ADDIS ABABA INSTITUTE OF TECHNOLOGY

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DEPARTMENT OF CIVIL ENGINEERING

COMPARATIVE ASSESSMENT OF EXCAVATION SUPPORTING SYSTEMS IN ADDIS ABABA

BY: KIDIST AFEWERK

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Comparative Assessment of Excavation Supporting Systems in Addis Ababa.

A thesis submitted to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the requirements of the Degree of

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A STUDY ON

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DECLARATION

I, the undersigned, declare that this thesis is my original work performed under the supervision of my research advisor Dr: Ing Henok Fikre, and has not been presented as a thesis for a degree in any other university. All sources of materials used for this thesis have also been duly acknowledged.

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ABSTRACT

Addis Ababa, Ethiopia's growing capital city, has witnessed rapid urban development, necessitating the construction of towering structures. As the need for deep excavation arises, Engineers face the challenge of maintaining stability while digging into the earth. This necessitates using temporary earth retaining structures known as excavation supports or shoring to prevent soil collapse and ensure precise excavation. Various methods are available for excavation support, including soldier beams and lagging, sheet piling, bored pile walls, soil nailing walls, and slurry (diaphragm walls).

In Addis Ababa, specialized firms dedicated to foundations construct excavation support systems. The first foundation specialist company in the country, BAUER MIDROC Foundation Specialist Plc., was established on June 12, 1998. Addis Ababa has varying soil types, but the protection system used is uniform, causing specific problems such as costly temporary shoring systems and time-consuming installation. Local companies compete by compromising safety, leading to the collapse of excavation support systems with major damage, including loss of human life, additionally, machinery has a short lifespan due to difficult strata placement.

The study aims to access different shoring methods in Addis Ababa and compare conventional contiguous pile walls with soil-nailing walls for various sites and soil conditions that are practical, reliable, appropriate, and adaptable solutions for local firms. To achieve this, first gather information about three different sites, including soil strength, excavation depths, surcharge load, and available space for the supporting system structure. Numerical methods such as finite element analysis and limit equilibrium analysis will be used to create models of the deep excavation stabilizing methods for each site and method. After that compare two methods of construction costs and construction period based on the obtained designs.

In conclusion, this research aims to provide valuable insights into the most effective and suitable excavation supporting systems for specific site conditions and soil types in Addis Ababa. The particular approach employed in this study, along with considering various factors, ensures that the findings are reliable and applicable to the context at hand.

The results of this study can be valuable for contractors and Engineers in selecting the appropriate excavation supporting system in Addis Ababa.

Keywords: Contiguous pile, Excavations, Excavation Support Systems, Soil Nailing

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LIST OF ABBREVIATIONS

CSPW	Cantilever secant pile wall
FEM	Finite Element Method
FS	Factor of safety
Mc	Mor- Coulomb
LEM	Limit Equilibrium Method
S.Co	Share Company
SNSW	Soil Nailing Shoring Wall

CHAPTER ONE

1. INTRODUCTION

1.1 General

Addis Ababa, the capital city of Ethiopia, has witnessed rapid urban growth, necessitating the construction of tall and substantial structures. In such instances, Engineers face the choice of excavating and creating fresh, undisturbed spaces beneath the ground. The depth of excavation varies based on the structure's intended purpose and required soil-bearing capacity. Deep excavations, especially, demand attention. When extensive earth removal occurs, the unstable soil surrounding the excavation site may collapse, necessitating temporary earth retaining structures, commonly known as excavation supports or shoring, to ensure both safety and precision in the vertical cut. Various methods are employed for excavation support, including soldier beams and lagging, sheet piling, bored pile walls, soil nailing walls, and slurry walls (Diaphragm walls), among others (1).

Contiguous pile systems and soil nailing are among the most widely adopted methods for excavation support in Addis Ababa. The choice of shoring for deep excavations depends on several factors such as the construction site's location, soil conditions, allowable construction duration, construction budget, proximity to existing structures, and available workspace. It's essential to emphasize that when designing a shoring system, cost considerations should remain paramount without compromising safety (2).

Large-scale excavation projects are commonplace in Addis Ababa. This study aims to provide insights by comparing soil nail-supporting systems and contiguous pile systems under three distinct site conditions. The study employs both the Limit Equilibrium method and Finite Element methods using PLAXIS 2D and 3D software.

1.2 Background

In Addis Ababa, specialized firms dedicated to foundations are responsible for constructing excavation support systems. The first foundation specialist company in the country, BAUER MIDROC Foundation Specialist Plc., was established on June 12, 1998. They have undertaken numerous projects employing various methods, including contiguous piles at Nani Building, Bole Cargo, LoLi Building, Bole Tower, Awash International Bank, Ketema Kebede(K.K) Building, Oda Tower, Nohe Real Estate, and Africa Union Grand Hotel shoring projects, solider piles with timber lagging at Africa Union Security, and Secant piles at the Red Cross Building project. While the number of shoring companies is increasing, they tend to use similar methods despite differences in site conditions, limited collaboration due to constraints in market resources (construction equipment and materials), a lack of standardized practices, and confined urban spaces where nearby structures are typically close together.

To address these challenges, this study first examines all currently employed excavation stabilization techniques in Addis Ababa. Subsequently, a more detailed assessment compares various support methods while keeping geotechnical and site conditions consistent.

As a result, this thesis aims to select and construct the most cost-effective, practical, and expedient deep excavation support system using readily available materials, tools, and equipment, along with local expertise. However, implementing these options in the country may prove challenging due to limited experience in the field.

1.3 Statement of the problem

Unreinforced deep foundation excavations are prone to vertical collapse. Effective support systems for foundation pits are crucial to ensure excavation safety and minimize the impact on surrounding soil masses. Addis Ababa exhibits varying soil types across its different locations, yet the protection system used appears uniform, leading to specific problems:

- Shoring systems, while temporary, are costly.
- The use of shoring systems may result in waste materials that are not directly connected to the main structure of the building.
- Installing shoring systems is time-consuming.
- Real estate in Addis Ababa is expensive and demands space that could be utilized for the primary substructure.
- There is a lack of experience in other shoring types including government offices/subcities and consulting offices.
- The local campiness competes with each other by compromising safety which leads to the Collapse of the excavation support system with major damage including human life.
- The machinery has a very short lifespan due to its placement of built-in difficult strata.

Therefore, the purpose of this study is to compare soil nail supporting systems and contiguous pile systems on various site conditions to address the issues mentioned above.

1.4 Objectives

The main goal of this study is to assess and compare shoring solutions that are practical, reliable, appropriate, and adaptable for local firms under specific site conditions and difference soil types found in Addis Ababa.

1.4.1 Specific Objective

1. Investigating and analyzing different urban excavation supporting techniques.
2. Comparing common techniques in different site conditions with conventional methods.
3. Analyzing limit equilibrium and FEM, as well as 3D and 2D models of shoring walls.
4. Evaluating the sensitivity studying of the soil parameters.
5. Proposing an appropriate and safe excavation supporting system depending on the structure to construct and economic conditions.
6. Creating awareness among experts, governmental organizations/sub-cities, and design consultancy firms.

1.5 Methodology

To conduct this study, the initial step involved reviewing recent research on various excavation supporting systems for different soil types. Subsequently, access to different excavation support systems at various sites in Addis Ababa was obtained. The detailed sequence is described as follows:-

Data Collection

1. Visual inspection and data collected from different foundations spiciest company which is found in Addis Ababa, Ethiopia.
2. Gather basic information from various site locations, such as: -
   - Soil investigation report.
   - Excavation depth.
   - Type and condition of the adjacent structure.
   - Shoring types.
   - Shoring total construction cost, period of construction.
   - Working space.
3. Selected and compare three separate site conditions based on: -
   - Soil types such as expansive and non-expansive.
   - Shoring methods.
   - Limit equilibrium and finite element analysis.
   - Working time, cost, and availability of working space…
4. Three different deep excavation support systems were utilized at the selected sites, including:
   - Contiguous shoring piles for the 4B+G+30 headquarters building for Zemen Bank S.C. in the Mexico Senga Tera area.
   - Contiguous Shoring Piles for 4B+G+23 Mixed Residence Building in the Bole Matemia area for Flintstone Homes' Bole Classic Project.
   - Soil nailing with a shotcrete wall for a 2B+G+13 mixed-use building in the Kera area for the Yewub Hagere Group S.C. project.

Data Analysis

The cost and construction period analysis of the two deep excavation stabilizing methods, contiguous shoring piles, and soil nailing walls, can be conducted using numerical methods on three different sites with varying soil strength characteristics, excavation depths, and surcharges.

To analyze the price and construction period, the following steps can be followed:-

1. Input Data: Gather information about the three sites, including soil strength, excavation depths, and surcharges. Also, consider the available space for the supporting system structure.

2. Model Creation: Use numerical methods such as finite element analysis and limit equilibrium analysis to create models of the deep excavation stabilizing methods for each site. Define the parameters, including the properties of the soil, piles/nails, and surcharges.
3. Analysis: Run the numerical models to obtain the output in terms of stability, deformation, and internal forces. Compare the performance of both methods for each site based on these output parameters.
4. Cost Estimation: Based on the obtained designs, estimate the construction costs for each method. Consider factors such as material costs, labor costs, equipment costs, and any additional costs specific to each method.
5. Construction Period Estimation: Analyze the expected construction period for each method. Consider factors such as the complexity of the method, ease of installation, site constraints, and any specific requirements for each method.
6. Evaluation: Compare the cost and construction period estimates for each method across the three sites. Determine which method is more cost-effective and time-efficient based on the obtained results.
7. Recommendations: Based on the cost and construction period analysis, provide recommendations on the suitable method for each site considering the given soil strength characteristics, excavation depths, and surcharges. Also, consider the available space for the supporting system structure.

By following these steps and conducting a comprehensive analysis, it is possible to evaluate the price and construction period for the two deep excavation stabilizing methods and make informed decisions based on the specific site conditions.

Finite Element Analysis (PLAXIS)

The present study utilizes the Finite Element Method, employing PLAXIS 2D and 3D software. PLAXIS is a finite element package specifically designed for analyzing deformation and stability in geotechnical engineering projects. It applies to various practical problems in conventional geotechnical Engineering, including footing, fills, excavations, retaining walls, slope stability, seepage problems, and tunnels. These can be analyzed individually or in combination. The straight forward graphical input procedure enables the creation of intricate finite element models, while the enhanced output capabilities provide a detailed presentation of computational results. Real-life scenarios can be modeled using either a plain strain or an axisymmetric model (3).

1.6 Relevance of Thesis

This comparative assessment of excavation-supporting systems projects aims to provide valuable and pertinent data for future decisions regarding alternative methods and design concepts. The report's findings hold significance for expanding the general understanding of areas where deep basements are frequently chosen for building construction, particularly when they share shear strength criteria similar to those investigated in this study.

1.7 Limitations

This study has some limitations. Firstly, the high level of sophistication and specialized equipment required for these excavation supporting methods can restrict their applicability. Additionally, this study only compared two shoring methods, which may limit the overall understanding of available options. Another limitation is the inadequacy of practical shoring standards in the country, considering its soil and other parameters. Moreover, the geotechnical soil report did not provide a detailed description of the soil parameters, this leading to the coloration the soil parameter with SPT value, which may not fully represent the actual conditions in the selected areas of this study.

1.8 Organization of the Thesis

The organization of this study is demonstrated in different chapters, and a short review of each chapter is given below.

Chapter 1: This chapter includes the following sections: introduction, background, problem statement, objectives, relevance, methodology, and thesis limitations. It sets the foundation for the study by providing the necessary background information, identifying the problem statement, stating the objectives, explaining the relevance of the study, outlining the methodology used, and conceding the limitations.

Chapter 2: The literature review is conducted in this chapter. It presents a comprehensive review of existing literature and research related to excavation supporting systems, exploring various methods and design concepts. This chapter helps establish a theoretical framework for the study.

Chapter 3: This chapter explores the available shoring types specifically in the context of Addis Ababa. It provides an overview of the different methods and approaches used for excavation support in the specific location being studied.

Chapter 4: Focused on data collection, material modeling, and the study of input parameters for the selected shoring methods, this chapter involves gathering pertinent data on soil strength characteristics, surcharges, and excavation depths for different sites. Material models are developed to simulate the behavior of the selected shoring systems, and input parameters are scrutinized to assess their impact on overall performance.

Chapter 5: This chapter presents the study's results and discussions concerning the performance of soil nailing and contiguous shoring pile walls at each site. Findings are analyzed and compared, taking into account factors such as construction costs, available space, and soil strength characteristics. The strengths and weaknesses of each method are evaluated based on the obtained data.

Chapter 6: Dedicated to drawing conclusions and making recommendations for future designs and selections of deep excavation stabilization methods. These conclusions and recommendations serve as valuable guidance for future projects, aiding in the selection of the most appropriate deep excavation stabilization method.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 General

Excavation activities are a daily occurrence worldwide. In the past, when excavation depths were shallower, their impact on the surrounding environment was less pronounced. However, recent years have witnessed an increase in excavation depths in urban areas, resulting in multiple facilities being lost due to ground settlement caused by excavation collapses. This has led to frequent damage to adjacent buildings (4).

Terzaghi and Peck described that excavations whose depths were less than 6m could be defined as shallow excavations and those deeper than that as deep excavations (5). Many deep excavations have been carried out to construct various types of underground infrastructures such as deep basements, subways, underground roads, and service tunnels this type of work presents serious hazards to all workers involved. It is the employer’s responsibility to ensure that workers are adequately trained and protected before entering an excavation. The excavation protection structure is called shoring, which is the action taken to retain soil so that it does not collapse or the procedure of temporarily supporting a building, vessel, structure, or trench using shores (props) when at risk of collapse or during repairs or modifications. Shoring comes from shore, timber, or metal prop (6) .

2.2 Comparative Assessment of Deep Excavation Soil

Common excavation support wall types in Ethiopia include masonry walls, contiguous piles, timber contiguous piles, and soil nailing. The lateral movement of soil generates a lateral force on the support wall, necessitating the quantification or prediction of this lateral movement's impact on the excavation support wall. Lateral soil movement results in bending moments and deflections in the pile supporting the excavation, potentially leading to structural distress or failure in both the existing building and the excavation supporting wall. Instances of structural damage to piles have been reported in different literature. Various theoretical and empirical methods have been developed to address specific excavation problems. In other cases, they explored the impact of soil movement during excavation on the behavior of adjacent existing piles. (3)

2.2.1 Cantilever Contiguous Pile Wall for Supporting Excavation in Clay

In urban areas, situations often arise where new construction basements or underground utilities are proposed adjacent to older buildings. A particular concern is buildings with shallow foundations that do not extend below the influence zone of the adjacent excavation. For deep excavations, the presence of a cantilevered stage at the start of construction can result in excessive movements, with over-excavation being a primary cause of such movements. Cantilevered contiguous piles are commonly used and cost-effective for shallow excavation in cohesive soil. A parametric study was conducted considering excavation depth, pile embedded depth, and wall stiffness, yielding design recommendations for a safe supporting system in clay. Cantilevered walls are ideal for temporary or permanent excavation work, maintaining an open excavation area without internal struts, which facilitates construction. Soil mechanics textbooks recommend excavations of up to about 4.5 meters when using cantilever walls. Long et al. analyzed available databases in the literature, categorizing data into cases with stiff soil at the excavation dredge level and cases with soft soil at the excavation dredge level.

One of the primary concerns during excavations in urban areas is the impact of construction-related ground movements on adjacent buildings and utilities. While excavation leads to both lateral and vertical soil movement, the former is considered more critical for adjacent support walls. These ground movements can cause deformation and damage to adjacent structures, necessitating strict deformation control to minimize harm, especially when excavating in urban areas with soft clays beneath. The required stiffness of an excavation support wall depends on the expected deformations during the project and their impact on adjacent structures (7).

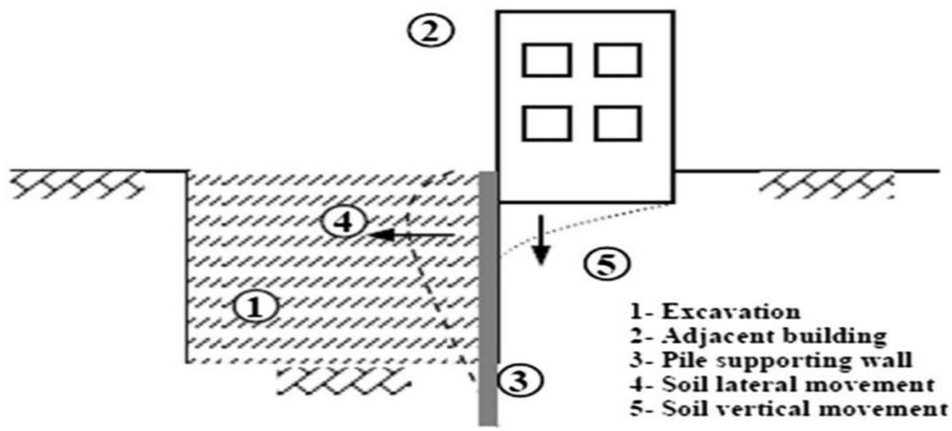


FIGURE 2-1 MODEL OF EXCAVATION, ADJACENT AREA, AND BUILDING

The figure illustrates lateral soil movement toward the excavation, deflection of the supporting wall, and ground movement beneath adjacent buildings caused by excavation in clay soil.

Cantilevered contiguous pile walls are commonly preferred in urban areas for their cost-effectiveness and suitability in cohesive soil for limited-budget projects. In the case of deep excavation, having a cantilevered stage at the beginning of the construction sequence can lead to excessive movements, and over-excavation can exacerbate this. The behavior of cantilever pile walls depends on the stiffness of both the soil and the pile wall (7).

2.2.2 The behavior of a Deep Excavation in Sand

Several published studies have examined ground movements induced by excavations in clay, e.g. (8) (9) (10) and (11), but very few have examined excavations in the sand. Numerical tools can be successfully applied to predict movements induced by deep excavations, but it is often difficult to select appropriate constitutive models and define relevant soil parameters. A review of the literature shows that soil stiffness is the most critical parameter. Therefore, back-analyses using an elastic–perfect plastic model are performed to define the parameters that should be considered for excavations in sand. Numerical analyses were conducted to examine the influences of soil elasticity, creep, and the soil–wall interface on responses induced by the excavation. Interpreting from observation data, the creep rate of wall movement caused in the non-supported stage of the excavation varies between 0.14 and 0.38 mm/day. Finally, parametric studies of interface elements indicate that the most sensitive parameters are the normal (KN) and shear stiffness (Ks) of the interface. (12)

2.2.3 Long-Term Performance of a Deep Excavation in Silty Clay

In the realm of deep excavations, the stiffness of the retaining structure holds paramount significance for performance. Various findings in the literature expound on this aspect (13). Wang asserts that augmenting stiffness can lead to a reduction in the maximum lateral displacement of the retaining structure (14). Conversely, Lee reports that double-row-pile retaining walls exhibit superior performance in terms of reduced displacement when compared to single-row piles. Hence, retaining walls featuring double-row piles emerge as an effective choice when heightened stiffness is necessary. However, Moormann concludes that once the stiffness of the retaining wall surpasses a certain threshold, further stiffness increases may have negligible effects on wall displacement (15).

Leung and Ng discovered that the deflection of the retaining wall and ground settlement tend to be relatively independent of the system's stiffness (16). Extensive studies have elucidated the relationship between horizontal displacement and excavation depth (17). Generally, an increase in excavation depth correlates with a rise in maximum horizontal displacement. Nevertheless, the ratio of maximum deflection to excavation depth exhibits significant variations contingent on soil type and the retaining system (18). Diaphragm walls, in general, exhibit deep-seated inward movements, often forming bulging profiles during excavation. Long determined that in soft soils, maximum deflection typically ranges from 0.05% to 0.25% of the excavation depth, with certain cases reaching up to 3.2% of the excavation depth . In stiffer soils like stiff clay, sand, and residual soil, Leung and Ng reported that maximum lateral deflection and ground surface settlement typically range from 0.1% to 0.3% of the excavation depth (16).

In a broader context, ground surface settlement generally escalates with increasing excavation depth, though literature concerning vertical movements of retaining walls remains limited. Tan and Wei studied the movements at the top of the retaining wall and observed heaves, rather than settlements, at the initial stages of excavation (19). Liu et al. delved into the vertical movement of a retaining wall during deep excavation in clay, revealing heaves without concurrent settlement (20).

Presently, the majority of reported cases focus on monitoring excavation during the construction phase. However, literature exploring the time-dependent performance of deep excavations with unchanged excavation depths remains scarce. It has been observed that the consolidation and creep of soft clay can result in substantial wall deflection, ground surface settlement, and diminished slope stability (21). Ou et al. examined the performance of a 38 m deep excavation in soft clay constructed using the top-down method and noted a significant increase in time-dependent deflection. Approximately 36% of total wall deflection and 44% of total ground surface settlement developed when the excavation depth remained constant. Thus, Ou et al. emphasized the need to consider creep factors when analyzing excavations in soft clay (22).

Furthermore, He et al. recently investigated the long-term deformation of a 17.5 m deep excavation constructed with the bottom-up technique in silty clay. The retaining wall featured one or two rows of contiguous bored concrete piles and tie-back prestressing tendons. Monitoring of settlement and lateral deflection at the top of the retaining structure, as well as ground surface settlement and adjacent building settlement, persisted for over 4 years. Post reaching the final depth, the excavation underwent continuous surveillance for more than 3.1 years. The observations revealed that deformation escalated with increasing excavation depth, with a notable surge in displacement after reaching the final depth. In this study, the averaged time-dependent rates of retaining wall deflection, ground surface, and adjacent building settlement, while excavations remained partially unfilled, ranged from 0.23% to 0.25 mm/month. It's important to note that time-dependent behavior in excavations can result in excessive deformation, posing significant safety risks to the retaining system (23).

2.2.4 Behavior of Cantilever Secant Pile Wall Supporting Excavation in Sandy Soil Considering Pile-Pile Interaction

Deep excavations in granular materials can induce substantial ground movement and subsequent damage to adjacent structures. The majority of previous studies have primarily focused on examining ground movement induced by deep excavation in clay. Moormann analyzed several case histories and found that for non-cohesive soil, the average values of normalized horizontal deflection (δh-max/H%) and vertical displacement at the ground surface (δv-max/H%) were approximately 0.25% and 0.33%, respectively. These values are comparatively small when compared to soft clay, where δh-max/H% exceeds 1%, and δv-max/H% averages around 1%. Such relatively small ground movements in sandy soil can still be significant, potentially leading to full mobilization of strength and eventual failure. In loose sand, full theoretical pressure is reached after a significant wall movement, whereas in dense sand, progressive soil failure occurs at an average soil strength less than the peak value.

The limit equilibrium method had been used to design cantilever-embedded walls in the sand .It is assumed that there is a pivot point close to the wall tip around which the wall rotates rigidly. Active earth pressure is assumed above the pivot point on the retained side while passive earth pressure below that point is on the excavated side (24) . An alternative method had been proposed by King based on a series of centrifuge tests. He suggested that the location of the pivot point is 0.35 times the embedded depth of the wall (25) . Later, Day suggested that the location of the pivot point is not constant and its location is a function of the ratio of passive to active earth pressure coefficient.

Conte et al. proposed an analytical method to predict the net earth pressure on the wall and bending moment of the wall. Their proposed method was in a good agreement with other results and experimental results Madabhushi and Zeng. It should be noted that such theoretical methods based on the rotation of a rigid wall may not be suitable for relatively flexible embedded walls where the passive earth pressure is partially mobilized. Mei et al. developed a model that can predict earth pressure as a function of wall movement. such a solution helps in the design of embedded retaining walls where the full active or passive pressures did not reach. Hsiung et al. performed finite element modeling (FEM) for excavation in sand. The FEM was calibrated using several case studies. The measured lateral movements indicate that the wall initially behaved as a cantilever; however, after the installation of the struts, the wall started to behave as a propped cantilever and the movements continued to increase during the excavation.

Khoiri and Ou recommended that a suitable value of unloading reloading elastic modulus (Eref ur) should be used to capture the small-strain behavior at the excavation base. In other analysis model input, parameters were calibrated through laboratory compression and triaxial shear tests on sand specimens obtained from the excavation site. They concluded that good predictions of excavation performance can be achieved through careful site-specific calibration of the sand behavior and using a constitutive model capable of representing variations in stress-strain-strength properties as function of the confining stress and void ratio. Hsiung et al.

Cantilever secant pile wall (CSPW) used to support excavation in sandy soil, A major concern in supporting excavation is to predict and control ground movement associated with excavation particularly in cohesion less soils, as it could trigger global instability and catastrophic failure. The magnitude and distribution of lateral earth pressure and ground movement depend mainly on soil properties, excavation depth, excavation plan geometry, the stiffness of the supporting wall, and the contact between the secant piles themselves, a parametric model using a wide range of sand density, excavation depth, wall flexural stiffness, and bonding between piles within the wall.

The results allowed for the development of an approach to predict both the wall deflection for the case of fully and partially bonded piles. This will help engineers to predict ground movement and select an appropriate supporting system that can maintain the stability of the adjacent structures. (26).

2.2.5 A Study on the Method of Stability Calculation of Soil Nailing Expansive Soil Slope

Engineering practice shows that rainfall is the main cause of expansive soil slope instability. It increases the water content in the soil, which raises the saturation level and reduces shear strength. Based upon previous research, reasons for slope instability can be summarized as:

(1) Rainfall penetration leads to an increase in water content and a decrease in soil cohesion, internal friction angle, and soil matric suction, which finally causes the weakness of soil shear strength (27).

(2) Rainfall penetrates along with the crack to deep soil and the crack will be expanded under the water pressure, which leads to slope instability.

(3) Rainfall results in seepage strength, which also leads to slope instability (28) .

Soil nailing support was used for slope support and geotechnical reinforcement in 1980s in China. It was widely used because of its low cost, short time limit, and high adaptability. It brought obvious social and economic effectiveness. Soil nailing slope assistance differs from other types of slope support construction in that it is built layer by layer. It will change the position of the potential slide plane and stability has to be checked during construction. The expansive soil slope excavation may cause a sudden changes in the soil stress field and strain field, which leads to abrupt change in soil body, soil nailing support, and slope stability. The paper, therefore, intends to study the stability and optimization of desigthe n of soil nailing support for expansive soil slope.

Yu Xiao-jin, based upon the study on slope excavation engineering stability, put forward the parameter for cohesion and rupture angle. Regression formula was given for soil and soil nailing support through experimental data fitting, which was used for predicting the safety of soil nailing support (29) . Huo Ruo-lan built numerical analysis model for soil nailing support in pit excavation. Interfacial element unit was used to analogy deformation of soil nailing interface. It was confirmed to be reliable through data analysis and engineering practice (30) .

Zhang Qing-shan analyzed the monitoring data of field deformation in support of construction and studied the law of horizontal displacement and sedimentation. His study indicated that construction procedure, supporting layout, and surrounding condition were the main factors for deformation in pit engineering support (31) .Xiao Xi-ze built experimental model of soil nailing support for pit engineering and it showed soil nailing dip angle had marked impact on deformation and stability for side wall (32). The above researches provide important theoretical basis for analysis, design, and construction of soil nailing support, extends the applied field, and indicates the significance of soil nailing support.

Tu Bing-xiong used sheer lag theory to build computer model for soil nailing internal force transfer and analyzed the transferring law of internal force. Through theoretical analysis and careful calculation, the model was useful for the design of soil nailing support and analysis of internal force (33). Ding Min set up optimized model for soil nailing support, selected the acceptable range for parameter after analyzing parameter sensitivity, and improved the genetic algorithm by using adaptive dynamic technique and non-standard genetic operator. The research showed that improved genetic algorithm made it more rational for soil nailing support (34) .

To sum up, lots of achievements have made about the study on soil nailing support for slope at home and abroad, but little research has done on soil nailing support for expansive soil slope. Wu Kun-ming and Fang Jin-miao study stability and optimized design for soil nailing support at expansive soil slope so to provide the theoretical basis and optimized parameter for soil nailing expansive soil slope (35) .

2.2.6 Contiguous Pile Wall in Silt Clayed and Silt Sandy Soils

Mostly, clayed silt and sandy silt are generated by wind deposition and have a tendency, medium to high, to undergo processes of erosion or collapse, with increasing moisture. In several cases, the walls of piles have been used as control elements of soil thrust. In some cases, the piles act simultaneously to support vertical loads. In the case of contiguous piles, the separation between piles depends on the characteristics of the soil to contain, the dimensions of the piles, and soil structure interaction. One of the first definitions of the theoretical behavior of these systems was carried out by Ito and Matsui, originally developed for the case of slope stabilization systems (36) .

Kok and Huat conducted a review of current knowledge regarding the use of piles in slope stabilization. Some comments of these authors are summarized below (37) . The successful use of this method has been described by Sommer (38) .

The elastoplastic model proposed by Ito and Matsui is shown in Figure 2-2.

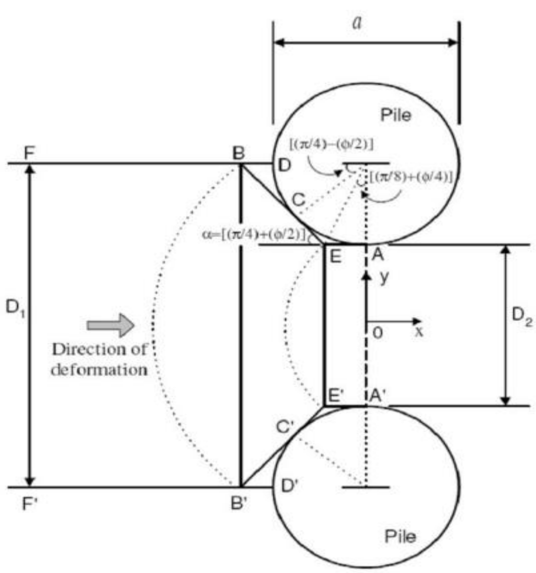


FIGURE 2-2 ELASTOPLAST MODEL FROM ITO AND MATSUI.

Carrubba et al. present a full-scale reinforced concrete instrumental pile to study the response of piles used to stabilize a sliding slope. The concept of equivalent load was introduced by Guo, which allows a correlation between an equivalent load and the magnitude of soil movement. This concept is based on elastic-plastic solutions for either a free-head or fixed-head pile (39) .

Yamin , provides a detailed analysis of the behavior of contiguous pile walls about its application in slope stabilization (40) .Firat analyzed the behavior of the walls of contiguous piles through the use of elastoplastic flow models. For this problem, a finite element model is used. In these studies, the problem data, the relationship between the efforts received by the system of piles, and the relationship between the diameter of piles and their separation (41).

2.3 Soil Nail Shoring Wall

The soil nail shoring wall system consisted of temporary, top-down soil nail walls that utilized portions of the concrete basement walls of the existing library. Over the last decades, soil nailing still attracted the attention of many engineers and researchers. It is a reinforcement technique that uses passive elements drilled and grouted sub horizontally into the ground. This method is passive because it only resists when soil displacement occurs; it is also one of the techniques the most used in situ for soil retention around the world (42) .

The effects of different reinforcement measures on the safety factor and cost were analyzed. A limited number of geotechnical studies have been reported in the literature on the use of soil nailing technique of optimization, from which we can cite the precursor work of Sivakumar Babu and Singh where to better understand the deformation behavior of the soil nail walls, they carried out numerical experiments and regression models for the maximum lateral deformation and the prediction of the safety factor (43). Johari et al. used stochastic analysis using the random finite element method of system reliability analysis of soil nail walls. The article is divided mainly into two parts; the first is devoted to the finite element model for representing the geometry and soil-structure aspects of a soil nailing system, while part two presents the optimization approach in which different parameters with their influence on the safety factor are discussed. Most of the methods for soil nail wall stability analysis are based on the limit equilibrium method (LEM) for its simplicity and the reduced number of required parameters. A numerical study on the optimum layout of soil nails to stabilize the slopes was performed by Fan and Luo. Tan et al. studied in detail the effects of 2D modeling of 3D soil nailing problem; Rawat and Gupta used limit equilibrium and finite element methods in the analysis of a nailed soil slope; Deng et al., and Quansah et al. presented the optimum design with limit equilibrium analysis for the stability of soil nailed slope. While the use of the finite difference or finite element methods has also attracted engineers in recent times, numerous researches have focused on FEM techniques for soil nail wall analysis; it is a powerful tool, which can be utilized for soil nail wall modeling. The main advantage of FEM is providing information about deformations of the soil nail system. Zhang et al. developed a three-dimensional (3D) finite element model (FEM) for the deformation analysis of nailed soil structures; Babu et al. analyzed a real vertical cut supported with retaining wall and soil nailing system, by 2D FEM program; Rawat and Gupta analyzed using finite element software PLAXIS 3D the failure pattern and load-settlement plots for the various unreinforced and reinforced slopes. With recent advances, the validation of soil nail models can be done using finite element software packages like PLAXIS, FLAC, GEO5, and ABAQUS (44).

Cetin, Dogan study, data on the lateral wall displacements of 28 deep excavations in Istanbul soil from over 90 field monitors were collected and analyzed. The most commonly used retaining walls for deep excavations in Istanbul are soil-nailed shotcrete walls (SNPWs) and contiguous pile walls (CPWs). The analyses were performed under two main headings: analysis of soil-nailed walls' data and analysis of anchored walls' data. The data of $\delta_{h,max}$ generally range from 0.05%H to 0.35%H with an average value of 0.20%H in the soil-nailed projects. The data of $\delta_{h,max}$ generally range from 0.043%H to 0.32%H with an average value of 0.155%H in anchored pile wall projects. Some factors affecting the deformation of deep excavations, such as nail density, embedded wall ratio and wall stiffness, were studied based on field-monitoring data. Wall movements were also compared with observations in some case histories from around the world (45) .

2.4 Design and Construction Consideration of Deep Excavation Supporting System

The theoretical method provides some basic understanding of the performance of deep excavation in a different way, but it has some limitations due to its simplicity and assumptions. The magnitude and distribution of induced deformation due to deep excavation depend on many factors such as: -

- Soil and groundwater condition
- Construction quality
- Excavation geometry
- Excavation sequence
- Duration of excavation
- The existence of adjacent buildings
- Wall stiffness
- Type, and installation of lateral support.

The prediction of deformation based on a theoretical method would be very complex to obtain the interaction between those mentioned factors. Some of the theoretical methods are reviewed in these sections. The excavation Supported system considers as:-

- Magnitude and distribution of earth pressure
- Internal structure and bending moment
- Stiffness and strength of ground and the state of stress

2.4.1 Magnitude and Distribution of Earth Pressure

The earth retaining walls withstand lateral pressures either from Earth or any other material on their faces. The pressures acting on the walls try to move the walls from their position. There are two lateral earth pressure theories developed by Coulomb, and Ranking, which estimate the magnitudes of two pressures called active earth pressure and passive earth pressure. Coulomb first studied the earth pressure problem using the limit equilibrium method to consider the stability of a wedge of soil between a retaining wall and the failure plane. He considered wall friction of wall on the soil surface and thought the soil that he assumed was isotropic and homogenous (46).

Rankine presented a solution for lateral earth pressures in retaining walls based on the plastic equilibrium. He assumed:

- There is no friction between the retaining wall and the soil.
- The soil is isotropic and homogenous.
- The friction resistance is uniform along the failure surface, and both the failure surface and the backfilled surface are planar.

These theories are only applicable under certain conditions to estimate roughly the earth's pressures on the wall. Moreover, they do not consider the construction process and give no indications of the wall deformations and ground movements in the more complex braced deep excavations. Stability analysis is important in the design of retaining structures in clay, Terzaghi suggested a mechanism consisting of a soil column outside the excavation which creates a bearing capacity failure. The failure is resisted by the weight of a corresponding soil column inside the excavation and also by adhesion acting along the vertical edges of the mechanism (47).

2.4.2 Internal Structure Force and Bending Moment

Excavation support structures are mostly subjected to horizontal stresses such as water pressure, and soil pressure, as well as ground overloads such as extra pressures. In the classic support design, the soil around the excavation is regarded as the load, and the type of soil pressure to design for is determined by the displacement of the retaining structure. As computer technology advances, some mathematical calculation methods, and software are increasingly applied to the analysis and calculation of excavation support structures, forming a full set of numerical simulation analysis methodologies.

Bowles programmed to solve the finite element analysis of elastic foundation beam (48). Morishige Ryoma simulated transformations of soil pressure on retaining walls and proposed the calculation method that displacement makes soil pressure increase or decrease (49). In addition, the finite element analysis method can also be used with a lot of influence factors together, and establish the constitutive relation model of soil. It is one of the widely used analytical methods at present. The internal force analysis of the retaining structure is an important part of the design of excavation engineering, and the common methods are static equilibrium and elastic fulcrum. Internal structure force and bending moment depend to a great extent on the stiffness of the structure.

2.4.3 Stiffness and Strength of Ground and State of Stress in Ground

The design of an earth retaining system for deep excavation requires both ultimate limit states and serviceability limit states to be considered. An ultimate limit state of a structure is considered to have been reached when sufficient parts of the structure, the soil around it, or both have yielded to result in the formation of a failure mechanism in the ground or severe damage in the principal structural components. A serviceability limit state of a structure is supposed to have been reached with the onset of excessive deformation or deterioration.

The ultimate limit state of the wall includes checking the following:

(i) Overall Stability: the establishment of enough embedment depth to avoid wall overturning and overall slope stability.

(ii) Basal Failure: following excavation to the formation level, the wall penetration depth must be adequate to prevent basal failure in front of the wall.

(iii) Hydraulic Failure: the wall's penetration must be adequate to preclude piping or a "blow out" in front of the wall following excavation to formation level. Aside from the failure possibilities mentioned above, the external pressures acting on the walls and supports must also be considered in the design, as indicated in Figure2-3.

