output (Brinkgreve et al., 2014b). The triangular finite elements which when added up represents the behaviour of the whole structure.

The Plaxis program is divided into 4 major sections. The input, calculation, output and curve. The first part of Plaxis has to do with the input program where we create our model and this model could be plain strain or axisymmetric. Elements created in the input program could be created using 6-node or 15-node triangle elements to model the soil layers but 15-node triangle is most accurate. Still on the input program after choosing the model and the number of nodes, we now create our geometry using points, lines after which the program generates the cluster in which we insert our loads and boundary conditions followed by the properties of the materials used and finally we generate our mesh. Calculation program can now proceed after the generation of the finite element model, the actual elements can be executed with the software and the calculation type chosen which could be plastic, consolidation or safety analysis (phi-c reduction). Last but not the list step is the output program that contains all the facilities to view and list the results of generated input data and finite element calculation. Figure 2.1 shows the interface of the modelling software PLAXIS 2D.

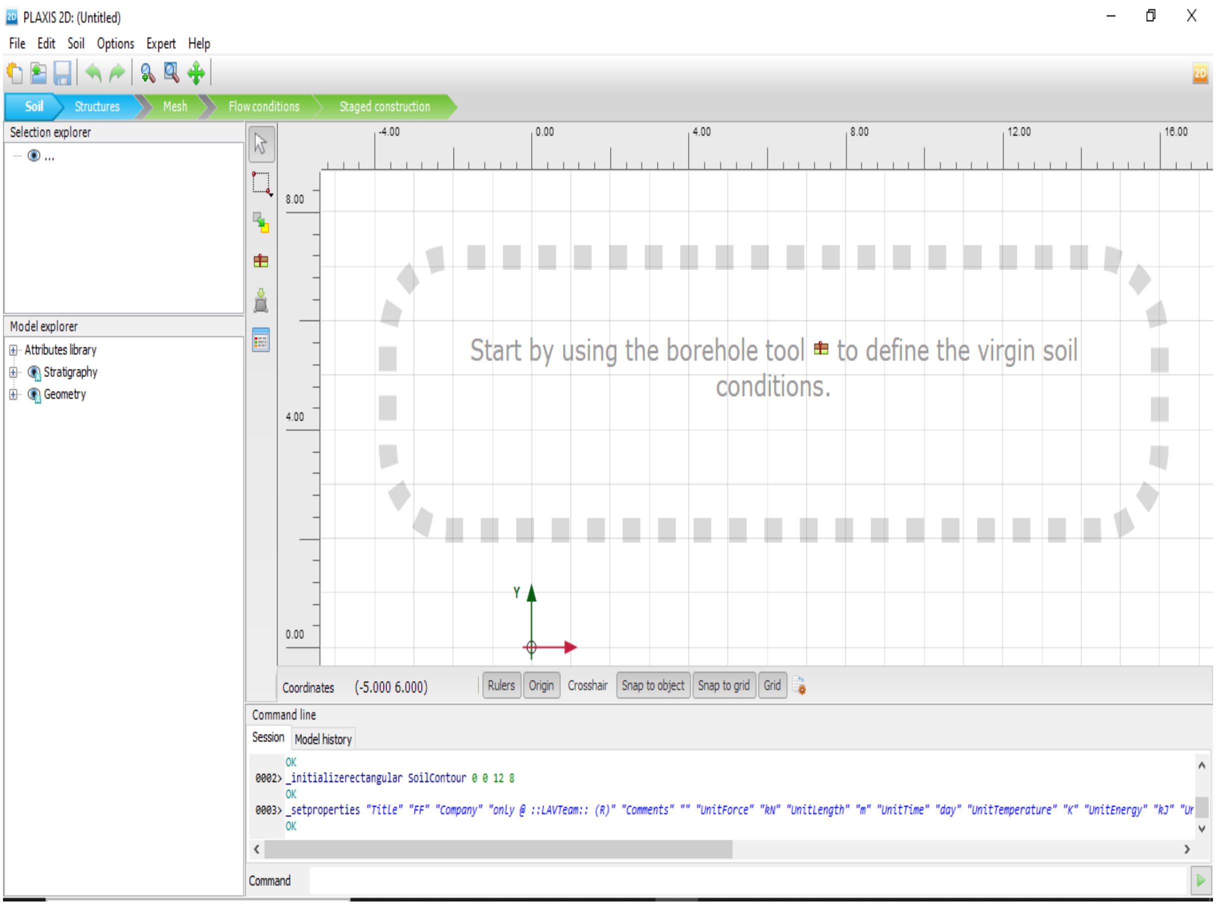


Figure 2. 1. Interface of PLAXIS 2D

2.3.2. Stability conditions in the numerical analysis

The code used to check for the stability of the retaining wall is Eurocode 7.

According to this code, the maximum allowable horizontal movement of the wall is 0.5%H, where H is the depth of the retained soil.

The maximum allowable settlement of the retaining wall is 50mm.

The minimum desirable safety factor for global stability is 1.5.

2.4. Structural design of the retaining wall

The structural design permits us to determine the reinforcement area of the retaining wall. We will focus on the design at ULS using EC 2. The bending moments, shear and axial forces of the retaining wall are obtained from the software PLAXIS. The wall is designed as three separate elements including two slabs and a side wall to determine moment and shear reinforcements. Each part is designed for one meter width and the steel reinforcement are distributed throughout the total width of the wall. The predesign is made by verifying the stability of arbitrary dimensions on PLAXIS.

2.4.1. Forces on retaining wall

To obtain the solicitations on the wall, a second analysis is performed on PLAXIS 2D using Design approach 1 combination 1 to design the wall at ULS. This design approach factors out actions and is the most unfavourable condition for this project. The partial factors for this design approach are shown on table 2.2.

Table 2.2. Partial factors for design approach 1 combination 1

Parameter		Symbol	DA1-1		
			A1	M1	R1
Permanent action (G)	Unfavourable	γ_G	1.35		
	Favourable	$(\gamma_{G,fav})$	1.0		
Variable action (Q)	Unfavourable	γ_Q	1.5		
	Favourable	-	(0)		
Shearing resistance (tan φ)		γ_φ			
Effective cohesion (c')		γ_c			
Undrained shear strength (c_u)		γ_{cu}		1.0	
Unconfined compressive strength (q_u)		γ_{qu}			
Weight density (γ)		γ_γ			
Bearing resistance (R_v)		γ_{Rv}			
Sliding resistance (R_h)		γ_{Rh}			1.0
Earth resistance (R_e)		γ_{Re}			
Factors given for persistent and transient design situations					

The following considerations are made in the design:

Table 2.3. Load types on structure

Load	Description
Surcharge loads from new building	permanent unfavourable
Surcharge loads from existing building	Permanent unfavourable
Driving load of micropiles	Variable unfavourable
Material partial factors are applied on the retained soil which accounts for lateral pressures on the wall	

2.4.2. Design of the top and bottom slabs

The slabs are considered as one-way slabs carrying uniform loads on the assumption that they consist of a series of rectangular beams 1 m wide spanning between supporting beams or walls. The continuous one-way slab is shown in Figure 2.2.

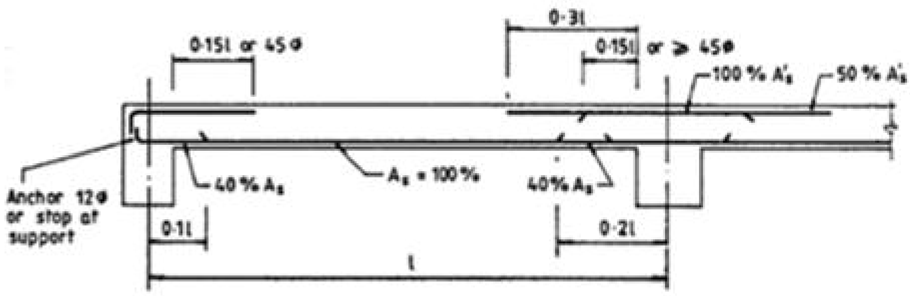


Figure 2. 2. Continuous one-way slab (BS-EN1992-1-1_E_2004)

The design process of slabs begins with the determination of concrete cover, then followed by the evaluation of the corresponding verifications at ultimate and serviceability limit states using the M-T-N (bending moment, shear force and axial force) diagrams obtained from PLAXIS.

2.4.2.1. Determination of concrete cover

To ensure the required design working life of the structure, it is necessary to protect each structural element against the environmental action. For concrete structures, Eurocode 2 ensured this protection by the definition of a concrete cover taking into account the structural class of the structure and the exposure class. This concrete cover is defined as the distance between the surface of the reinforcement closest to the nearest concrete surface and the nearest concrete surface as shown in the figure 2.3.

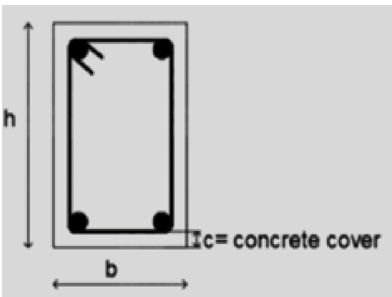


Figure 2. 3. Concrete cover

It is the nominal concrete cover, c_{nom} distance between the surface of the reinforcement closest to the nearest concrete surface. In the design, the nominal value of the concrete cover is defined as a minimum cover, C_{min}, plus an allowance for any probable deviation. The nominal cover, C_{nom}, is defined in equation 2.1.

$$C_{nom} = C_{min} + \Delta C_{dev} \tag{2. 1}$$

Where:

ΔC_{dev}= 10mm

$C_{min} = \max\{C_{min,}\ ; C_{min,\ dur};\ 10mm\}$

$C_{min,}$: is the minimum cover due to bond requirements, equal to the diameter of the bars or the equivalent diameter in the case of bundled bars;

$Cmin, dur$: the minimum cover due to environmental conditions obtain from table 2.4 of as a function of the exposure and the structural class of the building.

Table 2.4. Values of minimum cover

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

2.4.2.2. Moment reinforcement of top and bottom slabs

The flexural reinforcement of the top and bottom slabs is designed at the critical sections, that is, for the maximum positive and negative moments. The maximum bending moments per unit meter are obtained from the software PLAXIS. The design is done using formulae from EUROCODE 2. The steel reinforcement is computed for a rectangular section with the height h, the width b and the effective depth d as shown in the figure 2.4.

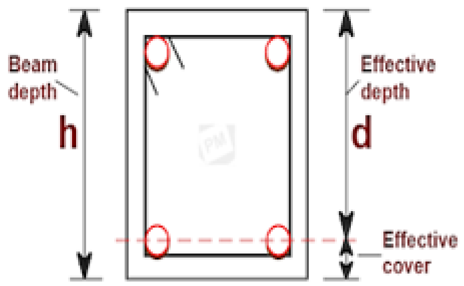


Figure 2. 4. Example of transversal slab section with longitudinal reinforcement

The procedure for the design of moment reinforcement is described below:

The effective depth is the length of the section minus the concrete cover.

$$d = h - c_c \tag{2.2}$$

The position of the neutral axis, X_{lim} and the limiting moment $M_{Rd,lim}$ are calculated from equation 2.3 and 2.4.

$$x_{lim} = \frac{\varepsilon_{cu}}{\varepsilon_{cu}+\varepsilon_{yd}}.d \qquad (2.3)$$

Where:

$\varepsilon_{yd} = f_{yd}/E_s$

ε_{cu} is obtained from table 2.5.

Table 2. 5. Values of ε_{cu} obtained from concrete class

Grade	ε_{cu}	ε_{c2}	ε_{c3}	n	λ	η
≤ C50	0.0035	0.0020	0.00175	2.0	0.800	1.000
C55	0.0031	0.0022	0.00180	1.75	0.788	0.975
C60	0.0029	0.0023	0.00190	1.60	0.775	0.950
C70	0.0027	0.0024	0.00200	1.45	0.750	0.900
C80	0.0026	0.0025	0.00220	1.40	0.725	0.850
C90	0.0026	0.0026	0.00230	1.40	0.700	0.800

$$M_{Rd,lim} = F_c.z_{lim} \qquad (2.4)$$

Where:

F_c is the resultant of compression is stresses given as:

$$F_c = \beta_1.b.x_{lim}.f_{cd} \qquad (2.5)$$

z_{lim} is the lever arm taken from the neutral axis given by the expression:

$$z_{lim} = (d - \beta_2.x_{lim}) \qquad (2.6)$$

With $f_{cd} = \frac{0.85 f_{ck}}{\gamma_c}$, γ_c = 1.5, β_1 and $\beta_2$are obtained from table.

Table 2. 6. Values of β_1 and β_2 according to concrete class

f_{ck} (N/mm²)	up to 50	55	60	70	80	90
β_1	0,80000	0,76781	0,73625	0,67500	0,61625	0,56000
β_2	0,40000	0,39375	0,38750	0,37500	0,36250	0,35000

If M_{Ed} is smaller than $M_{Rd,lim}$, that is; $M_{Ed} < MRd,lim$, the tension reinforcement As alone is needed.

The depth of the neutral axis, x is calculated as:

$$x = \frac{d}{2\beta_2} - \sqrt{\left(\frac{d}{2.\beta_2}\right)^2 - \frac{M_{Ed}}{\beta_1\,\beta_2 b f_{cd}}} \qquad (2.7)$$

The section of steel reinforcement A_s needed for the bending moment slab is estimated using the formula:

$$A_s = \frac{M_{ED}}{f_{yd}(d-\beta_2 x)} \quad (2.\ 8)$$

Where:

M_{ED} is the design moment (maximum moment)

$f_{yd} = \frac{f_{yk}}{1.15}$

The minimum area of steel reinforcement is given as:

$$A_{s,min} = \text{Max}\left\{0.26\frac{f_{ctm}}{f_{yk}} b.d\,; 0.0013b.d\right\} \quad (2.\ 9)$$

Where: $f_{ctm} = 0.3\, f_{ck}^{\frac{2}{3}}$ for $f_{ck} \leq C50$

The maximum area of steel reinforcement is given as:

$$A_{s,max} = 0.04A_c \quad (2.\ 10)$$

Where:

A_c is the area of concrete = $b*h$ for a rectangular section.

To verify steel section, we check if the actual steel section, A_s, is greater than the minimum value presented in the Euro-Code, $A_{s,}$.

$A_{s,\min} \leq A_s \leq A_{s,\max}$

Also, it should not exceed the maximum moment presented in the code.

$A_s \leq A_{s,\max}$

If M_{Ed} is greater than $M_{Rd,lim}$, that is, $M_{Ed} > M_{Rd,lim}$,some reinforcement steel, A'_s in compression is needed. To calculate it, we use $\Delta M_{Ed} = M_{Ed} - M_{Rd,\lim}$

$$A'_s = \frac{\Delta M_{ED}}{f_{yd}(d - d')} \quad (2.\ 11)$$

Where: $d' = \beta_2 . X_{\lim}$

The tensioned reinforcement is:

$$A_s = \left(\frac{M_{Rd,\lim}}{f_{yd}.(d - \beta_2 x_{\lim}}\right) + A'_s \quad (2.\ 12)$$

The spacing between longitudinal bars is:

$$\text{Spacing, } S = \frac{b - n\phi_b - 2C_{nom}}{n-1} \quad (2.\ 13)$$

The minimum spacing between longitudinal bars is given by:

$$s_{\min} = \max\{k_1\,\phi; d_g + k_2; 20\text{mm}\} \quad (2.\ 14)$$

The maximum spacing of longitudinal reinforcement is:

$$s_{max} = 0.75d(1 + \cot\alpha) \quad (2.\ 15)$$

Where dg is the maximum size of aggregate and ϕ is the bar diameter. The recommended values of k_1 and k_2 are 1 and 5mm respectively.

2.4.2.3. Distribution steel

The distribution, transverse or secondary steel runs at right angles to the main moment steel and serves the purpose of tying the slab together and distributing non-uniform loads through the slab. The area of this secondary reinforcement is the same as the minimum area for main reinforcement and should not be less than 25% of the main reinforcement.

$$A_{s,dis} = \max\left\{0.26\frac{f_{ctm}}{f_{yk}}b.d\,;0.0013b.d;25\%A_s\right\} \quad (2.\ 16)$$

2.4.2.4. Verification of shear reinforcement

The maximum shear force, V_{Ed} applied on the structure is obtained from PLAXIS.

