ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Parametric Study of Anchored Diaphragm Wall For Deep Excavations Using Plaxis 2D

By

Abenezer Shiferaw

November, 2020

Addis Ababa, Ethiopia

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A Thesis Submitted to the school of graduate studies of Addis Ababa University in partial fulfillment of the requirements for the Degree of Masters of Science in Geotechnical Engineering

By

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Advisor

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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. 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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Date November, 2020

Confirmation

This thesis has been submitted for examination with my approval as the Institute assistant professor.

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Abstract

There is a growing need for the construction of underground structures beneath buildings because of the scarcity of available space in Addis Ababa either for parking or other purposes. Because of that there is a need for deep excavations during construction of the structures, which shall minimize the ground movement caused by the structures to remain within the allowable limits. It's the geotechnical engineer's responsibility to properly study the soil-structure interaction and soil behavior to design a support structure which satisfies this requirement in addition to providing a safe working space.

There are many ways to support deep excavations; among which diaphragm walls and contiguous piles are the most popular ones in Addis Ababa. There is a need to study the effect of different parameters prior to the construction for a safer design. Previous studies on anchored diaphragm walls show that the effect of excavation depth, depth of embedment, soil type, strut spacing, and stiffness of the anchored diaphragm wall but no consideration is given for the geometric properties of the ground anchor used. Therefore, this research presents a study of anchored diaphragm walls using Plaxis 2D for deep foundation support in dry excavation by performing parametric study on the anchor orientation, anchor length and the grout (geo-grid) length to compare their influence on the stability of the diaphragm wall.

Available literatures on diaphragm wall had been carefully studied followed by a numerical analysis using Plaxis 2D on the representative base models. The comparison of the effect of the parameters are carried out by analyzing the result of the maximum horizontal displacement, vertical displacement, and bending moments induced in the diaphragm wall using FEM.

The results of the parametric studies showed that increasing the anchor orientation more than 25^{O} isn't advisable unless the site situation inhibited the installation. It was found out that free anchor length beyond the assumed wage slip surface should be a minimum of 1 meter for a safe anchorage. Beyond the assumed wage slip surface, for the same length of anchorage, it was found out that longer free anchor length and shorter grout result in a better anchorage than a short free anchor length with longer grout for both expansive and red silty clay soil.

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1. Introduction

1.1 Background

In big cities like Addis Ababa where space is rather limited for the construction of different kinds of buildings, it becomes apparent that the designer tries to use the available limited space without compromising the functional need of the structures to be built. Professionals in the construction business are now designing buildings with multiple basements as the solution for this space limitation that leads to deep excavation into the ground.

Traditional empirical methods such as those suggested by Peck (1969) and Clough and O'Rourke (1990) cannot always provide a reasonable prediction on the deformation pattern in complex modern construction in terms of the construction sequence, tie back system and development of pore water pressure. There are many other analytical methods of prediction proposed in the literature for the complex problem but these analyses are not sensitive to the construction procedure, overall stability, and movement pattern in the adjacent soil and effects on adjacent structures.

There have been several types of researches done in the parametric analysis of anchored diaphragm walls using FEM for deep foundation supports considering different Parameters like depth of embedment, type of soil, anchor spacing, the thickness of the diaphragm wall and depth of excavations (Fekadu, 2010), (Yimam, 2019), since these are the main Parameters which affect the stability of the ground movement. In this research focus is given to different geometric Parameters (anchor angle of orientation, free anchor length, and grout length) on the stability of the diaphragm wall on the maximum horizontal deflection, vertical deflection and bending moment of the diaphragm wall.

To simulate soil-structure interactions the software requires different input Parameters obtained from various soil testing techniques. For this research Plaxis 2D was used with the Hardening soil model to simulate the soil material type since it's more realistic than the traditional Mohr-Coulomb model (Brinkgreve, 2004).

1.2 Problem Statement

The design of appropriate ground anchors for diaphragm wall is very important in the design of anchored diaphragm wall for deep excavation support. There are very limited research's which can be used as a guide line for design of anchored diaphragm walls in Ethiopia. Moreover, this studies mainly focused on the stiffness, depth of excavation and embedment depth of the diaphragm wall and little is considered when it comes to ground anchors.

Hence, it is important to investigate the effect of ground anchors geometric properties which are used to counter the earth pressure developed and act on the diaphragm wall during and after the excavation. Thus, it's significant to study and put forth a guideline for anchored diaphragm wall to fill the gap on the information available regarding geometric anchor properties for soils found on Addis Ababa.

1.3 Objectives

The general objective of this study is to investigate the effect of the ground anchor geometry on anchored diaphragm wall by performing a parametric study using Plaxis 2D for Addis Ababa region.

The specific objectives of the study are as follows:-

- ✓ To develop a recommended range for anchor length based on the result from anchor length vs displacement relationship for anchored diaphragm wall.
- ✓ To study the bending moment, settlement and horizontal displacement variations on the diaphragm wall for different anchor length and orientations.
- ✓ To formulate and propose appropriate ground anchor geometry to be used as a guidelines when designing anchored diaphragm wall.

1.4 Methodology

The finite element software Plaxis 2D was used to simulate the model and analyze the results with different parameters to observe the effect of the parameters which we consider for this study keeping all the other parameters constant.

First the available literature in anchored diaphragm wall, ground anchors and soil mechanics are examined to establish the parameters that influence the proper functioning of a diaphragm wall. Next the finite element software plaxis 2D had been used for modeling the excavation in the top- down construction sequence. After that, the result obtained for the maximum horizontal displacement, vertical displacement and bending moment of the diaphragm wall had been analyzed and compared to reach a qualitative conclusion which will be used as a guideline for the engineers for future design and construction works.

In general the research proceeds as follows:-

1. Review the available literatures on diaphragm walls, ground anchors, and soil mechanics
2. Perform a parametric study using Plaxis 2D on anchored diaphragm wall
3. Interpret the result found by the parametric study by comparing the maximum vertical displacement, horizontal displacement and bending moment of the diaphragm wall
4. Develop guide lines, recommendations and conclusion to be used for the design of anchored diaphragm wall.

1.5 Significance of the research

The need for deep excavation support has been growing fast since there is scarcity of land in the city of Addis Ababa and that engineers considering “safety first” and “economy second” had been designing deep excavation support and anchored diaphragm walls are among them. Assessment of the Parameters which influence the stability of structures give designer a good support and ways to predict the influence of each Parameters and decide based on their degree of influence which parameter to work with. Thus this research is expected to have practical implications on how the Parameters affect the stability and will be used as a base line for the engineers during the design stage.

1.6 Structure of the thesis

The thesis consists of five chapters. The first chapter is the introduction part while literature review is covered in the second one. The third chapter covers the base model and chapter four the parametric study whereas the fifth chapter is left for the conclusions and recommendations.

2. Literature review

2.1 Introduction

The demand for the construction of underground structures is increasing recently and since it's the geotechnical engineer's responsibility to study and design the appropriate supporting structures for this excavation thus leading to pursuit for the developments of tools and methods which can simulate and predict the ground reaction and the soil and structure interaction behaviors.

The development of finite element methods is a breakthrough for geotechnical engineers which work with deep excavation. Some of the programs developed are ABAQUS, CRISP and PLAXIS. In this study PLAXIS 2D is used to study the effect of different geometrics Parameters (free anchor length, anchor orientation and grout length) influences. It had been proved by previous studies which used PLAXIS 2D to predict the soil structure interaction before the construction of the structure or after the construction of the structure to monitor and compare the actual values with those one which are obtained from the model to validate the precision of the software to be used for future construction.

2.2 Types of retaining walls

The most commonly used retaining wall types to support excavations are:-

a. Sheet pile wall
b. Soldier pile wall
c. Contiguous bored piles wall
d. Secant pile wall
e. Diaphragm wall

2.2.1 Sheet pile wall

Sheet pile walls are retaining walls constructed to retain earth, water or any other filling materials. These walls are thinner in section compared to other retaining structures.

Sheet pile walls can be of timber, reinforced concrete or steel.

Timber sheet piles are generally used for short spans and to resist light lateral loads. They are mostly used for temporary structures such as braced sheeting in cuts. When timber sheet piles are used in permanent structures above water level, they require preservative treatment and even, but still their life span is relatively short. Timber sheet piles are joined to each other by tongue and grove joints. Timber sheet piles are not suitable for soils consisting of stones, as the stone would dislodge the joints (Tan, 1998).

Figure 1. Timber sheet pile wall (source:wikiwand.com).

Reinforced concrete sheet piles are precast concrete members, usually with a tongue and groove joints. These piles are relatively heavy and bulky. They displace large volumes of solid during driving. This large volume displacement of soil tends to increase the driving resistance. Due to this reason, suitable reinforcement is provided considering large driving stresses (Tan, 1998).

Figure 2. Precast reinforced concrete sheet pile wall (source:jhs-system.com).

Steel sheet piles are most commonly used. Steel sheet piles possess several advantages over other piles (Tan, 1998). They are:

- ✓ Steel sheet piles are resistant to high driving stresses
- ✓ They are lighter in section
- ✓ They can be used several times
- ✓ They can be used either below or above water and have long life span
- ✓ Suitable joints can be provided to have a continuous wall
- ✓ The pile length can be increased either by welding or bolting.

Figure 3. Steel sheet pile wall (source:civilengineeringbible.com).

Disadvantages of sheet pile walls are:

- ✓ Sections can rarely be used as part of the permanent structure.
- ✓ Installation of sheet piles is difficult in soils with boulders or cobbles. In such cases, the desired wall depths may not be reached.
- ✓ Excavation shapes are dictated by the sheet pile section and interlocking elements.
- ✓ Sheet pile driving may cause neighborhood disturbance
- ✓ Settlements in adjacent properties may take place due to installation vibrations

Sheet pile wall is commonly used as temporary retaining wall system. The suitability of sheet pile to be used in basement construction is generally influenced by the following factors:-

a. Soil conditions and the ease of pile installation:
   The subsoil must allow the sheet pile to be easily driven in with Standard Penetration Test (SPT) ‘N’ values lower than 50 or else it would be difficult to achieve the required penetration. The selection of sheet pile to be used would depend on the requirements of the flexural strength and strength to resist driving. Driving of sheet piles in loose sandy soils can also result in settlements in adjacent ground.
b. Depth of excavation:
   Sheet pile is usually suitable for shallow excavation and as temporary works due to its lower stiffness compared to other types of retaining wall such as diaphragm wall, contiguous bored piles or secant piles.
c. Water tightness:
   Some seepage is expected to pass through the interlocking steel sheet piling if there is a difference in hydraulic head.
d. Ability to withdraw temporary sheet pile after used:
   It would be more economical if extraction of any of the temporary sheet piles is allowed. However, extraction causes vibration unless silent piler is used and also lateral soil movement when the void created during extraction collapses.

2.2.2 Soldier pile walls

Soldier pile wall has two major components; soldier piles (vertical component) and lagging (horizontal component). Solider piles usually consist of steel H sections and driven maintained in full contact with the soil. Its installation resistance is quite similar to the sheet pile. Solider piles provide the primary support to the retained soil, and lagging serves as a secondary support to the soil face. Lagging prevents progressive deterioration of the soil arching between piles (Tan, 1998).

Soldier pile wall is normally used for small shallow excavation in stiff soils and in soils above ground water table as temporary support. Soldier pile wall is not suitable for soft clays and loose sands.

Advantages

- ✓ Relatively cheap materials
- ✓ Suitable as a sacrificial casing
- ✓ Can be recuperated

Disadvantages

- ✓ Cannot be used next to existing structures as this system is not rigid enough
- ✓ Cannot be applied in the case of underground obstacles
- ✓ Not completely vibration-free

Figure 4. Soldier pile wall (source:theconstructor.org).

2.2.3 Contiguous bored piles wall

Contiguous bored piles wall can be either temporary or permanent wall for excavation. Usually contiguous bored piles wall is used in stiff soil and lower water table. The advantage of contiguous bored piles wall are lower cost and speed in construction for temporary or permanent wall where drilling conditions are conducive. The system has higher capacity to overcome obstructions like rock compared to other system. However additional works are needed to form an acceptable surface to the wall (Tan, 1998).

Advantages of choosing a Contiguous Pile Wall are:

- ✓ Easier to build in load variations, with no requirement for a guide wall
- ✓ It's a cost-effective bored pile solution, compared to Diaphragm walls
- ✓ They can be used for more flexible geometry than a diaphragm wall Load bearing capacity
- ✓ Can be installed in difficult ground conditions
- ✓ Contiguous Walls are ideal for metropolitan cities. (Urbanization makes the cost of land high and so it is increasingly common to go deeper into the ground)
- ✓ CFA piles are considered more economical than diaphragm walls in small to medium scale excavations due to reduction in cost and time of site operations
- ✓ Contiguous piles are suitable in crowded urban areas, where traditional retaining methods would otherwise encroach the adjoining properties
- ✓ The presence of footings of adjacent buildings and underground utilities on a site make contiguous pile wall the ideal choice for supporting an excavation
- ✓ Contiguous piled walls have a faster speed of installation when compared with secant piled walls

Figure 5. Contiguous piled wall (source:martellopiling.com).

2.2.4 Secant piles wall

The major disadvantage of contiguous bored piles wall which is the lack of water tightness has been effectively overcome by interlocking as in secant piles wall. Other than water tightness, secant piles wall has similar advantages and disadvantages as contiguous bored piles wall. This method consists of boring and concreting primary piles, at center to center spacing of slightly less than twice the nominal pile diameter, secondary piles ae then bored at mid distance between the primary piles before the concrete has achieved its full strength. Reinforcement is usually concentrated in secondary piles. The main advantage of the secant pile is the possible full temporary protection in sensitive and collapsible soils and ease of coring into rock (Tan, 1998).

The main advantages of secant pile walls are:

- ✓ Increased construction alignment flexibility.
- ✓ Increased wall stiffness compared to sheet piles.
- ✓ Can be installed in difficult ground (cobbles/boulders).
- ✓ Less noisy construction.
- ✓ No need to install timber lagging.

The main disadvantages of secant pile walls are:

- ✓ Verticality tolerances may be hard to achieve for deep piles.
- ✓ Total waterproofing is very difficult to obtain in joints.
- ✓ Increased cost compared to sheet pile walls.

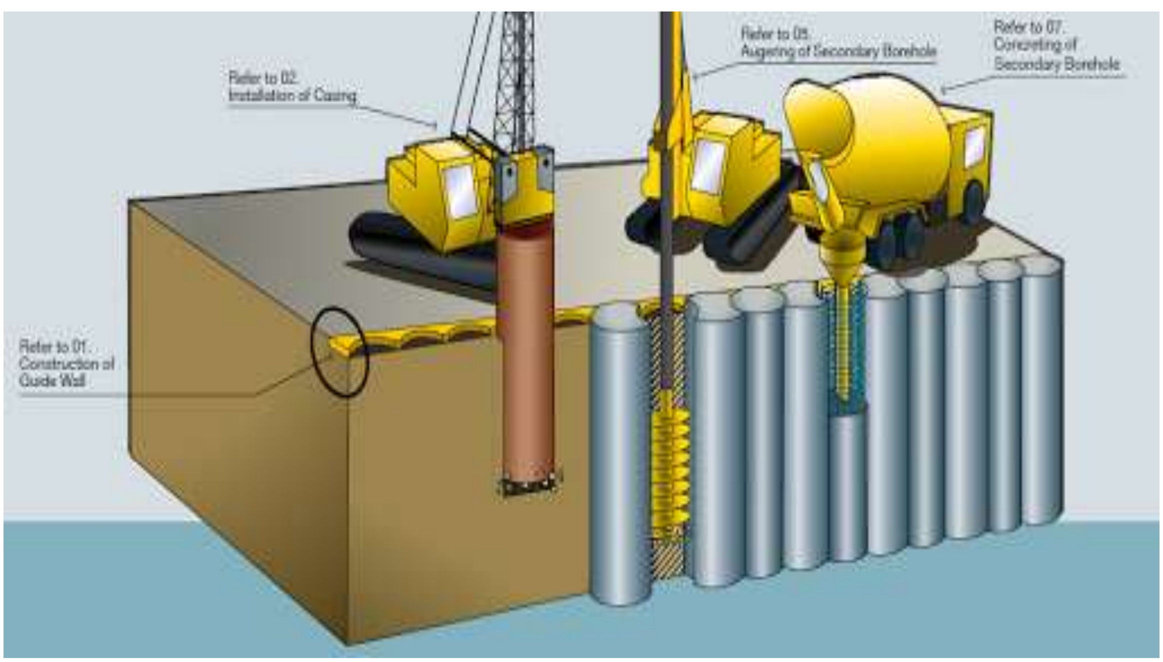


Figure 6. Construction of secant pile wall (source:railsystem.net).

2.2.5 Diaphragm wall

Diaphragm wall is commonly used as a permanent wall system. Diaphragm wall offers most efficient water tightness compared to other wall types. Similar to contiguous bored piles and secant piles wall, diaphragm wall construction also causes minimum noise and vibration disturbance. However, it’s not suitable for highly collapsible soil during trenching.

Diaphragm walls are also called slurry trench walls due to the reference given to the construction technique where excavation is made possible by filling and keeping the wall cavity full with bentonite-water mixture during excavation to prevent collapse of the vertical excavated surfaces. Typical wall thickness varies between 0.6 to 1.1m. The wall is constructed panel by panel in full depth. Panel width varies from 2.5m to about 6m. Short widths of 2.5m are selected in less stable soils, under very high surcharge or of very deep walls. It must be remembered that diaphragm walls are constructed as a series of alternating primary and secondary panels. Alternate primary panels are constructed first which are restrained on either side by stop-end pipes. Before the intermediate secondary panel excavation is taken up, the pipes are removed and the panel is cast against two primary panels on either side to maintain continuity (Yimam, 2019).

The first diaphragm walls were tested in 1948 and the first full scale slurry wall was built by Icos in Italy in 1950 (Puller, 1996) with bentonite slurry support as a cut-off wall. The first application in the US was in New York City [1962] for a 7m diameter by 24m deep shaft (Tamaro, 1990), that was followed by the Bank of California in San Francisco (Clough and Buchignani, 1980), the CNA building in Chicago (Cunningham and Fernandez, 1972), and the World Trade Center in New York (Kapp, 1969, Saxena, 1974).

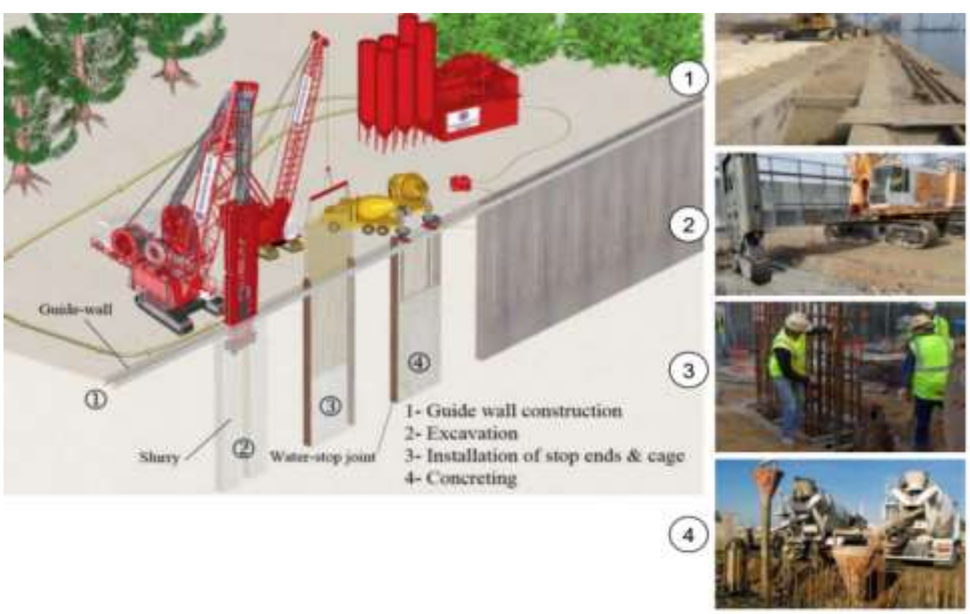


Figure 7. Construction sequences of diaphragm wall (source:gfwa.com.au)

Diaphragm wall is a reinforced concrete structure constructed in-situ panel by panel. The wall is usually designed to reach very great depth, sometimes up to 50m, mechanical excavating method is thus employed. Typical sequence of work includes:

- ✓ construct the guide wall
- ✓ excavation to form the diaphragm wall trench
- ✓ support the trench cutting using bentonite slurry
- ✓ insert reinforcement and placing of concrete to form the wall panel

Major advantages of diaphragm wall are listed below.

- ✓ used as permanent structural wall
- ✓ water retainable
- ✓ less temporary propping needed
- ✓ applied for top-down construction method
- ✓ rigid structure so that ground movement induced by basement excavation is less than other flexible retaining wall
- ✓ vibration and noise generated from installation of diaphragm wall is less than other methods.

Previous studies on the diaphragm wall

Computer aided numerical analysis has been proven quite effective in the prediction of the ground movements. The deep excavation pit with multi anchored diaphragm wall in Tirana show that the proposed numerical model has been able precisely to describe the effects of the anchoring and excavation (Josifovski, 2012).

In order to obtain a realistic simulation of the excavation the problem was analyzed using the finite element method on two- and three dimensional models. The popular program Plaxis specialized for geotechnical engineering has been proven as efficient in combination with analytical solutions. The program enables simple but efficient spatial modelling of different structural elements and accurate material definition. The ground stress-strain state with the occurring soil effect during the excavation process has been simulated on a two-dimensional plane strain finite element model. The soil material is discretized using the Mohr-Coulomb model while the concrete diaphragm wall with linear material law. The spatial discretization had been varied depending on the situation but in general triangular plane elements with 15 nodes had been used for the soil and beams for the structural elements of the wall and anchors.

The technical solution of multi anchored diaphragm wall had been proved during the actual construction of which wall and excavation of the pit as very effective. At all-time the diaphragm wall deflections had been measured. The recordings had confirmed all predictions made in the Plaxis 2D analysis. (Josifovski, 2012).

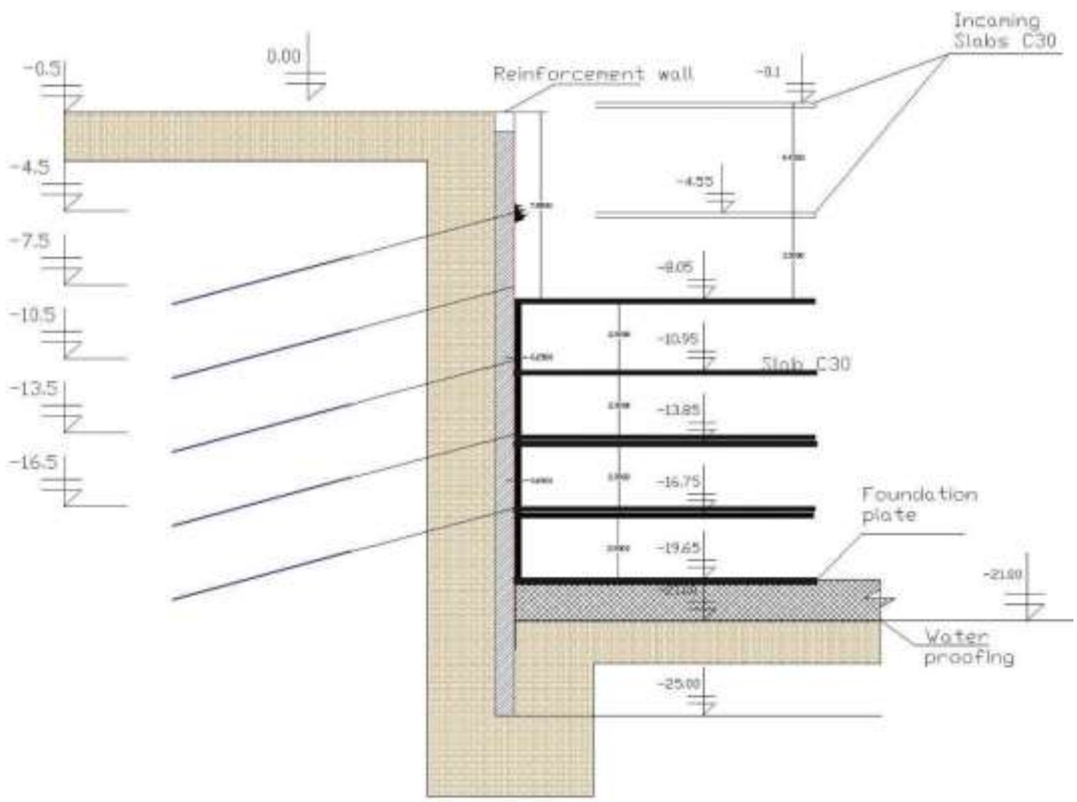


Figure 8. Multi anchored diaphragm wall model by Josifovski, 2012.

(Whittle, 1993), Studies the application of finite element analysis for modelling the top-down construction of seven-story building having underground parking garage at post office square in Boston. An advance constitutive model (MIT-E3) is used for describing the nonlinear and inelastic behavior of clays. The predictions which are obtained from the study are evaluated through comparisons with extensive field data, including wall deflections, soil deformation, surface settlements, and piezometric elevations. The analysis obtained was fairly similar with the measured data. The research also indicate there should be adequate characterization of engineering properties for the entire soil profile.

