

ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING



**Bored Soldier Pile Analysis and Design Practice in Addis
Ababa**
(Case of 4B+G+6 Podium and 15 floors Assem Building)

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Ababa**

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A Project Submitted to School of Graduate Studies in Partial Fulfillment
of the Requirements for the Degree of Masters of Engineering in
Geotechnical Engineering

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Declaration

I hereby declare that the project entitled Bored Soldier Pile Analysis and Design Practice in Addis Ababa (Case of 4B+G+6 podium and 15 floors Assem Building) is my work performed under the supervision of Dr. Tezera Firew and Mr. Leamlak Minwuyelet submitted to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the requirements for the Degree of Master of Engineering in Geotechnical Engineering. I further declare that this work has not been submitted to any other institutions for the award of any degree and all sources of materials used for the project have been fully acknowledged.

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List of Symbols and Notations

2D	Two Dimensional
FEM	Finite Element Method
MC	Mohr-Coulomb
HS	Hardening Soil
HSS	Hardening Soil Small
U_x	Horizontal Displacement
U_y	Vertical Displacement
K_o	Coefficient of earth pressure at rest, $K_o = 1 - \sin(\phi)$
E	Young's modulus
ν	Poisson's ratio
ϕ	friction angle
c	cohesion
ψ	dilatancy angle
f	yield function
E_{50}	triaxial loading stiffness
E_{oed}	Oedometer loading stiffness
E_{ur}	triaxial unloading stiffness
p^{ref}	Reference Pressure, $p^{ref} = 100$ kPa
σ_3'	minor principal stress
m	power value
G_o^{ref}	reference shear modulus
$\gamma_{0.7}$	Shear strain
γ	Bulk unit weight
γ_{sat}	saturated unit weight
q	Surcharge
EA	axial stiffness
Ls	out-of-plane spacing

EI	Flexural Stiffness
R_{inter}	Interface strength
H_e	Excavation depth
ΣP_{ux}	sum phase displacement
L_p	Length of the pile

Abstract

The purpose of this research is to show the benefit of the Finite Element Method over the Conventional Analysis Method for deep excavation design. This research is conducted on the Assem Building excavation, which has a 4B+G+6 podium and 15 stories and is located in Addis Ababa, Ethiopia, and is surrounded by busy roads and buildings. Following a comprehensive assessment of the literature on the analysis and design of excavation support walls, specifically anchored soldier pile walls, FEA and conventional analytic methods are used to model the excavation of the Assem Building.

In the conventional analysis approach, the Assem Building's excavation support walls were analyzed and designed using the DeepEX software program and the GEO5 "Sheeting Check" software tool. Similarly, the FEA method uses the Plaxis 2D software program. Similar model sizes and soil and structural element parameters are used throughout all software programs.

The following result was achieved by analyzing the Assem Building excavation. According to the FEA and optimized FEA results, the bending moments along the pile length are 40 % and 36 % lower, respectively, than those found using conventional analysis methods. Similarly, shear force calculated using FEA and optimized FEA are 41 % and 44 % lower, respectively, than those calculated using conventional analysis. The conventional method makes considerable assumptions about the solutions, but the FEA method makes no such assumptions and treats the problem as it is. As a result, the FEM result is lower than that of the conventional method. Because of the lower results, the FEA approach saves 9% to 11 % of the cost of the conventional analysis method. The optimized FEA also results in a cost reduction of 14 to 15%.

1 Introduction

1.1 Back Ground

In large cities, where structures with multiple basements are required for parking and other purposes. Where adjacent buildings are already functional, the excavation support system is an essential component.

Different calculation methods are now used in the design of the excavation support system in daily practices. The conventional design process implies repeated analysis of a structure and resizing of its members until a satisfactory design is obtained.

This research focused on three analysis methods namely; the Limit Equilibrium Analysis method, Beam on Elastic Foundation Analysis method, and the FEA method. Limit-equilibrium method is an iterative process in which we consider possible failure mechanisms and then apply the equilibrium equations for each failure mechanism to calculate the failure load. Beam on Elastic Foundation method based on Winkler method. The Winkler soil model assumes that the displacement appears only in the loaded zone. Outside this zone, the deflections are zero. This assumption leads to a discontinuous displacement field. The benefit of numerical methods is accounting for soil behavior and the soil-wall interface more precisely.

In the geosystem study, there are three common numerical methods. The Finite Difference Method (FDM), the Boundary Element Method (BEM), and the Finite Element Method (FEM) are among such methods. The Finite Element Method has developed into the most widely used numerical tool in engineering science to analyze a wide range of problems. It requires discretization of the domain of the problem.

In this research, the Mohr-Coulomb model was used to simulate stress-strain relationships to describe the general response of the soil to external and internal disturbances.

The "DeepEX software program" was used to perform Limit Equilibrium Analysis and Beam on Elastic Foundation Analysis. The GEO5 "Sheeting Check" software is another design software used for conventional analysis. The analyses for all methods focus on the wall's lateral deflection, bending moment, and shear force.

The conventional analysis method has an unrealistic structure deflection. That is critical for serviceability considerations, and the determined internal forces are over-estimated. As a result, the Finite Element Method is used for numerical modeling and analysis to assess the wall's structural response and behavior. PLAXIS 2D software program used to perform the FEA.

The present work is to study the behavior of anchored pile walls penetrating layered soils for the excavation depth of 12 m. Due to the practical application in the chosen project, the soldier pile walls used to support the excavation were reinforced by two rows of pre-stressed tieback anchors, with a diameter of 0.6 m and a pile length of 17 m. These tieback anchors are fixed at 30 degrees of inclination below the horizontal, with a surcharge load of 90 kN/m² (4.7m thick backfill soil above the top of the wall plus 10 kN/m² from the highway). This surcharge load is expected throughout the excavation and construction process.

1.2 Problem Statement

Due to the increased demand for underground infrastructure and basements in urban areas, cost-effective and safe retaining wall designs are more important. In Addis Ababa, the soldier pile retaining wall system is used, and most geotechnical offices use conventional analysis methods, which are not cost-effective.

The deflection obtained by the conventional analysis method is unrealistic. Which, is critical for serviceability considerations, and the computed internal forces are over-estimated. Since the development of software programs allow for improved simulation of actual field conditions, the Finite Element Method (FEM) is increasingly being used for retaining wall design. Even complex geometries and supporting systems can simulate using 2D or 3D FEM modeling.

1.3 Research Objectives

1.3.1 General Objective of the Study

The purpose of this research is to show the benefit of the Finite Element Method over the Conventional Analysis Method for deep excavation design.

1.3.2 Specific Objective of the Study

This study has the following specific objectives:

- determining the lateral deflection, bending moment, and shear force in an anchored pile wall at the final stage of excavation using a conventional analysis method and finite element method.
- comparing the finite element method with the conventional analysis method.
- making cost comparison.

1.4 Scope of the Research

The scope of this research is to compare the Finite Element Method with the Conventional Analysis Methods (Limit Equilibrium Method and Beam on Elastic Foundation Analysis Method); for the lateral deflection, bending moment, and shear force in an anchored soldier pile wall at the final stage of excavation. Finally, there will be a cost comparison.

1.5 Methodology

The following methodologies are followed to achieve the above objective:

- Literature Review
Some of the available literature reviewed are; Foundation and Earth Retaining Structures, Soil Models for Excavation Using Retaining Walls, Finite element analysis of deep excavation, Comparison of finite element approach with the conventional method, and Plaxis, DeepEx, and GEO5 software Manual.
- Data Collection
All necessary secondary data for modeling and analysis of excavation with retaining walls are got through investigation reports from laboratory tests and actual project analytical data.
- Software Modeling
The following software; PLAXIS 2D, DeepEX, GEO5 "Sheeting Check", and Microsoft Office are used to model excavation with retaining walls.

2 Literature Review

2.1 Introduction

In urban environments, deep excavations dig near existing structures. As a result, they frequently generate uncomfortable motions, which can affect the safety of nearby existing structures. Different methodologies or processes are employed to limit the impact of deep excavation on surrounding structures. The anchored wall system is one of the methodologies. The anchored walls can be; either discrete or continuous vertical elements that are either driven or drilled to depths below the finished excavation grade. The objective of this literature review, assess different analysis methods used to analyze discrete vertical elements wall systems and assess early works on discrete vertical elements wall systems.

Performance of deep excavation is related to both stability and deformation. A stable deep excavation is an excavation whose walls do not collapse. Ground deformations can damage adjacent buildings, streets, and utilities. The severity and extent of damage depend on the magnitude and pattern of ground movements around the excavation.

Prediction of deep excavation performance involves analysis of both stability and deformation. Experience has shown that excavation stability can be evaluated with sufficient accuracy using simple limit equilibrium calculations. Deformations are difficult to predict using the conventional method, and finite element analyses are often used for this task when ground movements are critical.

The literature review is divided into the following categories:

- anchored soldier pile walls,
- approaches to design and analysis of excavation support walls,
- related work,

2.2 Anchored Soldier Pile Wall

(Deep Excavation manual) Soldier piles and lagging walls, commonly known as soldier piles, are some of the oldest forms of retaining wall systems used in deep excavations. Soldier pile walls have been successfully used, since the late 18th century in metropolitan cities like New York, Berlin, and London. The method is also commonly

known as the "Berlin wall" when steel piles and timber lagging are used. Alternatively, caissons, circular pipes, or concrete piles can be soldier piles. But, with an increased cost. Timber lagging is typical, although reinforced concrete panels can be used, for permanent conditions. Soldier pile walls are formed by:

1. Constructing soldier piles at regular intervals (1.8m to 3.6m, typical)
2. Excavating in small stages and installing lagging.
3. Backfilling and compacting the void space behind the lagging

Moment resistance in soldier piles and lagging walls is provided solely by the soldier piles. Passive soil resistance is from embedding the soldier piles beneath the excavation grade. The lagging bridges retain soil across the pile wall and transfer the lateral load to the soldier pile system.

2.2.1 Advantages & Disadvantages of Soldier Pile Walls

Soldier pile and lagging walls are the most inexpensive systems compared to other retaining walls. They are also easy and fast to construct.

The advantages of soldier pile walls are:

1. They are easy and fast to construct.
2. Construction is cheaper when compared to other systems.
3. Installation is changeable, and adjustments can make in the field. It can easily accommodate changes.
4. Lagging construction can be quick.
5. The construction of soldier piles and lagging walls does not require very advanced construction techniques.

Common lagging materials include timber, shotcrete, precast concrete panels, or steel plating. Permanent walls usually utilize precast concrete panels, while temporary soldier pile walls in the US utilize timber lagging.

Among the disadvantages;

1. They are primarily limited to temporary construction.
 2. They can't used in high water table conditions without extensive dewatering.
-
-

3. Poor backfilling and associated ground losses can result in significant surface settlements.
4. They are not as stiff as other retaining systems.
5. Because only the flange of a soldier pile is embedded beneath the subgrade, it can't control basal soil movements.

Soldier pile and sheet pile walls over 4.5 m height typically require additional lateral resistance to maintain stability and limit wall movements. This lateral resistance gets from ground anchors. Anchor spacing is decided by; including anchor capacity, unsupported cut slope stability, subsurface obstructions in the anchorage zone, and the strength capacity of lagging or facing elements. Proof testing is performed on every anchor.

2.2.2 Soldier Pile Retaining Wall- Components and Design

(The Construction Encyclopedia) where the earth's backfill is needed to support, a soldier pile retaining wall can use. The soldier pile wall consists of soldier piles and lagging materials. The former is either forced into the ground to an adequate depth or installed into holes drilled in advance, and the spacing is commonly between 1.8 m to 3.6 m. The latter is installed horizontally, after excavation is completed and supported by soldier pile flanges at its ends. Mostly soldier piles are cantilevered, but sometimes tie-back anchors be used, when the wall height is greater than 4.5 m, and enough space is available for using the ties to decrease embedment depth and size of the beams.

Components of Soldier Pile Retaining Wall

The Components of the Soldier Pile Retaining Wall are presented in Figure 2.1.

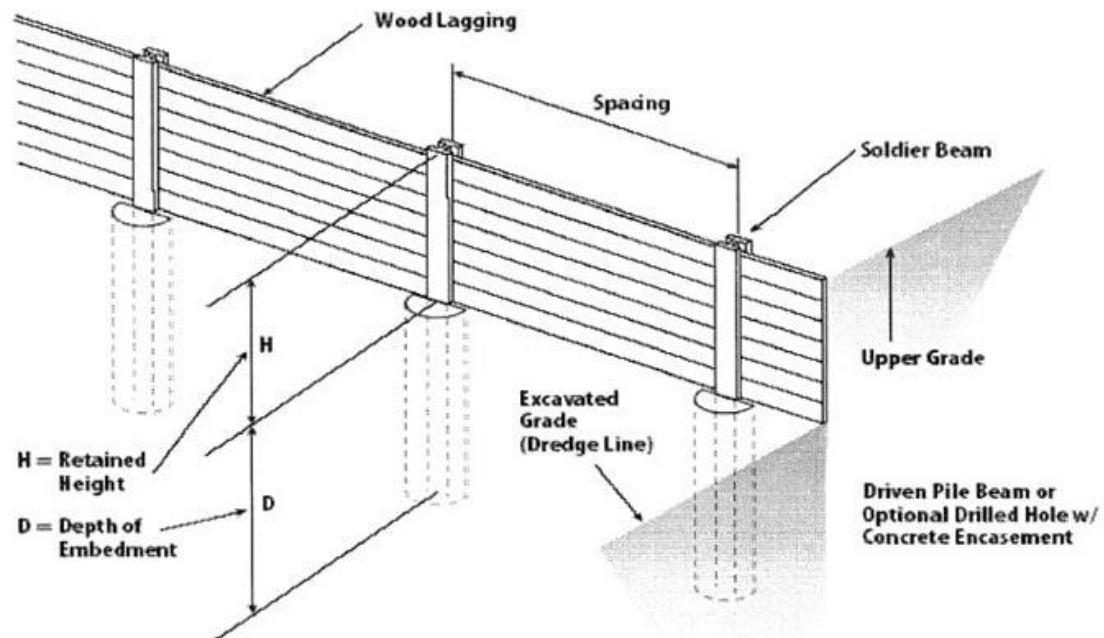


Figure 2. 1: Illustrates different parts of the soldier pile retaining wall.

Design of Soldier Pile Retaining Wall

Design Procedures:

In the design methodology, certain assumptions are. Firstly, the soil is non-cohesive or sandy. Secondly, this procedure is applicable for cantilevered soldier pile walls.

1. Compute forces from construction surcharges and pressure of active soil tributary to each soldier pile. The Rankine formula can be used to calculate the Active Pressure coefficient (K_a).
2. Spacing of the piles is determined to depend on beam size, embedment depth, and choice of lagging. Several design calculations may be necessary to determine the best spacing option.
3. Determine embedment depth after both surcharge (P_w) and active pressure (P_a) are found. Figure-2.2 shows different forces under which the cantilevered soldier pile in sandy soil, Embedment depth (d) of the pile wall, using equation-1.

$$d = \sqrt{\frac{2 \cdot P_p \cdot SF}{P \cdot D \cdot A}} \rightarrow \text{Equation-1}$$

Where: d : is the effective depth at zero shears.

P_p : is the force that opposes and is equal to $(P_w + P_a)$

SF: is the Safety factor used for allowable passive pressure

P : is the Allowable passive pressure

D : is the hole diameter or flange width whichever is utilized.

A : arching factor multiplier (*taken as $0.8 \leq 2.5$*)

4. Calculate maximum beam moment by taking moment summation above zero shear point. Equation-2 can be used to calculate.

$$M_{max} = P_a(0.33H + 0.67d) + P_w(0.50H + 0.67d) \rightarrow \text{Equation-2}$$

5. The maximum moment is opposed by a passive pressure couple consisting of $0.67D \cdot F_p$, so the required depth is calculated as:

$$D = \sqrt{\frac{\text{Maximum moment} \cdot SF}{(P \cdot D \cdot A \cdot D + 0.25) \cdot 0.67}} \rightarrow \text{Equation-3}$$

And required embedment depth which is denoted as (E) in Figure-2.2, is combined with $(D+d)$, and as a rule of thumb, it can be taken between $1.3H$ to $1.5H$.

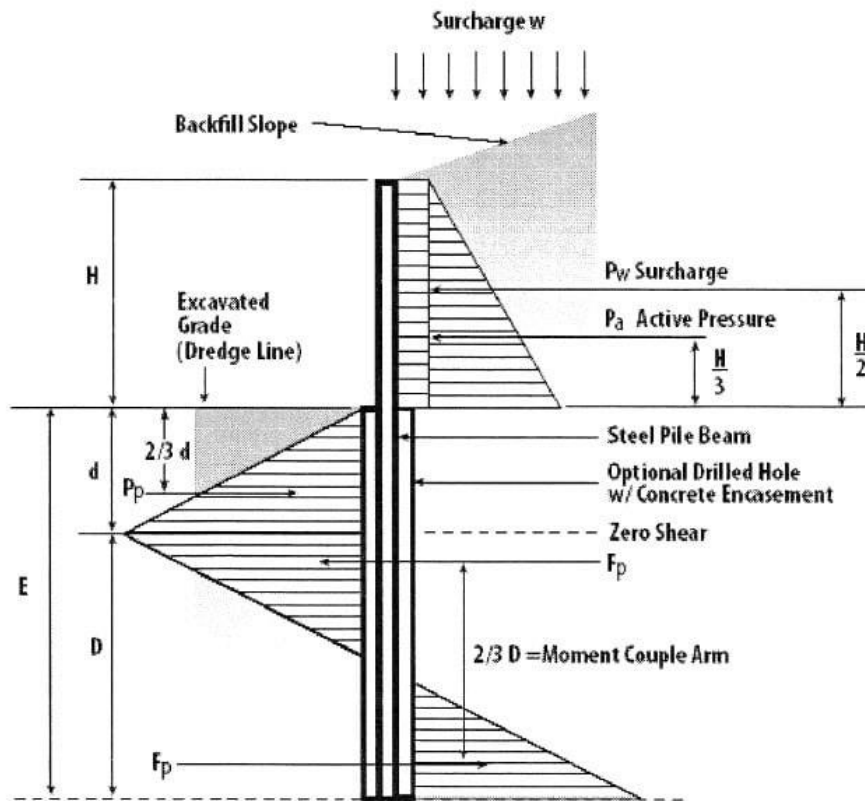


Figure 2. 2: Various forces acting on cantilever soldier pile on non-cohesive soil

6. Load resistance factor design (LRFD), is maximum moment times load factor of 1.6, is used to select various beam options from the AISC 13-edition, LRED, steel design handbook after choosing the most economical and available beam.
7. Choose lagging which must be treated wood, and conservative fiber stress is 6.2 MPa. Lateral earth pressure is decreased over the wall height from the base to the top. Therefore, the different lagging thicknesses can be used at various depths. Due to soil arch action between beams, 80% of the simple moment of each lag can be used, in addition to providing 25 cm space between lags for allowing water drainage.

Using Tiebacks in Soldier Pile Retaining Walls

When the wall height exceeds 4.5 m, and adequate space is available, tiebacks can be used. Not only is tieback utilization declining the size of soldier piles but also the embedment depth. Generally, tiebacks are steel rods placed into a 7.62 mm drilled hole into the backfill at an adequate distance that exceeds the failure plane for achieving

anchorage after grouting. Tiebacks are inclined downward at 15° to enhance pulling out resistance and facilitate grouting. The ends of tiebacks are welded to steel beams. Figure-2.3 shows forces acting on soldier piles when applying tiebacks.

Moreover, the embedment depth should adequately withstand the applied lateral forces P_w and P_a . Finding (d) is an iterative process; the value by which active and passive balance will be (d). than the maximum bending moment computed by applying statics.

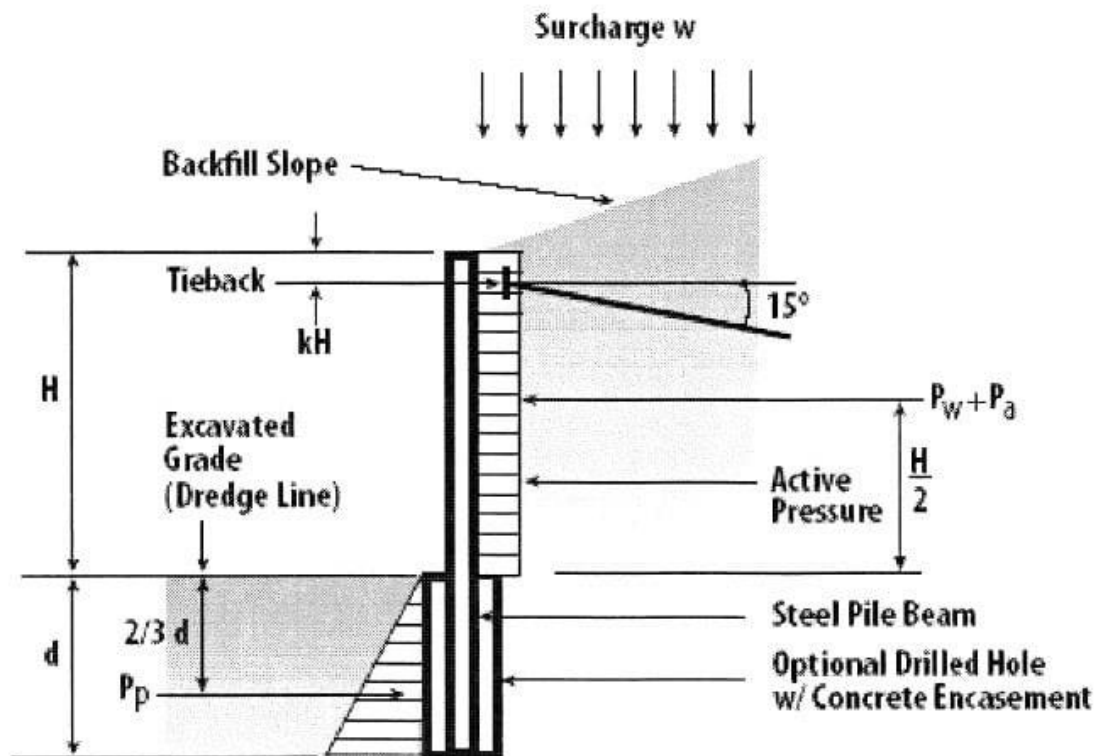


Figure 2. 3: Forces Acting on Soldier Pile after using Tieback

2.3 Approaches to Design and Analysis of Excavation Support Walls

Major approaches to designing and analyzing excavation support walls are: (a) conventional methods and (b) finite element methods

2.3.1 Conventional Methods

Focus on two analytical methods, namely, the limit equilibrium analysis method and beam on elastic foundation analysis method limit-equilibrium method is an iterative process in which we consider possible failure mechanisms and then apply the equilibrium equations for each to calculate the failure load.

2.3.1.1 Limit Equilibrium Analysis Method

(John Krahn (2001)) Limit equilibrium types of analysis to assess stability in geotechnical engineering for decade, specially for stability analysis of earth slopes. Discretizing a potential sliding mass into vertical slices was introduced early in the 20th century. During the next couple of decades, Fellenius (1936) introduced the Ordinary or Swedish method of slices in the mid-1950s, Janbu (1954) and Bishop (1955) developed advances in the method. The advent of electronic computers in the 1960s made it possible to more readily handle the iterative procedures inherent in the method, which led to mathematically more rigorous formulations such as those developed by Morgenstern and Price (1965) and by Spencer (1967).

The introduction of powerful desktop personal computers in the early 1980s made it economically viable to develop commercial software products based on these techniques. And the ready availability today of such software products has led to the routine use of limit equilibrium stability analysis in geotechnical engineering practice.

Limit equilibrium methods of slices are routinely used for analyzing the stability of slopes and embankments. And to examine the stability of anchored walls and landslide stabilization systems. However, the current state of the practice does not include a generally accepted method of modeling the restraint force provided by the prestressed ground anchors. LEM used in practice distributes the anchor forces to slices, and each slope stability computer program includes one or several of these methods. For this reason, caution when using limit equilibrium methods to calculate the required forces to restrain a slope. Calculated force should be reviewed critically and compared to solutions based on “hand-calculation” methods.

2.3.1.2 Beam on Elastic Foundation Analysis Method

(Doromir Dinev (2012)) The elastic modeling of the soil bed is based on an assumption for the behavior of the subgrade reaction under loading. The elastic subgrade reaction, is represented by one parameter Winkler analog subsoil model. The contact soil is replaced by a system of independent elastic support of stiffness K_h . The wall is considered as an elastic beam of unit width, and the value of the horizontal elastic soil reaction at an examined point is directly proportional to horizontal wall displacement at the same point as is illustrated by the equation below:

$$P_z = K \cdot y \leftrightarrow K = \frac{P_z}{y}$$

Where P_z is the pressure stress at depth z and y is the horizontal displacement.

The Winkler soil model assumes that the displacement appears only in the loaded zone. Outside this zone, the deflections are zero. This assumption leads to a discontinuous displacement field. And this is the main disadvantage of the Winkler model.

(Lourenço D., Schnaid F. & Rocha M.M.) Winkler Spring Model (WSM) is iterative, and the soil is represented by one-dimensional elastic-plastic spring elements, as shown in Figure 2.4(b). The yielding point of the elastic- perfectly plastic springs is related to the active and passive limit soil pressures. The wall is represented by linear beam elements, with two degrees of freedom per node, horizontal displacement, and in-plane rotation. Deformations by axial compression are not taken into consideration. The discretization length is 5cm, ensuring an adequate accuracy level for the problem of a floating concrete wall behavior that serves as a restraint for excavations.

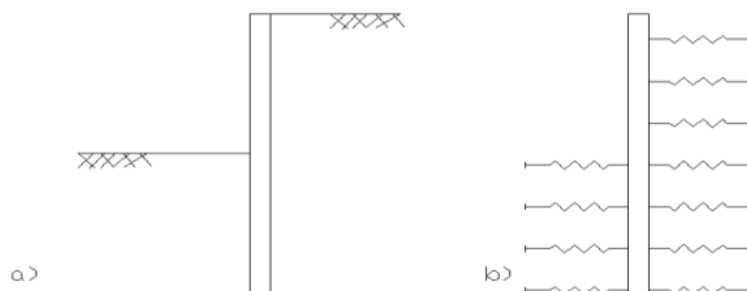


Figure 2. 4: Representation of a RW an excavation: a) real situation, b) Winkler Model

The calculations for the wall deformation process are iterative and summarized below.

Firstly, the soil pressures along the wall are estimated, as well as the resulting horizontal forces and the active and passive limits. The soil stiffness, represented by the Winkler springs, is added to the diagonal of the wall stiffness matrix, and the horizontal loads are to the corresponding degree of freedom in the load vector. The system is solved, and the wall displacements are known. Once these displacements are known, the elastic-plastic constitutive relations of the springs are checked out. If the plastic limit is reached, the spring stiffness is decreased to keep the soil pressure within the active or

passive limits. Hence the system is solved again, iteratively, until it converges. If the convergence does not occur, equilibrium is not possible, and the system is unstable.

2.3.2 Finite Element Method

(Bin-Chen Benson Hsiung and Sy-Dan Dao, 2014) Commercial FEM software such as Plaxis and Midas GTS, designed primarily for geotechnical problems, are now widely utilized to investigate the behaviors caused by deep excavations. Many constitutive soil models have been awarded in recent decades, ranging from a linear elastic model to non-linear elastoplastic cap models. However, utilizing the numerical analysis to predict movements generated by deep excavations has a challenge.

Geometry and boundary conditions, mesh generation, constitutive models, and input parameters all play a role in the accuracy of numerical analysis. Understanding the numerical methods and constitutive models of soil is critical for obtaining accurate findings from numerical analysis.

(Arjun Gaur, 2017) Plaxis enables thorough modeling of this particular problem type. It incorporates simulating the anchored retaining wall as well as the pre-stressing condition. Furthermore, the dry excavation includes calculations for groundwater flow (if any), displacements, stresses, ϕ - c reduction, and other factors which discover new water pressure distributions, deformations, and forces.

2.3.3 Comparison of Design Methods

Although classical methods are quick, a computer-aided analysis for complex soil and site conditions is required. The computer-aided analysis type is advanced in use and represents the future trend. The comparison of the design methods is in Table 2.1.

Table 2. 1: Comparison of the Design Methods

Method	Data Required	Advantages	Disadvantages
Limit Equilibrium	C' or S_u , ϕ' , γ , m, Eoed, Eur, Wall Section Properties, Wall Position, and Depth. Support Section Properties and Elevations.	<ul style="list-style-type: none"> • They form a well-known analysis method • can provide quick answers • The assumed limit conditions are well understood 	<ul style="list-style-type: none"> • The analysis is valid only for simple conditions • Soil structure interaction is not properly captured • Calculated wall displacements are unrealistic for excavations with multiple bracing levels • Computed wall BM is possibly unconservative when more than one support level is used • Construction stage history is ignored
Beam on Elastic Foundation based on Winkler's approach	the wall length and thickness, the soil parameters, the elevation of the bottom of each soil layer, excavation depth, specific weight, friction angle, cohesion intercept, soil elastic stiffness per area, K_{Es} , and the dimensionless coefficients K_{wA} (Winkler active coefficient) and K_{wP} (Winkler	<ul style="list-style-type: none"> • Helps to understand the soil-structure interaction phenomena and predict the contact pressure distribution and deformation within the soil. 	<ul style="list-style-type: none"> • Discontinuous displacement field • Deformations by axial compression are not taken into consideration

	passive coefficient).		
Finite Element	$\gamma_{unsat}, \gamma_{sat}, c, \phi, \psi, E_s, \nu, R_{inter}, E_{pile}, EA$	<ul style="list-style-type: none"> • Simulates construction sequence. • Good bending moment predictions. • Good movement predictions. • Unlimited geometry. • Multilayer. • Any loading. 	<ul style="list-style-type: none"> • It requires a longer execution time. • It requires a digital computer and is fairly extensive. • The output result will vary considerably.