It is not usual for a slab to contain shear reinforcement, therefore it is only necessary to ensure that the concrete section on its own may have sufficient shear capacity ($V_{Rd,c}$) to resist the ultimate shear force (V_{Ed}) resulting from the worst combination of actions on the structure, although in most cases a nominal or minimum amount of shear reinforcement will usually be provided. In those sections where $V_{Ed} \leq V_{Rd,c}$, no calculated shear reinforcement is required. The shear capacity of the concrete, $V_{Rd,c}$ in such situations is given by an empirical expression:

$$V_{Rd,c} = \left[C_{Rd,c}k(100\rho_1 f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp}\right]b_w d \quad (2.\ 17)$$

With a minimum value of:

$$V_{Rdc,min} = \left[0.035k^{\frac{3}{2}}f_{ck}^{\frac{1}{2}} + k_1\sigma_{cp}\right]b_w d \quad (2.\ 18)$$

Where:

$V_{Rd,c}$: The design shear resistance of the section without shear reinforcement

f_{ck} is in MPa

$C_{Rdc} = 0.18/\gamma_c$

γ_c : partial safety factor for the concrete (assumed=1.5)

$k = \left(1 + \sqrt{\frac{200}{d}}\right) \leq 2.0$ With d expressed in mm

$\rho_1 = \frac{A_s}{b_w d} \leq 0.02$ Reinforcement ratio corresponding to A_s

A_s: Longitudinal steel area

$k_1 = 0.15$

$\sigma_{cp} = \frac{N_{Ed}}{A_c}$ average stress in the concrete due to the axial compressive force N_{ED}

A_C: Concrete area

b_w : Minimum width of the cross section

d: Effective depth

If $V_{Ed} \geq V_{Rd,c}$, shear reinforcement is required

The design of members with shear reinforcement is based on a truss model (Figure 2.5)

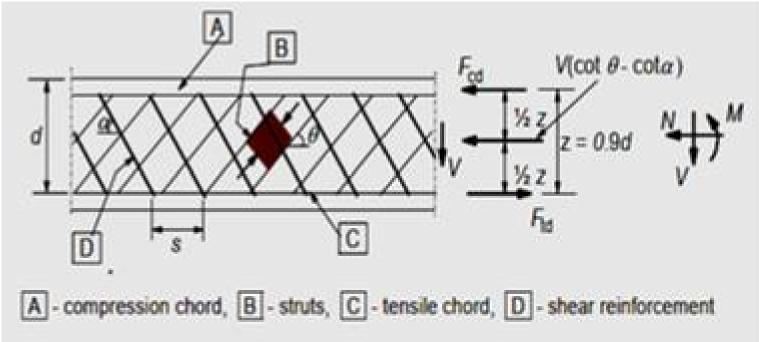


Figure 2. 5. Truss model for beam requiring web reinforcement

In this case the shear capacity is given by the smallest of the terms V_{Rdmax} and V_{Rds}

That is: $\begin{cases} V_{Ed} \leq V_{Rdmax} \\ V_{Ed} \leq V_{Rds} \end{cases}$

Where:

V_{Rdmax}= resisting contribution due to the inclined concrete strut given by the expression:

$$V_{Rdmax} = \alpha_{cw} b_w z \nu_1 f_{cd} (\cot\theta + \cot g\alpha) \,/\, (1 + \cot^2\theta) \qquad (2.\ 19)$$

V_{Rds} = resisting contribution due to the inclined steel tie given as:

$$V_{Rds} = \frac{A_{sw}}{s} z f_{ywd} (\cot\theta + \cot\alpha)\sin\alpha \qquad (2.\ 20)$$

With:

z = 0.9 d for rectangular cross-sections

s = stirrup spacing

f_{ywd} = design yield stress of the web reinforcement

A_{sw} = area of the web reinforcement

α = angle between the shear reinforcement and the axis of the beam (α = 90° for stirrups)

θ= inclination of the cracks or the concrete struts

α_{cw} coefficient of interaction between compressive stresses which can be assumed to be 1

ν_1= reduction coefficient for shear cracked concrete, assumed = 0.5.

f_{cd} = cylindrical concrete compressive strength (design value).

$$\frac{\cot\theta}{1+\cot^2\theta} = \frac{\sin 2\theta}{2}$$

The angle of inclination between cracking lines obtained by taking $V_{Ed} = V_{Rdmax}$ is:

$$\theta = \frac{1}{2}.\arcsin\left(\frac{2V_{Ed}}{\alpha_w * \gamma * Z * b * f_{cd}}\right) \tag{2. 21}$$

Where:

$\alpha_\omega = 1$; $\gamma = 0.5$; $Z = 0.9 \cdot d$

The required range for θ is $21.8° \leq \theta \leq 45°$

If $\theta \leq 21.8°$, take θ= 21.8

If $\theta > 45°$, the beam is not able the resist the force and the section has to be changed.

The spacing between transversal bars is calculated from:

$$\frac{A_{sw}}{s_t} = \frac{V_{Ed}}{0.9 * d * f_{ydw}} \tag{2. 22}$$

The area of transversal reinforcement is given as:

Where:

A_{sw} is the steel frame area

n: number of strands

The maximum spacing of shear reinforcement is:

$$s_{max} = 1.5d \tag{2. 23}$$

$\boldsymbol{\alpha}$=90° for vertical stirrups and cot $\boldsymbol{\alpha} = 0$

$$\text{Number of steel frames, } \#_{steelframes} = \frac{\text{lenghth}}{\text{spacing}} + 1 \tag{2. 24}$$

2.4.3. Design of the side wall

The design of sidewalls consists of determining the reinforcement area. The retaining wall bears lateral loads from backfill and also axial loads from the building. The calculations are carried out for a side wall, assuming a width b per meter linear, as a beam section of $b \cdot h$, subjected to a flexural bending and axial force; where h here represents the thickness of the side wall. Both the maximum positive and negative moments are considered for the design of the steel section.

2.4.3.1. Design for moment reinforcement

From Euro-code, the longitudinal steel area necessary is given by equation:

$$A_s = \frac{M_{ED}}{f_{yd}(d - \beta_2 x)} \tag{2. 25}$$

The minimum area for longitudinal reinforcement is:

$$A_{s,min} = \mathrm{Max}\left\{0.26\frac{f_{ctm}}{f_{yk}}b.d\,;0.0013b.d\right\} \tag{2. 26}$$

The maximum area for longitudinal reinforcement is:

$$A_{s,max} = 0.04Ac \tag{2. 27}$$

The maximum spacing between longitudinal bars is given as:

$$S_{max} = \mathrm{Min}\,\{3h; 400mm\} \tag{2. 28}$$

The area of horizontal reinforcement is the greatest of either 25% of vertical reinforcement or 0.001 Ac.

$$A_{s,dis} = max\{25\%A_s; 0.001A_c\} \tag{2. 29}$$

2.4.3.2.Verification for shear reinforcement

The shear verification of side walls has the same procedure as that of slab described in 2.4.2.4.

2.4.4. Detailing

Structural steel detailing involves the creation of detailed drawings from the results obtained by calculation. It is done using the software AutoCAD.

Conclusion

The main aim of this chapter was to establish the main procedures used to design a retaining wall working also as a floor slab. Firstly, a site recognition is done in order to assess the geographical location, its relief, the climatic conditions, the hydrology, the geology, population and socio-economic activities. This is followed by a collection of geometric and geotechnical data to have the necessary parameters for the study. Afterwards, the analysis of the retaining wall to check for its stability is performed on the modelling software PLAXIS. The different steps for the structural designed is presented and a cost analysis of the project is done.

CHAPTER 3. ANALYSIS AND RESULTS INTERPRETATION

Introduction

In this part we will present and interpret the results obtained according to the methodology we adopted in chapter 2. This part will be depicted as follows. The first section deals with the general presentation of the site focusing on its geographical location, climate, hydrology and hydrogeology, relief geology, population and socio-economic activities. The second part focuses on the presentation of the project and the description of the study area, followed by a presentation of the data collected. After this, we will model the retaining structure on PLAXIS 2D and check for its stability under lateral earth pressures, axial and surcharge loads. Finally, the analytical design to determine appropriate steel reinforcement is done using the procedure described in the previous chapter.

3.1. General presentation of the site

Here, we present the study area through its location, climate, hydrology, relief, geology, population and socio-economic activities.

3.1.1. Location

Yaounde is the capital of Cameroon and the headquarter of the central region. It is located 200 km from the Atlantic coast, between 4° North latitude and 11°35 East longitude. The retaining wall to be designed is located in Bastos in the division of Yaounde I and subdivision of Mfoundi. Figure 3.1 shows the map of Yaounde with the quarter of Bastos where the project is located being highlighted.

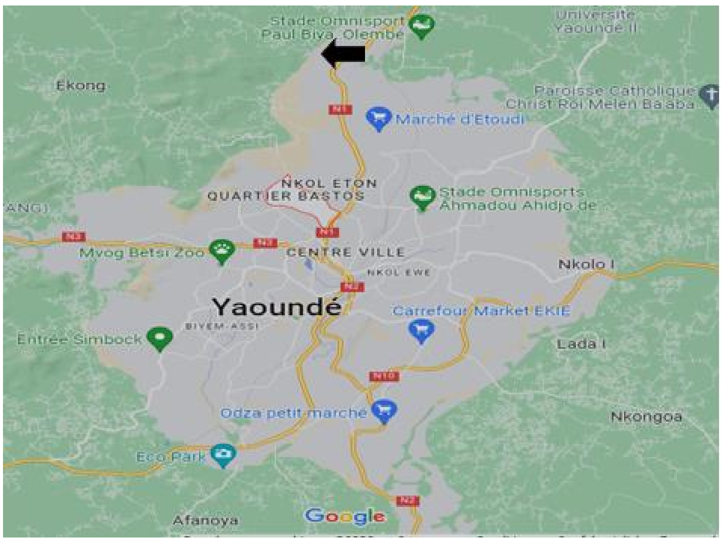


Figure 3.1. Map of Yaoundé City (source: Google map)

3.1.2. Climate, rainfall and vegetation

The climate in the city of Yaoundé is equatorial characterized by the alternation of two dry seasons and two rainy seasons. The average temperature is 23.5°C, varying between 16 and 31°C depending on the season, and the average annual rainfall is 1650 mm and average humidity is 80%. The average humidity is 80% and varies during the day between 35 and 98%. The frequent winds are humid and blow in a south-western direction; the strong winds are oriented towards the north-west. The vegetation is of the intertropical type with a predominance of the southern humid forest (Wéthé J. 1999; 2001).

The climate details of the city of Yaoundé are shown in figure 3.2.

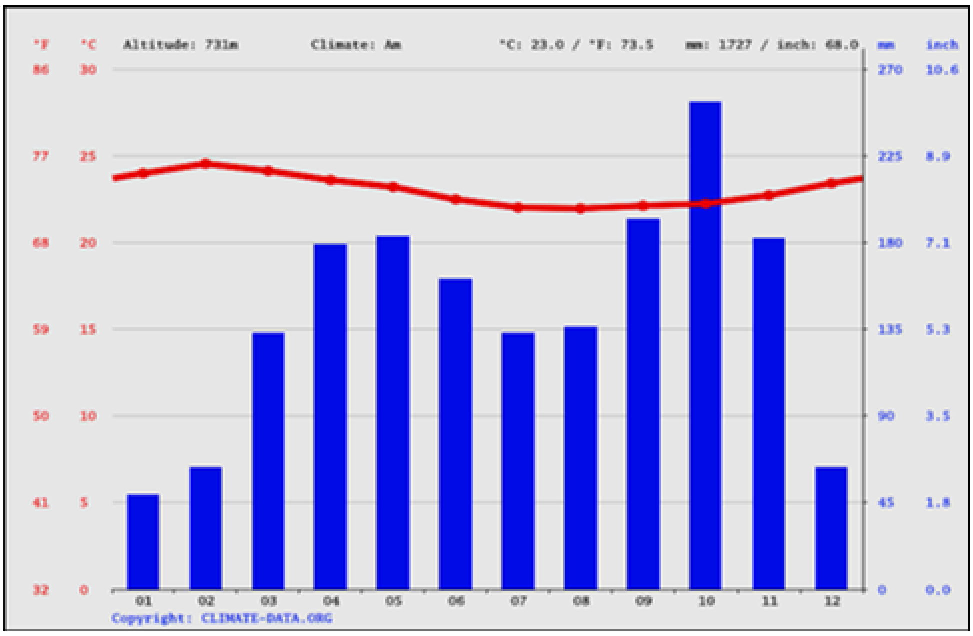


Figure 3.2. Climate details in the city of Yaounde (source: climate-data.org)

3.1.3. Hydrology and hydrogeology

The City's hydrographic network is a set of waterways arranged in a fan shape from two convergences towards the Mfoundi and Mefou rivers which are the main outlets for rainwater. These ensure the natural drainage of runoff and surface water that is discharged into the Mefou River, which in turn discharges its water into the Nyong River. In addition to these waterways, the city has a number of natural or artificial lakes and ponds whose waters are dangerous to public health because of the discharge of water from wastewater treatment plants (as in the case of the municipal lake), household waste and water from latrines located in marshy areas.

3.1.4. Relief

The city of Yaounde is located largely in the watershed of the river Mfoundi river. There are four types of land, namely the low-slope ridges whose land is easily urbanized, hills that can be developed with slopes varying from 5 to 15%, hills that are 15%, hills that are very difficult to develop with a slope greater than 15%, and valley bottoms that are generally with a slope of less than 5%. From these types of land, two main zones arise; the non-buildable zones include, on the one hand, the sectors of low slope (less than 5%) of which the valley bottoms and on the other hand, the steep slope areas, permanent seats of erosion and landslides. The zones that can be built on or urbanized are the slopes and sites with a slope between 5 and 15%.

3.1.5. Geology

The soil of Yaoundé is a red lateritic or ferrallitic soil. However, at depth, these soils are often ochre-yellow in color under the influence of a more which tends to modify the color of the individualized iron. On the surface, the color remains still quite dark and it is often difficult on the ground, in the absence of cutting, to judge the degree of evolution of these soils.

3.1.6. Population and economic activities

Yaoundé has a total population estimated at 3.8 million in 2019. The city of Yaoundé being a cosmopolitan city, there is a considerable portion of population coming from several other region of the country (west, far north etc.). Most of Yaoundé's economy is centred around the administrative structure of the civil services and the diplomatic services. Due to these, Yaoundé has a higher standard of living and security than the rest of Cameroon. However, Yaounde is a tertiary city and there are a few industries: breweries, sawmills, carpentry, tobacco, paper mills, machinery and building materials.

3.2. Presentation of the project

In this section will be evoked the general description of the study area, followed by the presentation of the results obtained from geotechnical test and parameters used for calculations in this work.

3.2.1. Site description

The case study is located in the Golf - Bastos district in the city of Yaounde along a main road. It has an area of approximately 600 m^2 (20*30). The earthworks have already been carried out. They made it possible to clear the entire platform of the future building. The area to be built is surrounded by three embankments with a maximum vertical height of around 8.2 m. On the rear slope, there is a large building (R + 12) whose property line 2.5 m from the slope's surface. This slope is 8.2 m and 7.5 m high and wide respectively. The load generated by the existing building is 96 kN /m^2. The load generated by the new building applied on the top of the retaining wall and backfill is 144 kN /m^2. Figure 3.3 shows a view of the project in plan, side and plan view while figure 3.4 presents a section of the project.

(a) (b)

Figure 3.3. View of the project (a) Plan view (b) Side and front view

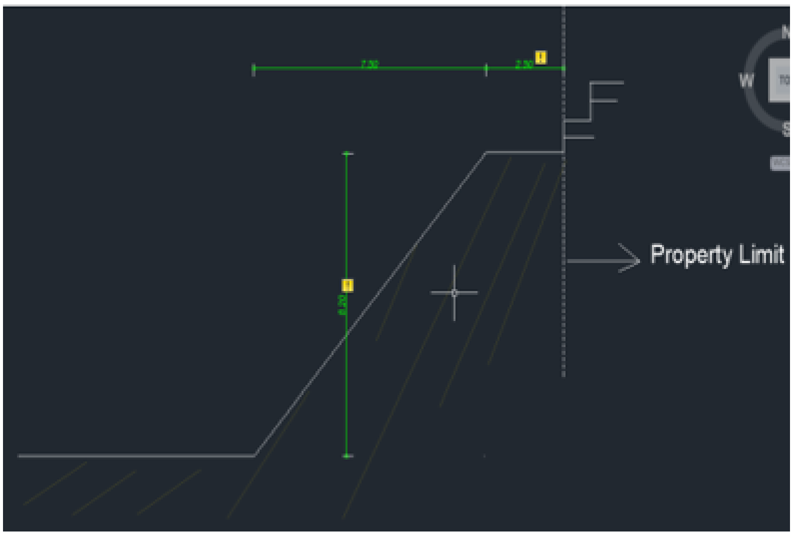


Figure 3.4. Section of the project

3.2.2. Geotechnical characteristics

Information on the site soil were obtained from geotechnical tests conducted at the construction site and in the laboratory.

The section obtained from drilling with a mechanical auger resulting from SPM pressure meter shows the succession of the following soil layers, from top to bottom:

❖ From 0.00 to 1.80 m depth / TN: Red Clay + Decomposed Rock;

❖ From 1.80 to 3.20 m deep / TN: Red Lateritic Breastplate;

❖ From 3.20 to 8.50 m deep / TN: motley Decomposed Rock;

3.2.2.1. Hydrological Data

The piezometric surface was located following the depths presented in the table below.