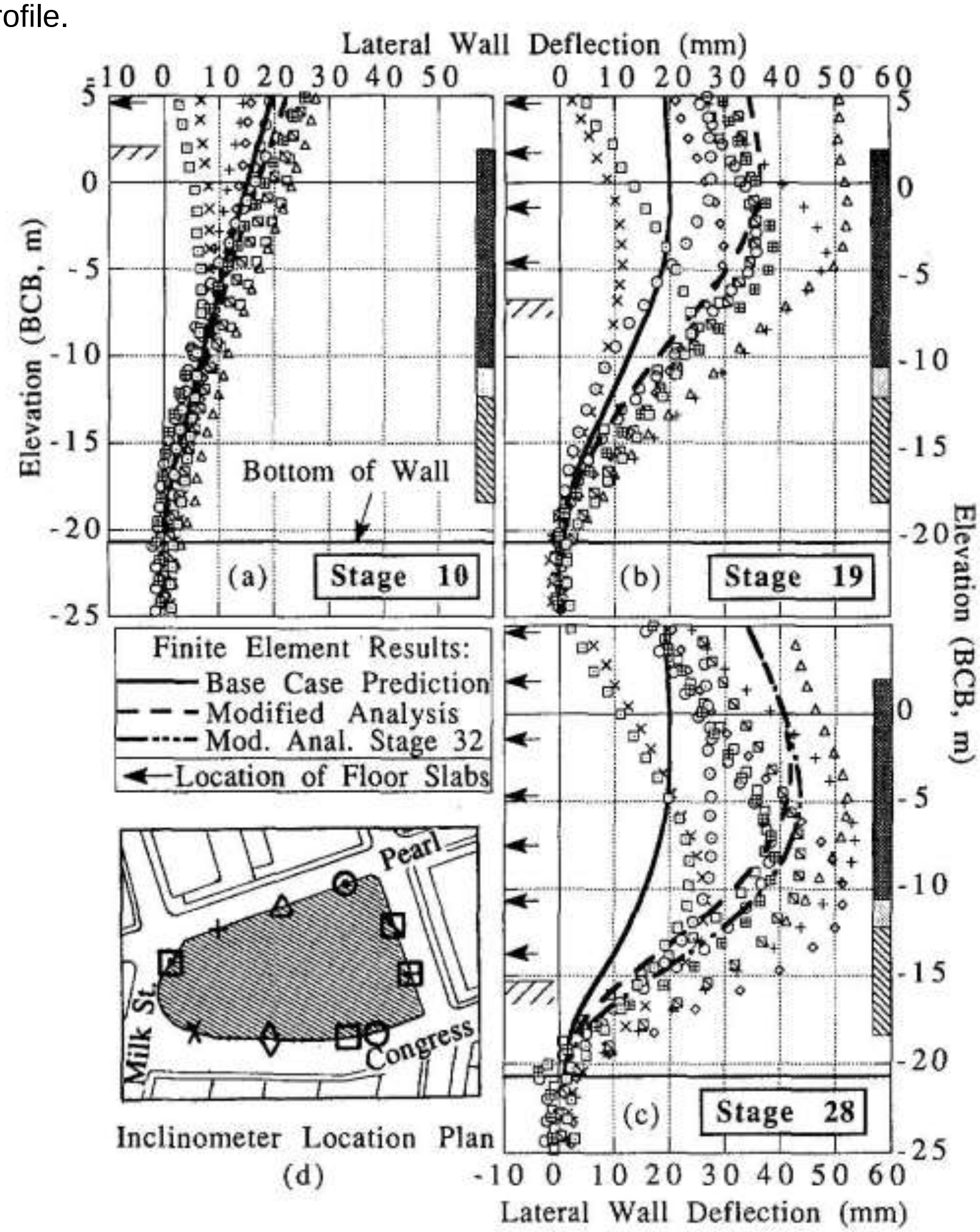


Figure 9. Comparison of predicted and measured lateral wall deflections (Whittle, 1993)

(Teparaksa, 2017), on Rosewood Hotel Project which consists of 6 basements design a diaphragm wall as soil protection system. The final excavation depth of this project is 24.2m from the ground surface therefore, the effect of water pressure was also considered. The behaviour of the diaphragm wall is predicted by numerical analysis by means of Finite Element Method (FEM). The result obtained from the FEM analysis of the diaphragm wall behaviour is presented in terms of bending moment, lateral displacement, and shear force in the diaphragm wall. Plaxis 2D is used in this project as the FEM program analysis to predict the diaphragm wall behaviour with Mohr-Coulomb soil modelling. The predicted diaphragm wall displacement by the FEM agrees well with field performance of the diaphragm wall which is obtained by measuring the lateral movement of the diaphragm wall by means of inclinometer at all stages of the construction.

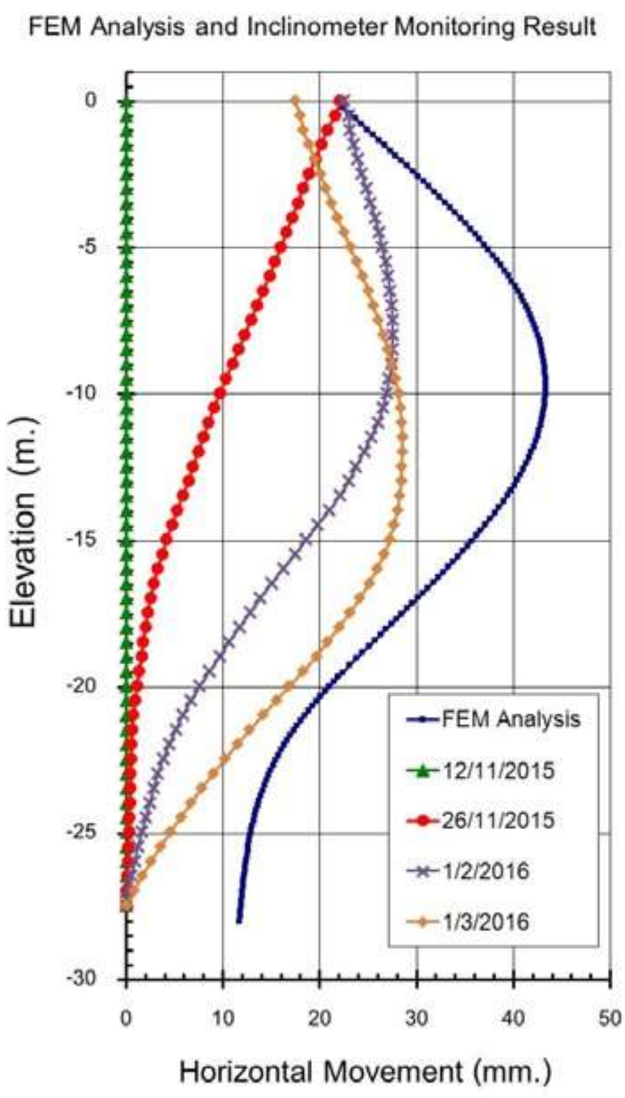


Figure 10. The lateral diaphragm wall displacement (Teparaksa, 2017)

(Konstantakos, 2004) on his study used PLAXIS 2D to analyze and control the ground movements for multi-level anchored diaphragm wall during excavation. The excavation was up to 23 m deep for the Dana Farb research tower in Longwood medical area of Boston. Summarizes the performance of the lateral earth support system based on field monitoring data measured during excavation of the basement from January 1995 to October 1995, Back analysis was used to evaluate and interpret wall and ground movement.

The calculations comprise a series of four 2-D, plane strain models representing each of the four sides of the excavation. Since all have different structures next to them.

After the finite element analysis and field measurement, back-analyses of the excavation performance using 2-D finite element analyses were able to give consistent estimates of the measured wall deflections on each of the four sides of the excavation. Below fig.11 shows the comparison between the results obtained by using Plaxis 2D and measured data.

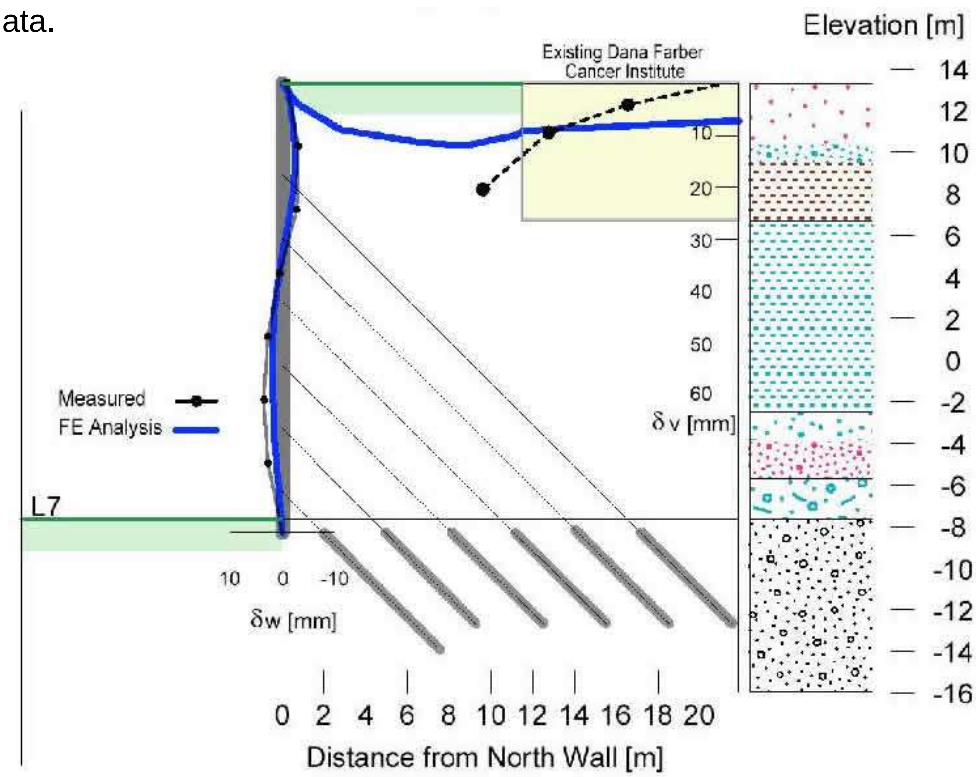


Figure 11. Measured and predicted behavior of the diaphragm wall at end of excavation Konstantakos, 2004

In conclusion back-analyses of the excavation performance using 2-D finite element analyses were able to give consistent estimates of the measured wall deflections on each of the four sides of the excavation. Ground losses have also been simulated in the FE analyses by including local volumetric strains in clusters of soil elements around the tiebacks. These simulations are able to replicate the measured surface settlements with relatively small volume strains, generating additional surface settlements in the range, 50mm – 60mm. The simulated ground losses also cause the perimeter wall to deflect 10mm –15mm from the excavation under the action of the applied anchor pre-stress loads. This result improves the overall agreement between the computed and measured behavior. The results appear to confirm the hypothesis that local ground losses during anchor installation can explain the unexpectedly large surface settlements that occurred on two sides of the excavation.

(Kokona, 2016), Studied design concept for an anchored diaphragm wall in the central part of budva, montenegro. The finite element softwere plaxis was used in his calculation. The spatial discretization is performed using triangular plane finite elements with 15 nodes, as for the structural element a beam finite element with three nodes is used in analysis. The analysis is defined to be a two-dimensional plane strain. The soil is defined using the Mohr-Coulomb material behaviour drained. Plate is used to model diaphragm wall with bending and normal stiffness. Geogrids and node-to-node elements combination are used to model ground anchors at grout body and free length, respectively.

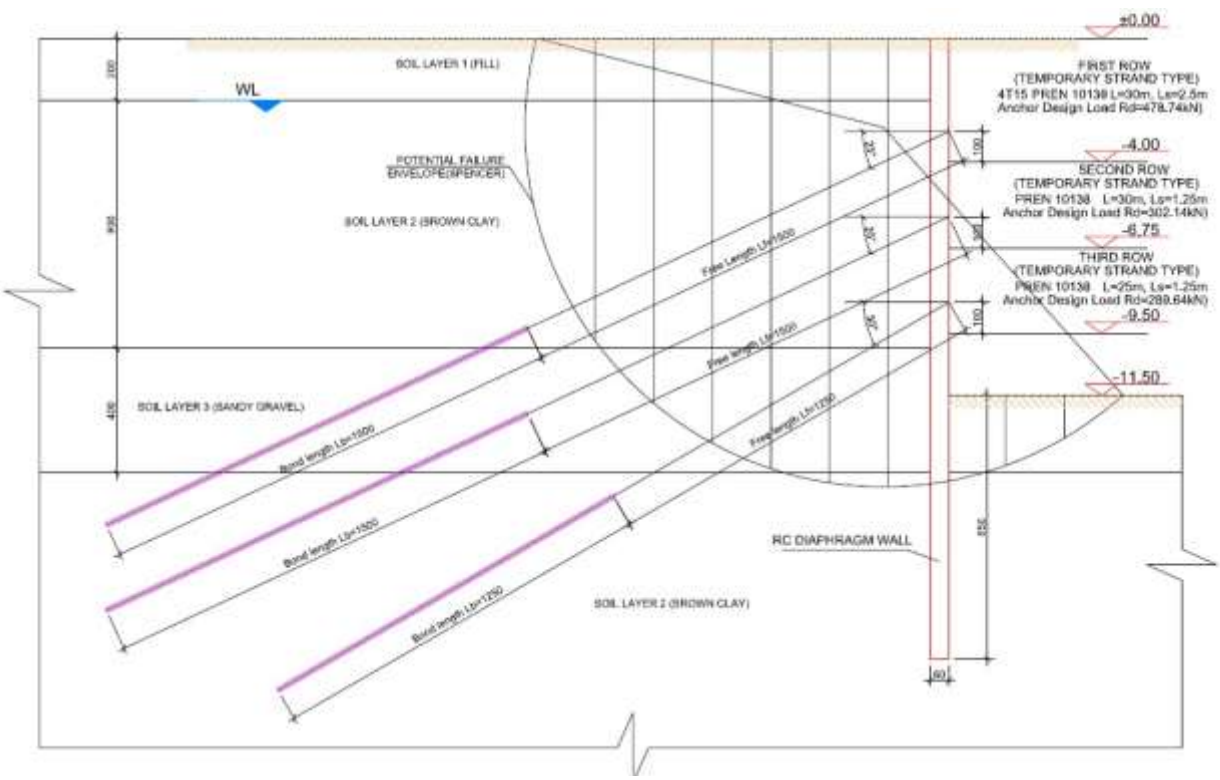


Figure 12. Design of an Anchored Diaphragm Wall in the Central Part of Budva, Montenegro

(Aissa Chogueur1, 2017) studied design of a self-stabilizing retaining diaphragm wall, using conventional analytical calculation method based on subgrade reaction coefficient and by numerical method with finite elements method FEM. two constitutive soil models are used such as Mohr-Coulomb MC and hardening soil model HSM.

The soil structure interaction was analyzed in phases (staged construction) with plastic loading steps analysis using drained conditions.

The lateral displacements obtained by using MC model (linear elastic) are not realistic because the stiffness is taken constant while those obtained by nonlinear hardening soil model (HSM) are more realistic.

The paper by Suched Likitlersuanga, 2013 presents a finite element analysis on the Bangkok MRT underground construction project. The excavation of Sukhumvit Station was selected as the case study for the FEM numerical modelling, in this paper the numerical study focuses on the initial input on the ground conditions and the constitutive soil models. The geotechnical Parameters were selected based on the soil investigation reports carried out for the purpose of the construction. The Parameters selected for the constitutive models used in the FEM analysis were calibrated against the laboratory testing results. Finally, all the FEM simulations were compared with the data from field investigations.

Sukhumvit Station was constructed using the Top-Down Construction method, with a configuration of the center plat-form type. The station box had a width, length and depth of 23 X 200 X 21 m. The reinforced concrete diaphragm walls (D-walls) were 1m thick and 27.9m deep; they were used for earth-retaining and permanent structures in the station.

(Fekadu, 2010), Studied deep excavations with tie back diaphragm wall in expansive clay, red silty clay and in cohesionless granular sandy soil. His main objectives being to investigate the effect of different Parameters on the ground movements which are caused by the deep excavation using the finite element software Plaxis 2D. The Parameters considered in his study where soil type, depth of excavation, wall embedment depth, wall stiffness, and strut spacing. These variables were used to conduct a series of finite element analyses using simplified geometry and ground conditions for the purpose of achieving the objective of his thesis. Results of these analyses were recorded in terms horizontal displacement of the diaphragm wall, ground settlement behind the diaphragm wall, and bending moments induced in the diaphragm wall due to an adjacent deep excavation.

(Muhamed,2018), studied the effect and performance of diaphragm wall for basement work in terms of bending moment, shear force, lateral wall deflections and ground movements by varying wall types, size of the wall and wall embedment length during excavation using the finite element software Plaxis 2D. The results were recorded in terms of horizontal displacement of the wall, ground settlement behind the wall, shear force and bending moments induced in the wall due to an adjacent deep excavation.

2.3 Approaches to design and analysis of excavation support walls

The major approaches for the design of excavation support walls can be classified in three and they are:-

1. The classical approach
2. The beam- column approach and
3. The finite element approach

2.3.1 The classical approach

The classical method for diaphragm walls bases its calculations on the Rankine failure criteria. With regard to their structural functions, walls are categorized into two types. Namely, cantilever walls and anchored walls.

2.3.1.1 Cantilever retaining walls

Cantilever retaining walls are normally used when the depth of excavation is small. The walls derive their support from the passive resistance developed in the soil below the excavation level in front of the wall (Fig. 13). The wall must penetrate to a sufficient embedment. The minimum embedment depth d for equilibrium can be determined by solving simultaneously two equations in which x and d are the unknowns, ΣH=0 and ΣMD=0 (Tefera, 2000).

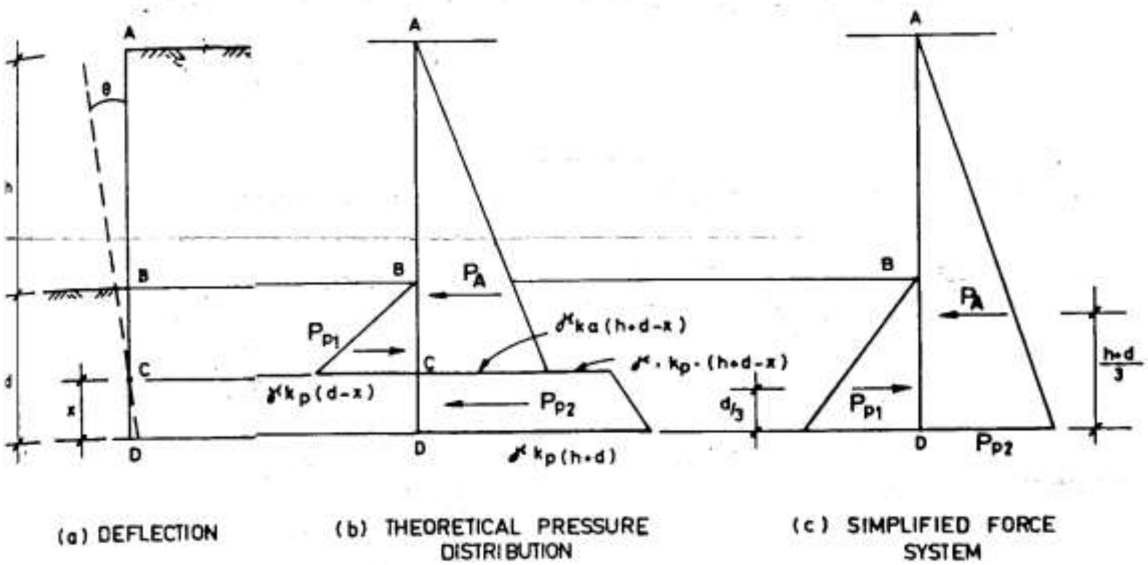


Figure 13. Cantilever retaining walls Tefera, 2000

$$P_A = \frac{1}{2} K_a \gamma (h+d)^2 \text{ --------------------------- Eq. 1a}$$

$$K_p d^3 - K_a \gamma (h+d)^3 = 0 \text{ --------------------------- Eq.1b}$$

The solution of Eq. (1) gives the guide to the embedment depth required and the calculated depth is increased by 20% to allow extra length for the development of the passive force P_{p2} (Tefera, 2000)

2.3.1.2 Anchored Retaining Walls

Anchored retaining walls are held in tact by tie-rods anchored into the soil. The principal methods of analyzing the equilibrium of anchored wall are based on two assumption related to the method of support of the driven end. These are known as the free earth support method and the fixed earth support method. Here, a single supported wall is considered.

Free earth support method

In this method, the wall is assumed to rotate freely about its base and freely supported above the ground by the anchorage forces and below the ground level by passive resistance (Tefera, 2000).

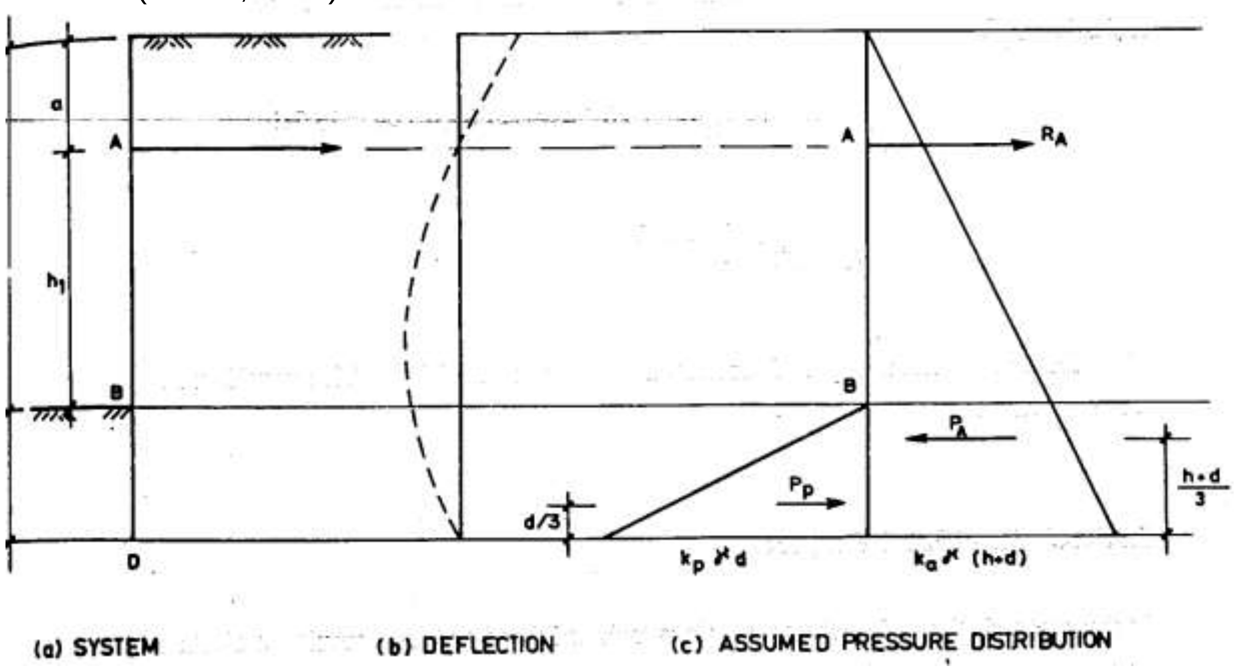


Figure 14. The free earth support methods Tefera, 2000

The minimum driving depth, d required for stability is obtained by taking moments about A,

$$\frac{1}{2} K_p d^2 \left(h_1 + \frac{2}{3} d\right) = \frac{1}{2} K_a \gamma (h+d)^2 \left(\frac{2}{3} d - \frac{1}{3} h + h_1\right) \text{ --------------- } Eq. 2$$

The above cubic equation is solved by assigning trial values of d and then the calculated minimum depth of embedment is increased by 20% for safety factor.

Fixed earth support method

In this method, the end of the retaining wall is assumed fixed and as a result the elastic line will be as indicated in Fig.15. Point C is considered as a hinge transmitting shear only and the moment at this point is zero. Using the method of the equivalent beam method, the structures may be regarded as two separate beams, AC and DC, with force RC as reaction at the hinge. The procedure of the analysis is as follows:

a) The position of C is estimated from the relationship that exists between the angle of internal friction Φ and the distance y as indicated in Table 1.

Table 1. Relationship between Φ and y (Tefera, 2000).

Φ(°)	Y	
20	0.25h	For normal frictional soils, it is sufficient to use y = o.1h
25	0.15h	
30	0.08h	
35	0.035h	
40	-0.006h	

b) The forces on the upper beam AC are calculated from the active and passive pressures.

$$P_a = \frac{1}{2} K_a \gamma \, (h + y), and$$

$$P_p = \frac{1}{2} K_p \gamma y^2 ----Eq.3$$

c) The reaction RC at the hinge C is found by taking moments of force about A.

$$R_C = \frac{\frac{1}{2} K_a \, \gamma (h+y)^2 \left[\frac{2}{3}(h+y) - a\right] - \frac{1}{2} K_p \gamma^2 \left(h - a \; + \frac{2}{3} y\right) \gamma}{h + y - a} ------Eq.4a$$

The anchor force R_A is found either by taking moment about C or from

$$\sum R_A + P_A + \; P_P + \; R_C ----------------------------Eq.4b$$

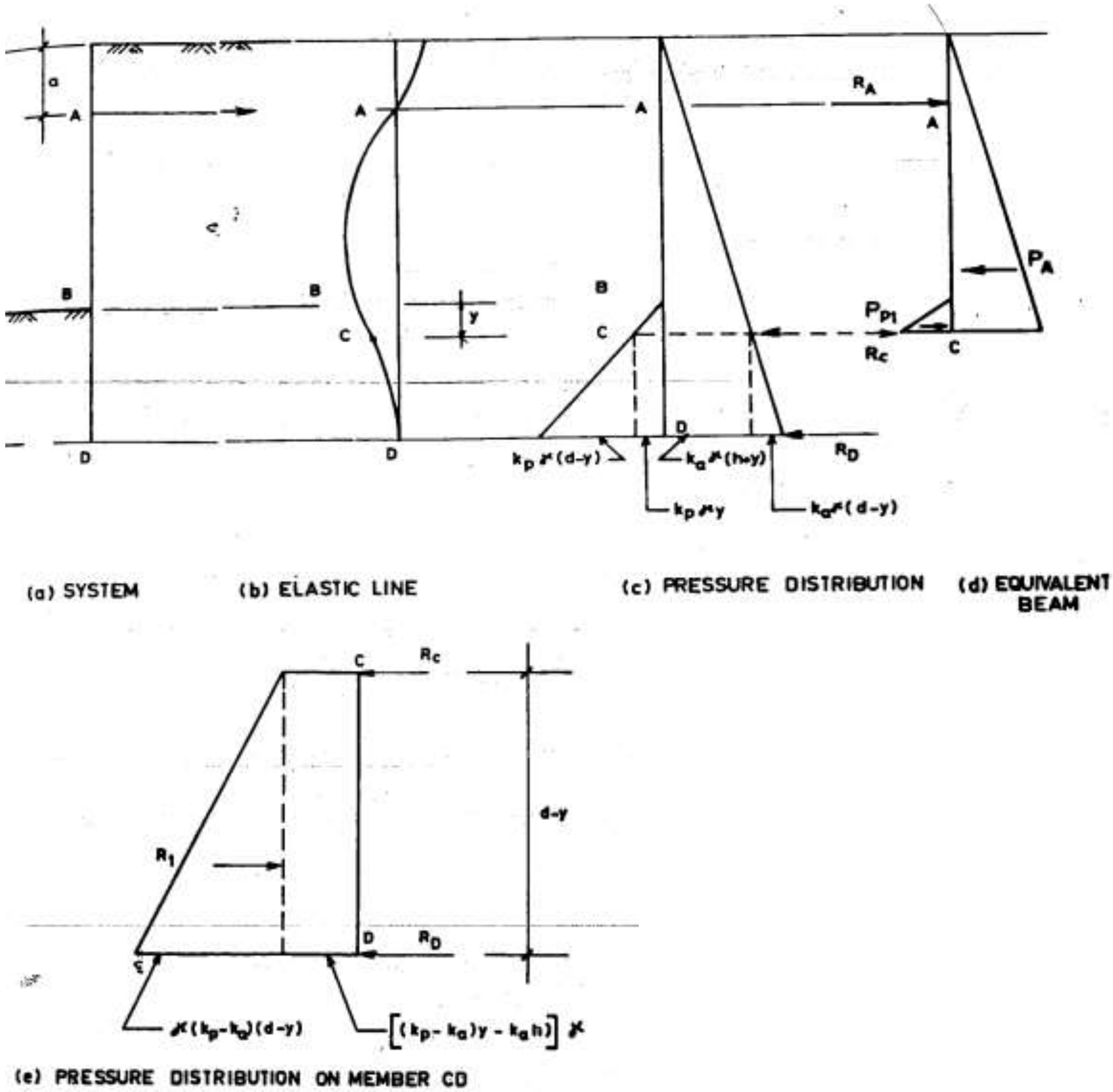


Figure 15. Anchored retaining wall for fixed earth support Tefera, 2000

d) For loads on the beam CD; taking the moments about D, re-arranging terms and solving a quadratic in (d-y), one obtains

$$d = \frac{3}{2}\left(\frac{K_a}{K_p - K_a}\right)h - \frac{y}{2} + \sqrt{\frac{9}{4}\left(\left(y - \frac{K_a h}{K_p + K_a}\right)^2 + \frac{6R_c}{\gamma\left(K_p - K_c\right)}\right)} \text{--------} Eq.5a$$

The first term under the root in Eq. 5a is very small compared with the second and may be neglected. Hence,

$$d = \frac{3}{2}\left(\frac{K_a}{K_p - K_a}\right)h - \frac{y}{2} + \sqrt{\left(\frac{6R_c}{\gamma\left(K_p - K_c\right)}\right)} \text{-------------------} Eq.5b$$

As in the previous cases, the calculated minimum embedment depth should be increased by 20% to allow for the fact that the lower passive resisting force RD is not a knife-edge reaction as assumed in the simplified calculation.