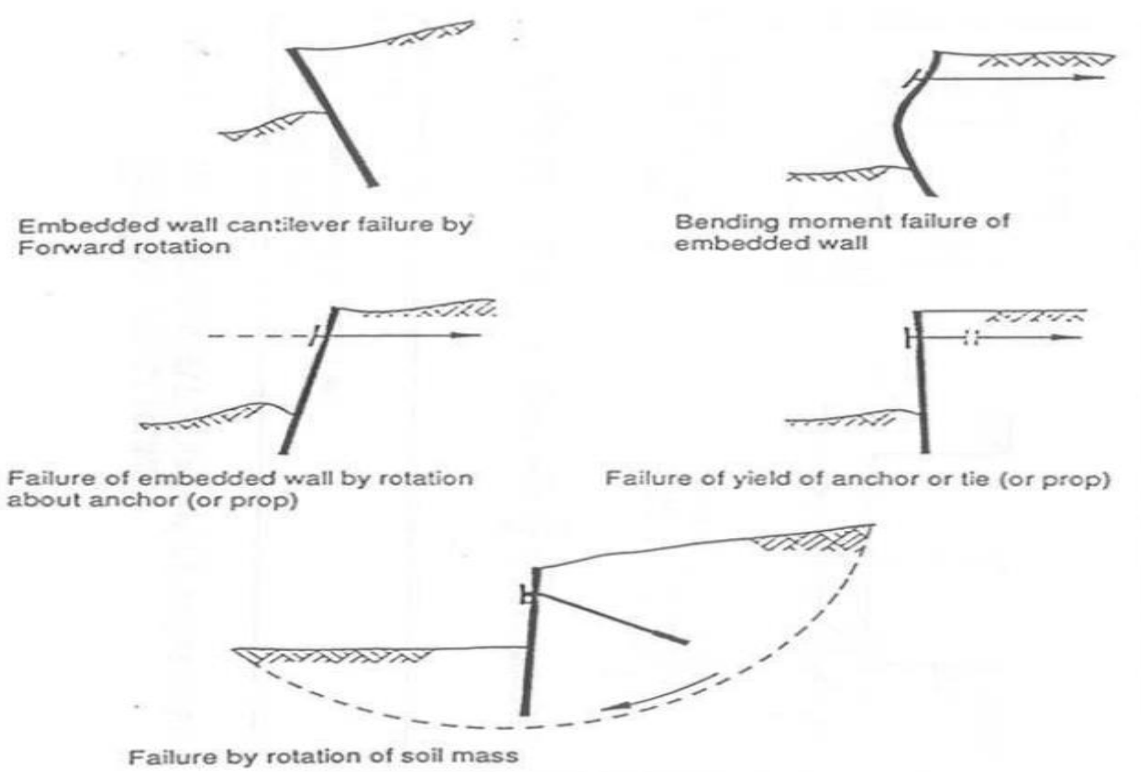


FIGURE 2-3 POSSIBLE FAILURE MECHANISMS OF RETAINING WALL

The serviceability limit state is to be considered in terms of wall and soil deformation at the rear of the wall. Usually, the check on the serviceability limit state is carried out using a computer program. There are also some empirical or semi-empirical methods for predicting wall and retained soil deformation. In the design, special care should be taken to control the groundwater level in the retained ground because the lowering of the groundwater level would increase settlement and cause damage to the adjacent structures and services.

2.4.4 Overall Stability

The overall stability of both retaining walls is often evaluated using limit equilibrium methods of analysis in which the conditions of failure are postulated, and a safety factor is applied to prevent its occurrence. There are several ways of applying the safety factor for overall stability. They are:

- Factor on embedment:- A safety factor of safety is applied to calculate embedment depth at limiting equilibrium. The method is described in the US Steel Sheet Piling Design Manual (United States Steel Corporation and the British Steel Corporation Pilling Handbook.

- Factor on moments of gross pressure: This method applies a factor of safety to moments of gross pressure on the passive side only. Water pressure is not factored in. The method is described in NAVFAC Design Manual 7.2 by the US Navy.
- Factors on moments of net total pressure: ` The net horizontal pressure distribution acting on the wall is calculated and the safety factor is defined as the ratio of moments of the net passive and active forces.
- Factors on net passive resistance or Potts and Burland Method: Developed by Potts and Burland it is analogous to the calculation of the bearing capacity for a strip load. This method defines the safety factor as the ratio of the moment of the net available passive resistance to the moment activated by the retained material including water and surcharge.
- Factors on shear strength on both active and passive sides: Soil shear strengths are reduced by dividing c’ and tan φ’ by a factor of safety, and the active and passive pressure diagrams are calculated using these reduced values. The reduced values approximate mobilized values. Bending moments and prop loads derived from the calculation can be used for wall design if they are treated as ultimate limit state values. This method is recommended in BS8002, 1994.

CHAPTER THREE

3. TYPES OF EXCAVATION SUPPORTING SYSTEMS IN ETHIOPIA

3.1 The Common Types of Excavation Supporting Systems

The Contiguous piles system is the most commonly used shoring method in Ethiopia during the excavation phase of construction projects. The choice of shoring method depends on various factors including excavation depth, surface water availability, soil type, and more. Selecting the right excavation support system significantly influences the duration, quality, safety, and profitability of construction projects involving deep excavations.

Shoring in the context of excavation involves both primary and auxiliary structural components. The primary components of excavation support systems include shoring pile walls, sheet piles, diaphragm walls, and others. These components serve as the main structural elements of the excavation support system. Depending on their role in load transfer, the supporting elements are categorized into two groups: external support systems, such as ground anchors, which function by transferring lateral excavation loads beyond the active soil movement zone, and internal supports like struts and rakers, which handle lateral loads within the excavation area.

3.2 Gravity Retaining Walls

Gravity retaining walls are robust structures employed for lateral support against soil or rock displacement due to excavation, gravity-induced landslides, or erosion. These walls solely rely on their self-weight to counteract lateral earth pressure, necessitating a substantial gravitational load to withstand soil pressure. This method is recognized as one of the means for supporting deep excavations in Addis Ababa, often achieved through the swift construction of masonry walls across the entire excavation or on selected sides. Materials such as masonry, stone, and mortar are commonly used in the construction of these walls.

FIGURE 3-1 EXAMPLE OF EXCAVATION SUPPORT SYSTEM BY GRAVITY RETAINING WALL

This method needed more space and not fully guaranteed especially when existing heavy structures surrounded the excavated site. Moreover, hazardous problems may occur during excavation and construction of retaining walls such as failure of existing roads, or nearby structures. Such problems were observed even in summer seasons when rainfall does exist. Another disadvantage of this method is that it will reduce the area of the site due to the large thickness of the retaining wall, especially at the bottom. This type of excavation support system always integrates into the structure itself (50).

FIGURE 3-2 FAILURE OF MASONRY RETAINING WALL AND EXISTING ROAD IN A. A

3.3 Contagious pile wall

Contiguous pile walls are rotary bored cast-in-situ concrete walls. It is suitable for granular soils, cohesive soils, and soft strata of rock, but it is often unsatisfactory for fine granular soils with high groundwater, very soft clay, and loose organic soils. Contiguous pile walls are simpler for any depth of excavation support than other excavation support systems. To reduce the lateral bending pressures, often the top pile bracing beams at the pile top are used to work together as one unit. The contiguous piles are spaced between sequenced piles, which are covered or not covered with arched shotcrete walls reinforced with a steel mesh or masonry wall.

Based on excavation depth the contiguous piles shoring system is classified into two categories: -

1. Cantilever shoring pile
2. Anchored contiguous piles

3.3.1 Cantilever Contiguous piles

The cantilever contiguous pile wall represents a retaining structure performed without an additional supporting system. The wall is executed by digging under the level of excavation or slide surface and stabilizing the soil mass due to the resistance of the material in the front. It is not disturbed excavation, a simpler constriction procedure due to a simpler stage of construction and it does not require support installation that may pass below adjacent structure.

The construction stages are:-

1. Borehole Drilling.

2. Installation of reinforcement or timber pole.
3. Casting concrete.
4. Bulk excavation.
5. After the bulk excavations apply shotcrete membranes or masonry walls.

FIGURE 3-3 CANTILEVER CONTIGUOUS PILES USUALLY USE REINFORCEMENT, TIMBER POLES, AND CONCRETE.

3.3.1.1 RCC contiguous pile

Contagious pile without shotcrete membranes

The use of contiguous piles without shotcrete membranes is particularly suitable for construction sites where the subsurface conditions consist of weathered rock and have a low water table. In such cases, these piles can act as self-stabilizing elements, eliminating the need for additional shotcrete walls between the piles.

Contiguous piles without shotcrete membranes offer several advantages in terms of cost and efficiency. By utilizing the inherent stability provided by the weathered rock, the construction process can be streamlined, resulting in lower overall costs. Additionally, the elimination of shotcrete membranes reduces the need for additional materials, saving both time and resources.

FIGURE 3-4 CONTIGUOUS PILES WITH OUT SHOTCRETE WALL IN ETHIOPIA, A. A

Contiguous pile with shotcrete or masonry wall

Shotcrete is often used between contiguous pile walls to provide additional structural support and increase the overall stability of the wall. The shotcrete is applied directly onto the surface of the contiguous piles, forming a continuous layer of reinforced concrete.

The primary purpose of using shotcrete between contiguous pile walls is to prevent soil movement and lateral displacement. The shotcrete acts as a protective layer, providing resistance against soil pressure. It reinforces the piles and helps to distribute the applied loads evenly along the wall, reducing the potential for deformation or failure.

Furthermore, shotcrete can be used to repair and strengthen existing contiguous pile walls that may have experienced deterioration or damage over time. The application of shotcrete can restore the wall's structural integrity and extend its service life.

In this type of shoring system, instead of shotcrete membranes, the masonry wall or brick wall is built to retain weak material that may be filled with or distributed among soil strata.

FIGURE 3-5 CONTIGUOUS PILES WITH SHOTCRETE WALL IN ETHIOPIA, A. A

Contiguous pile with masonry/brick of shoring system, a masonry, or brick wall is constructed instead of using shotcrete membranes to retain soil between the piles.

The use of a masonry or brick wall provides a cost-effective alternative for retaining the weak material while still offering structural stability. The wall is constructed by placing individual bricks or masonry units in a systematic arrangement, typically using mortar to bond them together. It helps distribute the lateral earth pressure and provides resistance against potential soil movements. The materials used, such as bricks or masonry units, can be chosen depending on the desired strength and aesthetics of the wall.

Compared to shotcrete membranes, the construction of a masonry or brick wall may require more time and labor. However, it often provides a more visually appealing and durable solution. The use of traditional masonry techniques can also offer added benefits such as thermal insulation and noise reduction.

It is important to note that the design and construction of the masonry or brick wall should be carried out by experienced professionals, taking into consideration factors such as the soil conditions, anticipated loads, and applicable building codes and regulations. Proper reinforcement, drainage, and waterproofing measures should also be incorporated as necessary to ensure the long-term stability and performance of the shoring system.

In summary, the use of a masonry or brick wall instead of shotcrete membranes in this type of shoring system offers an alternative solution for retaining weak soil strata. While it may require more time and labor during construction, it provides structural stability, versatility, and improved aesthetics to the overall project.

FIGURE 3-6 CONTIGUOUS PILES WITH MASONRY WALL IN ETHIOPIA, A. A

3.3.1.2 Timber cantilever contiguous pile

When the excavation depth is not more than 6 meters, and the water table is typically situated below the foundation level, for these specific conditions, the use of a cantilever contiguous pile system with timber poles proves to be a more economical choice.

The choice of using timber poles in this system is based on their availability and cost-effectiveness. Timber is a readily available material in the region, making it relatively affordable compared to other construction materials. Additionally, the construction process for installing timber poles is simpler and requires less specialized equipment, further reducing costs.

By utilizing timber poles, the cantilever contiguous pile system offers sufficient lateral support for the excavation, ensuring that the soil remains in place and that the stability of adjacent structures is maintained. This system is particularly suitable for relatively shallow excavations, such as those commonly encountered in Addis Ababa.

Construction Sequence: -

Step 1: Drill the hole as designed diameter and depth of the shoring pile, then install the timber pole into the drilled pile hole.

FIGURE 3-7 DRILLING AND TIMBER POLE INSTALLATION OF CANTILEVER CONTIGUOUS PILE

Step 2: Concrete casting the drilling pile hole after inserting the timbers and top pile bracing beam concrete casting.

FIGURE 3-8 BRACING BEAM CONCRETE CASTING

Step 3: Bulk excavation and shortcrete apply between contiguous piles.

FIGURE 3-9 SHOTCRETE APPLICATION OF TIMBER PILE

If the soil is very stiff or hard rock strata the shortcrete membrane will not applied because that is stabilized from collapse by itself.

When excavation soil is very stiff soil or hard rock strata, the need for a shotcrete membrane in shoring pile installations may not be necessary. The soil or rock strata itself serves as a stable barrier and can withstand lateral earth pressure without the requirement for additional stabilization measures.

However, this decision should be made based on thorough Engineering evaluations and geotechnical assessments to ensure the overall integrity of the shoring pile installation.

FIGURE 3-10 CANTILEVER CONTIGUOUS TIMBER PILE WITH OUT SHOTCRETE WALL

3.3.2 Anchored Contiguous Piles

Anchored wall system is one of the common methods used for deep excavation stabilization adjacent to sensitive structures in Addis Ababa urban areas. A key aspect of the stability analysis of deep excavations is the number of deformations occurring on the facing wall and the adjacent structure.

Ground anchor: -A prestressed grouted ground anchor is a structural element installed in soil or rock that is used to transmit an applied tensile load into the ground. Grouted ground anchors, referenced simply as ground anchors, are installed in grout-filled drill holes. Grouted ground anchors are also referred to as "tiebacks". The basic components of a grouted ground anchor include the:

- Free stressing (unbounded) length
- Bond length and
- Anchorage

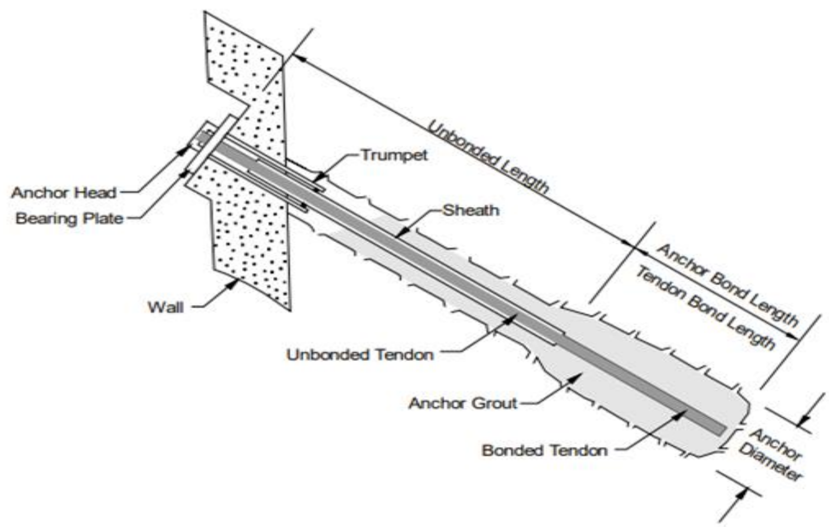


FIGURE 3-11 COMPONENT OF GROUND ANCHOR

1. The unboned length is that portion of the prestressing steel that is free to elongate elastically and transfer the resisting force from the bond length to the structure. A bond breaker is a smooth plastic sleeve that is placed over the tendon in the unboned length to prevent the prestressing steel from bonding to the surrounding grout. It enables the prestressing steel in the unboned length to elongate without obstruction during testing and stressing and leaves the prestressing steel unboned after lock-off.

2. The tendon bond length is that length of the prestressing steel that is bonded to the grout and is capable of transmitting the applied tensile load into the ground. The anchor bond length should be located behind the critical failure surface. A portion of the complete ground anchor assembly is referred to as the tendon. The tendon includes the prestressing steel element (strands or bars), corrosion protection, sheaths (also referred to as sheathings), centralizers, and spacers, but specifically excludes the grout. The definition of a tendon, as described in PTI (1996), also includes the anchorage; however, it is assumed herein that the tendon does not include the anchorage. The sheath is a smooth or corrugated pipe or tube that protects the prestressing steel in the unboned length from corrosion. Centralizers position the tendon in the drill hole such that the specified minimum grout cove r is achieved around the tendon. For multiple element tendons, spacers are used to separate

the strands or bars of the tendons so that each element is adequately bonded to the anchor grout. The grout is a Portland cement-based mixture that provides load transfer from the tendon to the ground and provides corrosion protection for the tendon.

3. Anchorage: - These and other components of a ground anchor are shown schematically in figure. The anchorage is the combined system of anchor head, bearing plate, and trumpet that is capable of transmitting the prestressing force from the prestressing steel (bar or strand) to the ground surface or the supported structure.

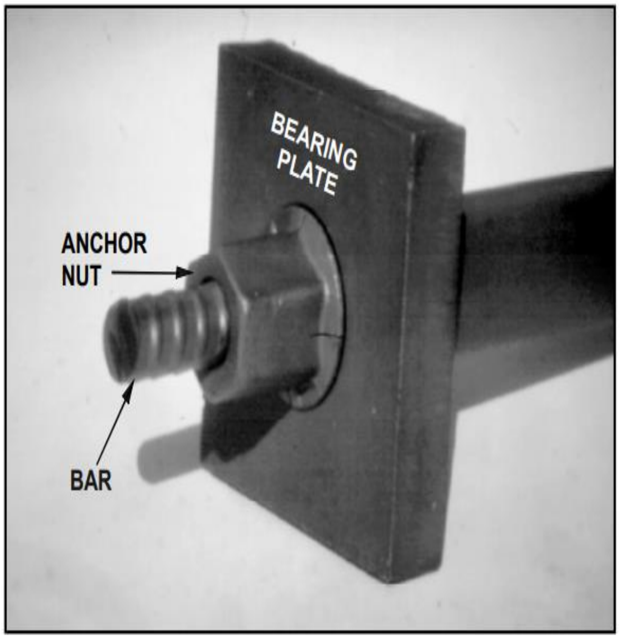
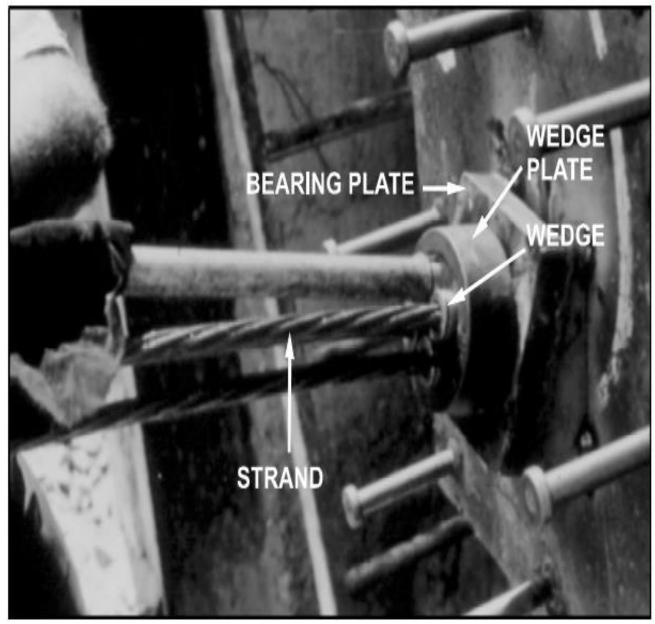


FIGURE 3-12 A, ANCHORAGE COMPONENTS FOR A BAR TENDON



B, ANCHORAGE COMPONENTS FOR A STRAND TENDON

Based of anchorage anchored contiguous piles are classified into three categories: -

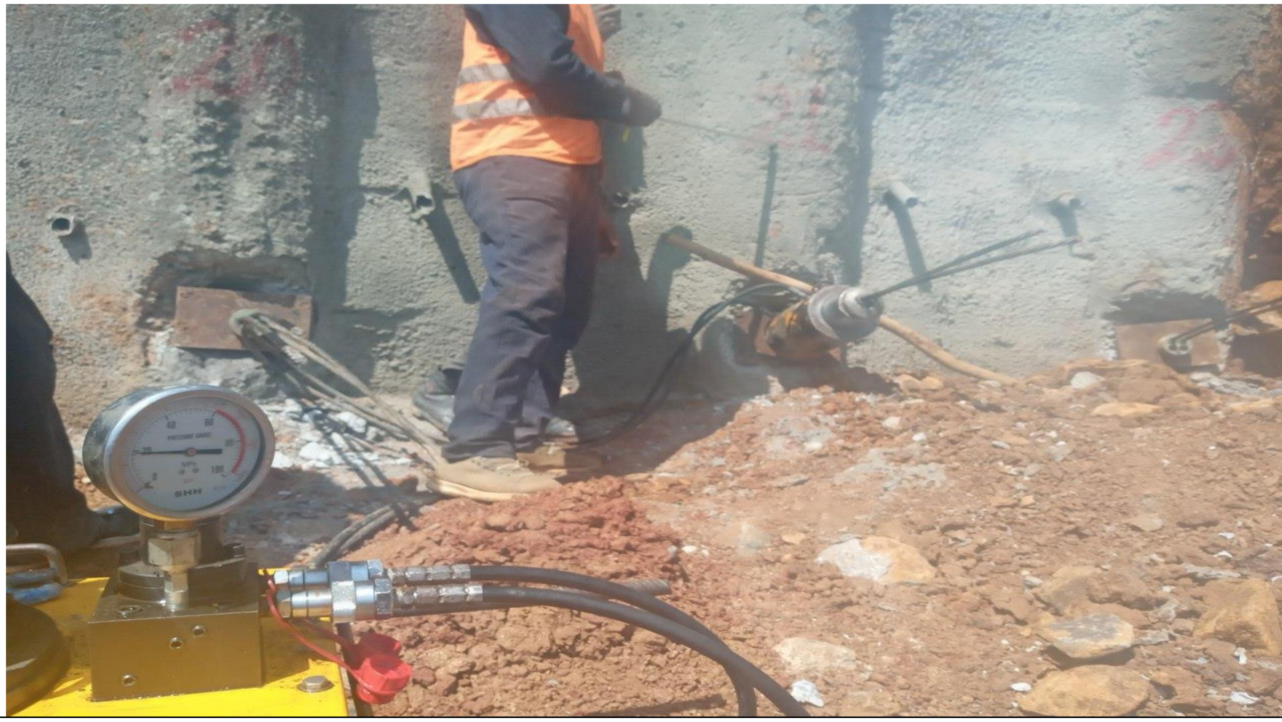
- Pile core anchored contiguous pile wall
- Steel I-Section anchor beam contiguous pile wall
- RCC anchor beam contiguous *piles*

3.3.2.1 Pile core anchored contiguous pile wall

This type of anchorage core the pile section with anchor drilling machine, which is cute minimum two main rebar and 130mm-150mm concrete from each pile. After application anchor strand installation and cement grout the anchor system is locked with anchorage. The main drawback of this type of anchoring system creates weak point on pile section when the anchor layers are large or deep excavation.

FIGURE 3-13 PILE CORE ANCHOR DRILLING

FIGURE 3-14 PLACING OF ANCHORAGE AND ANCHOR STRESS JACK TOOLS



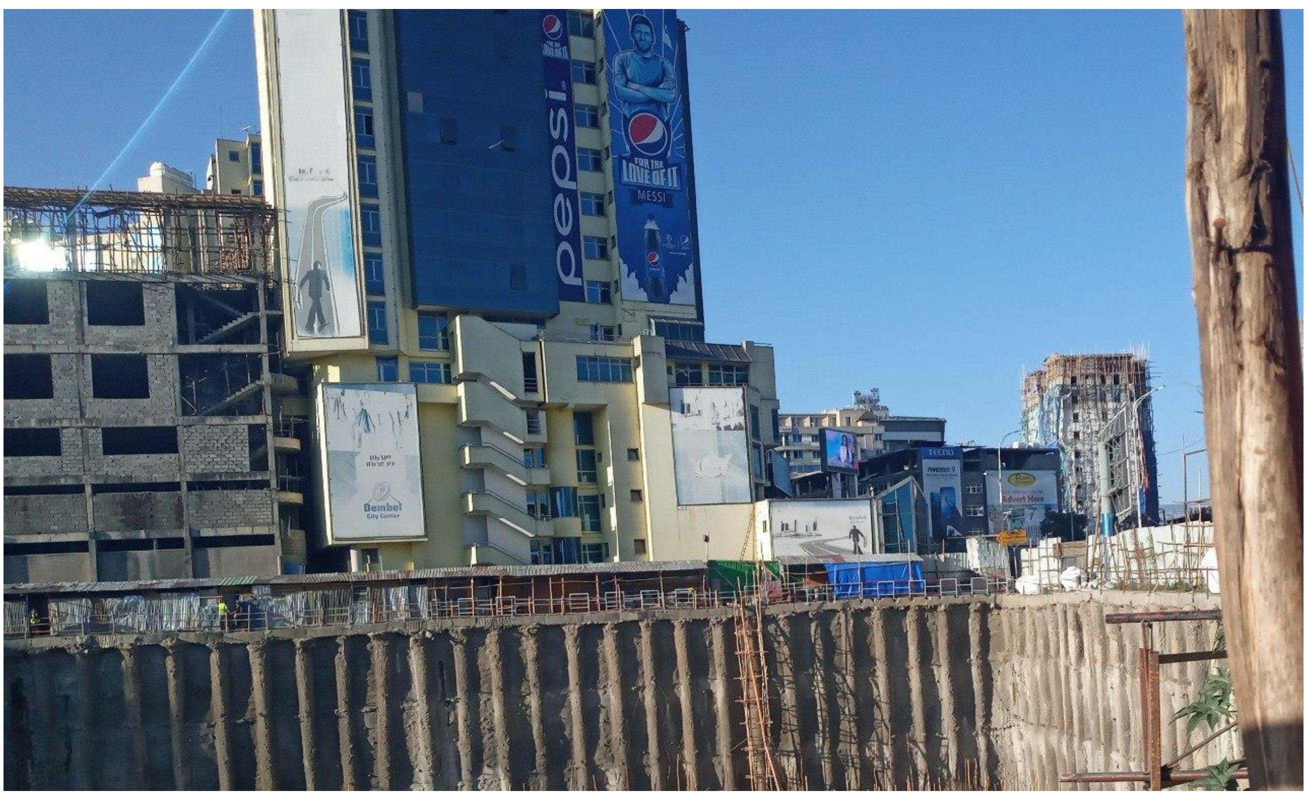


FIGURE 3-15 ANCHOR LOCKING AND PILE CORE ANCHORED PILES VIEW

3.3.2.2 Steel I-Section anchor beam contiguous pile wall

The anchors are drilled between the consecutive contiguous pile's installed strands and filled with cement grout then a steel I-section anchor beam is applied for the anchor locking. The steel I section beam availability in the local market with quality is very difficult, but the material can be reused for another projects after basement construction done.

FIGURE 3-16 PLACING OF STEEL I-SECTION ANCHOR BEAM AND LOCKING OF ANCHOR

FIGURE 3-17 VIEW OF STEEL I- SECTION ANCHOR BRAM WITHOUT SHOTCRETE WALL

3.3.2.3 RCC Anchor Beam Contiguous Pile

This system drilling procedure is same to I-section steel anchor drilling system only different is instated of I-section steel reinforced concrete anchor used as anchorage. It need space after pile face and the form work and concrete casting takes time relative to other anchorage type of anchorage.

FIGURE 3-18 PREPARATION OF REBAR AND FORMWORK FOR RCC ANCHOR BEAM

FIGURE 3-19 ANCHORED SHORING PILE WALL SYSTEM

FIGURE 3-20 RCC ANCHOR BEAM WITH MASONRY WALL BETWEEN THE CONTIGUOUS PILESSOIL NAILING WALL

In this country, this method of deep excavation supporting system is not frequently used like conventional contiguous pile shoring system. In recent years soil nailing has come into widespread use geotechnical engineering as a means to stabilize cut slopes and natural slopes in granular soil with some cohesion, stiff clay, and rocks. The term soil nailing after Jewell (1990b), refers to the use of relatively small diameter bars(typically 20mm to 30mm) inserted into the ground direction or more usually by drilling and grouting and oriented to develop the tensile (rather than compressive) axial force. Steel bare or rock bolts have been used for more than forty years to support excavation surfaces and sand portals in tunneling, the application of, and vigorous research on soil nailing were started in the 1970s in Germany, France, and the United states. Since then, soil nailing has been used successfully in temporary and permanent applications under a wide variety of ground conditions.

Recently several new techniques to decrease the displacements of nailing slopes due to the exaction have been proposed, where especially in urban areas excessive displacements, as well as problems with groundwater, were regarded as the major shortcomings in applying soil nailing. Extensive reviews of the historic evolution and major features of soil nailing are summarized by Bruce and Jewell (1986/1987) and Gassler (1990).

Applicable soil nailing type in Addis Ababa:-

- Soil nailing with timber and shortcrete wall.
- Soil nailing with steel rebar and shotcrete wall.
- Soil nailing with the contiguous pile.

3.3.3 Soil Nailing with Timber and Shortcrete Wall

When the excavation depth is shallow, driving wooden nailing by using hammer or anchor machine that assures that the energy needed to penetrate the nailing to the required depth is transmitted to the nailing head without damaging the nailing wood. After that the excavation face covered by shotcrete. This type of soil nailing wall is suitable for soft soil with a low water table and short-period protection work.

FIGURE 3-21 SOIL NAILING WITH TIMBER AND SHORTCRETE WALL

3.3.4 Soil Nailing with Steel Rebar and Shotcrete Wall

Soil nailing is practical and proven technique used in constricting excavations and stabilizing slopes by reinforcing the ground in-situ with relatively small, fully bonded inclusions, usually steel bars or a strand. These are introduced into the soil mass, the face of which has been locally stabilize by sprayed concrete (shotcrete), and act to produce a zone of reinforced ground. This zone then performed as a homogeneous and resistant unit to support the unreinforced ground behind. Sequence of construction:-

- The initial depth of the excavation lift is usually between 1 - 2m.
- Drill holes are drilled to a specified length, diameter, inclination, and horizontal spacing by using rotary drilling machine.
- Inserting nail bars and filled with cement grout then fixed reinforcement wire mesh of shotcrete.
- Shotcrete concrete sprayed on the face.
- Excavated the next level.

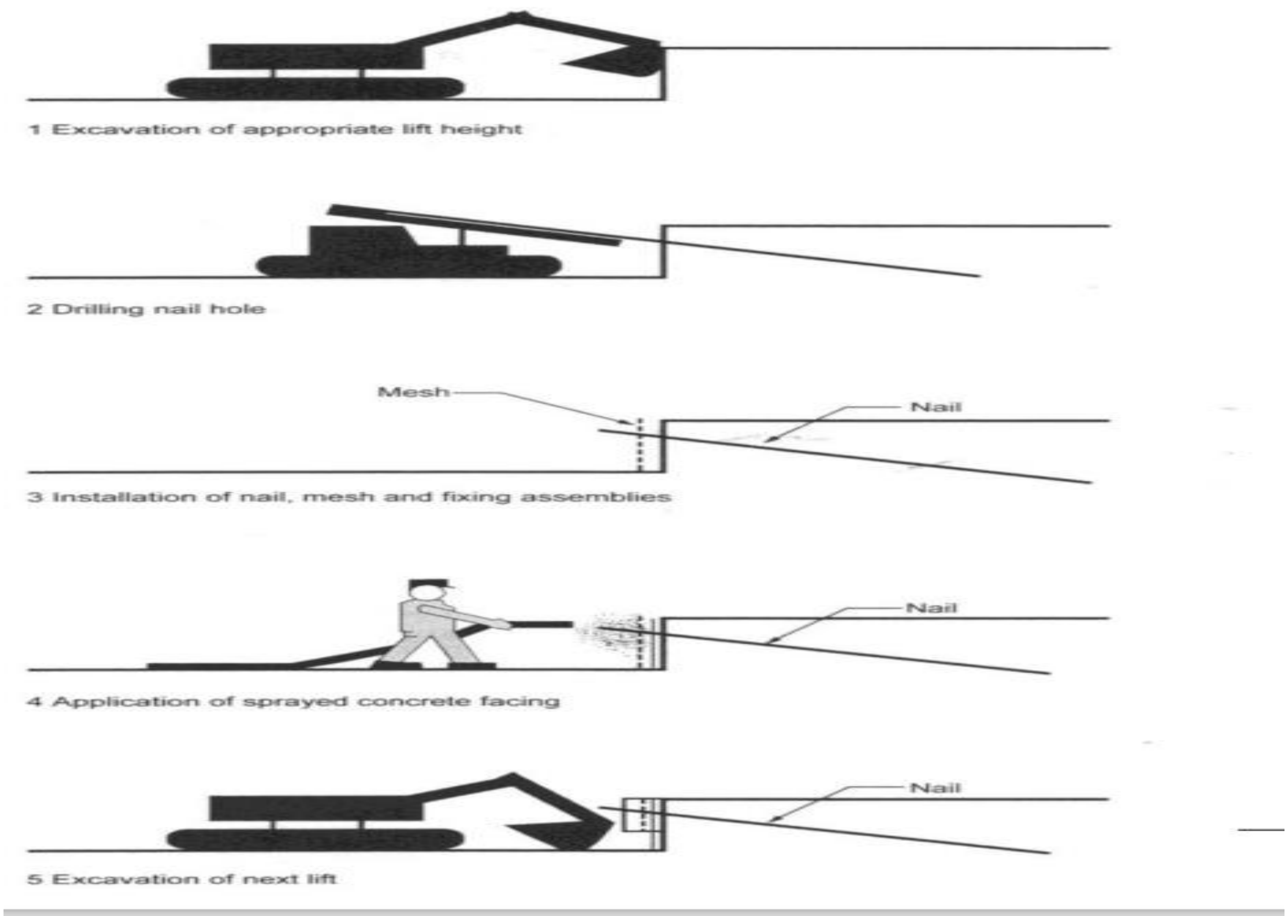


FIGURE 3-22 SEQUENCE OF CONSTRUCTION

Figure 3-23 View of Soil Nailing Wall


YOUTH CENTER

FIGURE 3-24 REINFORCEMENT SOIL NAILING WALL

3.3.5 Combination of Soil Nailing with the Contiguous Pile

Soil nailing wall used to minimized the excavation depth or replace the collapsed pile after excavation work that has flexibility to control the surrounding to failure. The adjacent structure and soil condition are the basic factor to select soil nailing with contiguous pile wall system.

FIGURE 3-25 VIEW OF TOP SOIL NAILING WITH CONTIGUOUS PILE WALL SYSTEM

Figure 3-26 View of Soil Nailing Wall Between Contiguous Pile Wall Systems

3.4 Tangent pile

Tangent pile walls are constructed in such a way that there is no connection between piles. Tangent pile walls offer a larger lateral resistance than contiguous walls, and anchor placement is simpler because weeps are not required. It has greater alignment flexibility and it could be inserted on challenging ground (such as boulders). One problem with this construction is that it uses more concrete. Since the water flows freely between the piles, no additional drainage is required. The walls of the tangent pile wall system must be stabilized. The soil must be able to span any small gaps between the piles. Tangent piles are therefore not advised for cohesion-less sand soils.

FIGURE 3-27 TANGENT PILE WALL

CHAPTER FOUR

4. DATA COLLECTION AND PARAMETER STUDY

4.1 Introduction

Deep excavations in the urban areas of Ethiopia are usually close to adjacent infrastructure, e.g., buildings, deep foundations, and underground roads, which are sensitive to excavation-induced ground movement. In such conditions, the design and construction of deep excavations must consider the possible adverse effect of the excavation and control it within a permissible level.

Deep excavations in Addis Ababa soil are challenging and affected by a few components such as geological conditions, retaining structures, construction methodology, and workmanship. Finite element analysis is a useful tool to investigate the excavation behavior, but its capacity to replicate the observed performance in the field needs to be evaluated through calibration with the field data. Moreover, a few undetermined parameters can be assessed through parametric studies.

This chapter will present the geological and geotechnical engineering properties derived from the soil investigation on the site. Field tests are also described here. An idealized excavation geometry is adopted in this chapter to conduct the parametric studies. Some useful findings and conclusions are generated for practical applications in the design and construction of deep excavations. Overall, the validation of the FEM model using LEM methods adds reliability to the findings and conclusions drawn from the FEM analysis. It assures that the FEM model can accurately simulate and predict the behavior of the system, thus increasing the reliability of the results obtained.

4.2 Geological and geotechnical engineering properties of study areas

In this study, three sites were selected for investigation: one in the Sengatera area of Mexico, the second in the Bole Matemia area, and the third in Kera. The geotechnical engineering properties specific to each site, which were determined through sub-soil investigations, will be described in the following sections.

4.2.1 Generalized geotechnical properties of the site in Mexico Sengatera area

This section provides an overview of the general geotechnical properties of the site for the construction of the 4B+G+30 headquarters building of Zemen Bank S.C. in Mexico. The information presented here is primarily derived from the sub-soil investigation report conducted in the project area.

According to the site investigation report, the subsurface soil is top area covered by medium stiff silty clay. Underlying this layer dense to very dense gravely silty sand with highly fragmented basalt, Very Stiff red sandy clay silt (decomposed rock) and highly fractured to fragmented Basalt. The ground water observed below the construction level and the soil parameters attached in the following table.

TABLE 4-1 A SUMMARY OF ENGINEERING PROPERTIES SOIL PARAMETERS FOR ZEMEN BANK S.C.

Parameter	*Name*	*Unit*	*Clayey Soil - Silty Clay, Medium stiff consistency*	*Strong, highly fractured basalt with some fine materials*	*Dense gravelly silty sand with some highly weathered core stone basalt*	*Clayey Soil - Silty Clay, Very stiff consistency*
Drainage type			Drained	Drained	Drained	Drained
Soil Model			Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Soil unit weight	γ_{unsat}	kN/m³	16.0	19.0	17.5	16.5
	γ_{sat}	kN/m³	18.0	23.0	21.0	19.5
Young's modulus	E	kN/m²	35000.0	120000.0	100000.0	42000.0
Shear modulus	G	kN/m²	13500.0	47240.0	40320.0	16800.0
Oedometric Modulos	E_{oed}	kN/m²	47120.0	149900.0	117900.0	50400.0
Poisson's ratio	ν (nu)		0.30	0.25	0.24	0.25
Effective Cohesion	C'_{ref}	kN/m²	25.0	75.0	100.0	30.0
Effective Friction angle	φ (phi)	°	20.0	34	40.0	24.0

4.2.2 Generalized geotechnical properties of the site in Bole matemia area

This section analyses the generalized geotechnical properties of a site in Bole for Flintstone Homes' Bole Classic 4B+G+23 residence building. The information is extracted mainly from the sub-soil investigation report of the project area.

According to the site investigation report, the subsurface soil is top area covered by Firm consistency Silty Clay, Underlying this layer Stiff consistency Silty Clay, and Very Stiff consistency Silty Clay,. The ground water observed below the construction level and the soil parameters attached in the following table 4-2.

TABLE 4-2 A SUMMARY OF ENGINEERING PROPERTIES SOIL PARAMETERS FOR FLINTSTONE HOMES'

Parameter	*Name*	*Unit*	*Clayey Soil - Silty Clay, Firm consistency*	*Clayey Soil - Silty Clay, Stiff consistency*	*Clayey Soil - Silty Clay, Very stiff consistency*
Soil Model			Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Soil unit weight	γ_{unsat}	kN/m³	16.5	17.0	18.0
	γ_{sat}	kN/m³	18.0	19.5	21.0
Young's modulus	E	kN/m²	16500.0	28500.0	42000.0
Shear modulus	G	kN/m²	6346.0	11220.0	16800.0
Oedometric Modulos	E_{oed}	kN/m²	22210.0	35610.0	50400.0
Poisson's ratio	ν (nu)		0.30	0.27	0.25
Effective Cohesion	C'_{ref}	kN/m²	15.0	25.0	30.0
Friction angle	φ (phi)	°	22.0	23.0	28.0

4.2.3 Generalized geotechnical properties of the site in Kera Bulgaria area

This section reviews the generalized geotechnical properties of the site in Kera Bulgaria 2B+G+13 Yeub Hagere Group S.C project. The information is extracted mainly from the sub-soil investigation report of the project area that attached in table 4-3.