Table 3.1. Hydrological data

Points of survey	Water level(m)
SPM 1	4.50
SPDL 1	5.00
SPDL 2	-
SPDL 3	-
SPDL 4	0.80

3.2.2.2. Laboratory Data

The results from the laboratory tests are presented in the table below.

Table 3.2. Laboratory data

Number of samples	PEI 1	PEI 2
Location	Rear embankment	Rear embankment
Nature	Red clay	Red clay
Natural water content (%)	21.7	25.3
Granulometry ≤ 80µm	57.1	61.1
Liquid limit LL	56.0	52.0
Plasticity index PI	37	32
LCPC Classification	Very plastic clays	Very plastic clays
Undrained shear strength UU		
Ψ(°) – C(bars)	19.3-0.18	19.0-0.23

These soils correspond to very plastic clays according to the LCPC classification of soils.

The parameters considered for the design are shown on table 3.3.

Table 3.3. Parameters considered for design

Soil parameters of the red clay

Unit weight, γ	18 kN /m^3
Stress-state	Effective
Angle of internal friction, ϕ_{ef}	18.00°
Cohesion of soil, c_{ef}	18.00 kPa
Saturated unit weight, γ_{sat}	22.00 kN /m^3

3.3. Numerical analysis of the stability of the retaining wall

In order to design the retaining wall, arbitrary dimensions are chosen and analyses performed on Plaxis 2d v20 to verify if the wall remains stable under the loads applied to it. The main checks are the serviceability checks which include vertical and horizontal displacements of the wall subjected to lateral pressures and surcharge loads and the global stability of the structure at ULS. This is done by trial and error of the wall's dimensions.

3.3.1. Geometry modelling

The generation of a model starts with the creation of the geometry which is a two-dimensional representation of the real three-dimensional. A working area of 85m width and 16.7m long was used. The general geometry is made up of materials, structures and loads.

3.3.1.1. Material properties

The soil is modelled with three soil layers whose properties are described in table 3.4 and table 3.5**.** The third layer on the top represents an embankment with a slope which is 8.2 m and 7.5 m high and wide respectively. The linear elastic perfectly-plastic Mohr-Coulomb model was used and it involves five input parameters, i.e. E and ν for soil elasticity; φ and c for soil plasticity and ψ as an angle of dilatancy. This Mohr Coulomb model represents a 'first order' approximation of soil or rock behaviour. Also, the Jointed Rock model which is an anisotropic elastic-plastic model, especially meant to simulate the behaviour of rock layers involving stratification and particular fault directions was used for the decomposed rock soil layer. The ground water table is located at 4.5 m below the soil surface: this implies the water influence is not taken into consideration. So the material model considered is the drained behaviour.

Table 3.4. Soil parameters of simulation of the Mohr Coulomb model

Parameters	Symbols (Units)	Layer 2(Red laterite)	Layer 3 (Red clay)	Backfill (laterite gravel)
Material model	-	MC- Drained	MC- Drained	MC- Drained
Type	-	Medium	Fine	Medium
Saturated unit weight	γ_{sat} (kN/m^3)	20	22	20
Unsaturated unit weight	γ_{unsat} (kN/m^3)	17	18	17
Effective young modulus	E'(kN/m^2)	19120	14620	56000
Drained poisson ratio	ν'(-)	0.3	0.3	0.3
Effective cohesion	c'_{ref} (kN/m²)	60	18	60
Effective friction angle	φ'	31	18	31
Dilantancy	ψ	1	0	1

Table 3. 5. Decomposed rock characteristics

Parameters	Symbols(Units)	Layer 1 (Decomposed rock)
Material model	-	Jointed Rock
Type	-	Coarse
unsaturated unit weight	γ_{unsat} (kN/m²)	24
saturated unit weight	γ_{sat} (kN/m²)	26
effective young's modulus	E'(kN/m²)	110000
Poisson's ratio	ν_1	0.3
Young's modulus perpendicular to "Plane 1	E_2(kN/m²)	110000
Poisson's ratio perpendicular to "Plane 1" direction	ν_2	0.3
Shear modulus perpendicular to "Plane 1" direction	G_2(kN/m²)	71500
Number of planes	-	1
Effective cohesion	C'_{ref}(kN/m²)	1
Effective friction angle	φ'	61
Dilatancy	Ψ	31
Dip angle	α_1	0

3.3.1.2. Structures modelling

Plate elements were used to model the retaining wall in the form of a box with a width of 12.5m and height of 8.2m. To simulate a more real behaviour, columns are added at a distance of 3m from each other and a slab is added 4m below the top slab. These plates are structural objects used to generate slender structures with a high flexural rigidity EI and the axial stiffness EA. Interfaces are defined to evaluate the interaction between the structure and the soil. The wall is made of reinforced concrete whose properties are shown in table 3.5.

Table 3. 6. Concrete characteristics

Property	Value	Unit
Class	C30/37	-
fck	30	N/mm²
$fcm = fck + 8$	38	N/mm²
γ_c	1.5	-
$f_{cd} = \frac{0.85\, f_{ck}}{\gamma_c}$	17	N/mm²
$f_{ctm} = 0.3\, f_{ck}^{\frac{2}{3}}$	2.90	N/mm²
$f_{ctd} = 0.7 f_{ctm}$	2.03	N/mm²
ν	0.5	-
E	33000	N/mm²
$EI = E * \frac{1}{12} bh^3$	176000	$\mathrm{kN/m^2}$
$EA = E * b * h$	13200000	$\mathrm{kN/m}$

3.3.1.3. Loads and boundaries

Surcharge loads from new and existing building were applied inside the model. A raft foundation of length 7.5 m is constructed on top of the backfill soil and will act as a floor slab for the new building that will be constructed.

3.3.1.4. Selection of backfill and compaction

Several soil types were tested including red clay, sand, gravel and laterite gravel. Red clay, sand and gravel were not able to sustain the high loads and caused high settlements in the backfill. The backfill used for the project is laterite gravel. These soils are formed mostly in tropical areas and are rich in iron and aluminium. They are of rusty-red coloration, because of high iron oxide content. Laterite soils in their natural state are granular in structure and are possessed of low plasticity and excellent free drainability. In this state they can carry heavy loadings. It is a local material and thus easily purchasable reducing the cost of the project.

Figure 3. 5. Laterite gravel for backfill (www._red-laterite-gravel-for-background.html)

The compaction of the backfill was simulated by using material properties for compacted laterite gravel. Figure 3.5 illustrates a typical laterite gravel whose properties are described in table 3.4.

Figure 3.6 presents the global geometry of the axisymmetric model of the retaining wall.

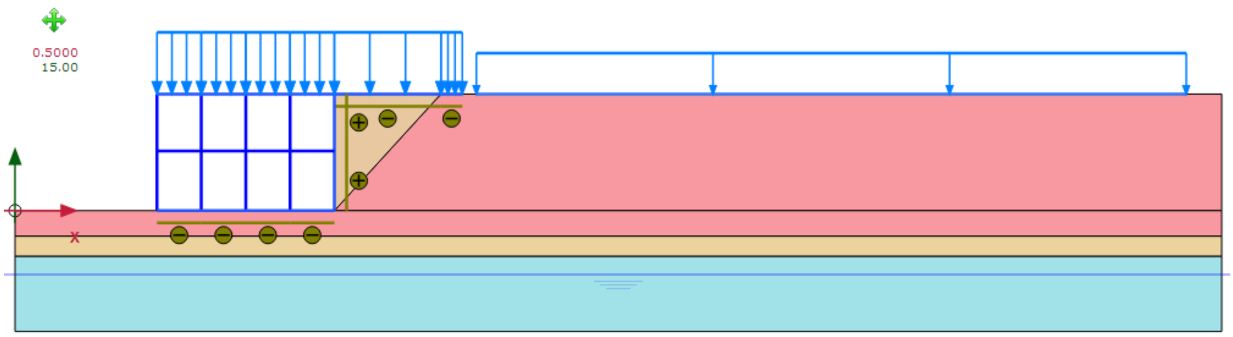


Figure 3. 6. General geometry of the plain strain model of the retaining wall (Plaxis 2D)

3.3.1.5. Drainage

The backfill is not exposed to moisture since the water table is far below it and a reinforced concrete slab is built on its top thus minimising infiltration. However, a drain is added at the bottom of the wall to eliminate any infiltration that may occur.

3.3.2. Mesh generation

Generating a proper Finite Element mesh is an important intermediate step between the definitions of the geometry and the construction stages. In order to have a smooth and accurate calculation the finite element mesh has to fulfil several criteria. For the numerical stability of the calculation, the mesh should have a good quality, that is to say, the elements should be regular without being excessively long and thin. For the accuracy of the calculation, the elements should be small enough, especially in those areas where significant changes in stress or strain can be expected during the analysis (Plaxis2DCE-V21.01-02-reference). In our study a medium mesh is used. The powerful 15-node element used provides an accurate calculation of stresses and failure loads. The meshes composed of 15-node elements are actually much finer and much more flexible than meshes composed of 6-node elements, but calculations are also more time consuming.

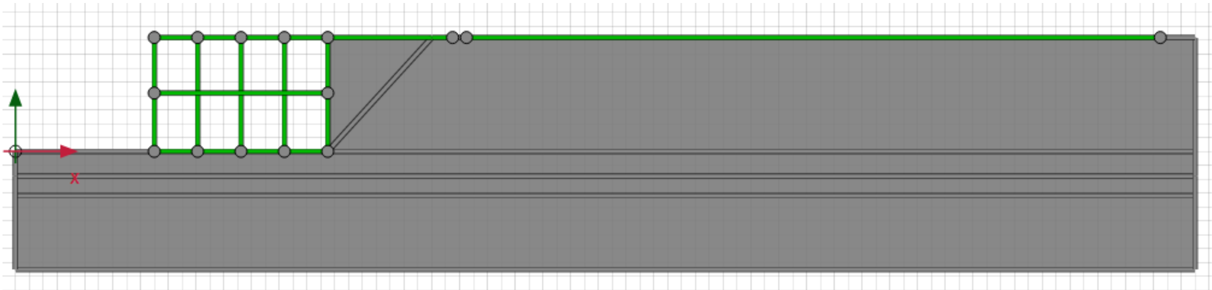


Figure 3. 7. Activation of mesh for refinement

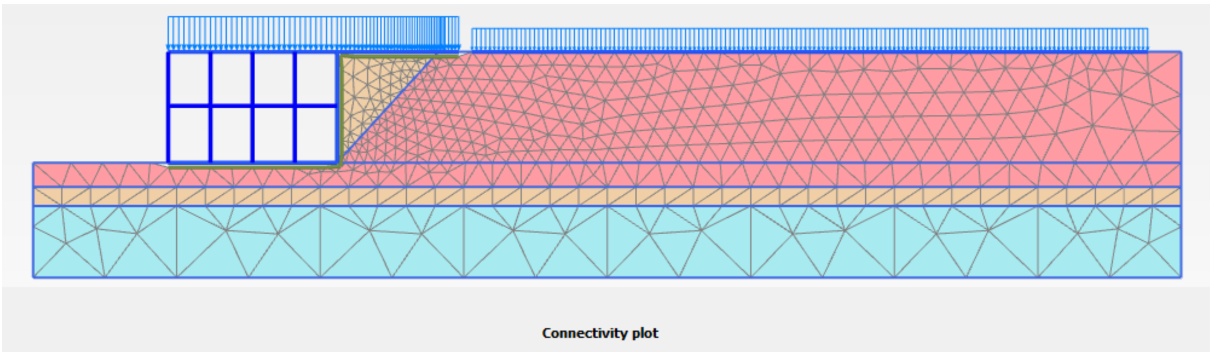


Figure 3. 8. Mesh generation

3.3.3. Calculation phase

The analysis performed is the plastic analysis with 5 stages of construction. A Plastic calculation is used to carry out an elastic-plastic deformation analysis in which it is not necessary to take the change of pore pressure with time into account. The stiffness matrix in a normal plastic calculation is based on the original un-deformed geometry. This type of calculation is appropriate in most practical geotechnical applications.

The loading input is Staged construction which enables an accurate and a realistic simulation for various loading, construction and excavation processes is used for this project. Stage construction is the most important type of loading input.

The different stages of construction are defined starting from the initial phase which is the geometry of the project followed by the addition of the load from existing building, the construction of the retaining wall, the driving of the micropile, the addition of the backfill with slab on its top and finally the application of the loads from the new building as shown in figure 3.9.

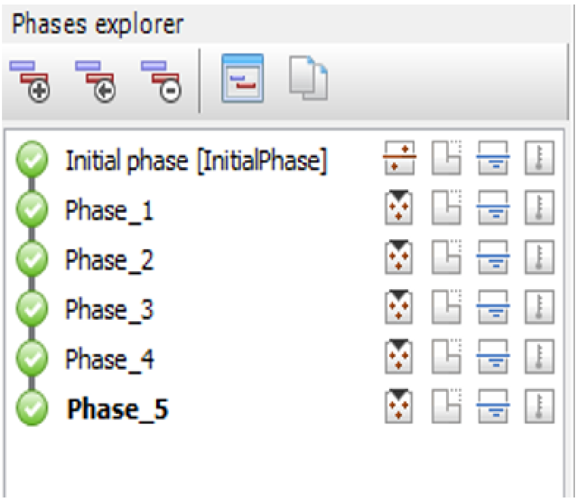


Figure 3. 9. Stage construction phases of the retaining wall

The screen shots of the different steps of the stage construction are shown in annexe B.

3.3.4. Presentation of output results from analysis on Plaxis

The FEM conception by PLAXIS was able to estimate the following results: deformations, horizontal and vertical displacements, gloabal stability, axial forces, shear forces and bending moments on the retaining wall.

3.3.4.1. Deformation of the model

The deformed mesh (figure 3.10) of the model scaled up to 50 times the true scale shows large deformations of the raft foundation on the backfill. This is due to the high loads from building.

After a first analysis, the settlement of the backfill are seen to be 118mm which is above the prescribed limit of 50 mm stipulated by EC7 part 1 due to the high surcharge loads as shown in figure 3.11

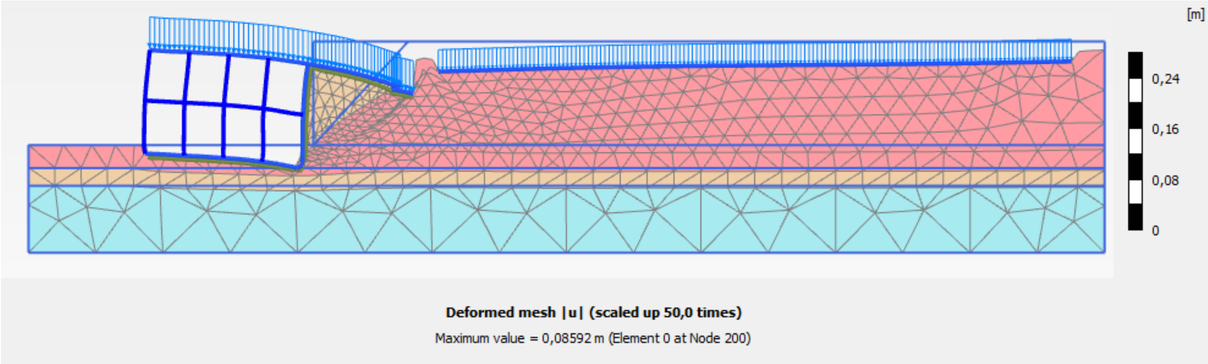


Figure 3. 11. Deformed mesh of the geometry

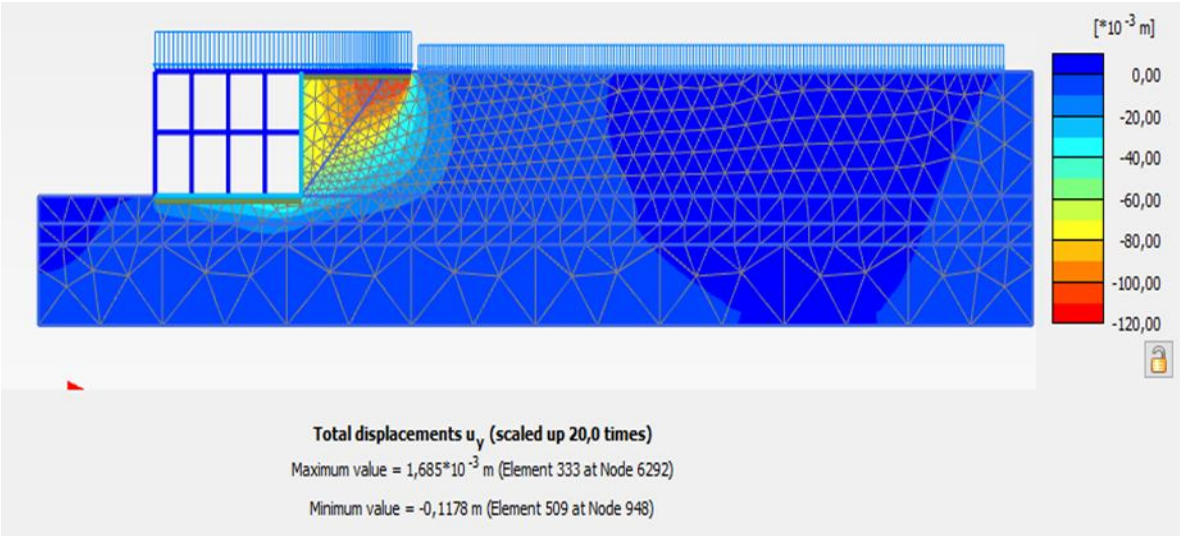


Figure 3.10. Vertical displacement of geometry

The global stability with respect to the most critical failure surface shown in figure 3.12 is checked.