2.3.2 Beam-Column Method

The boundary element method, BEM (Figure 16), consists of modeling the wall as a set of vertical elements Δz long with a bending stiffness, EI, and an axial stiffness, AE. The soil is represented by a series of vertical and horizontal springs placed along the wall. Spring models for tieback walls have been recommended by Briaud and Kim (1998). Typical programs include BMCOL and TBWALL. A typical input for the BEM is the length of the wall, the length of the wall elements, and the wall stiffness

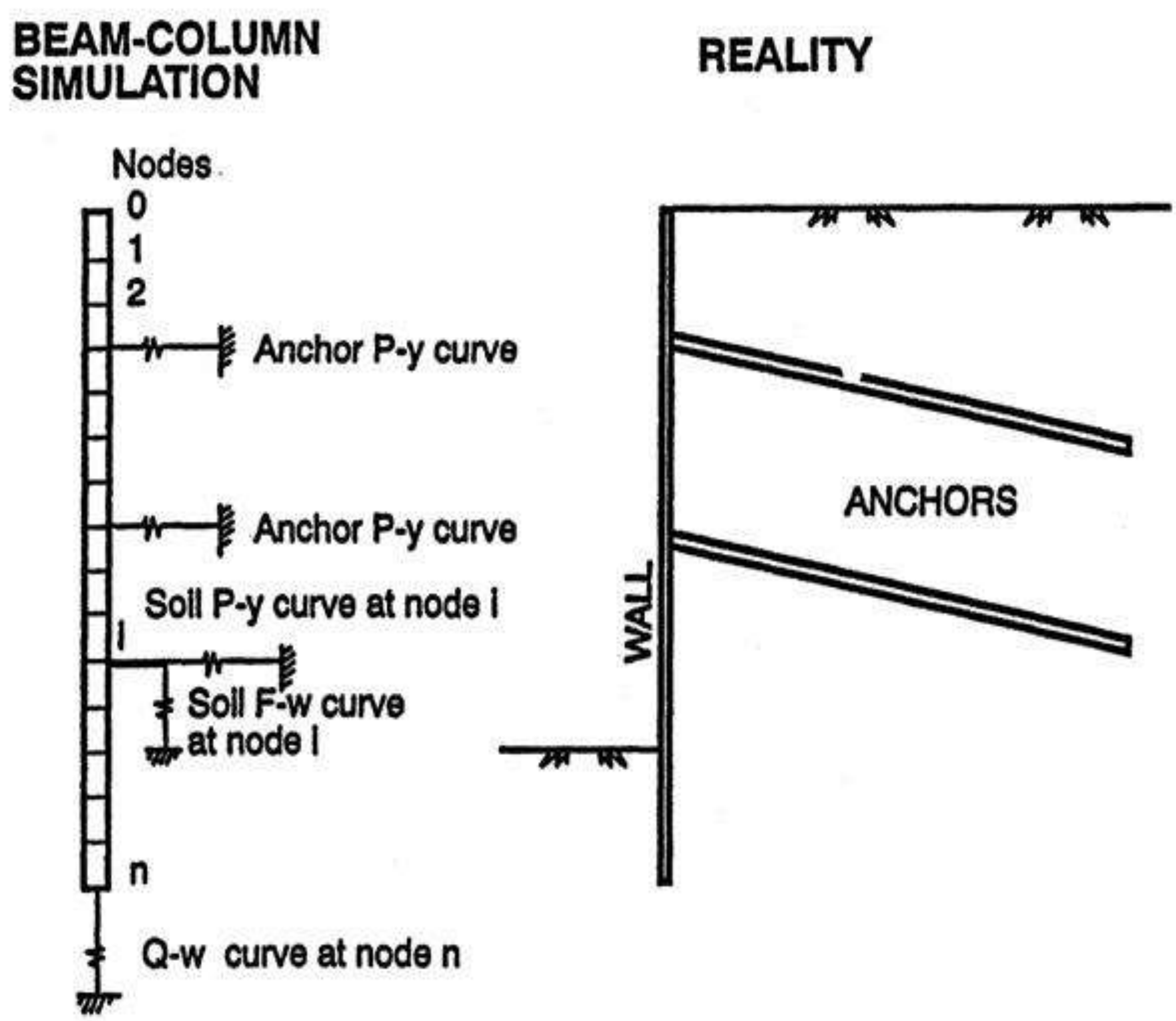


Figure 16. Boundary Element Model of Repeatable Section of Wall Briaud and Kim ,1998

The beam-column method for tieback walls deals with the analysis of the wall as a structural element interacting with the soil and the anchors (Briaud and Kim, 1998). An element of wall is considered and horizontal equilibrium of this element together with the constitutive law for the wall in bending (M=EI d2y/dz2) and the constitutive law for the soil [P = P(y, z)] leads to one of the governing differential equations (Matlock et al., 1981).

$$EI\ \frac{(d^4y)}{dZ^4} + Q\frac{d^2y}{dZ^2} - P(y,z) = 0 - - - - - - - - - - - - - - - - - Eq.6$$

Where:

E = wall modulus

I = wall moment of inertia

y = wall horizontal deflection at depth z

Q = axial load in the wall at depth z

P = horizontal soil reaction for a wall deflection y at a depth z.

The soil reaction P is a load per unit height of wall (kN/m, for example).

Vertical equilibrium of the same element together with the constitutive law for the wall in compression (Q=AE dw/dz) and the constitutive law for the soil [F = F(w, z)] lead to second governing differential equation (Matlock et al., 1981).

$$AE\frac{d^2w}{dz^2} + F(w,z) = 0 - Eq.7$$

Where:

E = wall modulus

A = wall cross section

w = wall vertical deflection at a depth z

F = vertical soil reaction for a wall deflection w at a depth z.

The soil reaction F is a load per unit height of wall. Eq. 6 and Eq. 7 are solved by the finite difference technique by considering that the wall is made of n elements having n + 1 horizontal deflections y_i, and n + 1 vertical deflections w_i. Once the deflections y_i and w_i are known, the bending M, the shear V, the soil reaction P, and the axial-load W can be obtained through their relation to y and w.

2.3.3 Numerical Methods

The need for numerical methods became apparent because classical elasticity problems are normally accomplished for regions and loadings with relatively simple geometry. Since most real-world problems involve structures with complicated shape and loading, a gap exists between what is needed in applications and what can be solved by analytical closed-form methods. Some of the numerical methods developed are described as follows:-

2.3.3.1 Finite Difference Method

The finite difference approximations for derivatives are one of the simplest and of the oldest methods to solve differential equations (Euler, 1768).

The finite-difference method is the most direct approach to discretizing partial differential equations. You consider a point in space where you take the continuum representation of the equations and replace it with a set of discrete equations, called finite-difference equations. The finite-difference method is typically defined on a regular grid and this fact can be used for very efficient solution methods. The method is therefore not usually used for irregular CAD geometries, but more often for rectangular or block-shaped models.

The principle of finite difference methods is close to the numerical schemes used to solve ordinary differential equations. It consists in approximating the differential operator by replacing the derivatives in the equation using differential quotients. The domain is partitioned in space and in time and approximations of the solution are computed at the space or time points. The error between the numerical solution and the exact solution is determined by the error that is committed by going from a differential operator to a difference operator.

2.3.3.2 Boundary Element Method

Boundary integral equations are a classical tool for the analysis of boundary value problems for partial differential equations. The term “boundary element method” (BEM) denotes any method for the approximate numerical solution of these boundary integral equations. The approximate solution of the boundary value problem obtained by BEM has the distinguishing feature that it is an exact solution of the differential equation in the domain and is parametrized by a finite set of Parameters living on the boundary (Heinz, 2010).

The BEM have some advantages over other numerical methods like finite element methods (FEM) or finite differences (Martin, 1986):

1. Only the boundary of the domain needs to be discretized. Especially in two dimensions where the boundary is just a curve this allows very simple data input and storage methods.

2. Exterior problems with unbounded domains but bounded boundaries are handled as easily as interior problems.

3. In some applications, the physically relevant data are given not by the solution in the interior of the domain but rather by the boundary values of the solution or its derivatives. These data can be obtained directly from the solution of boundary integral equations, whereas boundary values obtained from FEM solutions are in general not very accurate.

4. The solution in the interior of the domain is approximated with a rather high convergence rate and moreover, the same rate of convergence holds for all derivatives of any order of the solution in the domain. There are difficulties, however, if the solution has to be evaluated close to, but not on the boundary.

Some main difficulties with BEM are the following:

1. Boundary integral equations require the explicit knowledge of a fundamental solution of the differential equation. This is available only for linear partial differential equations with constant or some specifically variable coefficients. Problems with in homogeneities or nonlinear differential equations are in general not accessible by pure BEM. Sometimes, however, a coupling of FEM and BEM proves to be useful.

2. for a given boundary value problem there exist different boundary integral equations and to each of them several numerical approximation methods. Thus every BEM application requires that several choices be made. To evaluate the different possibilities, one needs a lot of mathematical analysis. Although the analysis of BEM has been a field of active research in the past decade, it is by no means complete. Thus there exist no error estimates for several methods that are widely used. From a mathematical point of view, these methods, which include very popular ones for which computer codes are available, are in an experimental state, and there might exist problems of reliability.

3. The reason for the difficulty of the mathematical analysis is that boundary integral equations frequently are not ordinary Fredholm integral equations of the second kind. The classical theory of integral equations and their numerical solution concentrates on second kind integral equations with regular kernel, however. Boundary integral equations may be of the first kind, and the kernels are in general singular. If the singularities are not integrable, one has to regularize the integrals which are then defined in a distributional sense. The theoretical framework for such integral equations is the theory of pseudo differential operators. This theory was developed 20 years ago and is now a classical part of Mathematical Analysis, but it is still not very popular within Applied Mathematics.

4. If the boundary is not smooth but has corners and edges, then the solution of the boundary value problem has singularities at the boundary. This happens also if the boundary conditions are discontinuous, e.g. in mixed boundary value problems. BEM

clearly have to treat these singularities more directly than FEM. Because the precise shape of the singularities frequently contains important information, e.g. stress intensity factors in fracture mechanics, this is a positive aspect of BEM. But besides practical problems with the numerical treatment of these singularities, non-smooth domains also present theoretical difficulties. These have so far been satisfactorily resolved only for two-dimensional problems. The analysis of BEM for three-dimensional domains with corners and edges is still in a rather incomplete stage.

2.3.3.3 Discrete Element Method

The discrete element method (DEM) is a numerical technique to simulate the behavior of a population of independent particles (Cundall and Strack, 1979). In this technique, each particle is represented numerically and is identified with its specific properties (e.g., shape, size, material properties, and initial velocity). The interior shape of the vessel containing the particle is used as the domain of the simulation and is separated into a grid to identify the particle’s position. Particles are then subjected to a small motion (based on Newton’s laws) over a small time interval (iteration). The small motion will cause some particles to contact other particles or the domain boundaries (Barry and James, 2016).

The discrete element method provides a useful tool for understanding a wide spectrum of dynamic problems in fractured rock masses, including potential sources of seismic excitation, e.g., earthquakes and rock bursts, as well as the effects of dynamic loading as in blasting. The DEM method has the capability to simulate and visualize displacements and stresses with real time, model large displacements and post-peak behavior of faults and block systems and stress wave propagation. Consequently, large displacements and rotations, general non-linear constitutive behavior for both the rock mass and the faults and joints, and time domain calculations can be accommodated in a straightforward manner (Jing and Stephansson, 2007).

2.3.3.4 Finite Element Method

Over the years, extensive research has clearly established and tested numerous FEM formulations, and the method has spread to applications in many fields of engineering and science. FEM techniques have been created for discrete and continuous problems including static and dynamic behavior with both linear and nonlinear response.

The method discretizes the domain under study by dividing the region into subdomains called elements. In order to simplify formulation and application procedures, elements are normally chosen to be simple geometric shapes

The power and utility of the finite element method lies in the use of computer codes that implement the numerical method for problems of general shape and loading. A very large number of both private and commercial FEM computer codes have been developed over the past few decades. Many of these codes (e.g., ABAQUS, ANSYS, ALGOR, NASTRAN, ADINA and Plaxis) offer very extensive element libraries and can handle linear and nonlinear problems under either static or dynamic conditions.

Advantages of Finite Element Method

- ✓ Modeling of complex geometries and irregular shapes are easier as varieties of finite elements are available for discretization of domain.
- ✓ Boundary conditions can be easily incorporated in FEM.

- ✓ Different types of material properties can be easily accommodated in modeling from element to element or even within an element.
- ✓ Higher order elements may be implemented.
- ✓ FEM is simple, compact and result-oriented and hence widely popular among engineering community.
- ✓ Availability of large number of computer software packages and literature makes FEM a versatile and powerful numerical method.

Disadvantages of Finite Element Method

- ✓ Large amount of data is required as input for the mesh used in terms of nodal connectivity and other Parameters depending on the problem.
- ✓ It requires a digital computer and fairly extensive
- ✓ It requires longer execution time compared with FEM.
- ✓ Output result will vary considerably.

3. Base model

3.1 Constitutive modeling

Nonlinear elastic (hyperbolic) model

Soil response to loading is highly nonlinear and highly dependent on the magnitude of stress. This behavior has a significant influence on the stresses and displacements developed within the reinforced structure. Nonlinear elastic (hyperbolic) model can be expected to provide acceptable prediction of the behavior of soil at relatively low shear stress levels. The soil stiffness modeled in this manner increases with increasing confining pressure and decreases with increasing stress level as shown in Figure below. A very low stiffness is assigned to elements with stress condition at failure (Brinkgreve, 2004).

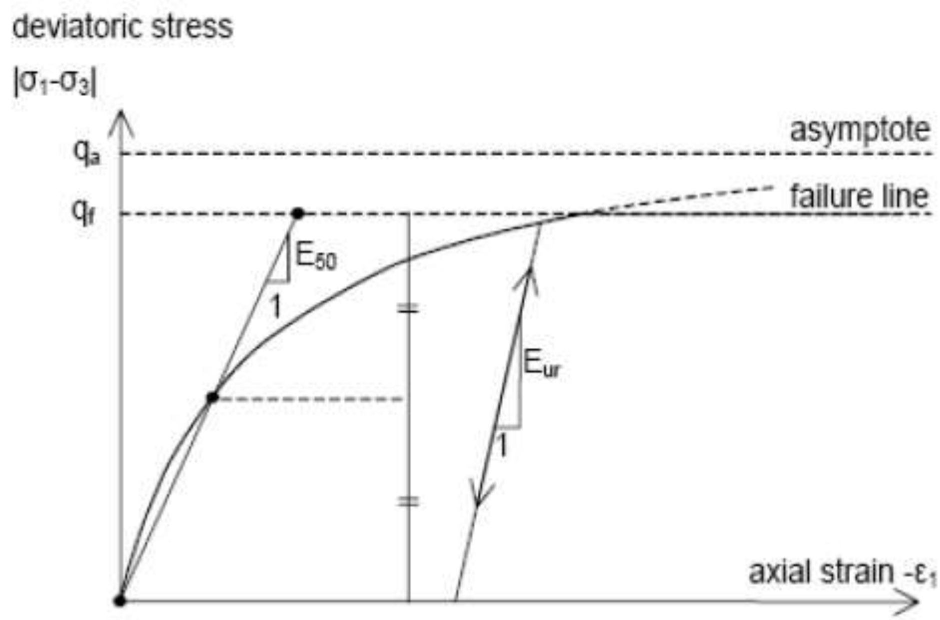


Figure 17. Hyperbolic stress-strain relation in primary loading for a standard drained triaxial test Brinkgreve, 2004

The hyperbolic model is relatively simple, well validated and reliable to represent soil behavior. The hardening soil model takes its basic idea from the hyperbolic relationship between the vertical strain and deviatoric stress, in primary triaxial loading

Soil hardening model

Similar to the Mohr-Coulomb model, limiting states of stress in the HS model are described in terms of effective stress para, i.e. the friction angle, φ, the cohesion, c, and the dilatancy angle, ψ, or in terms of undrained shear strength of soil, S_u, by specifying zero values for φ and ψ and setting c equal to S_u. The soil stiffness, however, is described

much more accurately in the HS model by using three different input stiffness values – the triaxial loading stiffness, E_{50}^{ref}, the triaxial unloading/reloading stiffness, E_{ur}^{ref}, and the odometer loading stiffness, E_{oed}^{ref}. Unlike the Mohr-Coulomb model, the HS model also accounts for stress-dependency of soil stiffness, i.e. the elastic stiffness values increase with confining stress in the HS model.
The HS model allows for plastic volume change (volumetric hardening) as well as plastic shearing due to deviatoric loading (shear hardening). Compared with the Mohr-Coulomb model, the unloading behavior of the soil is better taken into account in the HS model. The HS model may be used to calculate realistic pressure distribution below raft foundations and behind soil retaining structures (Brinkgreve, 2004). The HS model does not account for softening in which the modulus decreases with strain whereas strain hardening model for which the modulus increase with strain. The use of the HS model generally results in longer calculation time than the Mohr-Coulomb model since the material stiffness matrix is formed and decomposed in each calculation step.

Some Parameters required in the strain hardening model are the (effective) cohesion c' [kN/m2], (effective) angle of internal friction Φ [°] and the angle of dilatancy [ψ, °]. The basic Parameters for soil stiffness in soil hardening model are the secant stiffness in standard drained triaxial test E_{ref}^{50}, [N/m2], the tangent stiffness for primary odometer loading E_{ref}^{oed}, [kN/m2] and power for stress-level dependency of stiffness [m].

3.2 Parameter identification

Different Parameters had been tested in the study of anchored diaphragm walls in previous researches; some of the Parameters are soil type, depth of embedment, strut spacing and stiffness of the diaphragm wall (Yimam, 2019) (Fekadu, 2010) but less attention has been given to Parameters like anchor orientation, anchor length and grout length. Hence this study focused on the effect of the above geometric Parameters in the stability of the anchored diaphragm wall by comparing the diaphragm wall maximum horizontal deflection, vertical deflection and bending moment.

This research considers three soil types; namely red clay, black cotton and sand in combination with stratification namely red clay – sand and black cotton – sand with 3 meters depth for the red clay and black cotton soil and the sand extending to the depth of 20 m which is assumed to represents most of the soil types in Addis Ababa.

The following table presents the Parameters which are used for this study for the diaphragm wall with wall thickness of 60cm, tie-back wall Parameters and the soil properties.

Note: - EA and EI are per-unit length of the diaphragm wall

EA= Et where t = thickness of the diaphragm wall

EI = $Et^3/12$ and the concrete compressive strength = 30 MPa

Table 2. Diaphragm wall, anchor and grout Parameters for the model

Parameters	Unit	Diaphragm wall	Anchor	Grout
Type	-	Elastic	Elastic	Elastic
EA	kN/m	1.80E+07	2.00E+05	1.00E+05
EI	kNm	5.40E+05	-	-
Weight	kN/m/m	14.40	-	-
Poisson's ratio		0.15	-	-
Thickness	m	0.6	-	-
L_s (Strut spacing)	m	-	2.5	-
F_{max} (Prestress Load)	kN	-	150	-

Note: - The node to node anchor and geo-grid Parameters are adapted from (Fekadu, 2010)

Appropriate Parameters for sand is taken from literature which are assumed to be typical for fine grained granular soil and ground water level is assumed to be 3.0 below existing ground level.

For the geotechnical design which are going to be used for the base model, the design Parameters are correlated and established based on the soil investigation carried out and the design Parameters recommended for the design are as follow.

The initial stress coefficient for the expansive soil and for red silty clay soil are computed using Brooker and Ireland (1965) correlation as follows:

$K_o = 0.95 – \sin \Phi' = 0.95 – \sin 13^o = 0.725$ for the expansive soil and

$= 0.95 – \sin 20^o = 0.608$ for the red silty clay soil

$K_o = 1- \sin \Phi' = 1- \sin 38^o = 0.384$ for non-cohesive soil

The modulus of elasticity was determined considering a soft clay E = 1.75-4.2 MPa for Poisson's ratio = 0.15-0.25 and medium dense sand with Poisson's ratio = 0.3-0.35 with E = 24.5-49 MPa (Tefera, 2000)

Table 3. Soil Parameters for the model

Soil Parameters				
Parameters	Unit	Expansive soil	Red silty clay soil	Sand
Depth	m	0-3	0-3	3-20
Model*	--	HS	HS	HS
Type of material property		Undrained	Drained	Drained
Total unit weight	γ_{unsat} (kN/m^3)	16.3	16.5	19
Bulk unit weight (saturated)	γ_{sat} (kN/m^3)	19	17	20
Permeability	kx (m/day)	0.0001	0.001	0.1
	ky (m/day)	0.0001	0.001	0.1
Poisson's ratio	v (-)	0.2	0.2	0.3
Secant modulus at 50%**	E_{50ref} (kNm^2)	3,000	4,000	30,000
Odometer modulus	E_{oedref} (kNm^2)	3,000	4,000	30,000
Unloading reference Young's modulus	E_{urref} (kNm^2)	9,000	12,000	90,000
Cohesion	c' (kNm^2)	27	21	-
Effective friction angle	Φ' (º)	13	20	38
Coefficient of lateral earth pressure***	K_{0nc}	0.725	0.608	0.384
Interface	One	Rigid	Rigid	Rigid

Note:

* - HS – Hardening soil; (U) – 'undrained' capability to develop excess pore

Pressures Default Parameters used: $E_{oed}^{ref} = E_{50}^{ref}$, $E_{ur}^{ref} = 3\ E_{50}^{ref}$, $R_f = 0.9$

** - Stiffness Parameters assumed to be at reference pressure, P_{ref} = 100 kPa

*** - nc: normally consolidated

For the expansive soil the use of undrained material property is due to its low permeability

- The soil and sand material data are adapted from (Fekadu, 2010)

Table 4. Anchor length, Anchor angle and Grout length variant

Parameters	Unit	A_L1	A_L2	A_L3	A_L4	A_L5	A_L6	A_L7
Anchor length	m	7	8	9	10	11	12	13
Parameters	Unit	A_A1	A_A2	A_A3	A_A4	A_A5		
Anchor Angle	Degree(º)	15	25	35	45	55		
Parameters	Unit	G_L1	G_L2	G_L3	G_L4	G_L5	G_L6	
Grout length	m	2	3	4	5	6	7	

For the base model, this research adapted diaphragm wall thickness of 0.6 meters and excavation depth 12 meters with embedment depth of 5 meters for the anchored diaphragm wall which makes the length of the diaphragm wall 17 meters with two anchors one located 3 meters below the ground and the second 7 meters below the ground.

The excavation and construction sequence are simulated as follows: -

1. diaphragm wall construction
2. excavation to depth 4 meters and dewatering
3. first anchor installation
4. excavation to depth 8 meters and dewatering
5. second anchor installation
6. excavation to depth 12 meters and finally dewatering.

3.3 Model Generation

A representative model is shown in the Figure below including the extents of the soil structure significantly affected by the anchored diaphragm wall and the deep excavation, which has been proven by a sensitivity analysis.

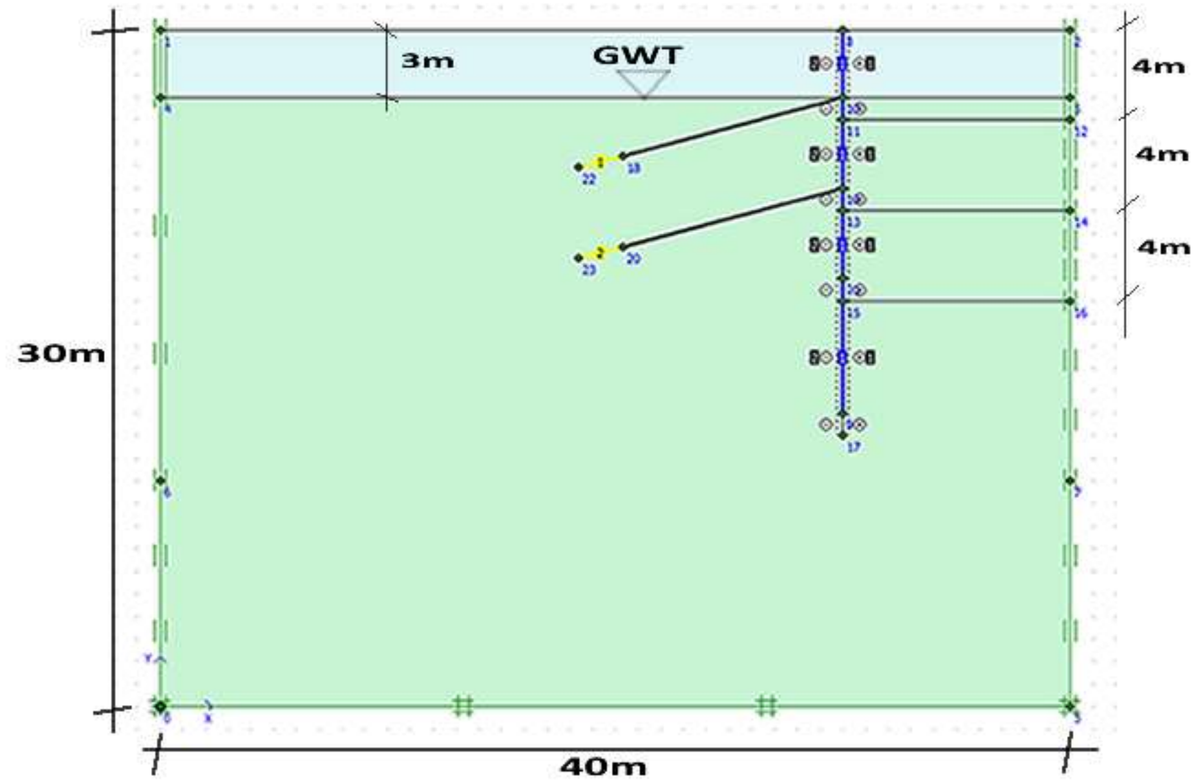


Figure 18. Typical sample model of the anchored diaphragm wall (Plaxis 2D)

The model is simulated as follows:-

- ✓ Model – Plain strain
- ✓ Elements – 15-node triangular elements
- ✓ Gravitational acceleration – 9.8 m/s^2
- ✓ Dimensions: width - 40m and depth 30m (Note:- since the model is symmetrical the dimensions represent half of the total geometry)

All the other Parameters such as soil type and their properties; anchor length, properties and orientation; grout length, orientation and properties are as stated in the tables on the previous section of chapter 3 and un-drained model is adapted for the expansive soil.