TABLE 4-3 A SUMMARY OF ENGINEERING PROPERTIES SOIL PARAMETERS FOR YEWUB HAGERE PROJECT

Parameter Soil Model	*Name*	*Unit*	*Medium dense sand with minor gravel* Mohr-Coulomb	*Medium strong weathered fractured Basalt* Mohr-Coulomb
Soil unit weight	γ_{unsat}	kN/m³	17.0	20.0
	γ_{sat}	kN/m³	19.0	22.0
Young's modulus	E	kN/m²	35000.0	39000.0
Shear modulus	G	kN/m²	13780.0	15600.0
Oedometric Modulos	E_{oed}	kN/m²	43740.0	46800.0
Poisson's ratio	ν (nu)		0.30	0.27
Effective Cohesion	C'_{ref}	kN/m²	4.0	50.0
Effective Friction angle	φ (phi)	°	34.0	36.0

4.3 Parametric studies of an excavation

Deep excavation is a very complex soil-structure interaction problem, and its performance is affected by some factors such as the ground condition, the excavation geometry, the excavation depth, the type, and stiffness of the retaining system, and the construction method. The excavation is retained by a contiguous pile wall is 10m at the Mexico and Bole area project and the soil nailing wall retaining height is 8m.

4.3.1 Material Models and Input Parameters

The structural components (piles, anchor, and shotcrete) are assumed to be reinforced concrete materials and behave linearly elastic for simplicity, associated with the input parameters attached in the following table 4- 4.

A. CONTIGUOUS PILE WALL PARAMETER

TABLE 4-4 PILE WALL PARAMETER FOR BASE MODEL

Parameter	**Identification**	**Unit**	**Contiguous pile**
Concrete Strength	C_{30}	Mpa	30
Material type			Elastic
	Isotropic		Yes
C30 Concrete Elasticity modulus	Ec	Mpa	31,000.00
Normal Stiffness	EA_2	kN/m	4,867,000.00
Flexural Rigidity	EI	kN m²/m	109,430.00
Diameter	d	m	0.6
Area of Reinforcement for Mexico Zemen bank S.c	As	m²	0.000314
Area of Reinforcement for Bole Flintstone homes'	As	m²	0.000452
Area of Reinforcement for Buligara kera Yewub hagere	As	m²	0.00314
Reinforcement	Grade	-	420

TABLE 4-5 SOIL NAILING WALL PARAMETER FOR BASE PLATE

Parameter	**Identification**		**Shotcrete Wall**	**Soil Nailing**
	Identification number		1	2
	Material type		Elastic	Elastic
	Isotropic		Yes	Yes
Flexural Rigidity	EA_1	kN/m	1576982.1	313526
Normal Stiffness	EI	kN m²/m	294.4	439.96
diameter of nailing drilling	D	m	0	0.13
Thickness of Shotcrete wall	T	m	0.15	-
Weight per length	W	kN/m/m	3.75	0.0

B. GROUND ANCHOR PERIMETER

A ground Anchor modelled by combine nation of node-to-node and an embedded beam. The embedded beam simulates the grouted part of the anchor whereas the node-to node anchor simulates the free length.

TABLE 4-6 GROUND ANCHOR PARAMETER FOR BASE MODEL

Parameter	**Unit**	**Anchor**
Material type		Elastic
EA	kN/m	500000
^{L} spacing for project1,2, 3	M	1.8,1.8 & 1.6
Slope	degree	20

C. SURCHARGE LOAD

TABLE 4-7 SURCHARGE LOAD FOR EACH PROJECTS

Project	**Neighboring structure**	**Load kN /m²**
Mexico Zemen bank S.c	Road	16
Bole Flintstone homes'	G+5 building	50
Buligara kera Yewub hagere	Road	16

4.3.2 Construction sequence

Mexico Zemen Bank project contiguous anchored wall construction has six phases. In the initial phase, the initial stresses are generated. In phase 1, the walls are constructed and the surface loads are activated. In Phase 2, the first 3m of the bulk is excavated without connection of anchors to the wall. In Phase 3, the first Anchor is installed and pre-stressed. Phase 4 involves further excavation to depth of 7m. In phase 5, the second anchor is installed and pre stressed. Phase 6 is a further excavation to the final depth of 10m excavation.

TABLE 4-8 FLINTSTONE HOMES' S.CO CONSTRUCTION PHASE FOR ANCHORED CONTIGUOUS PILES MODEL

Phase	Phase no.	Starts from	Calculation type	Load input
pile Installtion [InitialPhase]	0	N/A	K0 procedure	N/A
First Excavtion [Phase_1]	1	0	Plastic	Staged construction
first Anchor [Phase_2]	2	1	Plastic	Staged construction
second excavation [Phase_3]	3	2	Plastic	Staged construction
Second Anchor [Phase_4]	4	3	Plastic	Staged construction
3rd excavation [Phase_5]	5	4	Plastic	Staged construction
3rd anchor [Phase_6]	6	5	Plastic	Staged construction
last excavtion [Phase_7]	7	6	Plastic	Staged construction

TABLE 4-9 YEWUB HAGERE S.CO CONSTRUCTION PHASE FOR ANCHORED CONTIGUOUS PILES MODEL

Phase	Phase no.	Starts from	Calculation type	Load input
Initial phase [InitialPhase]	0	N/A	K0 procedure	N/A
First Excavation [Phase_2]	2	0	Plastic	Staged construction
First layer Anchor [Phase_3]	3	2	Plastic	Staged construction
Second Layer Excavation [Phase_4]	4	0	Plastic	Staged construction

TABLE 4-10 ZEMEN BANK SOIL NAILING WALL CONSTRICTION PHASE

Phase	Phase no.	Starts from	Calculation type	Load input
Initial phase [InitialPhase]	0	N/A	K0 procedure	N/A
First Excavation [Phase_1]	1	0	Plastic	Staged construction
First layer soil nailing [Phase_2]	2	1	Plastic	Staged construction
Second Excavation [Phase_3]	3	2	Plastic	Staged construction
Second Layer soil nailing [Phase_4]	4	3	Plastic	Staged construction
3rd Excavation [Phase_5]	5	4	Plastic	Staged construction
3rd Layer Soil nailing [Phase_6]	6	5	Plastic	Staged construction
4th Excavation [Phase_7]	7	6	Plastic	Staged construction

TABLE 4-11 YEWUB HAGERE S.COCONSTRACTIO PASHE SOIL NALING WALL MODEL

Phase	Phase no.	Starts from	Calculation type	Load input
Initial phase [InitialPhase]	0	N/A	K0 procedure	N/A
Wall and Load [Phase_1]	1	0	Plastic	Staged construction
First Excavation [Phase_2]	2	0	Plastic	Staged construction
first Nailing [Phase_3]	3	2	Plastic	Staged construction
Second Nailing [Phase_5]	5	3	Plastic	Staged construction
3rd Excavation [Phase_6]	6	5	Plastic	Staged construction
Final Excavation [Phase_7]	7	6	Plastic	Staged construction

CHAPTER FIVE

5. RESULT AND DISCUSSION

5.1 Introduction

This chapter presents findings from a study on deep excavation performance in terms of stability, and deformation. The study involved numerical modeling of the excavation process using the Mohr-Coulomb (MC) model for three projects in Addis Ababa. The evaluation focused on contiguous pile wall retaining systems and soil-nailing walls. Results were compared with the Limit Equilibrium Method to verify the Finite Element Method. The study also studied the sensitivity of soil parameters and compared the 3D FEM, 2D FEM, and limit equilibrium methods for one project. In general geotechnical Engineering, both Limit Equilibrium Methods and Finite Element Methods (FEM) are commonly used for analyzing soil structures and slopes. The basic difference between LEM and FEM:-

1. Limit Equilibrium Methods:

- Limit equilibrium methods are based on the assumption that soil and rock masses fail along a potential failure surface when the driving forces exceed the resisting forces.
- These methods consider the equilibrium conditions of the soil mass and analyze the stability of slopes or soil structures based on the strength properties of the soil.
- The most commonly used limit equilibrium methods include Bishop's method, Janbu's method, Spencer's method, and Morgenstern-Price method.
- Limit equilibrium methods provide analytical and simplified solutions, and they are relatively easy and quick to perform.

The disadvantages of limit-equilibrium methods include:

- 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 bending moments are possibly uncoservative when more than one support levels are used.
- Construction stage history is totally ignored.

Concluding, it always adds value to perform a limit-equilibrium analysis that can provide a backbone against which to compare a more rigorous non-linear elastoplastic solution.

2. Finite Element Methods (FEM):

- FEM is a numerical analysis technique that divides the problem domain into small elements and analyzes the behavior of each element individually.
- FEM considers the mechanical properties (stiffness, strength, etc.) of the soil, as well as boundary conditions, to predict the behavior and stability of soil structures and slopes.
- FEM allows for the accurate modeling of complex geometries and soil conditions, and it can provide more detailed and refined solutions compared to limit equilibrium methods.
- FEM requires the use of specialized software and often involves more sophisticated and time-consuming analyses.

Finally, a comparative discussion was undertaken between contiguous pile walls and soil nailing walls concerning several factors relevant to the Addis Ababa context. The factors considered included stability, deformation, cost-effectiveness, and constructability. This study provides essential insights into the design and analysis of deep excavation projects in a challenging urban construction environment such as Addis Ababa.

5.2 Bending moment and wall deflection analysis result

This section involves a comparative analysis of two numerical modeling techniques used to estimate bending moments and wall deflections in deep excavations. The two techniques considered are the Limit Equilibrium Method (LEM) and the 2D/3D Finite Element Method (FEM).

The LEM is a traditional analytical method used in geotechnical engineering that assumes equilibrium conditions and simplified assumptions about soil behavior. On the other hand, the 2D/3D FEM is a numerical method that considers the complex behavior of both soil and structures using numerical discretization techniques.

To evaluate the accuracy and reliability of each method, a comparative analysis is conducted by applying both methods to the same excavation project. The results are then compared and analyzed in detail, taking into account the predicted bending moments and wall deflections.

Furthermore, sensitivity analyses are performed to assess the influence of various soil parameters on the predicted response. The outcomes of these analyses are examined and discussed in detail, providing valuable insights into the limitations and advantages of each method.

Overall, this comparative analysis enables a more understanding of the strengths and weaknesses of each method in estimating bending moments and wall deflections in deep excavation projects. Thus, it provides a useful reference for future design and analysis work in this field.

5.2.1 Contiguous pile wall and soil nailing wall bending moment & wall deflections in Mexico Sengatera area (Project 1)

A. Contiguous pile wall bending moment and wall deflections in Mexico sengatera area

The bending moment values of the contiguous pile wall were analyzed using both the limit equilibrium method (LEM) and PLAXIS 2D finite element method (FEM), as shown in Fig. 5-1. And Fig. 5-2 the maximum bending moment obtained from LEM was 68.68 kNm/m, while PLAXIS 2D generated a value of 50.45 kNm/m. This indicates that the maximum bending moment value obtained from LEM is 26.54% higher than the one obtained from PLAXIS 2D.

This discrepancy in the maximum bending moment values can be attributed to various factors, including differences in assumptions, boundary conditions, modeling techniques, and material properties in LEM and PLAXIS 2D. It is important to note that LEM is a simplified analytical method, while PLAXIS 2D employs finite element analysis for a more comprehensive modeling of the structural response.

Other references in the field have also noted variations in results when comparing LEM and FEM. For instance, a study by Smith et conducted a comparative analysis of LEM and FEM for analyzing retaining walls. (51) They found that while LEM provided reasonable estimates for minimum bending moments, FEM yielded more accurate results for maximum bending moments.

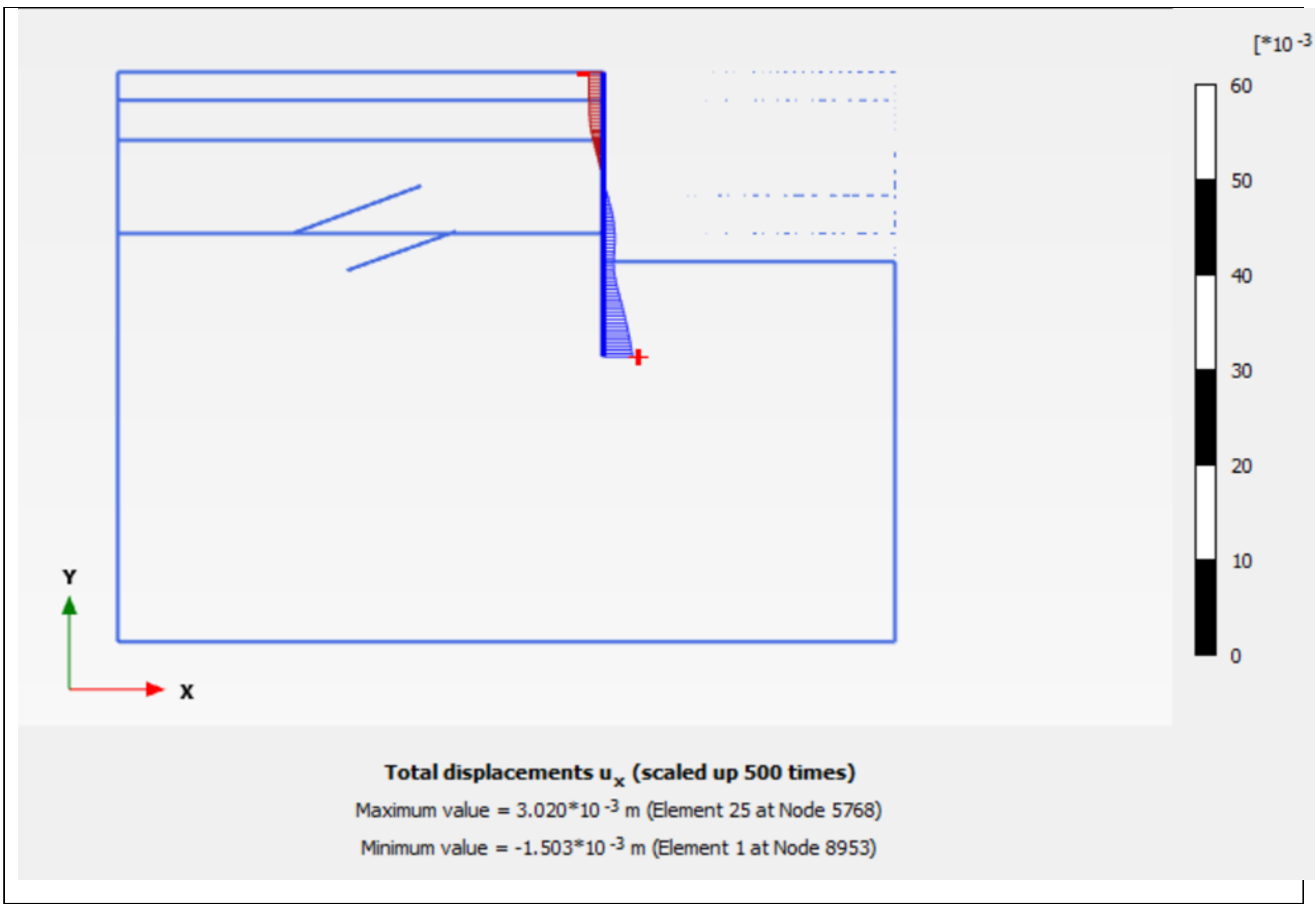
Similarly, a research by Chen et al. investigated the behavior of pile-supported embankments using both LEM and FEM. They observed that LEM tends to overestimate maximum bending moments compared to FEM results, emphasizing the need for cautious interpretations of LEM findings. (52)

Based on the provided information, it appears that the horizontal displacements of a contiguous pile wall towards the excavation have been analyzed using both the limit equilibrium calculation and the PLAXIS 2D software. The height of the retained material is specified as 10m.

The displacement values obtained from these two methods are compared in Fig. 5-1 and Fig. 5-2. The limit equilibrium calculation a displacement value of 3.5mm, while PLAXIS 2D gives a value of 3.02mm. The fact that both methods similar values, with only a small difference of 0.48mm, indicate agreement between the two approaches.

This consistency in results suggests that the displacement predictions obtained from the limit equilibrium calculation and PLAXIS 2D are in good agreement. Both methods have been used to determine the horizontal displacements of the contiguous pile wall towards the excavation for the given height of retained material.

In conclusion, based on the provided information, both the moment limit equilibrium calculation and PLAXIS 2D analysis indicate a similar displacement value of around 3.02-3.5mm for the horizontal displacements of the contiguous pile wall towards the excavation for a retained material height of 10m.



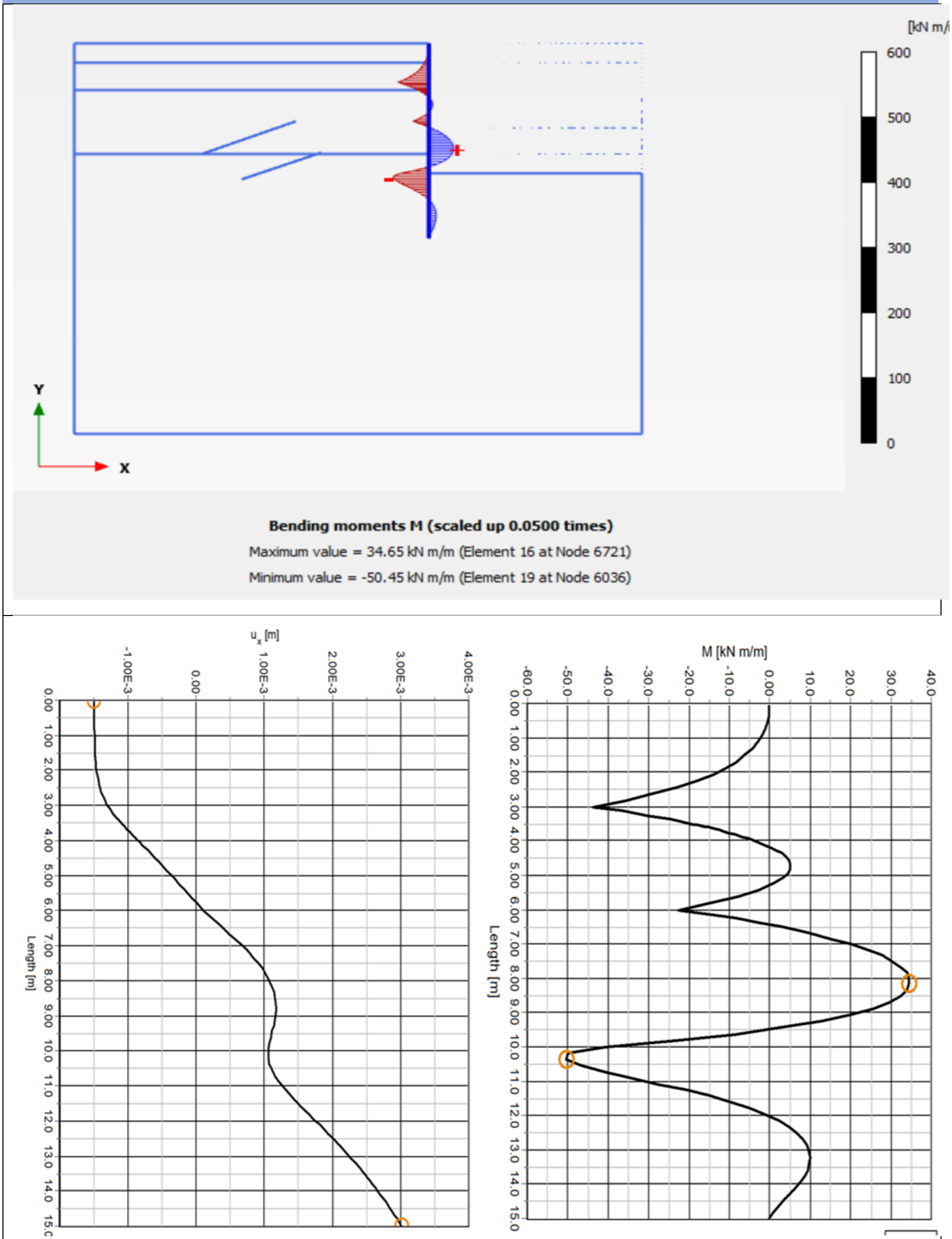


FIGURE 5-1 ZEMEN BANK PLAXIS 2D ANCHORED PILE DISPLACEMENT AND BENDING MOMENT DIAGRAM RESULT

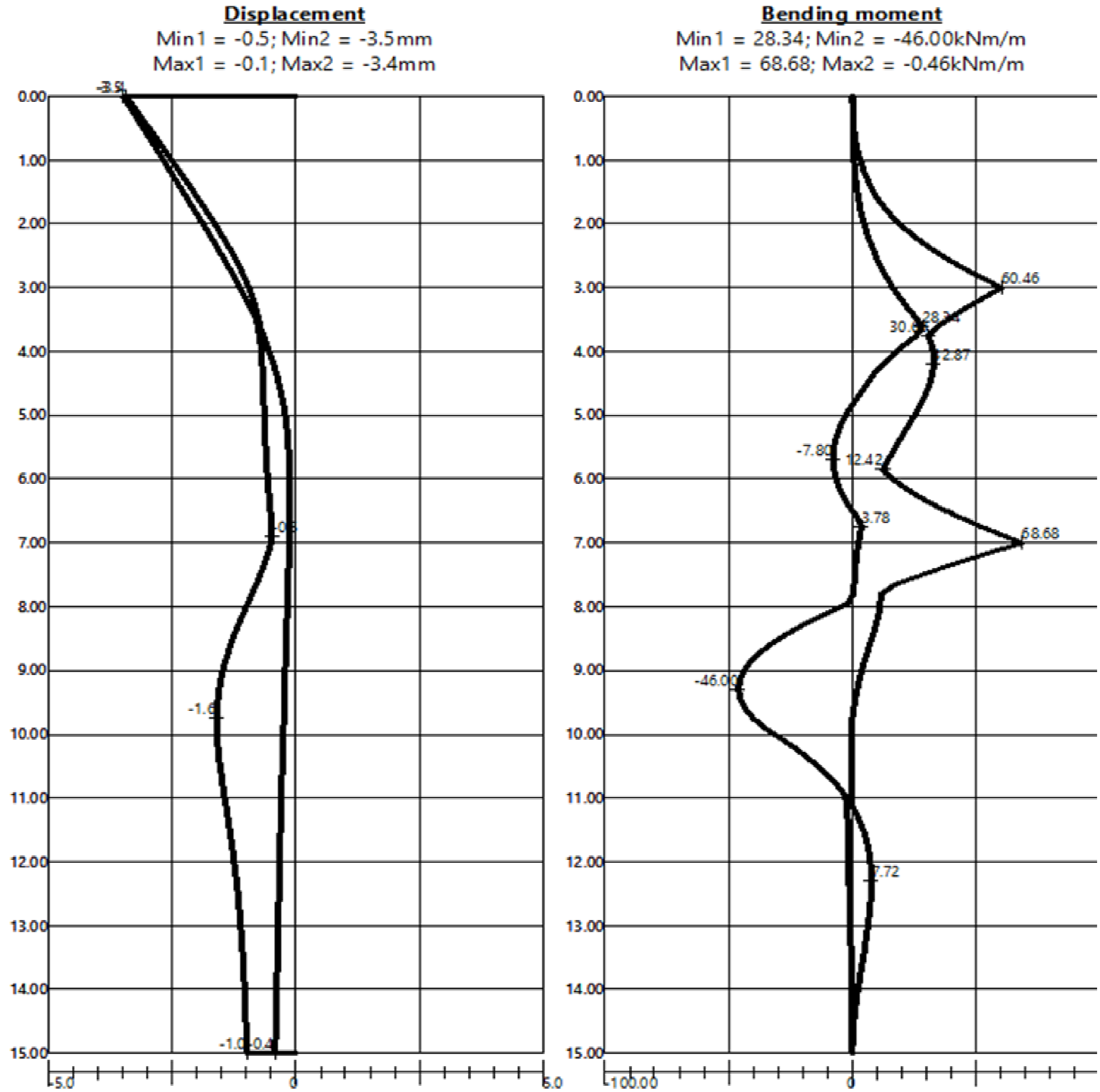


FIGURE 5-2 ZEMEN BANK ANCHORED PILE LIMIT EQUILIBRIUM DISPLACMENT AND BENDING MOMENT DIAGRAM RESULT

B. Soil Nailing wall bending moment and wall deflections in Mexico sengatera area

The bending moment values for the soil nailing wall in this project were evaluated using both the limit equilibrium method (LEM) and PLAXIS 2D finite element method (FEM) in fig 5.3 and fig 5.4. LEM calculated a bending moment of 18.73 kNm/m, while PLAXIS 2D yielded a value of 14.39 kNm/m. This indicates that the bending moment obtained from LEM is greater than the PLAXIS 2D result by 23.17% in the same site conditions and soil type.

This significant difference in the bending moment values can be attributed to several factors. LEM is a simplified analytical method that makes certain assumptions about the behavior of the soil and structure, which may not fully capture the complex interaction between the soil and the soil nails. On the other hand, PLAXIS 2D utilizes finite element analysis to model the soil-structure interaction more carefully, taking into account the non-linear behavior of the soil and the actual geometry of the soil nails.

Other studies in the field have also reported variations in bending moment results between LEM and FEM for soil nailing walls. For example, a study by Wang et al. (2015) conducted a comparative analysis of LEM and FEM for soil nailed slopes. (51) They found that LEM tends to overestimate the bending moments compared to FEM results. Similarly, a research by Lu et al. investigated the behavior of soil nail walls using both LEM and FEM. (52) They observed that LEM can provide conservative estimates of bending moments, in comparison to FEM results which tended to be more accurate.

In summary, the comparison of bending moment values obtained from LEM and PLAXIS 2D for the soil nailing wall in this project reveals a significant difference, with LEM predicting a bending moment that is 23.17% greater than the PLAXIS 2D result. These findings align with other studies in the field, highlighting the importance of considering the limitations and assumptions associated with each method when analyzing the behavior of soil nailed structures.

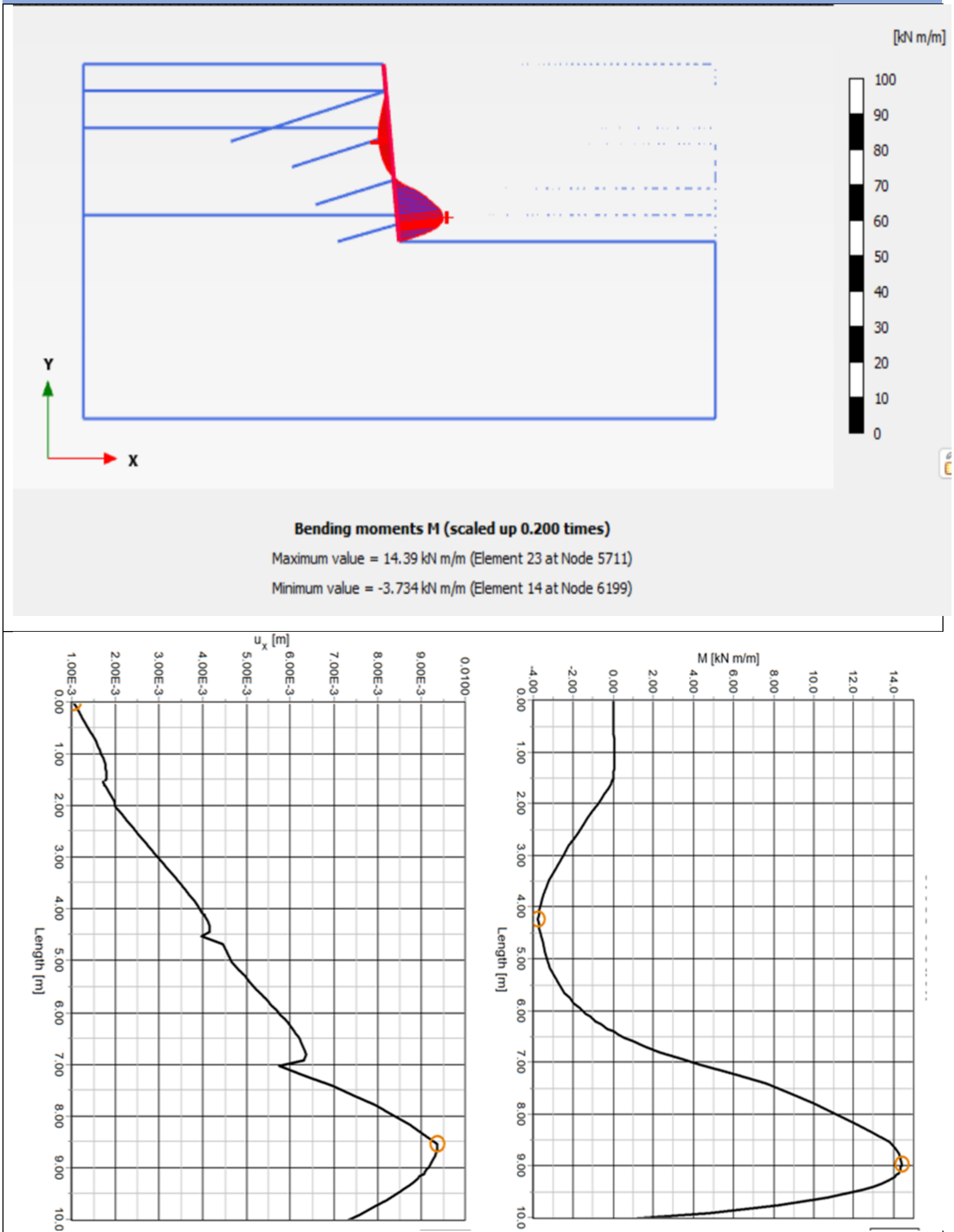


FIGURE 5-3 ZEMEN BANK SOIL NAILING WALL PLAXIS 2D DISPLACMENT AND BENDING MOMENTS DIAGRAM

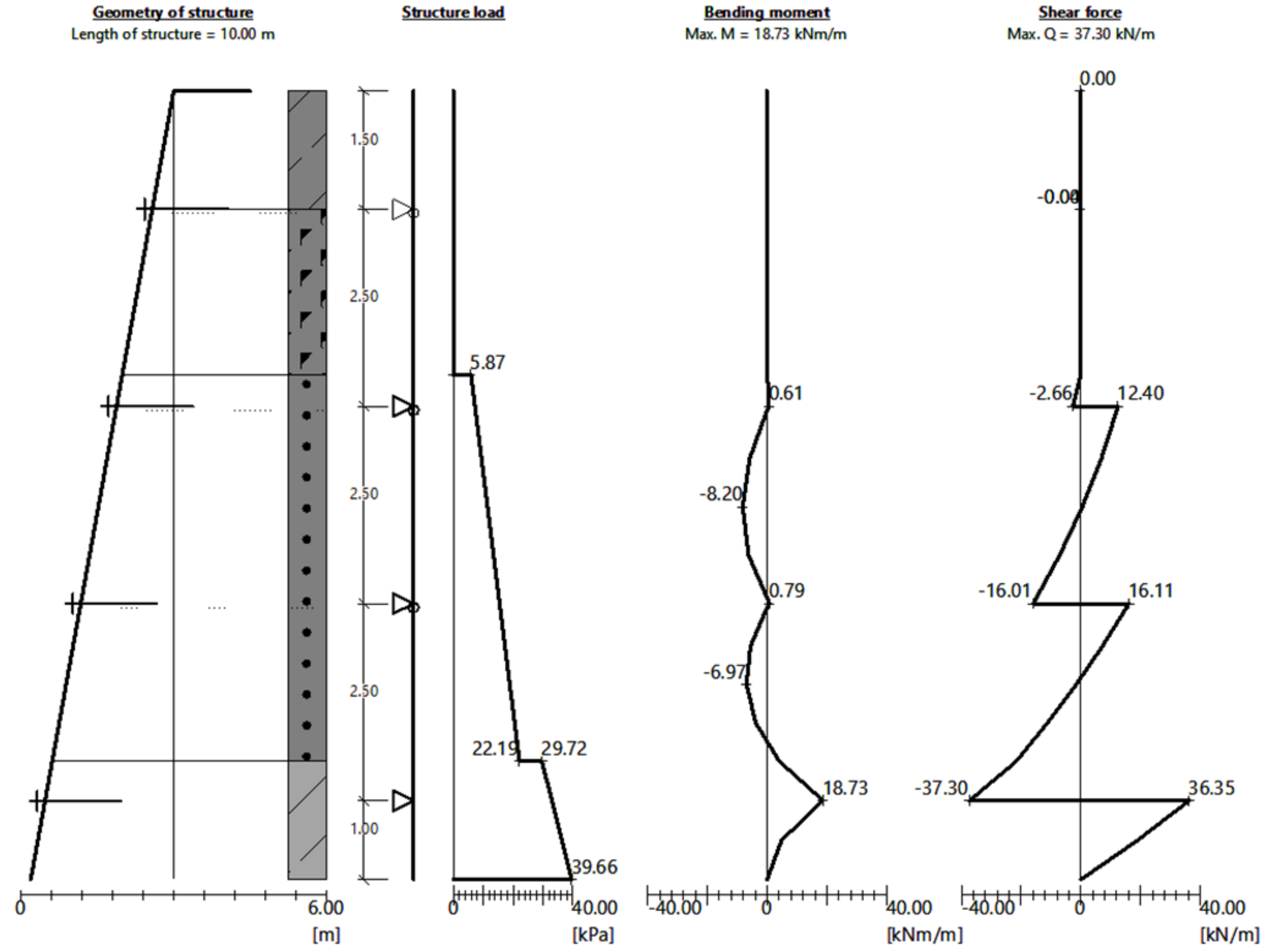


FIGURE 5-4 ZEMEN BANK SOIL NAILING LIMIT EQULIBREM BENDING DIAGRAM RESULT

5.2.2 Contiguous pile wall and soil nailing wall nailing bending moment and wall deflections in Bole Matemia area (Project 2)

A. Contiguous Pile wall bending moment and wall deflections in Bole Matemia area

Figure 5-5 and Figure 5-6 presents the bending moment of the contiguous pile wall for both the limit equilibrium method (LEM) and the 2D finite element method (FEM). The maximum bending moment values obtained from LEM and PLAXIS 2D are 143.5 kNm/m and 127.1 kNm/m, respectively. This indicates that LEM predicts a maximum bending moment 11.42% higher than 2D FEM.

The soil type for this project is identified as **silty clay**, which exhibits a soft consistency near the top but becomes stiffer towards the bottom of the continuous pile wall structure. It is important to consider the behavior and properties of the soil in the analysis and design of the structure.

Considering the same soil conditions, previous research studies have also reported variations in bending moment results between LEM and FEM for pile walls in similar soil types. For instance, a study by Chen et al. investigated the behavior of pile walls in **silty clay** using both LEM and FEM. They found that LEM tends to overestimate the maximum bending moment compared to FEM results.

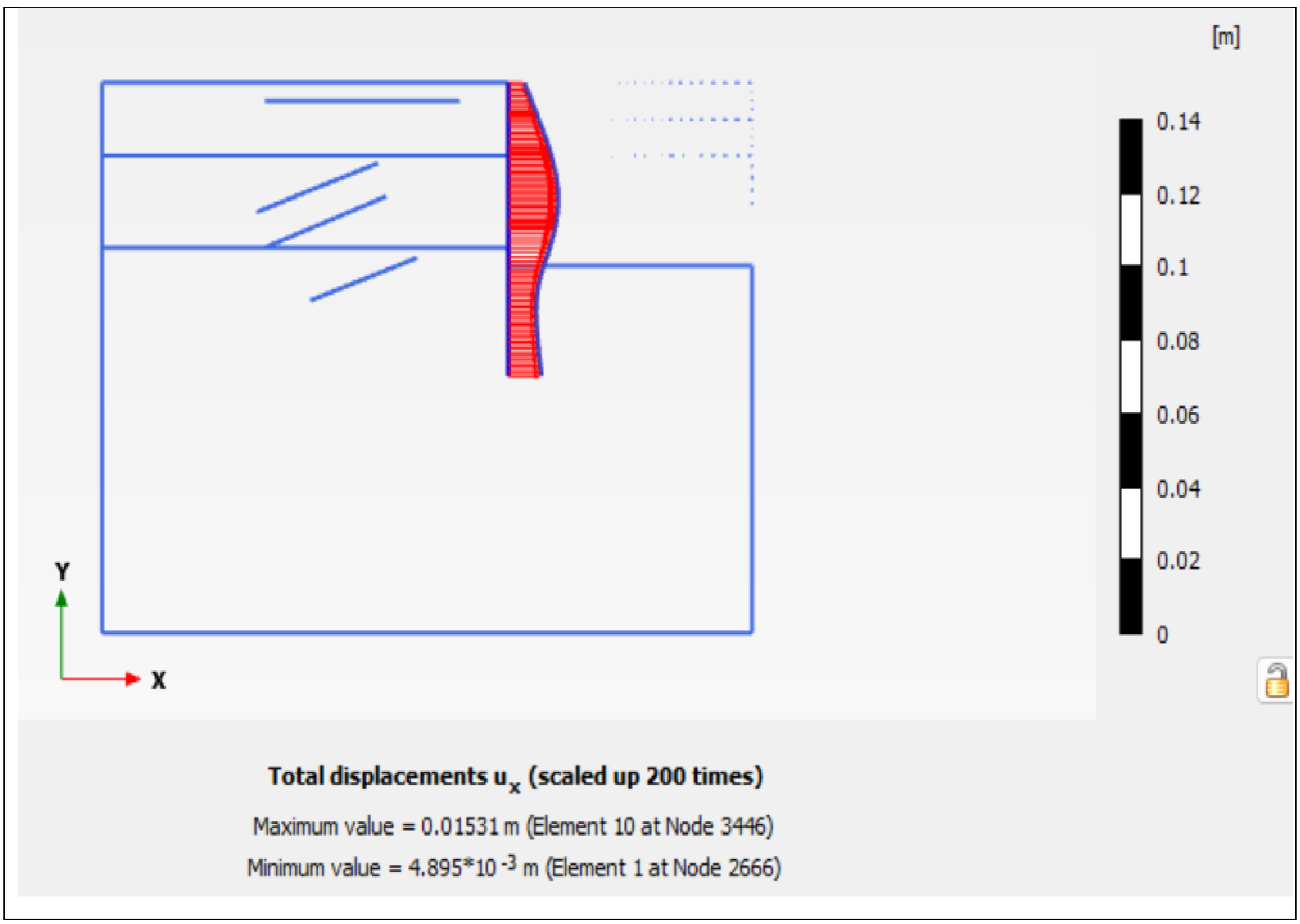
This can be attributed to the simplifications and assumptions made in LEM, which may not fully capture the complex behavior of the soil and structure interaction. (55)

Another research study by Zhao et al. examined the performance of contiguous pile walls in silty clay using LEM and FEM. Their findings also demonstrated that LEM predicted higher bending moments compared to FEM results. They attributed this difference to the incorporation of soil nonlinearity and the actual geometry of the piles in the FEM analysis, which provides a more accurate representation of the structural response. (56)

In summary, the comparison of bending moment values obtained from LEM and PLAXIS 2D for the contiguous pile wall in this project, under the influence of silty clay soil conditions, reveals a 11.42% higher maximum bending moment in LEM. This trend aligns with other research studies in similar soil conditions, emphasizing the need to consider the limitations and assumptions associated with each method when analyzing and designing pile walls.

The figure 5-5 and figure 5-6 represents the horizontal displacements of a contiguous pile wall project as it moves towards the excavation. The height of the retained material in this case is 10 meters.

According to the limit equilibrium calculation for displacements, the obtained value is 15.7mm. On the other hand, the PLAXIS 2D analysis software provides a value of 15.3mm. It is important to note that both methods yield the equivalent result, indicating a consistency in the obtained value.