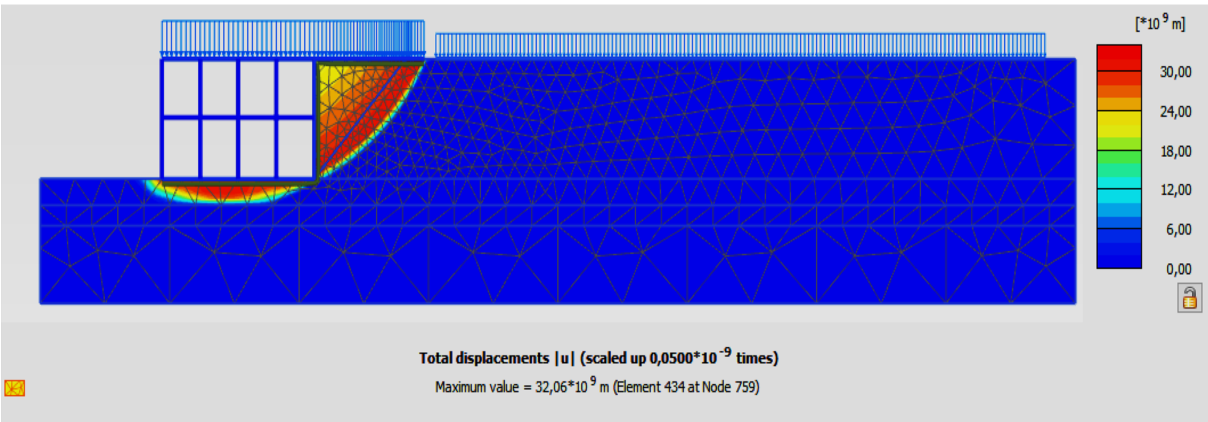


Figure 3. 12. Slip surface of the model

In the safety approach, the shear strength parameters of the soil (friction angle and cohesion) are successfully reduced until failure of the structure occurs.the principal results of a safety calculation are the failure mechanism and the corresponding $\sum Msf$, which is the safety factor. A graph with steps in the ordinate and incremental multiplier Msf in the abscissa is plotted as illustrated in figure 3.13 and a safety factor of 1.13 is obtained.

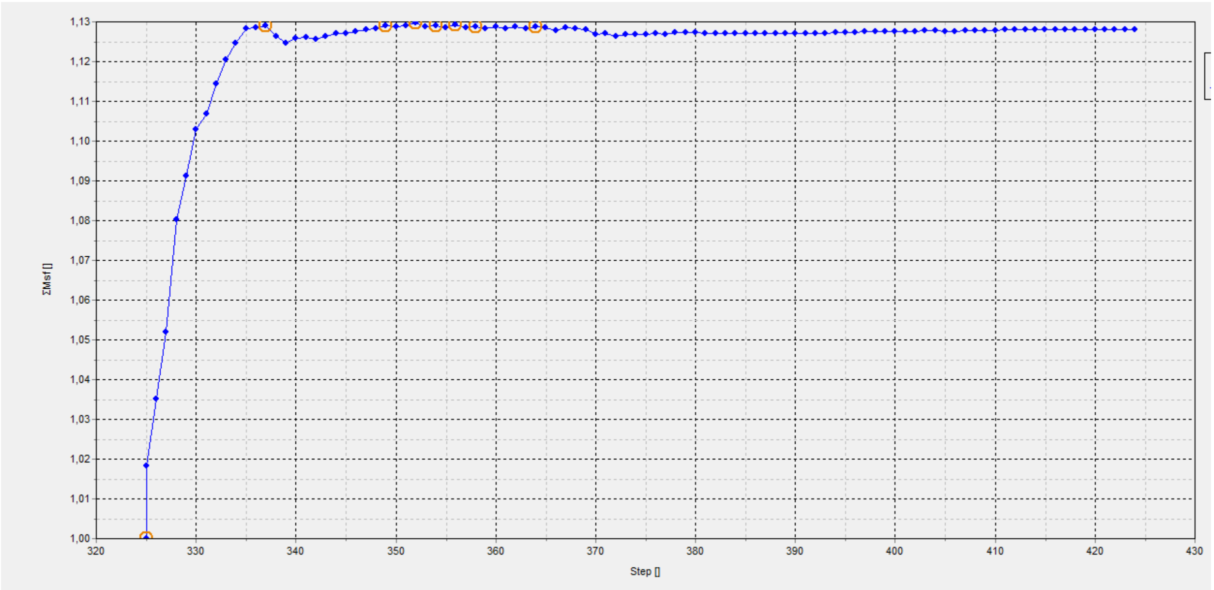


Figure 3. 13. Safety factor for glabal stability

The maximum settlement is above the prescribed limit of 50mm. This means the raft foundation on the backfill is unable to bear the loads applied to it. Furthermore, the factor of safety is below the desirable value of 1.5 for global stability. A micropile is added below the raft foundation as a support system to improve its bearing capacity and reduce the settlements.

3.3.4.2. Results with micropile

Micropiles are feasible when working in proximity to existing structures which is an existing building in our case, they minimise vibrations and noise.

The simulation of a micropile on PLAXIS was done creating an embedded beam rows with a depth of 10m whose properties are defined in table 3.7. A point load of 10kN/m was applied on the pile to represent the driving force.

Table 3. 7. Micropiles characteristics

Parameters	symbols	Units	Piles
Young's modulus	E	kN/m²	210000000
unit weight	ɣ	kN/m^3	78.5
Predefined beam type	-	-	Massive circular beam
Diameter	D	m	0.25
Axial skin resistance at start	$T_{skin,start,max}$	kN/m	500
Axial skin resistance at end	$T_{skin,end,max}$	kN/m	800
Base resistance	F_{max}	kN	1000

The deformation of the model scaled up to 50 times the true scale shows the vertical displacement of the top raft foundation and the upward vertical movement of the raft foundation at the bottom. This movements are induced by lateral pressures from backfill and by the surcharge loads from new and existing building. The deformed mesh of the model is shown in figure 3.14.

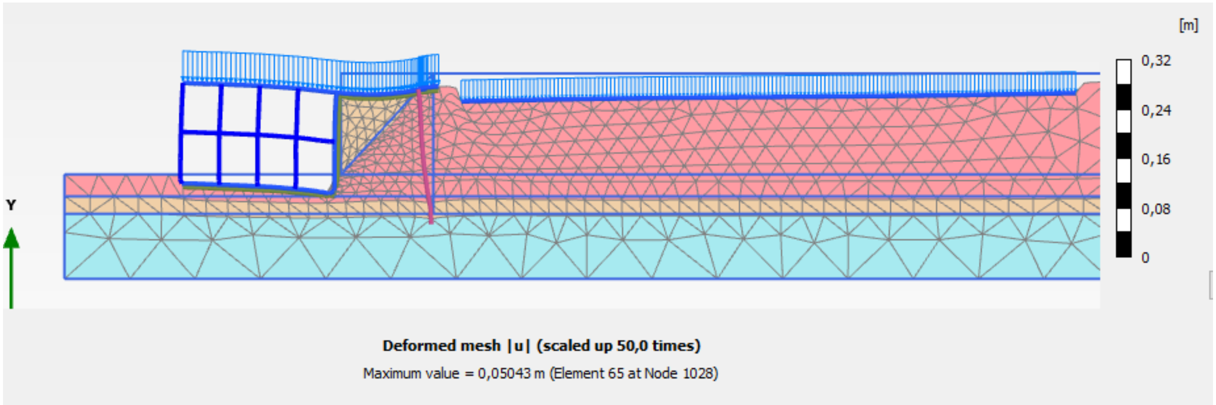


Figure 3. 14. Deformed mesh with micropile

After the addition of the micropile of diameter 0.25m, the settlements reduce to a maximum of 45mm. Figure 3.15 shows that the highest settlements in the model are found below the existing building and raft foundation on the top of the backfill. This is due to the fact that they receive high loads from the existing and new building respectively. The micropile reduces the settlements to acceptable limits stipulated by Eurocode.

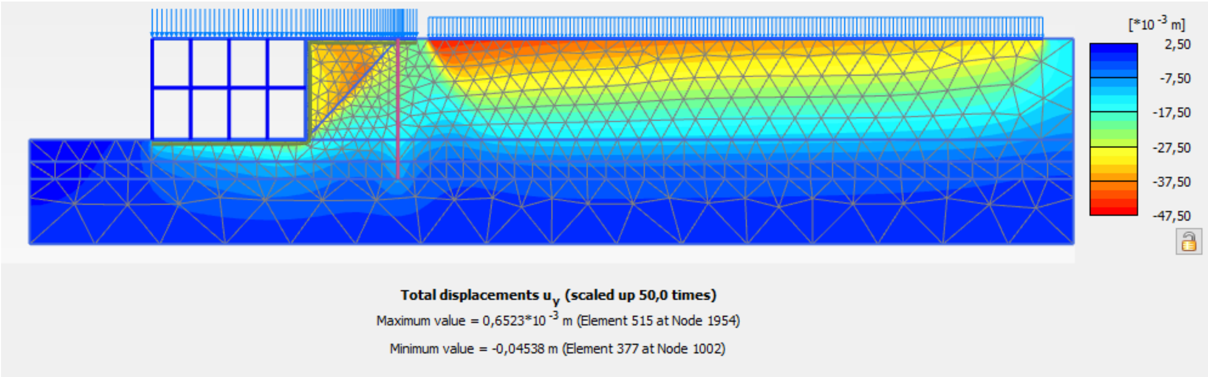


Figure 3. 15. Vertical displacements of geometry

Figure 3.16 shows that the greatest horizontal displacements are found at the bottom part of the retaining wall. This is due to the lateral pressures from backfill which increase with depth. The maximum allowable horizontal movement is 0.5%H and is 41mm. The maximum horizontal displacement is 32mm which is below the allowable limit.

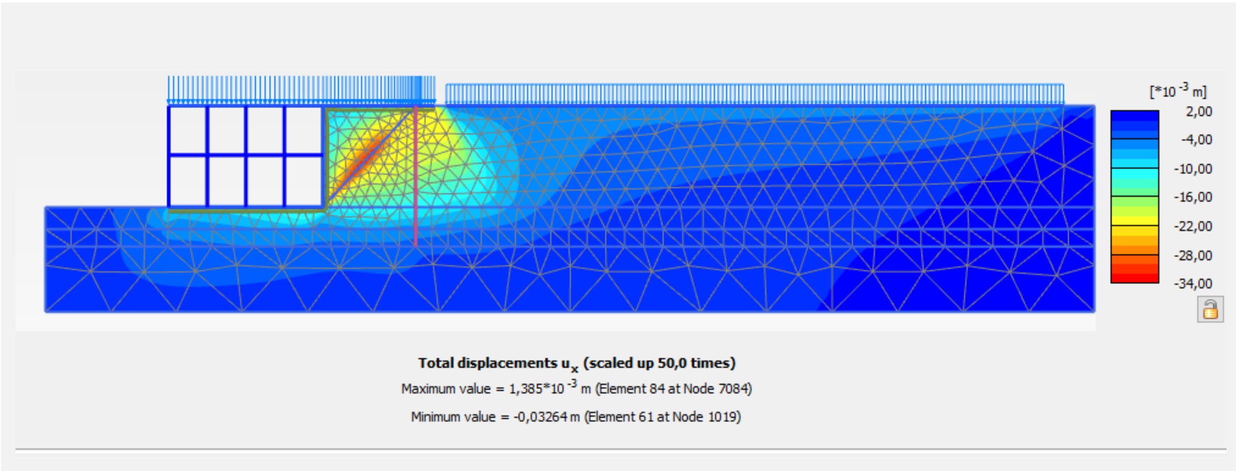


Figure 3. 16. Horizontal displacement of geometry

The global stability with respect to the most critical failure surface shown in figure 3.17 is checked and a safety factor of 2.37 is obtained from the graph illustrated in figure 3.18 using the c-phi reduction method.

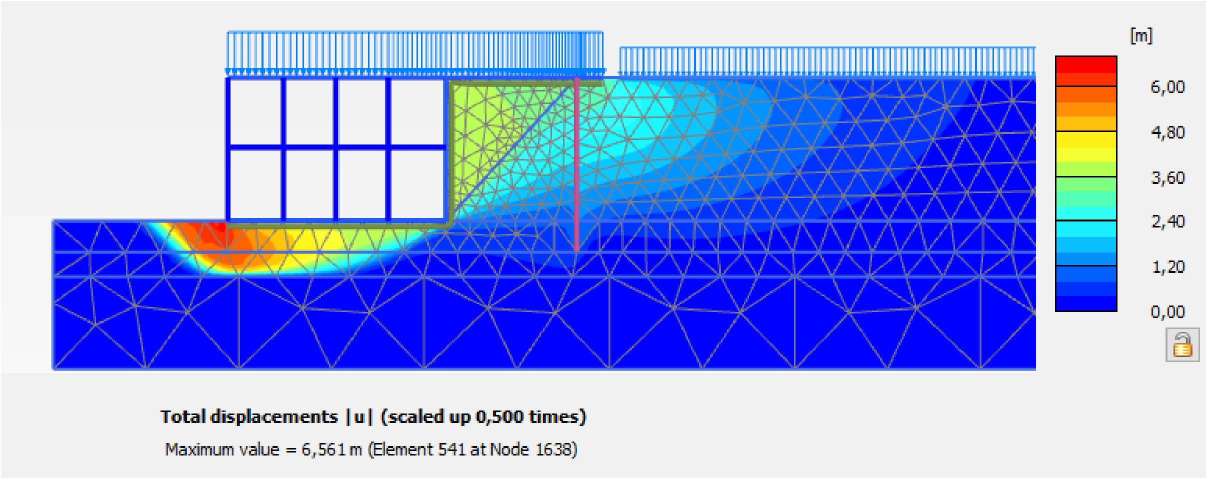


Figure 3. 17. Slip surface of model with micropile.

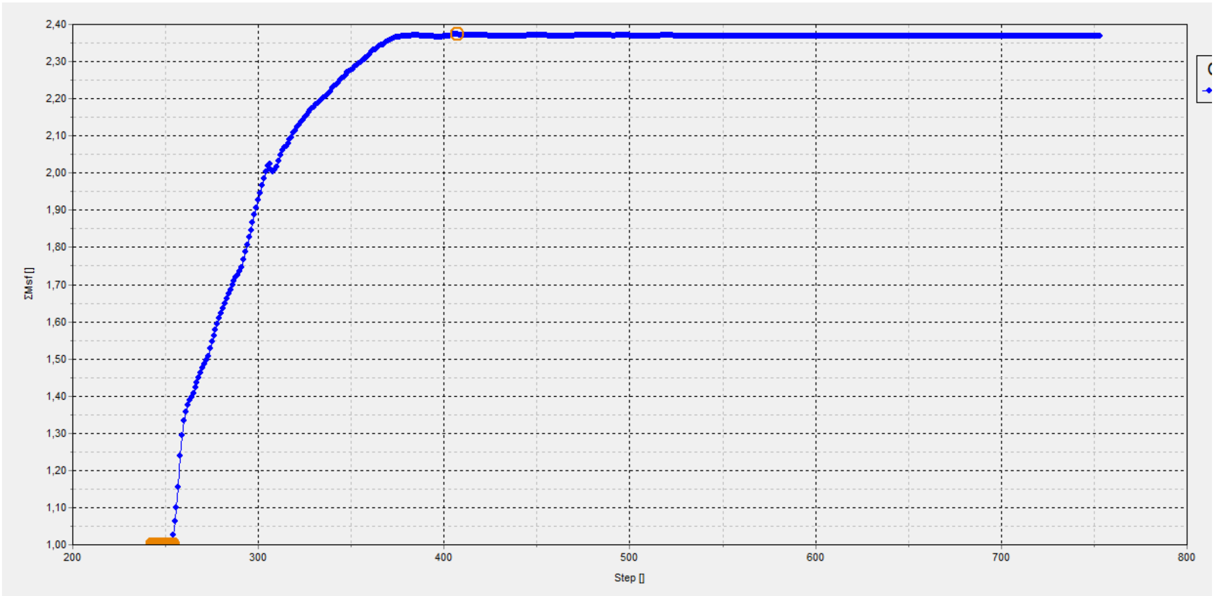


Figure 3. 18. Safety factor for global stability with micropile

3.3.5. Stability check on the retaining wall

The structure is a box and is fixed at both ends limiting rotations and translations thus making is stable to sliding and overturning. The main checks on the structure are the global stability at ULS and the maximum vertical and horizontal displacements at SLS. According to Eurocode 7, total settlements up to 50mm are often acceptable for normal structure.