3.4 Interface modeling

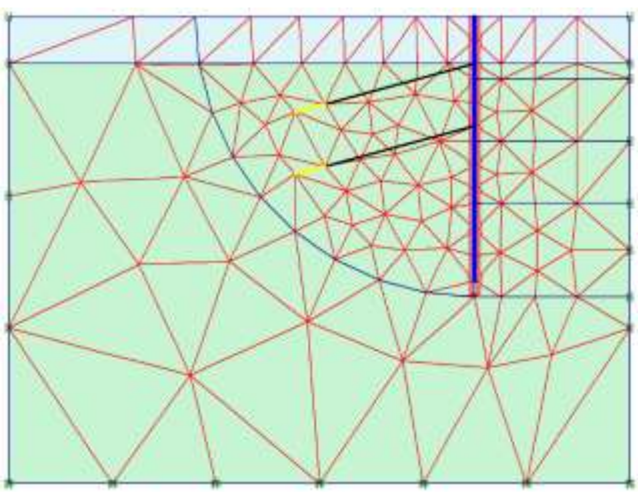
Interface modeling for representing the soil-structure interaction is mandatory for geotechnical analyses. For the finite element simulation of anchored diaphragm wall using Plaxis 2D the diagram wall interactions with the surrounding soil requires application of appropriate interaction modelling. In Plaxis 2D the interaction between the structure and the surrounding soil is simulated using the interfaces which are composed of interface elements. The stiffness matrix for interface elements is obtained by means of Newton Cotes integration (Brinkgreve, 2004).

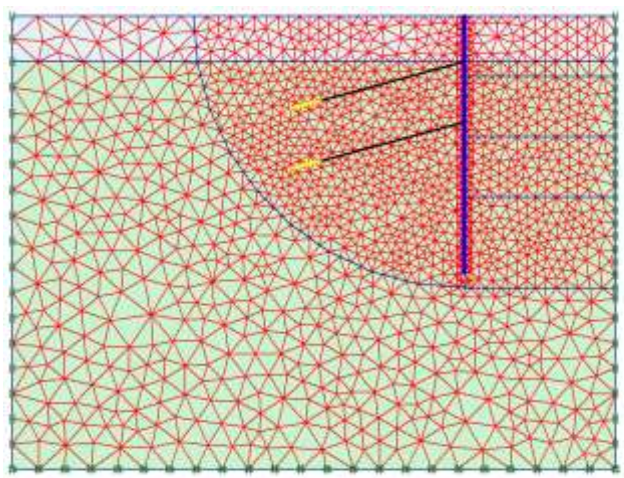
In Plaxis 2D the interface strength can be set as either rigid or manual. Rigid option is used when the interface should not have a reduced strength with respect to the strength of the surrounding soil. When rigid option is selected the corresponding R_{inter} (Strength reduction factor) equals to 1. As the result, the interface properties are the same as the soil properties in the data set. On the other hand the value of R_{inter} can be entered manually if the interface strength is set to manual. Generally speaking for the real soil-structural interaction the interface is weaker and more flexible than the surrounding soil therefore, R_{inter} should be less than 1 (Brinkgreve, 2004). For this study, a rigid interface is considered (R_{inter} = 1) based on (Brinkgreve and Shen, 2011) suggestion for reduction factor for clay/concrete interaction which is between 1 and 0.7. Therefore, the interaction is a perfectly rough soil-structure interaction and thus, the interface will have the same strength as that of the surrounding soil.

3.5 Mesh Sensitivity analysis

This section covers a mesh optimization for the base model to identify the appropriate type of mesh to be used for the analysis. The Parameters used are:-

- ✓ Soil type – Expansive soil
- ✓ Diaphragm wall thickness = 0.6 m
- ✓ Anchor strut spacing = 2.5 m
- ✓ Depth of embedment = 5 m
- ✓ Depth of excavation = 12 m
- ✓ Grout length = 2 m
- ✓ Anchor length = 10 m
- ✓ Anchor orientation = 15º





A) Very course mesh (209 element) B) Very fine mesh (2,967 element)

Figure 19. Alternative mesh sizes for sensitivity analysis

For each of the above mesh types, the maximum horizontal displacement are analyzed and compared, the results are summarized in Fig 20.

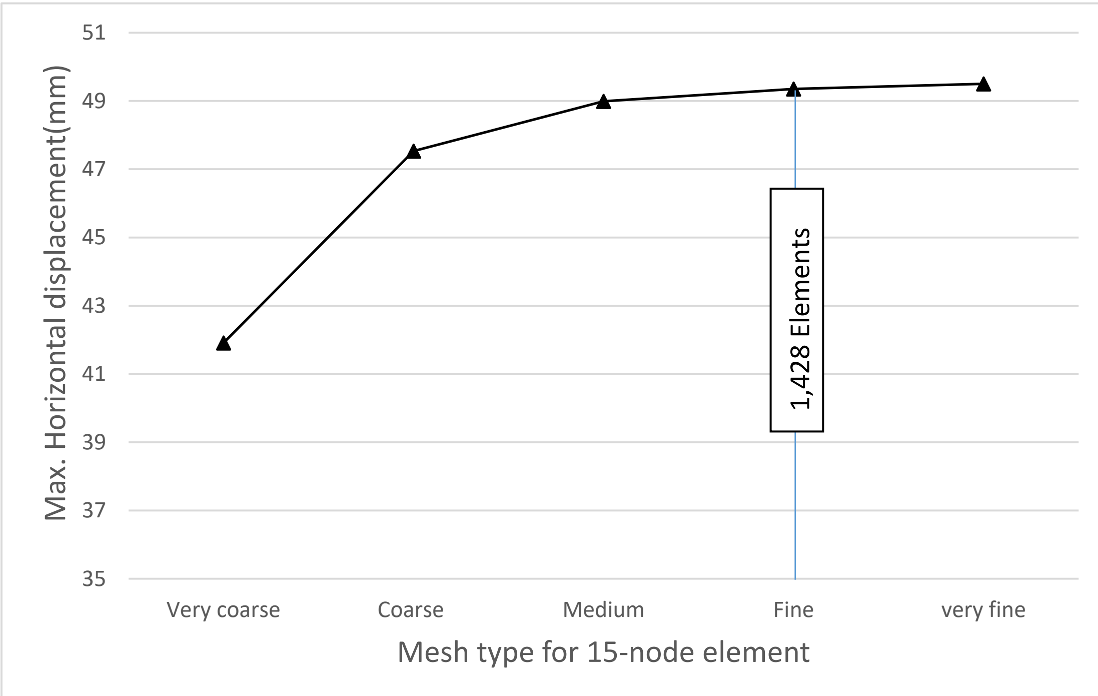


Figure 20. Mesh type VS Maximum horizontal displacement graph for the anchored diaphragm wall

It is observed that the change in the maximum horizontal displacement decreases with increasing the number of elements. Since no significant change is observed between the results of the fine mesh and the very fine mesh, the model with 1,428 elements has been used for further studies in the research.

4. Parametric study

The control parameters used in this study are the maximum horizontal displacement, vertical displacement (settlement) and bending moment in the anchored diaphragm wall.

The Plaxis 2D analysis predicts the overall deformation behavior of the wall with sufficient accuracy (Schweiger, 2009) and will be used for this parametric study.

The variables which are considered for the study are the free anchor length, grout length and orientation of the anchor with respect to the diaphragm wall. They are classified in three part and discussed the result in the following unit's as: -

- ✓ the effect of anchor length variation
- ✓ the effect of grout length variation and
- ✓ the effect of anchor orientation variation.

The effect of the three parameters has been covered in the section below in detail for the two soil types but keeping the following parameters constant for all three cases:-

1. Diaphragm wall thickness = 0.6 m
2. Anchor strut spacing = 2.5 m
3. Depth of embedment = 5 m
4. Depth of excavation = 12 m
5. The excavation sequences, as described in the previous section.

4.1 The effect of anchor length variation

The effect of the variation in anchor length has been covered in this section in detail. The Parameters varied are as follows:-

1. Anchor length: 7 m, 8 m, 9 m, 10 m, 11 m, 12 m, and 13 m
2. Anchor orientation: 15º, 35º and 45º
3. Grout length: 2 m, 4 m and 6 m

A total of 126 variants have been analyzed for studying the effect of anchor length variation which are shown in the table below.

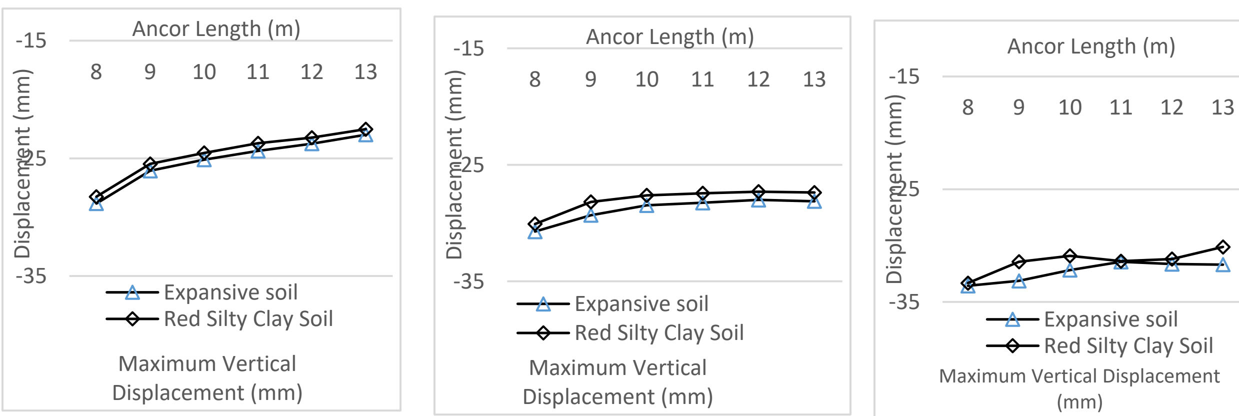
Table 5. Variants for the anchor length variation study

Expansive Soil			Red Silty Clay Soil		
Grout Length(m)	Anchor Angle(º)	Anchor Length(m)	Grout Length(m)	Anchor Angle(º)	Anchor Length(m)
2/4/6	15	7	2/4/6	15	7
		8			8
		9			9
		10			10
		11			11
		12			12
		13			13
	35	7		35	7
		8			8
		9			9
		10			10
		11			11
		12			12
		13			13
	45	7		45	7
		8			8
		9			9
		10			10
		11			11
		12			12
		13			13

4.1.1 The effect of anchor length variation for grout length 2 m.

Maximum vertical displacement/settlement

After performing the parametric study by using the input geometric, material and loading parameters, the variation of the vertical settlement with the increase in anchor length has been presented in Fig.21 for three anchor orientations and two soil profiles



a) Anchor angle 15º **b) Anchor angle 35º** **c) Anchor angle 45º**

Figure 21. Maximum vertical displacement of the diaphragm wall for GL 2 m.

For the three orientations; the increase in anchor length leads to a gradual decrease in the maximum vertical displacement. This decrease in maximum vertical displacement is due to the increase in the horizontal component of the anchorage for an increased anchor length.

It's evident that when the anchor length increases, the decrease in maximum vertical displacement reduces. The reduction of settlement is pronounced at lower angles while the use of longer anchors doesn't make sense at higher anchor orientations, as the settlement reduction is limited. For expansive soil at lower anchor orientation of 15º the length increment is effective in reducing the settlement only up to anchor length 11 m.

For anchor orientation 15º the graph become almost straight after anchor length 11 meters on the other hand for anchor orientation 35º and 45º the graph is fairly straight showing the increase in anchor length have a great effect on the lower anchor orientation. That's due to the fact that for smaller degree of anchor orientation most of the anchor part is in the assumed wedge slip surface and also the horizontal component of the anchorage force takes more portion than the vertical component.

The result for the red clay soil as observed from the parametric study is almost the same as that of the expansive soil. Therefore only the result for the expansive soil is discussed here and the result of the red clay soil is presented in the Appendix.

Maximum horizontal displacement

In the same way as the vertical displacements, the horizontal displacements are evaluated for the above cases and summarized in Figure 22.

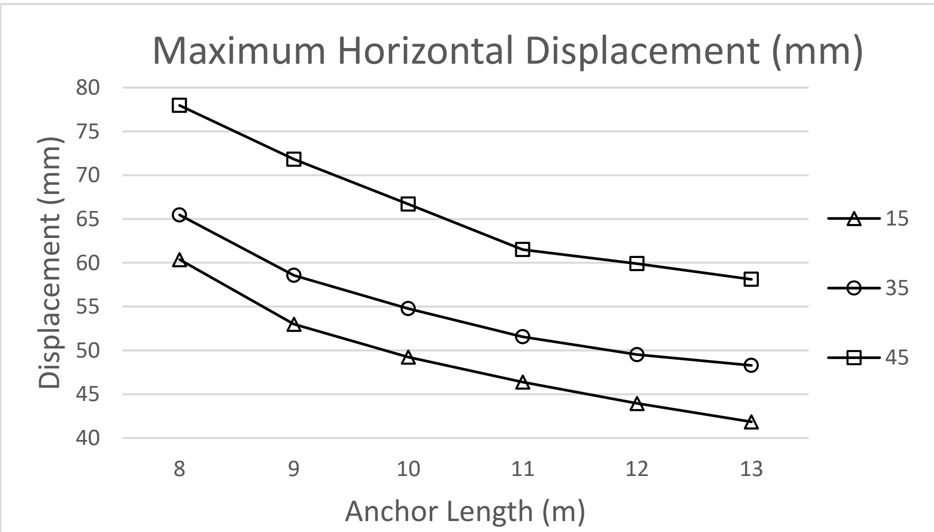


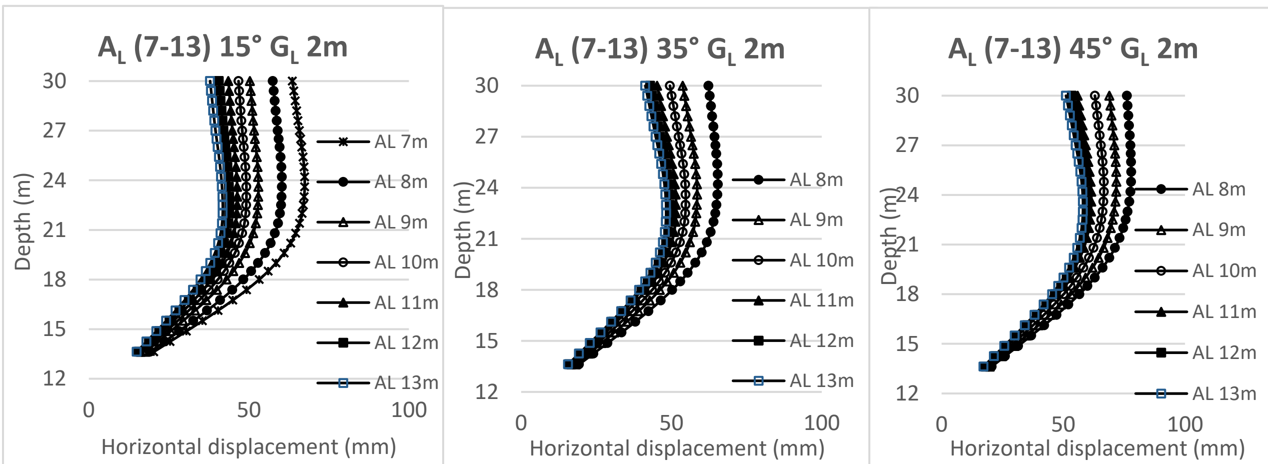
Figure 22. Maximum Horizontal displacement of the diaphragm wall for GL 2m.

The maximum horizontal displacement for the three angles, 2 meters long grout with anchor length varying from 8 to 13 is decreasing with respect to increasing the anchor length.

Based on the results of numerical analysis of maximum horizontal displacement for the three-anchor orientation, it’s obvious that the increase in the anchor length result in a decrease in the maximum horizontal displacement. The trend shows similar behavior for the three anchor angles. The relation between the anchor length and the maximum horizontal displacement is nonlinear as seen from the figure above. The decrease in the maximum horizontal displacement observed to be more at lower anchor length values but becomes insignificant for longer anchors. That’s due to the fact that after achieving adequate anchor length to transmit the stress on the diaphragm wall safely to the ground increasing the anchor length won’t increase the anchor capacity significantly. In this case after anchor length 10 meters the curve seems to be fairly straight for all three anchor orientations.

Free anchor length is provided because the anchor bond zone must be established behind the potential slip surface for a safer load transfer and anchorage (Sabatini, 1999)

A relationship between depth of diaphragm wall and horizontal displacement of the diaphragm wall is shown in Figure 23.



a) Expansive soil A_A 15º **b) Expansive soil A_A 35º** **c) Expansive soil A_A 45º**

Figure 23. Horizontal displacement of the diaphragm wall for GL 2m.

For anchor orientation 15º the decrease in horizontal displacement is bigger for the first 2 meters increase of anchor length but begin to narrow down beyond anchor length 9 meters. On the other hand for anchor orientation 35º, the decrease in horizontal displacement narrow down, for the last 3-meters increase in the anchor length.

The top half of the diaphragm wall with anchor orientation 15º seems fairly straight but when the anchor orientation change to 35º, the diaphragm wall arch back. Further increase of the anchor orientation to 45^0 result in the diaphragm wall to bend more. The increase in anchor orientation decrease the effective horizontal anchorage force and increase the vertical force which results in the bending of the diaphragm wall.

The results for 4m and 6m grout length shows a similar property and presented in the Appendix.

An investigation into the assumed wedge slip surface according to Rankine`s earth pressure theory (Tefera, 2000) has been performed for the three-anchor orientation in Fig. 24 below.

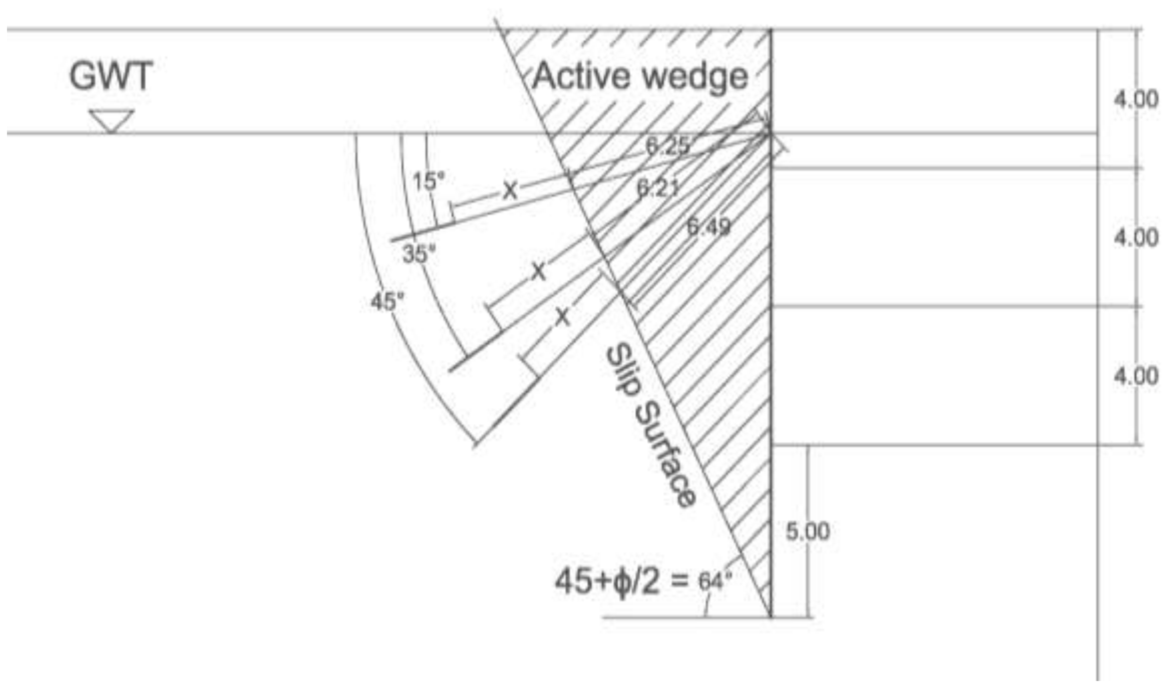


Figure 24. Assumed Wedge slip surface for G_L 2m.

For anchor orientation 15º, the assumed wedge slip surface starting from the bottom of the anchored diaphragm wall and extend to the surface with angle 45º+ϕ/2 = 64º. From the figure, it's evident that 6.25 meters is inside the active wedge which is a discounted anchor length and the anchor length can be described as 6.25 + X meters and based on the above geometry X is 2.75 meters for anchor length 9 meters.

For anchor orientation 35º its shows that 6.21 meters is inside the active wedge; therefore, the anchor length can be describes 6.21 + X meters. Similarly, for anchor length 9-meters x=2.79 meters.

Based on the parametric study it's found out that beyond anchor length 9 meters the decrease in the maximum horizontal displacement is less than 5mm for anchor orientation 15º and 35º which is insignificant in both soil types and therefore it's recommended not to increase the anchor length further than 9 meters. For anchor orientation 45º past anchor length 11 meters the change in maximum horizontal displacement is less than 5mm.

Increasing the anchor inclination also increases the vertical component of the anchor load, thus increasing the vertical load on the wall members and the underlying foundation materials (Nicholson, 1982) therefore should be avoided.

Maximum bending moment

The maximum bending moment also shows the same behavior as the maximum horizontal displacement as seen from Fig. 25.

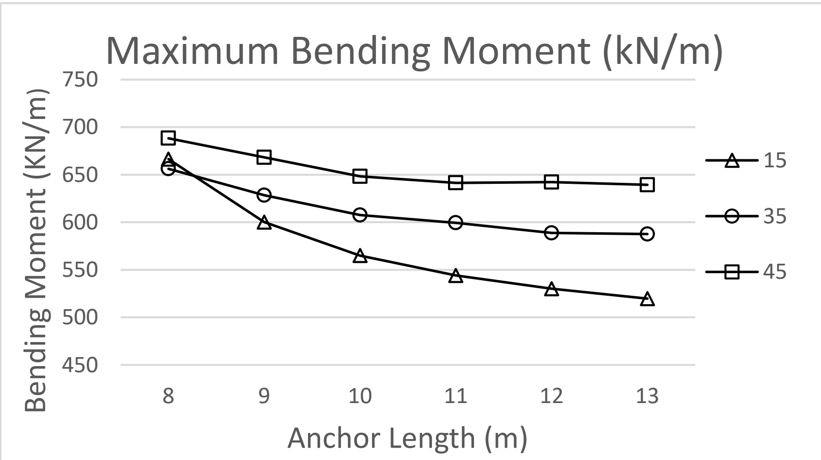
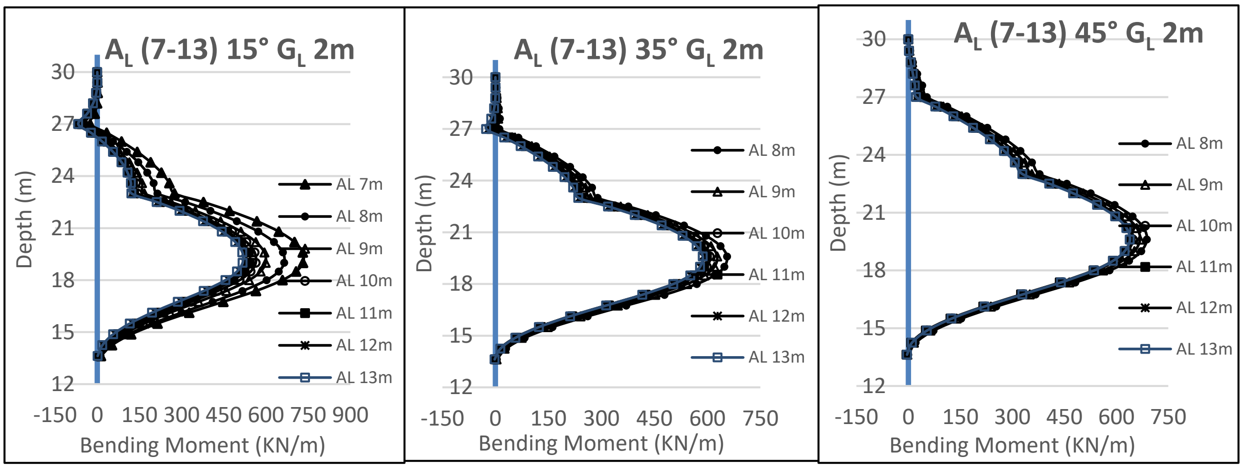


Figure 25. Maximum Bending moment of the diaphragm wall for GL 2m.

In general when the anchor length increase the maximum bending moment decrease for all three anchor orientations. It is observed that when the anchor length increase the decrease in the maximum bending moment diminishes, for example for 15º anchor orientation when the anchor length changed from 8 meters to 9 meters the corresponding drop in the maximum bending moment is 10% however for the change of anchor length from 11 meters to 12 meters the change in the maximum bending moment is only 3% even if both of them shows an increase in 1 meter of anchor length.

The decrease in the maximum bending moment is more vivid in lower anchor orientations 15º but seems almost a straight line for anchor orientations 35º and 45º.that is because when the anchor orientation increases the effective horizontal anchorage force decreases. At the lower anchor length regardless of the angel of orientation the bending moment shows a decrease.

Fig.26 presents the bending moment of the diaphragm wall for the three anchor orientation with grout length 2m.



a) Expansive soil A_A 15º **b)Expansive soil A_A 35º** **c)Expansive soil A_A 45º**

Figure 26.Bending moment of the diaphragm wall for GL 2m.

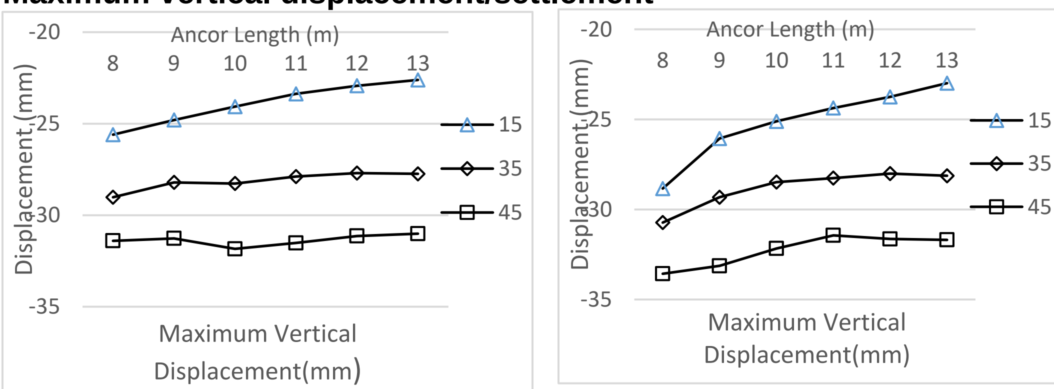
It's evident that anchor length variation affects the bending moment only at the smallest angle of orientation (15º) because of the fact that increasing the anchor orientation reduces the horizontal effective stress on the diaphragm wall; therefore it's better to use a smaller anchor orientation if the site situation allows.