2.4 Related work

F. A. Villalobos and P. L. Oróstegui are analyzing the design and construction of an anchored Soldier Pile Wall (SPW) for a big underground car park in the city of Concepción, the capital of the Bo Bo region in Chile's south. Before digging, they employed the construction technique to drive steel H section profiles (soldier piles) into the soil.

2.4.1 The Tribunals Excavation Support Project

The building has a quarter circle shape and is a reinforced concrete structure with masonry confined shear walls which also can be considered structural elements.

The soil in the project region primarily consists of silty sands SM with no plastic fines. Table 2.2 shows the geotechnical properties assumed in the project. For the active and passive sides, the effective soil-wall interface, soil angle of internal friction to be $\delta'/\phi' = 2/3$. According to the Hazen formula, the permeability coefficient is 10^{-5} m/s.

Table 2. 2: Values of the soil parameters

Soil	h	γ	γ'	G_s	ϕ'_{cr}	ϕ'_{max}	DR	C	$(N_1)_{60}$
	m	kN/m ³	kN/m ³				%	kN/m ²	
Fill	0-2	17.5	7.5	2.6	30	30	45	0	15
SM	2-7	17.5	7.5	2.8	33	34	60	0	18
SM	7-16	20.7	10.7	2.8	34	37	82	0	36

In an excavation, for example, 10 m wide and 3 m deep, it is highly likely that deformation calculations result in large horizontal movements of the soil, particularly close to the surface. Due to the inherent flexibility of this type of support system, even with relatively rigid H sections, they are not stiff enough to control horizontal displacements. Anchors can be incorporated, in SPWs to solve this problem, which is not related to stability nor the capacity to hold the excavation but to reduce soil deformations.

When the water table is high, anchored SPW is Diffecult (EAB 2008). The water table should ideally be below the SPW. However, when groundwater is removed, such as through well points, it is possible to accept the presence of a certain level of groundwater while limiting sediment transport.

SPWs provide enough space between timber laggings to allow the flow of groundwater. In case, gaps between timbers do not let pass easily the groundwater drains should be installed, perforating holes in timbers. The idea is to avoid any build-up of pore water pressure behind the SPW, which could add hydrostatic or hydrodynamic lateral pressure and result in undesirable deformations. Using well points to lower the water table in case of seepage behind the SPW avoids flooding and transportation of soil to the excavation.

2.4.2 Static loading conditions on SPW

The lateral earth pressure on an SPW has very little chance be at rest. Since soil deformations are highly likely to occur, the soil is not at rest. A mobilized condition should be assumed, between the at-rest and the active lateral earth pressure condition. Sowers (1979) proposed that, an active lateral earth pressure develops when the maximum horizontal displacement u_{hmax} on top of a rigid wall of height h is $u_{hmax} \geq 0.002h$ in loose granular soils and $u_{hmax} \geq 0.0005h$ in dense granular soils. In case of, anchored flexible walls, the estimation of any lateral earth pressure will depend strongly on the anchor's pre-stressed loads.

Below the excavation level, passive pressures apply in front of the wall from the bottom level of the excavation to the end tip of the H section piles. Passive pressure develop for maximum horizontal displacements uh_{max} an order of magnitude less than that for

active pressure; $u_{hmax} \geq 0.01h$ in loose granular soils and $u_{hmax} \geq 0.005h$ in dense granular soils (Sowers 1979).

In addition to earth lateral pressures, dead and live loads can act as constant or variable loads. The EAB (2008) recommendations consider a uniform distributed load over the surface of 10 kPa, representing the effect of live loads on the pavement and street. The SPW calculation procedure follows the construction sequence. EAB (2008) suggests that if the height from the bottom of the future excavation to the support line is h , then the anchors or struts should be installed at $h/3$ from the bottom of the current excavation, leaving a distance of $2h/3$ between the current and future excavation (see Figure 2.5).

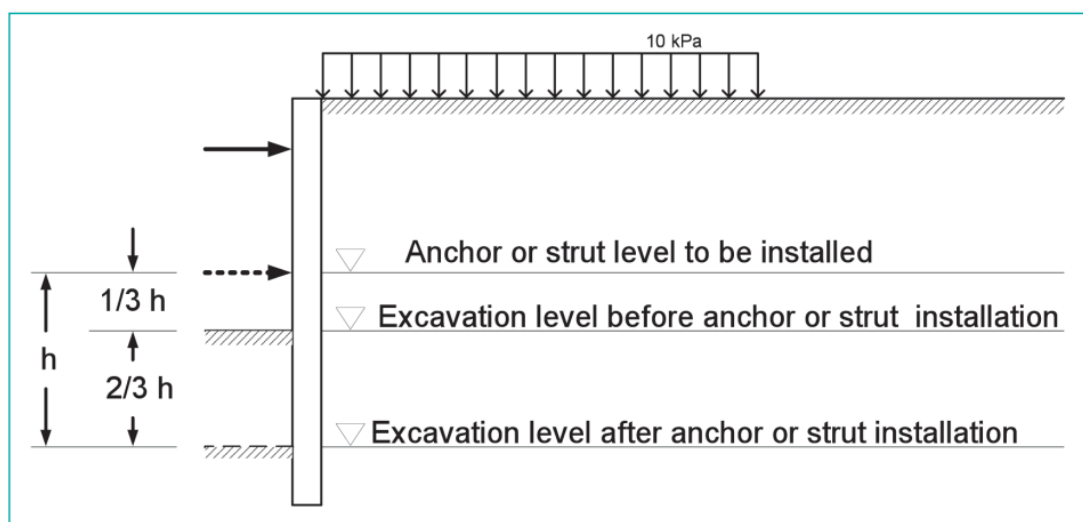


Figure 2. 5: Excavation limit before installing anchors or struts (EAB 2008)

They considered horizontal forces equilibrium within the height of the excavation for the excavation support design using an SPW, and from horizontal force equilibrium analysis plus moment equilibrium analysis, the embedding depth of the H section soldier piles is determined.

Design of Anchors

GGU-RETAIN calculates anchor loads and the required anchor free length to ensure SPW stability, the grouting length, and the number of cables in the anchor.

The anchor allowable load T_a was determined using the following expression,

$$T_a = n A_c f_y / FS$$

Where n is the number of cables, A_c is the area of each cable, f_y is the cable yield stress, and the factor of safety $FS = 1.5$. Table 2.3 resumes the cable technical characteristics for the post-stressed anchors installed in the project.

Table 2.4 shows the resulting anchor allowable load as a function of the number of cables. Table 2.4 and the values are shown in Table 2.5 calculate the number of cables required for each anchor.

Table 2. 3: Anchor cable properties (ASTM 416, GRADE 270)

Parameter	Value
Cable diameter D , mm	15.2
Cable area A_c , mm ²	140
Yield stress f_y , MPa	1670
Characteristic ultimate load T , kN	250
Characteristic yield load T_y , kN	235

Table 2. 4: Allowable load versus the number of cables

No. of Cables	Allowable load, kN
2	313
3	470
4	627
5	783
6	940

Table 2. 5: Anchor design using program GGU-RETAIN (Lancuyen 2008)

T _o kN	L m	L _s m	β °	buildings	D _f m
350 280	12.5 8.5	8 4	30 25	Fiscalia, Tucapel St	0
410 300	12 8.5	7.5 4	40 30	Hites	5
370 480	12.5 11	8 6.5	30 25	Entrances INP	1.5
450 325	11.5 9	7 4.5	45 35	INP	5
350 330	12.5 9	8 4.5	30 25	Tribunals	3
330 520	13 12.5	8.5 8	30 25	Tribunals	3
400 300	12.5 8.5	8 4	35 25	Tribunals	5.5
370 480	12.5 11	8 8.5	30 25	Barros Arana St	1.5

To verify the design loads taken by each anchor load test carried out in the first and the second row. The anchors had three steel cables. The maximum capacity is 90% of the steel yielding load.

2.4.3 Stability Analysis Following the Construction Sequence

A construction sequence that represents

- excavation without anchors,
- the first row of anchors at 3 m for a 6 m excavation, and
- the final two rows of anchors at 5.5 m for an 8 m excavation.

2.4.4 Discussions and Conclusions

It concluded that since no significant disturbance in terms of cracks or damage to neighboring structures. The anchored soldier pile wall offered an adequate solution for the support excavation required for a large underground car park.

However, it is believed that during the car park construction, the anchored soldier pile wall may have not been able to resist adequately the inertial forces imposed by the earthquake on 27th February 2010, its magnitude of 8.8 moments. As a result, large displacements of the soldier pile wall may have occurred owing to seismic lateral and vertical earth pressures, inducing serious damage to the buildings. They suggested that build a diaphragm wall instead of a soldier pile wall. Diaphragm walls are much stiffer, becoming the final walls of the structure, and therefore can offer a better response under seismic loads

The car park project contemplated 3596 m² of anchored SPW with 314 post-tensioned anchors totaling 3784 m under loads between 300 kN and 560 kN and 300 H section soldier piles totaling 3200 m. Once the definitive parking foundations, walls, and slabs are built and can resist the lateral earth pressures, anchors are distressed and the SPW lies buried with the H section piles and the timber laggings, except for the wailing. The final reinforced concrete walls and slabs stay in contact with the H piles of the SPW, assuring the transfer of loading from the retaining structure to the new and definitive structure.

3 Project Background

3.1 Project overview

The deep excavation investigated in this research was in Addis Ababa, Ethiopia, the site surrounded by busy roadways and buildings, see Figure 3.1.

The project excavation has two portions, Region A and Region B, with a total area of 2,694 m² (See Figure 3.2). Region A and Region B had average excavation depths of 12.0 m and areas of 1,630 m² and 1,064 m², respectively. Buildings for condominiums had built on the north side of the deep excavation, with a minimum distance of 13 m between the project excavation and the condominiums buildings. 15 m west of Region

A is a building with seven floors above ground and one basement. On the south side of the deep excavation lies the 40 m wide main streets that connect Mexico Square with Torhayloch.



Figure 3. 1: Location Map.

The pre-stressed tieback anchors hold the retaining pile walls used for the deep excavation. Mesh reinforcement and shotcrete reinforce the soils between the piles.

The south side of the deep excavation employed for the finite element analysis. There is a 40 m-wide Main Street that connects Mexico Square with Torehayloch. The surcharge load from 4.7 m thick backfill soil above the top of the wall plus 10 kN/m^2 from the highway, a total of 90 kN/m^2 surcharge used. This surcharge throughout the excavation and construction process. (See Figures 3.2 and 3.3). The pile walls used to support the excavation were reinforced by two levels of pre-stressed tieback anchors, with a diameter of 0.6 m and a pile length of 17 m. The tieback anchors have three strands of tensile strength of 220 kN/m and 180 kN/m for the upper and lower row, respectively. They are fixed at 30 degrees of inclination below the horizontal.

The tieback anchors spaced 1.8 m and 3.6 m horizontally for the upper and lower rows, respectively. The design parameters and profile of the retaining structures are presented in Figure 3.3.

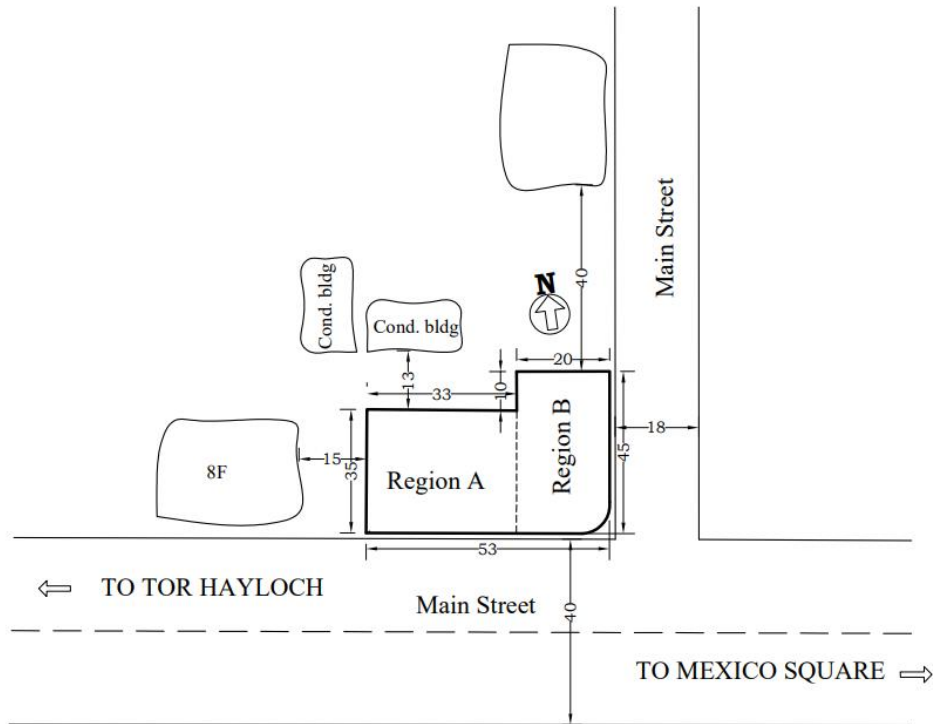


Figure 3. 2: Site layout.

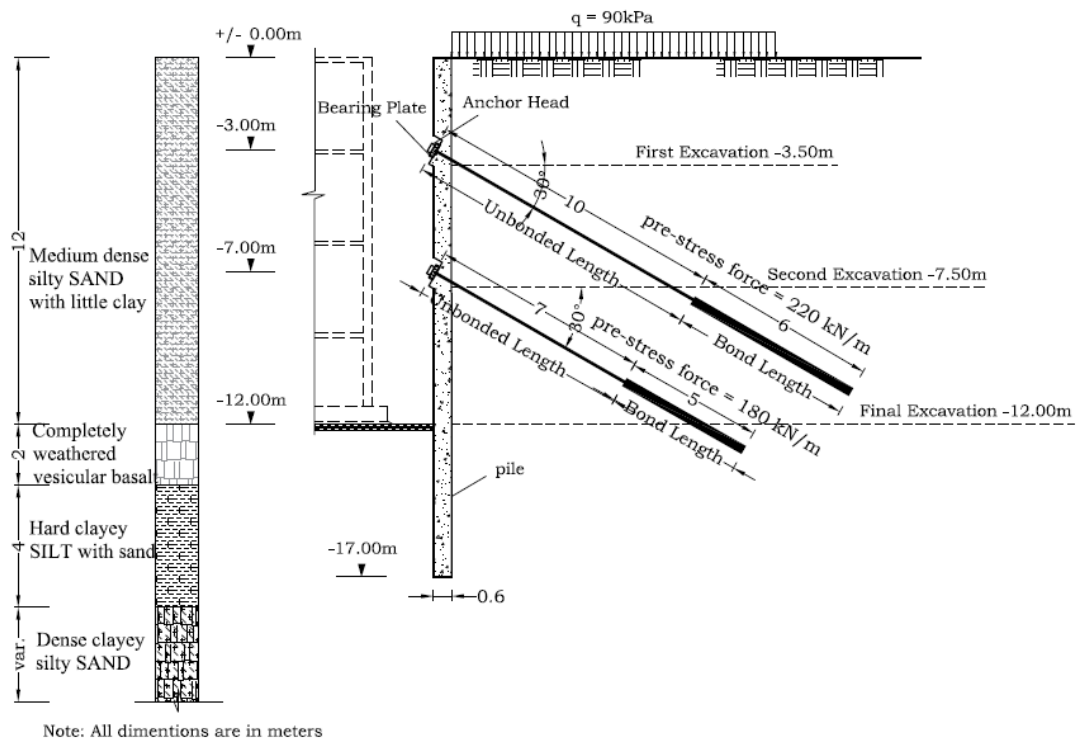


Figure 3. 3: Cross-section profile of retaining structure.

3.2 Geological Conditions

Four boreholes were bored to a maximum depth of 51 m to investigate the geological condition of the site. The project site comprises uniform geology, including basaltic flows of varying degrees of weathering. Consolidated paleosol layers separate the differently weathered basaltic flows. Weathered rocks degraded into silty SAND soil. At shallow depths, there is groundwater in the form of seepage. The site's topography is flat to gentle.

3.3 Geotechnical Conditions

According to a geotechnical investigation at the excavation site, the insitu soils included Medium dense silty SAND with little clay, completely weathered vesicular basalt, hard clayey SILT with sand, and dense clayey silty SAND. The geotechnical properties assumed in the project are shown in Table 3.1. Where ϕ' is effective angles of internal friction, C' is the cohesion, and E_s is Young's modulus.

Table 3. 1: Values of the soil parameters.

No.	soil	Thickness	Unit weight	C'	ϕ'	E_s
		m	kN/m ³	kN/m ²	°	MPa
1	Medium dense silty SAND with little clay	12	17	18	26	65
2	Completely weathered vesicular basalt	2	21	15	35	100
3	Hard clayey SILT with sand	4	19	20	29	60
4	Dense clayey silty SAND	30	18	18	28	70

The parameter of soil shear resistance take from laboratory direct shear tests of samples taken from boreholes. The modulus of stress-strain E_s values couldn't get from the investigation report, and the SPT N-value couldn't get on the specific layers. Thus, the E_s value was used from tables, according to Bowels. See Appendix A

4 Conventional Analytical Method

4.1 Conventional Analysis using DeepEx Software

One of the conventional analysis methods is the limit equilibrium method. LEM is the most common method for retaining structure analysis, and limited state conditions are assumed. For excavations and earth retaining structures, lateral earth pressures will occur on both the retained soil and excavated sides. These pressures may represent a failure state such as active or passive lateral earth pressures or an assumed redistribution such as diagrams by Terzaghi and Peck.

The retaining wall is analyzed to provide moment and force equilibrium. For models with multiple support levels, support reactions are analyzed either using the tributary area method or a series of beam analysis methods (Blum's Method, Simple Span Method, CALTRANS method, and more).

4.1.1 Concept of the LEA Method

In the limit equilibrium analysis, we need to calculate the net pressures diagram by taking into consideration:

1. The soil pressures on the driving and resisting side (active-passive - apparent)
2. The water pressures on the walls
3. The external loads on the walls (construction loads, buildings, traffic loads, cranes, and more)
4. The seismic pressures on the walls (when we examine seismic in long-term conditions)

4.1.2 Advantages and Disadvantages of Limit-Equilibrium Methods

The advantages of limit equilibrium methods include:

1. Form a well-known analysis method
2. Can provide quick answers
3. The assumed limit conditions are well understood

Among the Disadvantages;

1. The analysis is valid only for simple conditions.
2. Soil structure interaction is not well expressed.
3. Calculated wall displacements are unrealistic for excavations with multiple bracing levels.
4. Computed wall bending moments are possibly unconservative when there is more than one support level.
5. Construction stage history ignored.

4.1.3 Result of DeepEx Software

Computed results (lateral deflection, bending moment, and shear force) of the DeepEx software based on limit equilibrium analysis are shown in Figures 4.1, 4.2, and 4.3, respectively.

The maximum wall deflection at the ground surface and its value are 4.2mm (see Figure 4.1). The anchor load effect is seen in the bending moment diagram and shear force diagram. The maximum bending moment and shear force diagram are at 0.18Lp from the top of the pile and their values are 115 kN-m/m and 103 kN/m, respectively (see Figures 4.2 and 4.3). These maximum values are presented in Table 4.1. The factor of safety found in this analysis is 1.85.

Table 4. 1: Maximum; Lateral Deflection, Bending Moment, and Shear Force from the DeepEx software result, based on limit equilibrium analysis

Req. parameters	DeepEx result
Max. ΣP_{ux} (mm)	4.2
Max. BM (kN m/m)	115
Max. SF (kN/m)	103

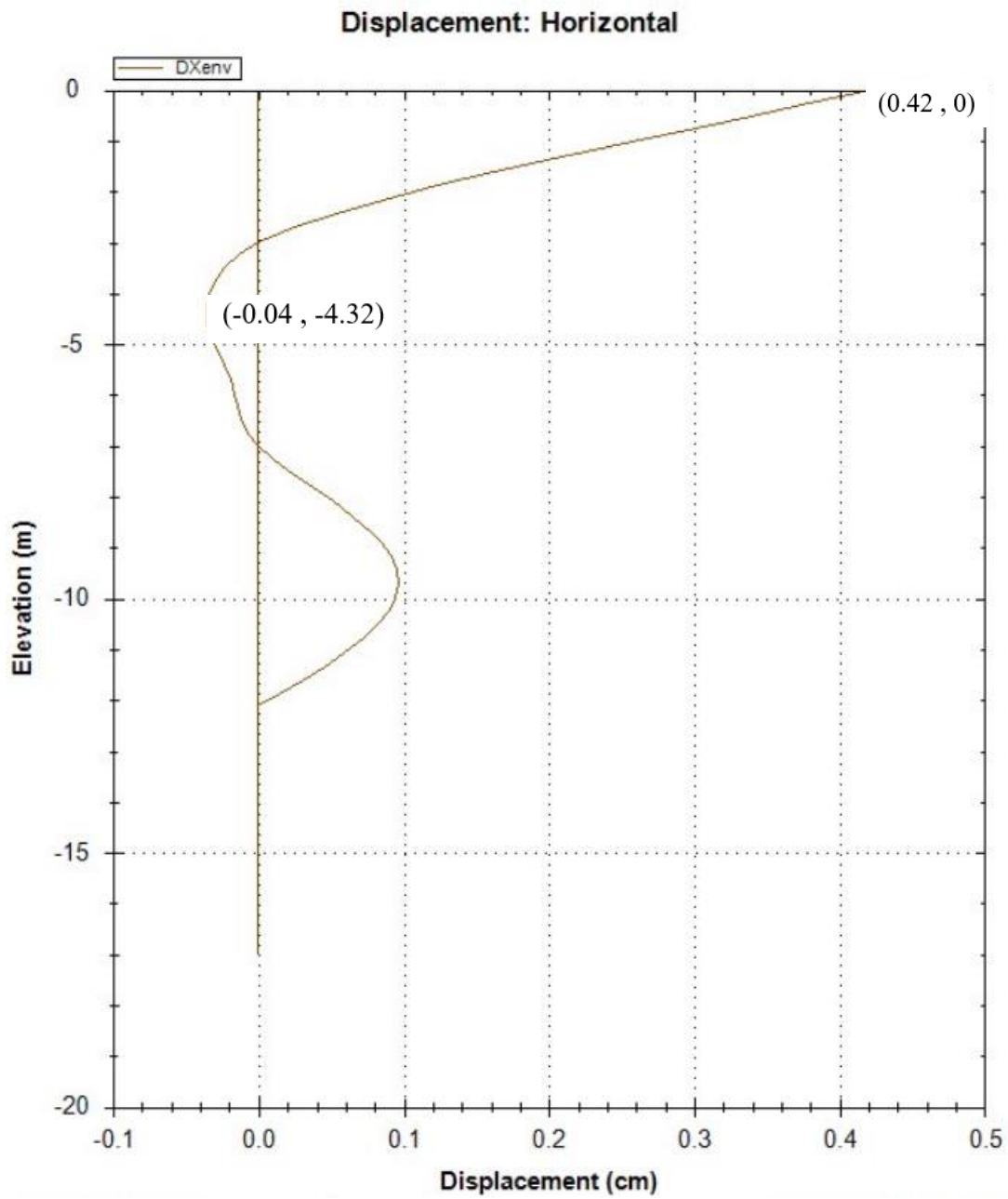


Figure 4. 1: Lateral deflections of the pile wall at the final stage of excavation when using DeepEX software

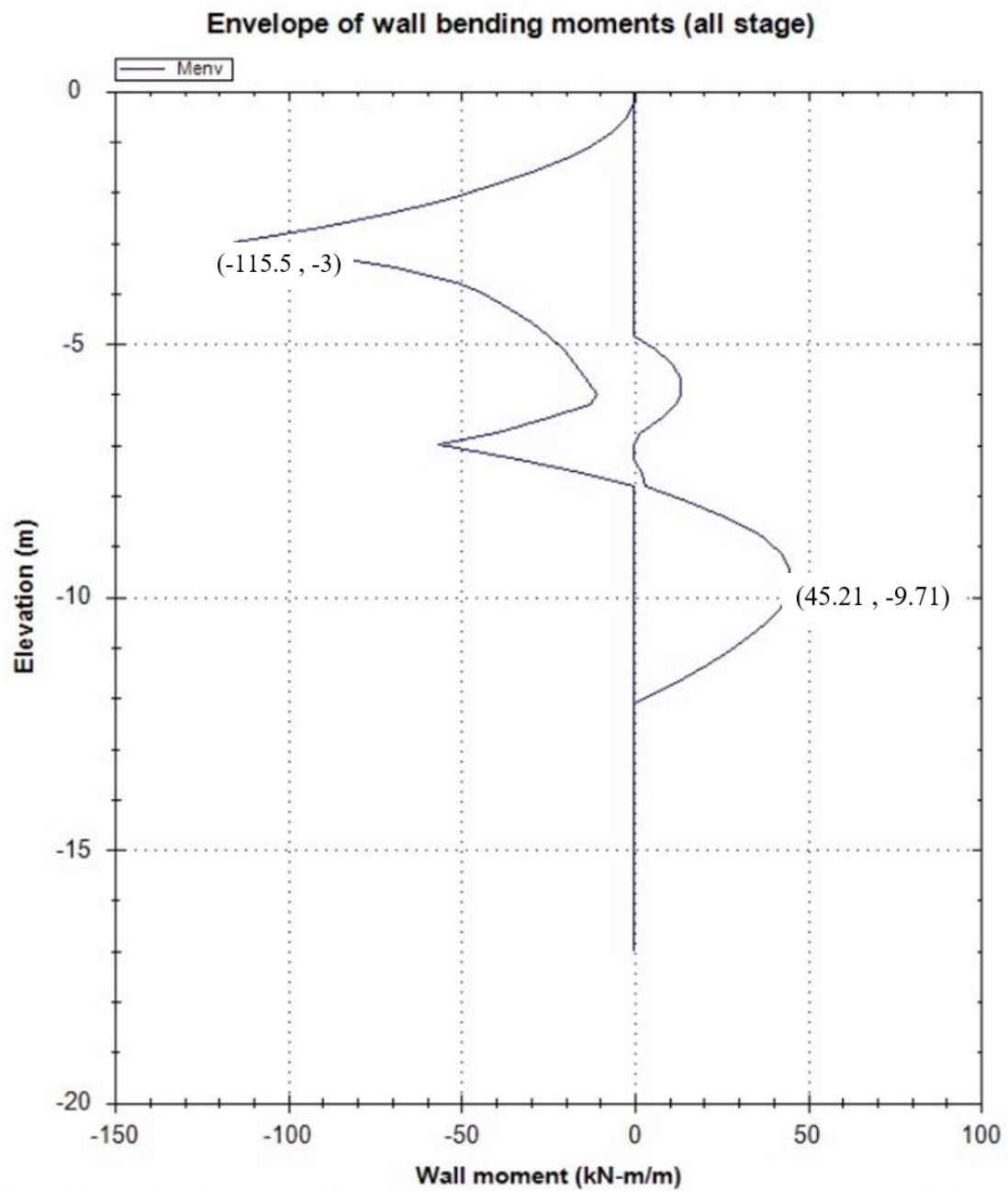


Figure 4. 2: Bending Moment of the pile wall at the final stage of excavation when using DeepEX software

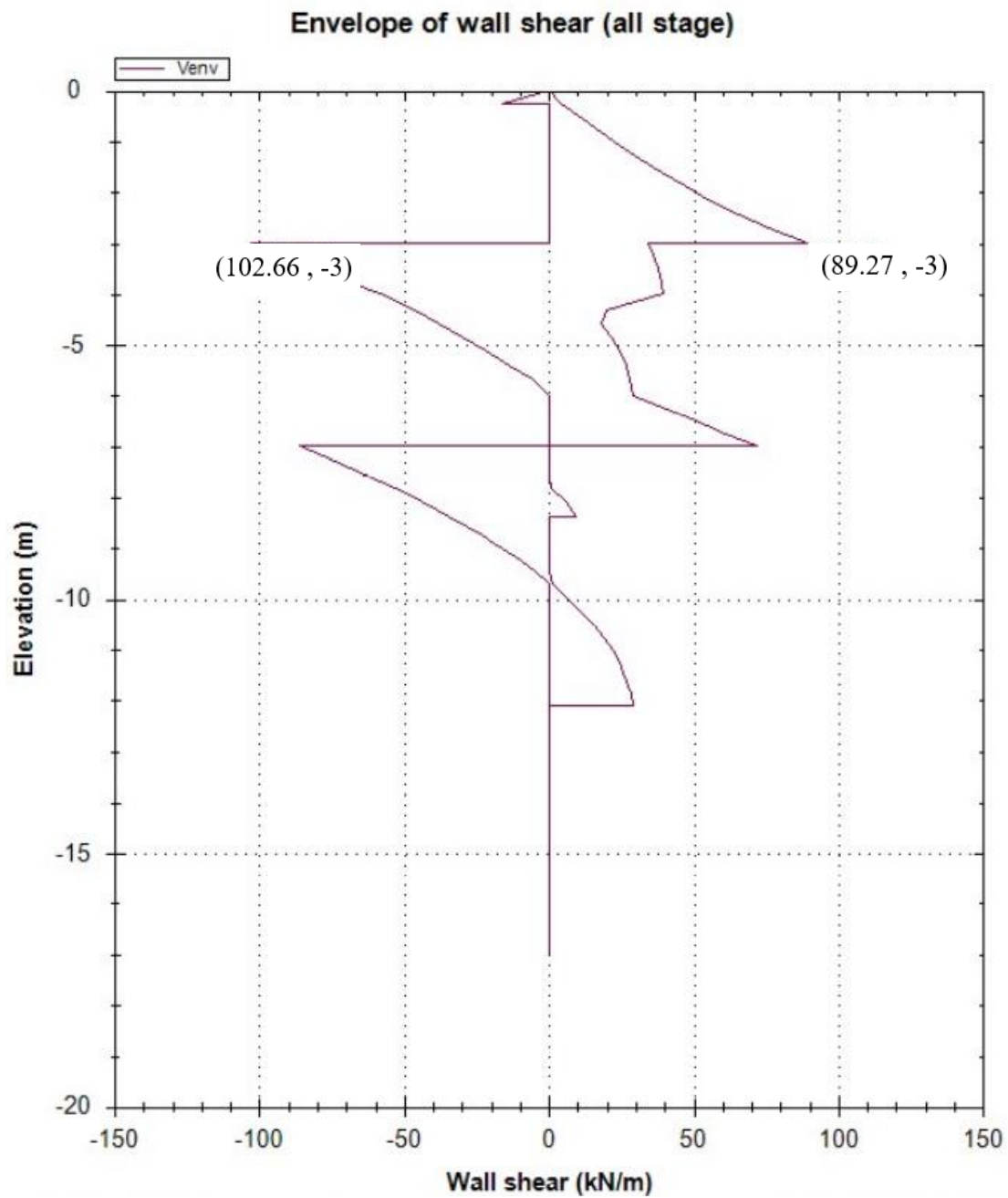


Figure 4. 3: Shear Force of the pile wall at the final stage of excavation when using DeepEX software

4.2 Conventional Analysis using GEO5 Software

The GEO5 “Sheeting Check” program makes the advanced design of embedded retaining walls.