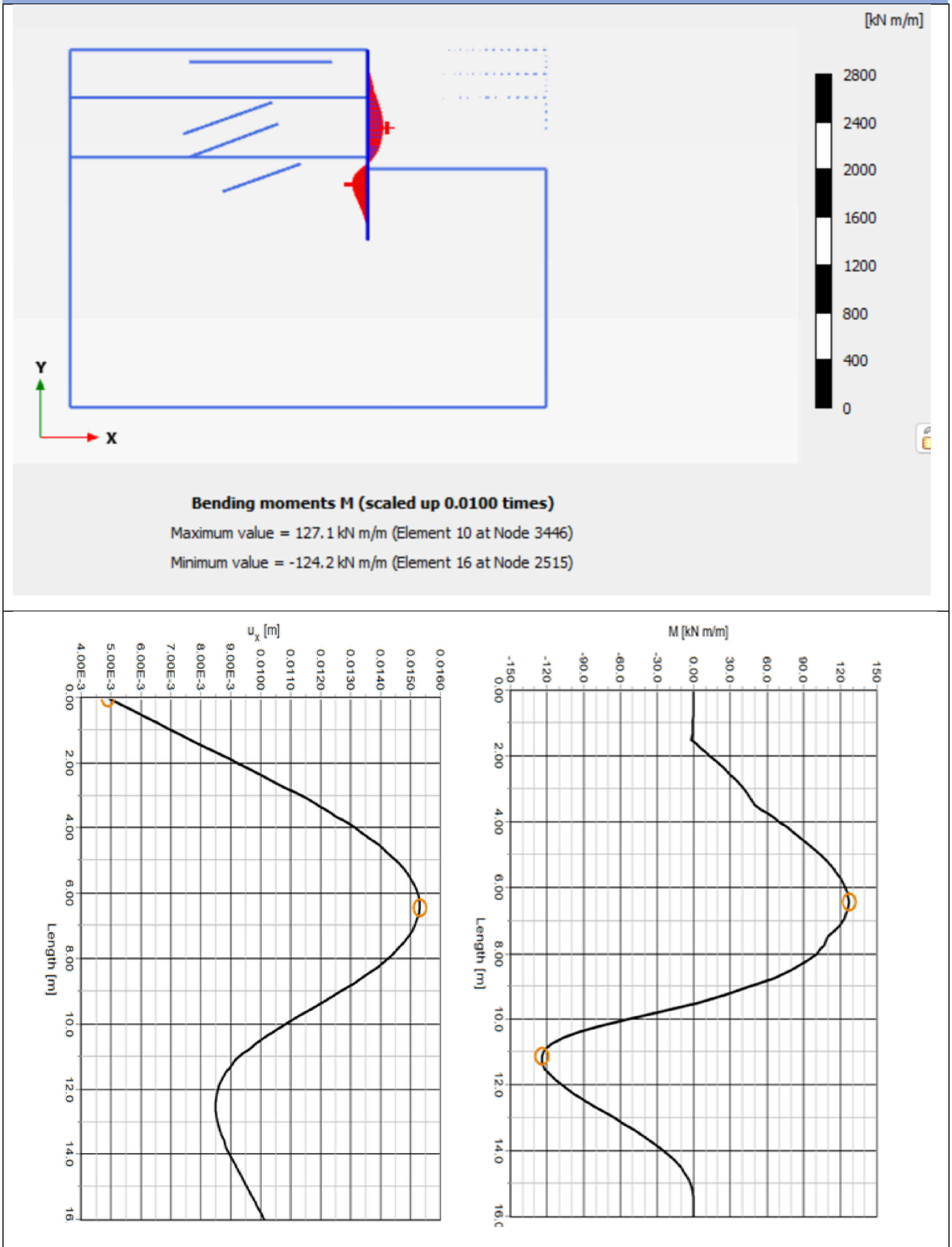


FIGURE 5-5 FILINTSTONE ENGINEERING S.CO PLAXIS 2D ANCHORED PILE DECPLACMENT AND BENDING MOMENT DIAGRAM RESULT

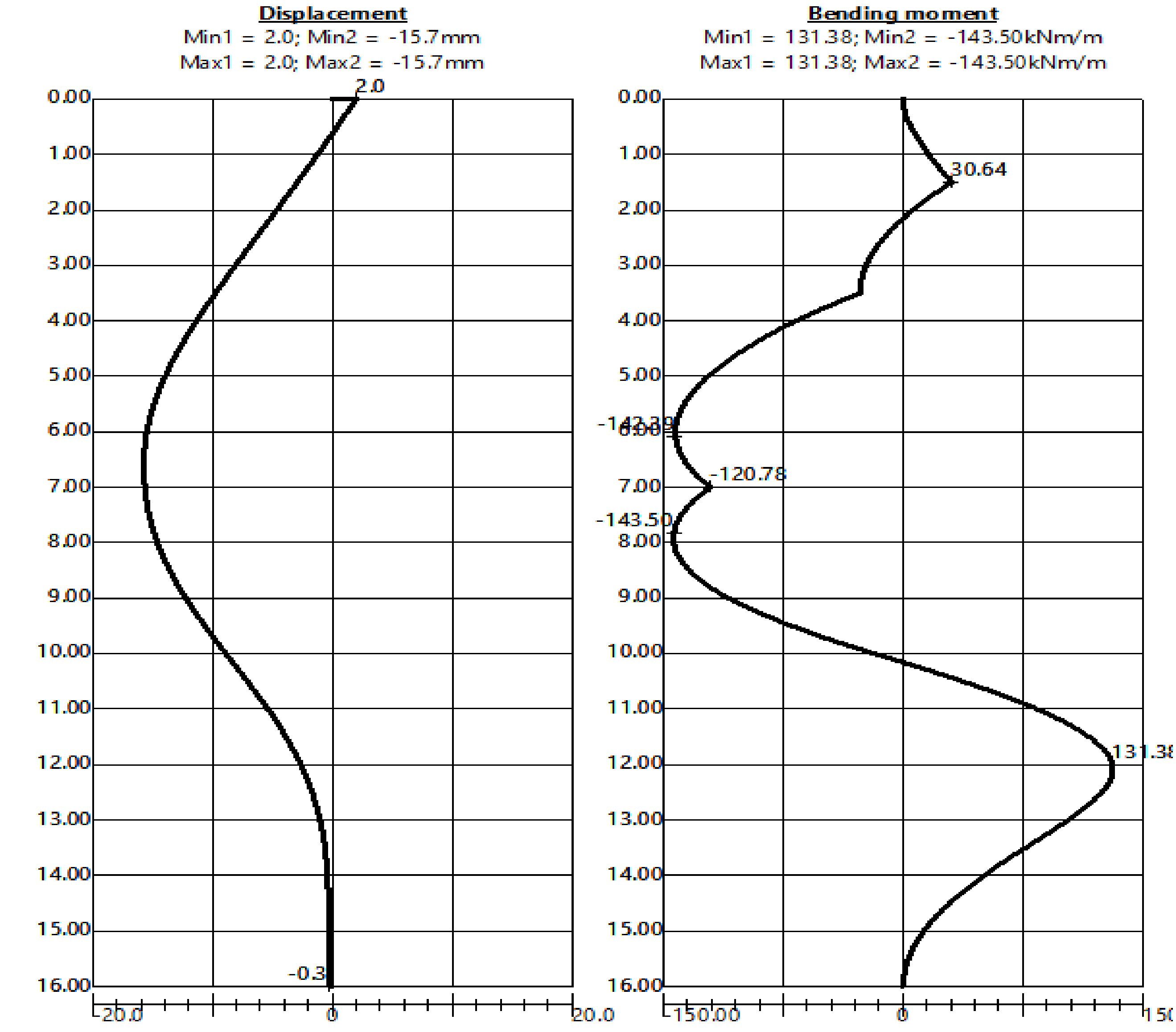


FIGURE 5-6 FLINTSTONE ENGINEERING S.CO- LIMIT EQULIBREM ANCHORED CONTIGUOUS PILE LATERAL DISPLACEMENT AND BENDING MOMENT DIAGRAM RESULT

B. Soil Nailing wall bending moment and wall deflections in Bole Material area

Geometry

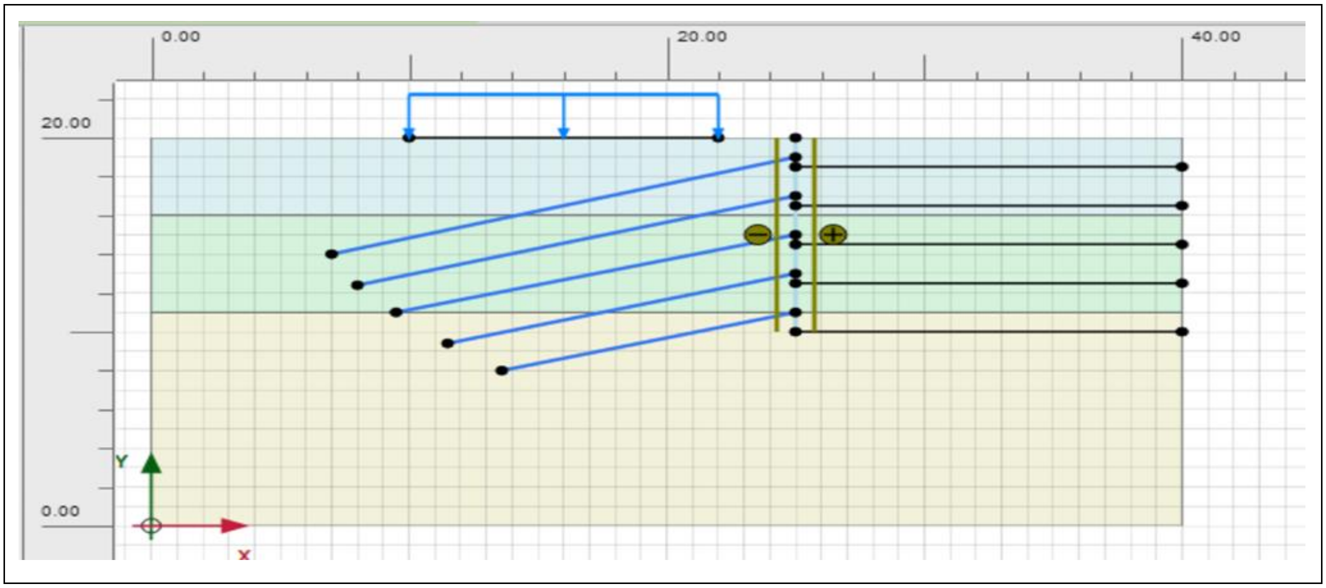
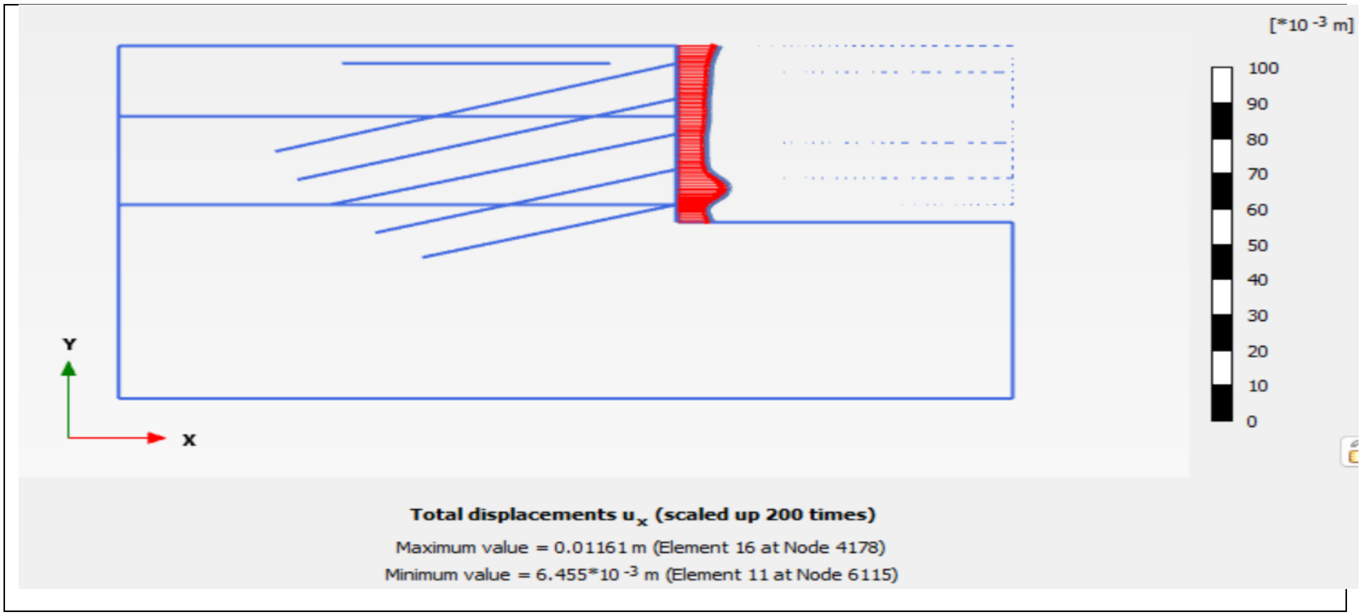


FIGURE 5-7 FLINTSTONE ENGINEERING S.CO SOIL NAILING WALL GEOMETRY

In the evaluation of the bending moment values for the soil nailing wall in this particular project, it is imperative to consider the soil composition. The top layer consists of soft silty clay, while the bottom layer is comprised of Stiff Stity clay. To ensure accuracy, both the limit equilibrium method (LEM) and the PLAXIS 2D finite element method (FEM) were working.

A bending moment of 17.23 kNm/m was estimated using the LEM. In contrast, a value of 12.12 kNm/m was obtained using the PLAXIS 2D methods. It is important to note that the site conditions and soil type under which these findings were obtained were the same. It follows that it is obvious that the bending moment obtained from the LEM exceeds the PLAXIS 2D result by an impressive 23.85%.



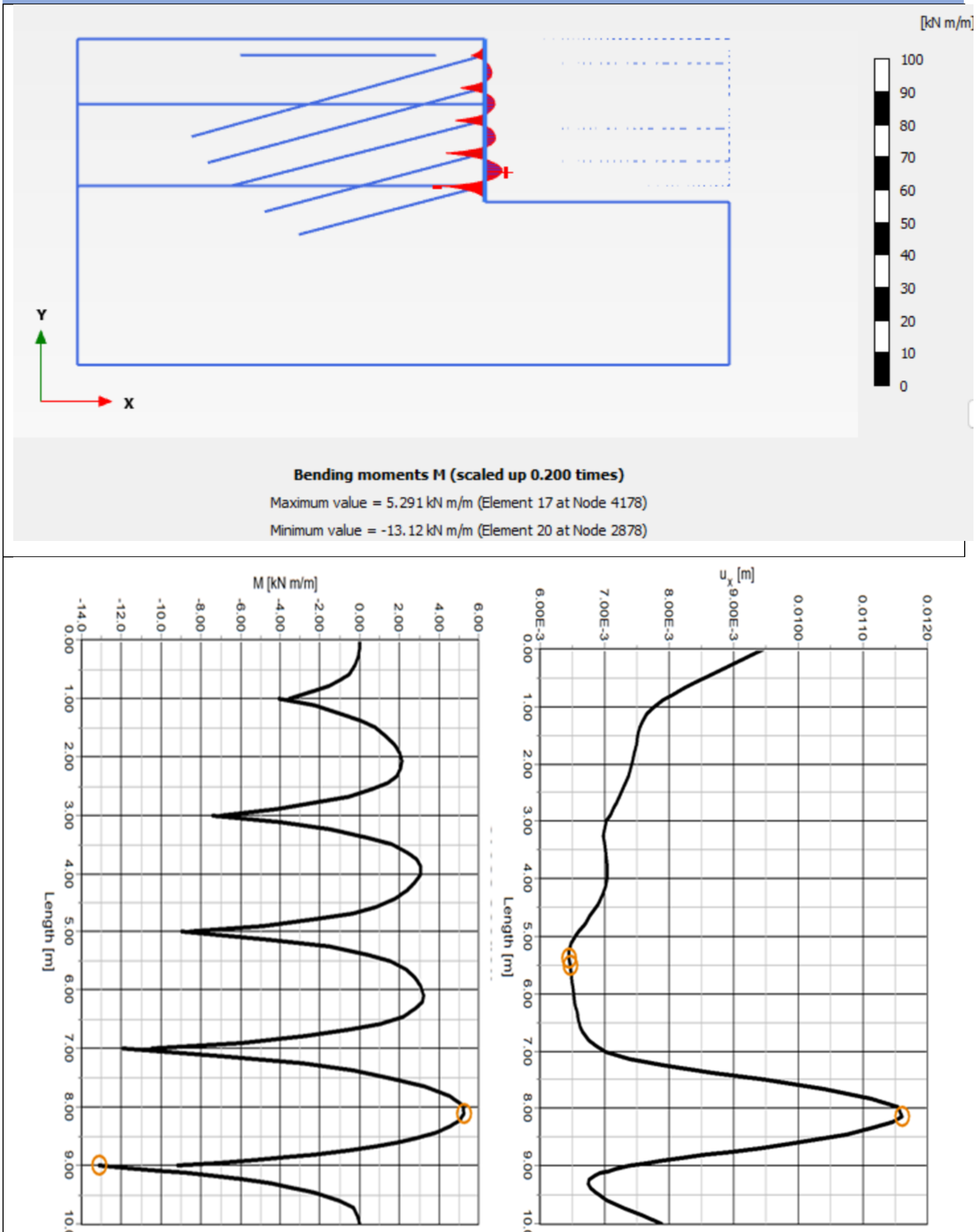


FIGURE 5-8 FLINTSTONE ENGINEERING S.CO PLAXIS 2D SOIL NAILING WALL DISPLACMENT AND BENDING MOMENTS DIAGRAM RESULT

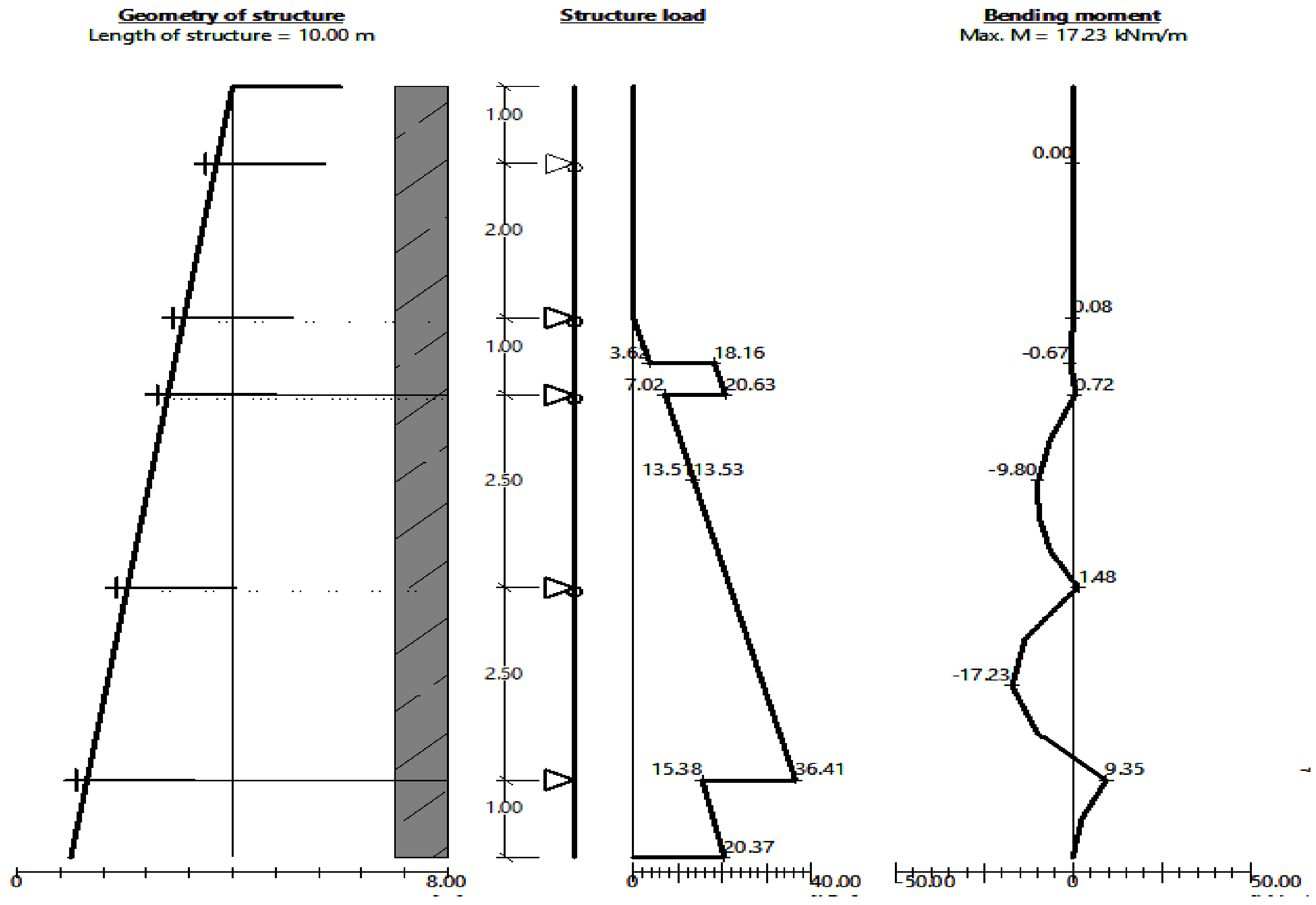


FIGURE 5-9 FLINTSTONE ENGINEERING S.CO LIMIT EQULIBREM SOIL NAILING WALL BENDING MOMENT RESULT

5.2.3 Contiguous pile wall and soil nailing bending moment and wall deflections in Bulgaria Kera area (Project 3)

A. Contiguous pile wall bending moment and wall deflections in Bulgaria Kera area

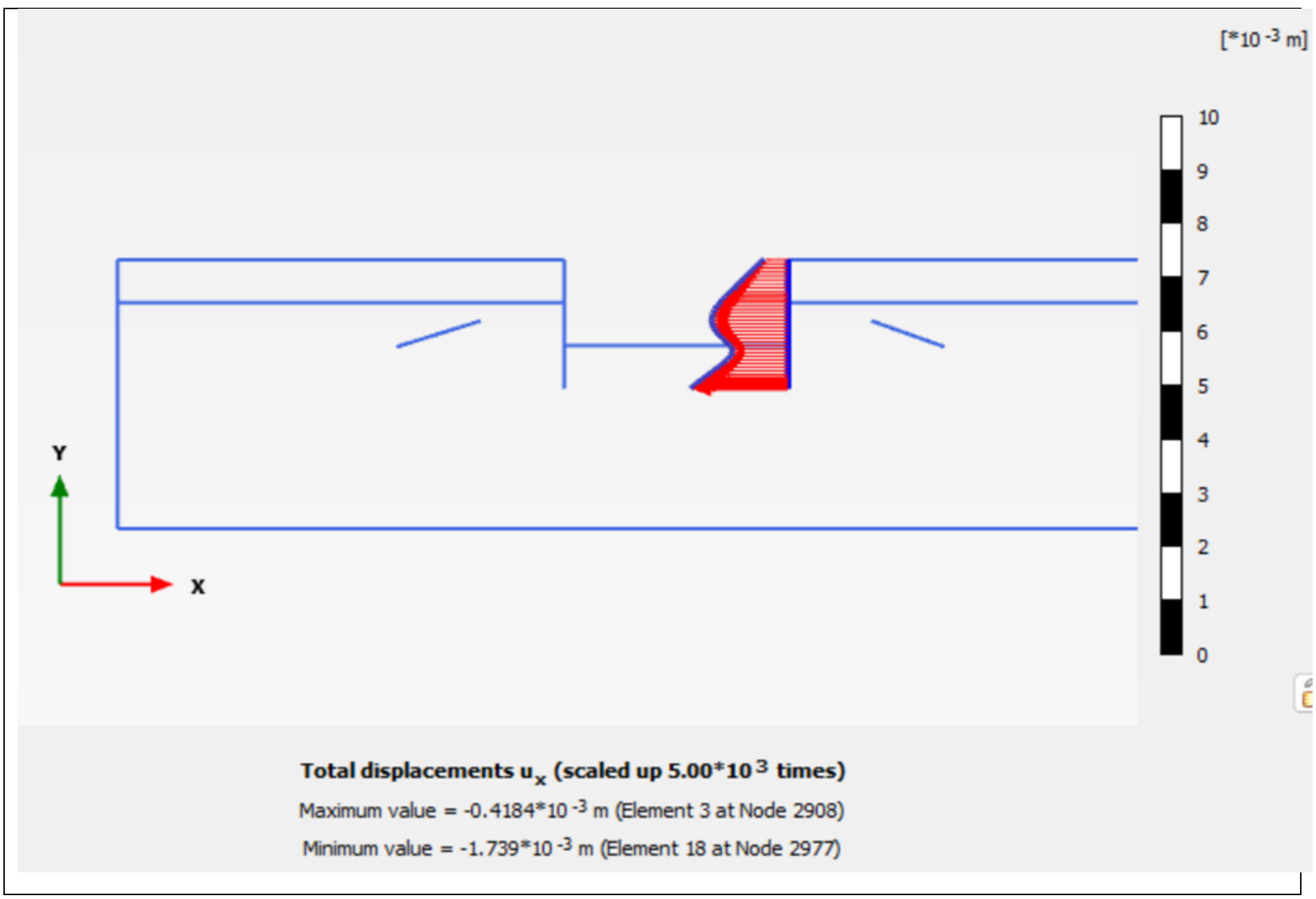
In the present study, the bending moment of a contiguous pile wall is analyzed using different methods and software. The obtained results were compared to evaluate the accuracy and reliability of each approach.

Figure 5-10, 12, 13 presents the bending moment of the project's contiguous pile wall. The bending moment values obtained from three different calculation methods were considered for comparison.

The first method used was the bending moment determined from the limit equilibrium calculation, which yielded a value of 55.19 kNm/m. The second method utilized was the PLAXIS 2D software, which resulted in a bending moment value of 44.74 kNm/m. PLAXIS 2D is a widely used finite element software that enables the modeling and analysis of various geotechnical problems.

Lastly, the PLAXIS 3D software was employed to determine the bending moment, yielding a value of 29.92 kNm/m. PLAXIS 3D extends the capabilities of PLAXIS 2D by allowing the analysis of three-dimensional geotechnical structures.

Comparing the results obtained from these different methods, it is evident that the bending moment obtained from the limit equilibrium calculation (55.19 kNm/m) exceeds the values obtained from both PLAXIS 2D (44.74 kNm/m) and PLAXIS 3D (29.92 kNm/m). Furthermore, the bending moment obtained from PLAXIS 2D is greater than that obtained from PLAXIS 3D.



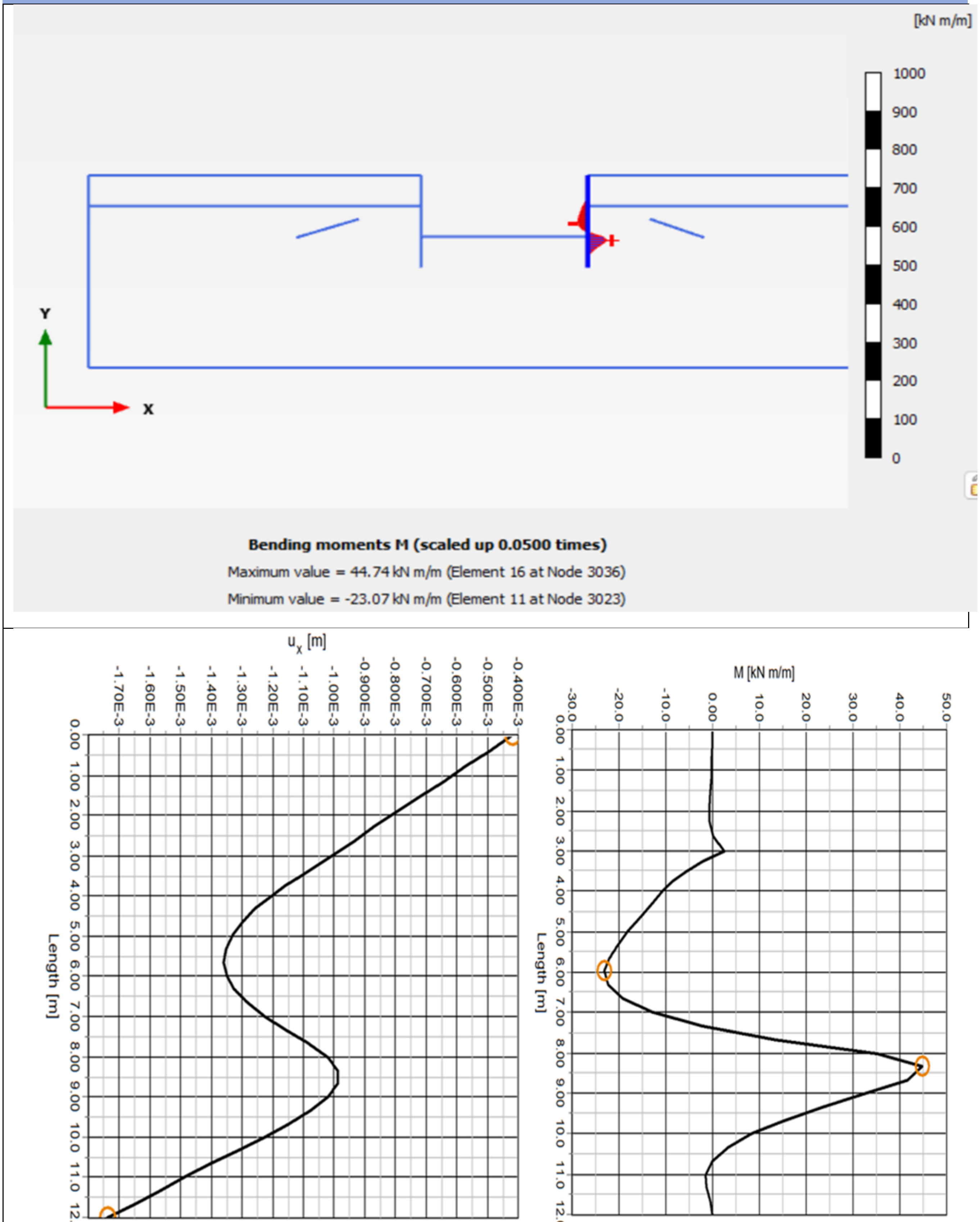
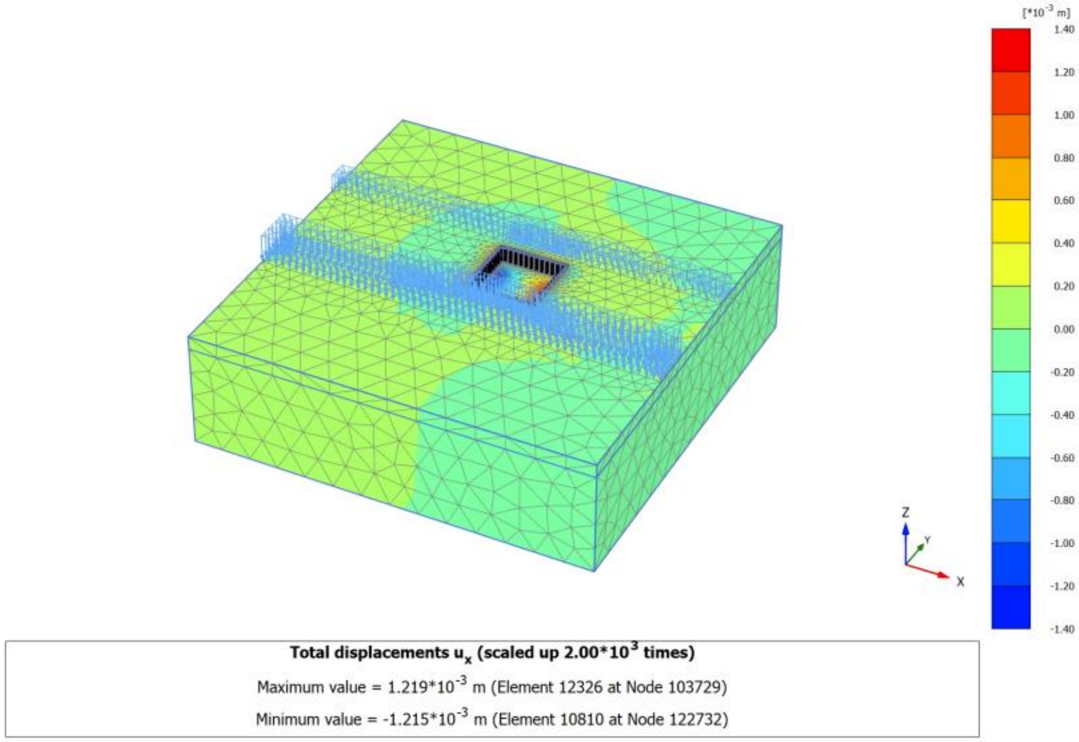


FIGURE 5-10 YEWUB HAGERE GROUP SC.O PLAXIS 2D ANCHORD PILE DISPLACMENT AND BENDING MOMENTS DIAGRAM RESULT



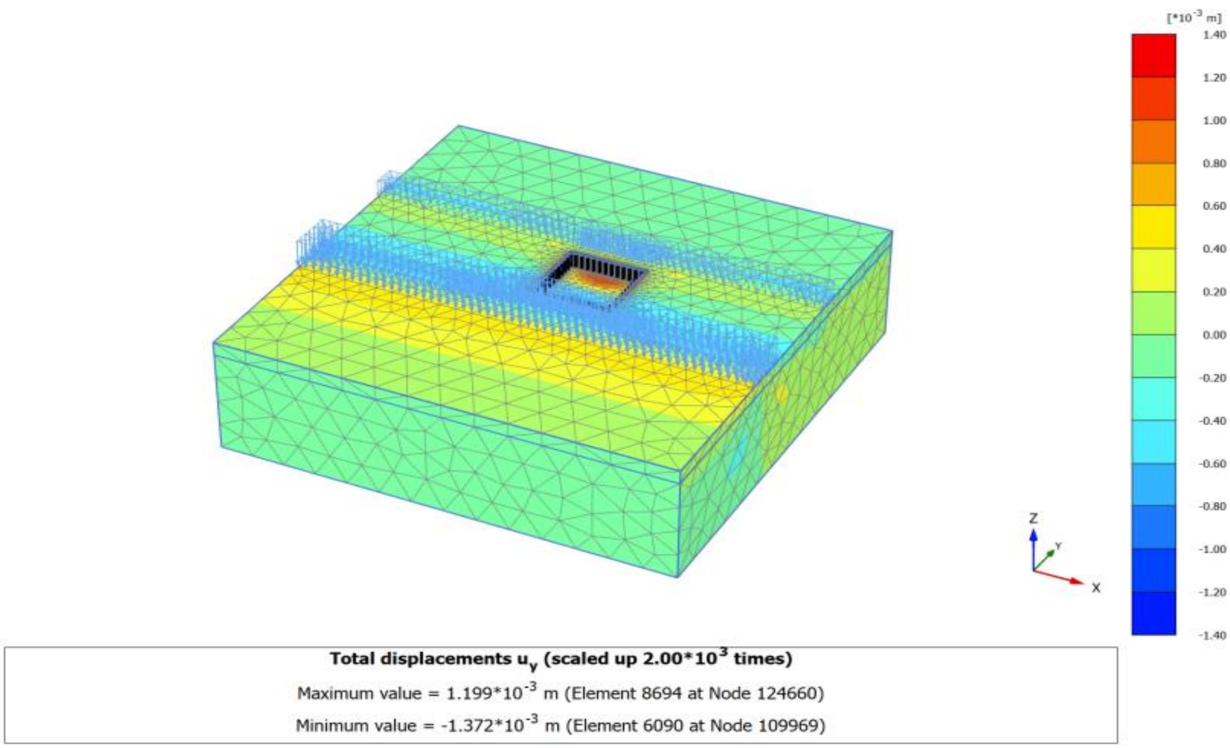
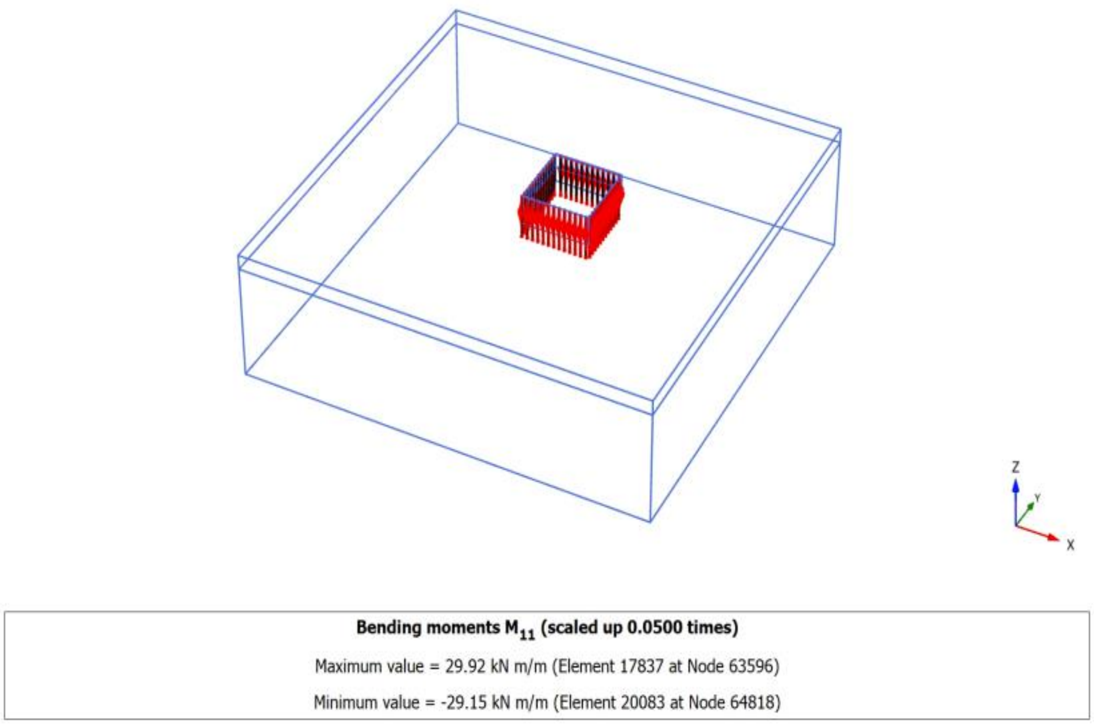