The results for 4m and 6m grout length shows a similar property and presented in the Appendix.

4.1.2 The effect of anchor length variation for grout length 4m.

This section covers about the effects of maximum horizontal displacement vertical displacement (settlement) and bending moment, in the two soil types with three anchor orientations grout length 4m and free anchor length 7-13m. Fig. 27 shows the relation between the maximum vertical displacements with the anchor length for the three anchor orientation for grout length 2m and 4m.

Maximum vertical displacement/settlement



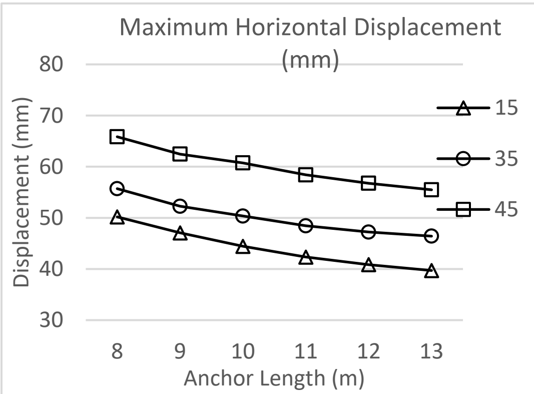
Grout length 4m **Grout length 2m**

Figure 27. Maximum Vertical displacement of the diaphragm wall for GL 2m & 4m.

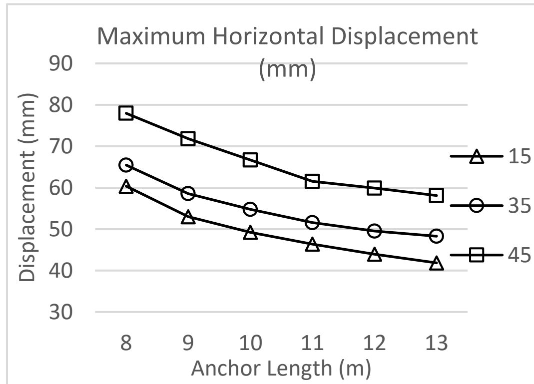
The change in maximum vertical displacement regarding the increase in anchor length for the grout length 4m shows similar trend with that of grout length 2m, the only difference being a minor increase in the magnitude. For longer anchor length the increase in the maximum vertical displacement become negligible for both grout length and anchor orientation signifying clearly that the use of very long anchors together with higher angle of orientation is redundant. The reason being once satisfactory anchorage is attained additional increase in the anchor length won't contribute to the stability of the structure significantly.

Maximum horizontal displacement

On Fig. 28 the effect of the maximum horizontal displacement of the diaphragm wall having grout length 4m and 2m for the three anchor orientation is presented as follows;



Grout length 4m



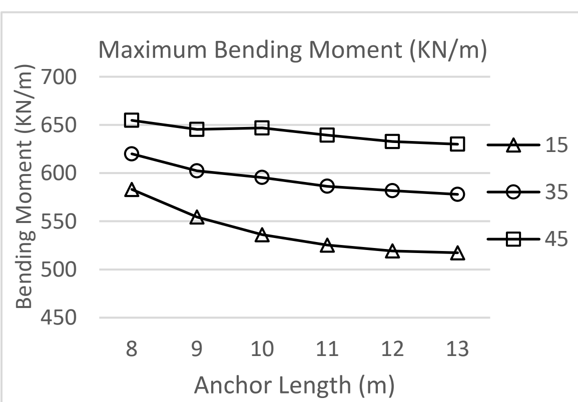
Grout length 2m

Figure 28. Maximum Horizontal displacement of the diaphragm wall for GL 4m & 2m.

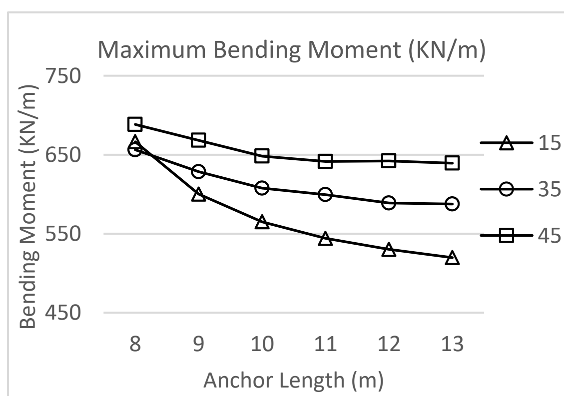
In the same way as the maximum vertical displacement, the change in maximum horizontal displacement as regards to the increase in anchor length for the grout length 4m shows similar inclination with that of grout length 2m.for grout length 4m the graph appear to be flatten compared to that of grout length 2m for all three anchor orientation in a similar manner that shows the effect of the increase in the grout length on the stability of the structure diminishes specially for very long anchor length.

Maximum bending moment

Fig. 29 compares the maximum bending moment vs anchor length for the three anchor angles for grout length 4m and 2m.



Grout length 4m



Grout length 2m

Figure 29. Maximum Bending moment of the diaphragm wall for GL 4m & 2m.

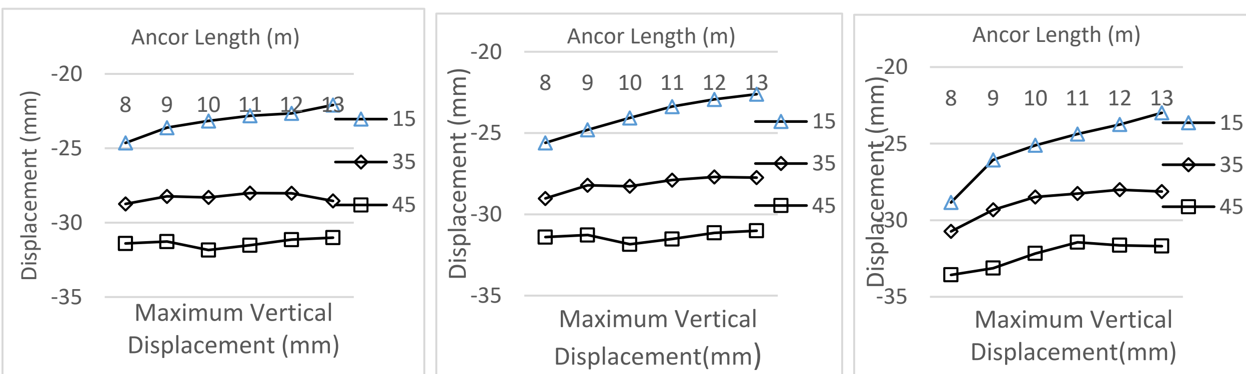
The maximum bending moment vs the anchor length graph shows a nonlinear relationship for both grout length and the three anchor orientation in general. For grout length two meters the bending moment decreases more pronounced at the beginning and flatten out near the end.

When comparing the two graphs it’s evident that the maximum bending moment vs anchor length plot shows a steady decrease for grout length 4m than that of grout length 2m. Which indicates the bending moment is affected by the anchor orientation only at the lower anchor length ranges.

4.1.3 The effect of anchor length variation for grout length 6m.

Maximum vertical displacement\settlement

Fig. 30 show the comparison of maximum vertical displacement for grout length 6m for the three orientations.



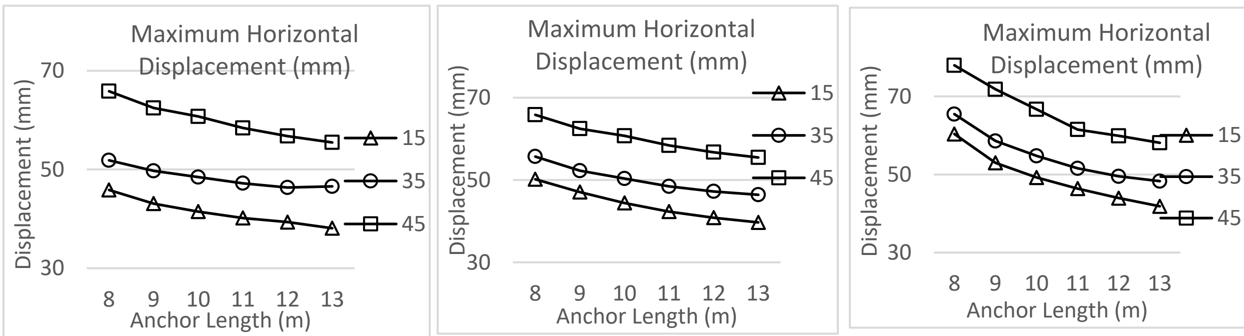
Grout length 6m **Grout length 4m** **Grout length 2m**

Figure 30. Maximum Vertical displacement of the diaphragm wall for GL 6m, 4m & 2m.

It’s noticeable that for longer grout length increasing the free anchor length is redundant. The 6m grout length graph for maximum vertical displacement verse anchor length shows almost a linear relation whereas, for grout length 4m it shows a slight decrease in the maximum vertical displacement when the anchor length increases and 2m grout length shows a clear curvature. The effect of anchor orientation is nearly the same for all three grout length variations.

Maximum horizontal displacement

On Fig. 31 the comparison of the maximum horizontal displacement of the diaphragm wall having grout length 6m vs the three anchor orientation is presented as follows;



Grout length 6m **Grout length 4m** **Grout length 2m**

Figure 31. Maximum Horizontal displacement of the diaphragm wall for GL 6, 4 & 2m.

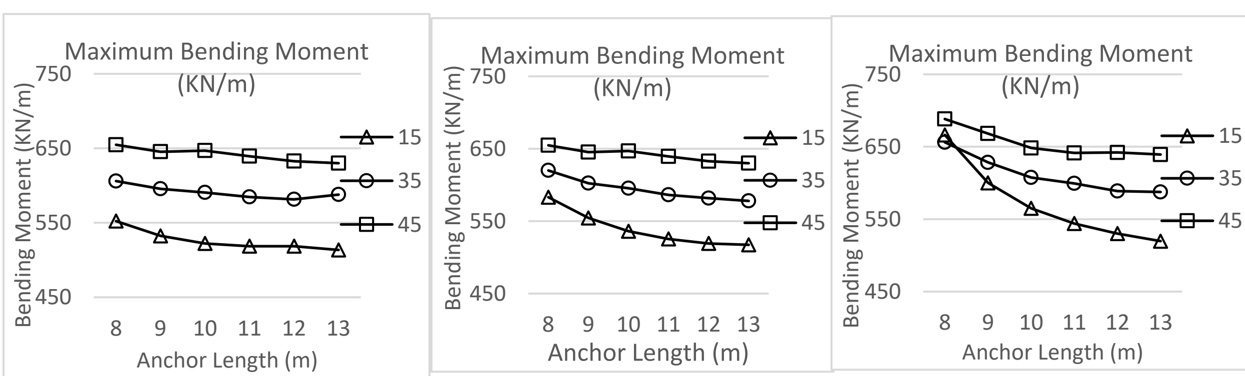
The decrease in the maximum horizontal displacement slows down as the anchor length increase for all three grout lengths and three anchor orientation.

The graph become more flat when the grout length is increased showing the increase in anchor length for anchors with longer grout diminishes. Therefore using anchors with very long grout with long anchor length is of no use and should be avoided.

The capacity of the anchor overall must balance the earth pressure developed due to the excavation and the vertical component of the anchor loads must not impose a wall force which would result in excessive deformation (Dennis, 1975). Longer anchor length more than required for the stability only result in a wall force which is responsible for excessive deformation.

Maximum bending moment

The maximum bending moment vs anchor length for the three anchor angles for grout length 6m is presented on Fig. 32 below.



Grout length 6m **Grout length 4m** **Grout length 2m**

Figure 32. Maximum Bending moment of the diaphragm wall for GL 6m, 4m & 2m.

Same as grout length 4m and 2m for 6 meters grout the maximum bending moment generally decrease as the anchor length increase but diminishing in decremented value as the anchor length increase.

From the study it’s evident that for anchor angle 15^o the bending moment shows more variation as the anchor length vary and anchor angle 35^0 and 45^0 shows that the rate of change in bending moment of the diaphragm decreases as the anchor orientation increase. The only difference between the three figures is that the rate of change in maximum bending moment vs anchor length further diminished as the grout length increased to 6m. Therefore, further increment in the anchor length will not result in a significant change in the maximum bending moment of the diaphragm wall particularly for the more inclined anchors.

Summary

The effect of free anchor length on the maximum vertical displacement/ settlement, maximum horizontal displacement and maximum bending moment are summarized below based on the above parametric study.

Increase of anchor length beyond certain limit doesn't significantly affect the maximum vertical displacement/ settlement, maximum horizontal displacement and maximum bending moment of the anchored diaphragm wall.

To quantify the limit beyond which the effect of the anchor length increasing is insignificant, a wedge slip surface is assumed which starts from the toe of the diaphragm wall and extend to the ground with angle $45^o+\phi/2$ as shown in the figure 33.

The free anchor length which is inside the active wedge is discounted since it's inside active wedge which is expected to have a lot of activities when the excavation is carried out. As shown in the figure below we appointed "X" as the free anchor length which is extended beyond the assumed slip surface. The X value which we found in this parametric study varies between 0.51m and 4.51 meters for the effect of the free anchor length become insignificant

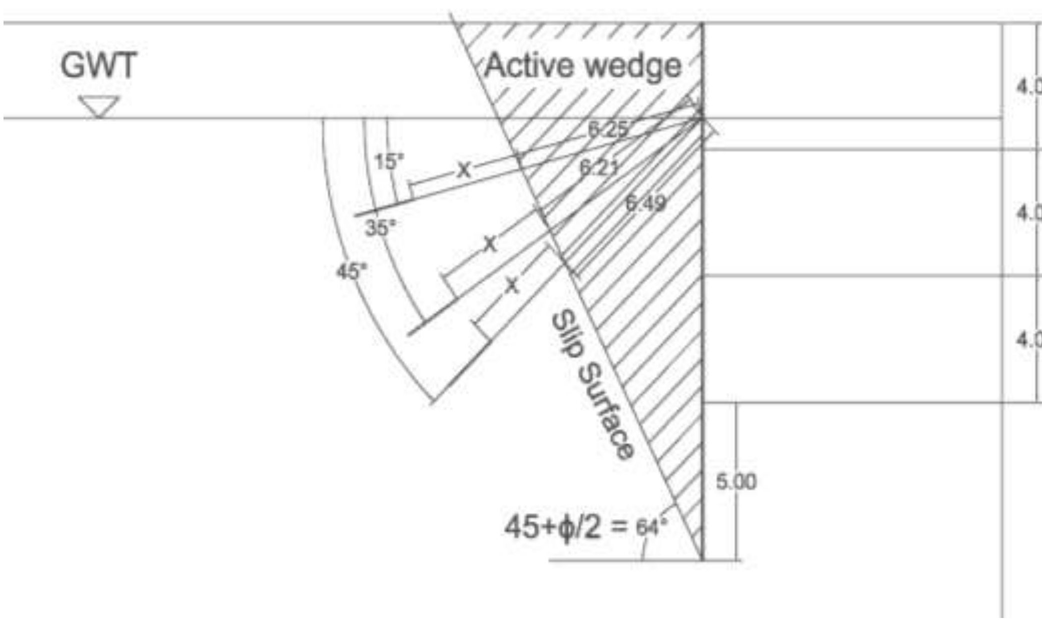


Figure 33. Determination of free anchor length beyond the slip surface or X.

X = 5 meters is the upper limit based on the parametric study and increasing the free anchor length more than 4 meters doesn't affect significantly the stability of the diaphragm wall. X = 1 meter should be adopted as the lower limit based on the parametric study since the results for anchor length 7 meters with orientation 45º and grout length 2m which gives X = 0.51 meters the prescribed ultimate state was not reached and the soil body collapses

When the anchor angle change from 15º to 45º the increase in the anchor length will significantly result in the rise of the vertical stress applied by the anchor resulting in an increase in vertical displacement/ settlement therefore it's advisable to keep the anchor length in the above recommended range

From the table below we also observe that beyond the slip surface for the same total anchor length the one which had a free anchor length more than the grout shows smaller deformation compare to the one having grout length greater than the free anchor length. Therefore, when designing the anchors it's preferable to make the length of the free anchor length greater than the grout length beyond the above assumed wedge slip surface. (Briaud, 2013) concluded that for the same overall anchor length, the grout with a short grout length had advantage over longer grout length since it will have a higher ultimate load, lower creep rate, and lower time dependent load loss.it also brings the soil stresses back further away from the wall.

Table 6. Variation in displacements and Maximum bending moment having similar total anchor length for expansive soil.

Expansive Soil						
Grout Length(m)	**Anchor Angle(°)**	**Anchor Length(M)**	**Total Anchor Length(M)**	**Maximum Verical Displacement(mm)**	**Maximum Horizontal Displacement(mm)**	**Maximum Bending moment(kN/m)**
2	15	10	12	-25.11	49.24	565
4	15	8	12	-25.6	50.19	583.09
2	15	12	14	-23.75	43.95	530.12
4	15	10	14	-24.07	44.43	536.1
2	35	10	12	-28.48	54.77	607.58
4	35	8	12	-29.02	55.7	619.98
2	35	12	14	-28.01	49.52	588.78
4	35	10	14	-28.27	50.36	595.45
2	45	10	12	-32.17	66.69	648.23
4	45	8	12	-31.4	65.85	654.74
2	45	12	14	-31.64	59.9	642.14
4	45	10	14	-31.84	60.74	646.87

4.2 The effect of anchor orientation variation

The effect of the orientation of the anchor with respect to the diaphragm wall will be covered in this section in detail for the two soil types and 10m anchor length is adapted. The excavation sequences will be as described in the previous section for all models.

The study is carried by varying the anchor orientation with respect to the diaphragm wall and take in consideration five angles 15º, 25º, 35º, 45º and 55º which can represent the possible anchor installation and analysis the results in detail as follow:-

This section covers the change in maximum horizontal displacement, vertical displacement (settlement) and bending moment for anchored diaphragm wall with anchor length 10m and grout length 2, 4 & 6m and anchor orientation 15º - 55ºm.

Maximum vertical displacement/settlement

Fig.34 presents the maximum vertical displacement of the diaphragm wall with anchor length 10 meters.

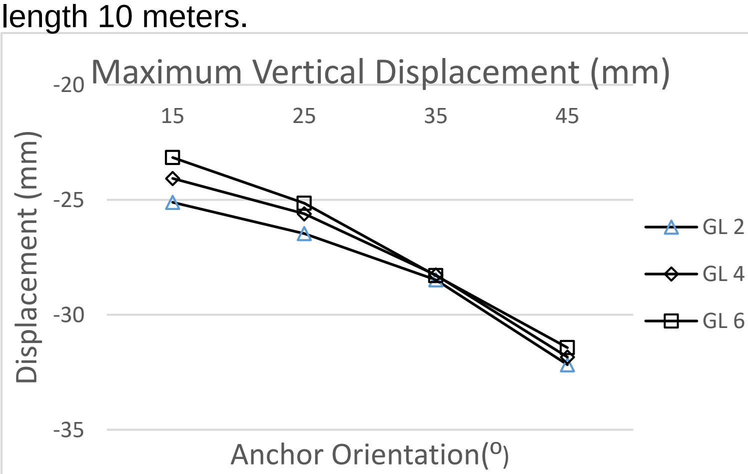


Figure 34. Maximum Vertical displacement of the diaphragm wall.

The change with respect to the increase of anchor orientation from 15º to 45º in the maximum vertical displacement/settlement on the anchored diaphragm wall where the grout length is 2, 4 and 6 meters and anchor length 10 meters is observed to increase with increasing the anchor orientation. That is because as the anchor orientation increase the effective horizontal anchorage force decrease and the effective vertical force increases resulting in an increase in vertical displacement (settlement).

It also shows for the three grout length the graph almost look identical further indicating the change in grout length doesn't have a significant impact on the magnitude of the maximum vertical displacement of the anchored diaphragm wall.

The parametric study furthermore showed that for anchor orientation 55^0 all the models failed to reach the prescribed safety factor and that anchor orientation 45^0 should be adopted as the maximum recommended orientation for the anchored diaphragm wall.

Maximum horizontal displacement

On Fig.35 below the effect of anchor orientation on the maximum horizontal displacement is presented for anchor length 10m.

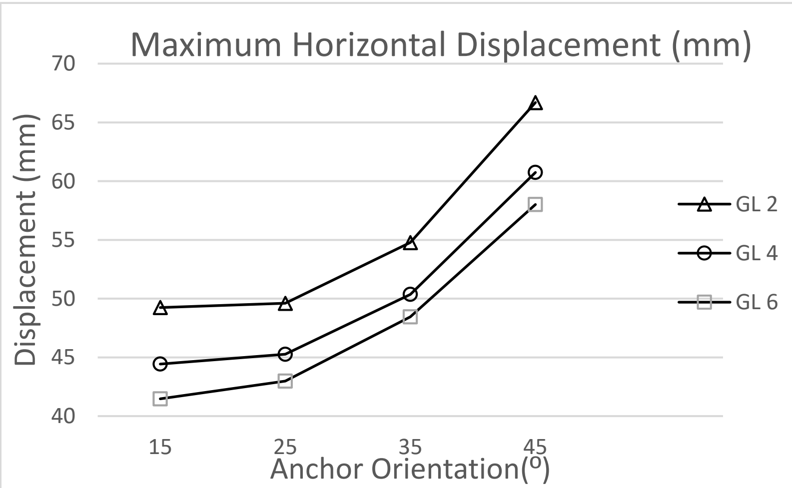


Figure 35. Maximum Horizontal displacement of the diaphragm wall.

The maximum horizontal displacement for 10 meters anchor for three grout length with anchor orientation varying from 15º-45º generally increase as the anchor orientation increase. As seen from the figure the maximum horizontal displacement increases exponentially as the anchor orientation increases. The increase in the vertical component and decrease in the horizontal component of the anchorage force as the anchor orientation increase is responsible for this increase in the maximum horizontal displacement. Therefore, lower anchor orientation should be used unless the site situation constrain the use it and required to increase the anchor orientation.

The US department of transportation on their publication regarding ground anchors and anchored systems suggest the use of 15º anchor inclination for ground anchors on the preliminary design and adjust the inclination based on the site situation but keeping the anchor inclination between 10 and 45 degrees (Sabatini, 1999). Based on the parametric study performed the US department of transportation publication which is published in 1999 is complimented.

Maximum bending moment

Fig.36 shows the effect of anchor orientation on the maximum bending moment for anchor length 10m.

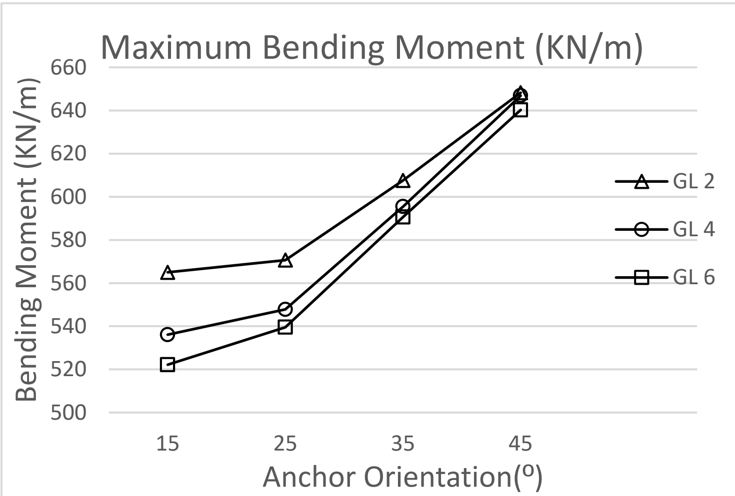


Figure 36. Maximum Bending moment of the diaphragm wall.

The results indicate that the diaphragm wall is more stable at lower anchor orientation since the effective horizontal anchorage is maximized with little vertical force and that the lowest possible anchor orientation should be adopted considering the site situation and the availability of equipment’s and methods of installation of anchors.

Similarly the curves for all three grout length are almost identical signifying that change in grout length have little or insignificant effect on the maximum bending moment of the diaphragm wall.

4.3 The effect of Grout length variation

The effect of the grout length variation in the diaphragm wall will be covered in this section in detail for 10m anchor length. The excavation sequences will be as described in the previous section for all models.

The study is carried by way of varying the grout length and considering six alternatives 2m, 3m, 4m, 5m, 6m or 7m which we believe can represent the possible length of grout. The effects of the grout length variation are discussed in detail based on the parametric study results obtained from this research.

The section covers the change in maximum horizontal displacement, maximum vertical displacement (settlement) and maximum bending moment in the two soil types for anchored diaphragm wall with anchor length 10m and anchor orientation 15º, 35º and 45º by varying the grout length from 2m to 7m in.

Maximum vertical displacement/settlement

Fig.37 shows the effect of grout length on the maximum vertical displacement for anchor length 10m.

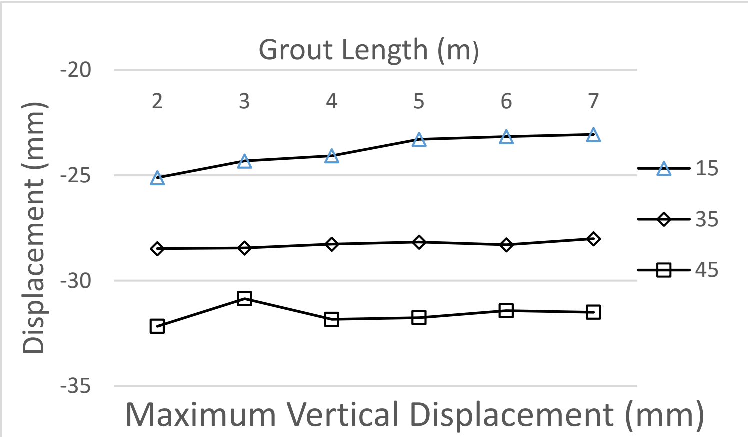


Figure 37. Maximum Vertical displacement of the diaphragm wall for G_L variation.

The change with respect to the increase of grout length from 2 m to 7 m in the maximum vertical displacement/settlement on the anchored diaphragm wall for three anchor orientation is observed generally to decrease with increase of grout length.

The decrease in the maximum vertical displacement is more pronounced for anchor orientation 15º. For anchor orientation 35º a fairly straight line is observed and for 45^0 also it's a straight line with exception in the result of anchor length 3 meters which shows variation from the other result which is understandable given the numerical representation isn't always perfect and might result in this kind of unexpected results which is tolerable.

Maximum horizontal displacement

Fig.38 shows the effect of grout length on the maximum horizontal displacement for anchor length 10m.