The "GEO5 Sheeting Check" program makes the advanced design of anchored or strutted retaining walls (Sheet pile, Diaphragm, and Soldier pile walls). It allows the user to model the actual structure behavior using stages of construction. The program calculates the required length of the structure in soil, the internal forces on the retaining wall, the deformation and pressures acting upon the retaining wall, and verifies the overall stability.

The "GEO5 Sheeting Check" program uses the following construction stages:

- Construction stage 1: excavation of a pit to a depth of 3.5 m, the geometry of the wall,
- Construction stage 2: first-row anchoring of the wall,
- Construction stage 3: excavation of a pit to a depth of 7.5 m,
- Construction stage 4: second-row anchoring of the wall,
- Construction stage 5: last excavation to a depth of 12 m.

A 2D geometry model in the XY plane with the surcharge load of 90 kN/m². The pile walls support the excavation and are reinforced by two levels of pre-stressed tieback anchors. The anchors are fixed at 30 degrees of inclination, below the horizontal (see Figure 4.4). Input data used on the software is shown in Tables 4.2 - 4.4.

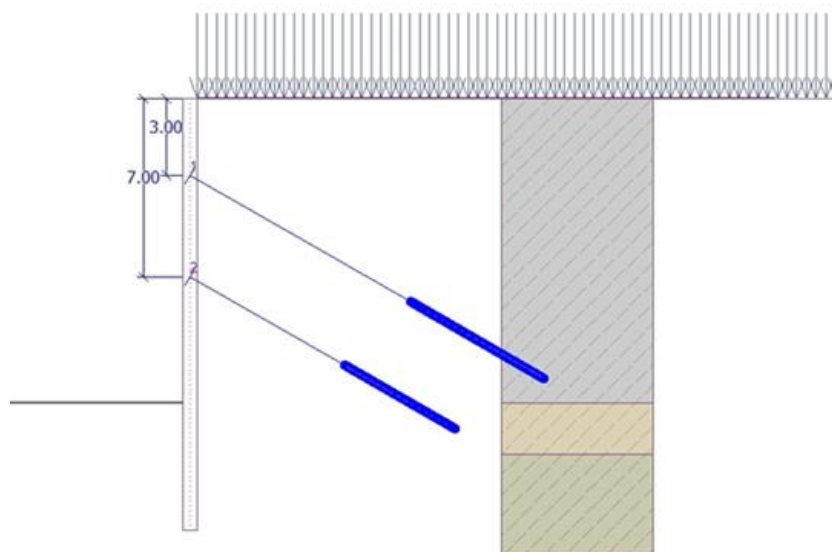


Figure 4. 4: Geometry of the structure

4.2.1 Input Data

Table 4. 2: Geometry of structure

Structure length	17.00 m
Cross-section name	Pile curtain
Diameter	0.6 m
Spacing	1.8 m
Material of pile	Concrete
Computed coefficient of pressure reduction below the ditch	0.7
Area of cross-section , A	1.57E-01 m ² /m
Moment of inertia, I	3.53E-03 m ⁴ /m
Elastic modulus , E	31000.00 MPa
Shear modulus, G	12917.00 MPa

Table 4. 3: Material of Structure





Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

Concrete	C 25/30
Cylinder compressive strength, f_{ck}	25.00 MPa
Tensile strength, f_{ctm}	2.60MPa
Elasticity modulus, E_{cm}	31000.00 MPa
Shear modulus, G	12917.00 MPa
Longitudinal steel	B400 (user defined)
Yield strength, f_{yk}	400.00 MPa
Transverse steel	B400 (user defined)
Yield strength, f_{yk}	400.00 MPa

Anchors forces

No.	Depth [m]	Displacement [mm]	Anchor force [kN]
1	3.00	-2.9	234.85
2	7.00	-2.3	183.00

Table 4. 4: Basic soil parameters

No.	Name	Pattern	ϕ_{ef} [°]	c_{ef} [kPa]	γ [kN/m ³]	γ_{su} [kN/m ³]	δ [°]	E_{oed} [MPa]
1	Silty SAND		26	18	17	8	9.00	65
2	Completely weathered vesicular basalt		35	15	21	12	11.00	100
3	Clayey SILT		29	20	19	10	9.00	60
4	Clayey silty SAND		28	18	18	9	9.00	70

4.2.2 Result of GEO5 Software

The lateral deflection, bending moment, and shear force results, using the GEO5 “Sheeting Check” software program presented in Figures 4.5, 4.6, and 4.7, respectively. The stability result is shown in Figure 4.8. The maximum service load values are shown in Table 4.5.

Table 4. 5: Maximum lateral deflection, Bending Moment, and Shear Force from the GEO5 software result

Req. parameters	GEO5 result
Max. ΣPu_x (mm)	26.1
Max. BM (kN m/m)	170.6
Max. SF (kN/m)	127

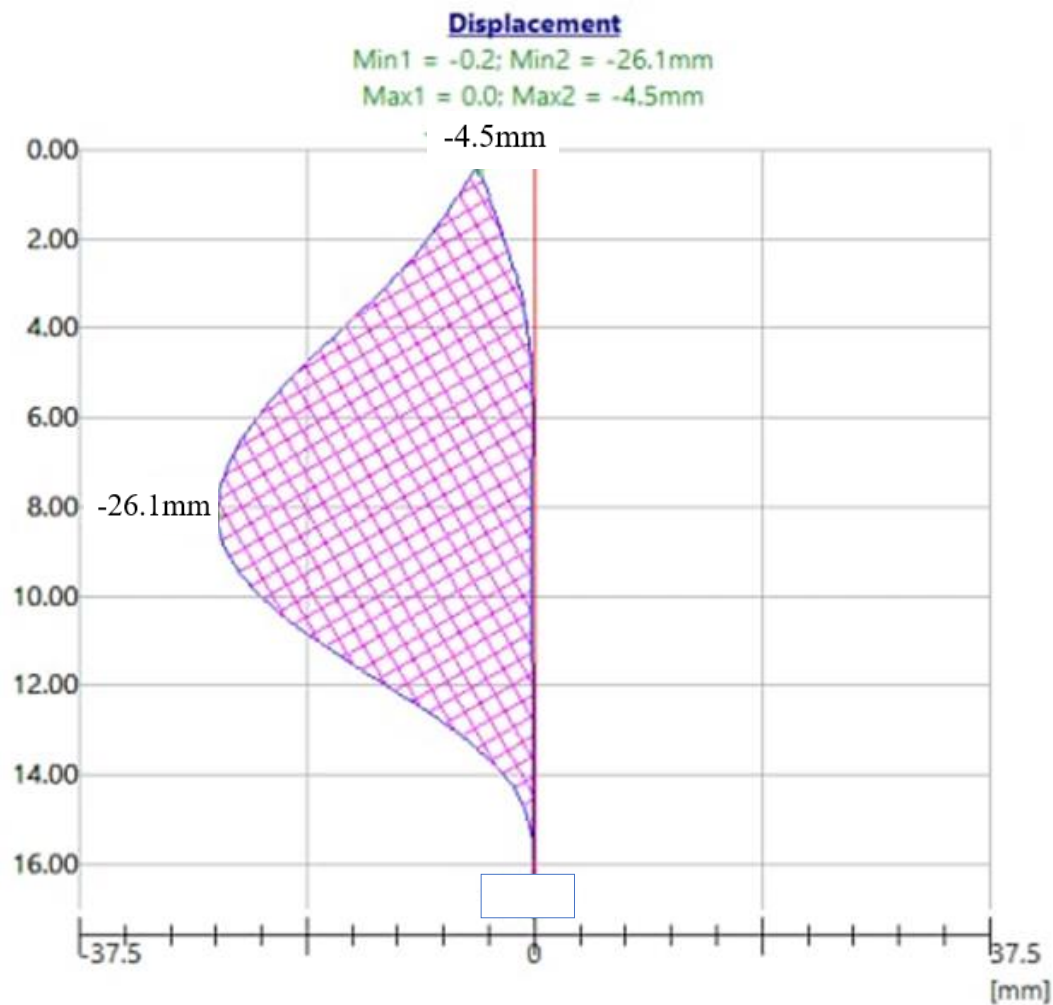


Figure 4. 5: Lateral deflections of the pile wall at the final stage of excavation when using GEO5 software

The maximum wall deflection is at $0.48L_p$ from the top of the pile, and its value is 26.1mm (see Figure 4.5). The maximum bending moment per meter width is at $0.82L_p$ from the top of the pile, and its value is 239 kN-m/m (see Figures 4.6). The maximum shear force per meter width is at $0.7L_p$ from the top of the pile, and its value is 178 kN/m (see Figures 4.7). The above loads are the maximum design load with a load factor of 1.4, and the maximum service load values are presented in Table 4.5. The safety factor found in this analysis is 1.85.

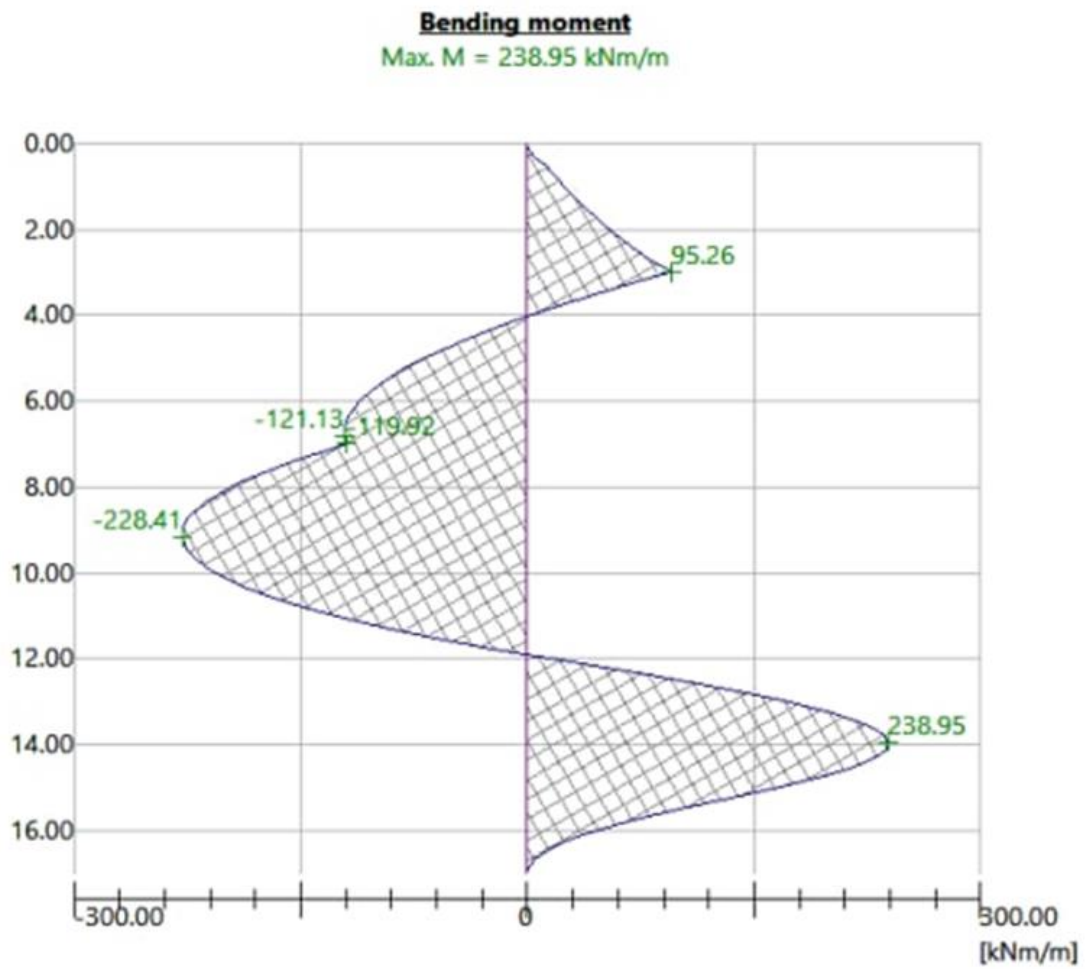


Figure 4. 6: Bending Moment of the pile wall at the final stage of excavation when using GEO5 software

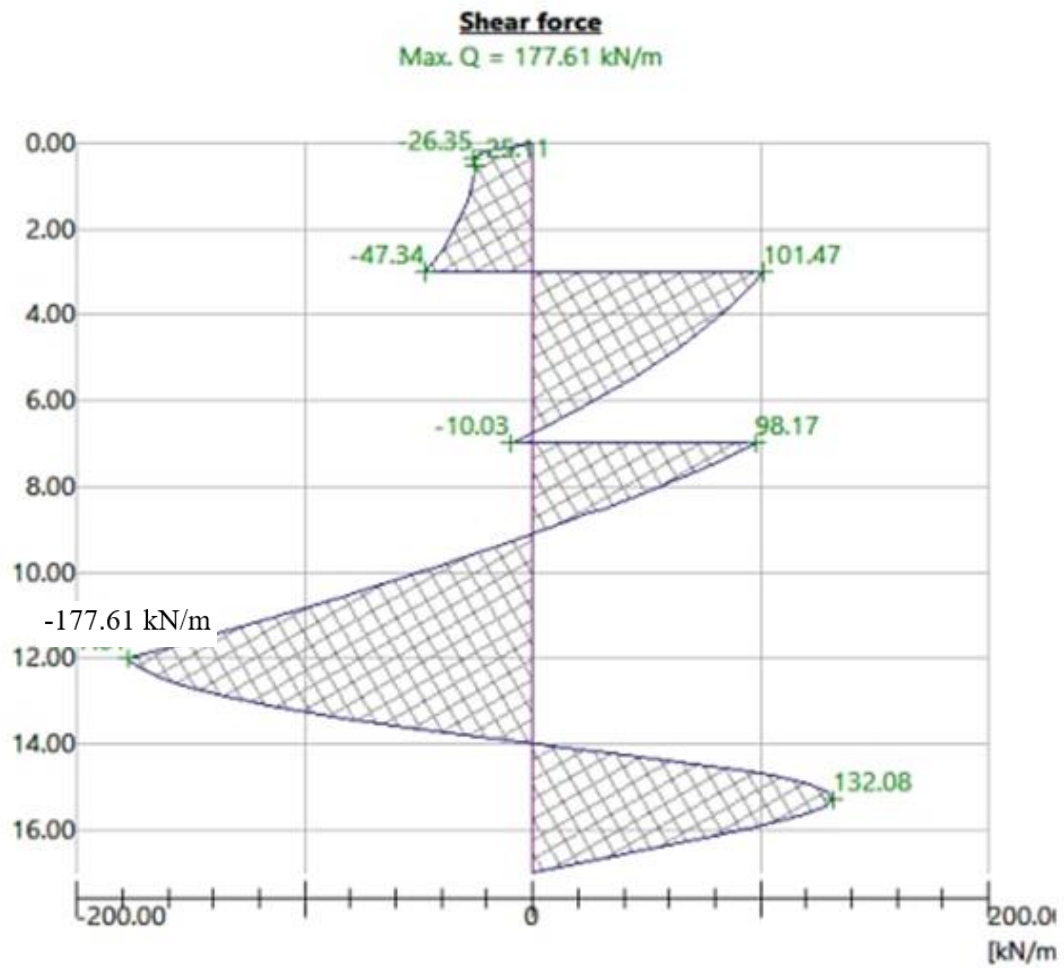
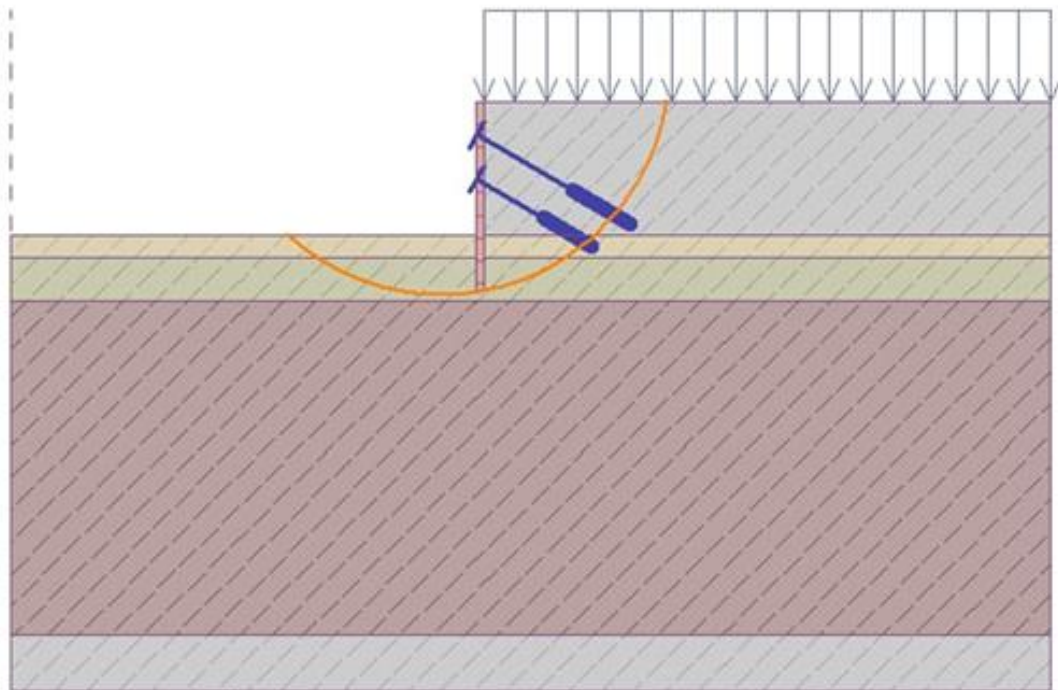


Figure 4. 7: Shear Force of the pile wall at the final stage of excavation when using GEO5 software



Slope stability verification (Bishop)

Sum of active forces : $F_a = 2033.19 \text{ kN/m}$

Sum of passive forces : $F_p = 4319.93 \text{ kN/m}$

Sliding moment : $M_a = 41416.01 \text{ kNm/m}$

Resisting moment : $M_p = 87997.05 \text{ kNm/m}$

Factor of safety = $2.12 > 1.50$

Slope stability **ACCEPTABLE**

Figure 4. 8: Bishop Slope stability verification

4.3 Beam on Elastic Foundation using DeepEx Software

The excavation model is reduced to a plane problem, in which a unit-wide slice of the wall analyzed. This method uses DeepEx software for the analysis, and for soil modeling Mohr-Coulomb model used.

Modeling the soil-wall interaction, the popular Winkler approach is adopted. The retaining wall is modeled as a beam element with transversal bending stiffness EI , the soil is modeled using a double array of independent springs at each wall grid point. According to the Winkler model, the behavior of every soil spring is un-coupled from

the behavior of adjacent elements. The actual interaction among different soil regions is left to the retaining wall.

The main input parameters are the wall length and thickness, the elevation of the bottom of each soil layer, excavation depth, and the soil parameters (shown in Table 4.6)

Table 4. 6: main input parameters

K_{wA}	Winkler active coefficient	[-]
K_{wP}	Winkler passive coefficient	[-]
K_{Es}	soil elastic stiffness per area,	[kN/m ²]
c	Cohesion intercept,	[kN/m ²]
φ	Friction angle	[°]
γ'	Specific weight	[kN/m ³]

4.3.1 Result of DeepEx software- Beam on Elastic Foundation Analysis

The "DeepEx" software analysis result based on the beam on elastic foundation analysis method, is presented below. The lateral deflection, bending moment, and shear force are seen in Figures 4.9, 4.10, and 4.11, respectively.

The maximum wall deflection is at 0.61Lp from the top of the pile, and its value is 6.4mm (see Figure 4.9). Similarly, a bending moment is at 0.61Lp from the top of the pile wall, and its maximum value is 126 kN-m/m (See Figure 4.10). The shear force is at 0.41Lp from the top of the pile wall, and its maximum value is 120 kN/m (see Figure 4.11). These maximum values are shown, in Table 4.7. The factor of safety found in this analysis is 1.65.

Table 4. 7: Maximum lateral deflection, Bending Moment, and Shear Force from the limit equilibrium analysis

Req. parameters	DeepEx result
Max. ΣP_{ux} (mm)	6.4
Max. BM (kN m/m)	126
Max. SF (kN/m)	120

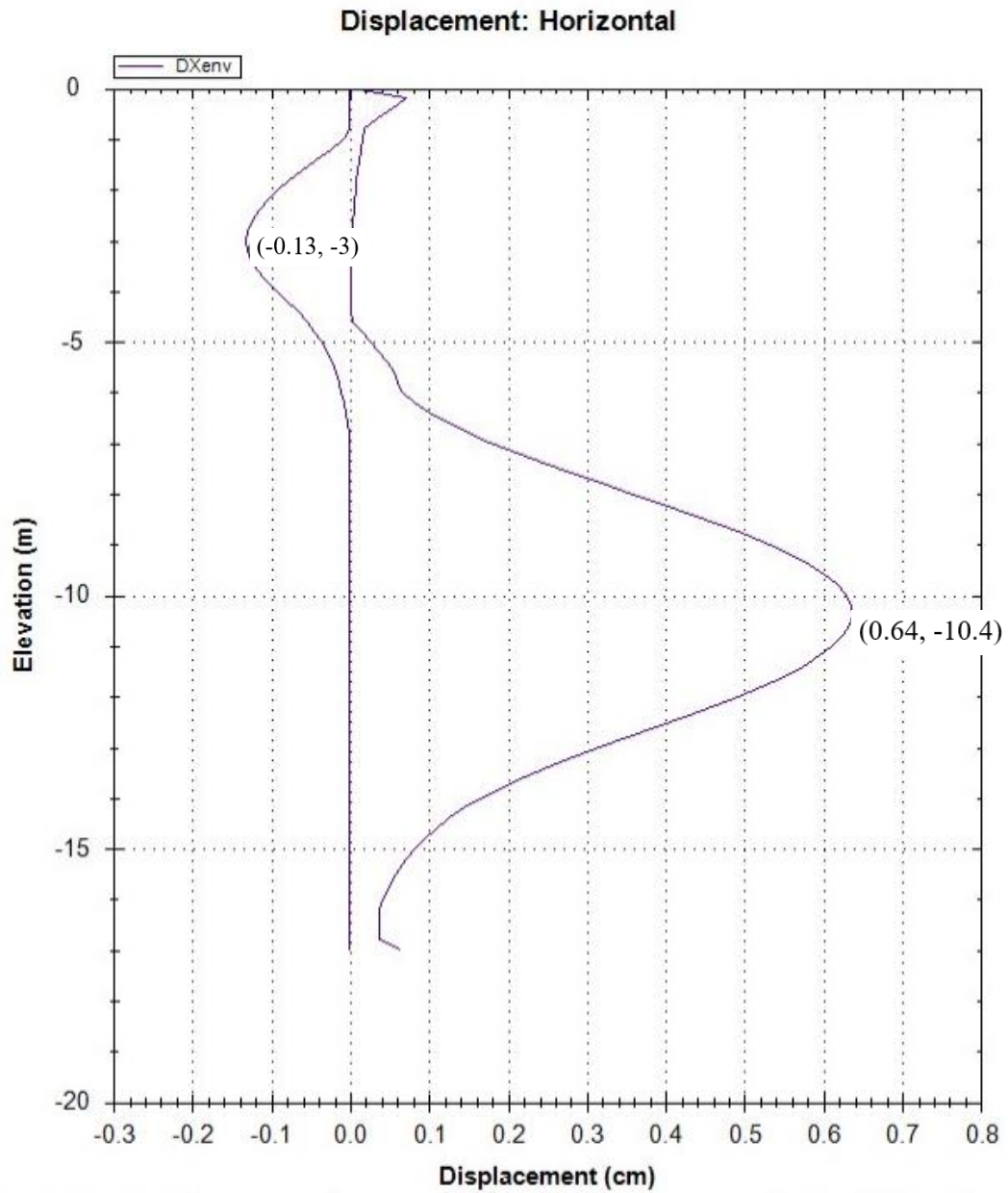


Figure 4. 9: Lateral deflections of the pile wall at the final stage of excavation when using DeepEX software

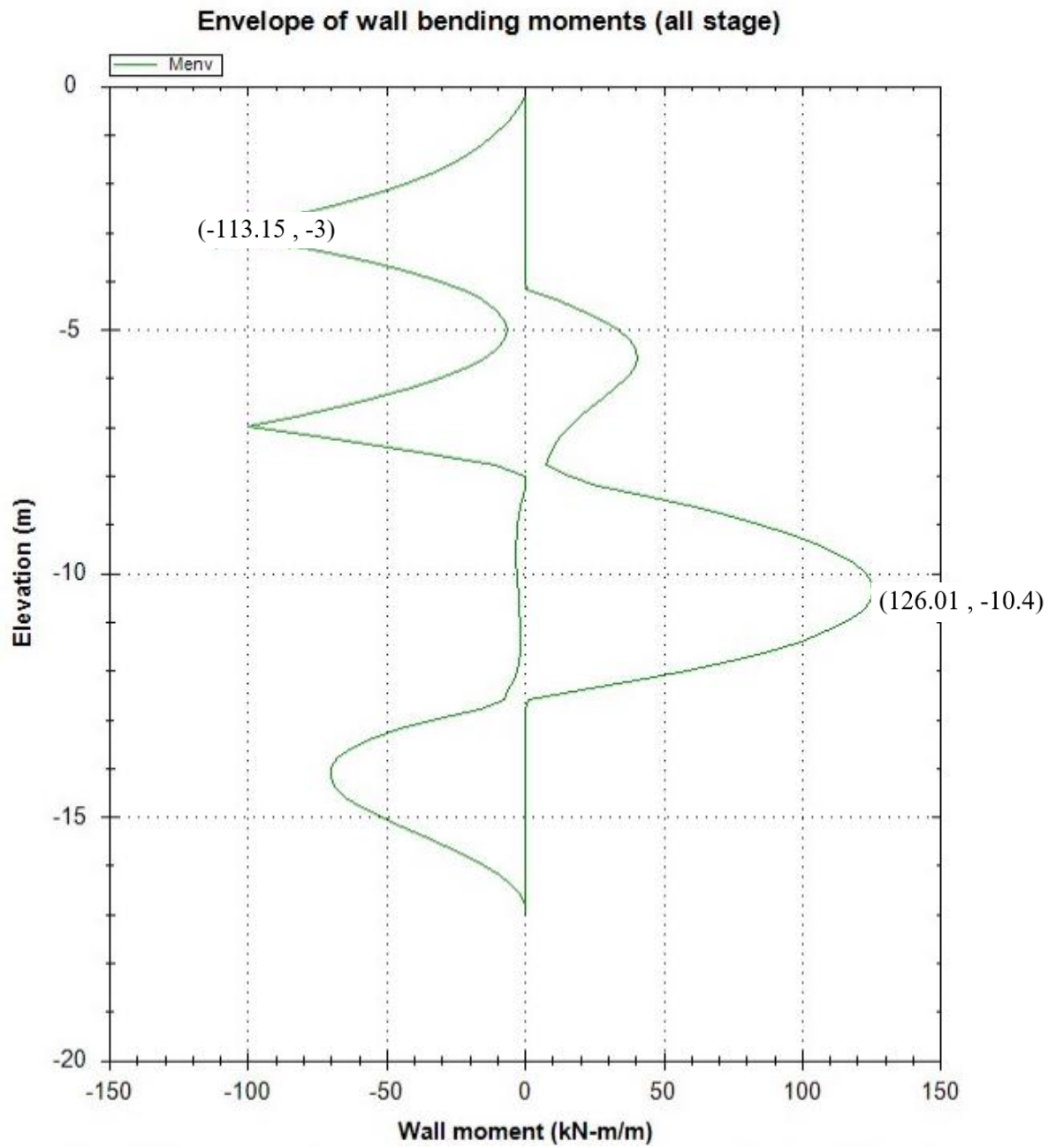


Figure 4. 10: Bending Moment of the pile wall at the final stage of excavation when using DeepEX software

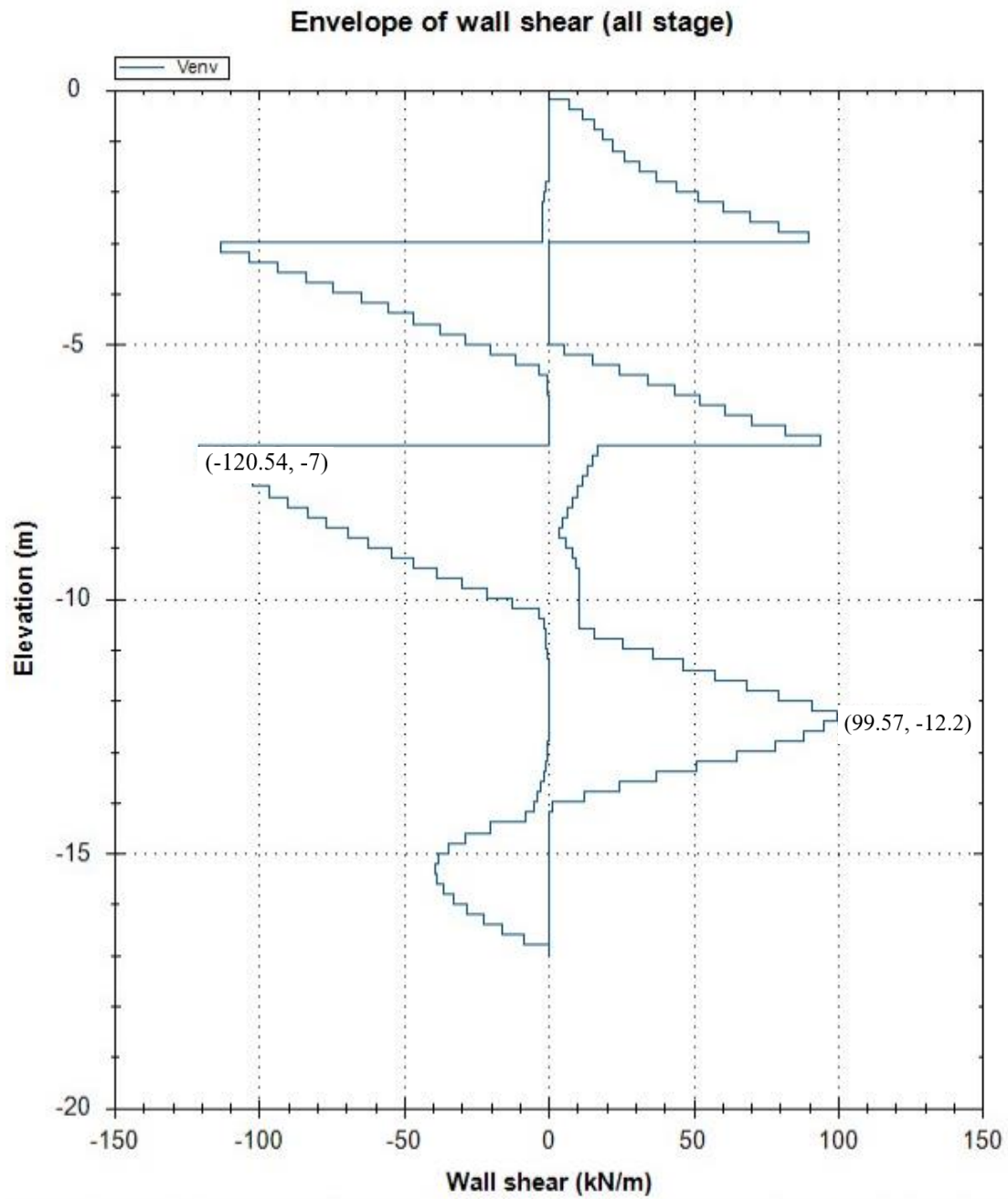


Figure 4. 11: Shear Force of the pile wall at the final stage of excavation when using DeepEX software

5 Finite Element Analysis Method (FEM)

5.1 Basic Steps Involved in FEM

The Finite Element Method has become the premier numerical tool for the analysis of a wide range of problems in engineering science. It requires the discretization of the domain of the problem. The basic steps involved in solving boundary-valued problems using FEM are as follow:

1. The domain is divided into a finite number of elements (see Figures 5.1). For two-dimensional problems, triangular and/or quadrilateral elements are often used. The corners of the element are called nodes. Irregular boundaries are easily incorporated by approximating them by discrete line segments or by using mathematical functions.