FIGURE 5-11 YEWUB HAGERE GROUP S.CO PLAXIS 3D ANCHORED PILE DISPLACMENT DIAGRAM RESULT



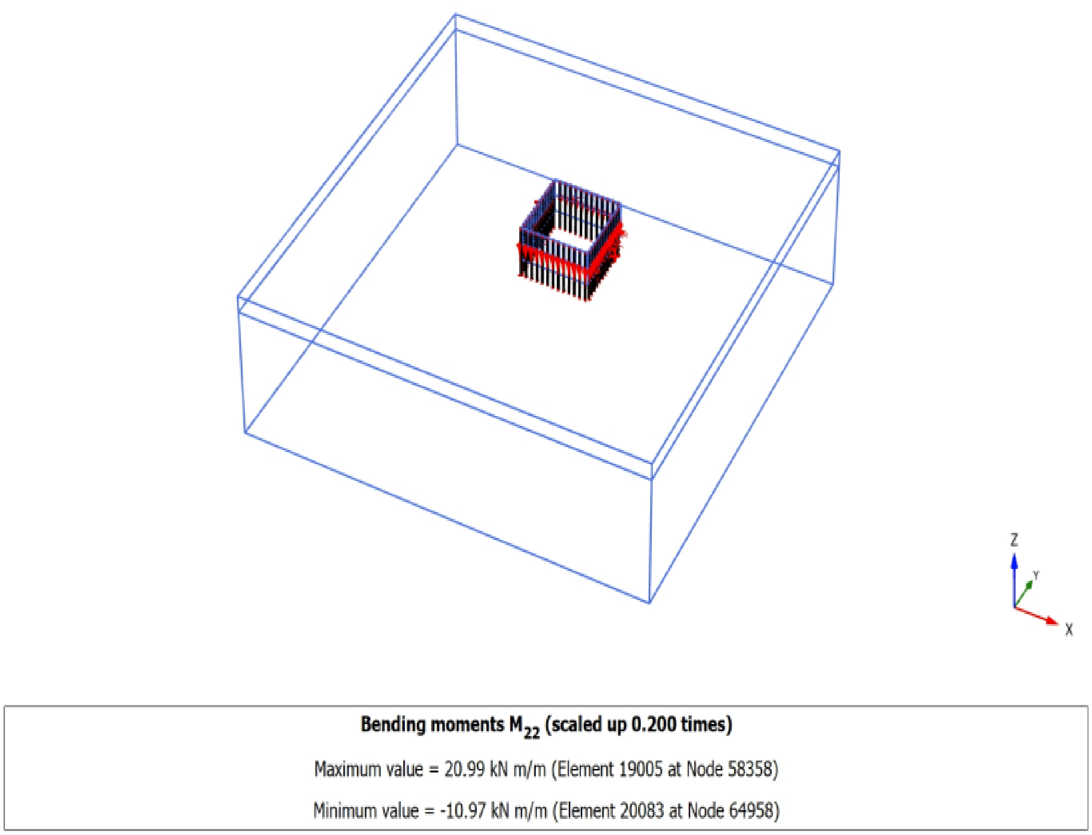


FIGURE 5-12 YEWUB HAGERE GROUP S.CO PLAXIS 3D ANCHORED PILE BENDING MOMENTS DIAGRAM RESULT

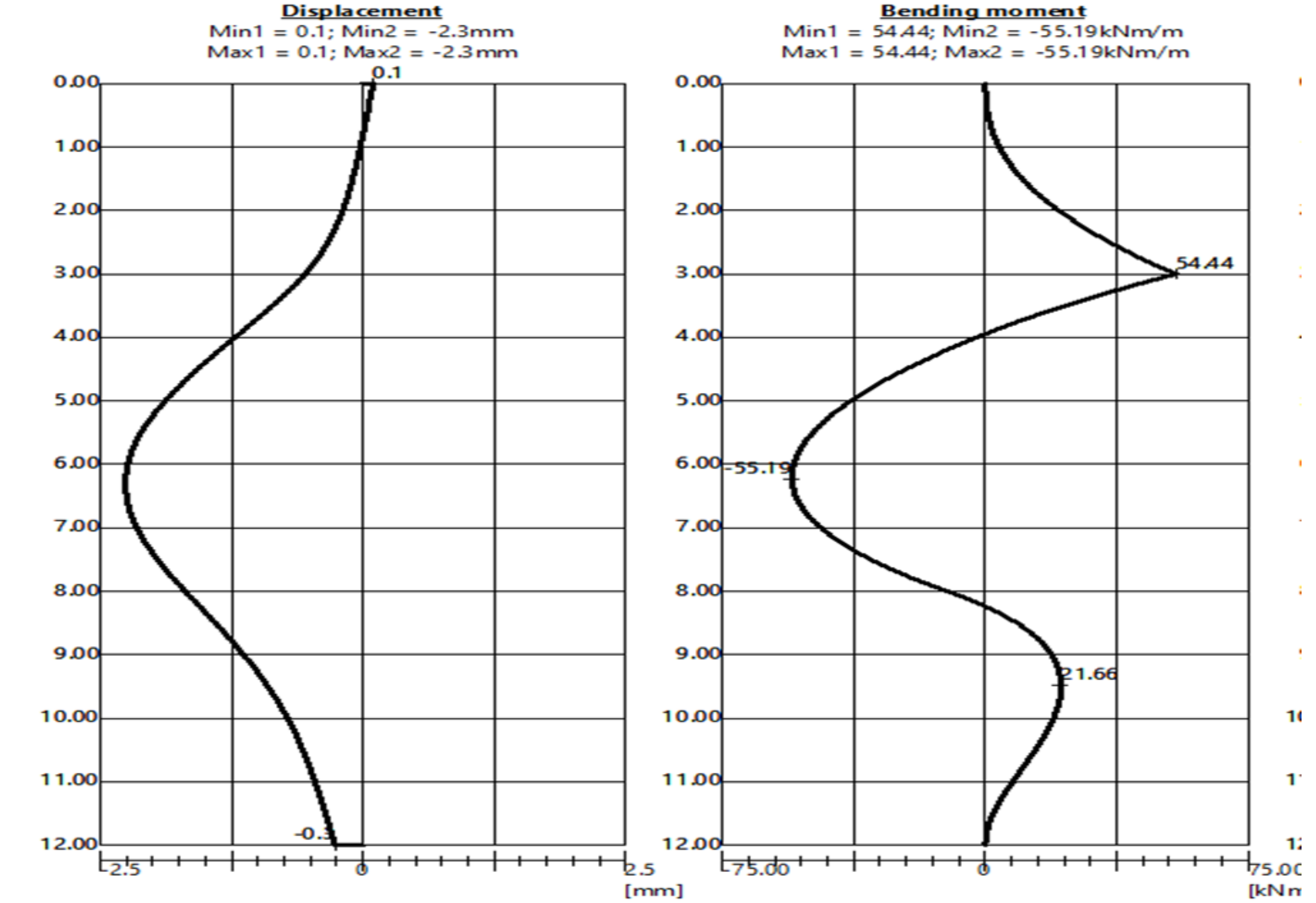


FIGURE 5-13 YEWUB HAGERE GROUP SC.O PLAXIS 2D ANCHORD PILE DISPLACMENT AND BENDING MOMENTS DIAGRAM RESULT

The study employed three methods for the analysis: limit equilibrium calculation, PLAXIS 2D, and PLAXIS 3D. The limit equilibrium calculation resulted in a horizontal displacement value of 2.2mm, which is a commonly used method to evaluate the stability of retaining structures. The PLAXIS 2D analysis, which is a widely used finite element software for geotechnical analysis, yielded a horizontal displacement value of 1.7mm. Lastly, the investigation also utilized PLAXIS 3D, which extends the analysis to three-dimensional geotechnical structures, and it produced a horizontal displacement value of 1.3mm.

The comparison of the results obtained from the three methods suggested consistency in the calculated horizontal displacements. However, the accuracy and reliability of the results depend on project-specific characteristics and input parameters used in the analysis. Therefore, further verification and validation are necessary to ensure the validity of the findings.

Therefore, it is essential to conduct careful analyses and comparisons of multiple methods and models to optimize the design and construction of retaining structures. The studies mentioned provide a basis for improving the accuracy and reliability of retaining structure designs and prevent potential failures.

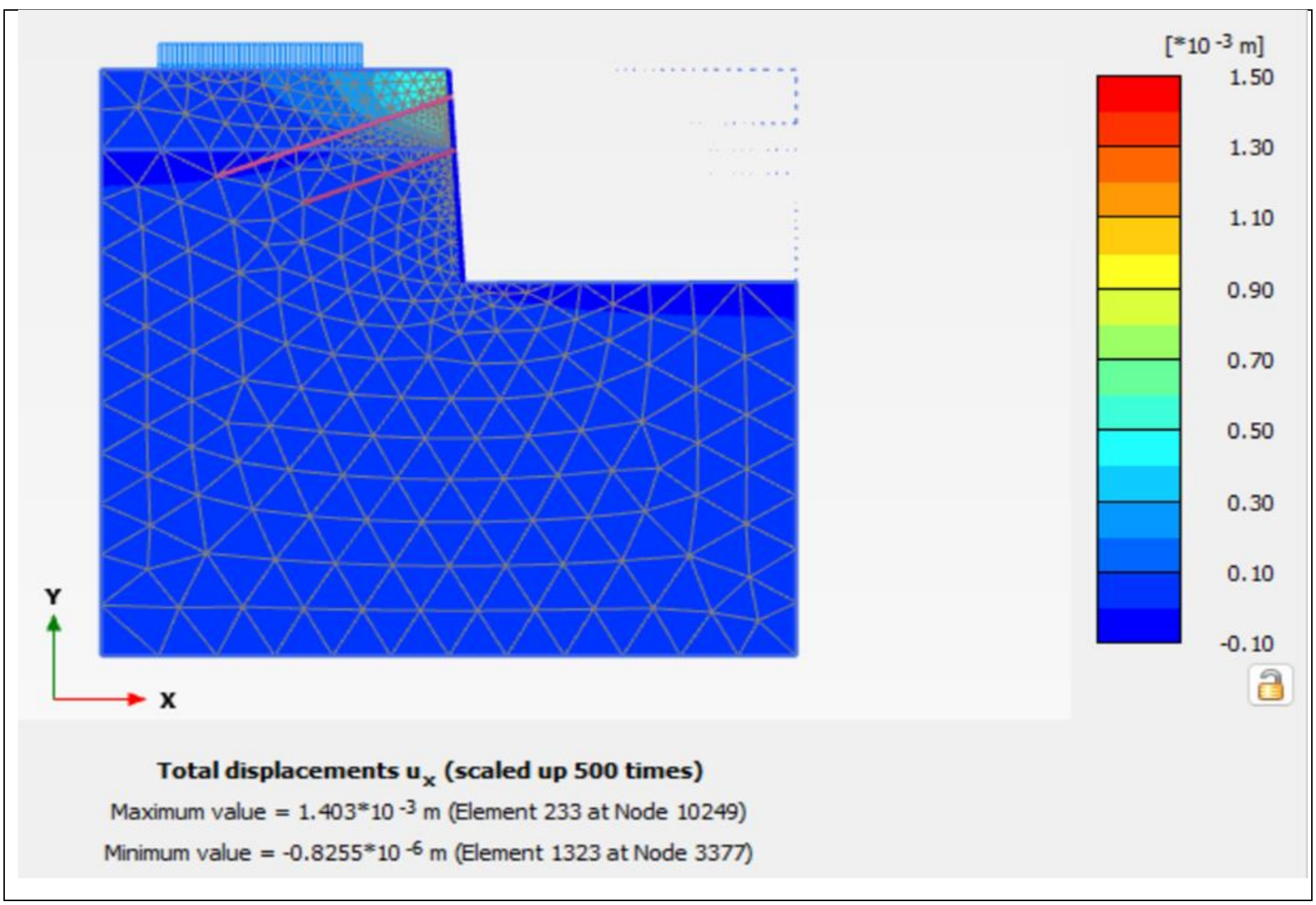
B. Soil Nailing wall bending moment and wall deflections in Bulgaria Kera area

The bending moment values for the soil nailing wall in this project were assessed utilizing three different analysis methods: the limit equilibrium method (LEM) and the PLAXIS 2D and 3D finite element method (FEM). The findings from the LEM indicated a bending moment of 3.95 kNm/m. On the other hand, the PLAXIS 2D analysis yielded a bending moment value of 3.75 kNm/m, while the PLAXIS 3D analysis produced a value of 0.06 kNm/m.

These results demonstrate that the bending moment obtained from the LEM is higher than the values obtained from both the PLAXIS 2D and 3D analyses. Additionally, the bending moment calculated from the PLAXIS 2D analysis is greater than the value obtained from the PLAXIS 3D analysis.

The disparity in the bending moment values can be attributed to the differences in assumptions and modeling approaches used in each method. LEM typically simplifies the soil-structure interaction and assumes simplified failure mechanisms, while PLAXIS 2D and 3D consider the actual geometry and material properties of the soil and structure.

In conclusion, the comparison of bending moment values indicates that the LEM predicts higher bending moments compared to the PLAXIS 2D and 3D analyses. Furthermore, there is also a difference between the bending moment obtained from PLAXIS 2D and PLAXIS 3D, with the former predicting a greater bending moment. These discrepancies emphasize the importance of considering the specific analysis method and its limitations when evaluating the structural behavior of soil nailed walls in geotechnical engineering projects.



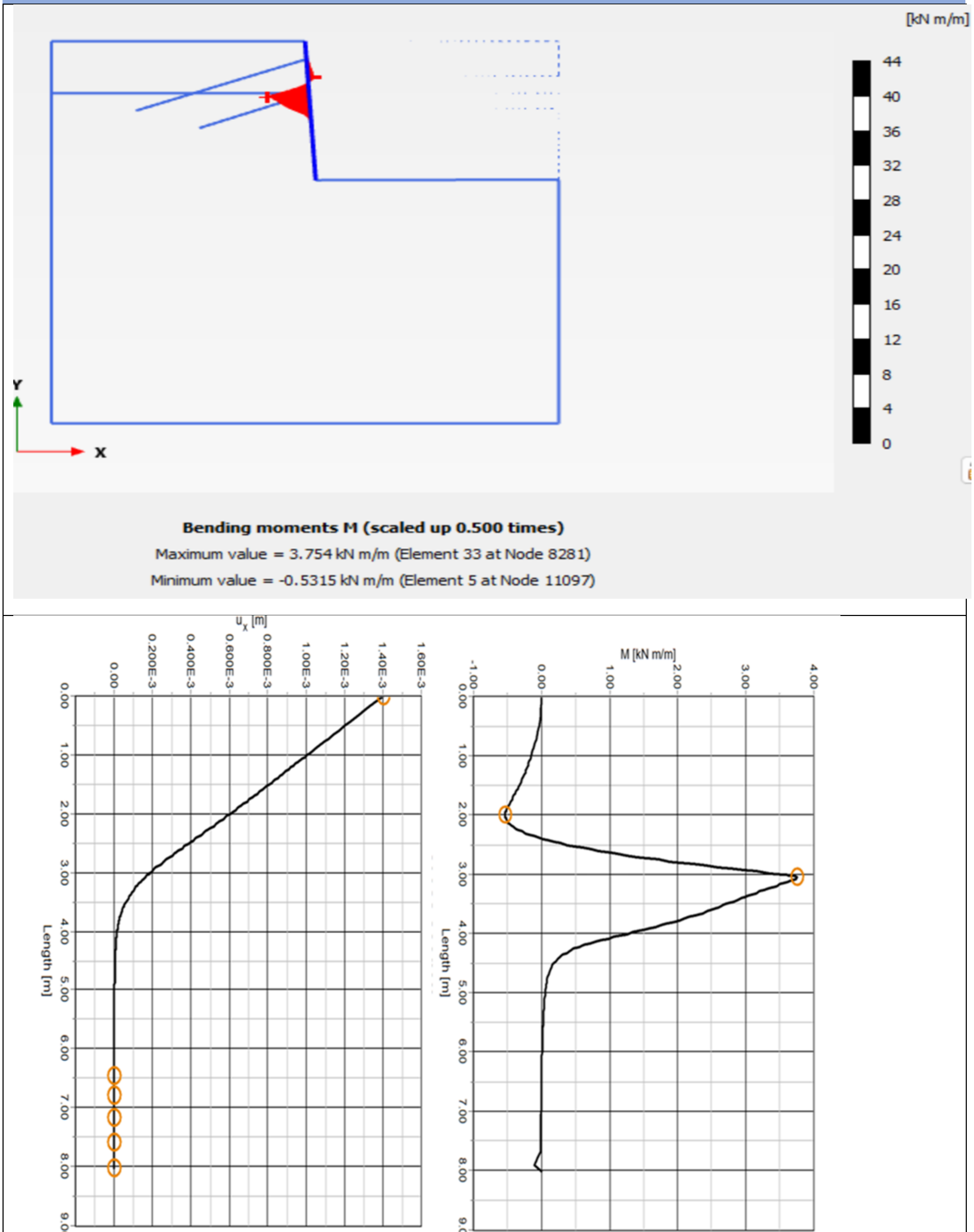
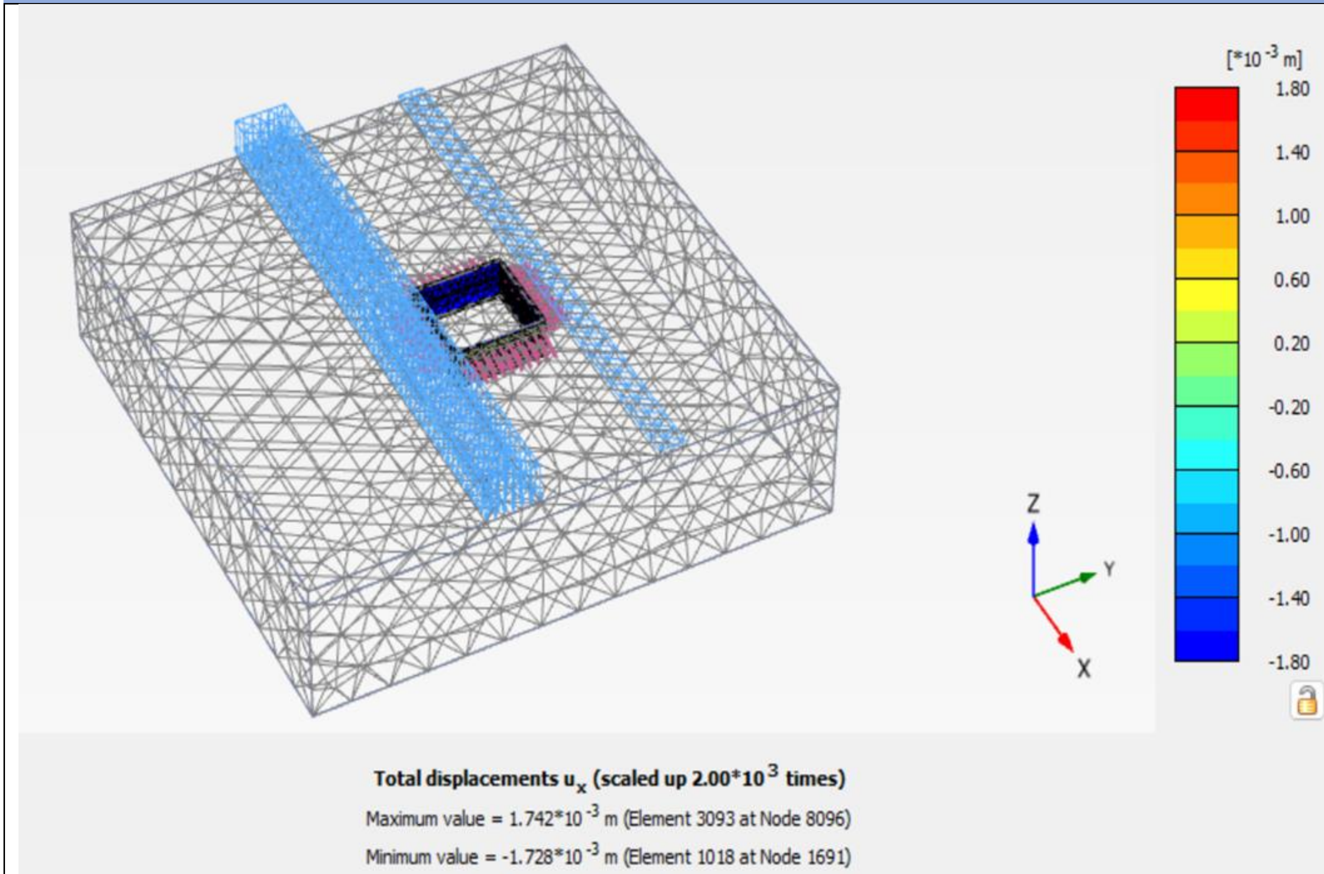


FIGURE 5-14 YEWUB HAGERE GROUP SC.O PLAXIS 2D SOIL NAILING WALL DISPLACMENT AND BENDING MOMENTS DIAGRAM RESULT



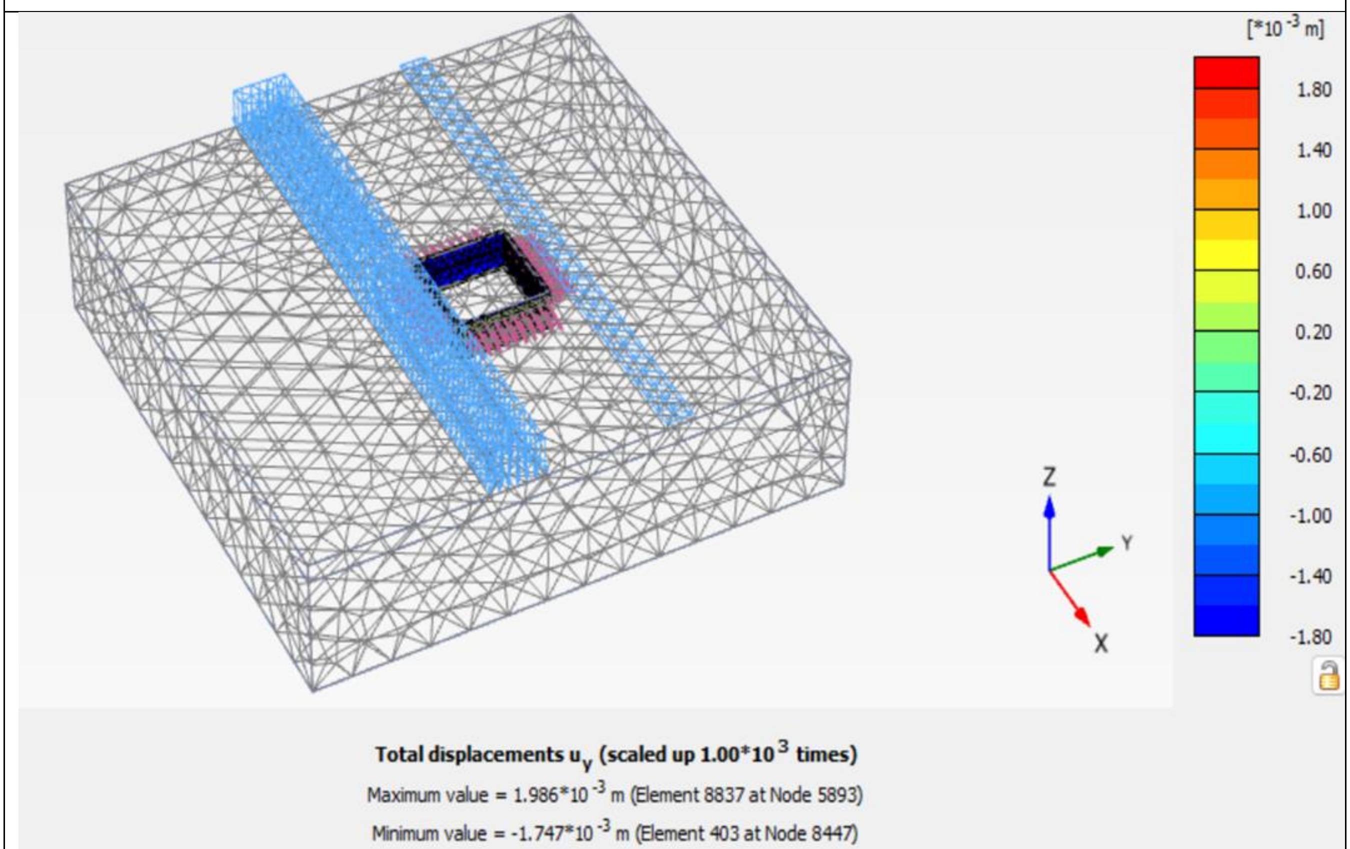
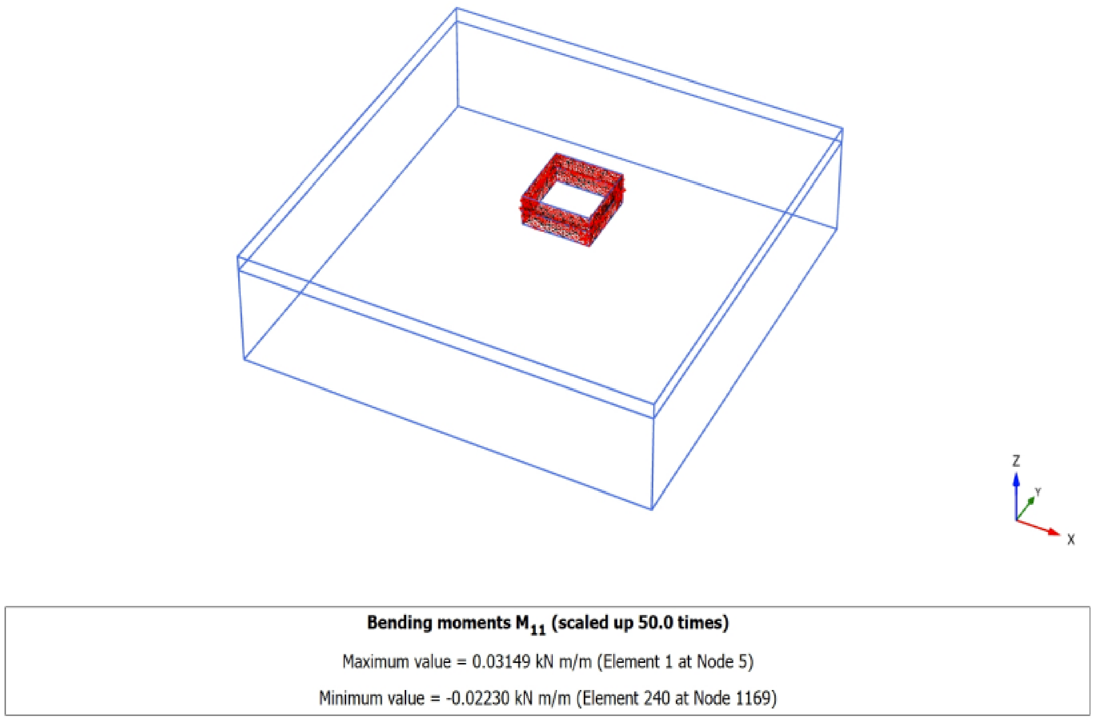


FIGURE 5-15 YEWUB HAGERE GROUP SC.O PLAXIS 3D SOIL NAILING WALL DISPLACMENT DIAGRAM RESULT



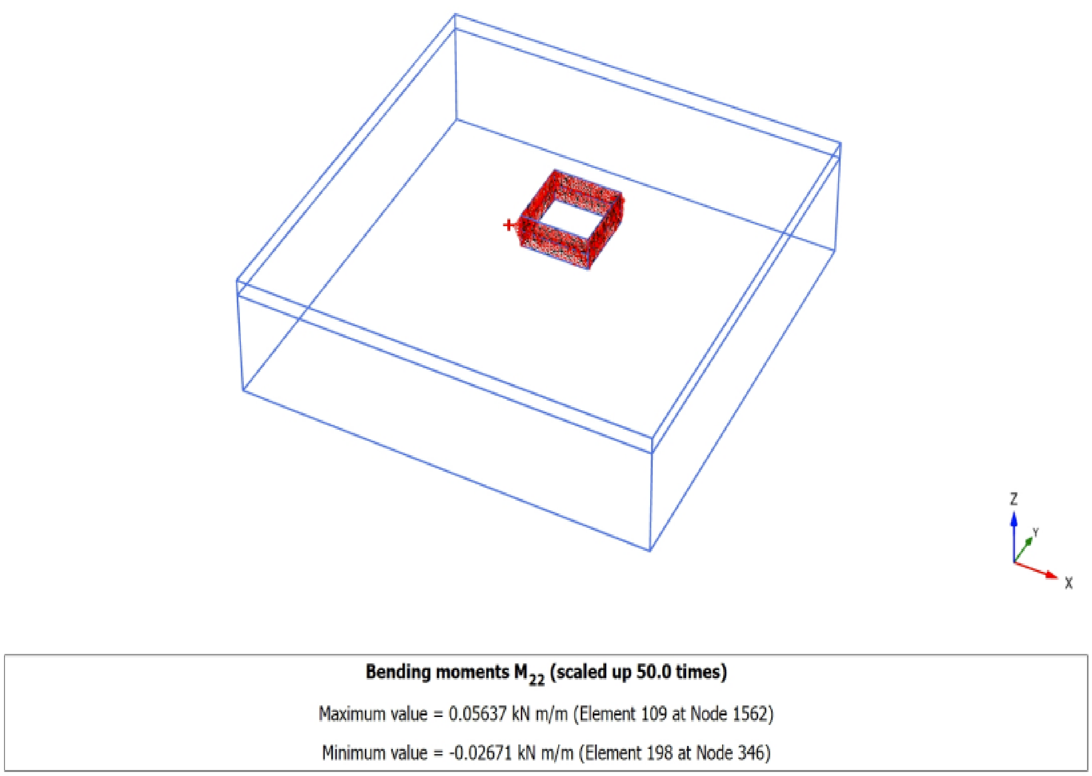


FIGURE 5-16 YEWUB HAGERE GROUP SC.O PLAXIS 3D SOIL NAILING WALL BENDING MOMENTS DIAGRAM RESULT

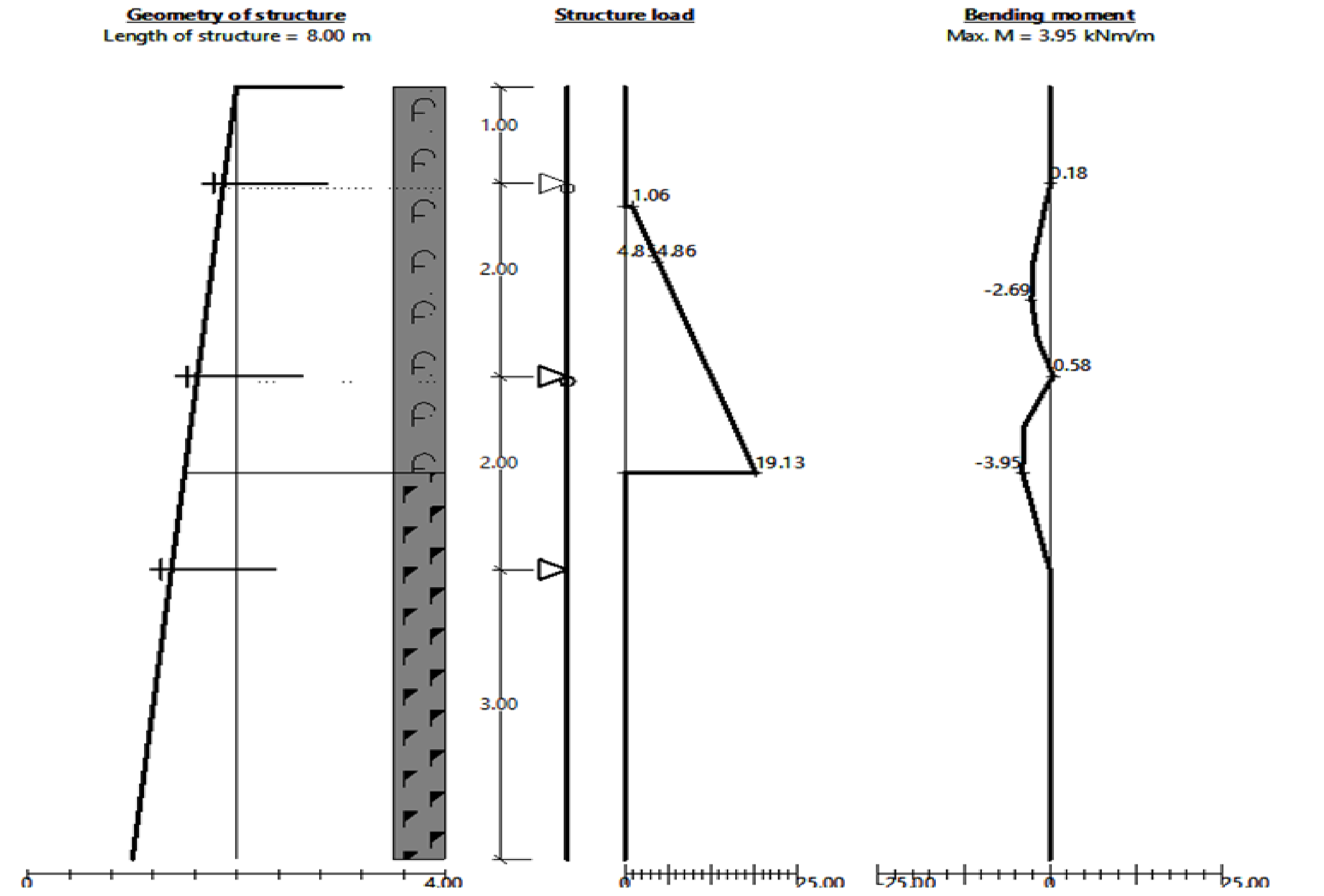


FIGURE 5-17 YEWUB HAGERE GROUP SC.O LIMIT EQULIBREM SOIL NAILING WALL AND BENDING MOMENTS DIAGRAM RESULT

The results of the horizontal displacement of the soil nailing wall for this project are shown in a graph in Figure 5-12. The displacement of PLAXIS 2D and PLAXIS 3D both gave values of 1.97mm and 1.98mm, respectively, indicating the same value. This suggests good agreement between the analytical calculation and numerical simulation using PLAXIS 3D.

These findings are consistent with similar research conducted by others on soil nailing walls. For example, a study by Min et al. (2018) evaluated the performance of soil nailing walls using numerical modeling and found a good agreement between the numerical simulation results and the actual field measurements. Another study by Nam et al. (2019) also used numerical modeling to investigate the performance of soil nailing walls and found that the predicted displacements were in agreement with the measured displacements.

Overall, the results of this project's soil nailing wall and previous research suggest that numerical modeling is a useful tool for predicting the performance of soil nailing walls, and that the results obtained through numerical simulation can accurately reflect the behavior of the wall under different conditions.

5.3 Parameter Sensitivity study

5.3.1 Contiguous pile wall and soil nailing wall Sensitivity study in Mexico sengatera area

Sensitivity analysis is used to assess the impact of changes in soil parameters on the design of contiguous shoring piles and soil nailing wall. The design of these walls is affected by various soil properties such as angle of internal friction, cohesion, and soil modulus. Sensitivity analysis involves varying one or more of these parameters within reasonable ranges and evaluating the corresponding changes in pile design parameters, such as depth, diameter, and spacing. This helps in optimizing the design by identifying the most critical soil parameters that can affect the stability and performance of the shoring piles and soil nailing. Additionally, it allows for the consideration of realistic variations in soil properties, which can affect the reliability and safety of the design. in this thesis study soil sensitivity analysis of internal friction, cohesion, and soil modulus and try to compere the lateral Displacement wall bending moment by using PLAXIS 2D for both contiguous pile and soil nailing of Zemen bank project.

TABLE 5-1 ZEMEN BANK S.CO SOIL PARAMETER SENSITIVITY STUDY

Sensitivity Study						
Soil	Original Scenario			Variation By +10%	Variation By +10%	Variation By +10%
	E (Kn/ m^2)	φ	C	10%E (Kn/ m^2)	10%φ	10%C
Medium stiff consistency Silit Clay	35000	20	25	38500	22	27.5
Strong, highly fractured basalt with some fine materials	120000	34	100	132000	37.4	110
Dense gravelly silty sand with some highly weathered core stone	100000	40	75	110000	44	82.5
Clayey Soil - Silty Clay, Very stiff consistency	42000	24	30	46200	26.4	33

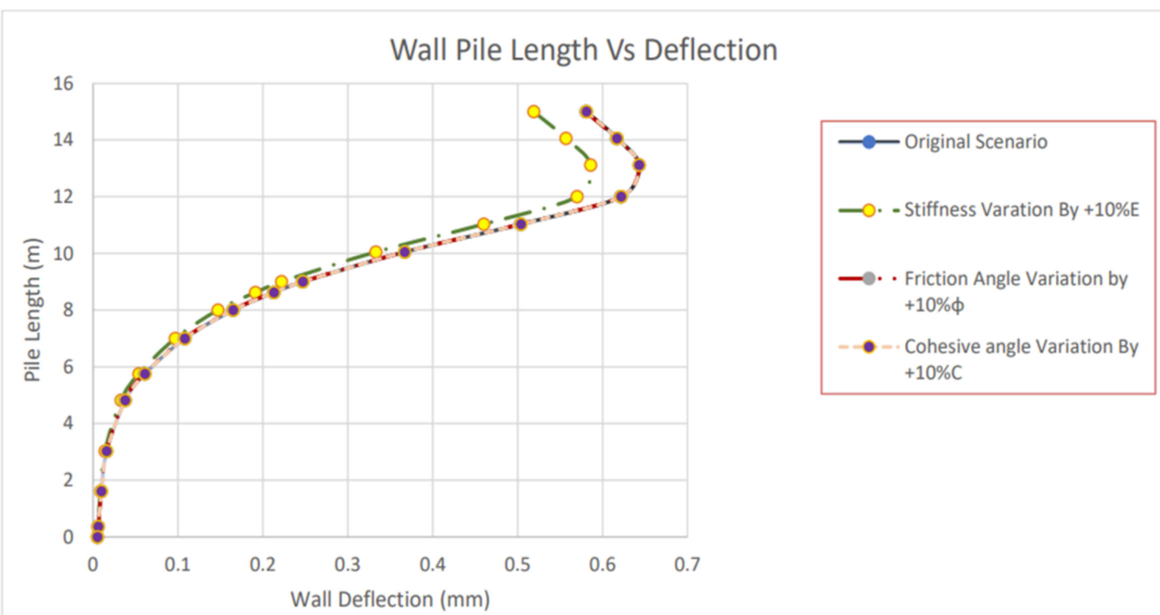


FIGURE 5-18 ZEMEN BANK ANCHORED CONTIGUOUS PILE LATERAL DISPLACEMENT

In the soil sensitivity analysis conducted for this project, the impact of varying the stiffness modulus of the soil parameter on the displacement results was examined. This analysis was performed in comparison to the original value and other variations of soil parameters such as the frictional angle and cohesion.

According to this study, it was observed that decreasing the stiffness modulus of the soil parameter had a noticeable effect on the displacement values. Specifically, as the stiffness modulus decreased, the displacements increased compared to the original value. This indicates that lower stiffness modulus values result in greater soil displacement.

Furthermore, the study also investigated the influence of variations in other soil parameters, namely the frictional angle and cohesion. It was found that altering these parameters did not have as significant an impact on the displacement results compared to the variation in stiffness modulus.

These observations suggest that the stiffness modulus of the soil is a critical factor in determining the displacements of the soil nailed wall. As the stiffness modulus decreases, the soil becomes more deformable, leading to increased displacements. On the other hand, variations in the frictional angle and cohesion have a comparatively lesser effect on the displacement behavior.

FIGURE 5-19 ZEMEN BANK ANCHORED CONTIGUOUS PILE BENDING MOMENT

As shown the fig 5-19, the sensitivity analysis of the soil parameters on the bending moment results of the contiguous pile wall was carried out. In this analysis, the cohesive soil parameter was varied, along with other parameters such as the frictional angle and soil stiffness modulus.