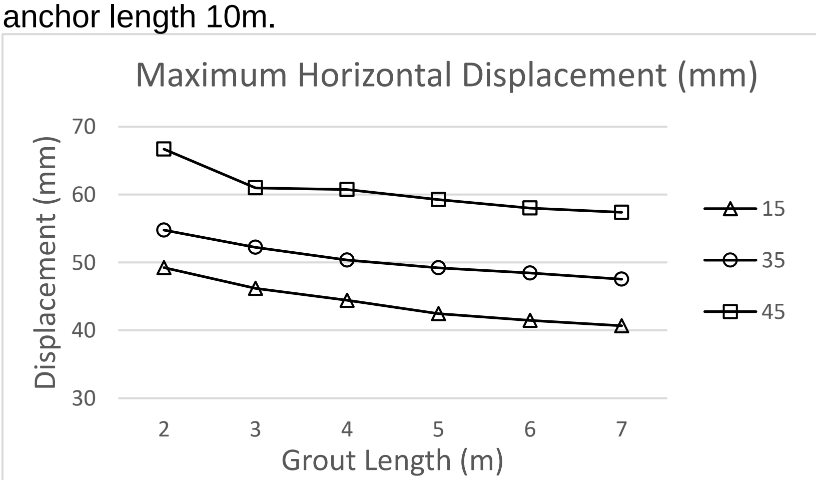


Figure 38. Maximum Horizontal displacement of the diaphragm wall for GL variation.

Anchor orientation 35º and 45º shows similar property as that of anchor orientation 15º indicating the rate of change in the maximum horizontal displacement decreases as the grout length increase. Indicating further increment in the grout length will not affect the maximum horizontal displacement significantly.

Also it’s shown that the increase in the anchor orientation increases the maximum horizontal displacement of the anchored diaphragm wall.

Maximum bending moment

Fig. 39 presents the relation between grout length vs maximum bending moment.

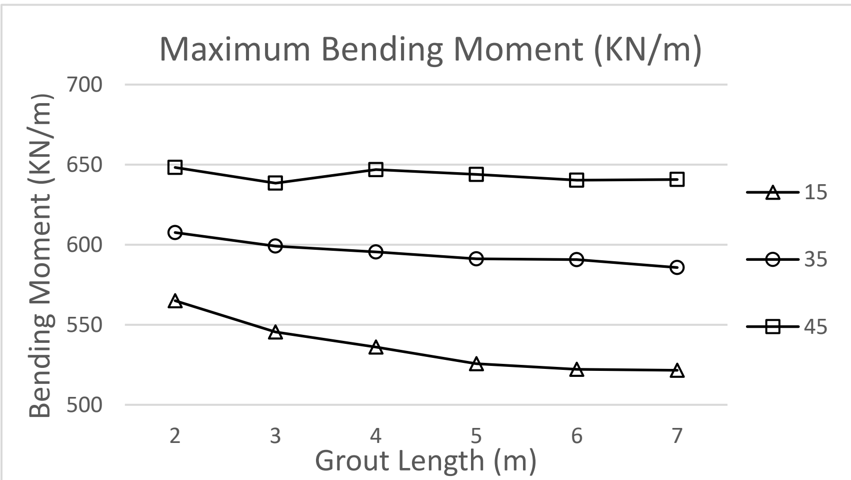


Figure 39. Maximum bending moment of the diaphragm wall for GL variation.

It’s evident that the rate of which the maximum bending moment decrease drop with increasing the grout length.

It also shows the increase in anchor orientation result in the increase in the maximum bending moment of the diaphragm wall.

For all the three anchor orientation the change in the maximum bending moment is 5% for an increase in grout length by 5 meters from 2m to 7m which indicates the increase in grout length have little or no effect on the maximum bending moment of the diaphragm wall.

(P.Xanthakos, 1991) Suggest the minimum grout length to be 3m and grout length greater than 10m don’t have any real effect on the anchor capacity and should be adopted as the upper limit. In general the estimation of grout length should be based on the relevant geological and geotechnical data.

5. Conclusion and recommendation

There are a number of researches on anchored diaphragm wall but little is done regarding the effect of geometric properties of the ground anchors. On this research by performing a parametric study using Plaxis 2D its been able to provide a guide line to follow regarding the ground anchors when designing anchored diaphragm walls.
The following conclusion and recommendation are reached based on the study:-

- ✓ The study show that increase of anchor length beyond certain limit doesn’t significantly affect the maximum (vertical & horizontal) displacement and the maximum bending moment of the anchored diaphragm wall since the main purpose of the free anchor length is to give adequate clearance from the slip surface for the grout\fixed anchor length which transfer the load from the diaphragm wall to the ground.
- ✓ The free anchor length which is inside the active wedge is discounted since it’s inside the active wedge which is expected to have a lot of movements when the excavation is carried out and also after excavation end.
- ✓ A minimum of 1m free anchor length beyond the assumed wage slip surface should be provided based on the study. (Sabatini, 1999) suggested that, the minimum free anchor length beyond the assumed wage slip surface should be 1.5m or 0.2H (depth of excavation) whichever is greater.
- ✓ When the anchor angle increase, the increase in the anchor length will significantly increase the vertical stress applied by the anchor resulting in an increase in vertical displacement/ settlement; The capacity of the anchor overall must balance the earth pressure developed due to the excavation and the vertical component of the anchor loads must not impose a wall force which would result in excessive deformation (Dennis, 1975); therefore it’s advisable to keep the anchor length in the above recommended range.
- ✓ The study also shows that beyond the slip surface for the same total anchor length the model which have a free anchor length more than the grout length displays smaller deformation compared to the model having grout length greater than the free anchor length. Therefore when designing the anchors, it’s preferable to make the length of the free anchor length greater than the grout length beyond the

assumed wedge slip surface. (Briaud, 2013) concluded that for the same overall anchor length, the grout with a short grout length had advantage over longer grout length since it will have a higher ultimate load, lower creep rate, and lower time dependent load loss.it also brings the soil stresses back further away from the wall.

- ✓ The study shows how the anchor orientation affect the stress induced on the diaphragm wall and that increasing the anchor orientation increase the vertical stress induced on the anchored diaphragm wall and decrease the effective horizontal bond reaction. The main objective of the anchors being to transfer the horizontal stress induced due to the earth pressure on the diaphragm wall to the ground safely decreasing the effective horizontal bond reaction affects the stability of the diaphragm wall negatively.
- ✓ Increasing the anchor orientation has a significance effect on the stability of the anchored diaphragm wall and a minimum orientation of 15º shall be used in the design due to practical technical drawbacks on the installation of anchors with smaller anchor orientation. Subsequent studies should be done to investigate the effect of anchor orientation less than 15º if equipment's and methods to install the anchors in such cases developed in the future.
- ✓ Increasing the anchor orientation beyond 45º shows failure in almost all of the models and should be avoided
- ✓ Based on the study, the recommended anchor orientation is between 15º to 45º and the smaller the orientation the more the structural stability. The US department of transportation on their publication regarding ground anchors and anchored systems suggest the use of 15º anchor inclination for ground anchors on the preliminary design and adjust the inclination based on the site situation but keeping the anchor inclination between 10 and 45 degrees (Sabatini, 1999).
- ✓ Reasons to increase the anchor orientation strictly should be to avoid obstacles, presence of adjacent foundations or structures or to reach a firm strata. Since the increase in anchor orientation reduced the effective horizontal reaction which tend to tie back the diaphragm wall to the ground to minimize ground movement and

increase the vertical force which pulls the anchored diaphragm wall into the ground.

Over all, based on the study it's evident that lower anchor orientation is effective and increasing the anchor length more than the value recommended in this study doesn't result in significant increase in the capacity of the anchors and is uneconomical thus should be avoided.

All the above design parameters are proposed to provide a range of representative design values to engineers who are unfamiliar with anchor design and should be adjusted based on the site condition and equipment's accessible for the construction.

Further study should be done with varying wall, soil and excavation Parameters to investigate the anchor geometric parameter effects in a broader sense to formulate a general specification

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Appendix

Appendix 1. Combinations for the parametric study

Soil 1	Soil 1	Soil 1
AL1	AL1	AL1
AA1	AA1	AA1
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL1	AL1	AL1
AA3	AA3	AA3
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL1	AL1	AL1
AA5	AA5	AA5
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL2	AL2	AL2
AA1	AA1	AA1
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL2	AL2	AL2
AA3	AA3	AA3
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL2	AL2	AL2
AA5	AA5	AA5
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL3	AL3	AL3
AA1	AA1	AA1
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL3	AL3	AL3
AA3	AA3	AA3
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL3	AL3	AL3
AA5	AA5	AA5
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL4	AL4	AL4
AA1	AA1	AA1
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL4	AL4	AL4
AA3	AA3	AA3
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL4	AL4	AL4
AA5	AA5	AA5
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL5	AL5	AL5
AA1	AA1	AA1
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL5	AL5	AL5
AA3	AA3	AA3
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL5	AL5	AL5
AA5	AA5	AA5
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1	Soil 1	Soil 1	Soil 1	Soil 1	Soil 1	Soil 1
AL6	AL6	AL6	AL6	AL6	AL6	AL6	AL6	AL6
AA1	AA1	AA1	AA3	AA3	AA3	AA5	AA5	AA5
GL1	GL3	GL5	GL1	GL3	GL5	GL1	GL3	GL5

Soil 1	Soil 1	Soil 1	Soil 1	Soil 1	Soil 1	Soil 1	Soil 1	Soil 1
AL7	AL7	AL7	AL7	AL7	AL7	AL7	AL7	AL7
AA1	AA1	AA1	AA3	AA3	AA3	AA5	AA5	AA5
GL1	GL3	GL5	GL1	GL3	GL5	GL1	GL3	GL5

Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2
AL1	AL1	AL1	AL1	AL1	AL1	AL1	AL1	AL1
AA1	AA1	AA1	AA3	AA3	AA3	AA5	AA5	AA5
GL1	GL3	GL5	GL1	GL3	GL5	GL1	GL3	GL5

Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2
AL2	AL2	AL2	AL2	AL2	AL2	AL2	AL2	AL2
AA1	AA1	AA1	AA3	AA3	AA3	AA5	AA5	AA5
GL1	GL3	GL5	GL1	GL3	GL5	GL1	GL3	GL5

Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2
AL3	AL3	AL3	AL3	AL3	AL3	AL3	AL3	AL3
AA1	AA1	AA1	AA3	AA3	AA3	AA5	AA5	AA5
GL1	GL3	GL5	GL1	GL3	GL5	GL1	GL3	GL5

Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2	Soil 2
AL4	AL4	AL4	AL4	AL4	AL4	AL4	AL4	AL4
AA1	AA1	AA1	AA3	AA3	AA3	AA5	AA5	AA5
GL1	GL3	GL5	GL1	GL3	GL5	GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL5	AL5	AL5
AA1	AA1	AA1
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL5	AL5	AL5
AA3	AA3	AA3
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL5	AL5	AL5
AA5	AA5	AA5
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL6	AL6	AL6
AA1	AA1	AA1
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL6	AL6	AL6
AA3	AA3	AA3
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL6	AL6	AL6
AA5	AA5	AA5
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL7	AL7	AL7
AA1	AA1	AA1
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL7	AL7	AL7
AA3	AA3	AA3
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL7	AL7	AL7
AA5	AA5	AA5
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL4	AL4	AL4
AA2	AA2	AA2
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL4	AL4	AL4
AA5	AA5	AA5
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL4	AL4	AL4
AA2	AA2	AA2
GL1	GL3	GL5

Soil 2	Soil 2	Soil 2
AL4	AL4	AL4
AA5	AA5	AA5
GL1	GL3	GL5

Soil 1	Soil 1	Soil 1
AL4	AL4	AL4
AA1	AA1	AA1
GL2	GL4	GL6

Soil 1	Soil 1	Soil 1
AL4	AL4	AL4
AA3	AA3	AA3
GL2	GL4	GL6

Soil 1	Soil 1	Soil 1
AL4	AL4	AL4
AA5	AA5	AA5
GL2	GL4	GL6

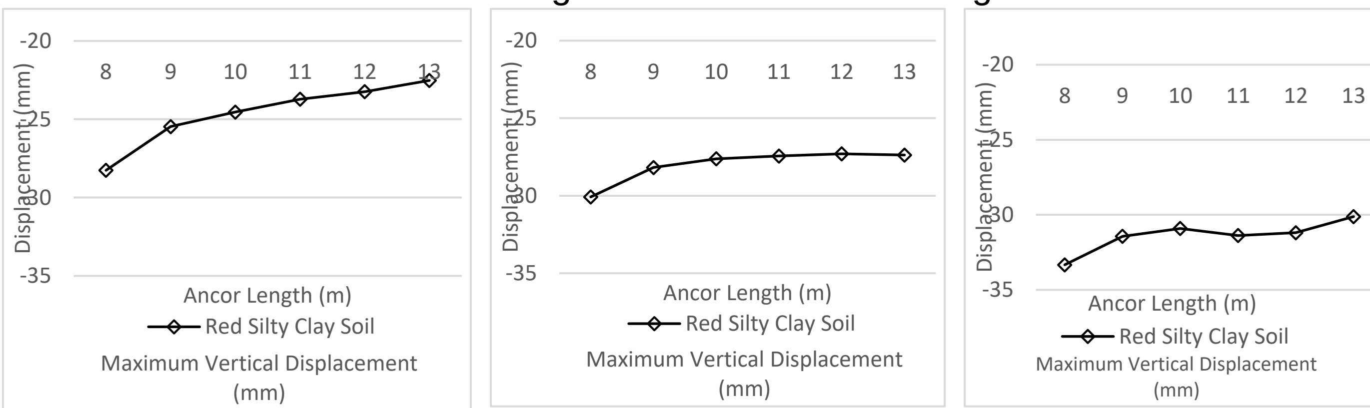
Soil 2	Soil 2	Soil 2
AL4	AL4	AL4
AA1	AA1	AA1
GL2	GL4	GL6

Soil 2	Soil 2	Soil 2
AL4	AL4	AL4
AA3	AA3	AA3
GL2	GL4	GL6

Soil 2	Soil 2	Soil 2
AL4	AL4	AL4
AA5	AA5	AA5
GL2	GL4	GL6

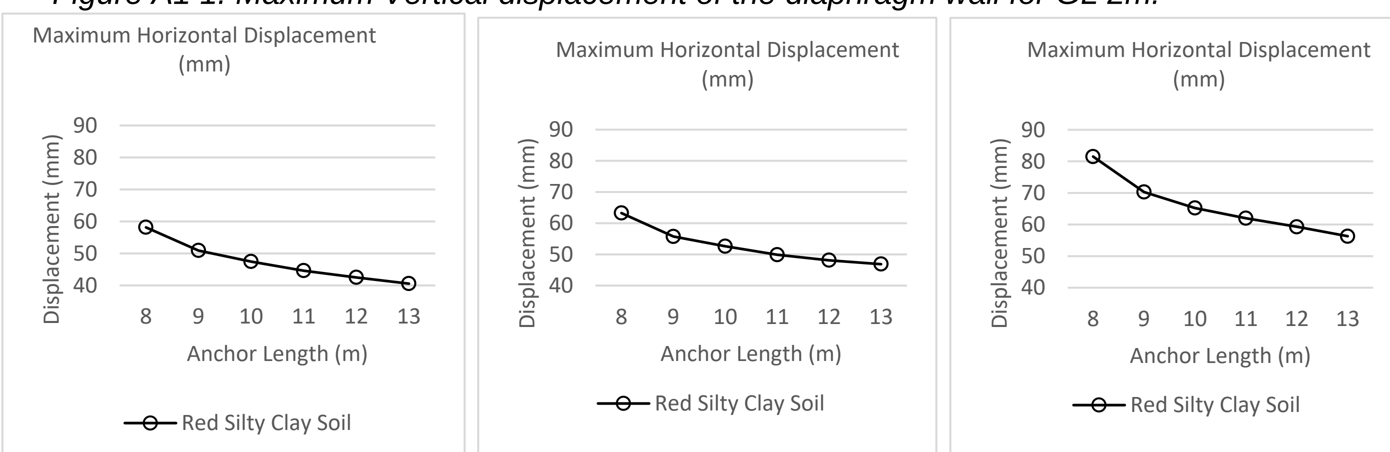
Appendix 2. Results from the parametric study

2.1 The effect of anchor length variation for Grout length 2m.



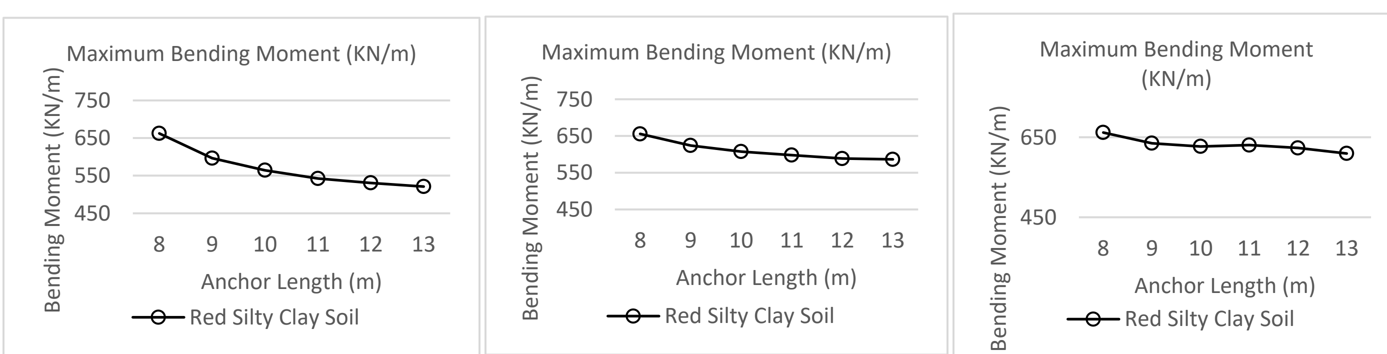
a) Anchor angle 15º **b) Anchor angle 35º** **c) Anchor angle 45º**

Figure A1 1. Maximum Vertical displacement of the diaphragm wall for GL 2m.



a) Anchor angle 15º **b) Anchor angle 35º** **c) Anchor angle 45º**

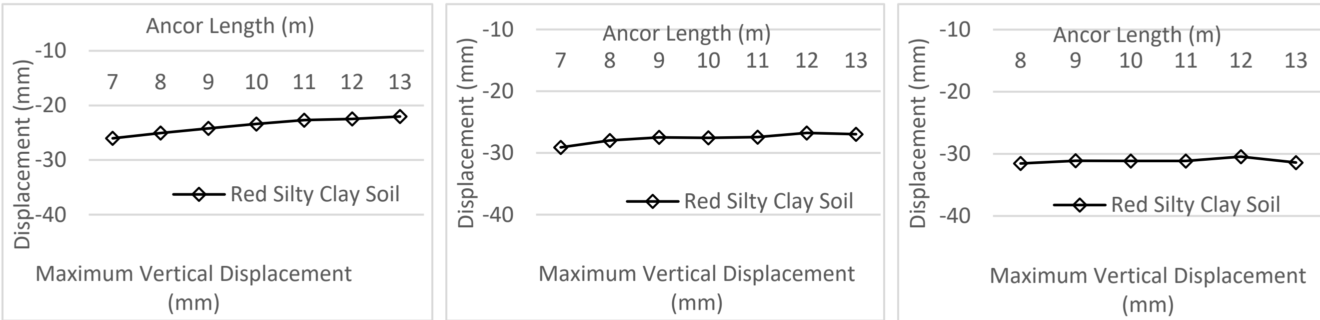
Figure A1 2. Maximum Horizontal displacement of the diaphragm wall for GL 2m.



a) Anchor angle 15º **b) Anchor angle 35º** **c) Anchor angle 45º**

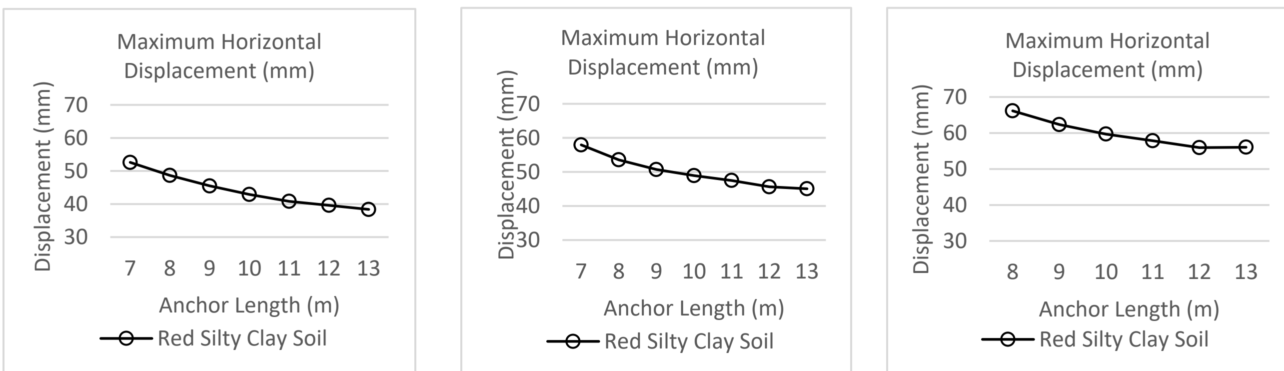
Figure A1 3. Maximum Bending moment of the diaphragm wall for GL 2m.

2.2 The effect of anchor length variation for Grout length 4m.



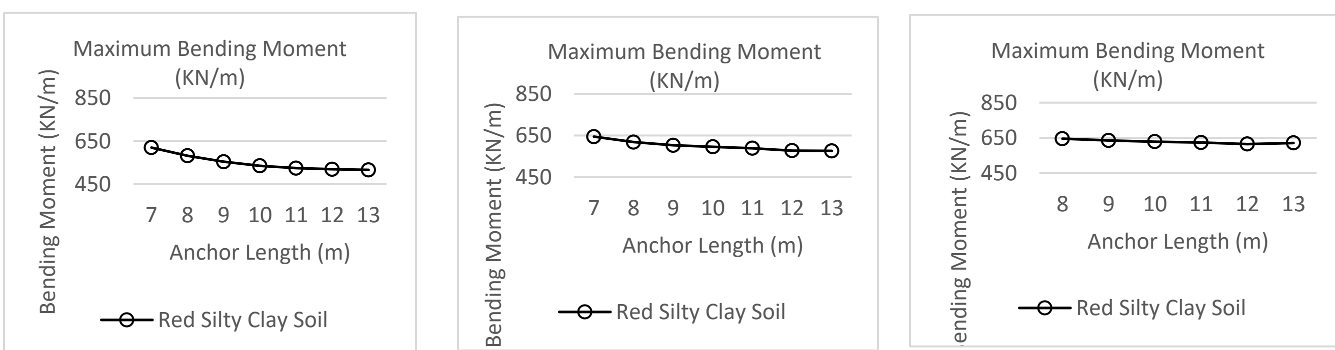
a) *Anchor angle 15º* **b) Anchor *angle 35º*** **c) Anchor *angle 45º***

Figure A1 4. Maximum Vertical displacement of the diaphragm wall for GL 4m.



a) *Anchor angle 15º* **b) Anchor *angle 35º*** **c) Anchor *angle 45º***

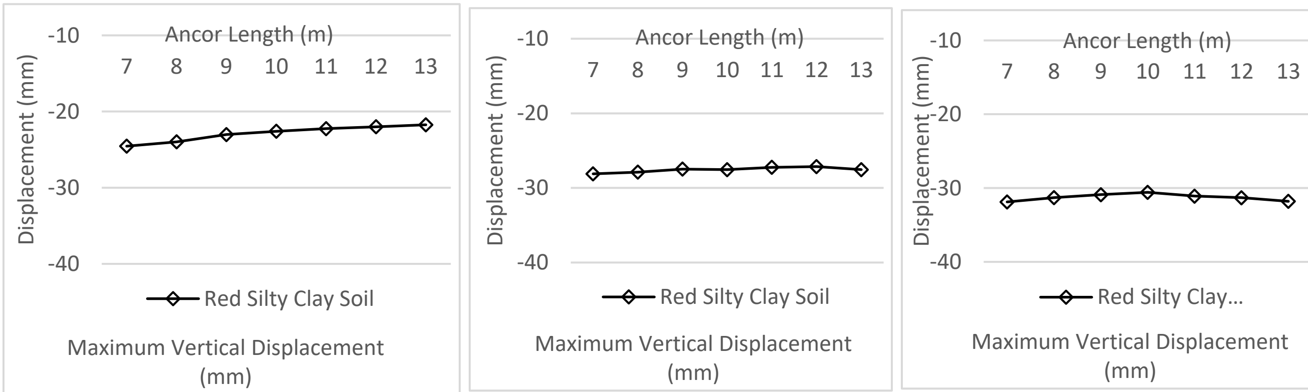
Figure A1 5. Maximum Horizontal displacement of the diaphragm wall for GL 4m



a) *Anchor angle 15º* **b) Anchor *angle 35º*** **c) Anchor *angle 45º***

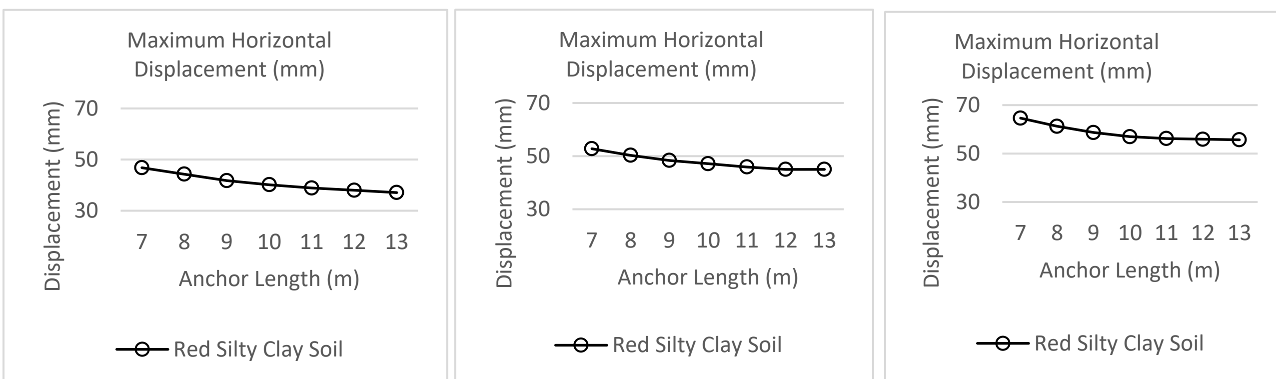
Figure A1 6. Maximum Bending moment of the diaphragm wall for GL 4m.