3.3.5.1. Stability check on retaining wall without micropile

Figure 3.19 and figure 3.20 show that the retaining wall has a maximum vertical and horizontal displacement of 83mm and 26mm respectively. The retaining wall is not safe at SLS.

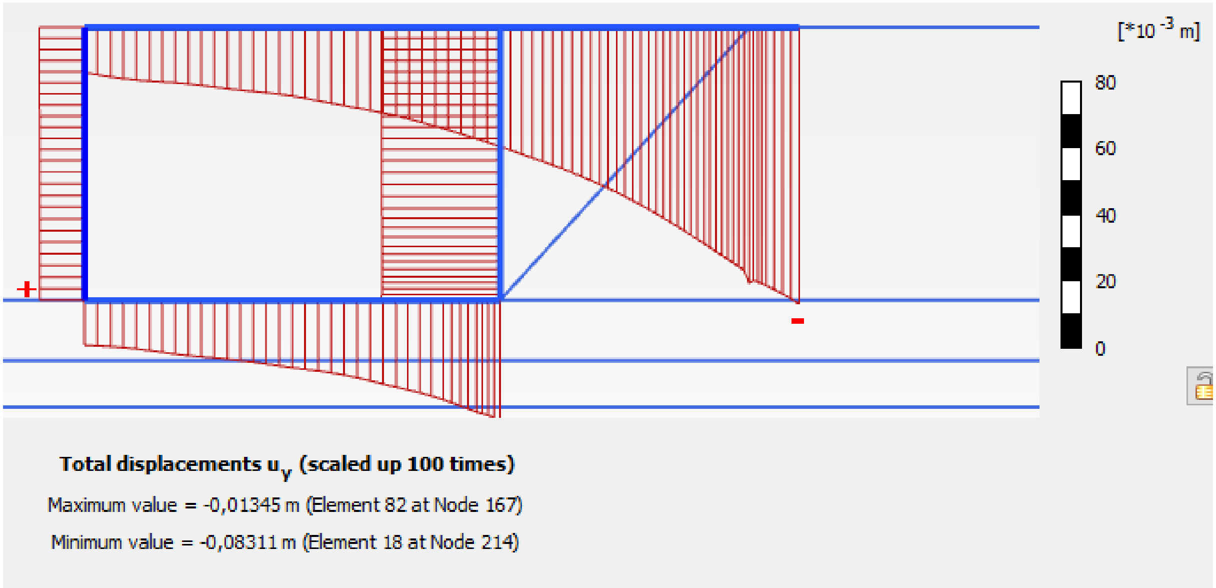


Figure 3. 19. Vertical displacement of retaining wall without micropile

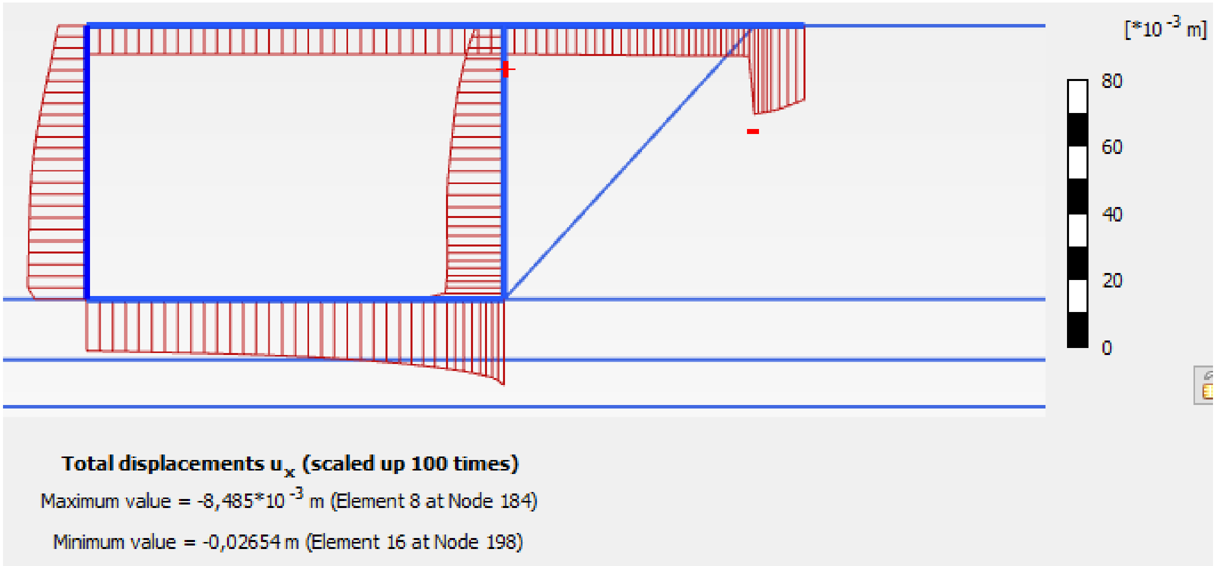


Figure 3. 20. Horizontal displacement of retaining wall without micropile

The safety factor for the global stability on the retaining wall is shown on figure 3.13 and is 1.13. This is less than the desirable value of 1.5.

3.3.5.2. Stability check on retaining wall with micropile

Figure 3.21 and figure 3.22 show that the retaining wall has a maximum vertical and horizontal displacement of 37mm and 24mm respectively which falls under the acceptable range. The retaining wall is safe at SLS.

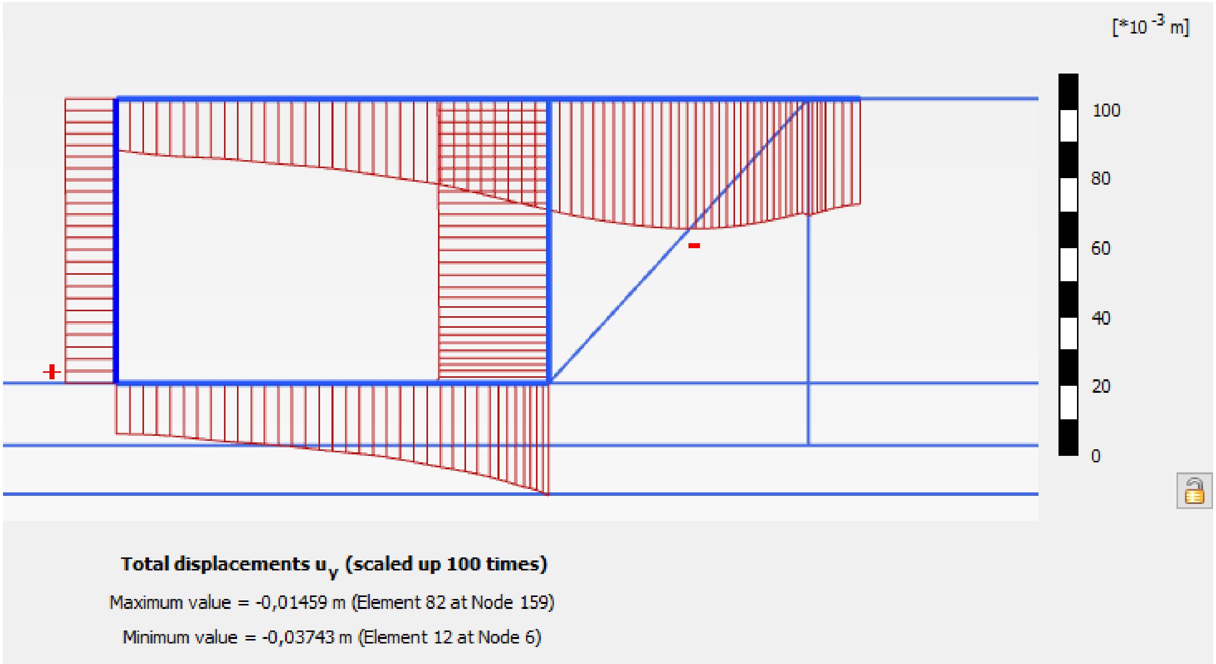


Figure 3. 21. Vertical displacements of the retaining wall

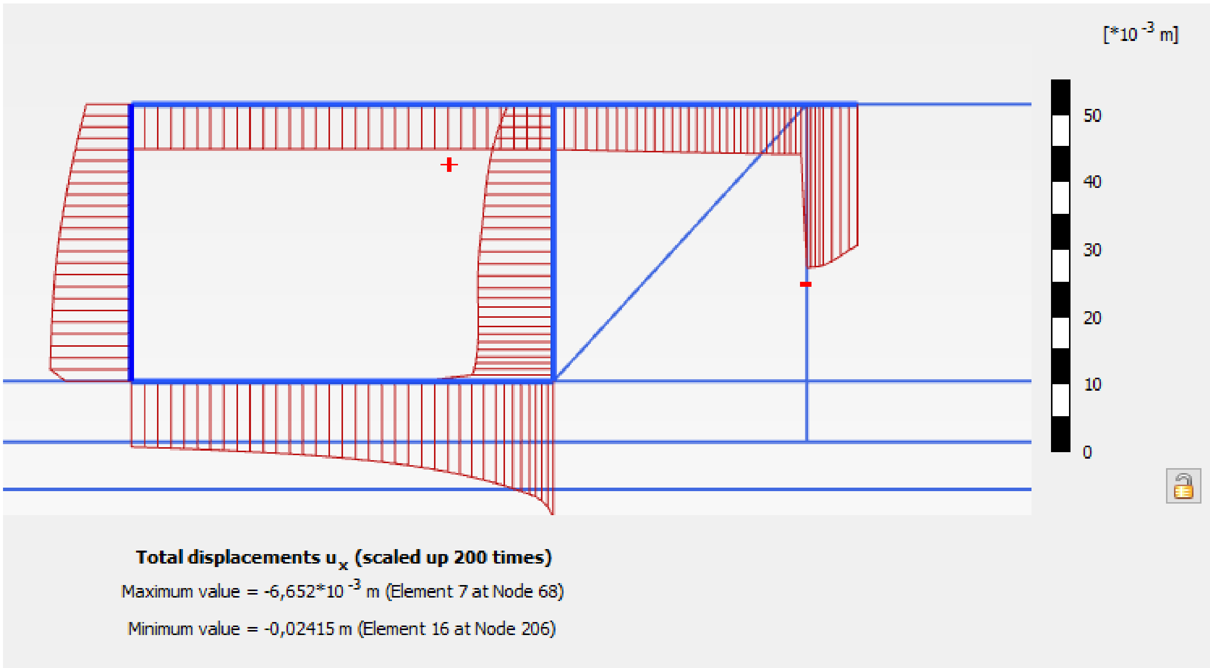


Figure 3. 22. Horizontal displacements of the retaining wall

At ULS, the safety factor for global stability is 2.37 as shown on figure 3.18 which means our structure is safe.

Table 3. 8. Summary of some results obtained from PLAXIS for the retaining wall without micropile

Thickness of wall,h	400mm	
At SLS		
Maximum horizontal displacement(mm)	26mm	SAFE
Maximum Vertical displacement(mm)	83mm	NOT SAFE
At ULS		
Safety factor for global stability	1.13	NOT SAFE

Table 3. 9. Summary of some results obtained from PLAXIS for the retaining wall with micropile

Thickness of wall,h	400mm	
At SLS		
Maximum horizontal displacement(mm)	24mm	SAFE
Maximum Vertical displacement(mm)	37mm	SAFE
At ULS		
Safety factor for global stability	2.37	SAFE

3.3.6. Significance of the results obtained from plaxis

The Objective of the analysis on PLAXIS 2D was to check if the earth retaining wall working also as a floor slab for a building would be stable to lateral pressures from retained soil and loads from surcharge loads due to new and existing building. From the results of the modelling above on PLAXIS 2D, we realize that the vertical and horizontal displacements of the retaining wall are below the acceptable limit when a micropile is added below the raft foundation meaning our structure is safe at long term from a geotechnical point of view. But this is not sufficient to conclude that the wall is safe. A design of the necessary reinforcement to resist to moment and shear in the reinforced concrete wall needs to be done at ULS.

3.4. Structural design of the retaining wall

The retaining structure is a box with two slabs (one on top and one at the bottom) and three vertical walls at each side of the embankments. The other side could be the entrance of a garage. In this design, we considered the two slabs (the bottom slab is a raft foundation) and the vertical wall that retains the soil on the rear side (the side with a slope). The predesign was made by verifying the stability of arbitrary dimensions on PLAXIS.

3.4.1. Forces on retaining structure

The forces acting on the wall were obtained from PLAXIS 2D using design approach 1 combination 1 for the ULS design. The surcharge loads from new and existing building were considered as permanent unfavourable loads with a partial coefficient of 1.35 as predicted by EC2. The loads due to the driving of the micropiles were considered as variable unfavourable loads with a coefficient of 1.5. Figures 3.23, 3.24 and 3.25 show the results of the maximum and minimum forces that were obtained from PLAXIS 2D.

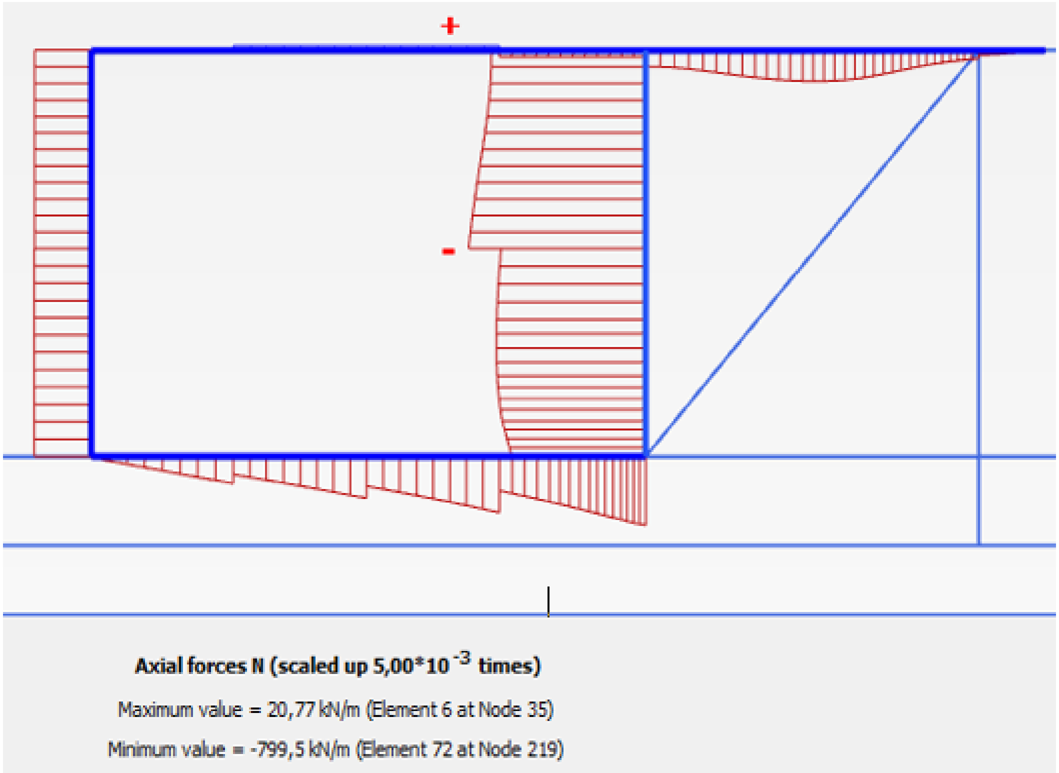


Figure 3. 23. Diagram of axial force on the retaining structure

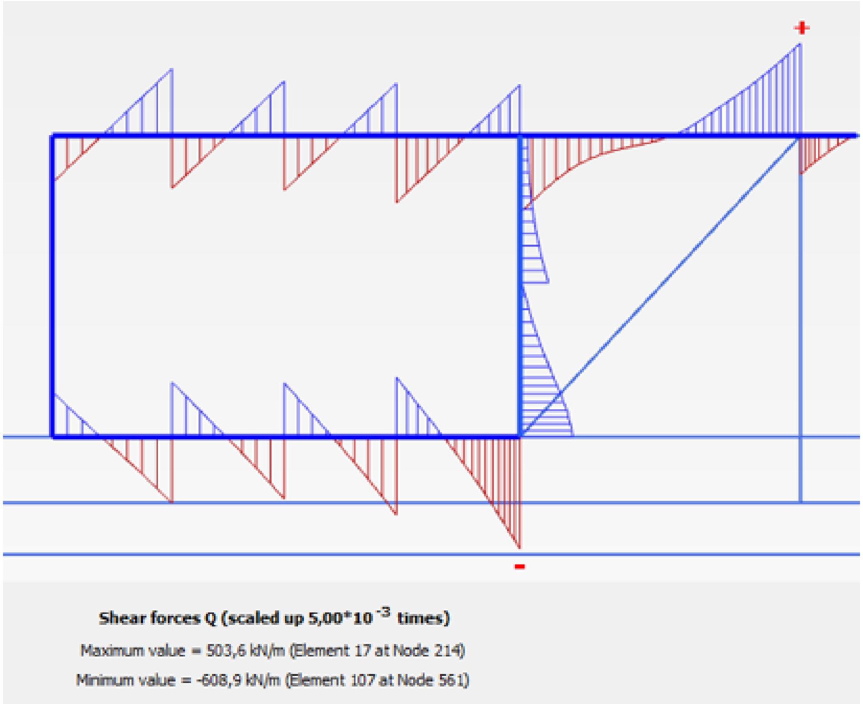


Figure 3.24. Diagram of shear force on the retaining structure

The greatest shear require greater amount of shear reinforcement. The greatest shear reinforcement is on raft foundation at the top of the backfill for the top slab and on the end support at the right side for the bottom slab.