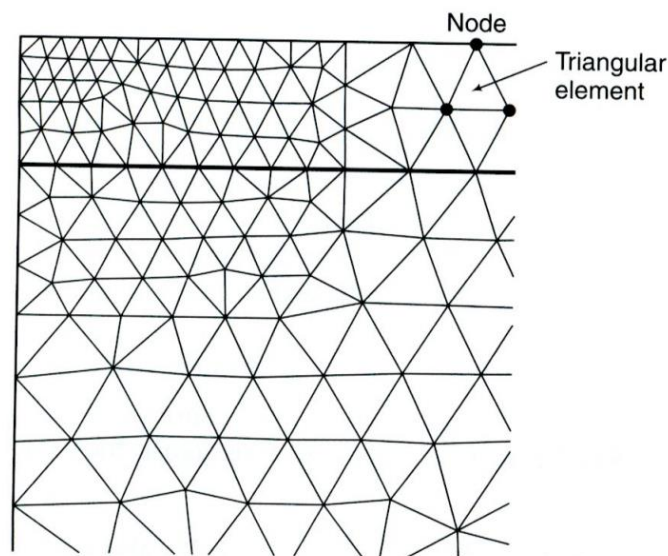


Figure 5. 1: Discretization of a two-layer of soil domain into triangular elements for FEM.

2. Determine the form of distribution of the unknown quantity (for ex. Displacement). We can write a mathematical function to describe how the unknown quantity varies across the element using the nodes. Polynomial interpolation functions are popular mathematical functions used in FEM.

3. Formulate the response of each element by expressing the unknown quantity as a function of the nodes and their position inside the element. The general form of the equations for the response of an element is:

$$\{f\} = [k]\{l\}$$

Where: k is the element stiffness matrix.

l is a vector list of the unknown quantity at the nodes, and

$\{f\}$ is a vector list of the applied loads.

4. Formulate the global response equations by assembling the equations for each element for the entire body.

$$\{F\} = [K]\{L\}$$

Where: $[K]$ is the global stiffness matrix,

$\{L\}$ is a vector list of the unknown quantity at the nodes within the body, and

$\{F\}$ is a vector list of the applied loads.

5. Solve the global equations imposing the boundary conditions for the unknown quantity.

5.2 Numerical Modeling of Supported Deep Excavation

The deep excavation is supported, by a concrete pile wall. The walls are tied back, using prestressed ground anchors.

5.2.1 Geometry Modeling

To generate a 2D geometry model in the XY plane the existing analytical data from the actual project was used. There are two things to think about when it comes to the geometry size:

- Geometry with an excessively large size incurs a significant computational cost. Furthermore, results from distant points may be irrelevant because they are outside the excavation's effect area.

- Geometry with a too-small size may produce incorrect results due to the influence of boundary conditions.

5.2.1.1 Model Size Sensitivity Analysis

The model's size must be large enough that the boundaries do not affect the results of the problem. The model size will be decided; by the height of the pile wall and the depth of building excavation. Sensitivity analysis is from $2L_p - 4L_p$ and $2H - 4H$ (L_p ; length of pile and H ; depth of building excavation) to determine the optimum model size. The variation of stresses in the horizontal and vertical directions are in Figure 5.2 and Tables 5.1 and 5.2.

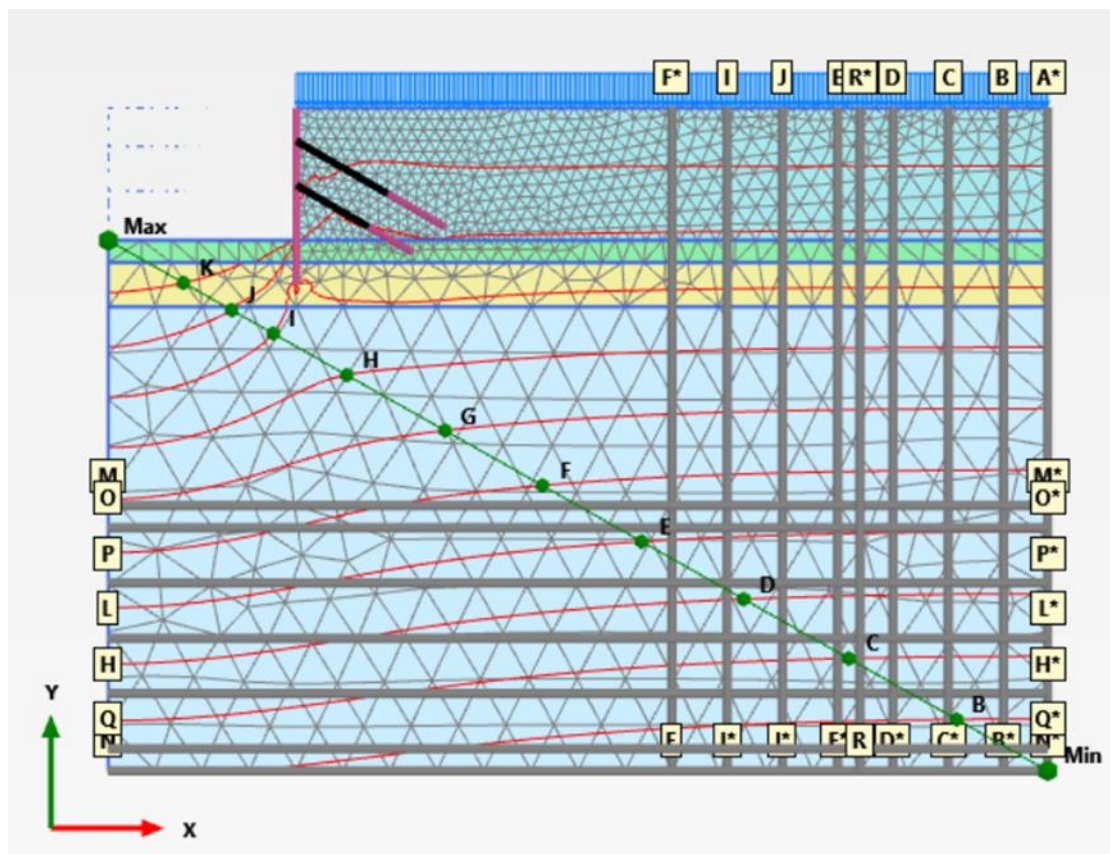


Figure 5. 2: Vertical (F-F, R-R, and A-A) and horizontal (M-M, L-L, and N-N) cross sections for sensitivity analysis of model size.

Table 5. 1: Major and minor stress away from the axis of the excavation

		away from the axis of excavation	σ'_1	σ'_2
		(m)	kN/m ²	kN/m ²
F-F	2Lp	51	1067	565.7
		56	1072	568.3
		61	1075	570.3
		66	1078	571.8
R-R	3Lp	68	1079	572.3
		71	1080	572.9
		76	1081	573.6
		81	1082	574.1
A-A	4Lp	85	1082	574.2

Table 5. 2: Major and minor stress below the ground surface

Section		below the ground surface	σ'_1	σ'_2
		(m)	kN/m ²	kN/m ²
M-M	2H	36	657.7	342.5
		38	693.4	361.3
		43	782.4	409.1
L-L	3H	48	871.1	457.3
		53	959.3	505.8
		58	1047	554.6
N-N	4H	60	1082	574.2

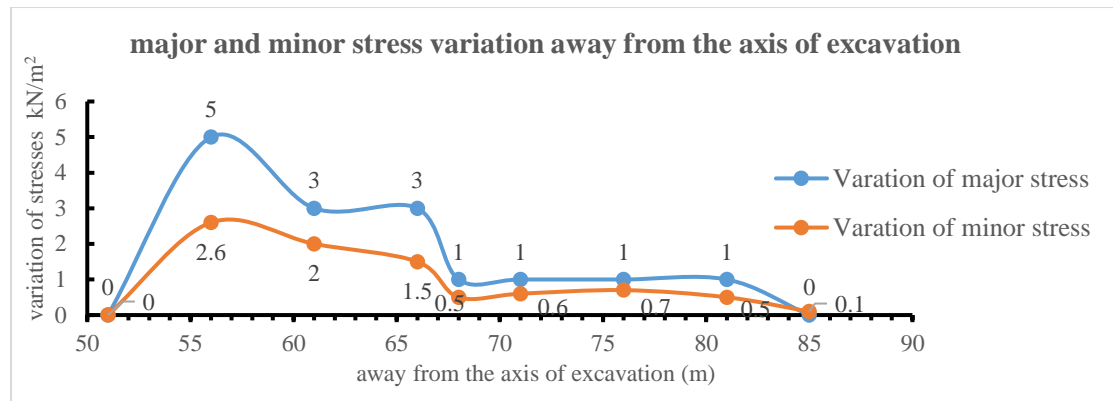


Figure 5. 3: Major and minor stress variation from the axis of excavation.

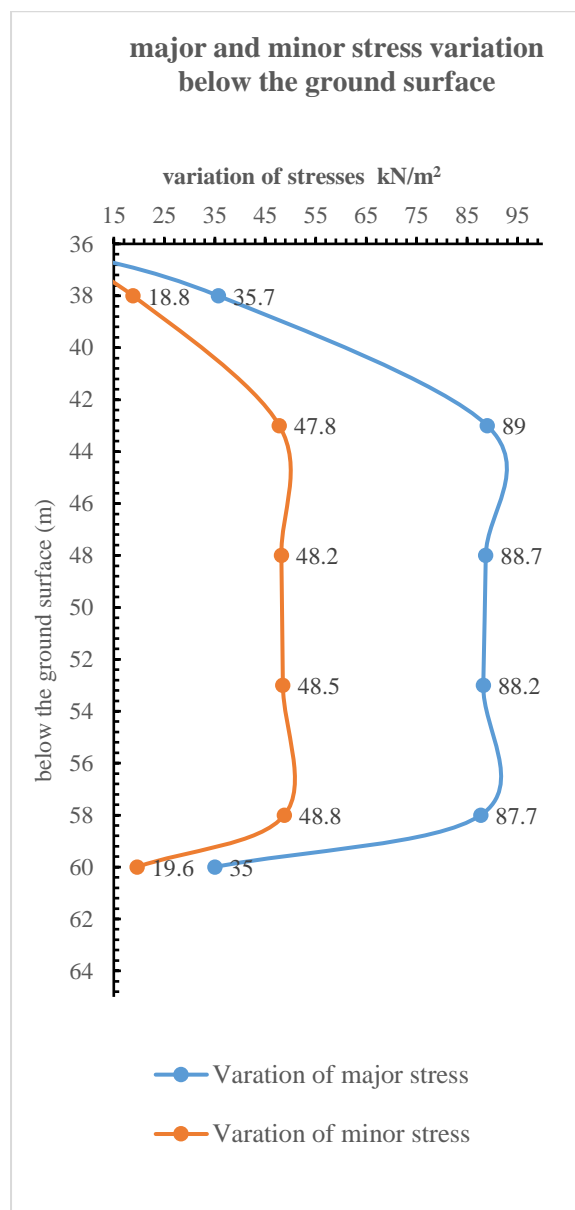


Figure 5. 4: Major and minor stress variation below the ground surface.

During excavation, the state of stresses in the groundmass around the excavation changes. Stress relief on the excavation face results in horizontal ground movement followed by the vertical ground movement for equilibrium. As shown in Tables 5.1 and 5.2, the major and minor stress increased in both; horizontal and vertical directions. Stress variation diminishes to zero in Figure 5.3 than in Figure 5.4. This implies that Stress variations in the horizontal direction are more sensitive than in the vertical direction.

Stress variation from distant points is irrelevant because they are outside the excavation's effect area. High-stress variation for the too-small size may produce incorrect results due to the influence of boundary conditions. Thus the optimum model size; is at sections R-R and L-L, is $3L_p$ from the edge of the excavation and $3H$ from the bottom of the excavation. Therefore, the excavation geometry is (68mx48m). See Figure 5.5

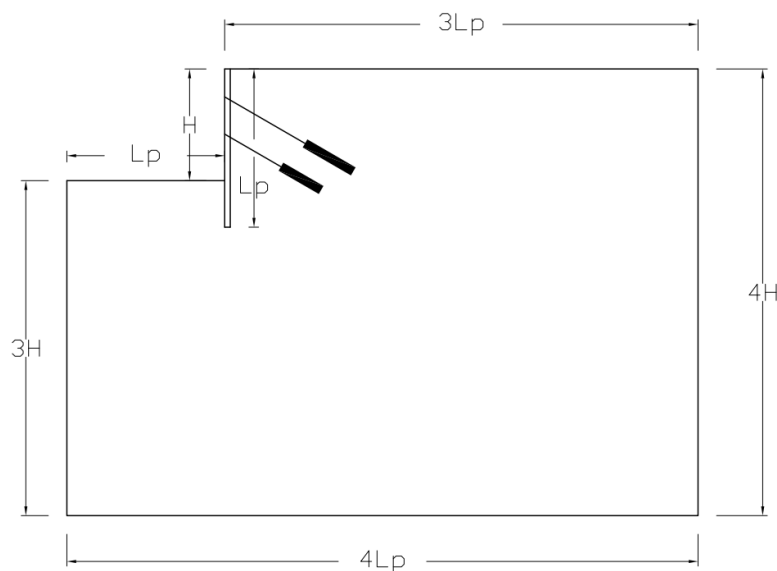


Figure 5. 5: Size of Model

All movements were restrained, at the bottom boundary. Whereas, for the vertical boundaries, all horizontal movements were restrained.

5.2.2 Soil Modeling

The soil stratigraphy is defined in the soil mode using the borehole feature of the program. The soil body has four sublayers. Table 3.1 shows the mechanical properties of each soil sublayer. The Young's Modulus of Elasticity, E_s values are critical for calculating the deformation of excavations.

Since the available geotechnical parameters are limited in the investigation report, it is difficult to use advanced soil models. Thus, the Mohr-Coulomb model (MC model) is used to simulate the soil layers to evaluate the wall deflections, bending moment, and shear force.

The Mohr-Coulomb model is a simple and well-known linear elastic perfectly plastic model. It is the first approximation of soil behavior. The linear-elastic part of the Mohr-Coulomb model gets from Hooke's law of isotropic elasticity. The perfectly plastic part is based on the Mohr-Coulomb failure criterion, formulated in a non-associated plasticity framework.

5.2.2.1 Basic parameters of the Mohr-Coulomb model

The linear elastic perfectly-plastic Mohr-Coulomb model requires a total of five parameters. These parameters with their standard units are listed below. (See Table 5.3)

The parameters related to the soil shear resistance, are based on laboratory direct shear tests of samples taken from boreholes. And the value of E_s , ν , and R_{inter} is taken from tables according to Bowels. (See Table 5.4)

Table 5. 3: Mohr-Coulomb model parameters.

E	Young's modulus	[kN/m ²]
ν	Poisson's ratio	[-]
c	Cohesion	[kN/m ²]
φ	Friction angle	[°]
ψ	Dilatancy angle	[°]

Table 5. 4: Input Properties of soil layers for the MC model.

Soil layer	Layer thickness	γ_{unsat}	γ_{sat}	c	ϕ	ψ	E'	ν	R_{inter}
	m	kN/m ³	kN/m ³	kPa	°	°	MPa	-	-
Medium dense silty SAND with little clay	12	17	17	18	26	0	65	0.35	0.65
Completely weathered vesicular basalt	2	21	21	15	35	5	100	0.35	0.65
Hard clayey SILT with sand	4	19	19	20	29	0	60	0.35	0.65
Dense clayey silty SAND	30	18	18	18	28	0	70	0.35	0.65

5.2.3 Structure Modeling

5.2.3.1 Ground Anchor

Basic components of a grouted ground anchor include

- anchorage (anchor head, bearing plate)
- free stressing (unbounded) length
- bond length

The unbounded length of prestressing steel is free to elongate elastically and transfer the resisting force from the bond length to the structure. And the tendon bond length can transmit the applied tensile load into the ground. The grout is a Portland cement-based mixture that transmits load from the tendon into the ground. While also protecting the tendon from corrosion (see Figure 5.6). (U.S. Department of Transportation, 1999)

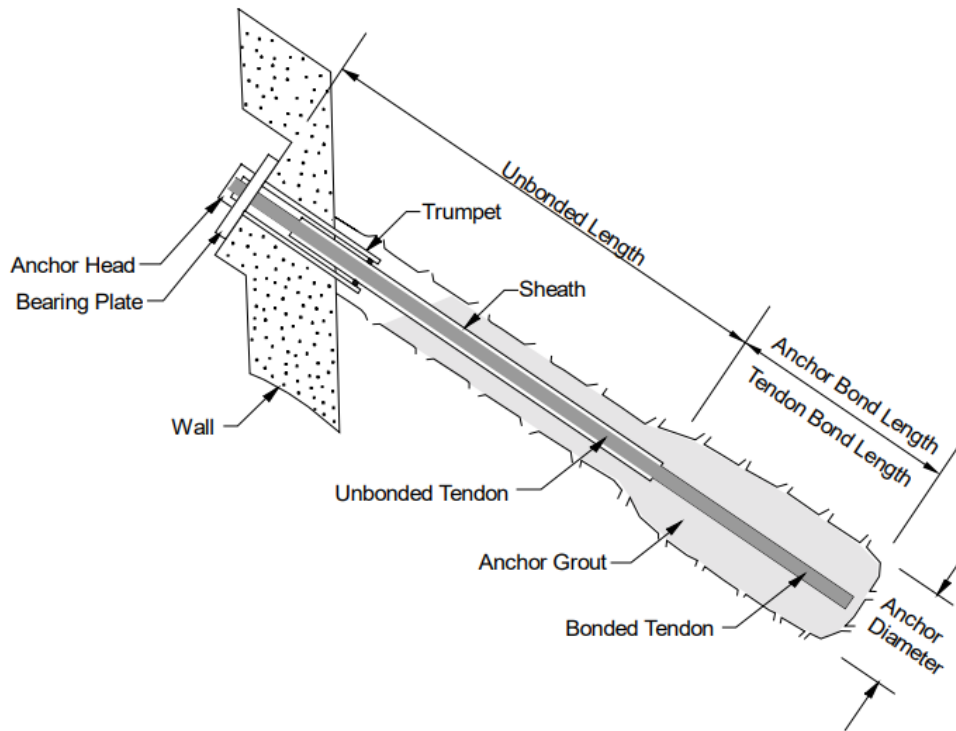


Figure 5. 6: Components of a ground anchor.

Finite Element Modeling of Ground Anchor

A ground anchor can be modeled; by a node-to-node anchor and an embedded beam. The embedded beam simulates the grouted part of the anchor. And the node-to-node anchor simulates the free length.

The axial stiffness EA , is the most required anchor property in the out-of-plane direction. It is entered per anchor rather than per unit width. Out-of-plane spacing, $L_s = 1.8$ m for top anchor and $L_s = 3.6$ m for bottom anchor, were used to determine equivalent stiffness per unit width. The top anchor had a maximum anchor force of 220 kN, while the bottom anchor had a maximum anchor force of 180 kN. The anchors are made up of three strands, each with seven wires. These are the most common strand in Addis Ababa. Table 5.5 shows the Anchor cable properties.

Table 5. 5: Anchor cable properties (ASTM 416, GRADE 270)

Parameter	Value
Cable diameter D, mm	15.24
Cable area A_c , mm ²	140
Yield stress f_y , MPa	1860
Characteristic ultimate load T, kN	260.70
Characteristic yield load T_y , at 1% extension, kN	234.60

Axial skin resistance of the grout body is 85% of the maximum anchor force. T_{skin} is 187 kN/m for the top anchor and 153 kN/m for the bottom anchor.

The coordinates and material properties of the anchor and grout body are listed in Tables 5.6 - 5.9

Table 5. 6: Node to node anchor coordinates

Anchor location	First point	Second point
Top	(17, 45)	(25.29, 40.22)
Bottom	(17, 41)	(23.53, 37.23)

Table 5. 7: Properties of the anchor rod (node-to-node anchor)

Parameter	Name	Value	Unit
Material type	-	Elastic	-
Axial stiffness	EA	106.7E3	kN
Out-of-plane spacing	L_s	1.8(top), 3.6(bottom)	m

Table 5. 8: Grout coordinates

Anchor location	First point	Second point
Top	(25.29, 40.22)	(30.49, 37.22)
Bottom	(23.53, 37.23)	(27.54, 34.92)

Table 5. 9: Properties of the grout body (embedded beam rows)

Parameter	Name	Value	Unit
Material type	-	Elastic	-
Stiffness	E	195.0E6	kN/m ²
Unit weight	γ	25	kN/m ³
Beam type	-	Predefined	-
Predefined beam type	-	Massive circular beam	-
Diameter	D	0.01524	m
Anchor spacing	$L_{spacing}$	1.8(top), 3.6(bottom)	m
Axial skin resistance		Linear	-
	$T_{skin, start, max}$	187(top), 153(bottom)	kN/m
	$T_{skin, end, max}$	187(top), 153(bottom)	kN/m
Lateral resistance		Unlimited	-
Base resistance	F_{max}	0	kN
Interface stiffness factor	Default values	Yes	-

5.2.3.2 Bored Soldier Pile Wall

Anchored walls consist of non-gravity cantilevered walls with one or more levels of ground anchors. Non-gravity cantilevered walls employ; either discrete or continuous vertical elements that are either driven or drilled to depths below the finished excavation grade. For non-gravity cantilevered walls, support is provided through the shear and

bending stiffness of the vertical wall elements and passive resistance from the soil below the finished excavation grade.

A discrete vertical wall element system and lagging construction by reinforced shotcrete were used. This kind of wall system can apply to most ground types. However, care must be on grounds such as cohesion-less soils and soft clays that may have limited “stand-up” time for lagging installation.

Finite Element Modeling of Bored Soldier Pile Wall

An embedded beam in PLAXIS 2D consists of a pile element with an embedded interface element to describe the interaction with the soil at the pile skin and the pile tip (bearing capacity). The pile axial skin resistance using α -METHOD according to Tomlinson and β -METHOD according to Burland. Base resistance is calculated based on the "c & ϕ " of soil according to Terzaghi.

Pile Skin Resistance Capacity

The α -METHOD according to Tomlinson

$$\alpha = C_1 [q/c]^{0.45}$$

$C_1 = 0.4 - 0.5$ for bored piles and ≥ 0.5 for driven piles

Layer No.	DL	γ	depth	q	C	C_1	α
1	4	17	2	34.00	18	0.5	0.666
	4	17	6	136.00	18	0.5	1.242
	4	17	10	306.00	18	0.5	1.789
2	2	21	13	579.00	15	0.5	2.588
3	3	19	15.5	873.50	20	0.5	2.736

$$f_s = \alpha C + q K_s \tan \delta$$

Where: f_s = pile shaft skin resistance

C = cohesion

C_u = undrained cohesion

K = coefficient of lateral earth pressure ranging from K_o to about 1.75

δ = effective friction angle between soil and pile material. (0.67ϕ to ϕ)

Q = effective average vertical stress on element ΔL

α = coefficient to be read from a graph or computed as above

Enter diameter of pile = 0.6 m

Layer	ΔL	ϕ	δ	K_a	K_o	K_p	F_w	C	α
1	4	26	19.5	-0.26	0.2	4.8	2	18	0.67
	4	26	19.5	0.26	0.2	16.0	2	18	1.24
	4	26	19.5	0.36	0.2	24.8	2	18	1.79
2	2	35	26.3	1.01	1.4	618.4	2	15	2.59
3	3	29	21.8	1.76	1.7	-23.7	2	20	2.74

γ	depth	q	K_s	$f_s(\text{kN/m}^2)$	$f_s(\text{kN/m})$
17	2	34	1.2	12.2	23.1
17	6	136	1.8	23.8	44.8
17	10	306	1.8	35.4	66.7
21	13	579	1.8	46.9	88.4
19	15.5	873.5	1.8	64.8	122.2

The average Axial Skin resistance (f_s) is 70kN/m.

The β -METHOD according to Burland

Recommended for cohesion less soils

$$f_s = \beta(q + q_s)$$

Where: f_s = pile shaft skin resistance

$$\beta = K \tan \delta$$

K = at-rest coefficient of lateral earth pressure, K_o

δ = effective friction angle between soil and pile material. (0.67ϕ to ϕ)

q = effective average vertical stress on element ΔL

q_s = surcharge

Enter diameter of pile = 0.6 m

Layer	ΔL (m)	γ (kN/m ³)	depth (m)	q (kN/m ²)	q_c (kN/m ²)	ϕ	K_o	δ	β	f_s (kN/m ²)	f_s (kN/m)
1	4	17	2	34	90	26	0.56	17.4	0.18	21.85	41.19
	4	17	6	102	90	26	0.56	17.4	0.18	33.83	63.78
	4	17	10	170	90	26	0.56	17.4	0.18	45.82	86.36
2	2	21	13	273	90	35	0.43	23.4	0.18	67.14	126.56
3	3	19	15.5	294.5	90	29	0.52	19.4	0.18	69.88	131.71

The average axial skin resistance (f_s) is 90kN/m.

For the analysis, axial skin resistance (f_s) has been taken as 70kN/m.

Ultimate Pile Base Resistance

Ultimate pile point resistance based on “c & ϕ ” of soil according to Terzaghi

$$P_{ub} = A (CN_c + qN_q + 0.5B'\gamma N_\gamma)$$

$\phi < 25$

$\phi > 25$

C (kN/m ²)	ϕ (deg)	ϕ (rad)	$Kp\gamma$	γ' (kN/m ³)	tip γ' (kN/m ³)	depth (m)	DIA M. (m)	length (m)	q (kN/m ²)	N_q	N_c	N_γ	P_{bu} (kN)	$Kp\gamma$
20	29	0.506	43	19	19	17	0.6	17	323	19.98	34.2	17.3	2,046	48.6

Ultimate pile point resistance is: $2046\text{kN} + Q = 2046 + 25.5 = 2070\text{kN}$.

The elastic behavior of the pile is defined by the following parameters. (See Table 5.10)

Table 5. 10: Elastic behavior of pile.

E	Young's modulus in the axial direction.
A	Pile cross-section area.
I	Moment of inertia against bending around the pile axis.