2.3 The effect of anchor length variation for Grout length 6m.



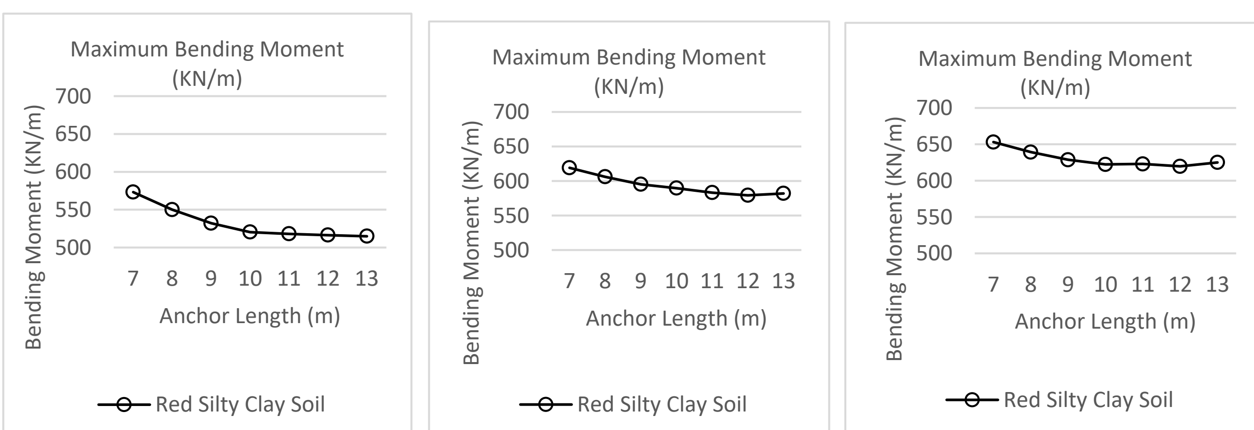
a) *Anchor angle 15º* **b) Anchor *angle 35º*** **c) Anchor *angle 45º***

Figure A1 7. Maximum Vertical displacement of the diaphragm wall for GL 6m.



a) *Anchor angle 15º* **b) Anchor *angle 35º*** **c) Anchor *angle 45º***

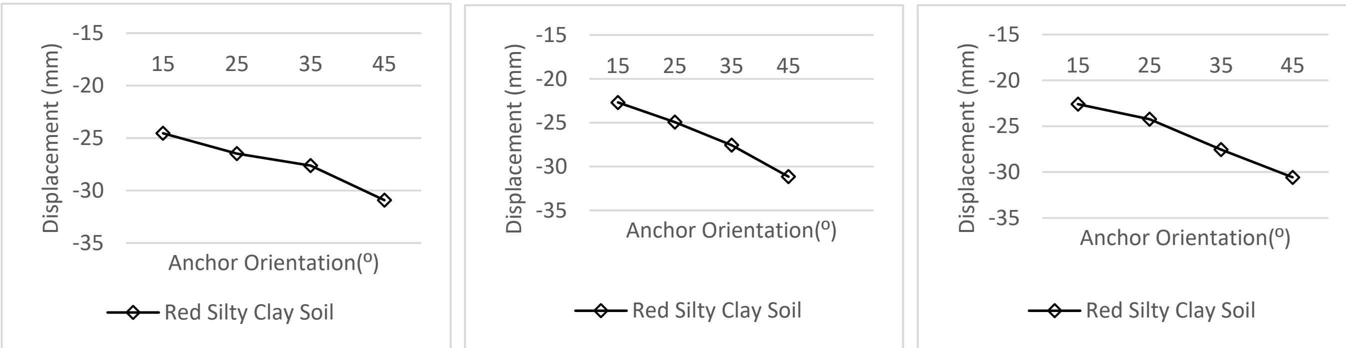
Figure A1 8. Maximum Horizontal displacement of the diaphragm wall for GL 6m.



a) *Anchor angle 15º* **b) Anchor *angle 35º*** **c) Anchor *angle 45º***

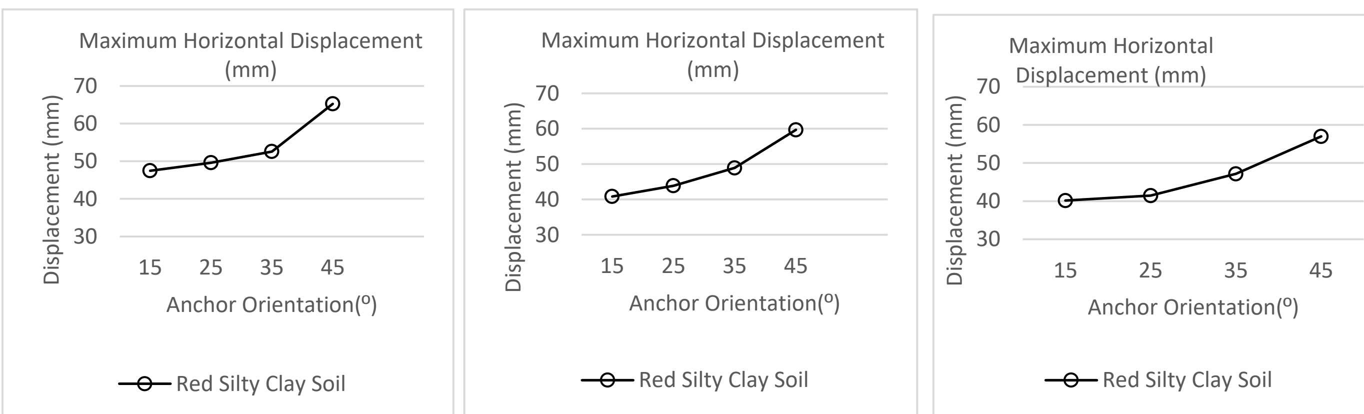
Figure A1 9. Maximum Bending moment of the diaphragm wall for GL 6m.

2.4 The effect of anchor orientation variation



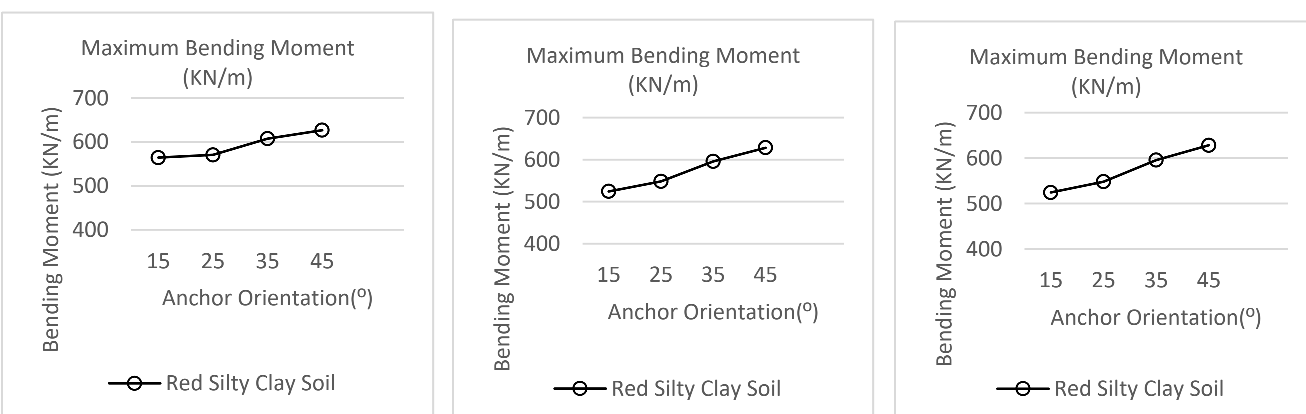
a) Grout length 2m **b) Grout length 4m** **c) Grout length 6m**

Figure A1 10. Maximum Vertical displacement of the diaphragm wall for A_A variation.



a) Grout length 2m **b) Grout length 4m** **c) Grout length 6m**

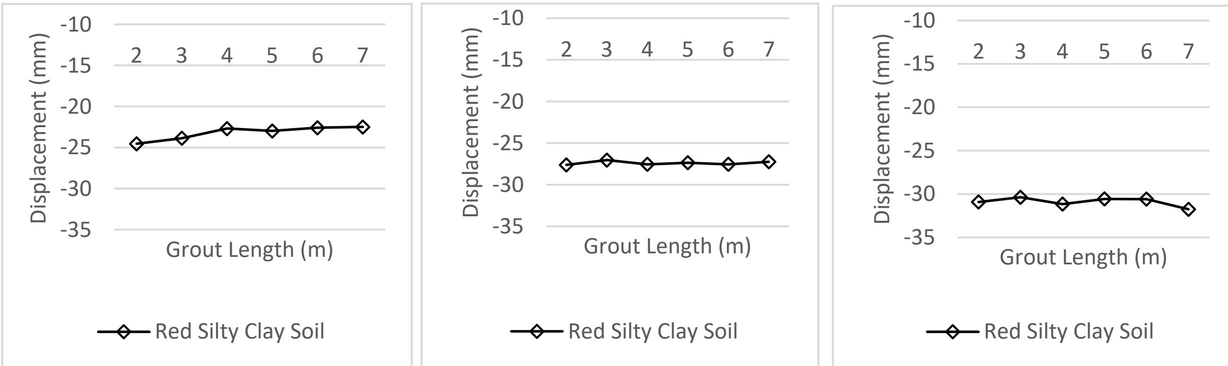
Figure A1 11. Maximum Horizontal displacement of the diaphragm wall for A_A variation.



a) Grout length 2m **b) Grout length 4m** **c) Grout length 6m**

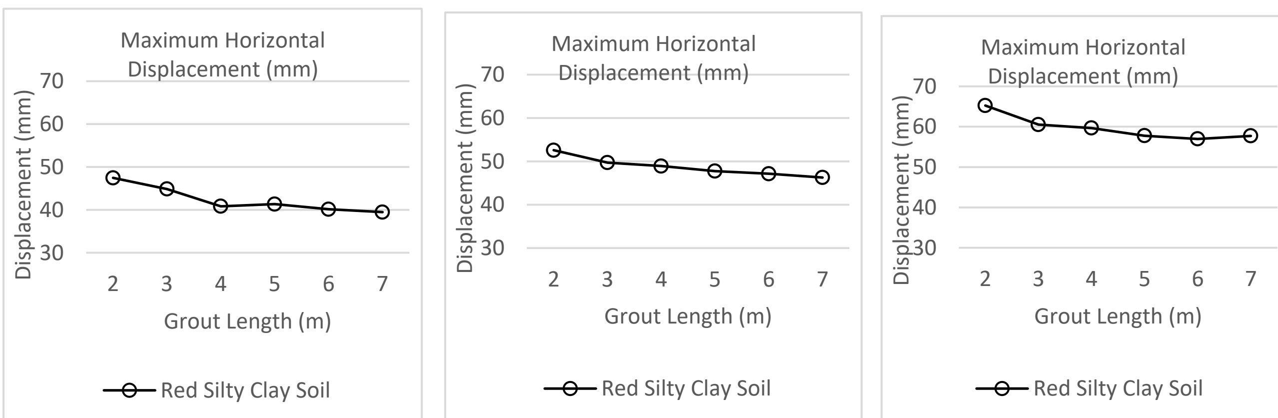
Figure A1 12. Maximum Bending moment of the diaphragm wall for A_A variation.

2.5 The effect of Grout length variation



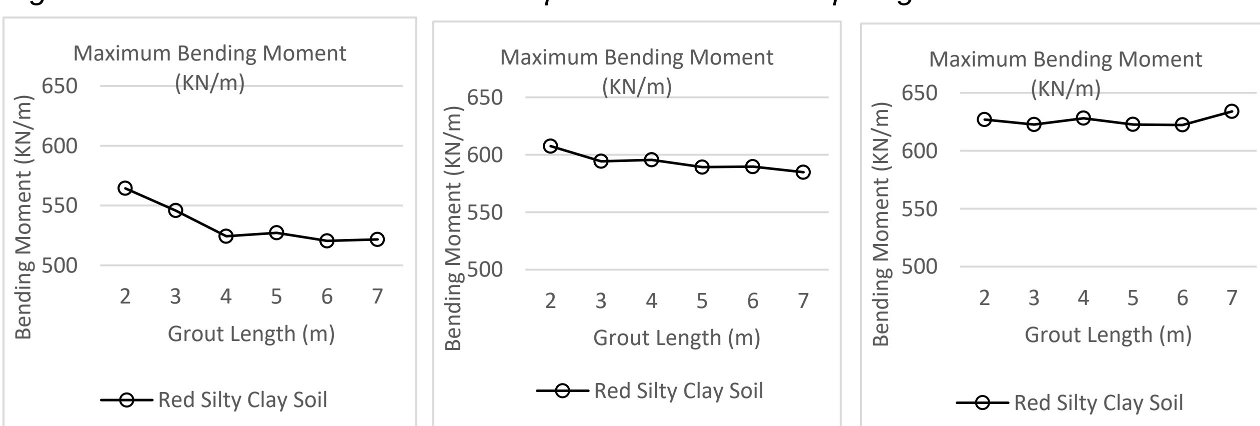
a) Anchor orientation 15⁰ b) Anchor orientation 35⁰ c) Anchor orientation 45⁰

Figure A1 13. Maximum Vertical displacement of the diaphragm wall for GL variation.



a) Anchor orientation 15⁰ b) Anchor orientation 35⁰ c) Anchor orientation 45⁰

Figure A1 14. Maximum Horizontal displacement of the diaphragm wall for GL variation



a) Anchor orientation 15⁰ b) Anchor orientation 35⁰ c) Anchor orientation 45⁰

Figure A1 15. Maximum Bending moment of the diaphragm wall for GL variation.

Appendix 3. Results from the parametric study

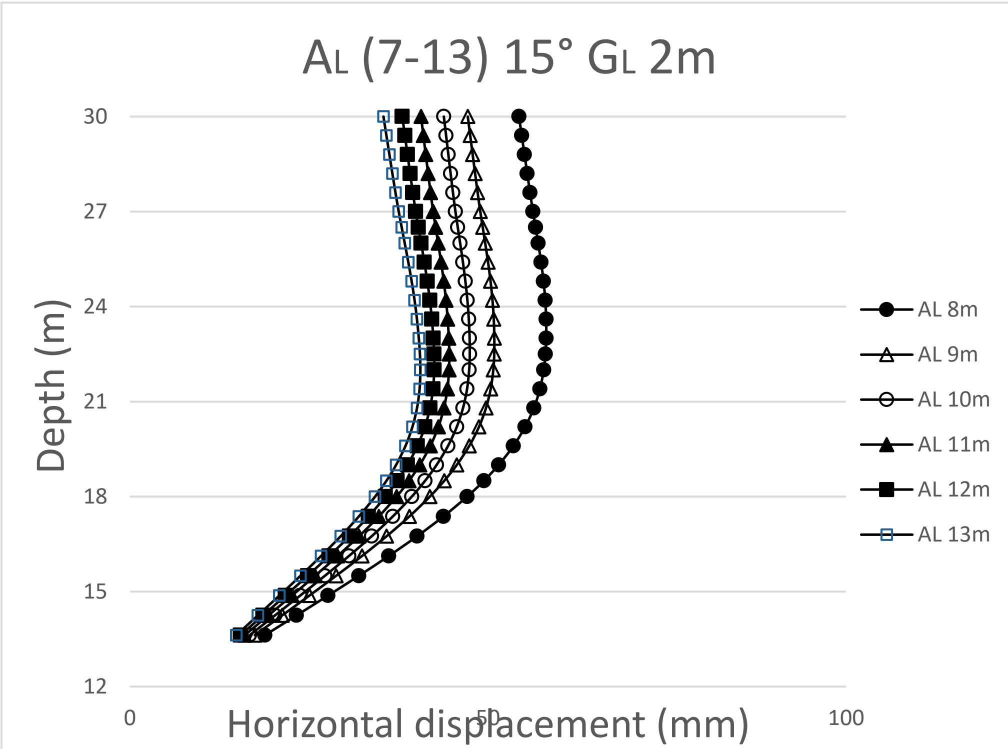


Figure A1 16. Horizontal displacement of the diaphragm wall for GL 2m AA 15º Soil 2.

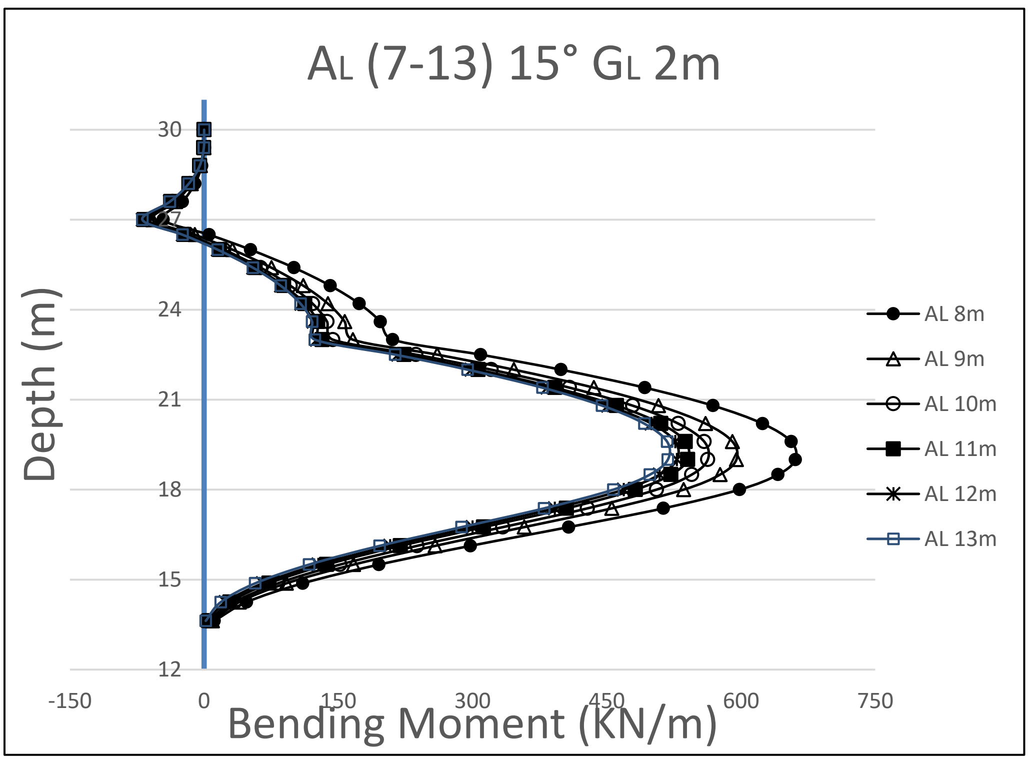


Figure A1 17.Bending moment of the diaphragm wall for GL 2m AA 15º Soil 2.

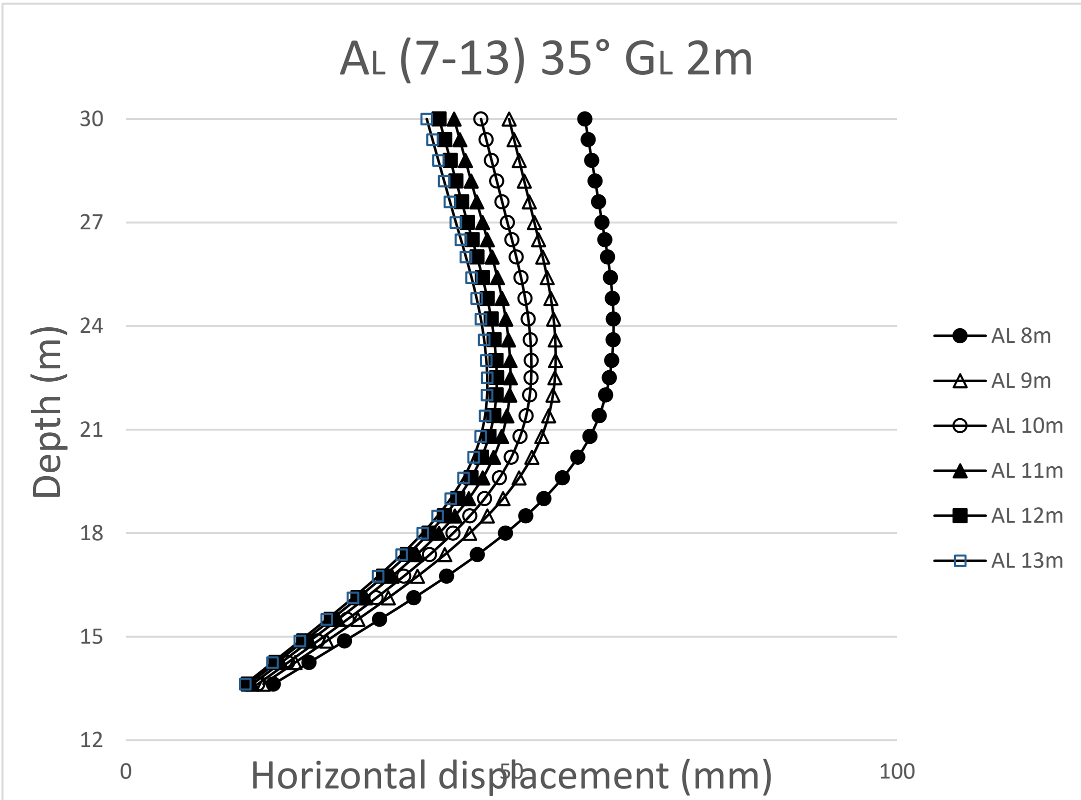


Figure A1 18. Horizontal displacement of the diaphragm wall for GL 2m AA 35⁰ Soil 2.

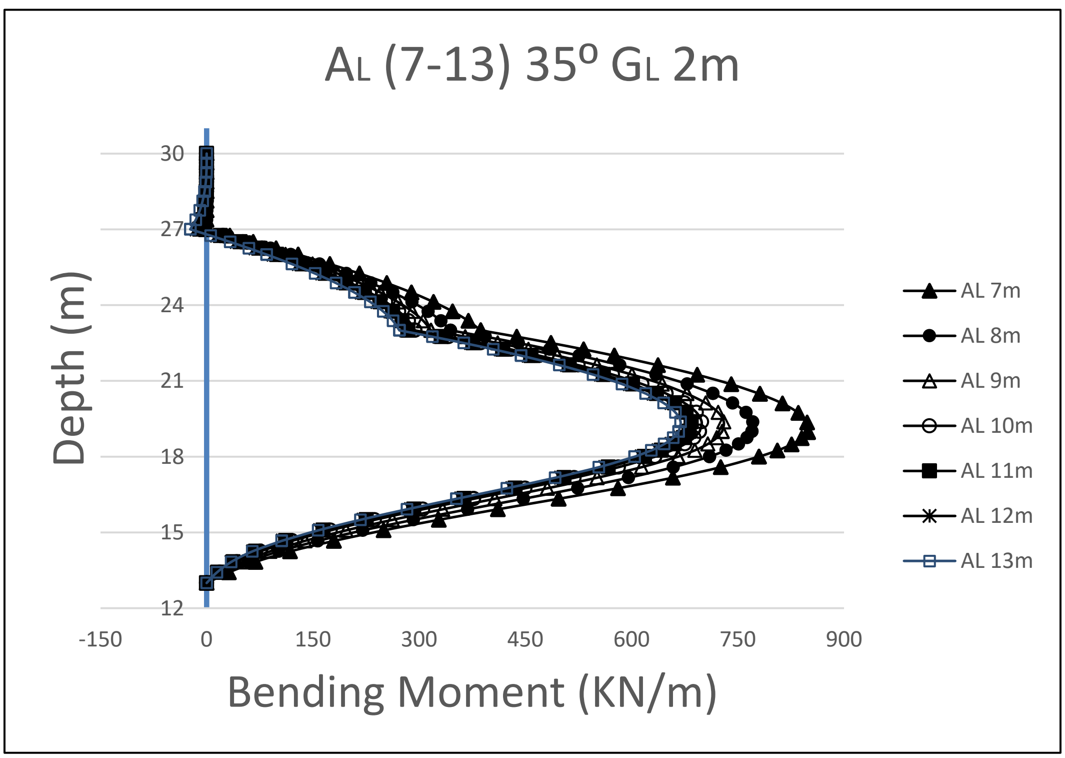


Figure A1 19.Bending moment of the diaphragm wall for GL 2m AA 35⁰ Soil 2.

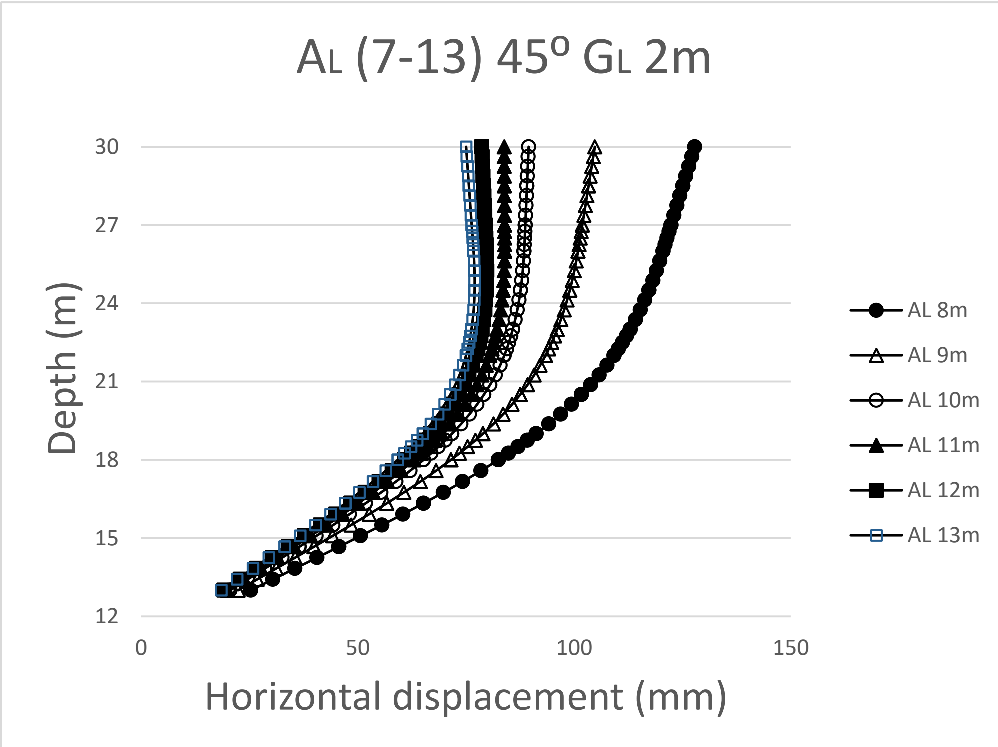


Figure A1 20. Horizontal displacement of the diaphragm wall for GL 2m AA 45º Soil 2.

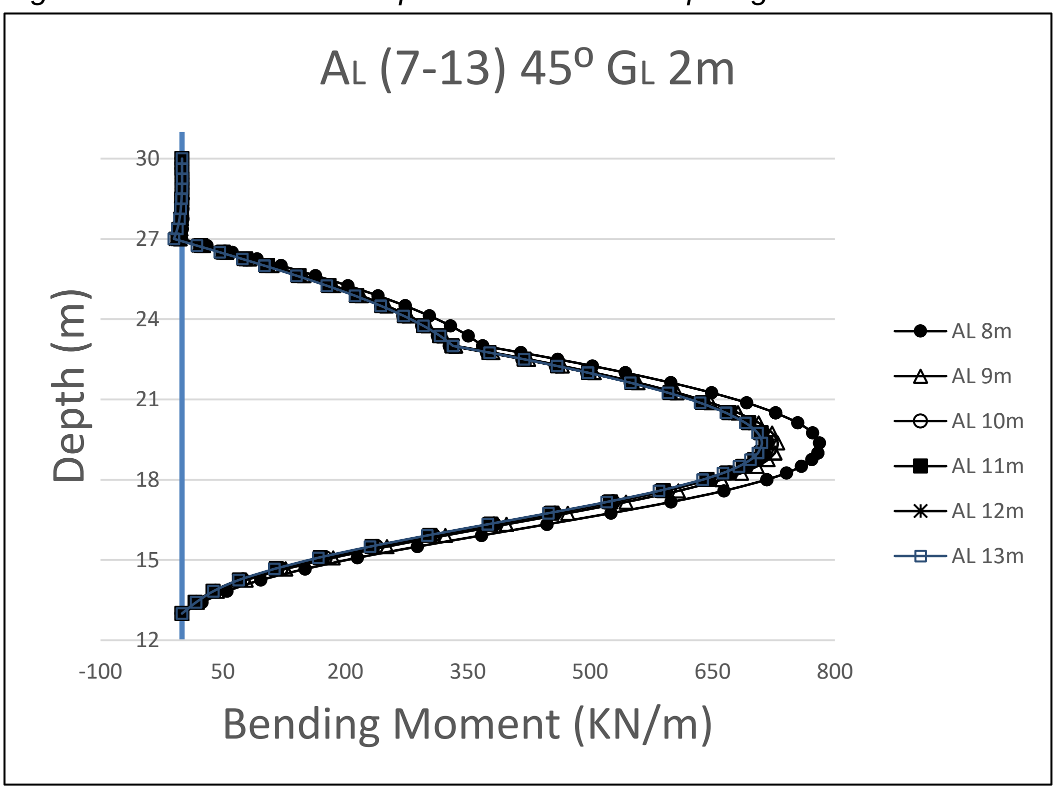


Figure A1 21. Bending moment of the diaphragm wall for GL 2m AA 45º Soil 2.