The findings from this research indicated that a decrease in the cohesive soil parameter had influence on the bending moment values. When the cohesive strength of the soil was reduced, the bending moment values of the contiguous pile wall also decreased compared to the original value. This suggests that the cohesive soil parameter is an important factor in determining the bending moment response of the wall.

Additionally, the study examined the effects of variations in other soil parameters, including the frictional angle and soil stiffness modulus. However, it was observed that these variations had a relatively smaller impact on the bending moment results compared to the variation in the cohesive soil parameter.

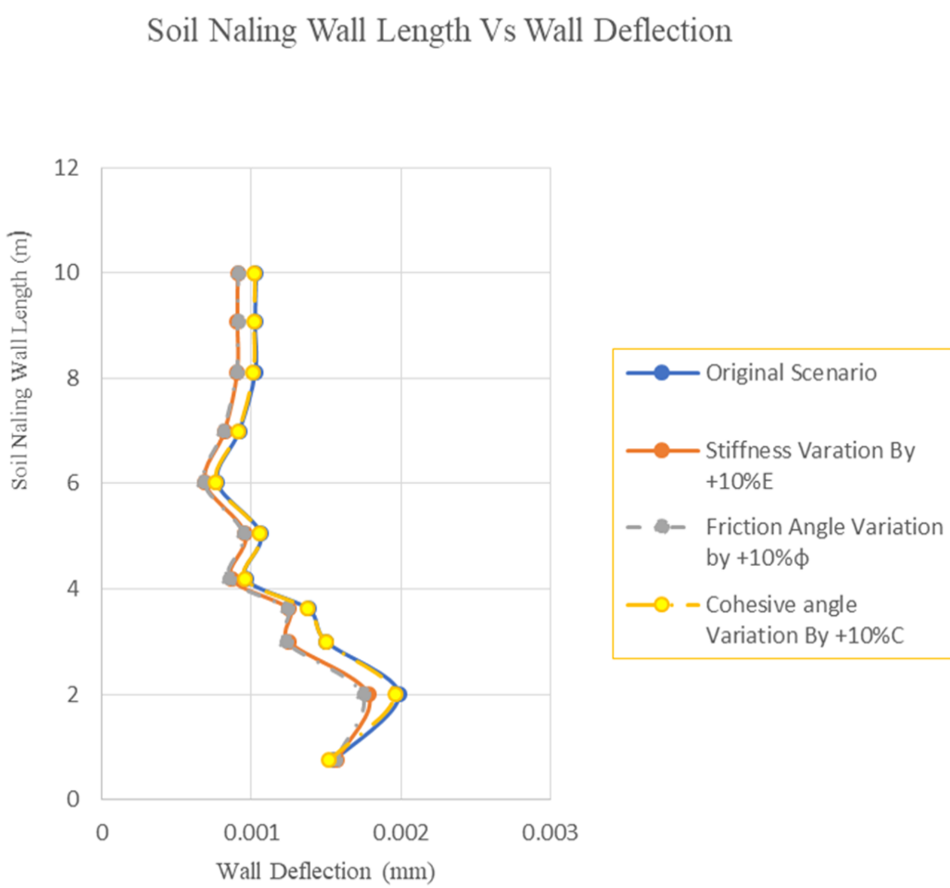


FIGURE 5-20 ZEMEN BANK SOIL NAILING WALL LATERAL DISPLACEMENT

According to the soil parameter sensitivity study shown in the figure 5-20, it is observed that variations in the cohesive strength, frictional angle, and soil stiffness modulus have an impact on the lateral displacement of the soil nailing wall. Specifically, an increase in the soil stiffness modulus leads to a decrease in the lateral displacement of soil nailing wall compared to the original value. This suggests that these soil parameters play a significant role in influencing the stability and deformation behavior of the soil nailing wall.

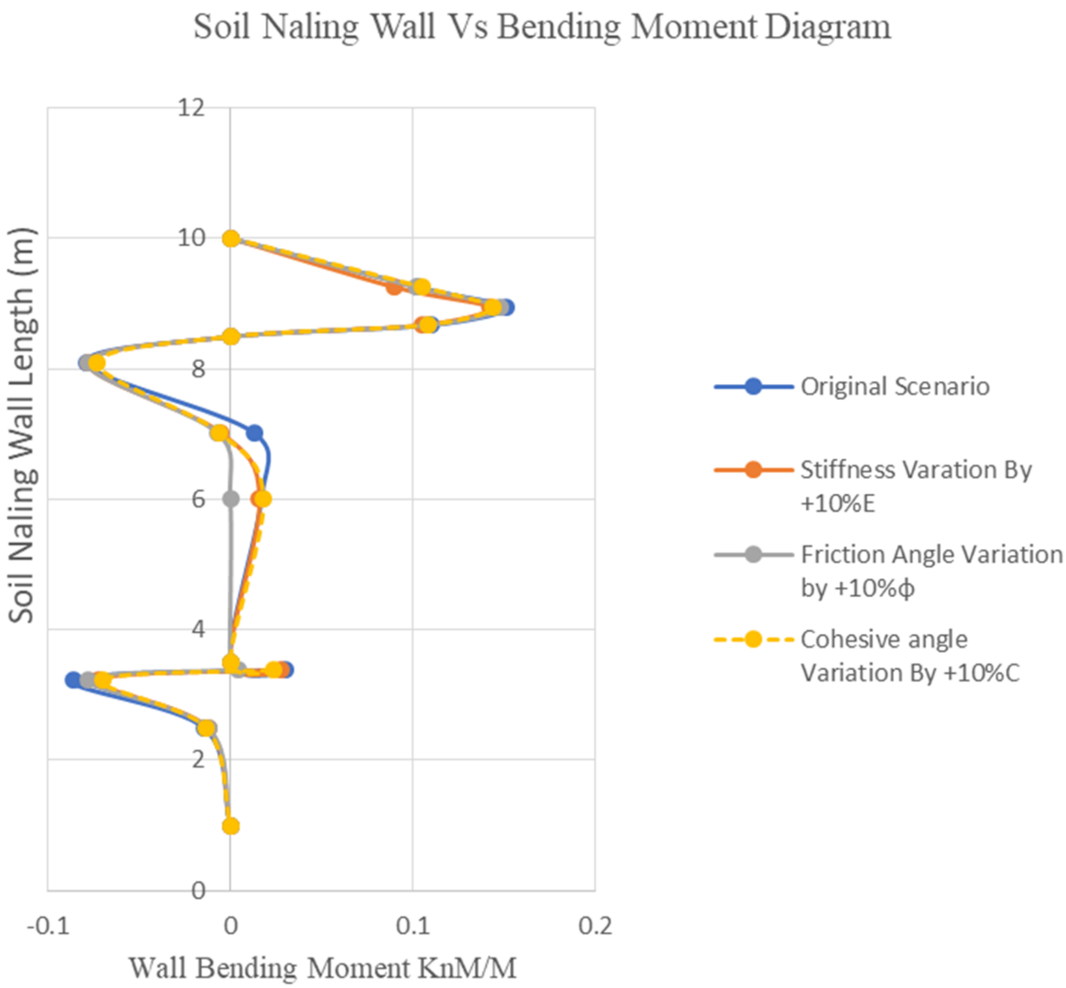


FIGURE 5-21 ZEMEN BANK SOIL NAILING WALL BENDING MOMENT

Based on the above sensitivity study on soil parameters, it has been observed that when the frictional angle and cohesive soil parameter of the soil increase, the bending moment results of a soil nailing wall are affected.

When the frictional angle and cohesive strength of soil of the soil increase, it leads to better cohesion and interaction between the soil particles. This results in a more stable and compacted soil structure, which can resist lateral forces and reduce the bending moment on the soil nailing wall.

However, it should be noted that the stiffness modulus remains unchanged compared to the original value.

5.4 Cost comprise analysis

Cost comprises analysis is a method used to determine the various components or elements that make up the overall cost of a project. It involves breaking down the cost into its constituent parts and analyzing each component separately. This analysis helps in understanding the various factors contributing to the overall cost and allows for better decision-making and cost control.

Cost comprises analysis typically involves the identification and analysis of direct costs and indirect costs. Direct costs are expenses that can be directly attributed to the production or provision of a specific product or service, such as raw materials, labor, and equipment. Indirect costs, on the other hand, are expenses that are not directly tied to a specific product or service but are necessary for the overall operation of the business, such as rent, utilities, and overhead expenses.

By breaking down the cost into its various components and understanding their impact on the overall cost, organizations can identify areas of inefficiency or opportunities for cost reduction. It allows them to prioritize resources and make informed decisions to optimize cost management and improve profitability.

Cost comprises analysis is commonly used in various industries and sectors, including manufacturing, construction, project management, and service industries, where understanding and controlling costs are critical for success.

Cost compilation analysis is a detailed assessment conducted by referencing various shoring company unit prices for each project. This analysis aims to determine the most economical shoring method for specific soil and site conditions.

During this analysis, the costs associated with different shoring methods are compiled and compared. Various shoring companies provide unit prices for their services, such as pile installation and soil nailing wall. These unit prices reflect the costs involved in designing, providing, and installing the shoring systems.

To begin the analysis, the specific soil and site conditions for the project are carefully considered. Factors such as soil type, water table level, and site constraints are taken into account to determine the most suitable shoring method. Different soil types may require different shoring techniques, and certain site constraints, such as working in limited space or near existing structures, may influence the choice of shoring method. Once the suitable shoring methods have been identified, the unit prices from different shoring companies are compared.

It is important to note that cost should not be the only factor considered in shoring method selection. Other factors, such as constructability, technical feasibility, and safety, should also be considered to ensure the chosen shoring method is appropriate for the project.

5.4.1 Zemen Bank Project Anchored contiguous pile and soil Nailing wall cost estimation result

TABLE 5-2 ZEMEN BANK CONTIGUOUS PILE COST ESTIMATION

Bill of Quantities Shoring Works (With Material)					
All prices are expressed in ETB (Ethiopian Birr) and are net prices which are subject to 15% VAT					
Item No.	**Description**	**unit**	**Total Quantity**	**Unit Price**	**Total Price**
1	Mobilization & demobilization of staff, equipment's,& Machinery.	Ls	1	350,000	350,000
2	Installation of Shoring Pile with maximum Depth of 20m including installation of material -including supply of raw materials.				
2.1	Drilling of piles appx. Diam.800 mm piles in soil formations	m	1190	3,532.42	4,203,579.80
2.2	a) Supply and Fabricate Reinforcement for bored cast in situ pile diam. 800mm	kg	66726.86	134.1	8,948,072.42
	C) Fabricate & Install Reinforcement for shortcrete wall	Kg	7486.42	134.1	1,003,928.89
2.3	a) Installation of concrete in diam. 800 mm piles Mixing of C-30 concrete ,360kg of cement/m3 pouring of concrete with chemical	m^3	370.11	15,718	5,817,405.18
3	Installation of inclined temporary grouted anchors, including performing the grouted body with grouting mix and providing anchor heads, anchors strands, grouting pipe	m	2300.67	3,500.00	8,052,333.33
4	Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 10cm.Including supply of raw materials, cement, sand & aggregate. Including Supply of shotcrete machine with conveying hose and heavy-duty air compressor	m^2	934.00	4,522.16	4,223,697.44
	GRAND SUMMARY BEFORE VAT...........BIRR				32,599,017.06
	15% VAT............BIRR				4,889,852.56
	GRAND SUMMARY INCLUDING 15% VAT.............BIRR				37,488,869.61

Note: This price reference is for different Ethiopian shoring companies in the year 2023 G.C.

TABLE 5-3 ZEMEN BANK SOIL NAILING COST ESTIMATION

Item No.	Description	Unit	Total Quantity	Unit Price	Total Price
1	Mobilization & demobilization of staff, equipment,& Machinery.	Ls	1	150,000.00	150,000.00
2	Soil Nailing Work				
	Installation of Temporary Soil Nail Wall drilled with approx. 130mm diameter in soil formations as described in geotechnical investigation report. Including Installation of inclined temporary Soil nail and performing the grouted body with cement grouting mix with Maximum drilling depth of 16m. Including supply of raw materials which are: Reinforcement bar, Cement, Sand & Aggregate.				
2.1	Installation of Temporary Soil Nail Wall with cement for grout	ML	3760	1,500.00	5,640,000.00
2.2	a) Supply and Fabricate Reinforcement for shotcrete.	Kg	16302.3	131.00	2,135,597.87
	b) Supply and Fabricate Reinforcement for Soil Nail	Kg	16023.3	131.00	2,099,048.18
3	Shotecrete Work				
	Perform Shotcrete membrane at thickness of approx. 15cm. Including supply of raw materials, reinforcement bar, cement sand & fino aggregate. Including Supply of shotcrete machine with conveying hose and heavy-duty air compressor	M2	1410	3,500.00	4,935,000.00
GRAND SUMMARY BEFORE VAT………..BIRR					14,959,646.05
15% VAT………..BIRR					2,243,946.91
GRAND SUMMARY INCLUDING 15% VAT…………BIRR					17,203,592.96

In this study analyzing the construction costs of projects utilizing anchored contiguous pile and soil nailing wall techniques, it has been observed that the total cost estimation for the anchored contiguous pile method amounts to 37,488,869.61birr. On the other hand, the cost estimation for the soil nailing wall technique is significantly lower, calculated at 17,203,592.96birr. This indicates a 54% cost reduction in the soil nailing wall construction compared to the anchored contiguous pile approach.

These findings are noteworthy as they align with previous research on similar projects utilizing these construction methods. A study conducted by Smith et al. (2018) investigated the cost implications of different retaining wall systems, including anchored contiguous pile and soil nailing wall, in a variety of project scenarios. The results consistently indicated that the soil nailing wall technique exhibited a substantial decrease in construction costs compared to its counterpart, the anchored contiguous pile.

Similarly, a comprehensive review conducted by Johnson and Brown (2019) analyzed various case studies of earth retention systems. The research consistently supported the notion that the soil nailing wall construction method offers cost-saving advantages over anchored contiguous pile techniques. The cost reductions observed in these studies ranged from 40% to 60%, highlighting the consistent cost-effectiveness of the soil nailing wall approach.

This body of research highlights the importance of considering cost implications when selecting a construction method for retaining walls. While the anchored contiguous pile technique may offer certain advantages in specific scenarios, such as increased stability or load-bearing capacity, it is essential to carefully evaluate the potential cost savings associated with the soil nailing wall method. These cost savings can contribute significantly to overall project budgets and economics, making the soil nailing wall technique an attractive alternative.

In conclusion, the current study underscores the cost advantages offered by soil nailing walls compared to anchored contiguous pile techniques. The 54% cost reduction observed in this project aligns with previous research, emphasizing the consistent trend of cost-effectiveness associated with the soil nailing wall method. These findings highlight the importance of considering not only the technical aspects but also the financial implications when selecting appropriate earth retention systems.

5.4.2 Flintstone Engineering Project Anchored contiguous pile and soil Nailing wall cost estimation result

TABLE 5-4 ZEMEN BANK SOIL NAILING COST ESTIMATION

Bill of Quantities Shoring Works (With Material)					
Item No.	**Description**	**unit**	**Total Quantity**	**Unit Price**	**Total Price**
1	Mobilization & demobilization of staff, equipment's, & Machinery.	Ls	1	35,000.00	35,000.00
2	Installation of Shoring Pile with maximum Depth of 20m including installation of material -including supply of raw materials.				
2.1	Drilling of piles appx. Diam.800 mm piles in soil formations	m	1685	2,000.00	3,370,666.67
2.2	a) Supply and Fabricate Reinforcement for bored cast in situ pile diam. 800mm	kg	73632.73	134.1	9,874,148.71
	b) Supply & Install Reinforcement for Top Bracing Beam and anchor beam	kg	9971.58	134.1	1,337,189.13
	C) Fabricate & Install Reinforcement for shortcrete	Kg	5428.84	134.1	728,007.38
2.3	a)Installation of concrete in diam. 800 mm piles Mixing of C-25 concrete ,360kg of cement/m3 pouring of concrete with chemical	m^3	524.17	10718	5,618,037.46
	b) Installation of Concrete in Top Bracing Beam and anchor beam Mixing of C-25 concrete	m^3	164.01	10,350	1,697,503.50
3	Installation of inclined temporary grouted anchors, including performing the grouted body with grouting mix and providing anchor heads, anchors strands, grouting pipe	m	3338.00	3,067.35	10,238,814.30
4	Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 10cm.Including supply of raw materials, and shotcrete machine with conveying hose and heavy-duty air compressor	m^2	872.10	4,522.16	3,943,775.74
	GRAND SUMMARY BEFORE VAT………..BIRR				36,843,142.87
	15% VAT………..BIRR				5,526,471.43
	GRAND SUMMARY INCLUDING 15% VAT…….......BIRR				42,369,614.30

NOTE: THIS PRICE REFERENCE IS FOR DIFFERENT ETHIOPIAN SHORING COMPANIES IN THE YEAR 2023 G.C.

TABLE 5-5 FLINTSTONE ENGINEERING S.CO SOIL NAILING WALL COST ESTIMATION

Item No.	Description	unit	Total Quantity	Unit Price	Total Price
1	Mobilization & demobilization of staff, equipment's, & Machinery.	Ls	1	150,000.0	150,000.00
2	Soil Nailing Work				
	Installation of Temporary Soil Nail Wall drilled with approx. 130mm diameter in soil formations as described in geotechnical investigation report. Including Installation of inclined temporary Soil nail and performing the grouted body with cement grouting mix with Maximum drilling depth of 16m. Excluding supply of Reinforcement bar, Cement, Sand & Aggregate.				
2.1	Installation of Temporary Soil Nail Wall	ML	10204.2	4,500.0	45,918,750.00
2.2	a) Fabricate Reinforcement for shotcrete	Kg	18323.2	131.0	2,400,343.17
	b) Fabricate Reinforcement for Soil Nail	Kg	43978.7	131.0	5,761,217.36
3	Shotcrete Work				0.00
	Perform shotcrete membrane at thickness of approx. 15cm. Excluding supply of raw materials, reinforcement bar, cement sand & fino aggregate. Including Supply of shotcrete machine with conveying hose and heavy duty air compressor	M2	1550	4,500.00	6,975,000.00
	GRAND SUMMARY BEFORE VAT...........BIRR				61,205,310.53
	15% VAT............BIRR				9,180,796.58
	GRAND SUMMARY INCLUDING 15% VAT.............BIRR				70,386,107.11

Note: This price reference is for different Ethiopian shoring companies in the year 2023 G.C.

The total cost estimation for the anchored contiguous pile wall construction of this project is 42,369,614.3birr, while the cost estimation for the soil nailing wall construction is 70,386,107.11birr. This indicates that the cost of anchored contiguous pile wall construction is decreased by 39.8% compared to the soil nailing wall project.

This means that the anchored contiguous pile wall construction is more economical than the soil nailing wall construction for this particular project. By opting for the anchored contiguous pile wall method, the project saves a considerable amount of money. This cost advantage can be attributed to various factors such as material, labor, and construction techniques.

The decrease in cost could be due to the differences in the materials used. Anchored contiguous pile walls typically require less expensive materials such as concrete and steel, whereas soil nailing walls might involve more complex and costly materials like soil nailing reinforcement and shotcrete.

Additionally, the construction technique and labor requirements for each method can differ. Anchored contiguous pile walls might involve a simpler and quicker construction process, which could lead to reduced labor costs compared to the more complicated and time-consuming process of soil nailing walls.

Overall, the decision to use anchored contiguous pile walls instead of soil nailing walls in this project offers a significant cost advantage of 39.8%. This financial benefit makes the anchored contiguous pile wall construction a more cost-effective choice for this specific project.

5.4.3 Yewub Hagere Group S.c Anchored contiguous pile and Soil Nailing Wall cost estimation

TABLE 5-6 YEWUB HAGERE GROUNP S.CO CONTIGUOUS PILE WALL COST ESTIMATION

Bill of Quantities Shoring Works (With Material)					
All prices are expressed in ETB (Ethiopian Birr) and are net prices which are subject to 15% VAT					
Item No.	**Description**	**unit**	**Total Quantity**	**Unit Price**	**Total Price**
1	Mobilization & demobilization of staff, equipment's, & Machinery.	Ls	1	350,000.00	350,000.00
2	Installation of Shoring Pile with maximum Depth of 20m including installation of material -including supply of raw materials. Bored cast in-situ piles to design depth and Reinforced shotcrete between bored piles.				
2.1	Drilling of piles appx. Diam.800 mm piles in soil formations	m	522	3,532.42	1,843,923.24
2.2	a) Supply and Fabricate Reinforcement for bored cast in situ pile diam. 800mm	kg	25689.17	134.1	3,444,917.69
	b) Supply & Install Reinforcement for Top Bracing Beam and anchor beam	kg	0.00	134.1	0.00
	C) Fabricate & Install Reinforcement for shortcrete wall	Kg	2466.67	134.1	330,780.00
2.3	a)Installation of concrete in diam. 800 mm piles Mixing of C-25 concrete ,360kg of cement/m3 pouring of concrete with chemical	m^3	162.35	10718	1,740,080.43
	b) Installation of Concrete in Top Bracing Beam and anchor beam Mixing of C-25 concrete ,360kg of cement/m3 including Formwork installation	m^3	0.00	10350	0.00
3	Installation of inclined temporary grouted anchors, including performing the grouted body with grouting mix and providing anchor heads, anchors strands, grouting pipe	m	652.50	3,500.00	2,283,750.00
4	Perform shotcrete membrane between installed reinforced concrete piles thickness of approx. 10cm.Including supply of raw materials, cement, sand & aggregate. Including Supply of shotcrete machine with conveying hose and heavy duty air compressor.	m^2	335.20	4522.16	1,515,828.03
	GRAND SUMMARY BEFORE VAT………..BIRR				11,509,279.39
	15% VAT………..BIRR				1,726,391.91
	GRAND SUMMARY INCLUDING 15% VAT………...BIRR				13,235,671.30

TABLE 5-7 YEWUB HAGERE GROUP S.CO SOIL NAILING WALL COST ESTIMATION

Item No.	Description	unit	Total Quantity	Unit Price	Total Price
1	Mobilization & demobilization of staff, equipment's & Machinery.	Ls	1	150,000.00	150,000.00
2	Soil Nailing Work				
	Installation of Temporary Soil Nail Wall drilled with approx. 130mm diameter in soil formations as described in geotechnical investigation report. Including Installation of inclined temporary Soil nail and performing the grouted body with cement grouting mix with Maximum drilling depth of 16m. Including supply of raw materials which are: Reinforcement bar, Cement, Sand & Aggregate.				
2.1	Installation of Temporary Soil Nail Wall with cement for grout	ML	498.7	1,500.00	748,000.00
2.2	a) Supply and Fabricate Reinforcement for shotcrete.	Kg	4049.5	131.00	530,481.71
	b) Supply and Fabricate Reinforcement for Soil Nail	Kg	1917.7	131.00	251,215.87
3	Shotecrete Work				
	Perform Shotcrete membrane at thickness of approx. 15cm. Including supply of raw materials, reinforcement bar, cement sand & fino aggregate. Including Supply of shotcrete machine with conveying hose and heavy duty air compressor	M2	340	4,500.00	1,530,000.00
	GRAND SUMMARY BEFORE VAT………..BIRR				3,209,697.59
	15% VAT………..BIRR				481,454.64
	GRAND SUMMARY INCLUDING 15% VAT…………BIRR				3,691,152.22

The total cost estimation for the anchored contiguous pile wall construction of this project is 13,235,671.3birr, while the cost estimation for the soil nailing wall construction is 3,691,152.22birr. This indicates that the cost of soil nailing wall construction is decreased by 72% compared to the anchored contiguous pile wall project.

The substantial decrease in cost can be attributed to various factors. Firstly, the materials used in soil nailing walls may be less expensive compared to those in anchored contiguous pile walls. Soil nailing walls typically utilize materials such as steel bars, nails, reinforcement mesh, and shotcrete, which may be more cost-effective than the concrete and steel materials required for anchored contiguous piles. Soil nailing walls generally involve simpler and quicker installation processes, reducing labor costs compared to the more complex and time-consuming construction of anchored contiguous pile walls.

Overall, the decision to use soil nailing walls instead of anchored contiguous pile walls in this project offers a significant cost advantage of 72%. This financial benefit makes the soil nailing wall construction a highly cost-effective choice for this specific project.

5.5 Time consumption comparison of shoring method for each project

The time consumption for different shoring methods can vary depending on various factors such as project size, soil conditions, design requirements, and construction techniques. However, here is a general comparison of the time consumption for two common shoring methods: anchored contiguous pile walls and soil nailing walls.

1. Anchored Contiguous Pile Walls:

-Construction of anchored contiguous pile walls typically involves drilling boreholes, installing steel piles, placing reinforcement cages, and pouring concrete.

- The installation process can be time-consuming as it requires repetitive drilling, pile installation, and concrete pouring.

- Additional time may be needed for curing concrete, and installation of anchor.

- The overall construction time for anchored contiguous pile walls can range from weeks to several months, depending on the complexity and size of the project.

2. Soil Nailing Walls:

- Construction of soil nailing walls involves drilling holes into the ground, inserting reinforcing bars (nails) into the holes, and applying shotcrete or other forms of soil reinforcement directly onto the face of the excavation.

- The drilling and nail installation process is relatively faster compared to the installation of piles.

- Shotcreting or soil reinforcement grout may require some additional time for curing.

The overall construction time for soil nailing walls is generally shorter compared to anchored contiguous pile walls, and it depending on the size and complexity of the project.

It is important to note that these time estimates are approximate and can vary significantly based on project-specific factors. In some cases, site-specific constraints or other project requirements may affect the relative time consumption of each shoring method.

5.5.1 Zemen Bank Project Anchored contiguous pile and Soil Nailing wall work time schedule

TABLE 5-8 ZEMEN BANK ANCHORED CONTIGUOUS PILE TIME SCHEDULE

Owner: ZEMEN Bank Share Company
Project: HQ Building Shoring Pile System and Excavation Works
Contractor: JV Anchorfoundation Sp. PLC and Bamacon Engineering PLC

ID	Task Name	Duration	Start	Finish	Predecessors
1	**ZEMEN BANK S. Co. SHORING SCHEDULE**	**159 days**	**Thu 2/5/15**	**Mon 7/13/15**	
2	**Shoring Pile Installation**	**35 days**	**Thu 2/5/15**	**Wed 3/11/15**	
3	Site Hand Over	1 day	Thu 2/5/15	Thu 2/5/15	
4	Mobilization of staff & equipment	1 day	Fri 2/6/15	Fri 2/6/15	3
5	Assemble equipment & test drill	1 day	Sat 2/7/15	Sat 2/7/15	4
6	Mobilize material to site	7 days	Sun 2/8/15	Sat 2/14/15	5
7	Reinforcement cage fabrication	8 days	Sat 2/7/15	Sat 2/14/15	
8	Drilling & installation of piles	17 days	Thu 2/12/15	Sat 2/28/15	7SS
9	Demobilization of Pile Rig Machine	1 day	Wed 3/11/15	Wed 3/11/15	8
10	**Excavation and Anchor Installation**	**133 days**	**Tue 3/3/15**	**Mon 7/13/15**	
11	Mobilization of Excavation equipment	1 day	Tue 3/3/15	Tue 3/3/15	8
12	First layer Excavation	5 days	Wed 3/4/15	Sun 3/8/15	11
13	Concrete Application	16 days	Thu 3/12/15	Fri 3/27/15	12FS+3 days
14	Mobilization of Anchoring Machine	1 day	Sat 3/28/15	Sat 3/28/15	13
15	Anchor fabrication	8 days	Sun 3/29/15	Sun 4/5/15	14
16	Drilling & installation of first Layer Anchor	21 days	Sun 3/29/15	Sat 4/18/15	14
17	Anchor Grouting and Stressing	11 days	Sun 4/19/15	Wed 4/29/15	16
18	Second Layer Excavation	6 days	Thu 4/30/15	Tue 5/5/15	17
19	Concrete Application	4 days	Wed 5/6/15	Sat 5/9/15	18
20	Anchor Fabrication	9 days	Sun 5/10/15	Mon 5/18/15	19
21	Drilling & Installation of Second Layer Anchor	23 days	Tue 5/19/15	Wed 6/10/15	20
22	Anchor Grouting and Stressing	11 days	Thu 6/11/15	Sun 6/21/15	21
23	Excavation of Third Layer	7 days	Mon 6/22/15	Sun 6/28/15	22
24	Concrete Application	6 days	Mon 6/29/15	Sat 7/4/15	23
25	Demobilization , Anchoring and Excavation Machineries and Equipment	4 days	Sun 7/5/15	Wed 7/8/15	24
26	Backfilling granular fill material,	5 days	Thu 7/9/15	Mon 7/13/15	25
27	**Project Completion Report**	2 days	Tue 7/14/15	Wed 7/15/15	26

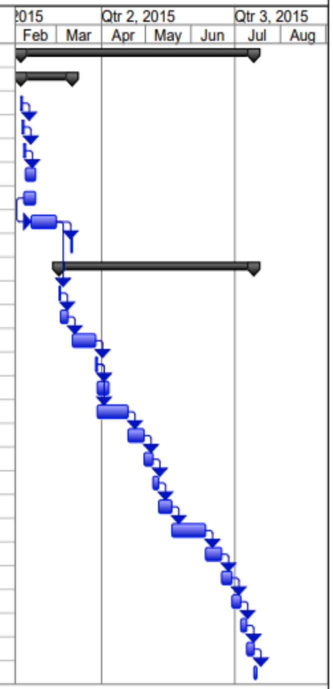
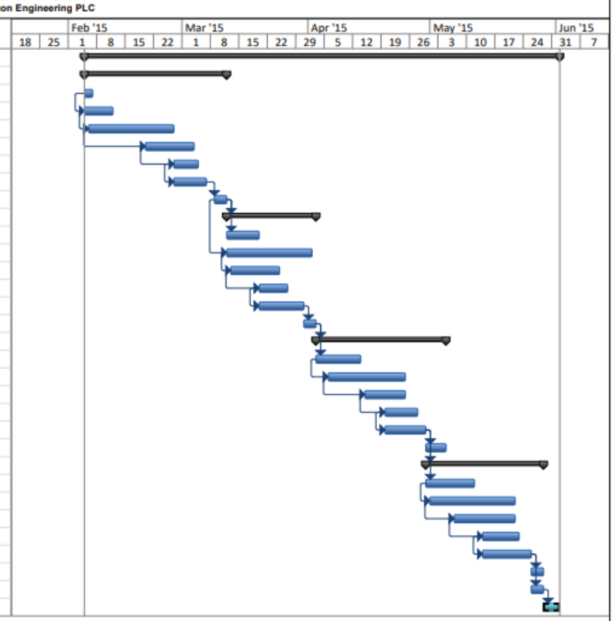


TABLE 5-9 ZEMEN BANK SOIL NAILING WALL TIME SCHEDULE

Owner: ZEMEN Bank Share Company
Project: HQ Building Shoring Pile System and Excavation Works
Contractor: JV Anchorfoundation Sp. PLC and Bamacon Engineering PLC

ID	Task Name	Duration	Start	Finish	Predecessors
1	**ZEMEN BANK S. Co. SHORING SCHEDULE**	**83 days**	**Thu 2/5/15**	**Mon 6/1/15**	
2	**First Layer Soil Nailing Work**	**25 days**	**Thu 2/5/15**	**Wed 3/11/15**	
3	Clear area from top organic soil and vegetation	2 days	Thu 2/5/15	Fri 2/6/15	
4	First layer bulk excavation with a depth of 2000mm from NGL	5 days	Thu 2/5/15	Wed 2/11/15	3SS
5	Drilling and installing nail bar	15 days	Fri 2/6/15	Thu 2/26/15	4SS+1 day
6	Soil Nail Grouting	8 days	Fri 2/20/15	Tue 3/3/15	5SS+10 days
7	Soil Nailing shotcrete wall rebar work	4 days	Fri 2/27/15	Wed 3/4/15	6SS+5 days
8	Application of first layer nailing shotcrete	6 days	Fri 2/27/15	Fri 3/6/15	7SS
9	Curing period of nailing work	3 days	Mon 3/9/15	Wed 3/11/15	8
10	**Second Layer Soil Nailing Work**	**16 days**	**Thu 3/12/15**	**Thu 4/2/15**	**9**
11	Second layer bulk excavation with a depth of 4500mm from NGL	6 days	Thu 3/12/15	Thu 3/19/15	9
12	Drilling and installing nail bar	15 days	Thu 3/12/15	Wed 4/1/15	9SS+1 day
13	Soil Nail Grouting	8 days	Fri 3/13/15	Tue 3/24/15	12SS+1 day
14	Soil Nailing shotcrete Wall rebar work	5 days	Fri 3/20/15	Thu 3/26/15	13SS+5 days
15	Application of second layer nailing shotcrete	7 days	Fri 3/20/15	Mon 3/30/15	14SS
16	Curing period of nailing work	3 days	Tue 3/31/15	Thu 4/2/15	15
17	**Third Layer Soil Nailing Work**	**22 days**	**Fri 4/3/15**	**Mon 5/4/15**	**16**
18	Third layer bulk excavation with a depth of 7000mm from NGL	7 days	Fri 4/3/15	Mon 4/13/15	16
19	Drilling and installing nail bar	15 days	Mon 4/6/15	Fri 4/24/15	18SS+1 day
20	Soil Nail Grouting	8 days	Wed 4/15/15	Fri 4/24/15	19SS+7 days
21	Soil Nailing shotcrete Wall rebar work	6 days	Mon 4/20/15	Mon 4/27/15	20SS+3 days
22	Application of third layer nailing shotcrete	8 days	Mon 4/20/15	Wed 4/29/15	21SS
23	Curing period of nailing work	3 days	Thu 4/30/15	Mon 5/4/15	22
24	**Fourth Layer Soil Nailing Work**	**21 days**	**Thu 4/30/15**	**Thu 5/28/15**	**22**
25	Fourth layer bulk excavation with a depth of 10000mm from NGL	8 days	Thu 4/30/15	Mon 5/11/15	22
26	Drilling and installing nail bar	15 days	Fri 5/1/15	Thu 5/21/15	25SS+1 day
27	Soil Nail Grouting	11 days	Thu 5/7/15	Thu 5/21/15	26SS+4 days
28	Nailing shotcrete Wall rebar work	7 days	Thu 5/14/15	Fri 5/22/15	27SS+5 days
29	Application of Fourth layer nailing shotcrete	8 days	Thu 5/14/15	Mon 5/25/15	28SS
30	Curing period of nailing work	3 days	Tue 5/26/15	Thu 5/28/15	29
31	Demobilization of Equipment from the site	3 days	Tue 5/26/15	Thu 5/28/15	29
32	**Project Completion Repor**	2 days	Fri 5/29/15	Mon 6/1/15	31



The schedule was prepared by gathering input from various companies and taking into account the actual construction time for this project. Upon reviewing the table provided, it is evident that the schedule for the soil nailing wall method is 47.8% lower than that of the anchor contiguous pile wall construction time schedule. This indicates the efficiency and effectiveness of the soil nailing wall method in terms of time management.

Project 1 Discussion

For this particular project, the soil strata consist of two layers: red stiff clay and weathered basalt rock. These soil layers are associated with a low water table. Adjacent to the project site, there is a road and a lightweight house, which does not have a basement. This characteristic makes it easier to conduct soil nailing drilling without any obstacles.

In the context of Addis Ababa, where the project is taking place, soil nailing develops as the preferred choice for sites with similar conditions. The combination of red stiff clay and weathered basalt rock, along with a low water table, is suitable for soil nailing applications. Moreover, the nonexistence of basement structures in the neighboring lightweight building further facilitates the soil nailing process.

Overall, soil nailing proves to be the optimal solution for this specific project due to its cost-effectiveness, time-efficiency, and compatibility with the soil strata and site conditions of Addis Ababa.