The pile wall system is, spaced 1.8m and meshed and shotcrete. It has a thickness of 0.6 m and a length of 17 m, with a 5 m embedment thickness (see Figure 5.7). The pile has the following material properties. (See Table 5.11)

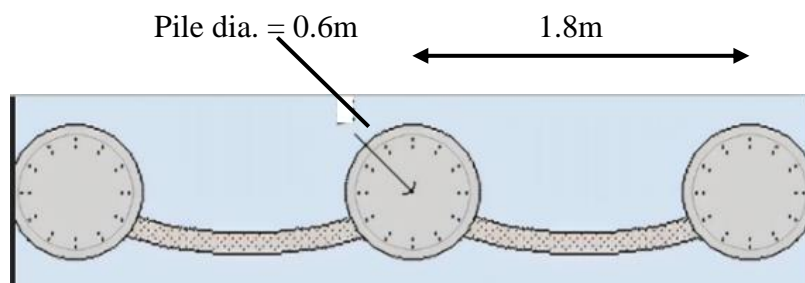


Figure 5. 7: Soldier Bored pile wall

Table 5. 11: Material properties of the pile.

Parameter	Name	Value	Unit
Material type	-	Elastic	-
Stiffness	E	29.96E6	kN/m ²
Unit weight	γ	25	kN/m ³
Beam type	-	Predefined	-
Predefined beam type	-	Massive circular beam	-
Diameter	D	0.6	m
Pile spacing	$L_{spacing}$	1.8	m
Axial Skin resistance		Linear	-
	$T_{skin, start, max}$	70	kN/m
	$T_{skin, end, max}$	70	kN/m
Lateral resistance		Unlimited	-
Base resistance	F_{max}	2070	kN
Interface stiffness factor	Default values	Yes	-

5.2.4 Discretization

When the geometry model is fully defined, the geometry has to be divided into finite elements to perform finite element calculations. A composition of finite elements is called a mesh. The mesh should be sufficiently fine to obtain accurate numerical results. On the other hand, very fine meshes must be avoided, since this will lead to excessive calculation times.

The PLAXIS 2D program uses a fully automatic generation of finite element meshes. Mesh generation uses a robust triangulation procedure. The mesh generation process account; for the soil stratigraphy and all structural objects, loads, and boundary conditions.

5.2.4.1 Mesh Sensitivity Analysis

Mesh sensitivity analysis within defined element distribution and element dimensions that are; from very coarse to very fine, and from 10 m to 2.5 m to check the effect of the variation on the maximum lateral deflection, bending moment, and shear force on the soldier pile wall. From Table 5.12, lateral displacement and bending moment are not sensitive to variation of mesh size, but the shear force is a little bit sensitive to mesh size. The fine element distribution is used in this study to construct the mesh. (See Figure 5.8)

Table 5. 12: Effect of mesh size on the result of pile wall displacement and internal forces

Element distribution	Element dimension (m)	Relative element size	Lateral displacement (mm)	Bending moment (kN-m/m)	Shear force (kN/m)
Very Coarse	10	2	11	77	76
Coarse	6.7	1.33	11	78	74
Medium	5	1	11	78	68
Fine	3.33	0.67	11	79	69
Very Fine	2.5	0.5	11	81	71

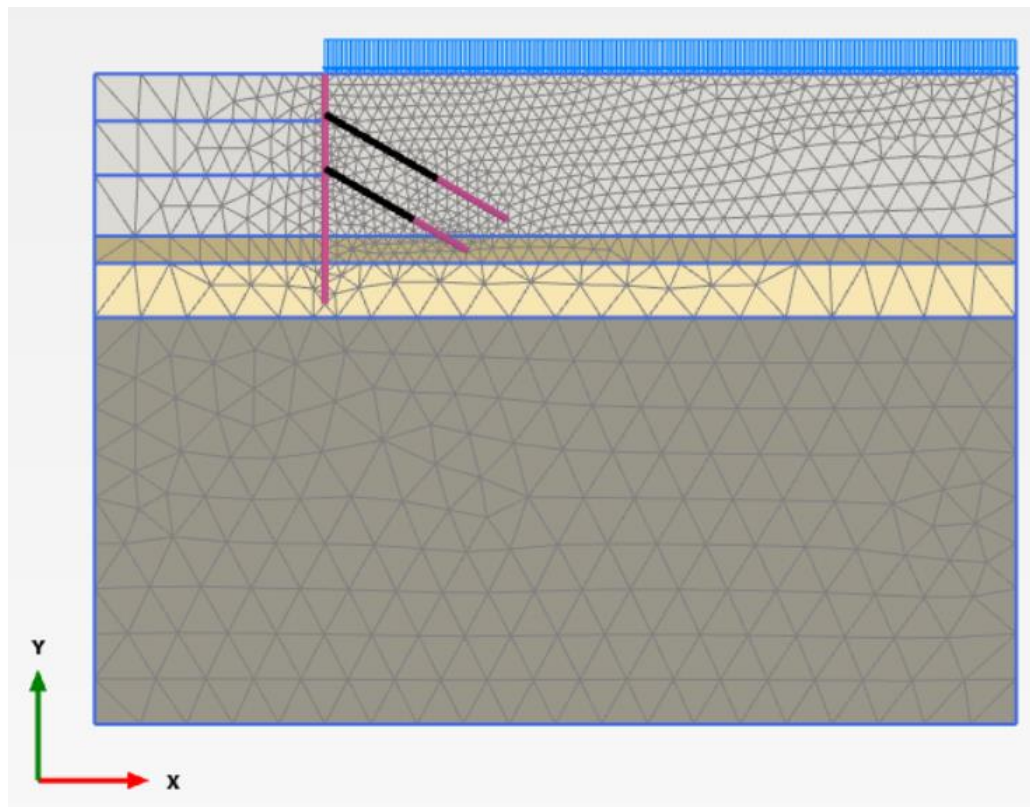


Figure 5. 8: Finite element meshing.

5.2.5 Staged Construction

During the soil parameters assigning and the finite element mesh generation, the initial stress was not accounted, for the generation of the finite element model. Calculating the initial stresses within the soil body requires a unique procedure. Only the original soil body exists, and all structural components and geometry changes, such as backfilling, excavation, and all structural elements, as well as groundwater modifications (e.g. dewatering, sudden water drawdown), must be turned off.

The K_0 technique generates the first effective stress in this investigation. This procedure is correct only and only when all the geometry of the ground surface, the ground layers, and the groundwater table are horizontal.

The excavation was simulated in seven phases (staged construction) with a plastic calculation type.

Phase 1: Initial phase, only the original soil body exists;

Phase 2: There is no excavation the wall and surcharge load are activated;

Phase 3: Excavation in 3.5m layer thickness;

Phase 4: Installation of the first-row tieback anchors and pre-stressed;

Phase 5: Excavation in 4 m layer thickness;

Phase 6: Installation of the second-row tieback anchors and pre-stressed;

Phase 7: Final excavation in 4.5 m layer thickness;

Phase 8: Factor of safety calculation;

The thickness of the building excavation was taken, from the practical application in the chosen project.

If there are doubts about excessive deformations, an additional safety calculation may be needed using the same design approach and then result in a stable ΣM_{sf} value larger than 1.0. For safety calculation, Phase 8 must be included, in stage construction.

5.3 The Finite Element Analysis Result

When an embedded beam row is displayed, the options sum phase displacement ΣP_{ux} is available from the deformation menu, and shear force Q and bending moment M are available from the forces menu in the plaxis 2D calculation result window. Figure 5.9 displayed the wireframe distribution of the sum phase displacement results in an embedded beam row cross-section, namely Pile.

The lateral deflection of the wall shows that it has moved to the excavation over its entire length (see Figure 5.9), with the wall deflection located at $0.6L_p$ from the top of the pile and its maximum value equal to 13 mm.

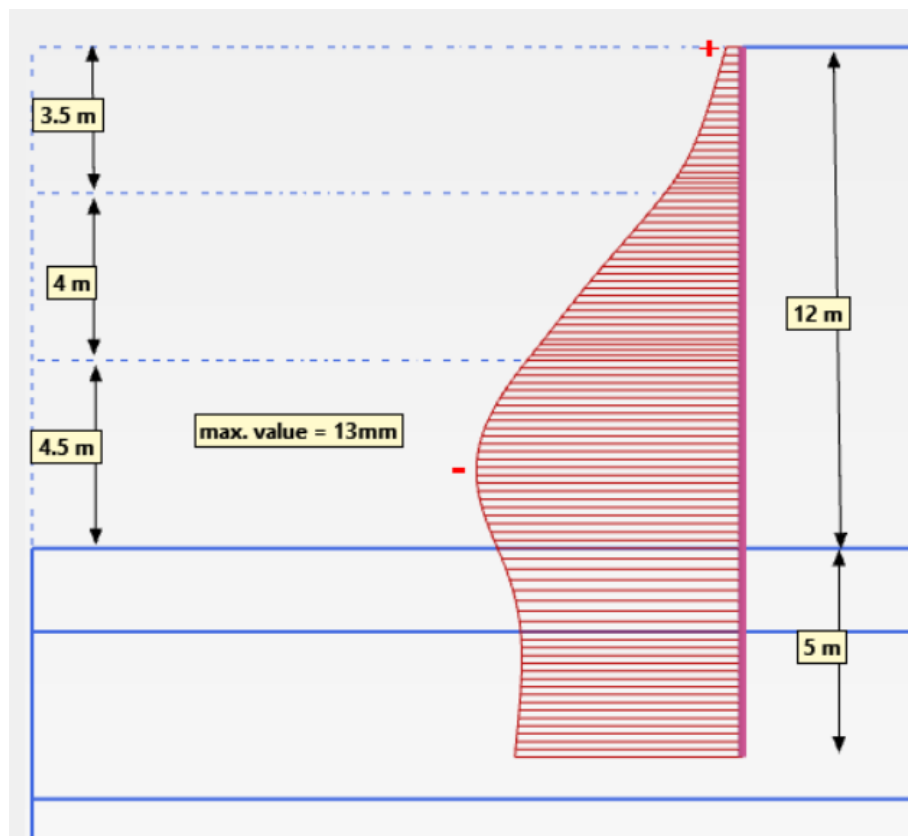


Figure 5. 9: Lateral deflections of the pile wall at the final stage of excavation when using Plaxis 2D software

The anchor load effect can become visible in the bending moment diagram. As the lateral deflections diagram, the maximum bending moment is at $0.6L_p$ from the top of the pile, and its value is equal to 80 kN- m/m. see Figures 5.10

Unlike the above diagram, the maximum shear force on the shear diagram is at $0.7L_p$ from the top of the pile, which is at the final depth of excavation, and its value is equal to 68 kN/m. see Figures 5.11

Safety calculation results indicate ΣM_{sf} value larger than 1.0. Figure 5.12 displays the $\Sigma M_{sf} - |u|$ plot for safety calculations of Phase 6 at the nodes located at the anchor wall connection. The factor of safety found in this analysis equals 2. Therefore, slope stability is acceptable.

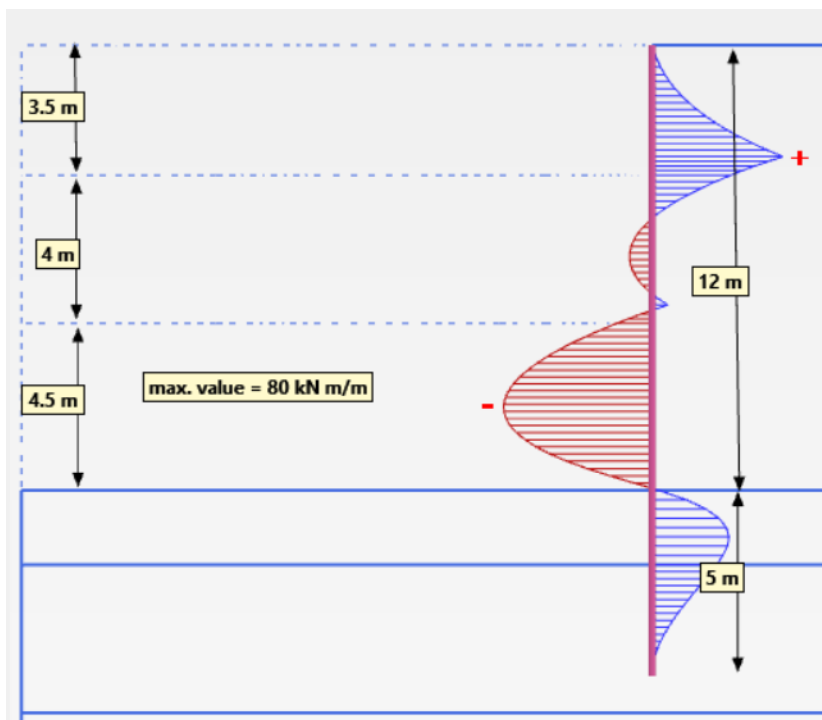


Figure 5. 10: Bending Moment of the pile wall at the final stage of excavation when using Plaxis 2D software

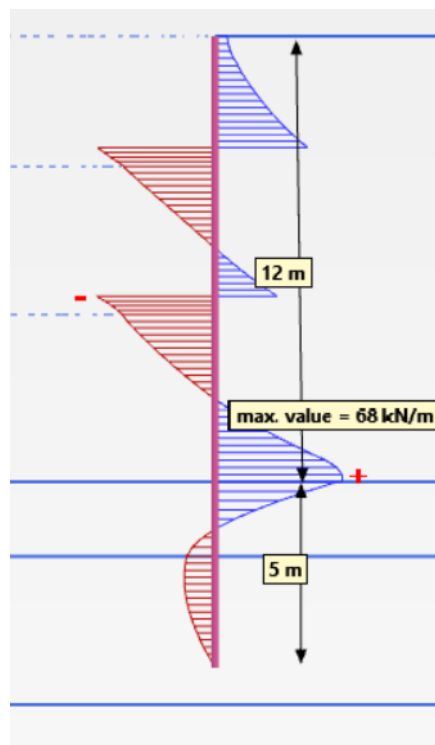


Figure 5. 11: Shear Force of the pile wall at the final stage of excavation when using Plaxis 2D software

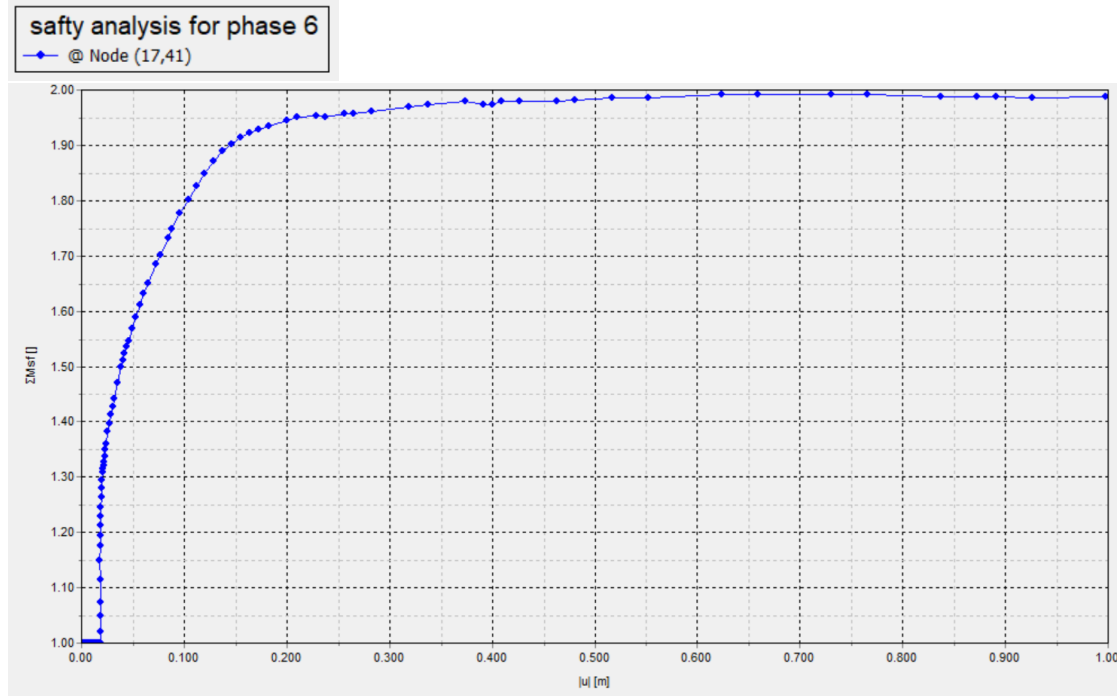


Figure 5. 12: ΣM_{sf} - $|u|$ plot for the last calculation phase

5.4 Optimized Finite Element Analysis Result

The wall system was reinforcement shotcrete with a 1.8 m span. Because our subsurface is a cohesion-less soil, the "stand-up" time for reinforcement shotcrete installation may be limited. As a result, optimizing pile spacing is difficult. It's also inconvenient to optimize pile diameter because there are only a few pile case diameters of 40 cm, 60 cm, and 80 cm available on the market. As a result, optimizing pile embedment depth is feasible.

The optimum embedded depth of the soldier piles should not be less than 0.22 H (H is the excavation depth of the foundation pit), equal to 2.64 m. Thus, optimization is done on 4 m embedment depth and 3m embedment depth.

Finite element results from Plaxis 2D software after optimization of embedment depth, from 5 m depth to 4 m and 3 m depth, shown in Table 5.13 and Table 5.14, respectively. Appendices B1 to B7 show the finite element result diagrams. On these embedment depths, slope stability is acceptable, and the factor of safety found is 1.9.

Table 5. 13: Lateral deflection, Bending Moment, and Shear Force after embedment depth optimization to 4 m

Req. parameters	Plaxis 2D result after 4m embedment depth optimization
Max. ΣPu_x (mm)	13
Max. BM (kN m/m)	83
Max. SF (kN/m)	66

Table 5. 14: Lateral deflection, Bending Moment, and Shear Force after embedment depth optimization to 3 m

Req. parameters	Plaxis 2D result after 3m embedment depth optimization
Max. ΣPu_x (mm)	13
Max. BM (kN m/m)	89
Max. SF (kN/m)	65

6 Result Discussion

Comparing different analysis options using the Mohr-Coulomb model, the following analysis results are required.

1. Maximum Lateral Deflections of the pile wall
2. Maximum Bending Moment of the Pile
3. Maximum Shear Force of the Pile

6.1 Comparison of Maximum Lateral Deflection, Bending Moment, and Shear Force

A comparison of the finite element analysis with conventional analysis results is presented in Table 6.1

Table 6. 1: Comparison of max. Lateral deflection, Bending moment, and Shear force

Req. parameters	Conventional Analysis			FEA		
	DeepEx	GEO5	DeepEx (Beam on Elastic Foundation)	Plaxis 2D(Original)	Plaxis 2D(Optimized to 4m)	Plaxis 2D(Optimized to 3m)
Max. ΣP_{u_x} (mm)	4	26	6.4	13	13	13
Max. BM (kN m/m)	115	171	126	80	83	89
Max. SF (kN/m)	103	127	120	68	66	65

The maximum Horizontal displacement value obtained with the above analysis methods should go together with the maximum bending moment's values. But as we see from Table 6.1, DeepEx's maximum BMs are 115 kN-m/m and 126 kN-m/m, and its maximum phasal horizontal displacements are 4mm and 6.4mm, which contradict the above assumption. Whereas those obtained from FEM are 80 kN-m/m to 89 kN-m/m, maximum bending moments and its maximum phasal horizontal displacement is 13mm, which agree with the above assumption. Shows the obtained DeepEx result is unrealistic and exaggerated.

6.2 ULS Design of Circular Reinforced Concrete Pile

According to:

EN 1992-1-1:2004+AC2:2010 Section 6.1

Input

Concrete characteristic strength f_{ck} , MPa	25
Steel characteristic yield strength f_{yk} , MPa	400
External diameter of cross-section D_{ext} , m	0.6
Diameter of bars Φ_1 , mm	20
Distance from bar centroid of external reinforcement c'_1 , m	0.06
The design value of the axial force N_{Ed} , kN surcharge load * pile cross-sectional area $90 \text{ kN/m}^2 * \pi * 0.6^2 * 0.25$	-25.5

From Conventional Analysis Result, DeepEx software

Design value of the bending moment $M_{Ed} = 207 \text{ kN-m} * 1.4 = 290 \text{ kN-m}$

Results

Reinforcement - Total $A = 37.19 \text{ cm}^2$

Required Reinforcement 12 $\Phi 20$, for detail calculation see Appendix C.1

From Conventional Analysis Result, GEO5 software

Design value of the bending moment $M_{Ed} = 239 \text{ kN-m/m} * 1.8\text{m} = 430 \text{ kN-m}$

The bending moment $M_{Ed} = 239 \text{ kN-m/m}$, found from software is a design load per meter width, thus no need for additional load factor only pile spacing applied.

Results

Reinforcement - Total $A = 59.69 \text{ cm}^2$

Required Reinforcement 19 $\Phi 20$, for detailed calculation, see Appendix C.2

From Beam on Elastic Foundation Analysis Result, the DeepEx software

Design value of the bending moment $M_{Ed} = 226 \text{ kN-m} * 1.4 = 317 \text{ kN-m}$

Results

Reinforcement - Total $A = 41.26 \text{ cm}^2$

Required Reinforcement 13 Φ 20, for detailed calculation, see Appendix C.3

From Finite Element Analysis Result

Design value of the bending moment $M_{Ed} = 144 \text{ kN-m} * 1.4 = 200 \text{ kN-m}$

Results

Reinforcement - Total A = 24.33 cm^2

Required Reinforcement 8 Φ 20, for detail calculation see Appendix C.4

From Optimized Finite Element Analysis Result (4m embed)

Design value of the bending moment $M_{Ed} = 149 \text{ kN-m} * 1.4 = 204 \text{ kN-m}$

Results

Reinforcement - Total A = 24.9 cm^2

Required Reinforcement 8 Φ 20, for detail calculation see Appendix C.5

From Optimized Finite Element Analysis Result (3m embed)

Design value of the bending moment $M_{Ed} = 160 \text{ kN-m} * 1.4 = 221 \text{ kN-m}$

Results

Reinforcement - Total A = 27.29 cm^2

Required Reinforcement 9 Φ 20, for detail calculation see Appendix C.6

Table 6. 2: Comparison of Required Reinforcement

	Conventional Analysis			FEA		
	DeepEx	GEO5	DeepEx (Beam on Elastic Foundation)	Plaxis 2D (Original)	Plaxis 2D(Optimized to 4m)	Plaxis 2D(Optimized to 3m)
Req. Rebar	12 Φ 20	19 Φ 20	13 Φ 20	8 Φ 20	8 Φ 20	9 Φ 20

From the above calculation result and Table 6.2 the Finite Element Analysis required less reinforcement than the conventional analysis. Due to the reduction of embedment depth, its moment increased to some amount that required an additional number of rebar than the original design.

6.3 Piling and Shotcrete work price comparison

Comparison of the total price of piling and shotcrete work based on the various analyses shown in Table 6.3. Appendix D1 - D6 includes a detailed Bill of Quantities.

Table 6. 3: Comparison of the total price of piling and shotcrete work

	Conventional Analysis			FEA		
	DeepEx	GEO5	DeepEx Beam on elastic foundation	Plaxis 2D(original)	Plaxis 2D(optimized to 4m)	Plaxis 2D(optimized to 3m)
Total Price (Birr)	34,861,319	38,014,465	34,230,690	31,077,544	30,061,920	29,602,733

From the above table, Finite Element Analysis is more cost-effective than the Conventional analysis method, 9% - 11% cost reduction. The optimized Finite Element Analysis is also a 14% to 15% cost reduction over the Conventional Analysis method.

7 Conclusion and Recommendation

7.1 Conclusion

The most important observations regarding the result obtained from the numerical analysis and analytical analysis can be summarized as follows:

- The conventional analysis gives bending moment values ranging from 115 kN-m/m to 171 kN-m/m, whereas FEA gives values ranging from 80 kN-m/m to 89 kN-m/m. Similarly, the shear force values from conventional analysis range

from 103 to 127 kN/m, while those from FEA range from 65 to 68 kN/m. Based on the above results, we may conclude that the internal force obtained from FEA is lower than that from conventional analysis methods.

- The deflection values of 4 mm, 6.4 mm, and 26 mm are conventional analysis results, whereas 13 mm is the FEA result. Greater bending moments result in more deflection than expected. But the above results show the opposite. As a result, we can conclude that the deflection value obtained by conventional analysis methods is unrealistic.
- The required reinforcement per pile for the supported deep excavation work is 8Φ20 from the Finite Element Analytical method, whereas from conventional analysis methods are 12Φ20, 13Φ20, and 19Φ20.
- As per optimized Finite Element Analysis, the required reinforcement per pile is 8Φ20 and 9Φ20.
- The quantity of reinforcement required by FEA and optimized FEA methods is less than that required by conventional analytical methods.
- The required number of Piles for the overall length of the proposed shored area, 196m, is 109pcs.
- Finite Element Analysis is more cost-effective than the Conventional analysis method, 9% - 11% cost reduction. The optimized Finite Element Analysis is also a 14% to 15% cost reduction over the Conventional Analysis method.

7.2 Recommendation

The most widely used conventional analysis result indicates that the structure's deflection is unrealistic. That is critical for serviceability considerations, and the determined internal forces are over-estimated. Since the developments of software programs allow for improved simulation of actual field conditions, thus the finite element method (FEM) is used for retaining wall design. Even complex geometries and supporting systems can be modeled in 2D and 3D using FEM. Despite of this, the finite element method for supported excavation analysis has yet to be adopted by geotechnical engineers.

The recommendation is that geotechnical engineers use the Finite Element technique rather than the conventional analytical method.