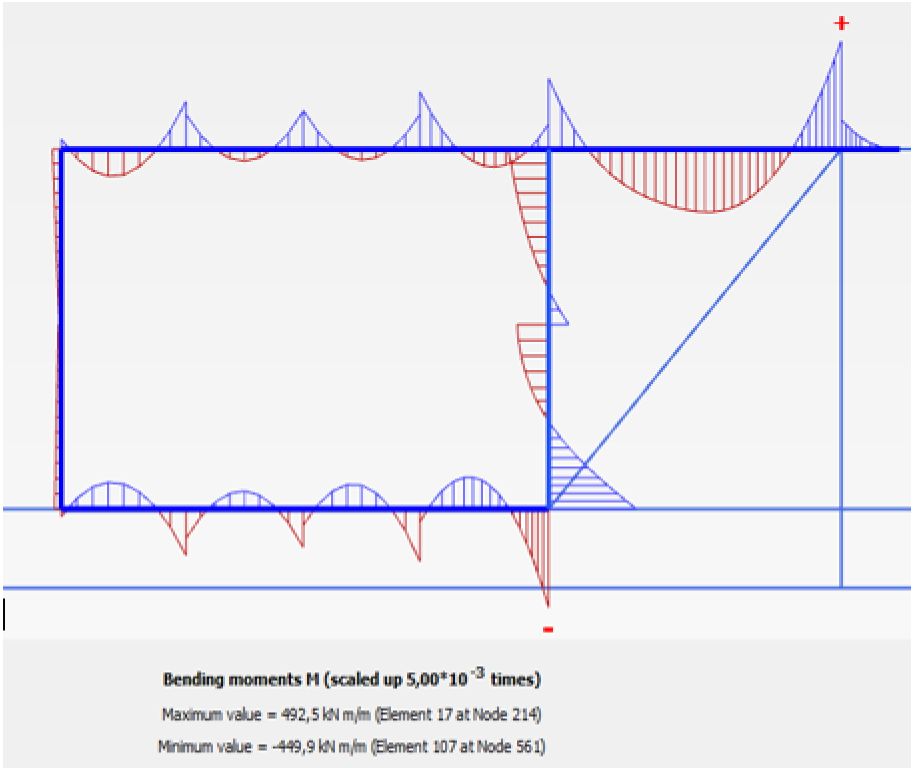


Figure 3.25. Diagram of bending moment on the retaining structure

The greatest moments on the top slab are found at the raft foundation on backfill meaning they require the greatest steel reinforcement. On the bottom slab, the greatest reinforcements will be done at the support on the right where the moments are greatest. The greatest reinforcements on the wall are done on the side that retains the soil.
The summary of the forces at critical points are obtained from the table in Plaxis.

Table 3.10. Summary of Data from PLAXIS

		M_{ED}(kNm/m)	V_{ED}(kN/m)	N_{ED}(kN/m)
Top slab	Maximum positive of right span	+447.3	+548.9	-121.79
	Maximum negative of right span	-492.1		
	Maximum positive of left span	+137.16	-374	-43.7
	Maximum negative of left span	-286.58		
Bottom slab	Maximum positive of span	+158.5	-421.04	-207.04
	Maximum negative of interior spans	-211.15		
	Maximum negative of exterior support	-459.5	-616.7	+260
Side wall	Maximum positive	+459.5	+260.44	-613.37
			+91.09	-687.08
	Maximum negative	-213	+143.14	-815.72
			+67.04	-771.96

The data required to do the structural design of the retaining structure is given below:

Table 3. 11. Data for structural design of the retaining wall

Symbol	Value
β_1 for fck ≤ 50Mpa	0.8
β_2 for fck ≤ 50Mpa	0.4
ε_{cu} for fck ≤ 50Mpa	0.0035
$C_{Rd,c}$	0.12
k_1	0.15
ν_1	0.5
f_{yk}	500 N/mm^2

3.4.2. Design of top and bottom slabs

The top slab consists of a part resting on columns of the building and a part acting as a raft foundation resting on the soil backfill. The bottom slab is a raft foundation. They are designed as one way slabs with formulae from EC2.

3.4.2.1. Determination of concrete cover

Considering a structural class S4 and the exposure class XC2,

C_{min} = max $\{C_{min,}\ ;\ C_{min,\ dur};\ 10mm\}$ = 25mm

$C_{nom} = C_{min} + \Delta C_{dev} =$ 35mm

We take C_{nom}=40mm

3.4.2.2. Moment reinforcement design of top and bottom slabs

The following data are calculated:

$C_{min,dur}$	25
$f_{yd} = \frac{f_{yk}}{1.15} = \frac{500}{1.15}$	434.78Mpa
$\varepsilon_{yd} = f_{yd}/E_s = \frac{434.78}{210000}$	0.0021
ε_{cu}	0.0035
$f_{cd} = \frac{0.85\,f_{ck}}{\gamma_c} = \frac{0.85*30}{1.5}$	17Mpa
$f_{ctm} = 0.3\,f_{ck}^{\frac{2}{3}} = 0.3*30^{2/3}$	2.8965Mpa

a. Design of top slab

In order to be most optimum, the design is done for the most critical sections.

In the top slab, the sections are divided into four as shown in figure 3.26. The procedure presented in the section 2.4.2 is used.

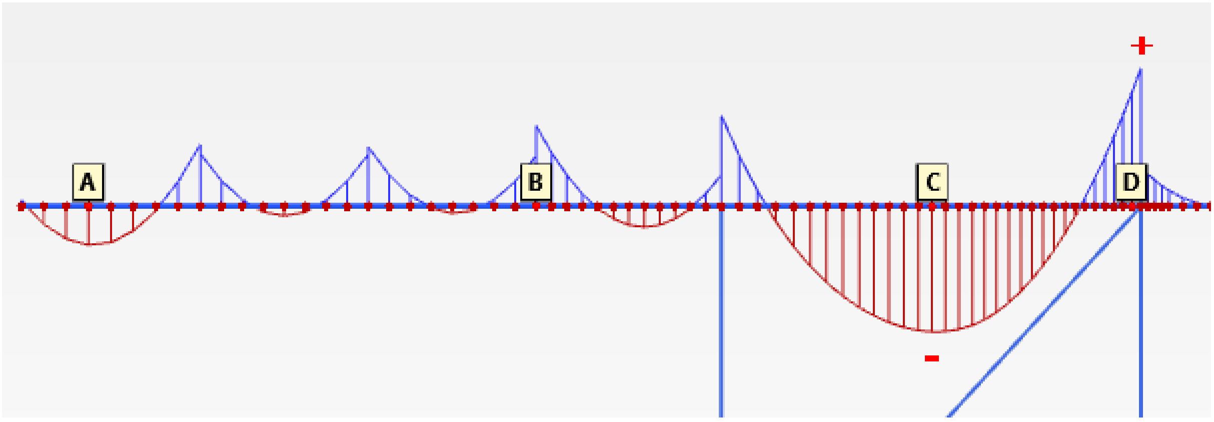


Figure 3. 26. Critical sections for moment design on top slab

i. For maximum negative at C, M_{ED}=447.3kNm

$d = h - c_c$ = 260 mm

$x_{\lim} = \frac{\varepsilon_{cu}}{\varepsilon_{cu}+\varepsilon_{yd}}.d$ = 219.91mm

$F_c = \beta_1.b.x_{\lim}.f_{cd}$ = 2990812.117N

$z_{\lim} = (d - \beta_2.x_{\lim})$ = 262.03mm

$M_{Rd,lim} = F_c.z_{lim}$ = 783.70Nmm

$M_{Ed} < M_{Rd,lim} \Longrightarrow$ Moment is verified

Therefore, tension reinforcement A_s alone is needed

$A_s = \frac{M_{ED}}{0.9*d*f_{yd}}$ = 3266mm²

$A_{s,\min} = \max\left\{0.26\frac{f_{ctm}}{f_{yk}}b_t d\,;0.0013b.d\right\}$ = 527.16mm²

$A_{s,max} = 0.04A_c$ = 16000mm²

Therefore, $A_{s,\min} \leq A_s \leq A_{s,\max} \Longrightarrow$ steel reinforcement section is VERIFIED. OK!

So we will provide 11ϕ20 at the bottom of the slab

$A_{s,provided}$= 3456mm²

Spacing, $S = \frac{b-n\phi_b-2C_{nom}}{n-1}$ = 68mm

We take S=65mm

$s_{min} = max\{k_1\,\phi; d_g + k_2; 20mm\}$= 25mm

$S_{min} < S$ so spacing is verified.

$$A_{s,dis} = \max\left\{0.26\frac{f_{ctm}}{f_{yk}}b.d\,;0.0013b.d;25\%A_s\right\}$$

$$= \max\left\{0.26 * \frac{2.8965}{500} * 1000 * 350\,; 0.0013 * 1000 * 350; 25\% * 3456\right\}$$

$$= 864 \text{ mm}^2$$

Provide $8\phi12$

$A_{s,dis,provide}$= 905mm²

The same procedure is done for all the other moments to obtain results.

ii. For maximum positive at D, M_{ED} = 492.1kNm/m

$M_{RD,lim}$= 783.7 kNm/m

$M_{Ed} < M_{Rd,lim} \Longrightarrow$ Moment is verified

A_s= 3594mm²

$A_{s,min} < A_s \Longrightarrow$ steel reinforcement section is VERIFIED!

Provide $12\phi20$

$A_{s,provided}$= 3770mm²

S= 60mm

S_{min} < S so spacing is verified.

$A_{s,dis} = 899\text{mm}^2$

Provide $8\phi12$

$A_{s,dis,provide}$= 905mm²

iii. For maximum negative at left span,A, MED = 137.16kNm/m

$M_{RD,lim}$= 783.7 kNm/m

$M_{Ed} < M_{Rd,lim} \Longrightarrow$ Moment is verified

A_s= 1002mm²

$A_{s,min} < A_s \Longrightarrow$ steel reinforcement section is VERIFIED!

Provide $5\phi16$

$A_{s,provided}$=1006mm²

S= 205mm

S_{min} < S so spacing is verified.

$A_{s,dis}$= 528mm²

Provide $5\phi12$

$A_{s,dis,provide}$= 566mm²

iv. For maximum positive at left span,B, MED = 286.58kNm/m

$M_{RD,lim}$= 783.7 kNm/m

$M_{Ed} < M_{Rd,lim} \Longrightarrow$ Moment is verified

A_s= 2093mm²

$A_{s,min} < A_s \Longrightarrow$ steel reinforcement section is VERIFIED!

Provide 11ϕ16

$A_{s,provided}$= 2212mm²

S= 70mm

S_{min} < S so spacing is verified.

$A_{s,dis}$ = 553mm²

Provide 5ϕ12

$A_{s,dis,provide}$= 566mm²

b. Bottom slab

For its design, the slab is divided into three section as shown in figure 3.27.

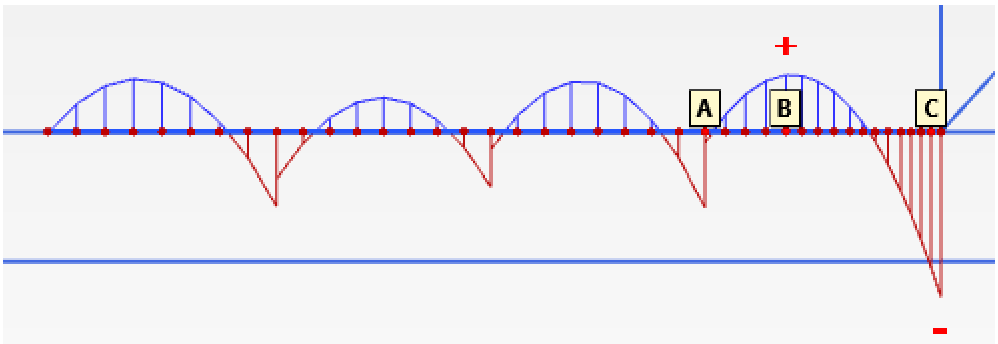


Figure 3. 27. Critical sections at bottom slab

i. For Maximum positive at B, M_{ED}=158.5 kNm/m

$M_{RD,lim}$=783.7kNm/m

$M_{Ed} < M_{Rd,lim} \Longrightarrow$ Moment is verified

A_s= 1157mm²

$A_{s,} < \Longrightarrow$ steel reinforcement section is verified

Provide 6ϕ16

$A_{s,provided}$= 1207mm²

S= 160mm

Smin= 25mm

$S_{min} < S$ so spacing is verified.

$A_{s,dis}$= 528mm²

Provide 5ϕ12

$A_{s,dis,provide}$= 566mm²

ii. For negative moment at A, M_{ED} = 211.15 kNm/m

$M_{RD,lim}$=783.7kNm/m

$M_{Ed} < M_{Rd,lim} \Longrightarrow$ Moment is verified

A_s= 1542mm²

$A_{s,\,min} < A_s \Longrightarrow$ steel reinforcement section is verified

Provide 8ϕ16

$A_{s,provided}$= 1609mm²

S= 110mm

$S_{min} < S$ so spacing is verified.

$A_{s,dis}$= 528mm²

Provide 5ϕ12

$A_{s,dis,provide}$= 566mm²

iii. For maximum negative at C, M_{ED}^- = 459.5 kNm/m

$M_{RD,lim}$=783.7kNm/m

$M_{Ed} < M_{Rd,lim} \Longrightarrow$ Moment is verified

A_s= 3356mm²

$A_{s,\,min} < A_s \Longrightarrow$ steel reinforcement section is verified

Provide 11ϕ20

$A_{s,provided}$= 3456mm²

S= 65mm

$S_{min} < S$ so spacing is verified.

$A_{s,dis}$= 864mm²

Provide 8ϕ12

$A_{s,dis,provide}$= 905mm²

3.4.2.3. Verification of shear reinforcement

a. For top slab

To design for shear, the section is divided into two as shown in figure 3.28. The procedure presented in the section 2.4.2.4 is used.