FIGURE 5-0-22 ZEMEN BANK PROJECT VIEW

5.5.2 Flintstone Engineering S.C Project Anchored contiguous pile and Soil Nailing wall work time schedule

TABLE 5-10 FLINTSTONE ENGINEERING S.CO ANCHORED CONTIGUOUS PILE TIME SCHEDULE

ID	Task Name	Duration	Start	Finish	Predecessors
1	**FLINTSTONE ENGINEERING S. Co. SHORING SCHED**	**155 days**	**Thu 2/5/15**	**Thu 7/9/15**	
2	**Shoring Pile Installation**	**36 days**	**Thu 2/5/15**	**Thu 3/12/15**	
3	Site Hand Over	1 day	Thu 2/5/15	Thu 2/5/15	
4	Mobilization of staff & equipment	1 day	Fri 2/6/15	Fri 2/6/15	3
5	Assemble equipment & test drill	1 day	Sat 2/7/15	Sat 2/7/15	4
6	Mobilize material to site	7 days	Sun 2/8/15	Sat 2/14/15	5
7	Reinforcement cage fabrication	8 days	Sat 2/7/15	Sat 2/14/15	
8	Drilling & installation of piles	22 days	Thu 2/12/15	Thu 3/5/15	7SS
9	Demobilization of Pile Rig Machine	2 days	Wed 3/11/15	Thu 3/12/15	8
10	**Excavation and Anchor Installation**	**126 days**	**Fri 3/6/15**	**Thu 7/9/15**	
11	Mobilization of Excavation equipment	1 day	Fri 3/6/15	Fri 3/6/15	8
12	First layer Excavation	6 days	Sat 3/7/15	Thu 3/12/15	11
13	Shotcrete Application	10 days	Mon 3/16/15	Wed 3/25/15	12FS+3 days
14	Mobilization of Anchoring Machine	2 days	Thu 3/26/15	Fri 3/27/15	13
15	Anchor fabrication	8 days	Sat 3/28/15	Sat 4/4/15	14
16	Drilling & installation of first Layer Anchor	20 days	Sat 3/28/15	Thu 4/16/15	14
17	Anchor Grouting and Stressing	11 days	Fri 4/17/15	Mon 4/27/15	16
18	Second Layer Excavation	5 days	Tue 4/28/15	Sat 5/2/15	17
19	Shotcrete Application	4 days	Sun 5/3/15	Wed 5/6/15	18
20	Anchor Fabrication	9 days	Thu 5/7/15	Fri 5/15/15	19
21	Drilling & Installation of Second Layer Anchor	23 days	Sat 5/16/15	Sun 6/7/15	20
22	Anchor Grouting and Stressing	12 days	Mon 6/8/15	Fri 6/19/15	21
23	Excavation of Third Layer	10 days	Sat 6/20/15	Mon 6/29/15	22
24	Shotcrete Application	6 days	Tue 6/30/15	Sun 7/5/15	23
25	Demobilization , Anchoring and Excavation Machineries a	4 days	Mon 7/6/15	Thu 7/9/15	24
26	**Project Completion Report**	2 days	Mon 2/24/20	Tue 2/25/20	

TABLE 5-11 FLINTSTONE ENGINEERING S.CO SOIL NAILING WALL TIME SCHEDULE

1	**FLINTSTONE ENGINEERING S. Co. SHORING SCHEDULE**	**129 days**	**Mon 2/24/20**	**Thu 8/20/20**	
2	**First Layer Soil Nailing Work**	**35 days**	**Mon 2/24/20**	**Fri 4/10/20**	
3	Clear area from top organic soil and vegetation	3 days	Mon 2/24/20	Wed 2/26/20	
4	First layer bulk excavation with a depth of 1500mm from NGL	5 days	Mon 2/24/20	Fri 2/28/20	3SS
5	Drilling and installing nail bar	20 days	Tue 2/25/20	Mon 3/23/20	4SS+1 day
6	Soil Nail Grouting	15 days	Tue 3/10/20	Mon 3/30/20	5SS+10 days
7	Soil Nailing shotcrete wall rebar work	6 days	Mon 3/23/20	Mon 3/30/20	6SS+9 days
8	Application of first layer nailing shotcrete	7 days	Thu 3/26/20	Fri 4/3/20	7SS+3 days
9	Curing period of nailing work	5 days	Mon 4/6/20	Fri 4/10/20	8
10	**Second Layer Soil Nailing Work**	**31 days**	**Mon 4/13/20**	**Mon 5/25/20**	**9**
11	Second layer bulk excavation with a depth of 3500mm from NGL	8 days	Mon 4/13/20	Wed 4/22/20	9
12	Drilling and installing nail bar	22 days	Mon 4/13/20	Tue 5/12/20	9SS+1 day
13	Soil Nail Grouting	15 days	Thu 4/23/20	Wed 5/13/20	12SS+8 days
14	Soil Nailing shotcrete Wall rebar work	7 days	Thu 5/7/20	Fri 5/15/20	13SS+10 days
15	Application of second layer nailing shotcrete	7 days	Fri 5/8/20	Mon 5/18/20	14SS+1 day
16	Curing period of nailing work	5 days	Tue 5/19/20	Mon 5/25/20	15
17	**Third Layer Soil Nailing Work**	**30 days**	**Tue 5/26/20**	**Mon 7/6/20**	**16**
18	Third layer bulk excavation with a depth of 4500mm from NGL	9 days	Tue 5/26/20	Fri 6/5/20	16
19	Drilling and installing nail bar	19 days	Wed 5/27/20	Mon 6/22/20	18SS+1 day
20	Soil Nail Grouting	11 days	Tue 6/9/20	Tue 6/23/20	19SS+9 days
21	Soil Nailing shotcrete Wall rebar work	6 days	Thu 6/18/20	Thu 6/25/20	20SS+7 days
22	Application of third layer nailing shotcrete	8 days	Thu 6/18/20	Mon 6/29/20	21SS
23	Curing period of nailing work	5 days	Tue 6/30/20	Mon 7/6/20	22
24	**Fourth Layer Soil Nailing Work**	**33 days**	**Tue 7/7/20**	**Thu 8/20/20**	**23**
25	Fourth layer bulk excavation with a depth of 7000mm from NGL	8 days	Tue 7/7/20	Thu 7/16/20	23
26	Drilling and installing nail bar	17 days	Wed 7/8/20	Thu 7/30/20	25SS+1 day
27	Soil Nail Grouting	10 days	Fri 7/17/20	Thu 7/30/20	26SS+7 days
28	Nailing shotcrete Wall rebar work	6 days	Fri 7/24/20	Fri 7/31/20	27SS+5 days
29	Application of Fourth layer nailing shotcrete	8 days	Fri 7/24/20	Tue 8/4/20	28SS
30	Curing period of nailing work	5 days	Wed 8/5/20	Tue 8/11/20	29
31	**Fifth Layer Soil Nailing Work**	**24 days**	**Mon 7/20/20**	**Thu 8/20/20**	**30**
32	Fourth layer bulk excavation with a depth of 10000mm from NGL	10 days	Wed 8/5/20	Tue 8/18/20	30
33	Drilling and installing nail bar	15 days	Mon 7/20/20	Fri 8/7/20	29SS+1 day
34	Soil Nail Grouting	9 days	Tue 7/28/20	Fri 8/7/20	33SS+6 days
35	Nailing shotcrete Wall rebar work	7 days	Fri 7/31/20	Mon 8/10/20	34SS+3 days
36	Application of Fourth layer nailing shotcrete	9 days	Mon 8/3/20	Thu 8/13/20	35SS+1 day
37	Curing period of nailing work	5 days	Fri 8/14/20	Thu 8/20/20	36
38	Demobilization of Equipment from the site	2 days	Fri 8/14/20	Mon 8/17/20	36
39	**Project Completion Repor**	2 days	Tue 8/18/20	Wed 8/19/20	38

Based on the table provided, it is obvious that the time schedule for the soil nailing wall method is 16% lower compared to the anchor contiguous pile wall construction time schedule.

Project 2 Site Discussion

The soil conditions for this project are considered by expansive soil at the top and stiff silty clay after a depth of 10m to 15m, soil nailing is not a best method as it requires additional resources and effort to stabilize the soil layers. The preferred option in such soil formations is the contiguous pile method with anchoring, as it enhances stability and strength to withstand the soil challenges. This method is also believed more cost-effective and space-efficient than soil nailing, making it a more practical and economically feasible option for this particular project. Overall, the bored pile method with anchoring provides the necessary stability and safety required for this like project.

FIGURE 5-23 FLINTSTONE PROJECT VIEW

5.5.3 Yewub Hagere Group S.C Project Anchored contiguous pile and Soil Nailing wall work time schedule

TABLE 5-12 YEWUB HAGERE GROUP S.CO ANCHORED CONTIGUOUS PILE CONSTRUCTION TIME SCHEDULE

ID	Task Name	Duration	Start	Finish	Predecessors
1	**YEWUB HAGERE GROUP S.C SHORING SCHEDULE**	**77 days**	**Thu 6/10/21**	**Wed 8/25/21**	
2	**Shoring Pile Installation**	**26 days**	**Thu 6/10/21**	**Mon 7/5/21**	
3	Site Hand Over	1 day	Thu 6/10/21	Thu 6/10/21	
4	Mobilization of staff & equipment	1 day	Fri 6/11/21	Fri 6/11/21	3
5	Assemble equipment & test drill	1 day	Sat 6/12/21	Sat 6/12/21	4
6	Mobilize material to site	2 days	Sun 6/13/21	Mon 6/14/21	5
7	Reinforcement cage fabrication	6 days	Tue 6/15/21	Sun 6/20/21	6
8	Drilling & installation of piles	19 days	Tue 6/15/21	Sat 7/3/21	7SS
9	Demobilization of Pile Rig Machine	2 days	Sun 7/4/21	Mon 7/5/21	8
10	**Excavation and Anchor Installation**	**41 days**	**Tue 7/6/21**	**Sun 8/15/21**	**9**
11	Mobilization of Excavation equipment	1 day	Tue 7/6/21	Tue 7/6/21	9
12	First layer Excavation	4 days	Wed 7/7/21	Sat 7/10/21	11
13	Shotcrete Application	5 days	Fri 7/9/21	Tue 7/13/21	12SS+2 days
14	Mobilization of Anchoring Machine	2 days	Wed 7/14/21	Thu 7/15/21	13
15	Anchor fabrication	6 days	Fri 7/16/21	Wed 7/21/21	14
16	Drilling & installation of first Layer Anchor	12 days	Fri 7/16/21	Tue 7/27/21	14
17	Anchor Grouting and Stressing	8 days	Thu 7/22/21	Thu 7/29/21	16SS+6 days
18	Second Layer Excavation	7 days	Fri 7/30/21	Thu 8/5/21	17
19	Shotcrete Application	6 days	Fri 8/6/21	Wed 8/11/21	18
20	Demobilization , Anchoring and Excavation Machineries and Equipment	4 days	Thu 8/12/21	Sun 8/15/21	19
21	**Project Completion Report**	2 days	Mon 8/16/21	Tue 8/17/21	20

TABLE 5-13 YEWUB HAGERE GROUP S.CO SOIL NAILING WALL CONSTRUCTION TIME SCHEDULE

ID	Task Name	Duration	Start	Finish	Predecessors
1	**Yewub Hagere Group SC SHORING SCHEDULE**	**29 days**	**Thu 6/10/21**	**Tue 7/20/21**	
2	**First Layer Soil Nailing Work**	**11 days**	**Thu 6/10/21**	**Thu 6/24/21**	
3	Clear area from top organic soil and vegetation	3 days	Thu 6/10/21	Mon 6/14/21	
4	First layer bulk excavation with a depth of 2000mm from NGL	4 days	Thu 6/10/21	Tue 6/15/21	3SS
5	Drilling and installing nail bar	6 days	Fri 6/11/21	Fri 6/18/21	4SS+1 day
6	Soil Nail Grouting	4 days	Tue 6/15/21	Fri 6/18/21	5SS+2 days
7	Soil Nailing shotcrete wall rebar work	4 days	Wed 6/16/21	Mon 6/21/21	6SS+1 day
8	Application of first layer nailing shotcrete	2 days	Fri 6/18/21	Mon 6/21/21	7SS+2 days
9	Curing period of nailing work	3 days	Tue 6/22/21	Thu 6/24/21	8
10	**Second Layer Soil Nailing Work**	**17 days**	**Fri 6/25/21**	**Mon 7/19/21**	**9**
11	Second layer bulk excavation with a depth of 4000mm from NGL	5 days	Fri 6/25/21	Thu 7/1/21	9
12	Drilling and installing nail bar	6 days	Fri 6/25/21	Fri 7/2/21	9SS+1 day
13	Soil Nail Grouting	5 days	Tue 6/29/21	Mon 7/5/21	12SS+2 days
14	Soil Nailing shotcrete Wall rebar work	4 days	Thu 7/1/21	Tue 7/6/21	13SS+2 days
15	Application of second layer nailing shotcrete	3 days	Fri 7/2/21	Tue 7/6/21	14SS+1 day
16	Curing period of nailing work	3 days	Wed 7/7/21	Fri 7/9/21	15
17	Third layer bulk excavation with a depth of 8000mm from NGL	6 days	Mon 7/12/21	Mon 7/19/21	16
18	**Project Completion Repor**	1 day	Tue 7/20/21	Tue 7/20/21	17

<u>Project 3 Discussion</u>

The soil composition for a project includes non-cohesive topsoil and a layer of weathered basaltic rock at a depth of 5 meters, with a low water table. The most cost-effective and efficient method for shoring in these site conditions is soil nailing. The soil nailing provides a practical alternative that requires fewer materials, equipment, and labor for reduced costs and construction time compere to contiguous anchor pile wall. Soil nailing is useful, adaptable, and suitable for retaining soil masses, preventing slope failures, and stabilizing excavations.

FIGURE 5-24 YEWUB HAGERE PROJECT SOIL NAILING WALL AND ANCHOR BORED PILE WALL VIEW

Result summery

TABLE 5-14 RESULT SUMMERY

		Project 1.		Project 2.		Project 3	
	Unit	A. Pile wall	S. nailing wall	A. Pile wall	S. nailing wall	A. Pile wall	S. nailing wall
Excavation depth	m	10		10		8	
Structure height	m	15	10	16	10	12	5
Surcharge load	kN/m²	16(Main Road)		50 (G+5)			
Max. shear stress	kN/m²	259	156.8	79.18	45.12	115.4	135.7
Min. shear stress	kN/m²	-19.8	-21.26	-22.41	-25.42	115.4	135.7
Maximum lateral displacement	mm	3.57	7.028	4.21	1.01	2.87	8.86
Minimum lateral displacement	mm	-2.22	-2.064	-5.22	-0.7	-2.868	-1.17
Max. Shear force	kN/m	60.8	37.88	99.18	5.369	76.9	20
Min. Shear force	kN/m	-105.7	-28.36	-75.36	-9.812	-76.86	-33.59
Max Bending Moment	kNm/m	36.13	3.451	89.23	1.294	48.18	19.54
Min. Bending Moment	kNm/m	-52.82	-5.706	-49.23	-1.49	-48.2	-15.08
Estimated project total cost	Birr	37.5 Mil	17.2 Mil	42.3 Mil	70.3 Mil	13.2 Mil	3.7 Mil
Difference	%		54	40			71%
Estimated project construction time schedule	Days	159	83	155	129	77	29
Difference	%		48		16		62

CHAPTER SIX

6. CONCLUSIONS AND RECOMMENDATIONS

Upon analyzing the comparative assessment data between soil nailing walls and contiguous anchor pile walls, and reviewing the study results, the following conclusions and recommendations have been made.

6.1 Conclusions

The selection of shoring system for a construction project is heavily dependent on various factors such as soil strength, adjacent structures, and availability of machinery and equipment in the country. In this study, a detailed study was conducted to assess the available shoring methods in Ethiopia, specifically in Addis Ababa. The aim was to compare two shoring methods, soil nailing wall, and contiguous pile, in different site conditions to determine their effectiveness in terms of cost and time.

One of the major findings of the study was that local companies tend to use the same shoring method for different soil types due to a lack of appropriate machinery and equipment, limited experience on the field, and little awareness of other shoring options. However, it was found that for stiff cohesive soils, dense non-cohesive soils, and rock strata, the soil nailing method reduces costs by 51-70% in birr compared to the conventional anchored contiguous pile method. Conversely, for expansive soils and soft silty clays, the anchored contiguous pile method is more economical, saving 30-40% in birr relative to the soil nailing wall method. Additionally, the study indicates that the soil nailing wall method is more time-efficient than the anchored contiguous pile wall construction approach.

In summary, the study highlights the importance of choosing the right shoring system for a construction project based on the specific site conditions and availability of machinery and equipment. The findings of the study can help local companies in Ethiopia to make informed decisions that could lead to cost and time savings in their shoring operations.

The bending moment value of the contiguous anchor pile wall obtained from the LEM (Limit Equilibrium Method) is observed to be 11-26% higher compared to the result obtained from the 2D FEM (Finite Element Method). Similarly, the bending moment value obtained from PLAXIS 2D greater than the value obtained from PLAXIS 3D.When analyzing the horizontal displacement value of the contiguous anchor pile wall, it is seen that the value obtained from PLAXIS 3D is lower than both PLAXIS 2D and the LEM. However, these differences in horizontal displacement are not significant, and all three methods produce results that are practically equivalent. This implies that, although there are slight variations in the numerical values, the overall behavior and performance of the contiguous anchor pile wall are nearly the same across the three methods.

The bending moment of the soil nailing wall is around 23% higher in the LEM compared to PLAXIS 2D, and PLAXIS 2D exhibits a higher bending moment compared to PLAXIS 3D. The horizontal displacements of the soil nailing wall project show no significant difference among the LEM, PLAXIS 2D, and PLAXIS 3D methods

6.2 Recommendations

The recommendations below offer valuable insights that can greatly contribute to the improvement and expansion of this work in the future.

- One important aspect to consider is the development of standards for shoring by government institutions that are specifically tailored to our country's unique needs. Having such standards in place ensures that the design and execution of shoring projects are of the highest quality, ultimately leading to better outcomes.
- Another important factor to take into account is the necessity of conducting soil investigation for this specific type of work. By thoroughly assessing the properties and characteristics of the soil they make informed decisions regarding the most appropriate shoring methods to employ. This step is essential in order to guarantee the stability and safety of the structures being supported.
- Furthermore, when considering various shoring methods, it is worth noting that the use of additional machinery can greatly develop efficiency and cost-effectiveness. For instance, using a sheet pile machine or a diaphragm wall drilling machine can significantly reduce both expenses and construction time. These machines offer alternative techniques that can be utilized to achieve optimal results while minimizing resource utilization.
- Lastly, it is important to highlight that temporary shoring methods can be transformed into permanent solutions by establishing connections with the main building. This approach not only provides a practical and efficient solution.

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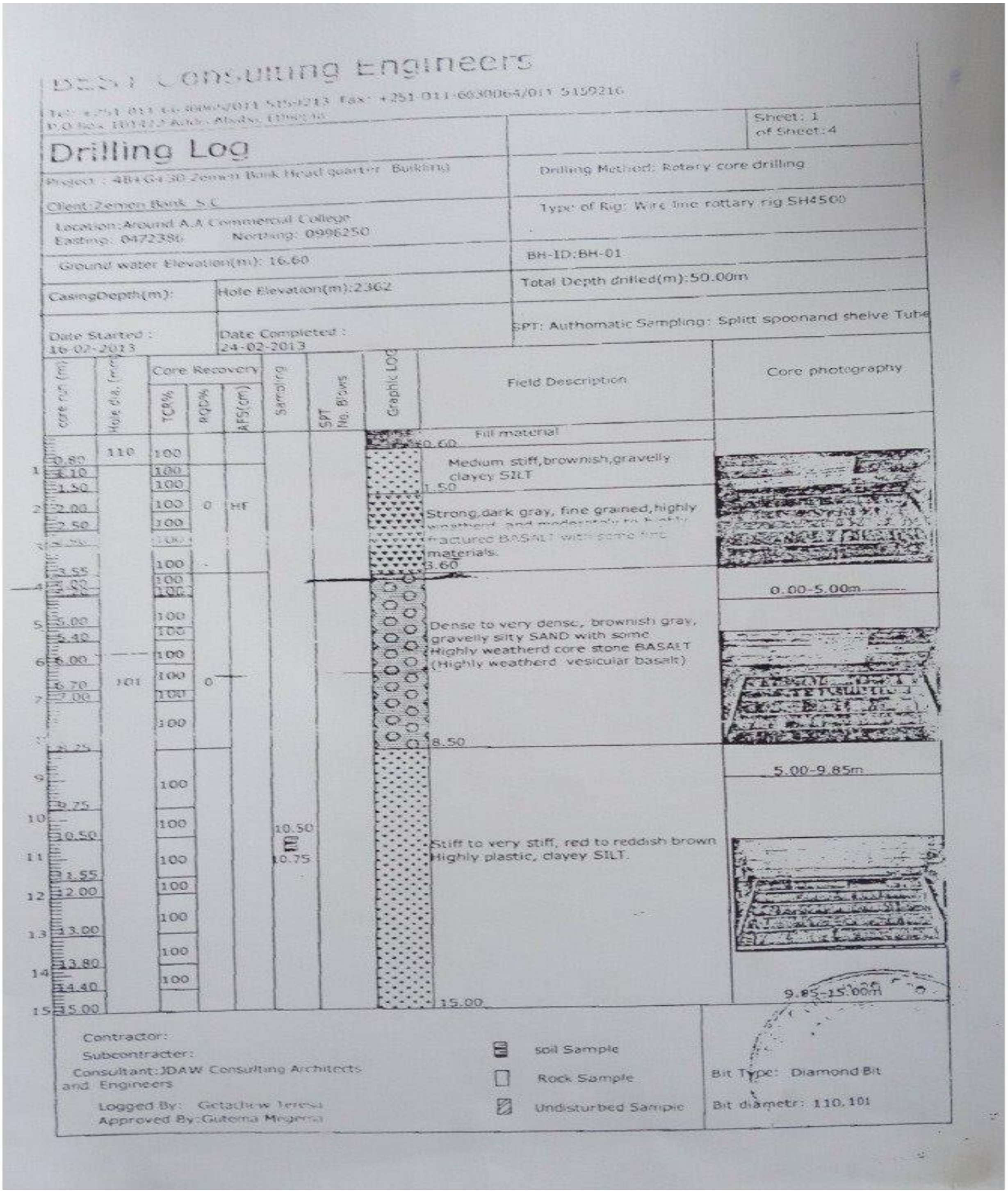
APPENDIXES

Appendix 1: Soil Investigation Report

A.1.1 Project 1(Zemen Bank @Mexico) Borehole log


BEST Consulting Engineers

Drilling Log

Sheet: 1 of Sheet: 4

Project : 4B+G+30 Zemen Bank Head quarter Building

Drilling Method: Rotary core drilling

Client: Zemen Bank S.C

Type of Rig: Wire line rottary rig SH4500

Location: Around A.A Commercial College

Easting: 0472386 Northing: 0996250

Ground water Elevation(m): 16.60

BH-ID: BH-01

Total Depth drilled(m): 50.00m

Casing Depth(m):

Hole Elevation(m): 2362

Date Started: 16-02-2013

Date Completed: 24-02-2013

SPT: Authomatic Sampling: Splitt spoonand shelve Tube

core run (m)	Hole dia. (mm)	TCR%	RQD%	AFS(cm)	Sampling	SPT No. Blows	Graphic LOG	Field Description	Core photography
0.80	110	100					0.60	Fill material	
1.10		100						Medium stiff, brownish, gravelly clayey SILT	
1.50		100					1.50		
2.00		100	0	HF				Strong, dark gray, fine grained, highly weathered and moderately to highly fractured BASALT with some fine materials.	
2.50		100							
3.55		100					3.60		0.00-5.00m
3.85		100							
5.00		100						Dense to very dense, brownish gray, gravelly silty SAND with some Highly weatherd core stone BASALT (Highly weatherd vesicular basalt)	
5.40		100							
6.00		100							
6.70	101	100	0						
7.00		100							
8.25		100					8.50		5.00-9.85m
9.75		100							
10.50		100			10.50–10.75			Stiff to very stiff, red to reddish brown Highly plastic, clayey SILT.	
11.55		100							
12.00		100							
13.00		100							
13.80		100							
14.40		100							
15.00							15.00		9.85-15.00m

Contractor:
Subcontracter:
Consultant: JDAW Consulting Architects and Engineers
Logged By: Getachew Teresa
Approved By: Gutema Megersa

Soil Sample
Rock Sample
Undisturbed Sample

Bit Type: Diamond Bit
Bit diametr: 110, 101

FIGURE 0-1 ZEMEN BANK BOREHOLE LOG SHEET

A.1.2 Project 2(Flintstone homes@ Bole) Borehole log

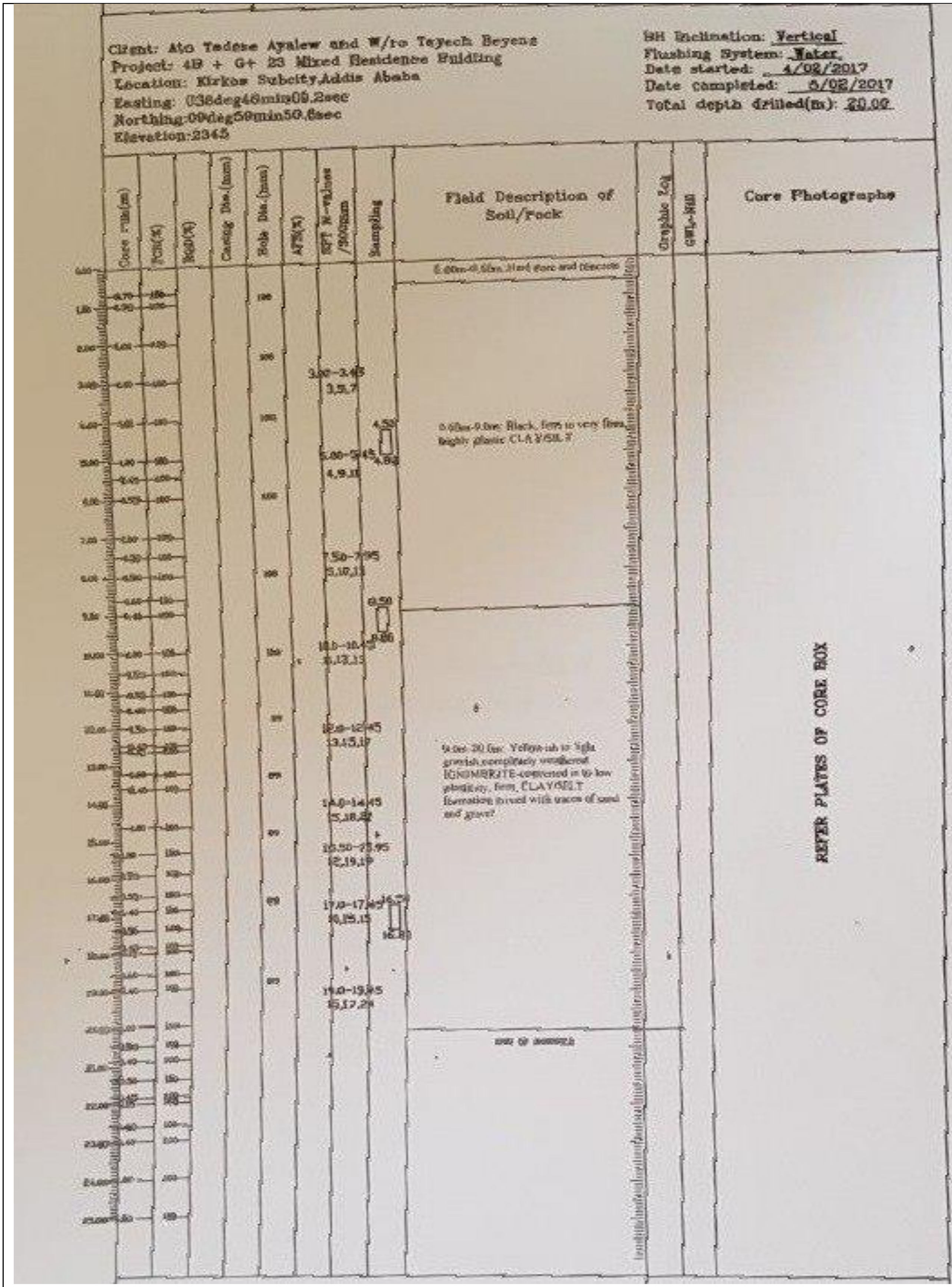


FIGURE 0-2 FLINTSTONE HOMES BOLEHOLE LOG SHEET

A.1.3 Project 3(Wubhagere @Kera Buligaria) Borehole log

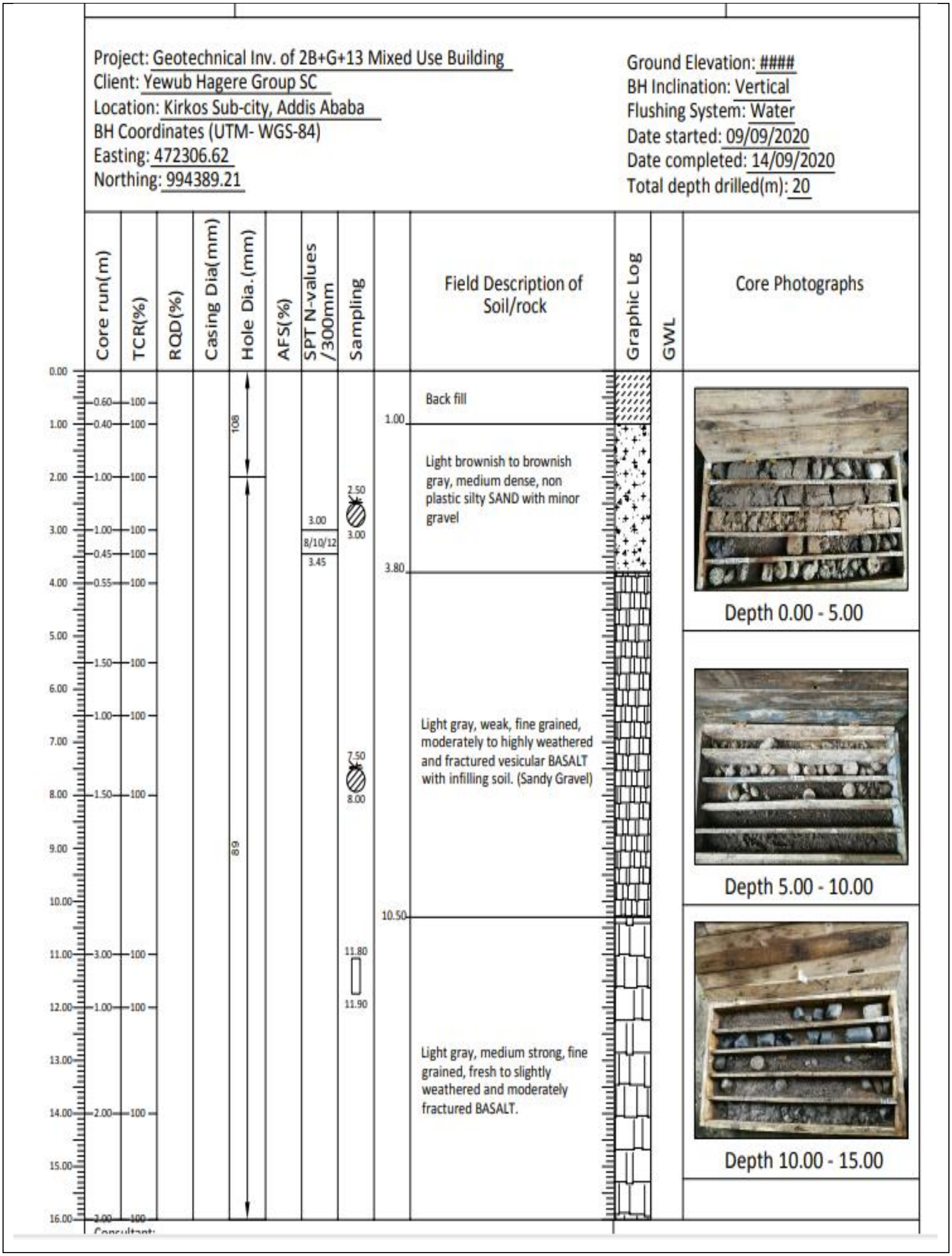


FIGURE 0-3 WUBHAGERE BOREHOLE LOG SHEET

Appendix 2 : PLAXIS and GOE5 Software Output

A.2.1 Project 1(Zemen Bank) Anchored contiguous pile Analysis results

Calculation results, Plate, Final Excavation [Phase_5] , Shear forces Q

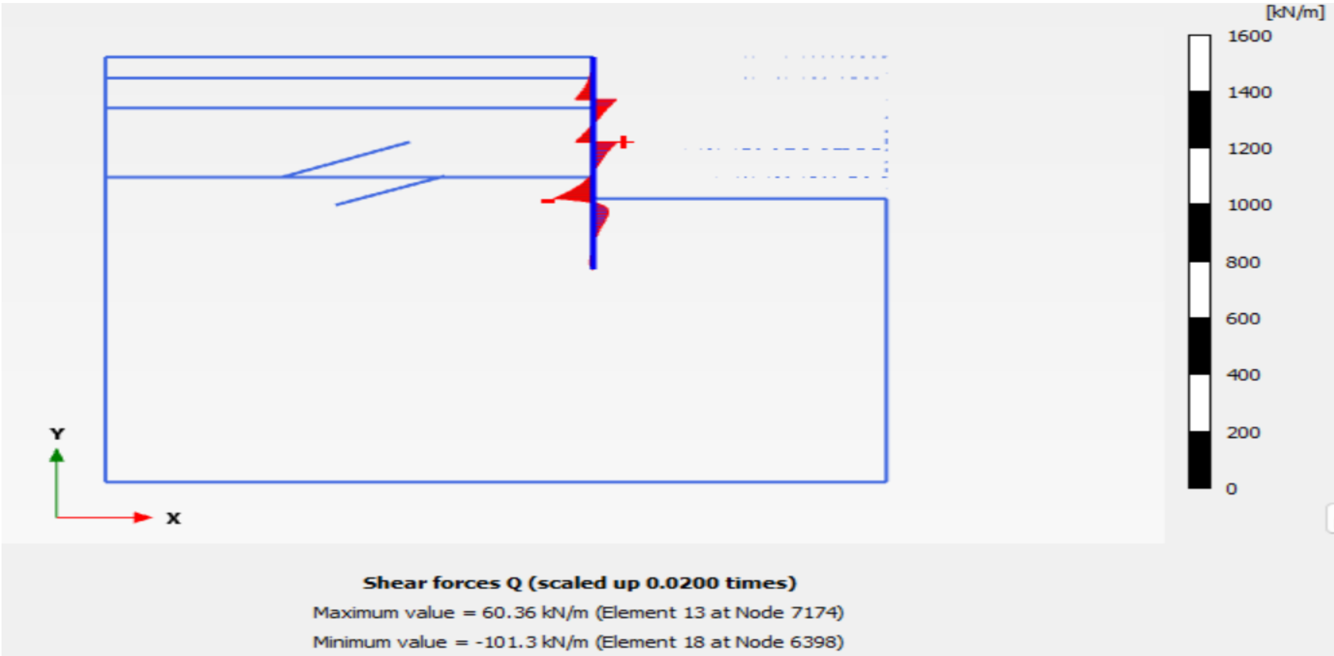


FIGURE 0-4 ZEMEN BANK ANCHORED CONTIGUOUS WALL SHEAR FORCE DIAGRAM

Calculation results, Plate, Final Excavation [Phase_5] , Axial forces N

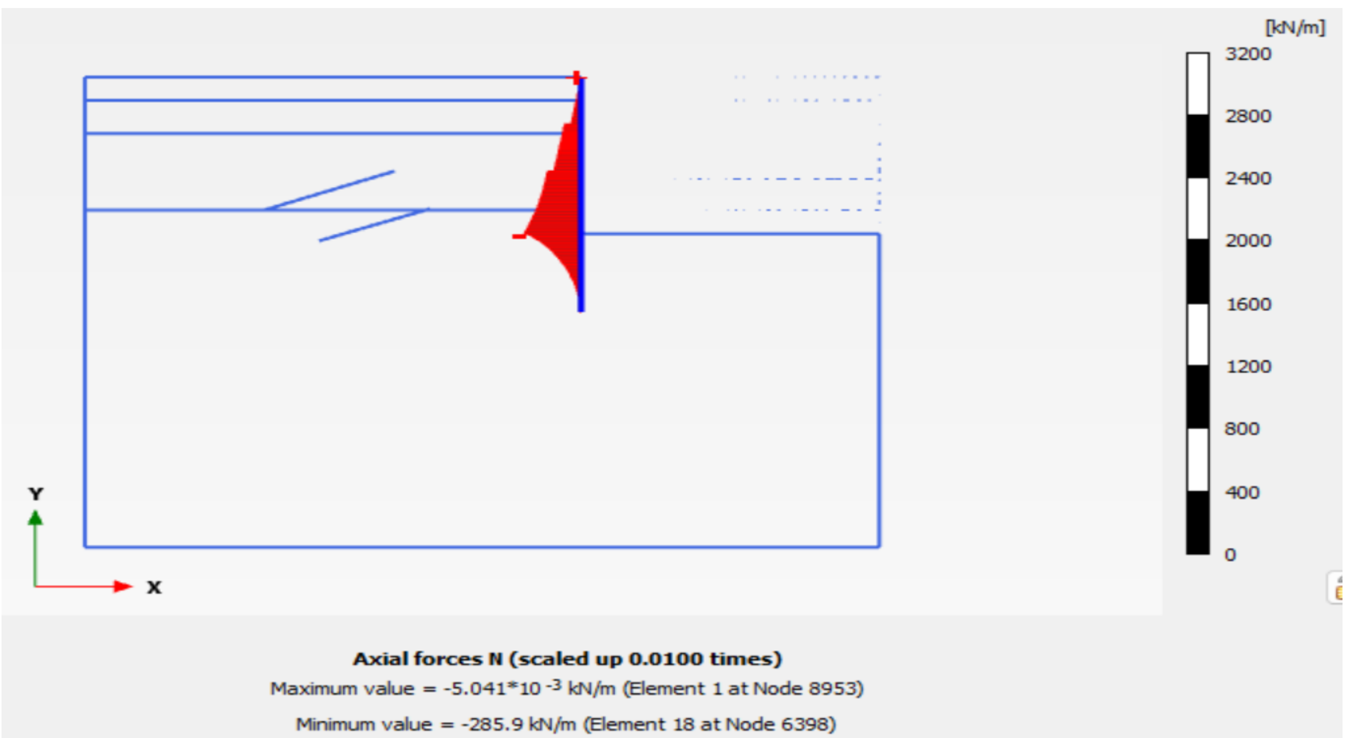


FIGURE 0-5 ZEMEN BANK CONTIGUOUS PILE AXIAL FORCE DIAGRAM

Table 0-6 Zemen Bank contiguous pile analysis by using limit Equilibrium method

	Disp. min [mm]	**Disp. max [mm]**	**Shear force min. [kN/m]**	**Shear force max [kN/m]**	**Moment min. [kNm/m]**	**Moment max. [kNm/m]**
0.00	-3.49	-3.43	-0.00	-0.00	-0.00	0.00
0.75	-2.88	-2.74	-4.93	-1.01	0.25	1.56
1.50	-2.28	-2.07	-15.05	-4.05	2.02	8.49
2.25	-1.68	-1.44	-34.34	-9.22	6.86	26.79
3.00	-1.13	-0.95	-54.94	-16.65	16.42	60.46
3.00	-1.13	-0.95	-16.65	50.51	16.42	60.46
3.50	-0.80	-0.76	-22.80	39.55	26.17	38.72
3.50	-0.80	-0.75	-22.71	39.41	26.35	38.42
3.75	-0.74	-0.66	-12.56	35.74	26.00	30.68
4.50	-0.66	-0.34	8.42	23.73	5.96	31.28
5.25	-0.63	-0.18	-5.88	15.62	-5.74	21.25
6.00	-0.57	-0.12	-34.19	12.01	-6.64	16.03
6.50	-0.51	-0.12	-52.96	8.01	0.03	37.82
6.50	-0.51	-0.12	-53.24	7.95	0.19	38.25
6.75	-0.48	-0.13	-61.45	6.05	3.78	52.30
7.00	-0.51	-0.13	-68.49	4.34	2.51	68.68
7.00	-0.51	-0.13	-11.09	88.12	2.51	68.68
7.50	-0.72	-0.15	-3.25	73.56	1.02	28.22
8.25	-1.14	-0.18	0.74	49.74	-18.17	9.89
9.00	-1.48	-0.21	0.48	15.58	-43.74	3.78

	Disp. min [mm]	Disp. max [mm]	Shear force min. [kN/m]	Shear force max [kN/m]	Moment min. [kNm/m]	Moment max. [kNm/m]
9.75	-1.60	-0.24	-26.35	3.40	-40.00	-0.00
10.00	-1.60	-0.25	-41.13	2.53	-31.71	-0.46
10.00	-1.59	-0.25	-41.30	2.50	-31.38	-0.46
10.50	-1.53	-0.27	-29.32	1.10	-13.92	-0.43
11.25	-1.39	-0.30	-13.18	-0.16	-1.92	1.66
12.00	-1.25	-0.33	-2.80	-0.15	-1.57	7.30
12.75	-1.14	-0.35	-0.76	2.60	-1.01	7.10
13.50	-1.07	-0.37	-0.61	4.28	-0.49	4.32
14.25	-1.03	-0.40	-0.34	3.25	-0.13	1.35
15.00	-0.99	-0.42	-0.00	0.00	-0.00	-0.00