8 References

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9 Appendix A: Es values for soils

Modulus of stress-strain, Es, values for soils (Joseph E. Bowles, RE., S.E)

soil description		Es(kPa)
clay	very soft	2,000-15,000
	soft	5,000-25,000
	medium	15,000-50,000
	hard	50,000-100,000
	sandy	25,000-250,000
Glacial till	loose	10,000-150,000
	dense	150,000-720,000
	very dense	500,000-1,440,000
loess		15,000-60,000
sand	silty	5,000-20,000
	loose	10,000-25,000
	dense	50,000-81,000
sand & gravel	loose	50,000-150,000
	dense	100,000-200,000
shale		150,000-5,000,000
silt		2,000-20,000

10 Appendix B: Optimized FEA Analysis Results Diagram

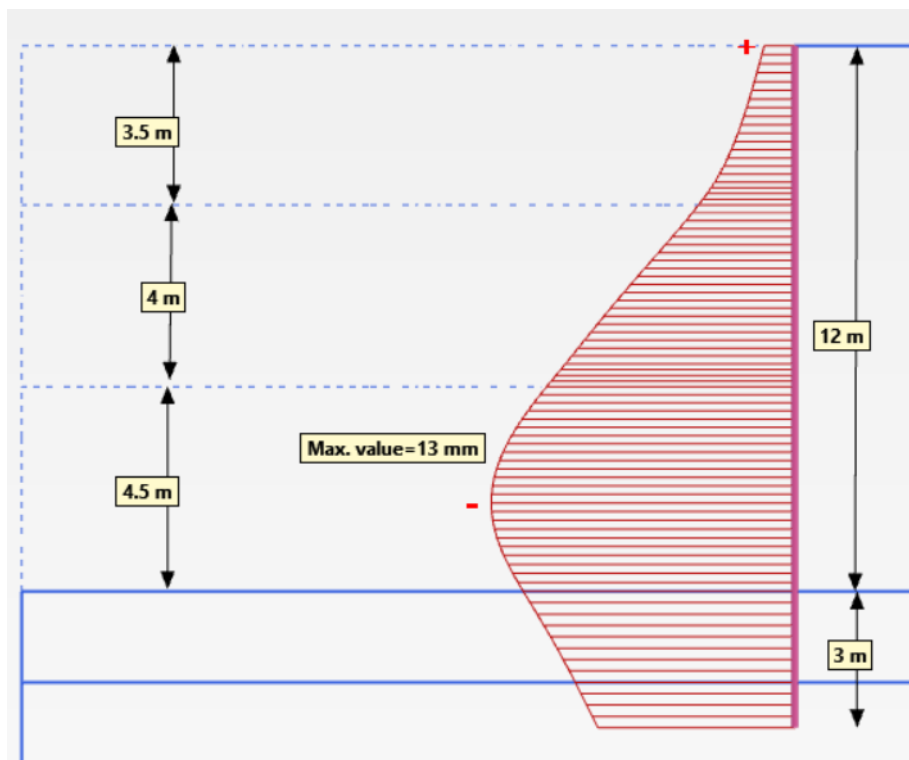


Figure B.1: Lateral deflections of the pile wall at the final stage of excavation when using Plaxis 2D software (3m embedment depth)

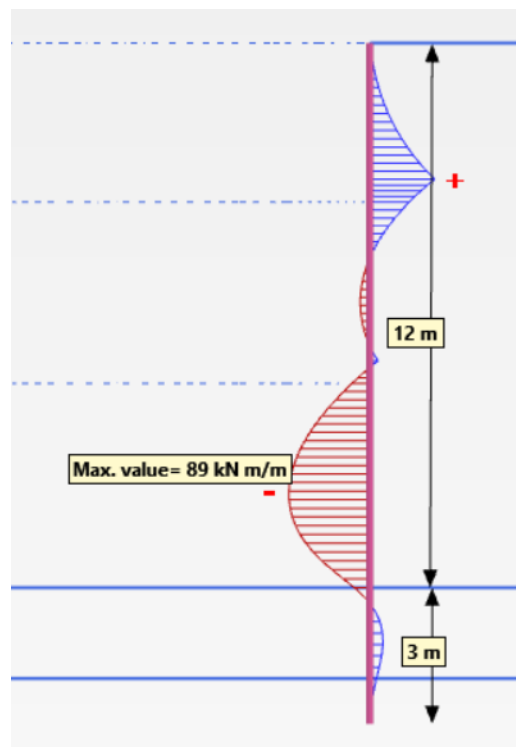


Figure B.2: Bending Moment of the pile wall at the final stage of excavation when using Plaxis 2D software (3m embedment depth)

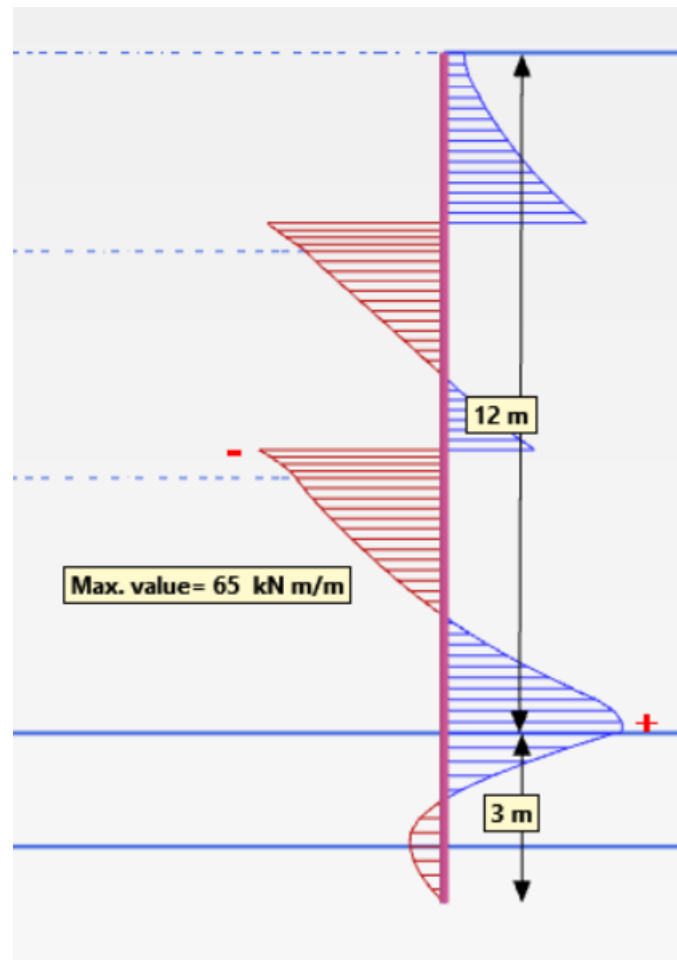


Figure B.3: Shear Force of the pile wall at the final stage of excavation when using Plaxis 2D software (3m embedment depth)

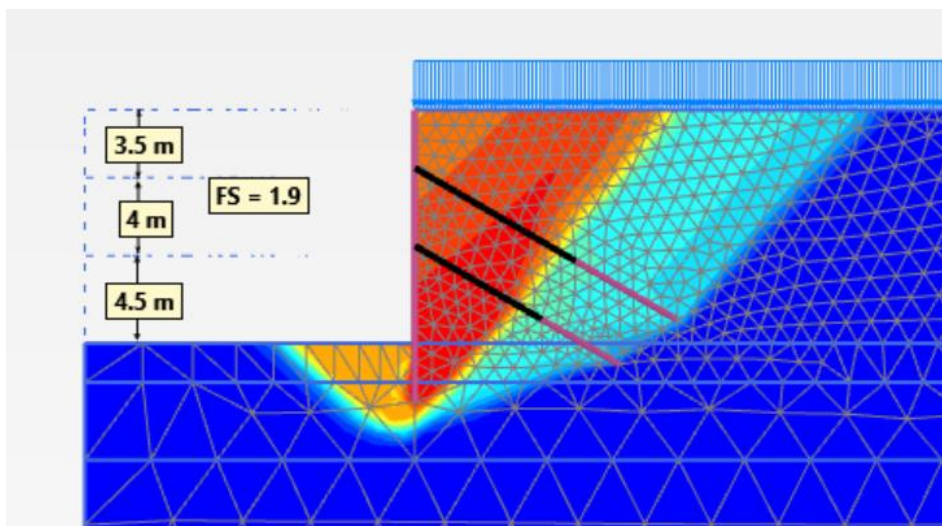


Figure B.4: slope stability

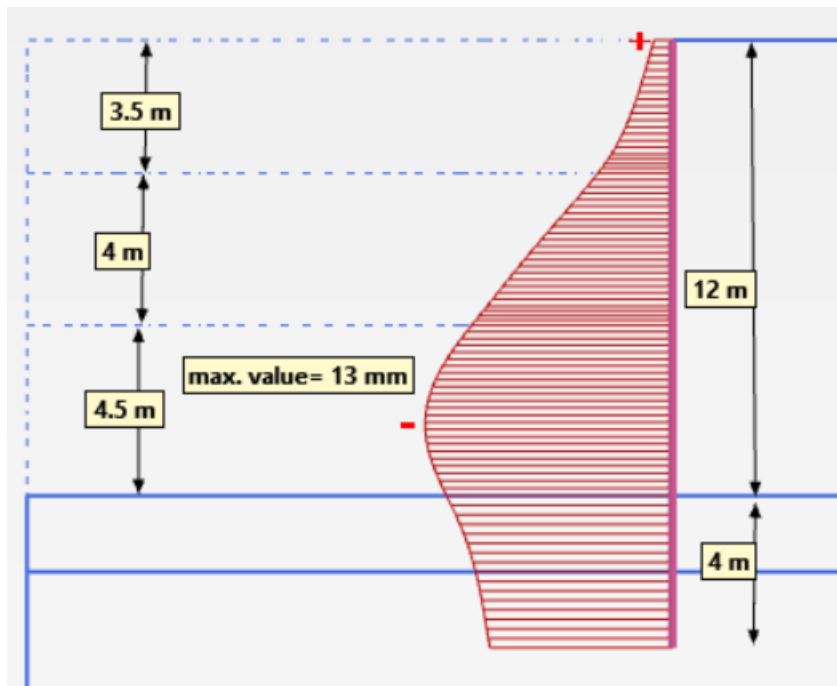


Figure B.5: Lateral deflections of the pile wall at the final stage of excavation when using Plaxis 2D software (4m embedment depth)

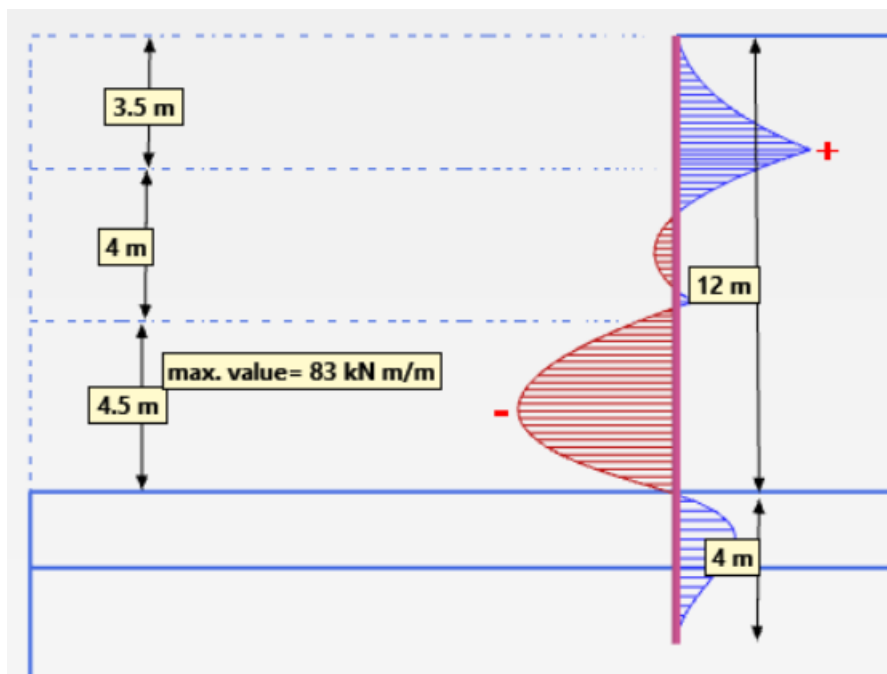


Figure B.6: Bending Moment of the pile wall at the final stage of excavation when using Plaxis 2D software (4m embedment depth)

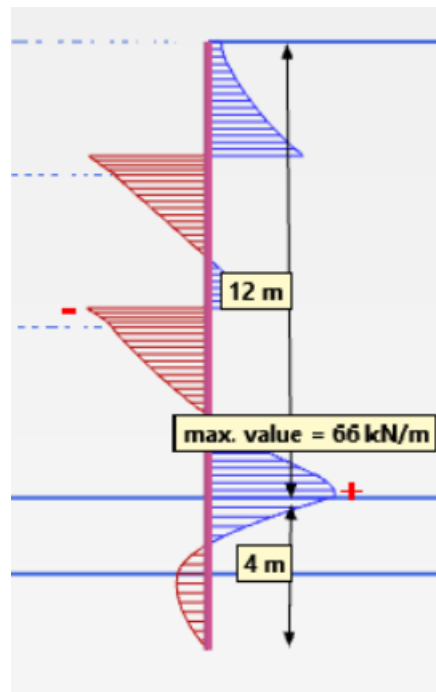


Figure B.7: Shear Force of the pile wall at the final stage of excavation when using Plaxis 2D software (4m embedment depth)

11 Appendix C: ULS design of circular reinforced concrete pile

11.1 Appendix C.1: DeepEx- steel Area



Project: 290

Subject:

Designer:

Date:

Eurocode 2

ULS design of circular (or tubular) reinforced concrete cross-section for bending and axial force

Description:

ULS design of circular or tubular reinforced concrete cross-section for bending and axial force (calculation of required reinforcement for given bending moment / axial force, maximum moment resistance MRd for given axial force, or complete M-N interaction diagram for given reinforcement layout)

According to:

EN 1992-1-1:2004+AC2:2010 Section 6.1

Supported National

Annexes:

Nationally Defined Parameters (NDPs) automatically filled for supported countries

Input

Concrete characteristic strength $f_{ck} = 25$ MPa

Steel characteristic yield strength $f_{yk} = 400$ MPa

Type of reinforcement stress-strain law = No hardening (Perfectly plastic) ▼

External diameter of cross-section $D_{ext} = 0.6$ m

Internal diameter (for hollow tube sections) $D_{int} = 0$ m

Initial reinforcement: When the required reinforcement is requested the initial reinforcement will be modified accordingly.

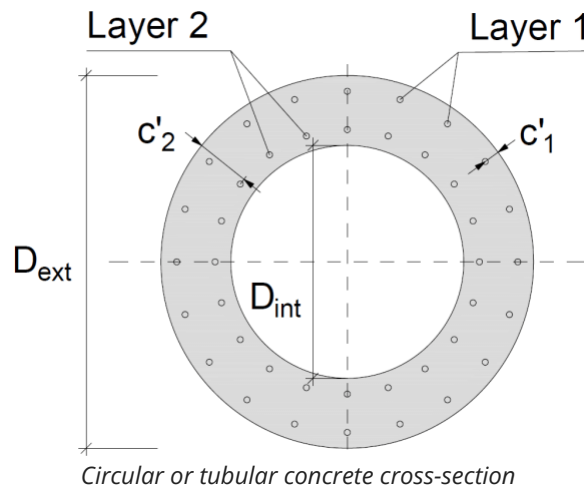
Number of bars of external reinforcement layer 1 $n_{bars,1} = 10$

Diameter of bars of external reinforcement layer 1 $\phi_1 = 20$ mm

Distance from bar centroid of external $c'_1 = 0.06$ m

Number of bars of internal reinforcement layer 2	$n_{bars,2} = 0$	
Diameter of bars of internal reinforcement layer 2	$\phi_2 = 20$	mm
Distance from bar centroid of internal reinforcement layer 2 to <u>external</u> concrete edge	$c'_2 = 0.175$	m

Type of analysis	= Required reinforcement A_s	▼
Design value of the bending moment (positive when tension at bottom side)	$M_{Ed} = 290$	kNm
Design value of the axial force (compression negative)	$N_{Ed} = -25.5$	kN
Reinforcement layers to be modified	= Layers 1 & 2	▼



Nationally Defined Parameters

Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending)	$\alpha_{cc} = 1$
Concrete partial material safety factor	$\gamma_c = 1.5$
Reinforcement steel partial material safety factor	$\gamma_s = 1.15$
Design value of reinforcement ultimate strain	$\epsilon_{ud} = 0.9\epsilon_{uk}$

Results

Maximum design bending moment resistance for given design axial force

Reinforcement - Layer 1 (external)

Reinforcement - Layer 2 (internal)

Reinforcement - Total

Check of total axial force convergence

Check of bending moment convergence

$$M_{Rd} = 290.00 \text{ kNm}$$

$$A_{s,1} = 37.19 \text{ cm}^2$$

(initial $\times 1.184$)

$$A_{s,2} = 0.00 \text{ cm}^2$$

$$A_{s,tot} = 37.19 \text{ cm}^2 (1.32\%)$$

$$|\Delta N| = 0.00 \text{ N} \leq 1 \text{ N} \Rightarrow \text{Ok} \quad |\Delta M| = 0.00 \text{ Nm} \leq 1 \text{ Nm} \Rightarrow$$

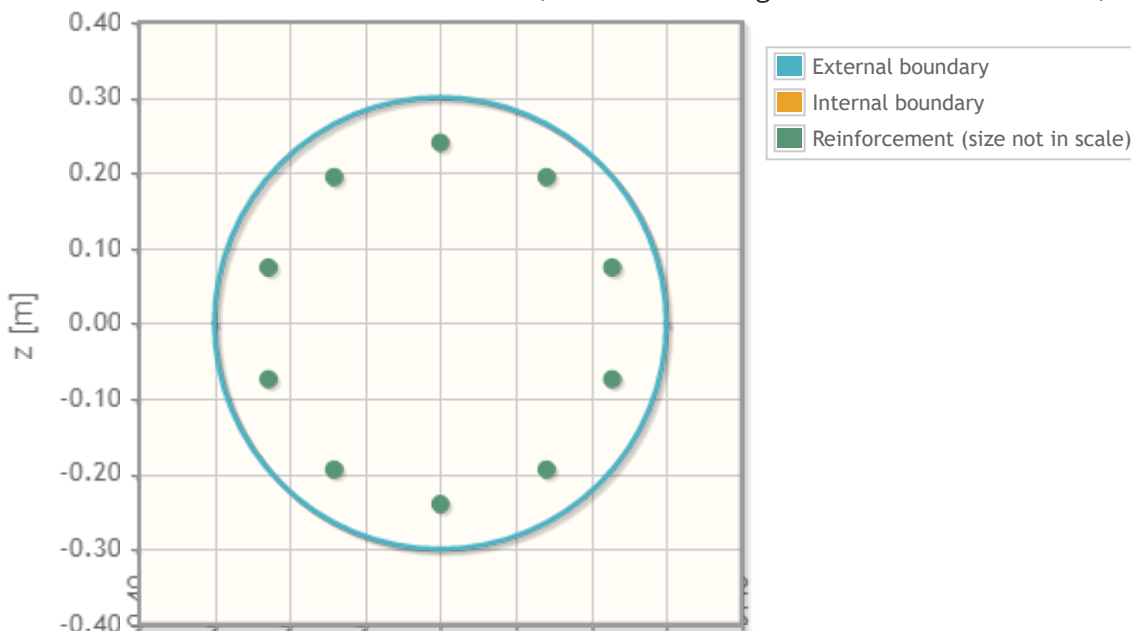
Ok

Notes

1. Positive bending moment produces tension at the bottom side. Negative bending moment produces tension at the top side.
2. In this calculation the area of the reinforcement bars is subtracted from the surrounding concrete area in order to avoid considering the same area for both steel and concrete. This correction is not significant in general, except for small highly-reinforced cross-sections.
3. The indicated reinforcement layers have been modified by multiplication by the same common factor that is indicated above. The layers in tension are generally more effective. When large compressive axial force is applied increasing the layers in compression may be beneficial.
4. According to EN1992-1-1 § 6.1(4) when the axial force N_{Ed} is compressive the absolute value of the design bending moment should be at least $|M_{Ed}| \geq e_0 \times |N_{Ed}|$ where e_0 is the minimum assumed eccentricity of the compressible load which is equal to $e_0 = \max(h/30, 20 \text{ mm})$, where h is the total depth of the cross-section.

Charts

Cross-section (hover=value, drag=zoom, double-click=reset)



Tables

Copy to Clipboard

Interaction diagram - Axial force resistance N_{Rd} vs bending moment resistance M_{Rd}

N-M interaction			Lever arm & neutral axis				
#	Design axial force resistance N_{Rd} [kN]	Design bending moment resistance M_{Rd} [kNm]	Location of neutral axis from centroid y_n [m]	Lever arm of internal forces z [m]	Normalized level arm z/d [-]	Depth of compressive zone x [m]	Normalized depth of compressive zone x/d [-]
1	-25.50	290.00	0.154	0.314	0.581	0.146	0.270

Details

Input Data

- Concrete characteristic strength: $f_{ck} = 25$ MPa
- Steel characteristic yield strength: $f_{yk} = 400$ MPa
- Type of reinforcement stress-strain law: = No hardening (Perfectly plastic)
- Ratio of characteristic ultimate strength to characteristic yield strength: $(f_t/f_y)_k = 1$
- Characteristic value of ultimate strain: $\epsilon_{uk} = 75$ ‰
- External diameter of cross-section: $D_{ext} = 0.6$ m
- Internal diameter (for hollow tube sections): $D_{int} = 0$ m
- Number of bars of external reinforcement layer 1: $n_{bars,1} = 10$
- Diameter of bars of external reinforcement layer 1: $\Phi_1 = 20$ mm
- Distance from bar centroid of external reinforcement layer 1 to external concrete edge: $c'_1 = 0.06$ m
- Number of bars of internal reinforcement layer 2: $n_{bars,2} = 0$
- Diameter of bars of internal reinforcement layer 2: $\Phi_2 = 20$ mm
- Distance from bar centroid of internal reinforcement layer 2 to external concrete edge: $c'_2 = 0.175$ m
- Type of analysis: = Required reinforcement As
- Design value of the bending moment (positive when tension at bottom side): $M_{Ed} = 290$ kNm
- Design value of the axial force (compression negative): $N_{Ed} = -25.5$ kN
- Reinforcement layers to be modified: = Layers 1 & 2

Nationally Defined Parameters

- Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending): $\alpha_{cc} = 1$
- Concrete partial material safety factor: $\gamma_c = 1.5$
- Reinforcement steel partial material safety factor: $\gamma_s = 1.15$
- Design value of reinforcement ultimate strain: $\epsilon_{ud} = 0.9\epsilon_{uk}$

11.2 Appendix C.2: GEO5- Steel Area



Project: 430

Subject:

Designer:

Date:

Eurocode 2

ULS design of circular (or tubular) reinforced concrete cross-section for bending and axial force

Description:

ULS design of circular or tubular reinforced concrete cross-section for bending and axial force (calculation of required reinforcement for given bending moment / axial force, maximum moment resistance MRd for given axial force, or complete M-N interaction diagram for given reinforcement layout)

According to:

EN 1992-1-1:2004+AC2:2010 Section 6.1

Supported National

Annexes:

Nationally Defined Parameters (NDPs) automatically filled for supported countries

Input

Concrete characteristic strength $f_{ck} = 25$ MPa

Steel characteristic yield strength $f_{yk} = 400$ MPa

Type of reinforcement stress-strain law = No hardening (Perfectly plastic) ▼

External diameter of cross-section $D_{ext} = 0.6$ m

Internal diameter (for hollow tube sections) $D_{int} = 0$ m

Initial reinforcement: When the required reinforcement is requested the initial reinforcement will be modified accordingly.

Number of bars of external reinforcement layer 1 $n_{bars,1} = 10$

Diameter of bars of external reinforcement layer 1 $\phi_1 = 20$ mm

Distance from bar centroid of external $c'_1 = 0.06$ m

concrete edge

Number of bars of internal reinforcement layer 2

$$n_{bars,2} = 0$$

Diameter of bars of internal reinforcement layer 2

$$\phi_2 = 20 \quad \text{mm}$$

Distance from bar centroid of internal reinforcement layer 2 to external concrete edge

$$c'_2 = 0.175 \quad \text{m}$$

Type of analysis

$$= \text{Required reinforcement } A_s \quad \checkmark$$

Design value of the bending moment (positive when tension at bottom side)

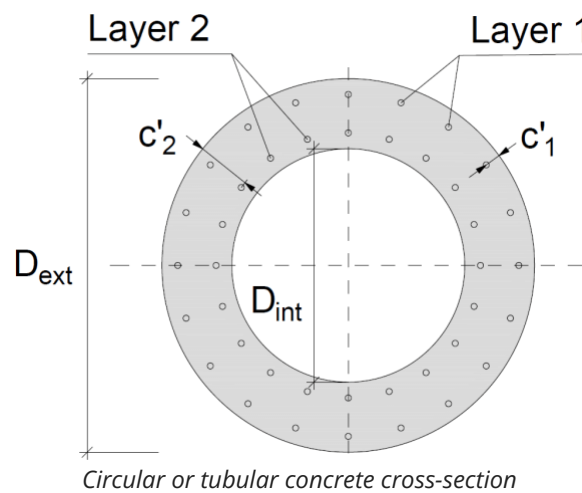
$$M_{Ed} = 430 \quad \text{kNm}$$

Design value of the axial force (compression negative)

$$N_{Ed} = -25.5 \quad \text{kN}$$

Reinforcement layers to be modified

$$= \text{Layers 1 \& 2} \quad \checkmark$$



Nationally Defined Parameters

Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending)

$$\alpha_{cc} = 1$$

Concrete partial material safety factor

$$\gamma_c = 1.5$$

Reinforcement steel partial material safety factor

$$\gamma_s = 1.15$$

Design value of reinforcement ultimate strain

$$\epsilon_{ud} = 0.9\epsilon_{uk} \quad \checkmark$$

Results

Maximum design bending moment resistance for given design axial force

Reinforcement - Layer 1 (external)

Reinforcement - Layer 2 (internal)

Reinforcement - Total

Check of total axial force convergence

Check of bending moment convergence

$$M_{Rd} = 430.00 \text{ kNm}$$

$$A_{s,1} = 59.97 \text{ cm}^2$$

(initial × 1.909)

$$A_{s,2} = 0.00 \text{ cm}^2$$

(initial × 1.909)

$$A_{s,tot} = 59.97 \text{ cm}^2 (2.12\%)$$

$$|\Delta N| = 0.00 \text{ N} \leq 1 \text{ N} \Rightarrow \text{Ok} \quad |\Delta M| = 0.00 \text{ Nm} \leq 1 \text{ Nm} \Rightarrow$$

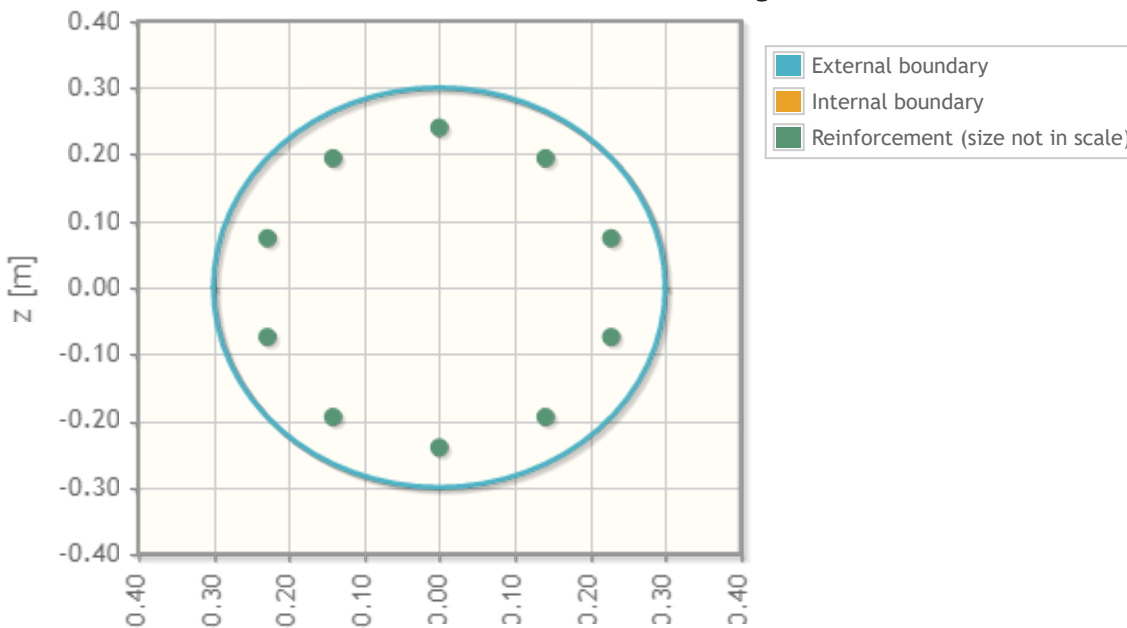
Ok

Notes

1. Positive bending moment produces tension at the bottom side. Negative bending moment produces tension at the top side.
2. In this calculation the area of the reinforcement bars is subtracted from the surrounding concrete area in order to avoid considering the same area for both steel and concrete. This correction is not significant in general, except for small highly-reinforced cross-sections.
3. The indicated reinforcement layers have been modified by multiplication by the same common factor that is indicated above. The layers in tension are generally more effective. When large compressive axial force is applied increasing the layers in compression may be beneficial.
4. According to EN1992-1-1 § 6.1(4) when the axial force N_{Ed} is compressive the absolute value of the design bending moment should be at least $|M_{Ed}| \geq e_0 \times |N_{Ed}|$ where e_0 is the minimum assumed eccentricity of the compressible load which is equal to $e_0 = \max(h/30, 20 \text{ mm})$, where h is the total depth of the cross-section.

Charts

Cross-section (hover=value, drag=zoom, double-click=reset)



Tables

Copy to Clipboard

Interaction diagram - Axial force resistance N_{Rd} vs bending moment resistance M_{Rd}

N-M interaction		Lever arm & neutral axis					
#	Design axial force resistance N_{Rd} [kN]	Design bending moment resistance M_{Rd} [kNm]	Location of neutral axis from centroid y_n [m]	Lever arm of internal forces z [m]	Normalized level arm z/d [-]	Depth of compressive zone x [m]	Normalized depth of compressive zone x/d [-]
1	-25.50	430.00	0.130	0.322	0.596	0.170	0.314

Details

Input Data

- Concrete characteristic strength: $f_{ck} = 25$ MPa
- Steel characteristic yield strength: $f_{yk} = 400$ MPa
- Type of reinforcement stress-strain law: = No hardening (Perfectly plastic)
- Ratio of characteristic ultimate strength to characteristic yield strength: $(f_t/f_y)_k = 1$
- Characteristic value of ultimate strain: $\epsilon_{uk} = 75$ ‰
- External diameter of cross-section: $D_{ext} = 0.6$ m
- Internal diameter (for hollow tube sections): $D_{int} = 0$ m
- Number of bars of external reinforcement layer 1: $n_{bars,1} = 10$
- Diameter of bars of external reinforcement layer 1: $\Phi_1 = 20$ mm
- Distance from bar centroid of external reinforcement layer 1 to external concrete edge: $c'_1 = 0.06$ m
- Number of bars of internal reinforcement layer 2: $n_{bars,2} = 0$
- Diameter of bars of internal reinforcement layer 2: $\Phi_2 = 20$ mm
- Distance from bar centroid of internal reinforcement layer 2 to external concrete edge: $c'_2 = 0.175$ m
- Type of analysis: = Required reinforcement As
- Design value of the bending moment (positive when tension at bottom side): $M_{Ed} = 430$ kNm
- Design value of the axial force (compression negative): $N_{Ed} = -25.5$ kN
- Reinforcement layers to be modified: = Layers 1 & 2

Nationally Defined Parameters

- Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending): $\alpha_{cc} = 1$
- Concrete partial material safety factor: $\gamma_c = 1.5$
- Reinforcement steel partial material safety factor: $\gamma_s = 1.15$
- Design value of reinforcement ultimate strain: $\epsilon_{ud} = 0.9\epsilon_{uk}$

11.3 Appendix C.3: DeepEx - Steel Area (Beam on Elastic Foundation Analysis)



Project: 317

Subject:

Designer:

Date:

Eurocode 2

ULS design of circular (or tubular) reinforced concrete cross-section for bending and axial force

Description:

ULS design of circular or tubular reinforced concrete cross-section for bending and axial force (calculation of required reinforcement for given bending moment / axial force, maximum moment resistance MRd for given axial force, or complete M-N interaction diagram for given reinforcement layout)

According to:

EN 1992-1-1:2004+AC2:2010 Section 6.1

Supported National

Annexes:

Nationally Defined Parameters (NDPs) automatically filled for supported countries

Input

Concrete characteristic strength $f_{ck} = 25$ MPa

Steel characteristic yield strength $f_{yk} = 400$ MPa

Type of reinforcement stress-strain law = No hardening (Perfectly plastic) ▼

External diameter of cross-section $D_{ext} = 0.6$ m

Internal diameter (for hollow tube sections) $D_{int} = 0$ m

Initial reinforcement: When the required reinforcement is requested the initial reinforcement will be modified accordingly.