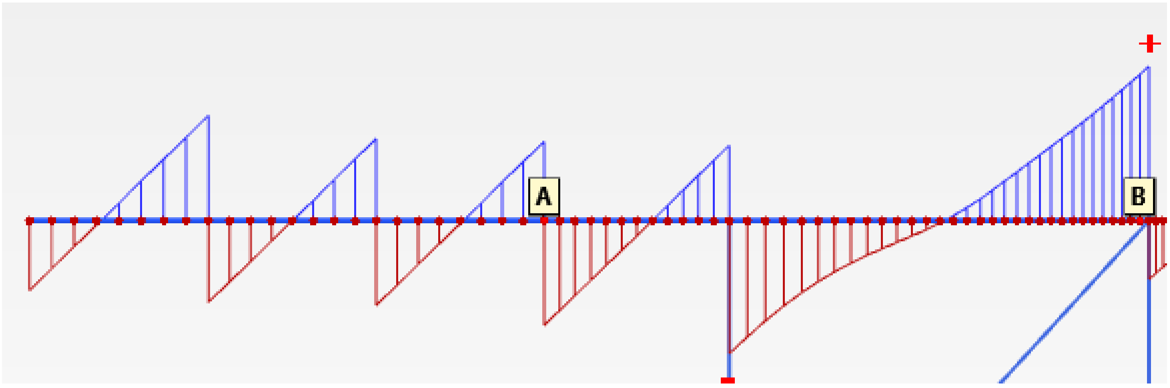


Figure 3. 28. Critical sections for shear on top slab

i. For maximum shear at B, V_{ED} = 548.9kN

$C_{Rdc} = 0.18/\gamma_c = 0.12$

$$k = \left(1 + \sqrt{\frac{200}{d}}\right) = \left(1 + \sqrt{\frac{200}{350}}\right) = 1.76 < 2$$

$$\sigma_{cp} = \frac{N_{ED}}{A_c} = \frac{121790}{1000*350} = 0.30$$

$$\rho_1 = \frac{A_s}{b_w d} == 0.01 < 0.02$$

We first need to calculate the shear resistance of the section.

$$V_{Rd,c} = \left[C_{Rd,c}k(100\rho_1 f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp}\right]b_w d \quad = 250.9\text{kN/m}$$

$$V_{Rdc,min} = \left[0.035k^{\frac{3}{2}}f_{ck}^{\frac{1}{2}} + k_1\sigma_{cp}\right]b_w d \ = 172.1\text{kN/m}$$

$V_{Rdc,min} < V_{Rd,c}$ is verified

Since $V_{Ed} > V_{Rdc}$, some steel frames will be needed to resist the demanding shear force;

It is first required to evaluate the angle of inclination θ between the cracks taking

$$V_{ED} = V_{Rdmax} = \alpha_{cw} b_w z v_1 f_{cd}(\cot\theta + \cot g\alpha) / (1 + \cot^2\theta)$$

$$\theta = \frac{1}{2}.\arcsin\left(\frac{2V_{Ed}}{\alpha_w * \gamma * Z * b * f_{cd}}\right)$$

$= 12.1°$

Since $\theta < 21.8°$, we take $\theta = 21.8°$

We take n=4 and ϕ=10 for steel frame

$n * \frac{A_{sw}}{s_t} = \frac{V_{Ed}}{0.9*d*f_{ydw}}$= 3.9

The Spacing between frames, S= 80.5

We take S= 80mm

$s_{\max} = 1.5d$ = 525mm

St $< s_{t,\max} \Longrightarrow$ Spacing is verified.

$$\#steelframe = \frac{\text{length}}{\text{spacing}} + 1 = 14$$

The same procedure is done for all the other shears and the results are obtained.

ii. For maximum shear at A, V_{ED} = 374kN/m

$V_{Rd,c}$ = 240.06kN/m

$V_{Rdc,min}$ = 161.9kN/m

$V_{Rdc,min} < V_{Rd,c}$ is verified

Since $V_{Ed} > V_{Rd}c$, some steel frames will be needed to resist the demanding shear force;

$$\theta = \frac{1}{2}.arcsin\left(\frac{2V_{Ed}}{\alpha_w * \gamma * Z * b * f_{cd}}\right)$$

= 8.1°

Since θ < 21.8°, we take θ= 21.8°

We take n=4 and ϕ=10 for steel frame

$n * \frac{A_{sw}}{s_t} = \frac{V_{Ed}}{0.9*d*f_{ydw}}$= 2.7

The Spacing between frames, S= 116.4

We take S= 115mm

St $< s_{t,\max} \Longrightarrow$ Spacing is verified.

$$\#steelframe = \frac{\text{length}}{\text{spacing}} + 1 = 10$$

b. For bottom slab

The design for shear is done in two sections as shown in figure 3.29.

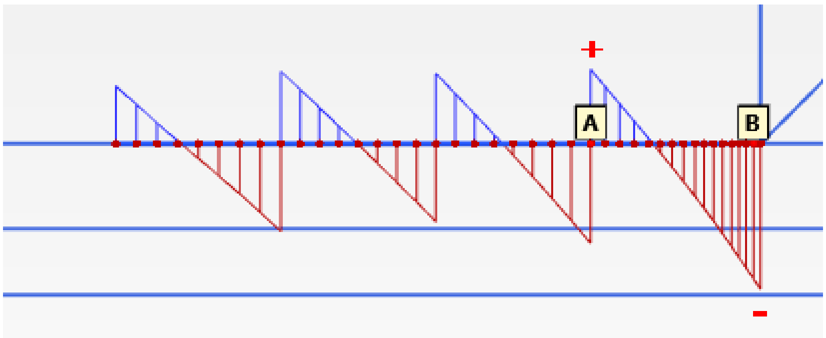


Figure 3. 29. Critical sections for shear on bottom slab

i. **For maximum shear at B, V_{ED}= 616.7kN/m**

$V_{Rd,c}$ =269.0kN/m

$V_{Rdc,min}$ = 190.2kN/m

$V_{Rdc,min} < V_{Rd,c}$ is verified

Since $V_{Ed} > V_{Rdc}$, some steel frames will be needed to resist the demanding shear force;

$$\theta = \frac{1}{2}.arcsin\left(\frac{2V_{Ed}}{\alpha_w * \gamma * Z * b * f_{cd}}\right)$$

= 13.7°

Since θ < 21.8°, we take θ= 21.8°

We take n=4 and ϕ=10 for steel frame

$$n * \frac{A_{sw}}{s_t} = \frac{V_{Ed}}{0.9 * d * f_{ydw}} = 4.5$$

The Spacing between frames, S= 69.8

We take S= 65mm

St $< s_{t,\max} \Longrightarrow$ Spacing is verified.

$$\#steelframe = \frac{\text{length}}{\text{spacing}} + 1 = 17$$

i. **For maximum shear at A, V_{ED}= 421.04kN/m**

$V_{Rd,c}$ =262.0kN/m

$V_{Rdc,min}$ = 183.3kN/m

$V_{Rdc,min} < V_{Rd,c}$ is verified

Since $V_{Ed} > V_{Rdc}$, some steel frames will be needed to resist the demanding shear force;

$$\theta = \frac{1}{2}.arcsin\left(\frac{2V_{Ed}}{\alpha_w * \gamma * Z * b * f_{cd}}\right)$$

$= 9.16°$

Since $\theta < 21.8°$, we take $\theta = 21.8°$

We take n=4 and ϕ=10 for steel frame

$$n * \frac{A_{sw}}{s_t} = \frac{V_{Ed}}{0.9*d*f_{ydw}} = 3.07$$

The Spacing between frames, S= 102.3 mm

We take S= 100mm

St $< s_{t,\max} \Longrightarrow$ Spacing is verified.

$$\#steelframe = \frac{\text{length}}{\text{spacing}} + 1 = 10$$

3.4.3. Design of the side wall

The side wall is designed for moment and shear reinforcement.

3.4.3.1. Design for moment reinforcement

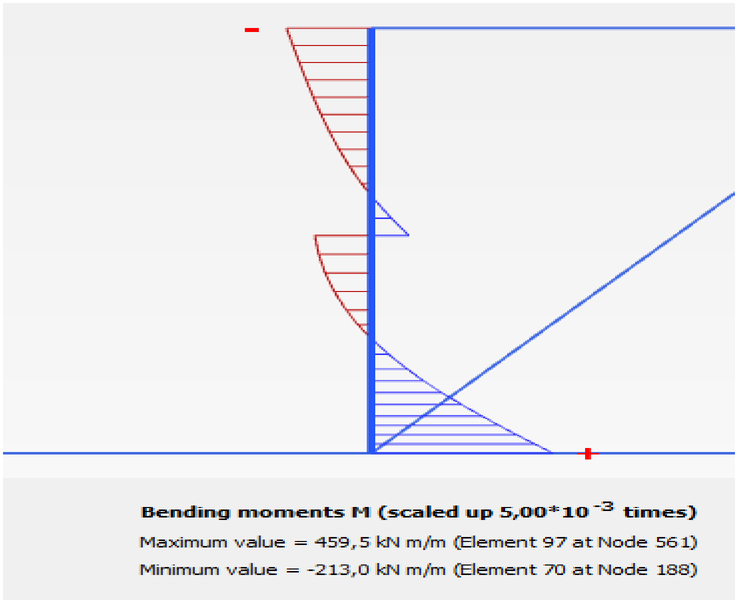


Figure 3. 30. Critical sections for moment on side wall

The design of moment for the side is done by dividing the wall into two sections as shown in figure 3.30.

a. For maximum positive, B, M_{ED} = 459.5kNm

Taking C_{nom}=50mm for calculations

$M_{Rd,lim}$=783.7kNm/m

$M_{Ed} < M_{Rd,lim} \Longrightarrow$ Moment is verified

A_s= 3356mm²

$A_{s,min}$=528mm²

$A_{s,} < A_s \Longrightarrow$ steel reinforcement section is verified

Provide 11ϕ20

$A_{s,provided}$= 3456mm²

The area of distribution steel is:

$$A_{s,dis} = max\{25\%A_s; 0.001A_c\}$$

$$= 864mm^2$$

Provide 8ϕ12

As,dis,provided=905mm²

b. For maximum negative,A, M_{ED}^- = 213kNm

A_s= 1556mm²

As,min=528mm²

$A_{s,} < \quad \Longrightarrow$ steel reinforcement section is verified

Provide 8ϕ16

$A_{s,provided}$= 1609mm²

S= 110mm

$$A_{s,dis} = max\{25\%A_s; 0.001A_c\}$$

$$= 402.25mm^2$$

Provide 4ϕ12

As,dis,provided=453mm²

3.4.3.2. Verification of shear

The shear is designed by dividing the side wall into four sections as shown in figure 3.31.

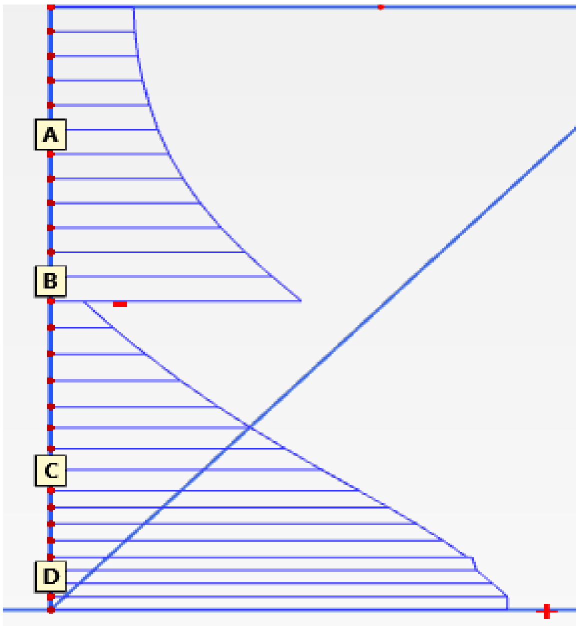


Figure 3. 31. Critical sections for shear on side wall

a. For shear at A, V_{ED}= 67.04kN/m

$V_{Rd,c}$ = 336.2kN/m

$V_{Rdc,min}$ = 257.4kN/m

$V_{Rdc,min} < V_{Rd,c}$ is verified

Since $V_{Ed} < V_{Rdc}$, no steel frames will be needed to resist the demanding shear force;

A minimum amount of steel frames is provided.

b. For shear at B, V_{ED}= 143.14kN/m

$V_{Rd,c}$ = 342kN/m

$V_{Rdc,min}$ = 263.2kN/m

$V_{Rdc,min} < V_{Rd,c}$ is verified

Since $V_{Ed} < V_{Rdc}$, no steel frames will be needed to resist the demanding shear force.

c. For shear at C, V_{ED}= 91.09kN/m

$V_{Rd,c}$ = 325.1kN/m

$V_{Rdc,min}$ = 246.3kN/m

$V_{Rdc,min} < V_{Rd,c}$ is verified

Since $V_{Ed} < V_{Rd}c$, no steel frames will be needed to resist the demanding shear force.

d. For shear at D, V_{ED}= 260.44kN/m

$V_{Rd,c}$ = 315.4kN/m

$V_{Rdc,min}$ = 236.6kN/m

$V_{Rdc,min} < V_{Rd,c}$ is verified

Since $V_{Ed} < V_{Rd}c$, no steel frames will be needed to resist the demanding shear force.

Table 3.12. Summary table for longitudinal steel reinforcement

		$A_{s,provided}$(mm^2)	Chosen section/ ml	$A_{s,dis,provided}$(mm^2)	Chosen section/ml
Top slab	Lower part of slab on left side	1002	$5\phi16$	566	$5\phi12$
	Upper part of slab on left side	2213	$11\phi16$	566	$5\phi12$
	Upper part of slab on right side	3770	$12\phi20$	905	$8\phi12$
	Lower part of slab on right side	3456	$11\phi20$	905	$8\phi12$
Bottom slab	Upper part of slab	1202	$6\phi16$	566	$5\phi12$
	Lower part at right edge	3456	$11\phi20$	905	$8\phi12$
	Lower part of interior part	1609	$8\phi16$	566	$5\phi12$
Side wall	Side retaining soil	3456	$11\phi20$	905	$8\phi12$
	Side not retaining	1609	$8\phi16$	453	$4\phi12$

Table 3.13. Summary table for transversal steel reinforcement

		Chosen section/ml	Spacing(mm)
Top slab	Slab on backfill	14ϕ10	50
	Slab not on backfll	10ϕ10	115
Bottom slab	At right edge	17ϕ10	65
	On the remaining slab	10ϕ10	100
Side wall	No shear reinforcement		

3.4.4. Detailing

The details of the steel reinforcements are shown below.