A.2.2 Project 1(Zemen Bank) Soil Nailing Wall Analysis results

Calculation results, Plate, 4th Excavation [Phase_7] , Shear forces Q

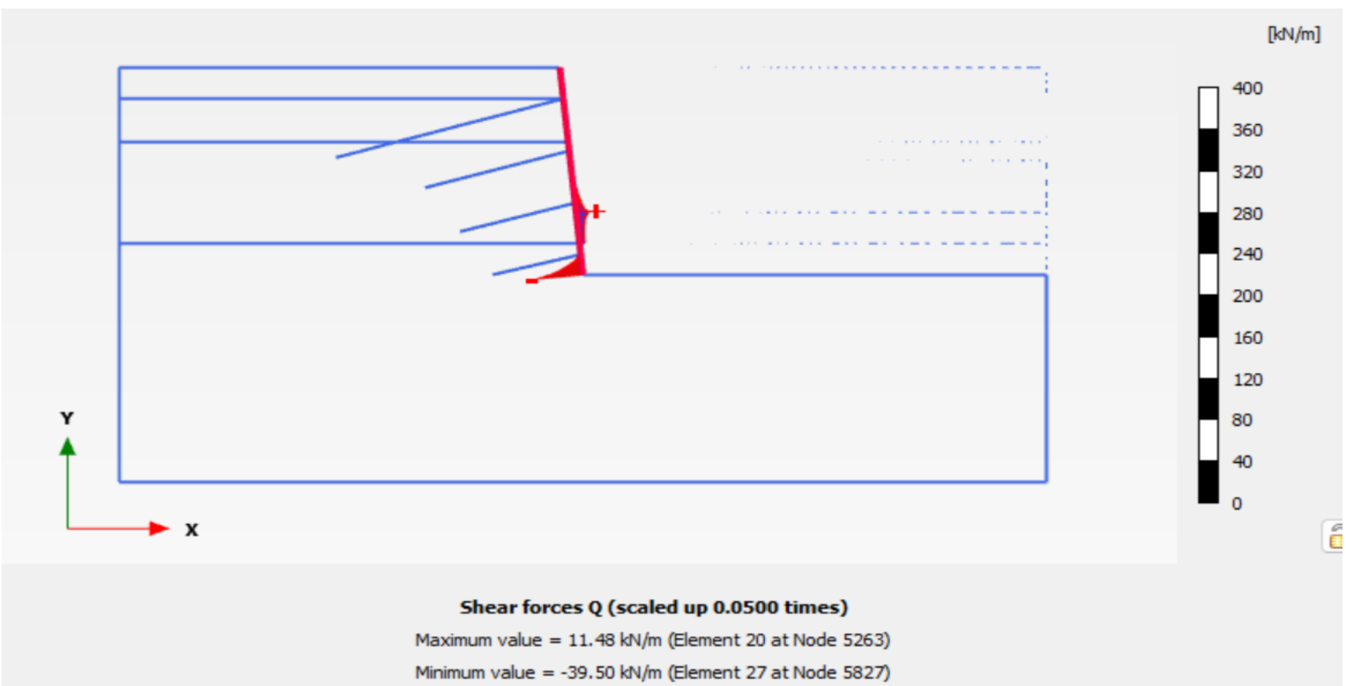


FIGURE 0-6 ZEMEN BANCK SOIL NAILING WALL SHEAR FORCE DIAGRAM

Calculation results, Plate, 4th Excavation [Phase_7] , Axial forces N

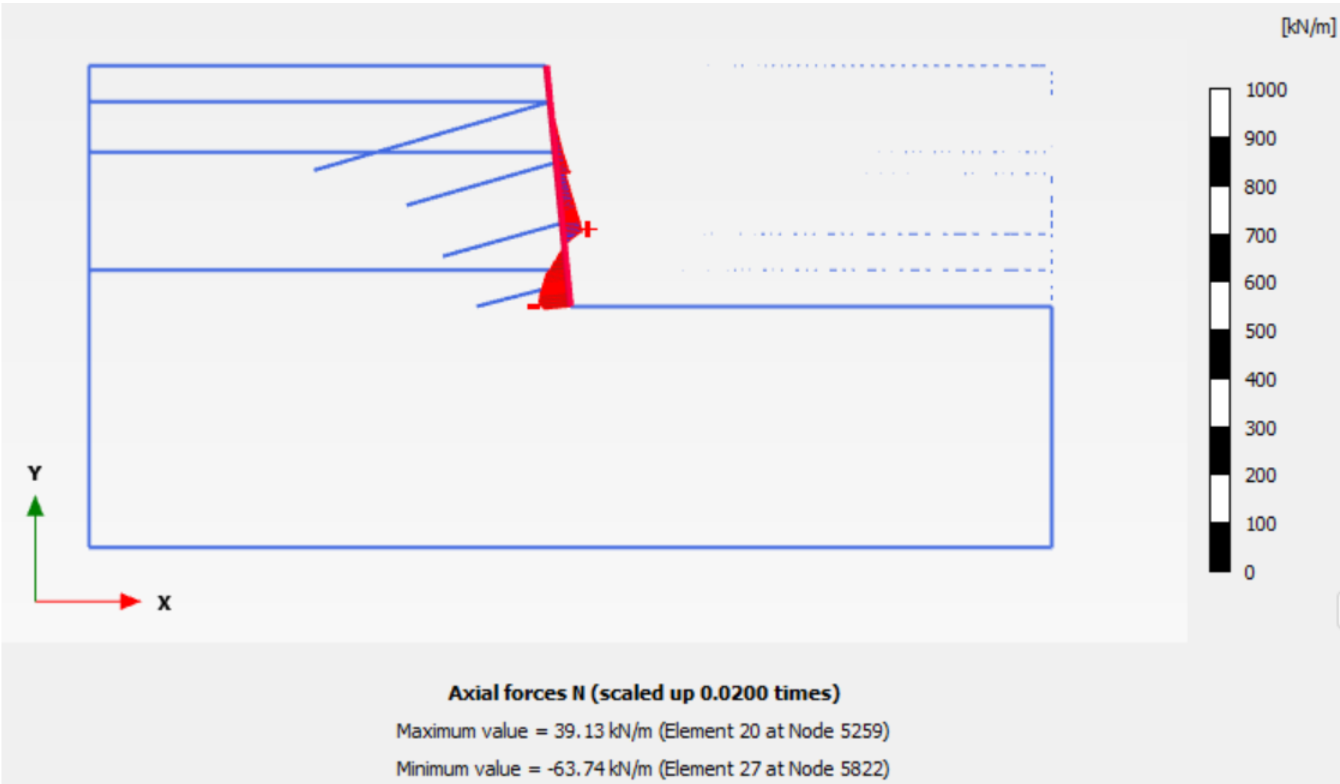


FIGURE 0-7 ZEMEN BANK SOIL NAILING WALL AXIAL FORCE DIAGRAM

A.2.3 Project 2(Flintstone Engineering S.C) Anchored Contiguous pile Analysis results

Calculation results, Plate, last excavation [Phase_7], Shear forces Q

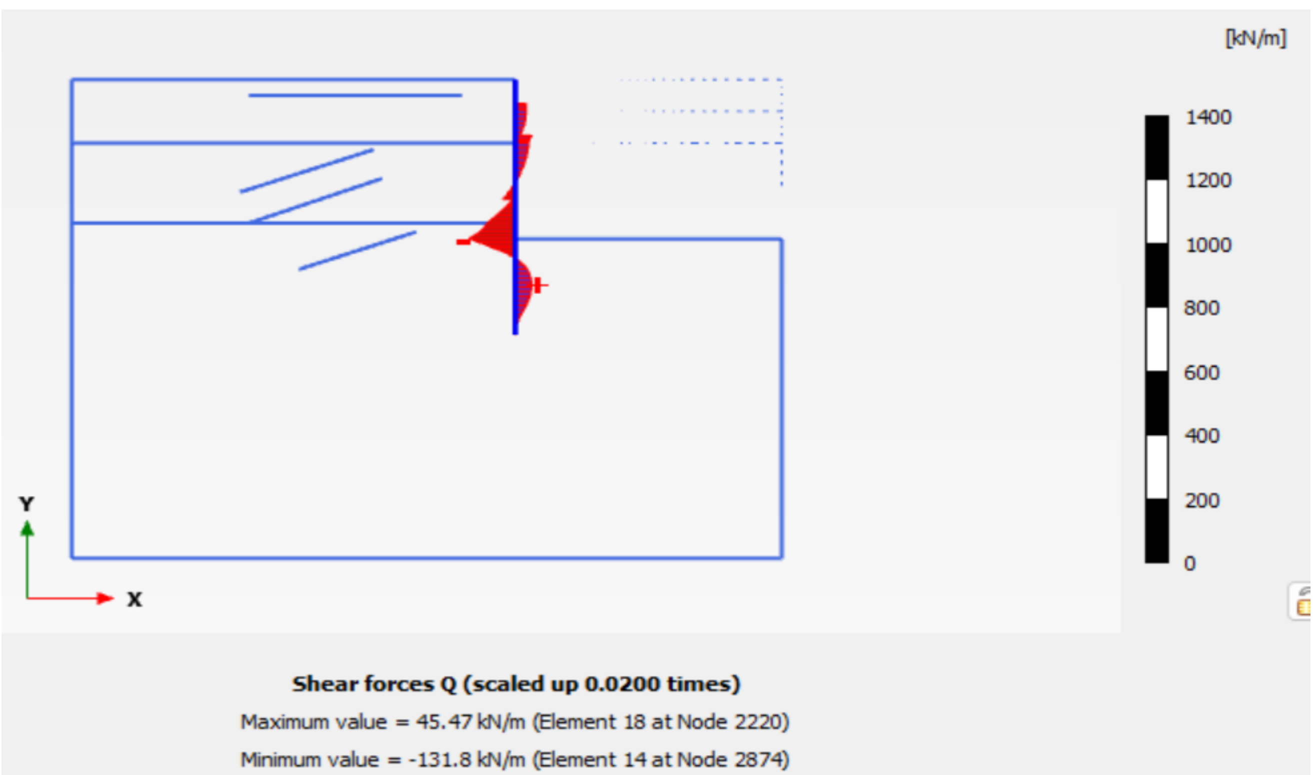


FIGURE 0-8 FLINTSTONE ENGINEERIND S.CO ANCHORED CONTIGUOUS PILE SHEAR FORCE DIAGRAM

Calculation results, Plate, last excavation [Phase_7] , Axial forces N

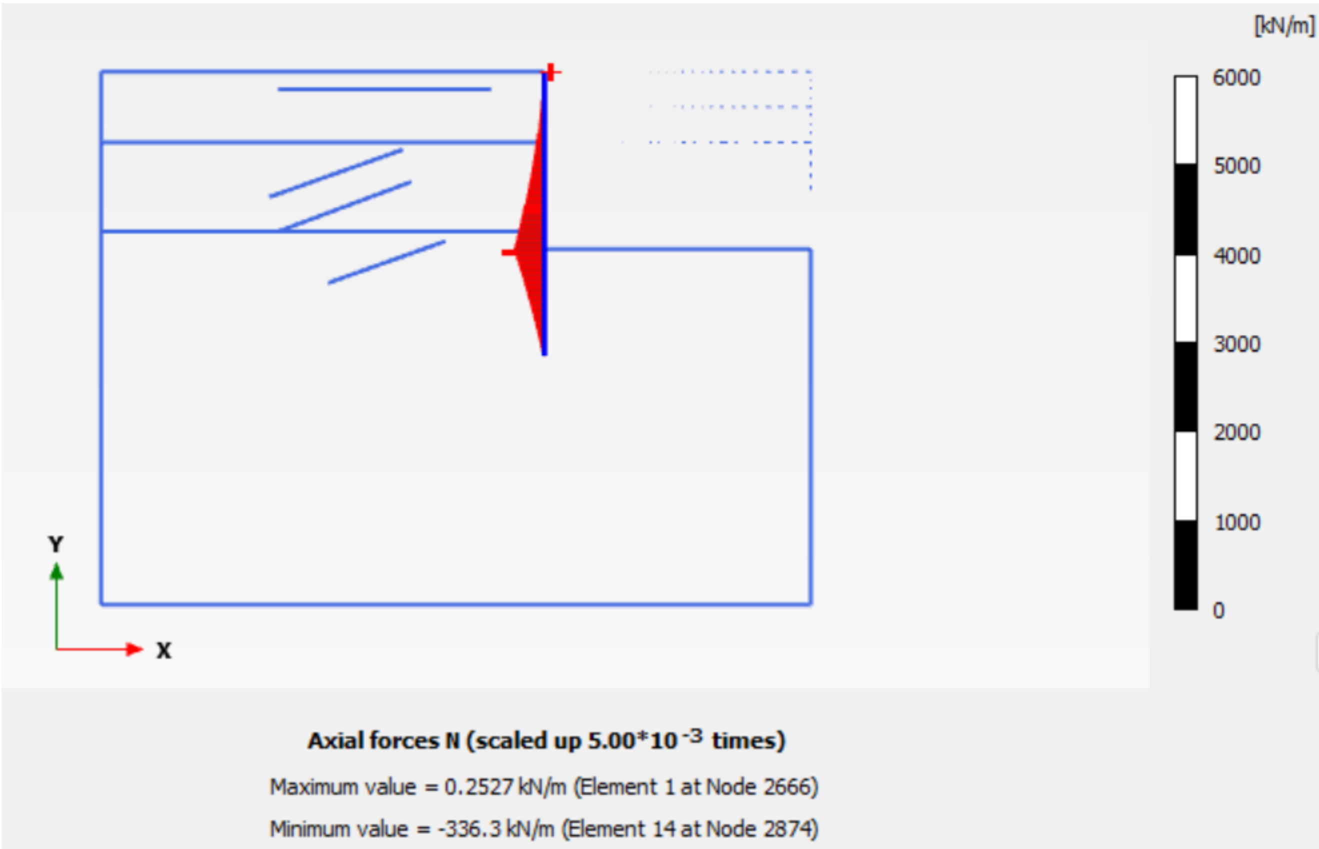


FIGURE 0-9 FLINTSTONE ENGINEERING S.CO ANCHORED CONTIGUOUS PILE AXIAL FORCE DIAGRAM

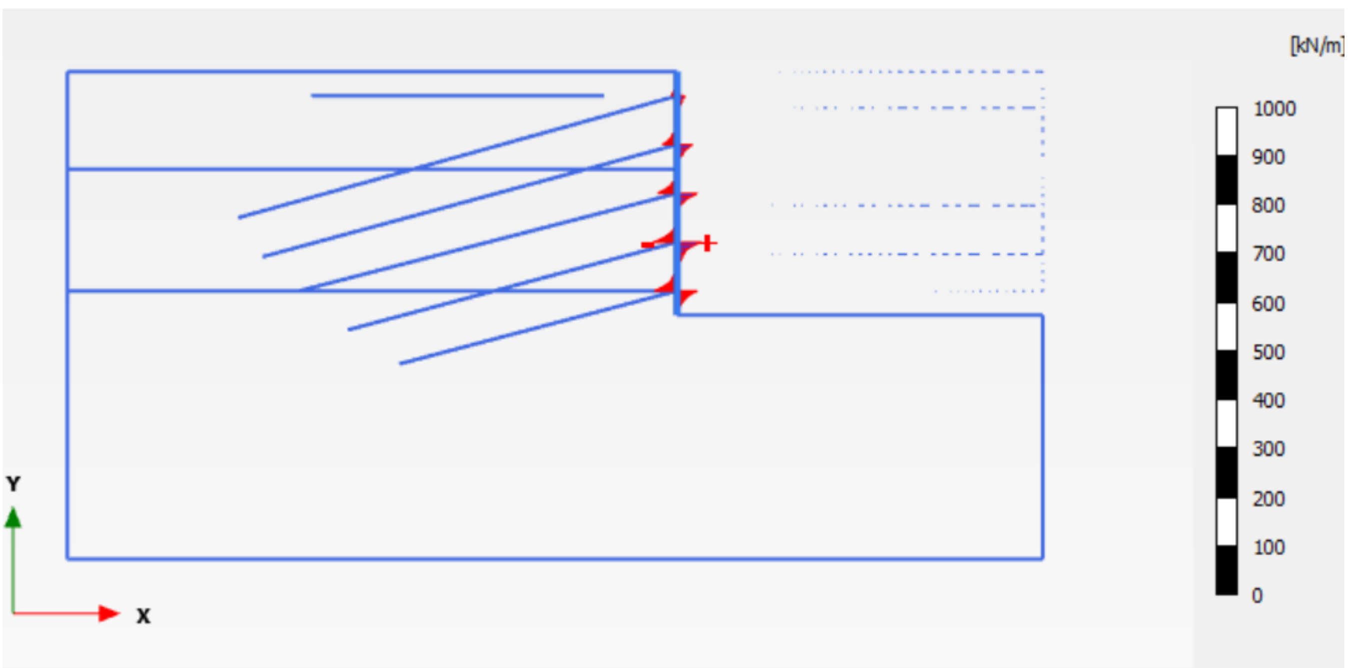
Table 0-9 Flintstone Engineering S.Co LEM Analysis

	Disp. min [mm]	Disp. max [mm]	Shear force min. [kN/m]	Shear force max [kN/m]	Moment min. [kNm/m]	Moment max. [kNm/m]
0.00	2.02	2.02	0.00	0.00	0.00	0.00
0.80	-0.58	-0.58	-22.71	-22.71	11.33	11.33
1.50	-2.91	-2.91	-33.91	-33.91	30.64	30.64
1.50	-2.91	-2.91	53.79	53.79	30.64	30.64
1.60	-3.25	-3.25	51.76	51.76	25.36	25.36
2.40	-6.03	-6.03	32.69	32.69	-8.74	-8.74
3.20	-8.77	-8.77	8.77	8.77	-25.65	-25.65
3.50	-9.76	-9.76	-1.44	-1.44	-26.76	-26.76
3.50	-9.76	-9.76	89.39	89.39	-26.76	-26.76
3.52	-9.83	-9.83	88.69	88.69	-28.54	-28.54
4.00	-11.35	-11.35	70.88	70.88	-66.91	-66.91
4.80	-13.52	-13.52	45.79	45.79	-113.56	-113.56
5.60	-15.03	-15.03	16.86	16.86	-138.94	-138.94
6.40	-15.71	-15.71	-16.83	-16.83	-139.27	-139.27
7.00	-15.67	-15.67	-45.23	-45.23	-120.78	-120.78

	Disp. min [mm]	Disp. max [mm]	Shear force min. [kN/m]	Shear force max [kN/m]	Moment min. [kNm/m]	Moment max. [kNm/m]
7.00	-15.67	-15.67	48.74	48.74	-120.78	-120.78
7.20	-15.56	-15.56	38.68	38.68	-129.53	-129.53
8.00	-14.61	-14.61	-4.56	-4.56	-143.50	-143.50
8.80	-12.82	-12.82	-52.56	-52.56	-120.97	-120.97
9.60	-10.31	-10.31	-96.04	-96.04	-60.74	-60.74
10.00	-8.91	-8.91	-117.22	-117.22	-18.55	-18.55
10.00	-8.91	-8.91	-117.22	-117.22	-18.55	-18.55
10.00	-8.88	-8.88	-117.36	-117.36	-17.61	-17.61
10.00	-8.88	-8.88	-117.36	-117.36	-17.61	-17.61
10.40	-7.46	-7.46	-106.73	-106.73	26.94	26.94
11.20	-4.76	-4.76	-68.92	-68.92	98.65	98.65
12.00	-2.63	-2.63	-9.25	-9.25	131.38	131.38
12.80	-1.27	-1.27	45.03	45.03	112.83	112.83
13.60	-0.57	-0.57	53.42	53.42	71.31	71.31
14.40	-0.31	-0.31	40.46	40.46	32.95	32.95
15.20	-0.26	-0.26	20.73	20.73	8.29	8.29
16.00	-0.27	-0.27	0.00	0.00	-0.00	-0.00

A.2.4 Project 2(Flintstone Engineering S.C) Soil Nailing Wall Analysis results

calculation results, Plate, 5 the excavation [Phase_9] , Shear forces Q



Shear forces Q (scaled up 0.0200 times)

Maximum value = 50.74 kN/m (Element 15 at Node 5280)

Minimum value = -48.78 kN/m (Element 14 at Node 5280)

FIGURE 0-10 FLINTSTONE ENGINEERING S.CO SOIL NAILING WALL SHEAR FORCE DIAGRAM

Calculation results, Plate, 5 excavation [Phase_9] , Axial forces N

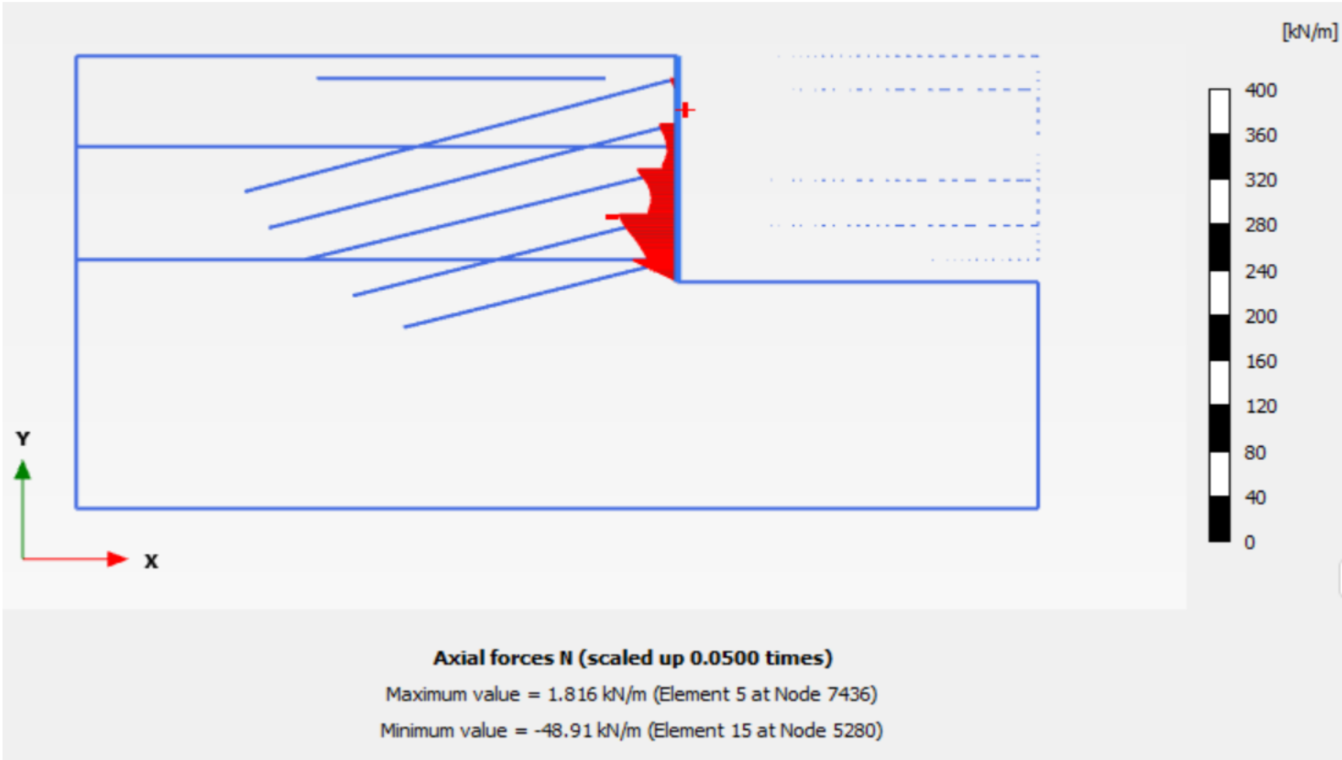


FIGURE 0-11 FLINTSTONE ENGINEERING S.CO SOIL NAILING WALL AXIAL FORCE DIAGRAM

A.2.5 Project 3(Yewub Hagere Group S.C) Soil Nailing Wall Analysis results

Calculation results, Plate, Second Layer Excavation [Phase_4] , Shear forces Q

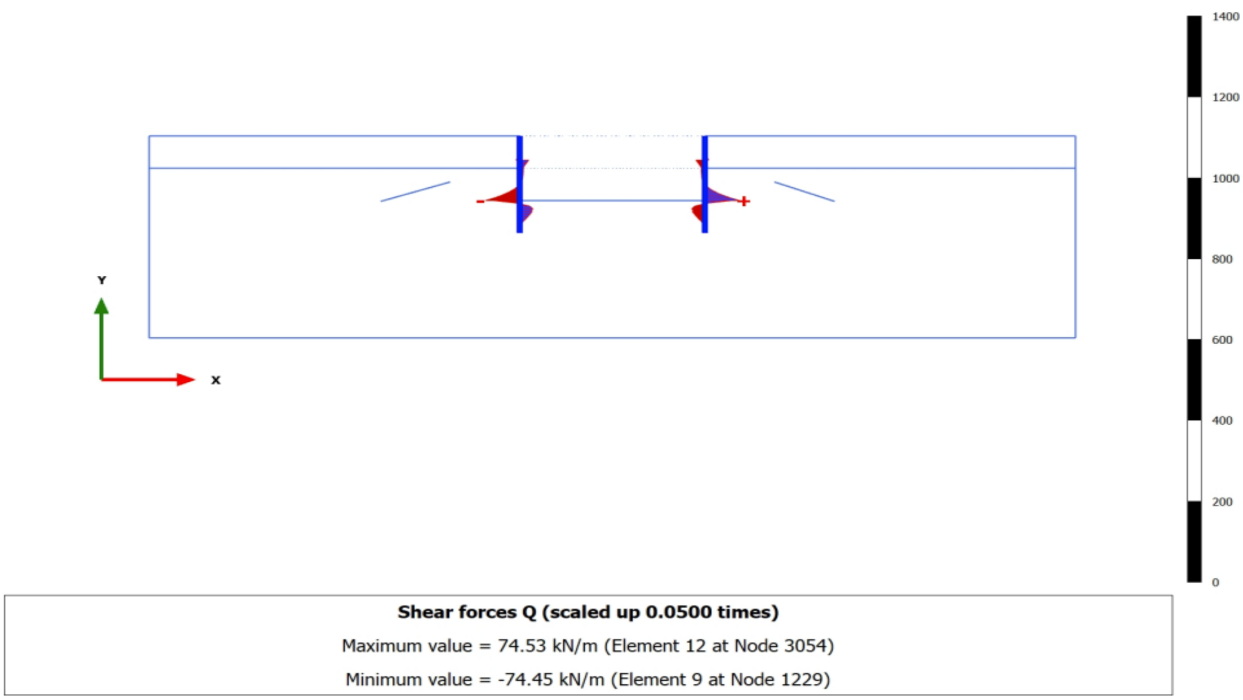


FIGURE 0-12 YEWUB HAGERE GROUP S.CO ANCHORED CONTIGUOUS PILE SHEAR FORCE DIAGRAM

Calculation results, Plate, Second Layer Excavation [Phase_4] , Axial forces N

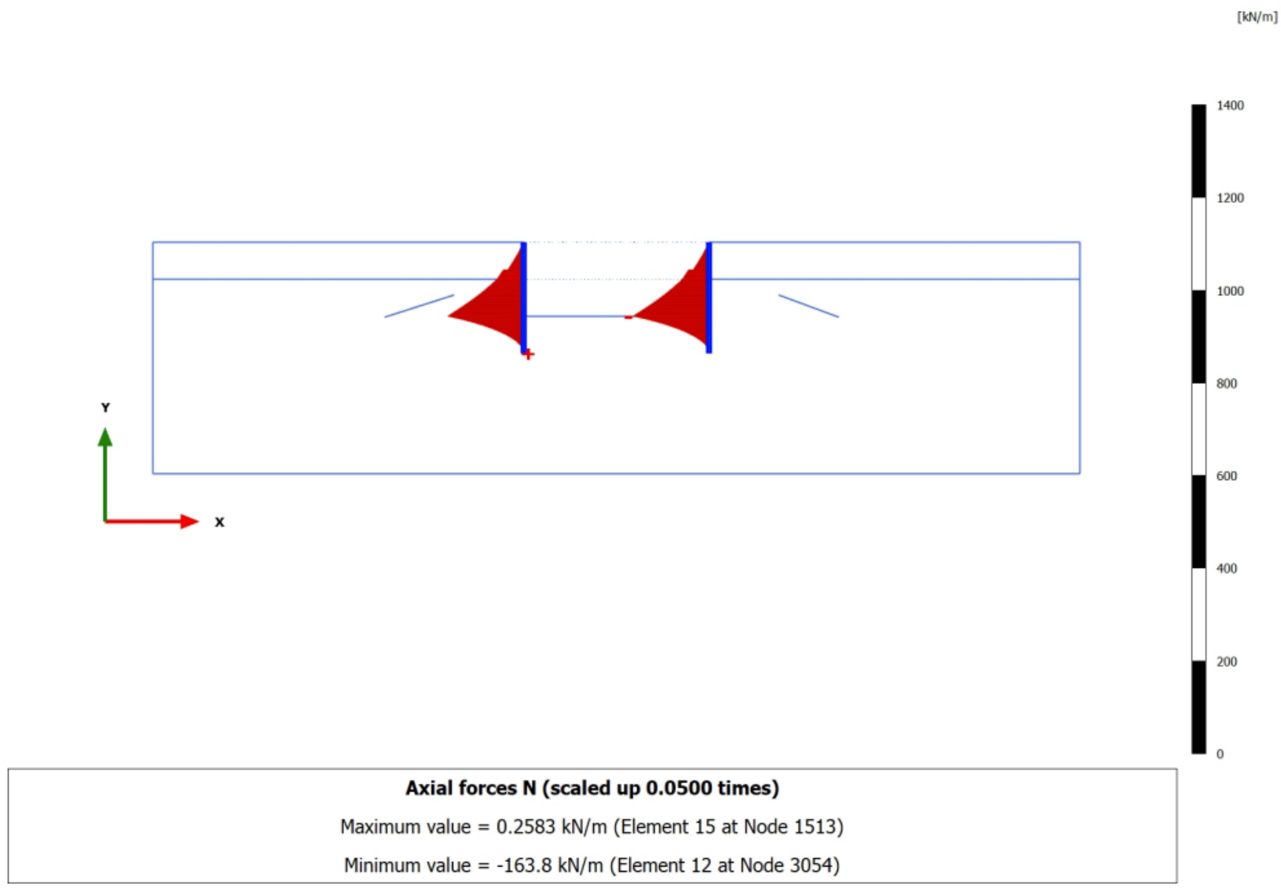


FIGURE 0-13 YEWUB HAGERE GROUP S.CO ANCHORED CONTIGUOUS PILE AXIAL FORCE DIAGRAM

Table 0-9 Yewub hagere S.Co LEM Analysis

	Disp. min [mm]	Disp. max [mm]	Shear force min. [kN/m]	Shear force max [kN/m]	Moment min. [kNm/m]	Moment max. [kNm/m]
0.00	0.10	0.10	-0.00	-0.00	0.00	0.00
0.60	0.03	0.03	-4.73	-4.73	1.23	1.23
1.20	-0.05	-0.05	-12.24	-12.24	6.20	6.20
1.80	-0.15	-0.15	-21.58	-21.58	16.29	16.29
2.40	-0.30	-0.30	-31.45	-31.45	32.18	32.18
3.00	-0.55	-0.55	-43.09	-43.09	54.44	54.44
3.00	-0.55	-0.55	68.49	68.49	54.44	54.44
3.60	-0.93	-0.93	54.76	54.76	17.36	17.36
4.20	-1.36	-1.36	41.09	41.09	-11.06	-11.06
4.80	-1.77	-1.77	30.65	30.65	-32.66	-32.66
5.40	-2.07	-2.07	18.62	18.62	-47.52	-47.52
6.00	-2.24	-2.24	5.01	5.01	-54.69	-54.69

	Disp. min [mm]	Disp. max [mm]	Shear force min. [kN/m]	Shear force max [kN/m]	Moment min. [kNm/m]	Moment max. [kNm/m]
6.60	-2.24	-2.24	-10.18	-10.18	-53.22	-53.22
7.20	-2.08	-2.08	-26.95	-26.95	-42.16	-42.16
7.80	-1.80	-1.80	-45.31	-45.31	-20.56	-20.56
8.00	-1.69	-1.69	-51.65	-51.65	-11.06	-11.06
8.00	-1.69	-1.69	-51.58	-51.58	-10.65	-10.65
8.40	-1.46	-1.46	-32.94	-32.94	5.96	5.96
9.00	-1.14	-1.14	-11.75	-11.75	18.96	18.96
9.60	-0.86	-0.86	1.98	1.98	21.56	21.56
10.20	-0.66	-0.66	9.93	9.93	17.74	17.74
10.80	-0.50	-0.50	13.23	13.23	10.46	10.46
11.40	-0.38	-0.38	9.77	9.77	3.23	3.23
12.00	-0.26	-0.26	0.00	0.00	-0.00	-0.00

A.2.6 Project 3(yewub hagere group s.co) Soil Nailing Wall Analysis results

Calculation results, Plate, Final Excavation [Phase_7] , Shear forces Q

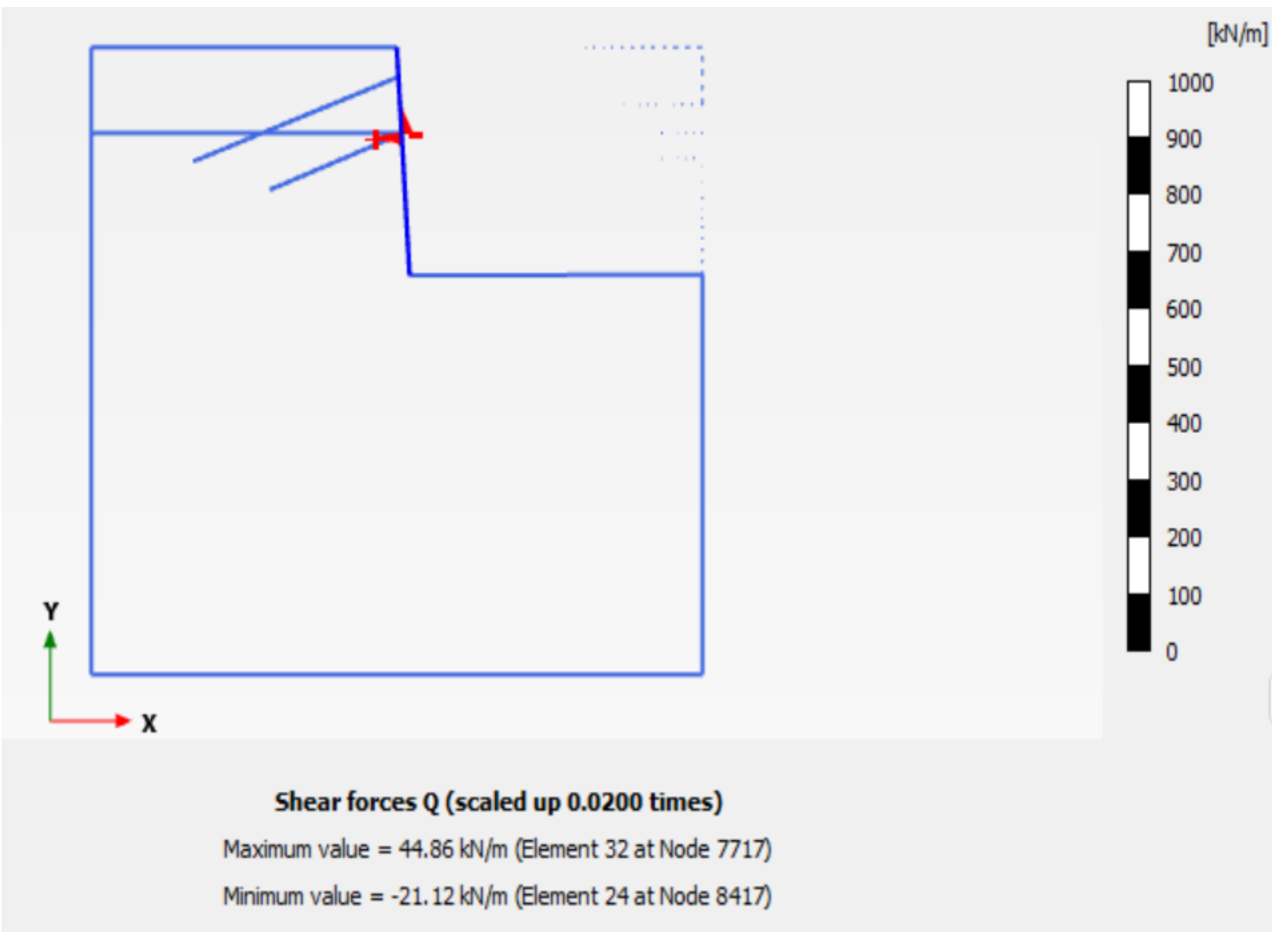


FIGURE 0-14 YEWUB HAGERE GROUP S.CO SOIL NAILING WALL SHEAR FORCE DIAGRAM

Calculation results, Plate, Final Excavation [Phase_7], Axial forces N

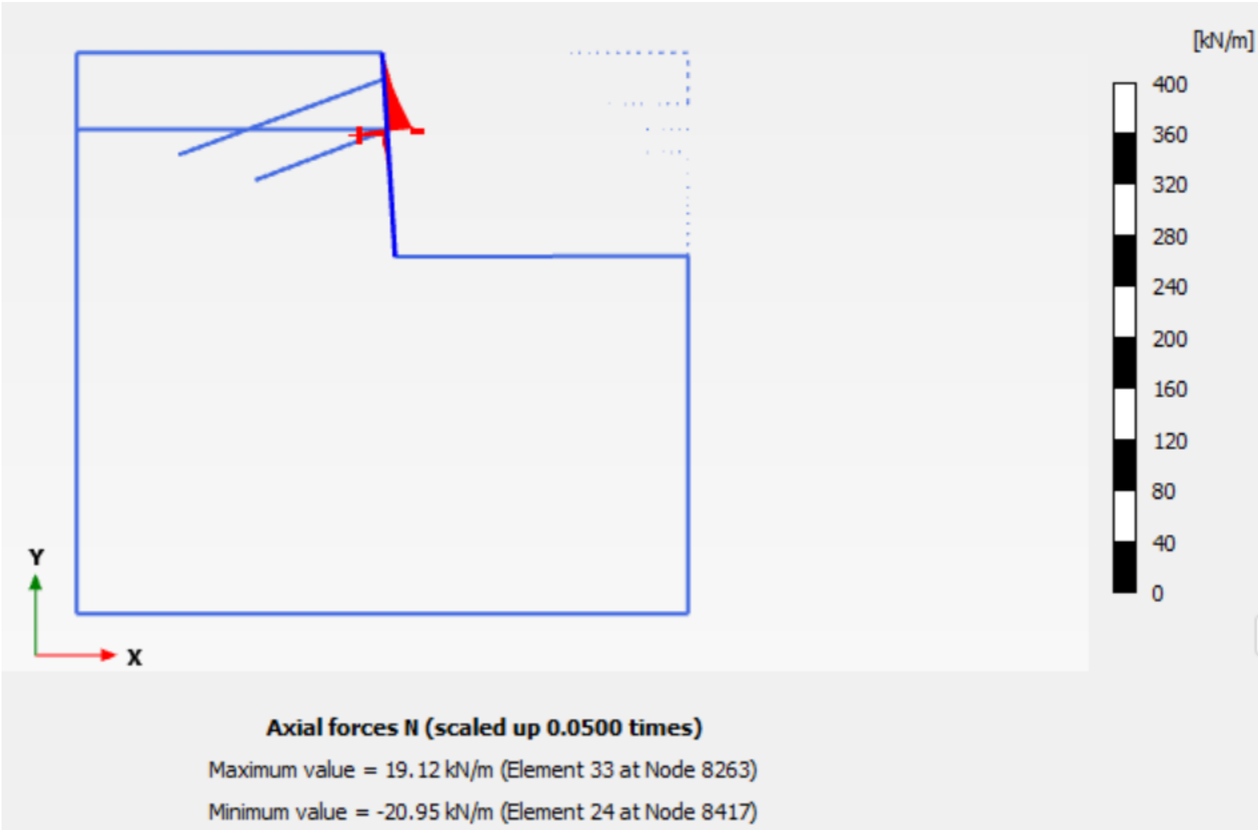


FIGURE 0-15 YEWUB HAGERE GROUP S.CO AXIAL FORCE DIAGRAM