Number of bars of external reinforcement layer 1 $n_{bars,1} = 10$

Diameter of bars of external reinforcement layer 1 $\phi_1 = 20$ mm

Distance from bar centroid of external $c'_1 = 0.06$ m

concrete edge

Number of bars of internal reinforcement layer 2

$$n_{\text{bars},2} = 0$$

Diameter of bars of internal reinforcement layer 2

$$\phi_2 = 20 \quad \text{mm}$$

Distance from bar centroid of internal reinforcement layer 2 to external concrete edge

$$c'_2 = 0.175 \quad \text{m}$$

Type of analysis

= Required reinforcement A_s

Design value of the bending moment (positive when tension at bottom side)

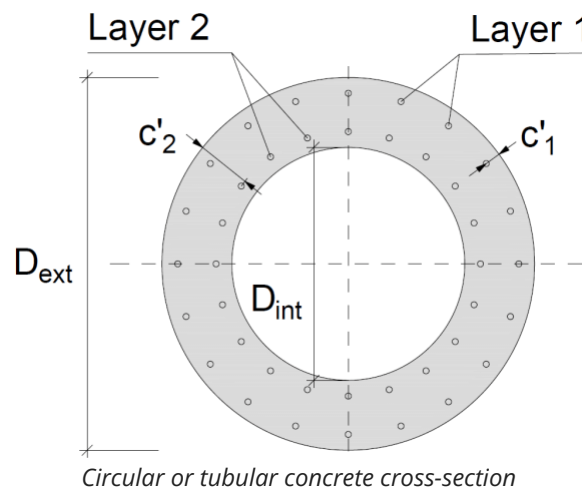
$$M_{Ed} = 317 \quad \text{kNm}$$

Design value of the axial force (compression negative)

$$N_{Ed} = -25.5 \quad \text{kN}$$

Reinforcement layers to be modified

= Layers 1 & 2



Nationally Defined Parameters

Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending)

$$\alpha_{cc} = 1$$

Concrete partial material safety factor

$$\gamma_c = 1.5$$

Reinforcement steel partial material safety factor

$$\gamma_s = 1.15$$

Design value of reinforcement ultimate strain

$$\epsilon_{ud} = 0.9\epsilon_{uk}$$

Results

Maximum design bending moment resistance for given design axial force

$$M_{Rd} = 317.00 \text{ kNm}$$

Reinforcement - Layer 1 (external)

$$A_{s,1} = 41.26 \text{ cm}^2$$

(initial $\times 1.313$)

Reinforcement - Layer 2 (internal)

$$A_{s,2} = 0.00 \text{ cm}^2$$

Reinforcement - Total

$$A_{s,tot} = 41.26 \text{ cm}^2 \text{ (initial } \times 1.313 \text{)}$$

Check of total axial force convergence

$$|\Delta N| = 0.00 \text{ N} \leq 1 \text{ N} \Rightarrow \text{Ok}$$

Check of bending moment convergence

$$|\Delta M| = 0.00 \text{ Nm} \leq 1 \text{ Nm} \Rightarrow \text{Ok}$$

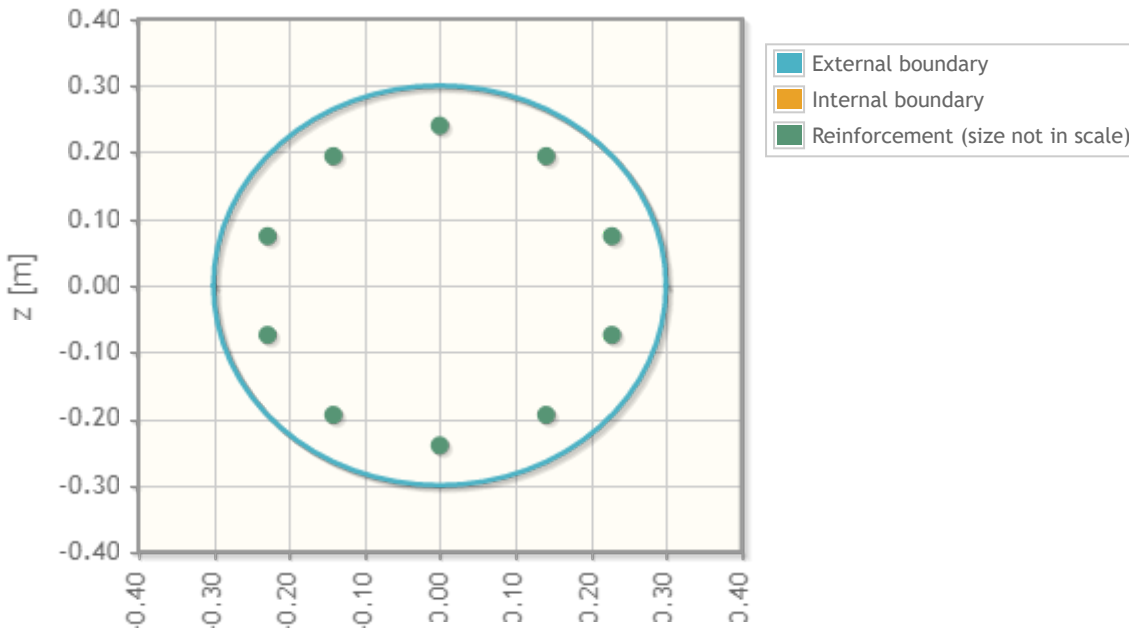
Ok

Notes

1. Positive bending moment produces tension at the bottom side. Negative bending moment produces tension at the top side.
2. In this calculation the area of the reinforcement bars is subtracted from the surrounding concrete area in order to avoid considering the same area for both steel and concrete. This correction is not significant in general, except for small highly-reinforced cross-sections.
3. The indicated reinforcement layers have been modified by multiplication by the same common factor that is indicated above. The layers in tension are generally more effective. When large compressive axial force is applied increasing the layers in compression may be beneficial.
4. According to EN1992-1-1 § 6.1(4) when the axial force N_{Ed} is compressive the absolute value of the design bending moment should be at least $|M_{Ed}| \geq e_0 \times |N_{Ed}|$ where e_0 is the minimum assumed eccentricity of the compressible load which is equal to $e_0 = \max(h/30, 20 \text{ mm})$, where h is the total depth of the cross-section.

Charts

Cross-section (hover=value, drag=zoom, double-click=reset)



Tables

Copy to Clipboard

Interaction diagram - Axial force resistance N_{Rd} vs bending moment resistance M_{Rd}

N-M interaction		Lever arm & neutral axis					
#	Design axial force resistance N_{Rd} [kN]	Design bending moment resistance M_{Rd} [kNm]	Location of neutral axis from centroid y_n [m]	Lever arm of internal forces z [m]	Normalized level arm z/d [-]	Depth of compressive zone x [m]	Normalized depth of compressive zone x/d [-]
1	-25.50	317.00	0.148	0.312	0.578	0.152	0.282

Details

Input Data

- Concrete characteristic strength: $f_{ck} = 25$ MPa
- Steel characteristic yield strength: $f_{yk} = 400$ MPa
- Type of reinforcement stress-strain law: = No hardening (Perfectly plastic)
- Ratio of characteristic ultimate strength to characteristic yield strength: $(f_t/f_y)_k = 1$
- Characteristic value of ultimate strain: $\epsilon_{uk} = 75$ ‰
- External diameter of cross-section: $D_{ext} = 0.6$ m
- Internal diameter (for hollow tube sections): $D_{int} = 0$ m
- Number of bars of external reinforcement layer 1: $n_{bars,1} = 10$
- Diameter of bars of external reinforcement layer 1: $\Phi_1 = 20$ mm
- Distance from bar centroid of external reinforcement layer 1 to external concrete edge: $c'_1 = 0.06$ m
- Number of bars of internal reinforcement layer 2: $n_{bars,2} = 0$
- Diameter of bars of internal reinforcement layer 2: $\Phi_2 = 20$ mm
- Distance from bar centroid of internal reinforcement layer 2 to external concrete edge: $c'_2 = 0.175$ m
- Type of analysis: = Required reinforcement As
- Design value of the bending moment (positive when tension at bottom side): $M_{Ed} = 317$ kNm
- Design value of the axial force (compression negative): $N_{Ed} = -25.5$ kN
- Reinforcement layers to be modified: = Layers 1 & 2

Nationally Defined Parameters

- Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending): $\alpha_{cc} = 1$
- Concrete partial material safety factor: $\gamma_c = 1.5$
- Reinforcement steel partial material safety factor: $\gamma_s = 1.15$
- Design value of reinforcement ultimate strain: $\epsilon_{ud} = 0.9\epsilon_{uk}$

11.4 Appendix C.4: FEA- Steel Area



Project: 200

Subject:

Designer:

Date:

Eurocode 2

ULS design of circular (or tubular) reinforced concrete cross-section for bending and axial force

Description:

ULS design of circular or tubular reinforced concrete cross-section for bending and axial force (calculation of required reinforcement for given bending moment / axial force, maximum moment resistance MRd for given axial force, or complete M-N interaction diagram for given reinforcement layout)

According to:

EN 1992-1-1:2004+AC2:2010 Section 6.1

Supported National

Annexes:

Nationally Defined Parameters (NDPs) automatically filled for supported countries

Input

Concrete characteristic strength	$f_{ck} = 25$	MPa
Steel characteristic yield strength	$f_{yk} = 400$	MPa
Type of reinforcement stress-strain law	= No hardening (Perfectly plastic)	▼
External diameter of cross-section	$D_{ext} = 0.6$	m
Internal diameter (for hollow tube sections)	$D_{int} = 0$	m

Initial reinforcement: When the required reinforcement is requested the initial reinforcement will be modified accordingly.

Number of bars of external reinforcement layer 1	$n_{bars,1} = 10$	
Diameter of bars of external reinforcement layer 1	$\phi_1 = 20$	mm
Distance from bar centroid of external	$c'_1 = 0.06$	m

concrete edge

Number of bars of internal reinforcement layer 2

$$n_{bars,2} = 0$$

Diameter of bars of internal reinforcement layer 2

$$\phi_2 = 20 \quad \text{mm}$$

Distance from bar centroid of internal reinforcement layer 2 to external concrete edge

$$c'_2 = 0.175 \quad \text{m}$$

Type of analysis

= Required reinforcement A_s ∇

Design value of the bending moment (positive when tension at bottom side)

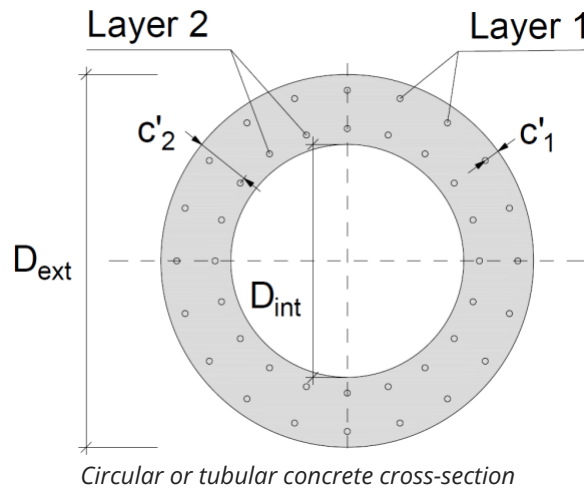
$$M_{Ed} = 200 \quad \text{kNm}$$

Design value of the axial force (compression negative)

$$N_{Ed} = -25.5 \quad \text{kN}$$

Reinforcement layers to be modified

= Layers 1 & 2 ∇



Nationally Defined Parameters

Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending)

$$\alpha_{cc} = 1$$

Concrete partial material safety factor

$$\gamma_c = 1.5$$

Reinforcement steel partial material safety factor

$$\gamma_s = 1.15$$

Design value of reinforcement ultimate strain

$$\epsilon_{ud} = 0.9\epsilon_{uk} \quad \nabla$$

Results

Maximum design bending moment resistance for given design axial force

$$M_{Rd} = 200.00 \text{ kNm}$$

Reinforcement - Layer 1 (external)

$$A_{s,1} = 24.33 \text{ cm}^2$$

(initial $\times 0.775$)

Reinforcement - Layer 2 (internal)

$$A_{s,2} = 0.00 \text{ cm}^2$$

Reinforcement - Total

$$A_{s,tot} = 24.33 \text{ cm}^2 \text{ (0.86\%)}$$

Check of total axial force convergence

$$|\Delta N| = 0.00 \text{ N} \leq 1 \text{ N} \Rightarrow \text{Ok}$$

Check of bending moment convergence

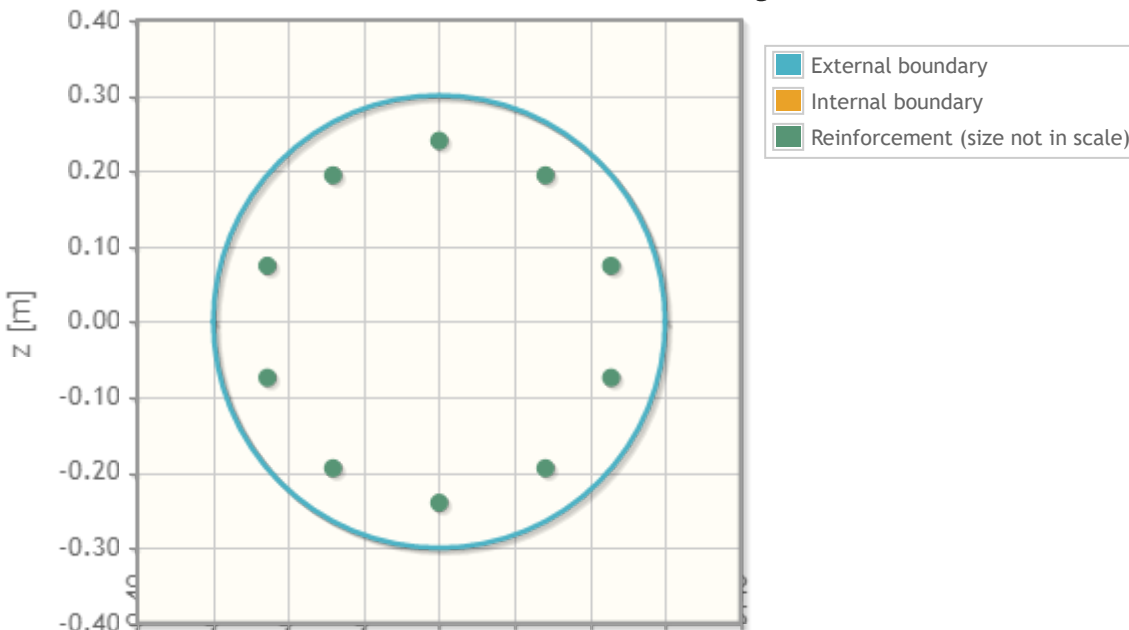
Ok

Notes

1. Positive bending moment produces tension at the bottom side. Negative bending moment produces tension at the top side.
2. In this calculation the area of the reinforcement bars is subtracted from the surrounding concrete area in order to avoid considering the same area for both steel and concrete. This correction is not significant in general, except for small highly-reinforced cross-sections.
3. The indicated reinforcement layers have been modified by multiplication by the same common factor that is indicated above. The layers in tension are generally more effective. When large compressive axial force is applied increasing the layers in compression may be beneficial.
4. According to EN1992-1-1 § 6.1(4) when the axial force N_{Ed} is compressive the absolute value of the design bending moment should be at least $|M_{Ed}| \geq e_0 \times |N_{Ed}|$ where e_0 is the minimum assumed eccentricity of the compressible load which is equal to $e_0 = \max(h/30, 20 \text{ mm})$, where h is the total depth of the cross-section.

Charts

Cross-section (hover=value, drag=zoom, double-click=reset)



Tables

Copy to Clipboard

Interaction diagram - Axial force resistance N_{Rd} vs bending moment resistance M_{Rd}

#	N-M interaction		Lever arm & neutral axis				
	Design axial force resistance N_{Rd} [kN]	Design bending moment resistance M_{Rd} [kNm]	Location of neutral axis from centroid y_n [m]	Lever arm of internal forces z [m]	Normalized level arm z/d [-]	Depth of compressive zone x [m]	Normalized depth of compressive zone x/d [-]
1	-25.50	200.00	0.181	0.327	0.606	0.119	0.221

Details

Input Data

- Concrete characteristic strength: $f_{ck} = 25$ MPa
- Steel characteristic yield strength: $f_{yk} = 400$ MPa
- Type of reinforcement stress-strain law: = No hardening (Perfectly plastic)
- Ratio of characteristic ultimate strength to characteristic yield strength: $(f_t/f_y)_k = 1$
- Characteristic value of ultimate strain: $\epsilon_{uk} = 75$ ‰
- External diameter of cross-section: $D_{ext} = 0.6$ m
- Internal diameter (for hollow tube sections): $D_{int} = 0$ m
- Number of bars of external reinforcement layer 1: $n_{bars,1} = 10$
- Diameter of bars of external reinforcement layer 1: $\Phi_1 = 20$ mm
- Distance from bar centroid of external reinforcement layer 1 to external concrete edge: $c'_1 = 0.06$ m
- Number of bars of internal reinforcement layer 2: $n_{bars,2} = 0$
- Diameter of bars of internal reinforcement layer 2: $\Phi_2 = 20$ mm
- Distance from bar centroid of internal reinforcement layer 2 to external concrete edge: $c'_2 = 0.175$ m
- Type of analysis: = Required reinforcement As
- Design value of the bending moment (positive when tension at bottom side): $M_{Ed} = 200$ kNm
- Design value of the axial force (compression negative): $N_{Ed} = -25.5$ kN
- Reinforcement layers to be modified: = Layers 1 & 2

Nationally Defined Parameters

- Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending): $\alpha_{cc} = 1$
- Concrete partial material safety factor: $\gamma_c = 1.5$
- Reinforcement steel partial material safety factor: $\gamma_s = 1.15$
- Design value of reinforcement ultimate strain: $\epsilon_{ud} = 0.9\epsilon_{uk}$

11.5 Appendix C.5: FEA- Steel Area (4m embedment depth)



Project: 204

Subject:

Designer:

Date:

Eurocode 2

ULS design of circular (or tubular) reinforced concrete cross-section for bending and axial force

Description:

ULS design of circular or tubular reinforced concrete cross-section for bending and axial force (calculation of required reinforcement for given bending moment / axial force, maximum moment resistance MRd for given axial force, or complete M-N interaction diagram for given reinforcement layout)

According to:

EN 1992-1-1:2004+AC2:2010 Section 6.1

Supported National Annexes:

Nationally Defined Parameters (NDPs) automatically filled for supported countries

Input

Concrete characteristic strength $f_{ck} = 25$ MPa

Steel characteristic yield strength $f_{yk} = 400$ MPa

Type of reinforcement stress-strain law = No hardening (Perfectly plastic) ▼

External diameter of cross-section $D_{ext} = 0.6$ m

Internal diameter (for hollow tube sections) $D_{int} = 0$ m

Initial reinforcement: When the required reinforcement is requested the initial reinforcement will be modified accordingly.

Number of bars of external reinforcement layer 1 $n_{bars,1} = 10$

Diameter of bars of external reinforcement layer 1 $\phi_1 = 20$ mm

Distance from bar centroid of external $c'_1 = 0.06$ m

concrete edge

Number of bars of internal reinforcement layer 2

$$n_{bars,2} = 0$$

Diameter of bars of internal reinforcement layer 2

$$\phi_2 = 20 \quad \text{mm}$$

Distance from bar centroid of internal reinforcement layer 2 to external concrete edge

$$c'_2 = 0.175 \quad \text{m}$$

Type of analysis

$$= \text{Required reinforcement } A_s \quad \checkmark$$

Design value of the bending moment (positive when tension at bottom side)

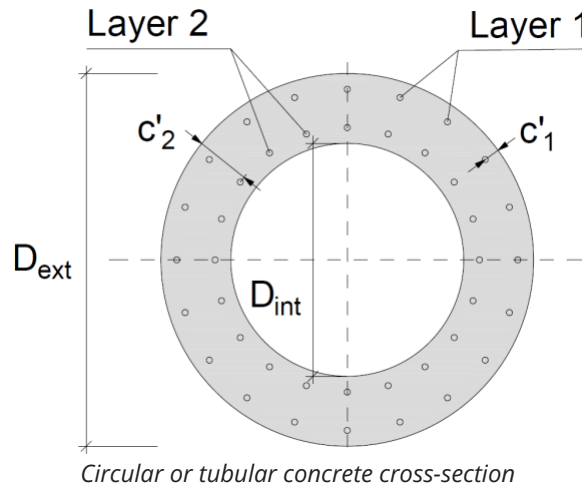
$$M_{Ed} = 204 \quad \text{kNm}$$

Design value of the axial force (compression negative)

$$N_{Ed} = -25.5 \quad \text{kN}$$

Reinforcement layers to be modified

$$= \text{Layers 1 \& 2} \quad \checkmark$$



Nationally Defined Parameters

Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending)

$$\alpha_{CC} = 1$$

Concrete partial material safety factor

$$\gamma_C = 1.5$$

Reinforcement steel partial material safety factor

$$\gamma_S = 1.15$$

Design value of reinforcement ultimate strain

$$\epsilon_{Ud} = 0.9\epsilon_{uk} \quad \checkmark$$

Results

Maximum design bending moment resistance for given design axial force

Reinforcement - Layer 1 (external)

Reinforcement - Layer 2 (internal)

Reinforcement - Total

Check of total axial force convergence

Check of bending moment convergence

$$M_{Rd} = 204.00 \text{ kNm}$$

$$A_{s,1} = 24.90 \text{ cm}^2$$

(initial $\times 0.792$)

$$A_{s,2} = 0.00 \text{ cm}^2$$

(initial $\times 0.792$)

$$A_{s,tot} = 24.90 \text{ cm}^2 \text{ (0.88\%)}$$

$$|\Delta N| = 0.00 \text{ N} \leq 1 \text{ N} \Rightarrow \text{Ok} \quad |\Delta M| = 0.00 \text{ Nm} \leq 1 \text{ Nm} \Rightarrow$$

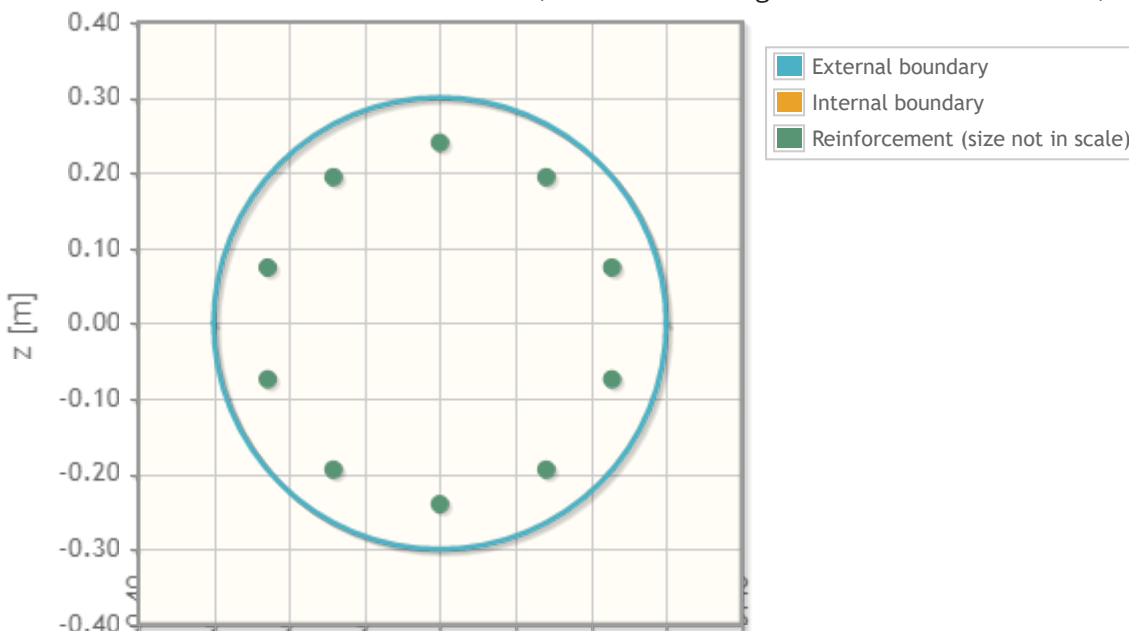
Ok

Notes

1. Positive bending moment produces tension at the bottom side. Negative bending moment produces tension at the top side.
2. In this calculation the area of the reinforcement bars is subtracted from the surrounding concrete area in order to avoid considering the same area for both steel and concrete. This correction is not significant in general, except for small highly-reinforced cross-sections.
3. The indicated reinforcement layers have been modified by multiplication by the same common factor that is indicated above. The layers in tension are generally more effective. When large compressive axial force is applied increasing the layers in compression may be beneficial.
4. According to EN1992-1-1 § 6.1(4) when the axial force N_{Ed} is compressive the absolute value of the design bending moment should be at least $|M_{Ed}| \geq e_0 \times |N_{Ed}|$ where e_0 is the minimum assumed eccentricity of the compressive load which is equal to $e_0 = \max(h/30, 20 \text{ mm})$, where h is the total depth of the cross-section.

Charts

Cross-section (hover=value, drag=zoom, double-click=reset)



Tables

Copy to Clipboard

Interaction diagram - Axial force resistance N_{Rd} vs bending moment resistance M_{Rd}

N-M interaction		Lever arm & neutral axis					
#	Design axial force resistance N_{Rd} [kN]	Design bending moment resistance M_{Rd} [kNm]	Location of neutral axis from centroid y_n [m]	Lever arm of internal forces z [m]	Normalized level arm z/d [-]	Depth of compressive zone x [m]	Normalized depth of compressive zone x/d [-]
1	-25.50	204.00	0.179	0.327	0.605	0.121	0.223

Details

Input Data

- Concrete characteristic strength: $f_{ck} = 25$ MPa
- Steel characteristic yield strength: $f_{yk} = 400$ MPa
- Type of reinforcement stress-strain law: = No hardening (Perfectly plastic)
- Ratio of characteristic ultimate strength to characteristic yield strength: $(f_t/f_y)_k = 1$
- Characteristic value of ultimate strain: $\epsilon_{uk} = 75$ ‰
- External diameter of cross-section: $D_{ext} = 0.6$ m
- Internal diameter (for hollow tube sections): $D_{int} = 0$ m
- Number of bars of external reinforcement layer 1: $n_{bars,1} = 10$
- Diameter of bars of external reinforcement layer 1: $\Phi_1 = 20$ mm
- Distance from bar centroid of external reinforcement layer 1 to external concrete edge: $c'_1 = 0.06$ m
- Number of bars of internal reinforcement layer 2: $n_{bars,2} = 0$
- Diameter of bars of internal reinforcement layer 2: $\Phi_2 = 20$ mm
- Distance from bar centroid of internal reinforcement layer 2 to external concrete edge: $c'_2 = 0.175$ m
- Type of analysis: = Required reinforcement As
- Design value of the bending moment (positive when tension at bottom side): $M_{Ed} = 204$ kNm
- Design value of the axial force (compression negative): $N_{Ed} = -25.5$ kN
- Reinforcement layers to be modified: = Layers 1 & 2

Nationally Defined Parameters

- Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending): $\alpha_{cc} = 1$
- Concrete partial material safety factor: $\gamma_c = 1.5$
- Reinforcement steel partial material safety factor: $\gamma_s = 1.15$
- Design value of reinforcement ultimate strain: $\epsilon_{ud} = 0.9\epsilon_{uk}$

11.6 Appendix C.6: FEA- Steel Area (3m embedment depth)



Project: 221

Subject:

Designer:

Date:

Eurocode 2

ULS design of circular (or tubular) reinforced concrete cross-section for bending and axial force

Description:

ULS design of circular or tubular reinforced concrete cross-section for bending and axial force (calculation of required reinforcement for given bending moment / axial force, maximum moment resistance MRd for given axial force, or complete M-N interaction diagram for given reinforcement layout)

According to:

EN 1992-1-1:2004+AC2:2010 Section 6.1

Supported National

Annexes:

Nationally Defined Parameters (NDPs) automatically filled for supported countries

Input

Concrete characteristic strength $f_{ck} = 25$ MPa

Steel characteristic yield strength $f_{yk} = 400$ MPa

Type of reinforcement stress-strain law = No hardening (Perfectly plastic) ▼

External diameter of cross-section $D_{ext} = 0.6$ m

Internal diameter (for hollow tube sections) $D_{int} = 0$ m

Initial reinforcement: When the required reinforcement is requested the initial reinforcement will be modified accordingly.