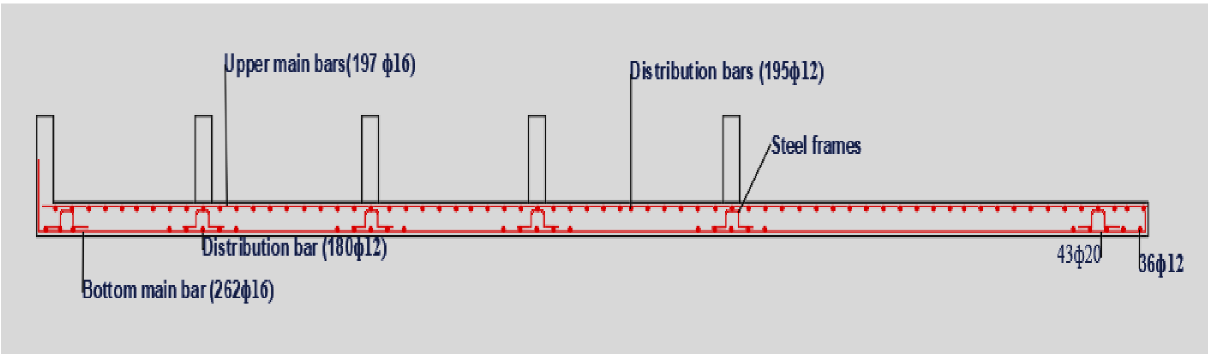


Figure 3. 32. Steel detailing of bottom slab

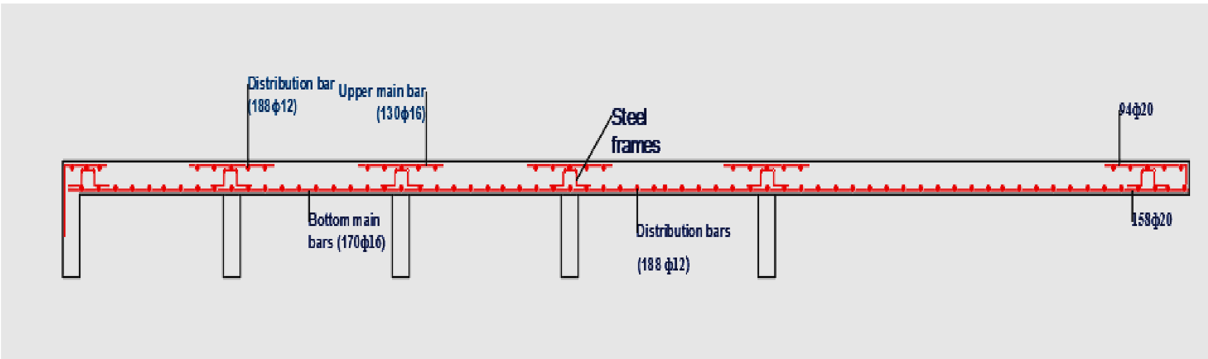


Figure 3. 33. Steel detailing of top slab

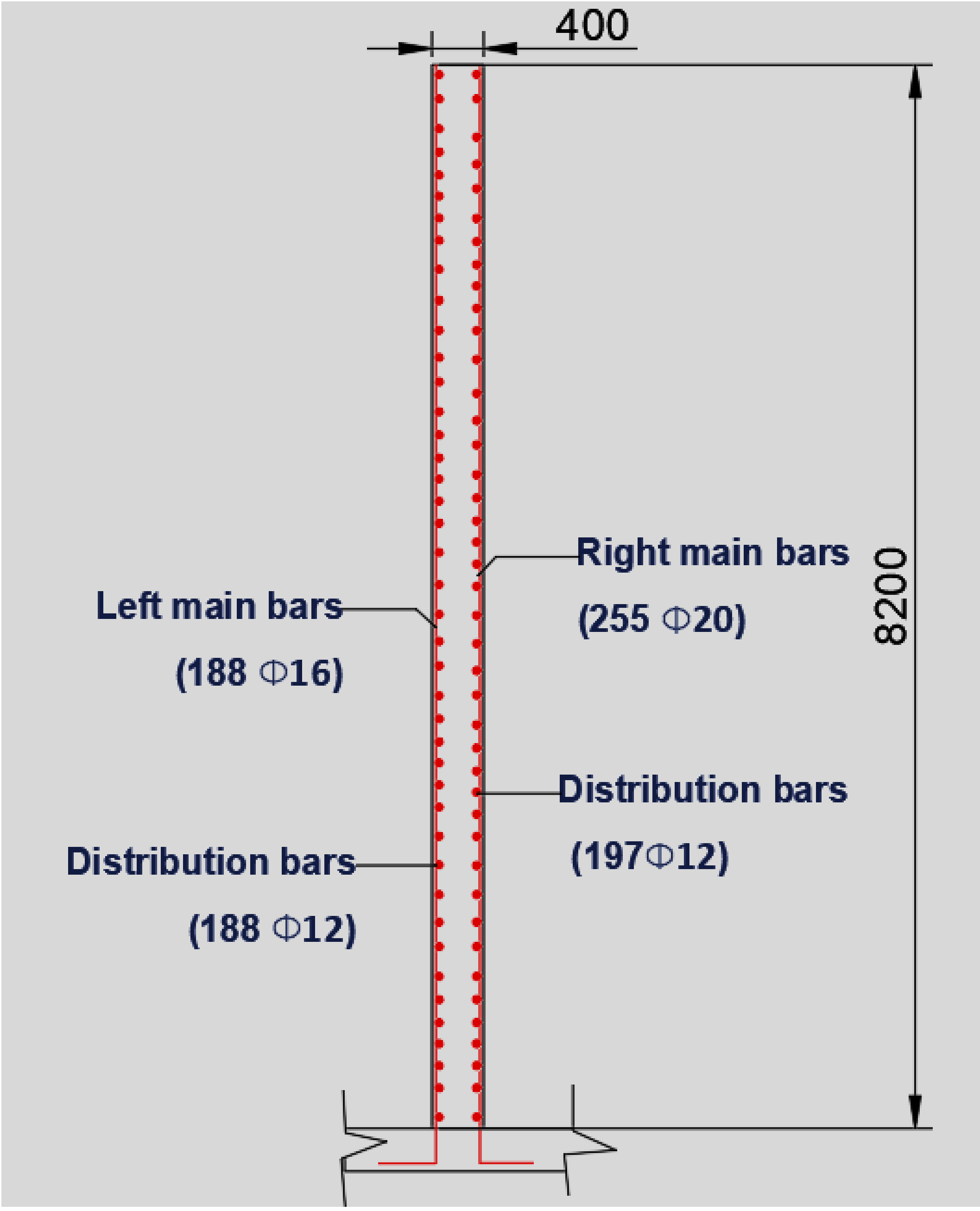


Figure 3. 34. Steel detailing of side wall

Table 3. 14. Cost analysis of construction of the retaining structure in Yaounde

Side wall		Top slab		Bottom slab	
Total number of rebars in right side of the wall(ϕ20)	255	Total number of rebars at the bottom (ϕ16+ϕ20)	170(ϕ16)+158 (ϕ20)	Total number of rebars at the bottom (ϕ16+ ϕ20)	262 (ϕ16)+43 (ϕ20)
Total number of rebars in left side of the wall (ϕ16)	188	number of rebars at the top (ϕ16+ϕ20)	130(ϕ16)+94 (ϕ20)	Total number of rebars at the top (ϕ16)	197
Total number of distribution rebars (ϕ12)	296	Total number of distribution rebars (ϕ12)	368	Total number of distribution rebars (ϕ12)	411
Total volume of concrete(m^3)	98.4	Total volume of concrete	240	Total volume of concrete	150
Total number of rebars of ϕ20	295				
Cost of 1 rebar of ϕ20	12,000 FCFA				
Total cost of rebars of ϕ20	3,540,000 FCFA				
Total number of rebars of ϕ16	759				
Cost of 1 rebar of ϕ16	9000 FCFA				
Total cost of rebars of ϕ16	6,831,000 FCFA				
Total number of rebars of ϕ12	779				
Cost of 1 rebar of ϕ12	5,950 FCFA				
Total cost of rebars of ϕ12	4,635,050				

Total volume of concrete	488.4 m^3
Cost of 1 m^3 of concrete	52,000 FCFA
Total cost of m^3 of concrete	25,396,800 FCFA
Total volume of gravel laterite	31 m^3
Cost of 1m3 of gravel laterite	1200 FCFA
Total cost of m^3 of gravel laterite	37,000 FCFA
Total cost of project	40, 439,850 FCFA

Conclusion

The main objective of this chapter was to present the results of the methodology adopted in the previous chapter to design an earth retaining structure working also as a floor slab for a building. The project at Bastos-Yaounde was presented through geographical location, climate, hydrology and hydrogeology, relief geology, population and socio-economic activities. This was followed by a description of the work area and the collection of data. Later on, the numerical analysis was made where our structure was modelled using PLAXIS 2D and at the end the total displacement of our structure was obtained to be small likewise the vertical and horizontal displacements which were within acceptable limits according to the EUROCODE. Therefore after doing all these verifications, we concluded that our structure is safe at ULS and SLS. Finally, the design for moment and shear reinforcement of the wall was done using formulae from EUROCODE to find steel sections that would resist to the loads and a cost analysis of the project was done.

GENERAL CONCLUSION

The main objective of this study was to design an earth retaining structure working also as a floor slab for a building. To achieve this objective, our study focused on 3 main parts; the first part concerned a comprehensive review of retaining walls, their various classification and types. The forces acting on retaining walls which include lateral pressures from retained soil and surcharge loads were also discussed. Different methods for their design of retaining walls were equally presented.

The second part was based on the presentation of the research methodology used to conduct this study. Geometric and geotechnical data were collected from project documents and a numerical analysis was performed on the modelling software PLAXIS 2D. The dimensions obtained from this analysis served as a predesign and the steps for the structural design were presented.

The third part dealt with the presentation of results obtained from data collection and results from numerical analysis. This was followed by the computation at ULS of the amount of steel reinforcement needed to resist to the bending moment and shear on the retaining structure. Finally, the results were interpreted and a cost analysis of the project was performed.

The results obtained from the analysis of our structure numerically all proved to be favourable meaning our design is in conformity with safety at ULS and SLS. Furthermore, the design of the wall at ULS to provide adequate steel reinforcement to resist to the applied loads was made.

From the results obtained, the following conclusions were obtained:

- The retaining structure working as a floor slab do not overturn nor slide due to its shape which makes it stable to applied forces.
- The retaining structure has acceptable settlements of 24mm when a micropile of diameter 0.25m is added below the raft foundation on the top of the backfill as support system.
- The global stability of the retaining structure with micropile is verified with a safety factor of 2.37.
- The retaining structure with a thickness of 0.4m is able to resist to lateral pressures from backfill, loads from new and existing building with appropriate steel reinforcement properly described in chapter 3.

- The choice of the backfill material has great influence on the stability of the wall. It's important to use coarse soils which are easy to compact and provide greater properties. They are also good for drainage.

Some limitations on this research include the fact that; project documents did not furnish enough information about the soil on our site so we had to reasonably assume certain data to facilitate our calculation. Also, the analysis was performed using Mohr- Coulomb as material model which represents a first order approximation of soil.

To improve this work, we can

- Adopt other material models for the Plaxis analysis. For instance, the Hardening Soil Model is more advanced and adapt for the simulation of soil behaviour because it describes much more accurately the soil stiffness variation with stress and strain.
- The design of the slabs may be considered in 2 directions and not as a one-way slab.
- If a minor displacement is required for particular needs of limiting deformation of the overstructure, other solution can be found to limiting the displacement of the slab: for instance, another micropile line parallel to the excavation may be added in the middle of the raft on top of the backfill or some concrete panels aligned perpendicularly to the excavation front could limit the vertical displacements, while an anchor connected to the lower part of the box retaining structure could limit the horizontal displacement or increase its overall stability.
- A drainage system could be added all around the structure to avoid water thrust. For instance, the provision of weep holes and geo-textile on the back-face of wall or the addition of perforated pipe draining system with filter.

The difficulties encountered in this work was due to the complexity of the design system, which was not easily handled with manual design, followed by the difficulties in getting exact costs for the realising the different support systems.

The present work could be completed and continued in various aspects. It would be relevant to extend this study with the use of adapted softwares or numerical tools considering the increase in complexity of the problem

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ANNEXES

ANNEX A: Tables for the methodology

Table A1. Exposure class related to environmental conditions

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

Table A2. Recommended structural classification

Structural Class							
Criterion	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2 / XS1	XD3 / XS2 / XS3
Design Working Life of 100 years	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2	increase class by 2
Strength Class [1) 2)]	≥ C30/37 reduce class by 1	≥ C30/37 reduce class by 1	≥ C35/45 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C40/50 reduce class by 1	≥ C45/55 reduce class by 1
Member with slab geometry (position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

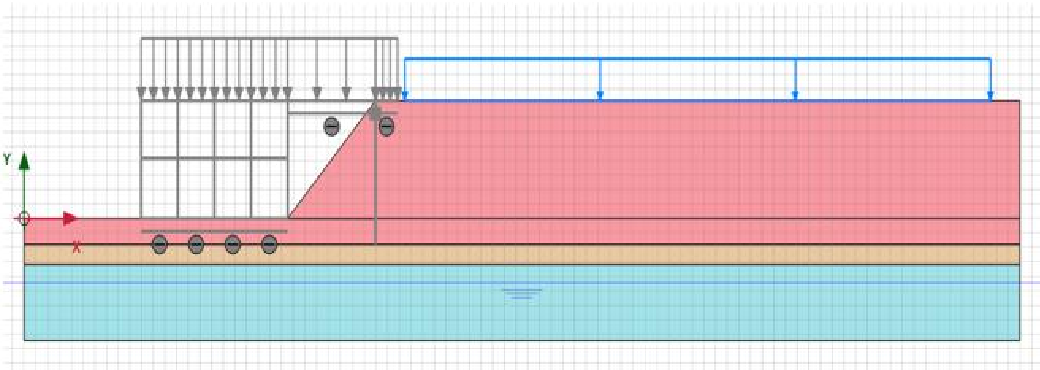
Table A3. Basic ratios of span /effective depth for reinforced concrete members

Structural System	K	Concrete highly stressed ρ = 1,5%	Concrete lightly stressed ρ = 0,5%
Simply supported beam, one- or two-way spanning simply supported slab	1,0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,3	18	26
Interior span of beam or one-way or two-way spanning slab	1,5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1,2	17	24
Cantilever	0,4	6	8

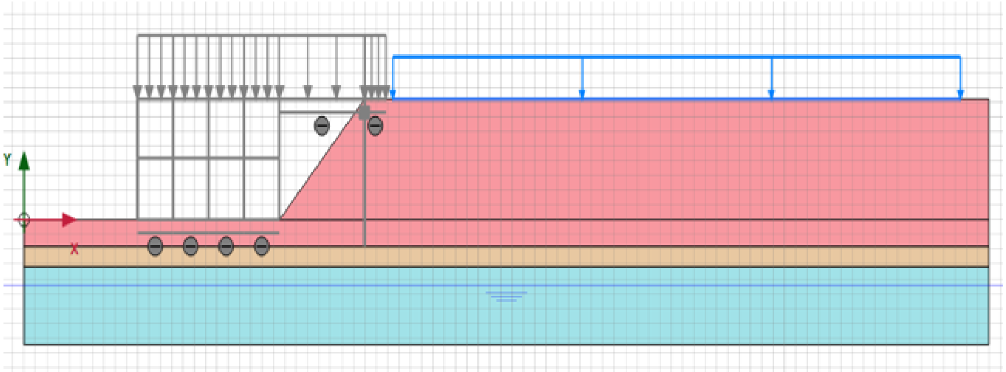
Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory.

ANNEXE B: Stages of construction in the numerical analysis in PLAXIS 2D

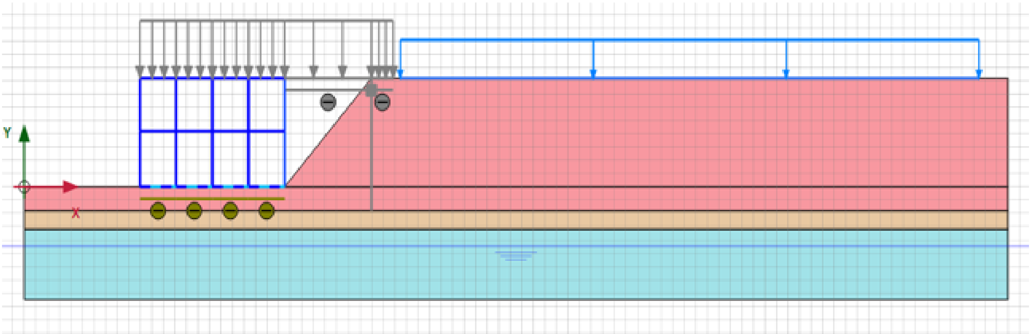
The screenshots of all the stages of construction are shown below.



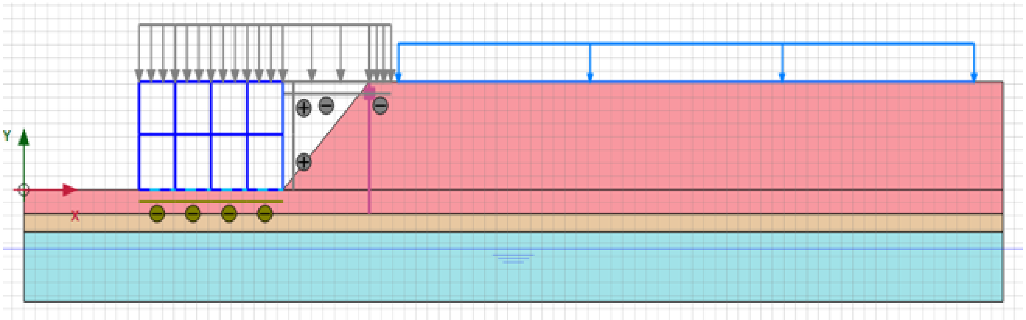
Initial phase of stage construction



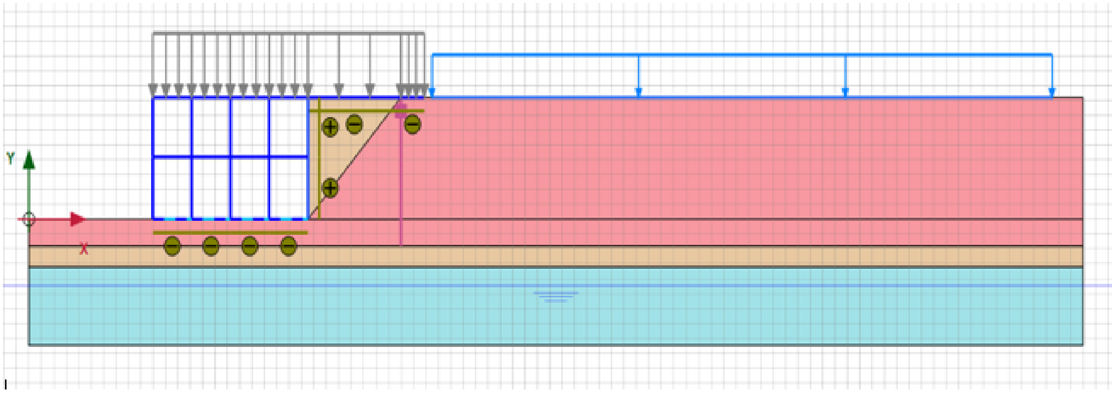
Phase 1 of the stage construction



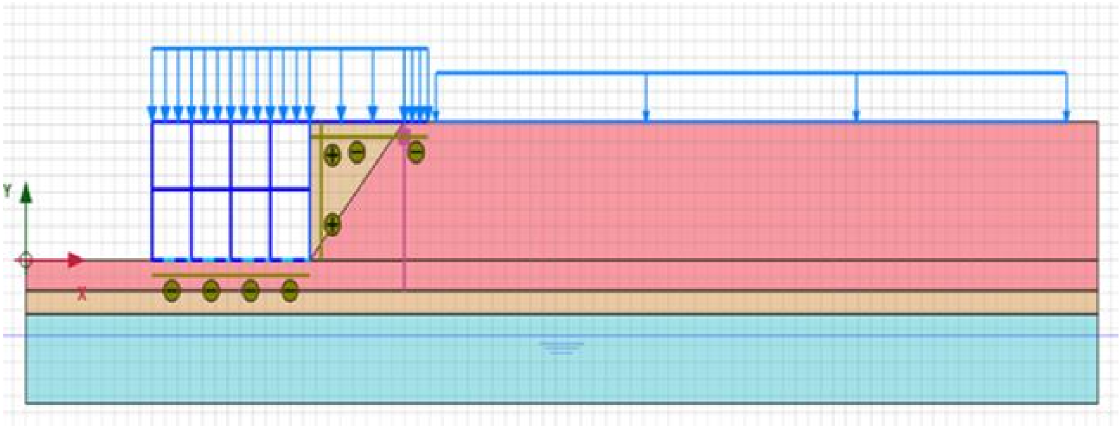
Phase 2 of the stage construction



Phase 3 of the stage construction



Phase 4 of the stage construction



Phase 5 of the stage construction

ANNEXE C: 3D simulation of project on ARCHICAD