Number of bars of external reinforcement layer 1 $n_{bars,1} = 10$

Diameter of bars of external reinforcement layer 1 $\phi_1 = 20$ mm

Distance from bar centroid of external $c'_1 = 0.06$ m

concrete edge

Number of bars of internal reinforcement layer 2

$$n_{\text{bars},2} = 0$$

Diameter of bars of internal reinforcement layer 2

$$\phi_2 = 20 \quad \text{mm}$$

Distance from bar centroid of internal reinforcement layer 2 to external concrete edge

$$c'_2 = 0.175 \quad \text{m}$$

Type of analysis

$$= \text{Required reinforcement } A_s \quad \checkmark$$

Design value of the bending moment (positive when tension at bottom side)

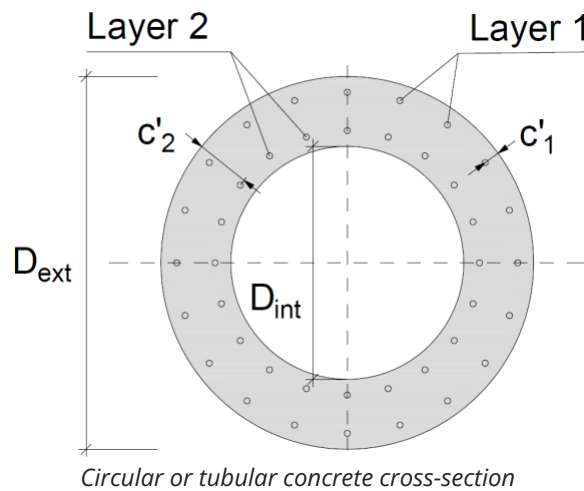
$$M_{Ed} = 221 \quad \text{kNm}$$

Design value of the axial force (compression negative)

$$N_{Ed} = -25.5 \quad \text{kN}$$

Reinforcement layers to be modified

$$= \text{Layers 1 \& 2} \quad \checkmark$$



Nationally Defined Parameters

Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending)

$$\alpha_{cc} = 1$$

Concrete partial material safety factor

$$\gamma_c = 1.5$$

Reinforcement steel partial material safety factor

$$\gamma_s = 1.15$$

Design value of reinforcement ultimate strain

$$\epsilon_{ud} = 0.9\epsilon_{uk} \quad \checkmark$$

Results

Maximum design bending moment resistance for given design axial force

$$M_{Rd} = 221.00 \text{ kNm}$$

Reinforcement - Layer 1 (external)

$$A_{s,1} = 27.29 \text{ cm}^2$$

Reinforcement - Layer 2 (internal)

$$A_{s,2} = 0.00 \text{ cm}^2 \quad (\text{initial} \times 0.869)$$

Reinforcement - Total

$$A_{s,tot} = 27.29 \text{ cm}^2 \quad (\text{initial} \times 0.869) \quad (0.97\%)$$

Check of total axial force convergence

$$|\Delta N| = 0.00 \text{ N} \leq 1 \text{ N} \Rightarrow \text{Ok} \quad |\Delta M| = 0.00 \text{ Nm} \leq 1 \text{ Nm} \Rightarrow$$

Check of bending moment convergence

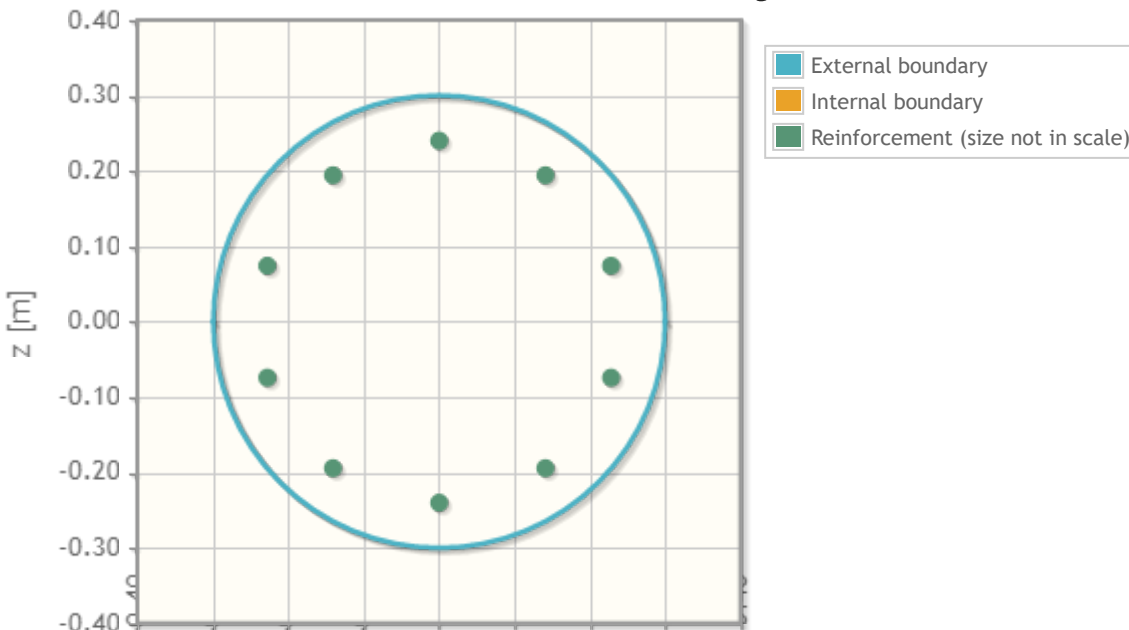
Ok

Notes

1. Positive bending moment produces tension at the bottom side. Negative bending moment produces tension at the top side.
2. In this calculation the area of the reinforcement bars is subtracted from the surrounding concrete area in order to avoid considering the same area for both steel and concrete. This correction is not significant in general, except for small highly-reinforced cross-sections.
3. The indicated reinforcement layers have been modified by multiplication by the same common factor that is indicated above. The layers in tension are generally more effective. When large compressive axial force is applied increasing the layers in compression may be beneficial.
4. According to EN1992-1-1 § 6.1(4) when the axial force N_{Ed} is compressive the absolute value of the design bending moment should be at least $|M_{Ed}| \geq e_0 \times |N_{Ed}|$ where e_0 is the minimum assumed eccentricity of the compressible load which is equal to $e_0 = \max(h/30, 20 \text{ mm})$, where h is the total depth of the cross-section.

Charts

Cross-section (hover=value, drag=zoom, double-click=reset)



Tables

Copy to Clipboard

Interaction diagram - Axial force resistance N_{Rd} vs bending moment resistance M_{Rd}

N-M interaction		Lever arm & neutral axis					
#	Design axial force resistance N_{Rd} [kN]	Design bending moment resistance M_{Rd} [kNm]	Location of neutral axis from centroid y_n [m]	Lever arm of internal forces z [m]	Normalized level arm z/d [-]	Depth of compressive zone x [m]	Normalized depth of compressive zone x/d [-]
1	-25.50	221.00	0.174	0.324	0.599	0.126	0.233

Details

Input Data

- Concrete characteristic strength: $f_{ck} = 25$ MPa
- Steel characteristic yield strength: $f_{yk} = 400$ MPa
- Type of reinforcement stress-strain law: = No hardening (Perfectly plastic)
- Ratio of characteristic ultimate strength to characteristic yield strength: $(f_t/f_y)_k = 1$
- Characteristic value of ultimate strain: $\epsilon_{uk} = 75$ ‰
- External diameter of cross-section: $D_{ext} = 0.6$ m
- Internal diameter (for hollow tube sections): $D_{int} = 0$ m
- Number of bars of external reinforcement layer 1: $n_{bars,1} = 10$
- Diameter of bars of external reinforcement layer 1: $\Phi_1 = 20$ mm
- Distance from bar centroid of external reinforcement layer 1 to external concrete edge: $c'_1 = 0.06$ m
- Number of bars of internal reinforcement layer 2: $n_{bars,2} = 0$
- Diameter of bars of internal reinforcement layer 2: $\Phi_2 = 20$ mm
- Distance from bar centroid of internal reinforcement layer 2 to external concrete edge: $c'_2 = 0.175$ m
- Type of analysis: = Required reinforcement As
- Design value of the bending moment (positive when tension at bottom side): $M_{Ed} = 221$ kNm
- Design value of the axial force (compression negative): $N_{Ed} = -25.5$ kN
- Reinforcement layers to be modified: = Layers 1 & 2

Nationally Defined Parameters

- Coefficient taking account of long term effects and loading effects on the compressive strength of concrete (for bending): $\alpha_{cc} = 1$
- Concrete partial material safety factor: $\gamma_c = 1.5$
- Reinforcement steel partial material safety factor: $\gamma_s = 1.15$
- Design value of reinforcement ultimate strain: $\epsilon_{ud} = 0.9\epsilon_{uk}$

12 Appendix D: Detailed Bill of Quantities of Piling and Shotcrete Work

12.1 Appendix D.1: DeepEx- Bill of Quantity

Project: Assem PLC 4B+G+6 podium and 15 Floors Building Shoring Work						
Piling and Shotcrete work Summary						
	TOTAL A	birr	30,314,191.12		Cost including per square meters of shored area including Vat	
	VAT 15%	Birr	4,547,128.67			14,821.99
	TOTAL Including Vat	Birr	34,861,319.78			birr/m ²
ASSUMPTIONS						
	Total length of proposed shored area		196	Meters		
	Maximum Shoring Wall Depth		12	Meters		
	Maximum Pile Depth		17	Meters		
	Diameter of Piles		600	mm		
	Number of Piles		109	pcs		
	Maximum pile spacing		1.8	Meters		
	Number of Anchor (1 st row)		109	Pcs		
	Number of Anchor (2 nd row)		55	Pcs		
	Free Anchor length (1 st row)		10	Meters		
	Free Anchor length (2 nd row)		7	Meters		
	Root Anchor length (1 st row)		6	Meters		
	Root Anchor length (2 nd row)		5	Meters		
	Anchor spacing (1 st row)		1.8	Meters		
	Anchor spacing (2 nd row)		3.6	Meters		
	Top Pile Beam of 600x500 mm of total length		196	Meters		
Section 1 : Bill of Quantities						
All prices are expressed in ETB (Ethiopian Birr) and are net prices which are subject to 15% VAT						
Item No.	Description	Unit	Quantity	Unit Price	Total Price	
1	Mobilization and demobilization of staff and equipment for piling foundation, anchoring, shotcreting and complete shoring system design.	Ls	1.00	60,000.00	60,000.00	
2	Installation of Shoring Pile with an average depth of 17000 mm for Dia 600 mm Pile including supply of raw materials. Bored cast in-situ piles to design depth and Reinforced shotcrete between bored piles and socketed in rock layer.					
2.1	Drilling of piles diameter of 600 mm piles in soil formations	m	1,308.00	2,000.00	2,616,000.00	
2.2	Drilling of 600 mm diameter piles in rock formation	m	545.00	3,250.00	1,771,250.00	
2.3	a) Fabricate pile reinforcement with steel grade of 60($f_{yk}=400MPa$) according to the structural detail drawing for bored cast in situ pile diameter of 600 mm					
	i. dia 20mm rebars	kg	63,976.87	120.00	7,677,224.32	
	ii. dia 10mm rebars	kg	14,356.38	120.00	1,722,765.11	
	b) Install Reinforcement for Top Pile Beam					
	i. dia 16mm rebars	kg	2,969.80	120.00	356,375.68	
	ii. dia 10mm rebars	kg	1,063.40	120.00	127,608.48	
	c) for shotcrete wall					
	i. dia 10mm rebars	kg	207.16	120.00	24,858.79	
2.4	a) Construction of shoring pile (diameter 600 mm) made of C-25 reinforced concrete with minimum cement content of 360kg per m ³ .	m ³	523.92	5,500.00	2,881,578.74	
	b) Installation of concrete in Tie Beam for Piles including trench excavation	m ³	58.80	5,800.00	341,040.00	
2.5	Provide, cut and fix in position sawn structural wood or steel formwork for Tie Beam	m ²	196.00	550.00	107,800.00	
2.7	Inter borehole movement	No	109.00	250.00	27,250.00	
2.8	a) Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 15cm.	m ²	2,352.00	1,320.00	3,104,640.00	
2.9	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 16 meters with an angle of 30° and spaced each with 1800mm @ 3.0 meter below NGL and between piles.	m	1,744.00	3,950.00	6,888,800.00	
2.10	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 12 meters with an angle of 30° and spaced each with 3600mm @ 7.0 meter below NGL and between piles.	m	660.00	3,950.00	2,607,000.00	
			TOTAL		30,314,191.12	
			VAT (15%)		4,547,128.67	
			GRAND TOTAL		34,861,319.78	

12.2 Appendix D.2: GEO5- Bill of Quantity

Project: Assem PLC 4B+G+6 podium and 15 Floors Building Shoring Work						
Piling and Shotcrete work Summary						
	TOTAL A	birr	33,056,056.95		Cost including per square meters of shored area including Vat	
	VAT 15%	Birr	4,958,408.54			16,162.61
	TOTAL Including Vat	Birr	38,014,465.49			birr/m ²
ASSUMPTIONS						
	Total length of proposed shored area		196	Meters		
	Maximum Shoring Wall Depth		12	Meters		
	Maximum Pile Depth		17	Meters		
	Diameter of Piles		600	mm		
	Number of Piles		109	pcs		
	Maximum pile spacing		1.8	Meters		
	Number of Anchor (1 st row)		109	Pcs		
	Number of Anchor (2 nd row)		55	Pcs		
	Free Anchor length (1 st row)		10	Meters		
	Free Anchor length (2 nd row)		7	Meters		
	Root Anchor length (1 st row)		6	Meters		
	Root Anchor length (2 nd row)		5	Meters		
	Anchor spacing (1 st row)		1.8	Meters		
	Anchor spacing (2 nd row)		3.6	Meters		
	Top Pile Beam of 600x500 mm of total length		196	Meters		
Section 1 : Bill of Quantities						
All prices are expressed in ETB (Ethiopian Birr) and are net prices which are subject to 15% VAT						
Item No.	Description	Unit	Quantity	Unit Price	Total Price	
1	Mobilization and demobilization of staff and equipment for piling foundation, anchoring, shotcreting and complete shoring system design.	Ls	1.00	60,000.00	60,000.00	
2	Installation of Shoring Pile with an average depth of 17000 mm for Dia 600 mm Pile including supply of raw materials. Bored cast in-situ piles to design depth and Reinforced shotcrete between bored piles and scoketed in rock layer.					
2.1	Drilling of piles diameter of 600 mm piles in soil formations	m	1,308.00	2,000.00	2,616,000.00	
2.2	Drilling of 600 mm diameter piles in rock formation	m	545.00	3,250.00	1,771,250.00	
2.3	a) Fabricate pile reinforcement with steel grade of 60($f_{yk}=400MPa$) according to the structural detail drawing for bored cast in situ pile diameter of 600 mm					
	i. dia 20mm rebars	kg	86,825.75	120.00	10,419,090.15	
	ii. dia 10mm rebars	kg	14,356.38	120.00	1,722,765.11	
	b) Install Reinforcement for Top Pile Beam					
	i. dia 16mm rebars	kg	2,969.80	120.00	356,375.68	
	ii. dia 10mm rebars	kg	1,063.40	120.00	127,608.48	
	c) for shotcrete wall					
	i. dia 10mm rebars	kg	207.16	120.00	24,858.79	
2.4	a) Construction of shoring pile (diameter 600 mmm) made of C-25 reinforced concrete with minimum cement content of 360kg per m ³ .	m ³	523.92	5,500.00	2,881,578.74	
	b) Installation of concrete in Tie Beam for Piles including trench excavation	m ³	58.80	5,800.00	341,040.00	
2.5	Provide, cut and fix in position sawn structural wood or steel formwork for Tie Beam	m ²	196.00	550.00	107,800.00	
2.7	Inter borehole movement	No	109.00	250.00	27,250.00	
2.8	a) Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 15cm.	m ²	2,352.00	1,320.00	3,104,640.00	
2.9	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 16 meters with an angle of 30° and spaced each with 1800mm @ 3.0 meter below NGL and between piles.	m	1,744.00	3,950.00	6,888,800.00	
2.10	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 12 meters with an angle of 30° and spaced each with 3600mm @ 7.0 meter below NGL and between piles.	m	660.00	3,950.00	2,607,000.00	
				TOTAL	33,056,056.95	
				VAT (15%)	4,958,408.54	
				GRAND TOTAL	38,014,465.49	

**12.3 Appendix D.3: DeepEx (Beam on Elastic Foundation) - Bill of
Quantity**

Project: Assem PLC 4B+G+6 podium and 15 Floors Building Shoring Work						
Piling and Shotcrete work Summary						
	TOTAL A	birr	29,765,817.95		Cost including per square meters of shored area including Vat	
	VAT 15%	Birr	4,464,872.69			14,553.87
	TOTAL Including Vat	Birr	34,230,690.64			birr/m ²
ASSUMPTIONS						
	Total length of proposed shored area		196	Meters		
	Maximum Shoring Wall Depth		12	Meters		
	Maximum Pile Depth		17	Meters		
	Diameter of Piles		600	mm		
	Number of Piles		109	pcs		
	Maximum pile spacing		1.8	Meters		
	Number of Anchor (1 st row)		109	Pcs		
	Number of Anchor (2 nd row)		55	Pcs		
	Free Anchor length (1 st row)		10	Meters		
	Free Anchor length (2 nd row)		7	Meters		
	Root Anchor length (1 st row)		6	Meters		
	Root Anchor length (2 nd row)		5	Meters		
	Anchor spacing (1 st row)		1.8	Meters		
	Anchor spacing (2 nd row)		3.6	Meters		
	Top Pile Beam of 600x500 mm of total length		196	Meters		
Section 1 : Bill of Quantities						
All prices are expressed in ETB (Ethiopian Birr) and are net prices which are subject to 15% VAT						
Item No.	Description	Unit	Quantity	Unit Price	Total Price	
1	Mobilization and demobilization of staff and equipment for piling foundation, anchoring,shotcreting and compete shoring system design.	Ls	1.00	60,000.00	60,000.00	
2	Installation of Shoring Pile with an average depth of 17000 mm for Dia 600 mm Pile including supply of raw materials.Bored cast in-situ piles to design depth and Reinforced shotcrete between bored piles and scoketed in rock layer.					
2.1	Drilling of piles diameter of 600 mm piles in soil formations	m	1,308.00	2,000.00	2,616,000.00	
2.2	Drilling of 600 mm diameter piles in rock formation	m	545.00	3,250.00	1,771,250.00	
2.3	a) Fabricate pile reinforcement with steel grade of 60(f_{yk}=400MPa) according to the structural detail drawing for bored cast in situ pile diameter of 600 mm					
	i. dia 20mm rebars	kg	59,407.09	120.00	7,128,851.16	
	ii. dia 10mm rebars	kg	14,356.38	120.00	1,722,765.11	
	b) Install Reinforcement for Top Pile Beam					
	i. dia 16mm rebars	kg	2,969.80	120.00	356,375.68	
	ii. dia 10mm rebars	kg	1,063.40	120.00	127,608.48	
	c) for shotcrete wall					
	i. dia 10mm rebars	kg	207.16	120.00	24,858.79	
2.4	a)Construction of shoring pile (diameter 600 mmm) made of C-25 reinforced concrete with minimum cement content of 360kg per m ³ .	m ³	523.92	5,500.00	2,881,578.74	
	b) Installation of concrete in Tie Beam for Piles including trench excavation	m ³	58.80	5,800.00	341,040.00	
2.5	Provide, cut and fix in position sawn structural wood or steel formwork for Tie Beam	m ²	196.00	550.00	107,800.00	
2.7	Inter borehole movement	No	109.00	250.00	27,250.00	
2.8	a)Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 15cm.	m ²	2,352.00	1,320.00	3,104,640.00	
2.9	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 16 meters with an angle of 30° and spaced each with 1800mm @ 3.0 meter below NGL and between piles.	m	1,744.00	3,950.00	6,888,800.00	
2.10	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 12 meters with an angle of 30° and spaced each with 3600mm @ 7.0 meter below NGL and between piles.	m	660.00	3,950.00	2,607,000.00	
			TOTAL		29,765,817.95	
			VAT (15%)		4,464,872.69	
			GRAND TOTAL		34,230,690.64	

12.4 Appendix D.4: FEA- Bill of Quantity

Project: Assem PLC 4B+G+6 podium and 15 Floors Building Shoring Work						
Piling and Shotcrete work Summary						
	TOTAL A	birr	27,023,952.12		Cost including per square meters of shored area including Vat	
	VAT 15%	Birr	4,053,592.82			13,213.24
	TOTAL Including Vat	Birr	31,077,544.94			birr/m ²
ASSUMPTIONS						
	Total length of proposed shored area		196	Meters		
	Maximum Shoring Wall Depth		12	Meters		
	Maximum Pile Depth		17	Meters		
	Diameter of Piles		600	mm		
	Number of Piles		109	pcs		
	Maximum pile spacing		1.8	Meters		
	Number of Anchor (1 st row)		109	Pcs		
	Number of Anchor (2 nd row)		55	Pcs		
	Free Anchor length (1 st row)		10	Meters		
	Free Anchor length (2 nd row)		7	Meters		
	Root Anchor length (1 st row)		6	Meters		
	Root Anchor length (2 nd row)		5	Meters		
	Anchor spacing (1 st row)		1.8	Meters		
	Anchor spacing (2 nd row)		3.6	Meters		
	Top Pile Beam of 600x500 mm of total length		196	Meters		
Section 1 : Bill of Quantities						
All prices are expressed in ETB (Ethiopian Birr) and are net prices which are subject to 15% VAT						
Item No.	Description	Unit	Quantity	Unit Price	Total Price	
1	Mobilization and demobilization of staff and equipment for piling foundation, anchoring,shotcreting and compete shoring system design.	Ls	1.00	60,000.00	60,000.00	
2	Installation of Shoring Pile with an average depth of 17000 mm for Dia 600 mm Pile including supply of raw materials.Bored cast in-situ piles to design depth and Reinforced shotcrete between bored piles and scoketed in rock layer.					
2.1	Drilling of piles diameter of 600 mm piles in soil formations	m	1,308.00	2,000.00	2,616,000.00	
2.2	Drilling of 600 mm diameter piles in rock formation	m	545.00	3,250.00	1,771,250.00	
2.3	a) Fabricate pile reinforcement with steel grade of 60(f_{yk}=400MPa) according to the structural detail drawing for bored cast in situ pile diameter of 600 mm					
	i. dia 20mm rebars	kg	36,558.21	120.00	4,386,985.33	
	ii. dia 10mm rebars	kg	14,356.38	120.00	1,722,765.11	
	b) Install Reinforcement for Top Pile Beam					
	i. dia 16mm rebars	kg	2,969.80	120.00	356,375.68	
	ii. dia 10mm rebars	kg	1,063.40	120.00	127,608.48	
	c) for shotcrete wall					
	i. dia 10mm rebars	kg	207.16	120.00	24,858.79	
2.4	a)Construction of shoring pile (diameter 600 mmm) made of C-25 reinforced concrete with minimum cement content of 360kg per m ³ .	m ³	523.92	5,500.00	2,881,578.74	
	b) Installation of concrete in Tie Beam for Piles including trench excavation	m ³	58.80	5,800.00	341,040.00	
2.5	Provide, cut and fix in position sawn structural wood or steel formwork for Tie Beam	m ²	196.00	550.00	107,800.00	
2.7	Inter borehole movement	No	109.00	250.00	27,250.00	
2.8	a)Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 15cm.	m ²	2,352.00	1,320.00	3,104,640.00	
2.9	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 16 meters with an angle of 30° and spaced each with 1800mm @ 3.0 meter below NGL and between piles.	m	1,744.00	3,950.00	6,888,800.00	
2.10	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 12 meters with an angle of 30° and spaced each with 3600mm @ 7.0 meter below NGL and between piles.	m	660.00	3,950.00	2,607,000.00	
			TOTAL		27,023,952.12	
			VAT (15%)		4,053,592.82	
			GRAND TOTAL		31,077,544.94	

12.5 Appendix D.5: FEA - Bill of Quantity (4m embedment depth)

Project: Assem PLC 4B+G+6 podium and 15 Floors Building Shoring Work						
Piling and Shotcrete work Summary						
	TOTAL A	birr	26,140,800.41		Cost including per square meters of shored area including Vat	
	VAT 15%	Birr	3,921,120.06			12,781.43
	TOTAL Including Vat	Birr	30,061,920.47			birr/m ²
ASSUMPTIONS						
	Total length of proposed shored area		196	Meters		
	Maximum Shoring Wall Depth		12	Meters		
	Maximum Pile Depth		16	Meters		
	Diameter of Piles		600	mm		
	Number of Piles		109	pcs		
	Maximum pile spacing		1.8	Meters		
	Number of Anchor (1 st row)		109	Pcs		
	Number of Anchor (2 nd row)		55	Pcs		
	Free Anchor length (1 st row)		10	Meters		
	Free Anchor length (2 nd row)		7	Meters		
	Root Anchor length (1 st row)		6	Meters		
	Root Anchor length (2 nd row)		5	Meters		
	Anchor spacing (1 st row)		1.8	Meters		
	Anchor spacing (2 nd row)		3.6	Meters		
	Top Pile Beam of 600x500 mm of total length		196	Meters		
Section 1 : Bill of Quantities						
All prices are expressed in ETB (Ethiopian Birr) and are net prices which are subject to 15% VAT						
Item No.	Description	Unit	Quantity	Unit Price	Total Price	
1	Mobilization and demobilization of staff and equipment for piling foundation, anchoring,shotcreting and compete shoring system design.	Ls	1.00	60,000.00	60,000.00	
2	Installation of Shoring Pile with an average depth of 16000 mm for Dia 600 mm Pile including supply of raw materials.Bored cast in-situ piles to design depth and Reinforced shotcrete between bored piles and scoketed in rock layer.					
2.1	Drilling of piles diameter of 600 mm piles in soil formations	m	1,308.00	2,000.00	2,616,000.00	
2.2	Drilling of 600 mm diameter piles in rock formation	m	436.00	3,250.00	1,417,000.00	
2.3	a) Fabricate pile reinforcement with steel grade of 60(f_{yK}=400MPa) according to the structural detail drawing for bored cast in situ pile diameter of 600 mm					
	i. dia 20mm rebars	kg	34,407.73	120.00	4,128,927.37	
	ii. dia 10mm rebars	kg	13,511.88	120.00	1,621,425.99	
	b) Install Reinforcement for Top Pile Beam					
	i. dia 16mm rebars	kg	2,969.80	120.00	356,375.68	
	ii. dia 10mm rebars	kg	1,063.40	120.00	127,608.48	
	c) for shotcrete wall					
	i. dia 10mm rebars	kg	207.16	120.00	24,858.79	
2.4	a)Construction of shoring pile (diameter 600 mmm) made of C-25 reinforced concrete with minimum cement content of 360kg per m ³ .	m ³	493.10	5,500.00	2,712,074.11	
	b) Installation of concrete in Tie Beam for Piles including trench excavation	m ³	58.80	5,800.00	341,040.00	
2.5	Provide, cut and fix in position sawn structural wood or steel formwork for Tie Beam	m ²	196.00	550.00	107,800.00	
2.7	Inter borehole movement	No	109.00	250.00	27,250.00	
2.8	a)Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 15cm.	m ²	2,352.00	1,320.00	3,104,640.00	
2.9	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 16 meters with an angle of 30° and spaced each with 1800mm @ 3.0 meter below NGL and between piles.	m	1,744.00	3,950.00	6,888,800.00	
2.10	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 12 meters with an angle of 30° and spaced each with 3600mm @ 7.0 meter below NGL and between piles.	m	660.00	3,950.00	2,607,000.00	
				TOTAL	26,140,800.41	
				VAT (15%)	3,921,120.06	
				GRAND TOTAL	30,061,920.47	

12.6 Appendix D.6: FEA - Bill of Quantity (3m embedment depth)

Project: Assem PLC 4B+G+6 podium and 15 Floors Building Shoring Work					
Piling and Shotcrete work Summary					
	TOTAL A	birr	25,741,507.36		Cost including per square meters of shored area including Vat
	VAT 15%	Birr	3,861,226.10		12,586.20
	TOTAL Including Vat	Birr	29,602,733.47		birr/m ²
ASSUMPTIONS					
	Total length of proposed shored area		196	Meters	
	Maximum Shoring Wall Depth		12	Meters	
	Maximum Pile Depth		15	Meters	
	Diameter of Piles		600	mm	
	Number of Piles		109	pcs	
	Maximum pile spacing		1.8	Meters	
	Number of Anchor (1 st row)		109	Pcs	
	Number of Anchor (2 nd row)		55	Pcs	
	Free Anchor length (1 st row)		10	Meters	
	Free Anchor length (2 nd row)		7	Meters	
	Root Anchor length (1 st row)		6	Meters	
	Root Anchor length (2 nd row)		5	Meters	
	Anchor spacing (1 st row)		1.8	Meters	
	Anchor spacing (2 nd row)		3.6	Meters	
	Top Pile Beam of 600x500 mm of total length		196	Meters	
Section 1 : Bill of Quantities					
All prices are expressed in ETB (Ethiopian Birr) and are net prices which are subject to 15% VAT					
Item No.	Description	Unit	Quantity	Unit Price	Total Price
1	Mobilization and demobilization of staff and equipment for piling foundation, anchoring, shotcreting and complete shoring system design.	Ls	1.00	60,000.00	60,000.00
2	Installation of Shoring Pile with an average depth of 17000 mm for Dia 600 mm Pile including supply of raw materials. Bored cast in-situ piles to design depth and Reinforced shotcrete between bored piles and scoketed in rock layer.				
2.1	Drilling of piles diameter of 600 mm piles in soil formations	m	1,308.00	2,000.00	2,616,000.00
2.2	Drilling of 600 mm diameter piles in rock formation	m	327.00	3,250.00	1,062,750.00
2.3	a) Fabricate pile reinforcement with steel grade of 60($f_{yk}=400MPa$) according to the structural detail drawing for bored cast in situ pile diameter of 600 mm				
	i. dia 20mm rebars	kg	36,289.40	120.00	4,354,728.08
	ii. dia 10mm rebars	kg	12,667.39	120.00	1,520,086.86
	b) Install Reinforcement for Top Pile Beam				
	i. dia 16mm rebars	kg	2,969.80	120.00	356,375.68
	ii. dia 10mm rebars	kg	1,063.40	120.00	127,608.48
	c) for shotcrete wall				
	i. dia 10mm rebars	kg	207.16	120.00	24,858.79
2.4	a) Construction of shoring pile (diameter 600 mmm) made of C-25 reinforced concrete with minimum cement content of 360kg per m ³ .	m ³	462.29	5,500.00	2,542,569.47
	b) Installation of concrete in Tie Beam for Piles including trench excavation	m ³	58.80	5,800.00	341,040.00
2.5	Provide, cut and fix in position sawn structural wood or steel formwork for Tie Beam	m ²	196.00	550.00	107,800.00
2.7	Inter borehole movement	No	109.00	250.00	27,250.00
2.8	a) Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 15cm.	m ²	2,352.00	1,320.00	3,104,640.00
2.9	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 16 meters with an angle of 30° and spaced each with 1800mm @ 3.0 meter below NGL and between piles.	m	1,744.00	3,950.00	6,888,800.00
2.10	Supply, drill and install cement grouted anchors using OPC with a minimum of 3 strands of each dia 15.2mm which can carry a total load of 500 KN with anchoring length of 12 meters with an angle of 30° and spaced each with 3600mm @ 7.0 meter below NGL and between piles.	m	660.00	3,950.00	2,607,000.00
				TOTAL	25,741,507.36
				VAT (15%)	3,861,226.10
				GRAND TOTAL	29,602,733.47