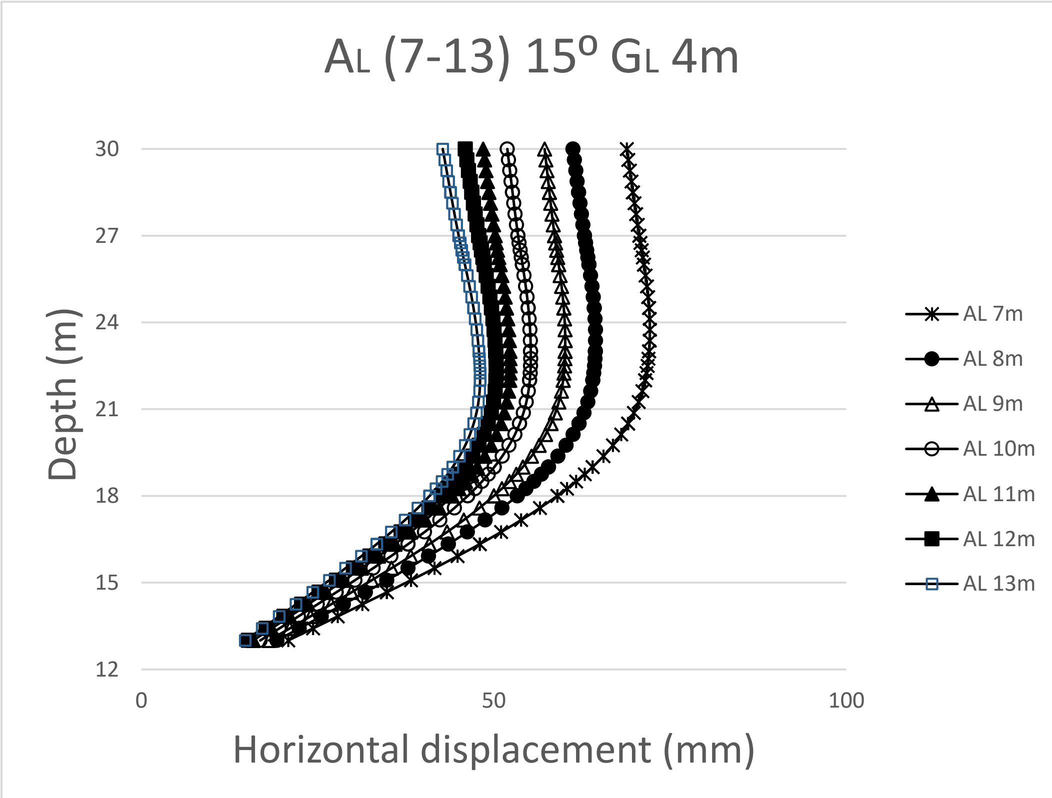


Figure A1 22. Horizontal displacement of the diaphragm wall for GL 4m AA 15º Soil 2.

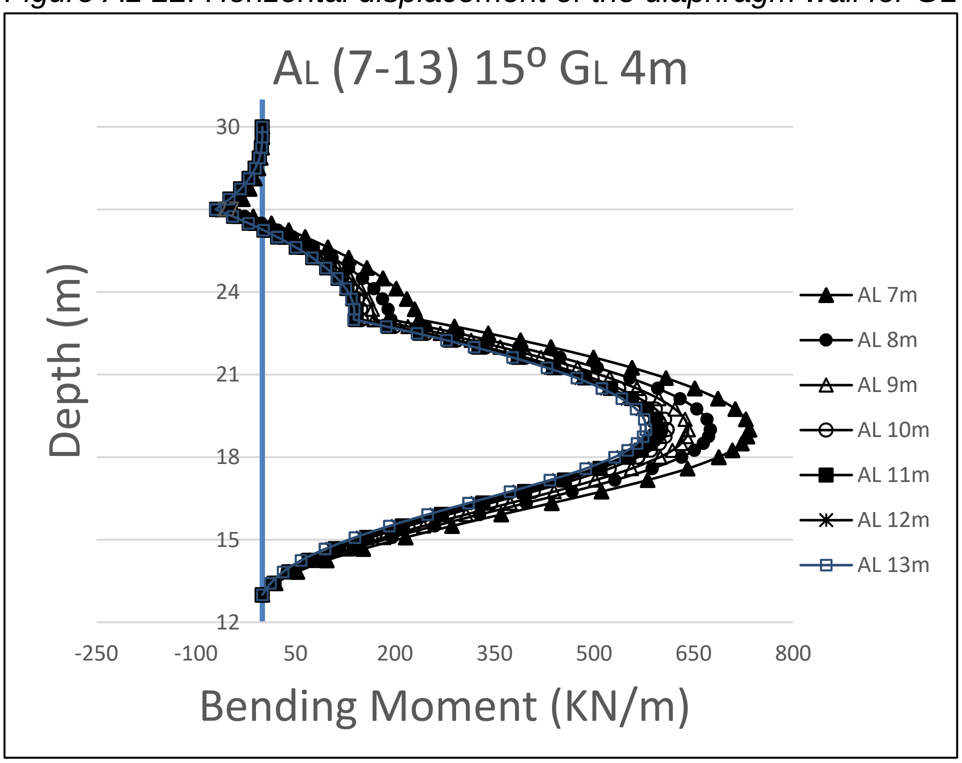


Figure A1 23. Bending moment of the diaphragm wall for GL 4m AA 15º Soil 2.

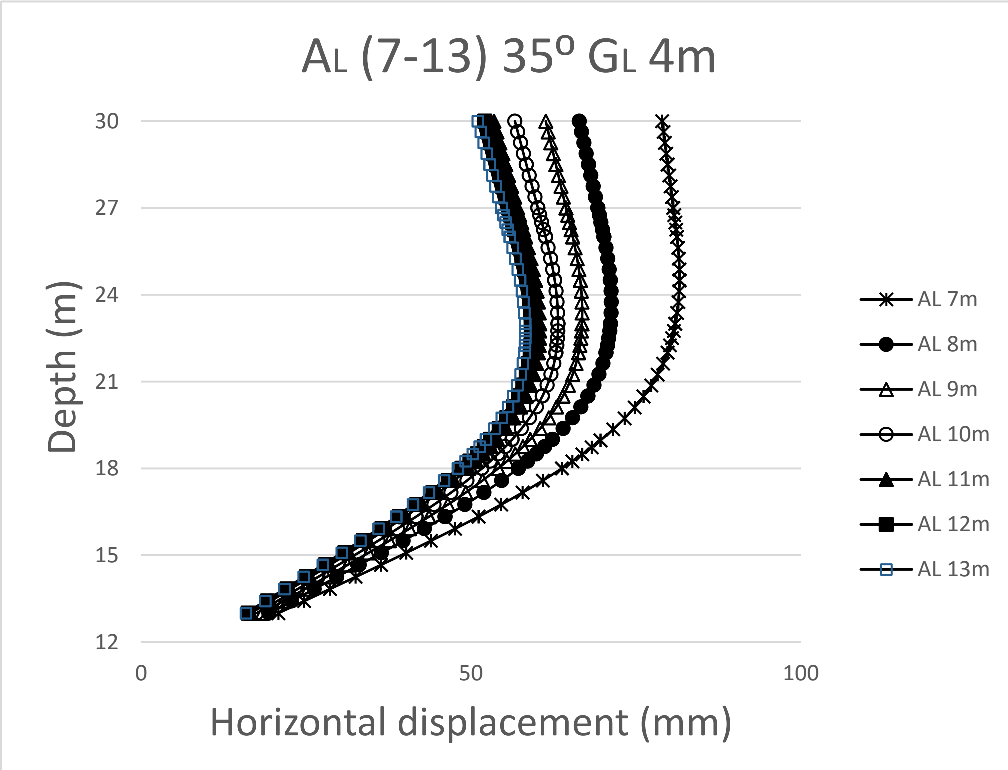


Figure A1 24. Horizontal displacement of the diaphragm wall for GL 4m AA 35º Soil 2.

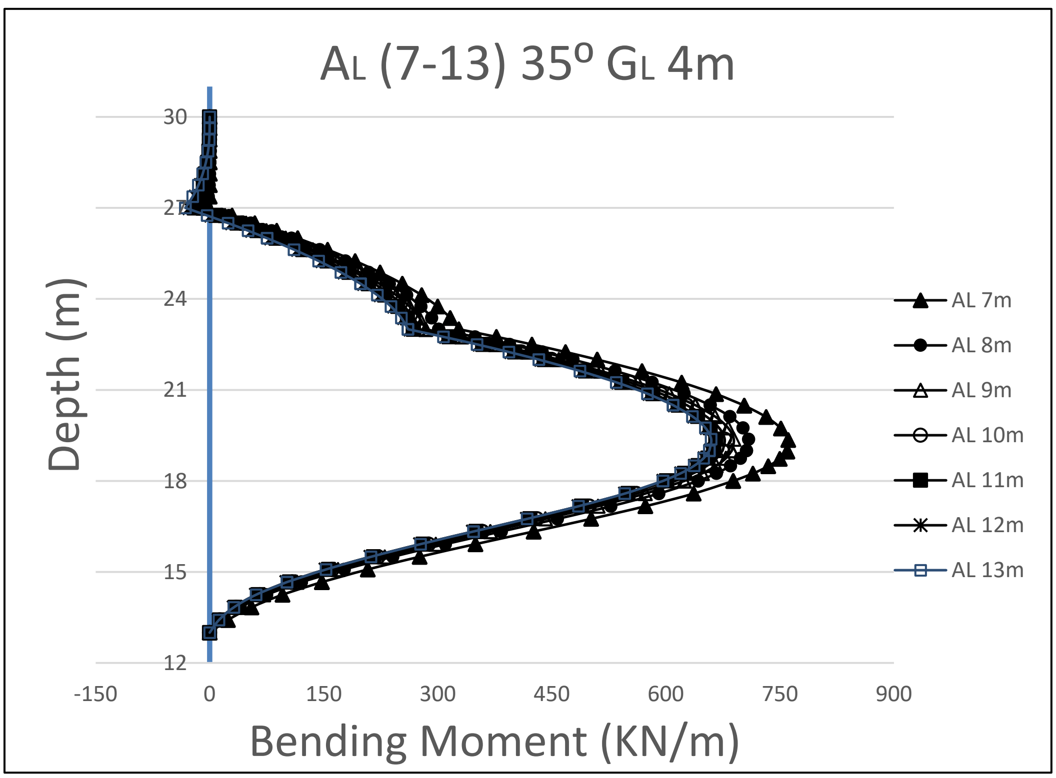


Figure A1 25. Bending moment of the diaphragm wall for GL 4m AA 35º Soil 2.

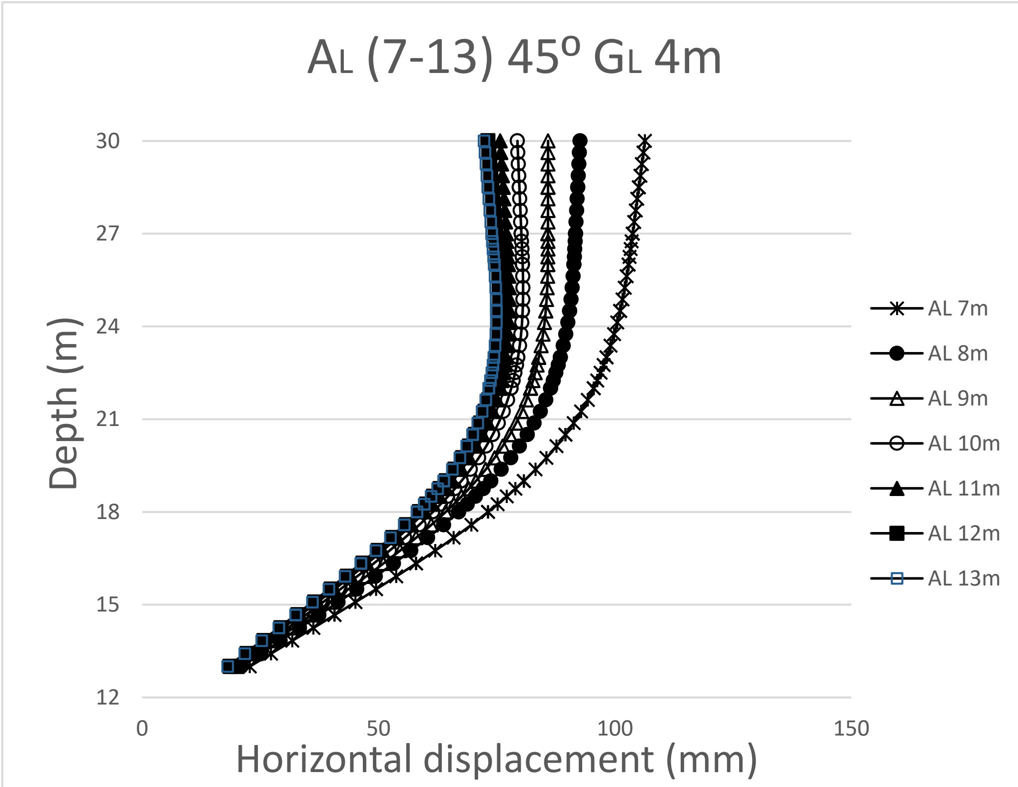


Figure A1 26. Horizontal displacement of the diaphragm wall for GL 4m AA 45º Soil 2.

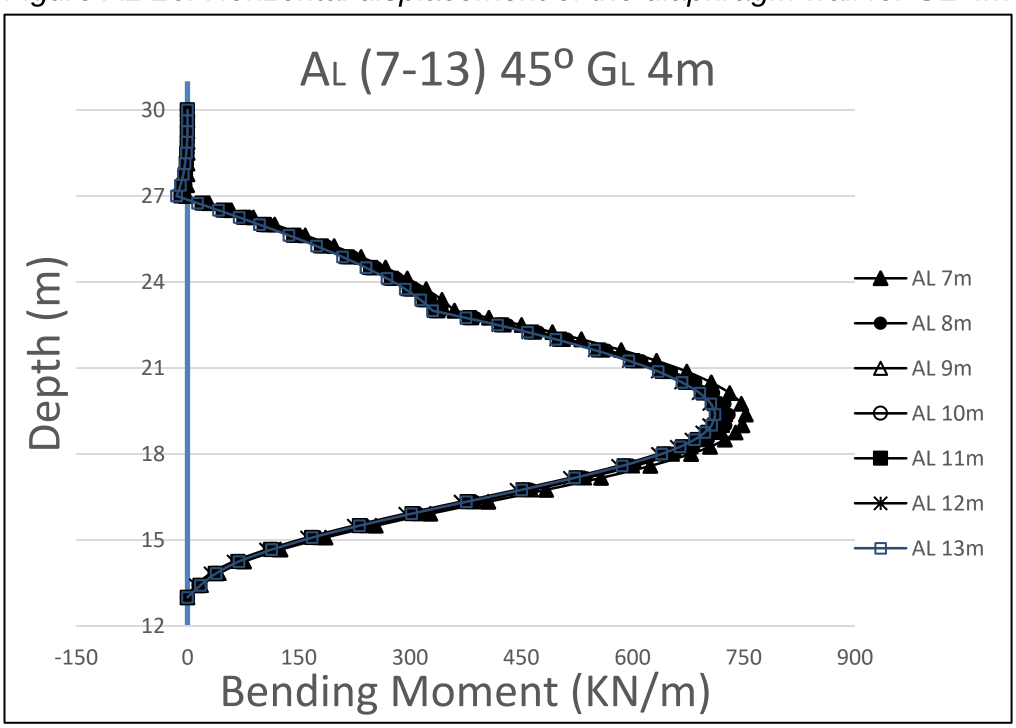


Figure A1 27. Bending moment of the diaphragm wall for GL 4m AA 45º Soil 2.

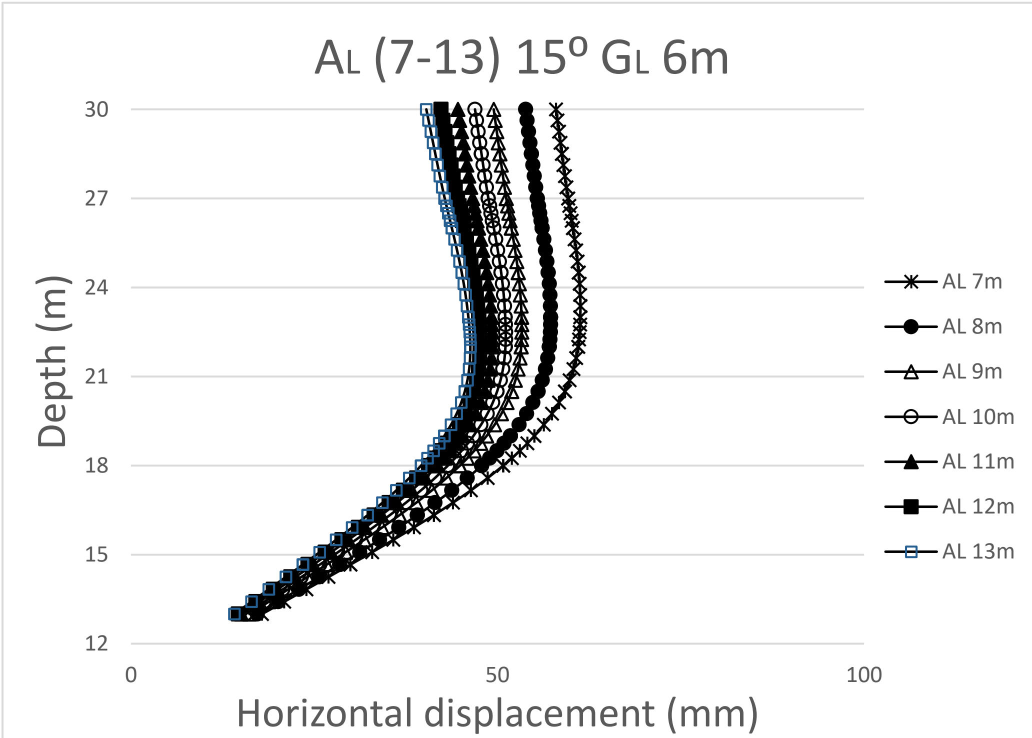


Figure A1 28. Horizontal displacement of the diaphragm wall for GL 6m AA 15º Soil 2.

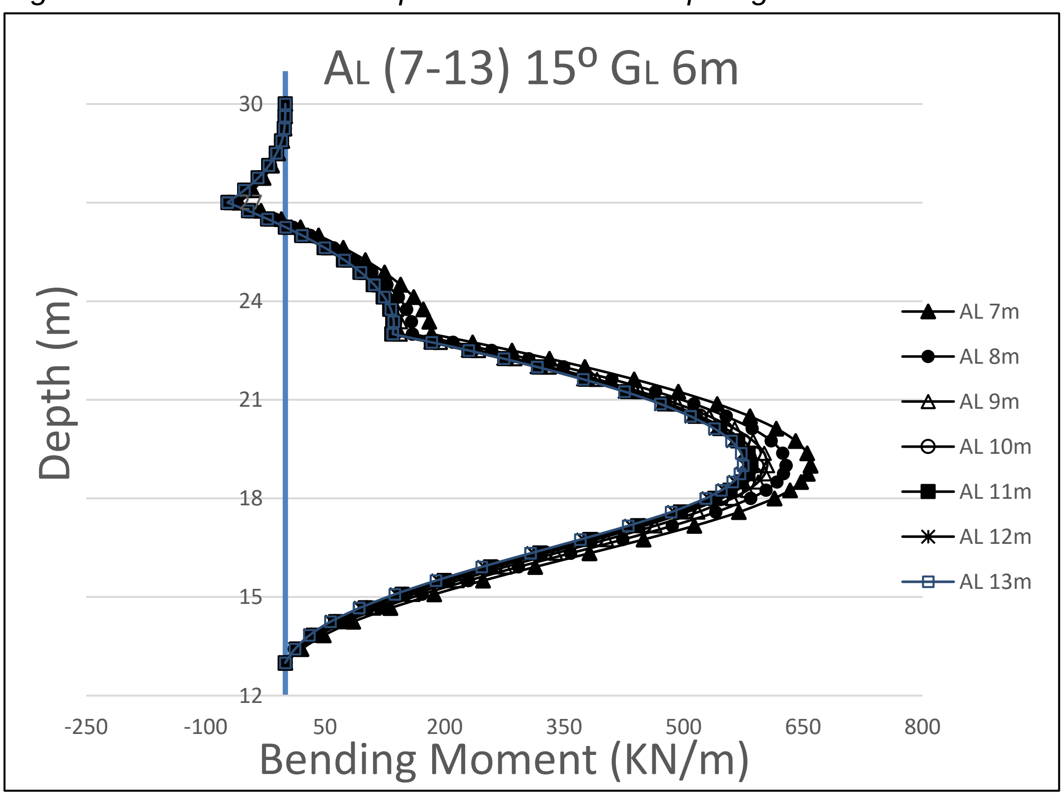


Figure A1 29. Bending moment of the diaphragm wall for GL 6m AA 15º Soil 2.

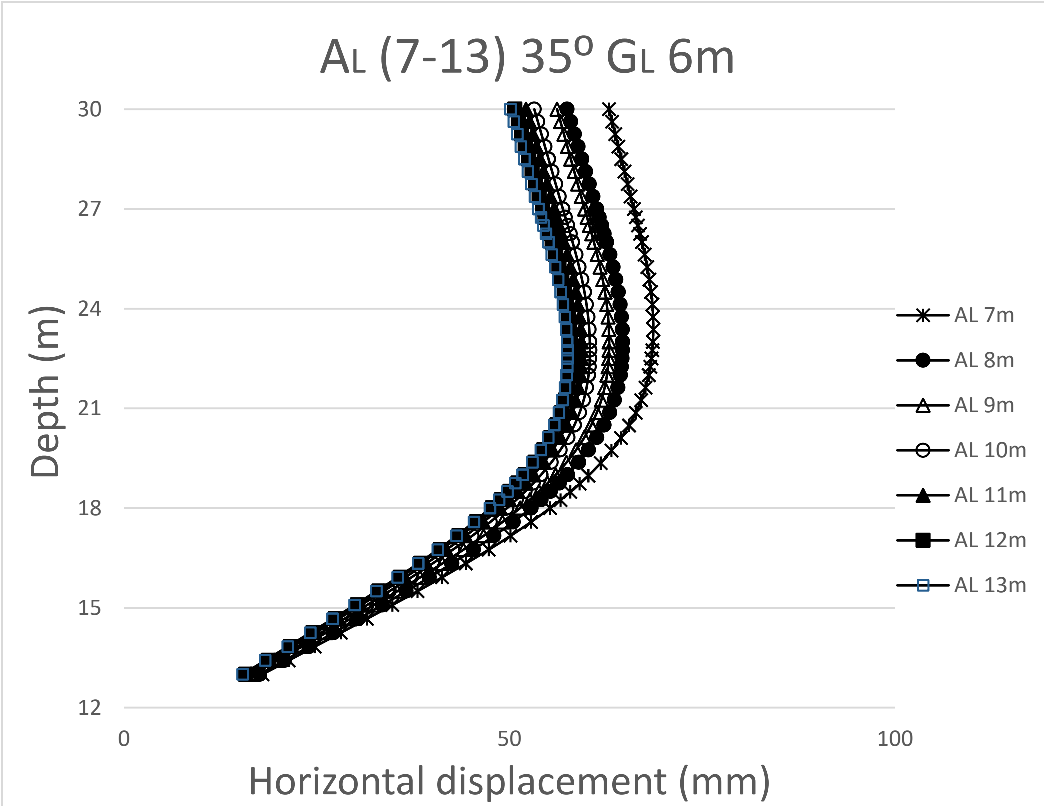


Figure A1 30. Horizontal displacement of the diaphragm wall for GL 6m AA 35º Soil 2.

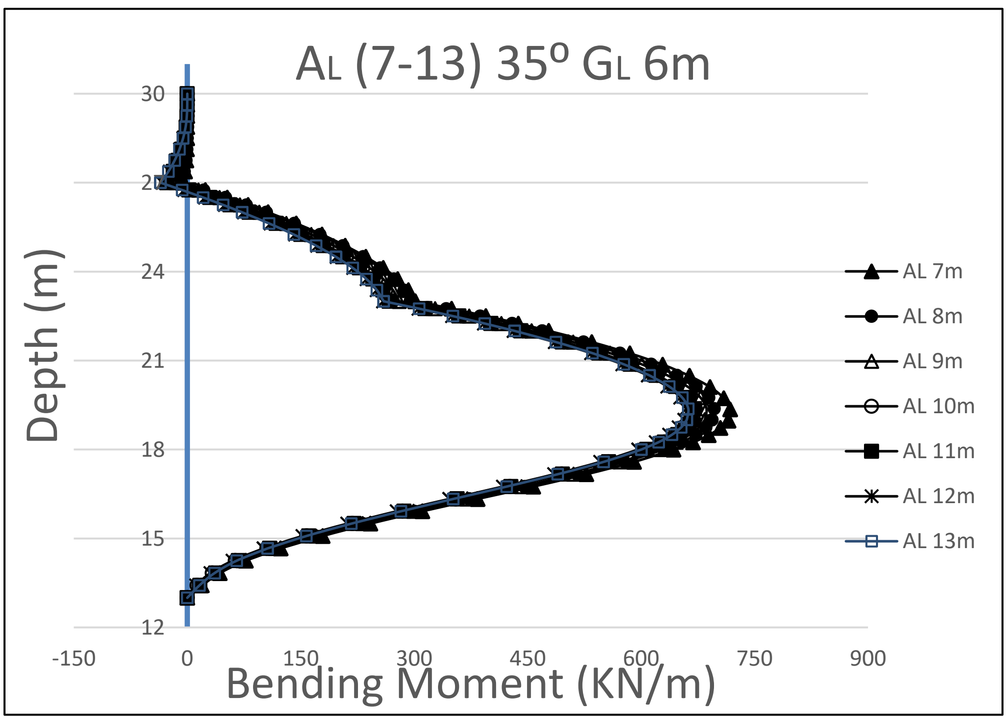


Figure A1 31. Bending moment of the diaphragm wall for GL 6m AA 35º Soil 2.

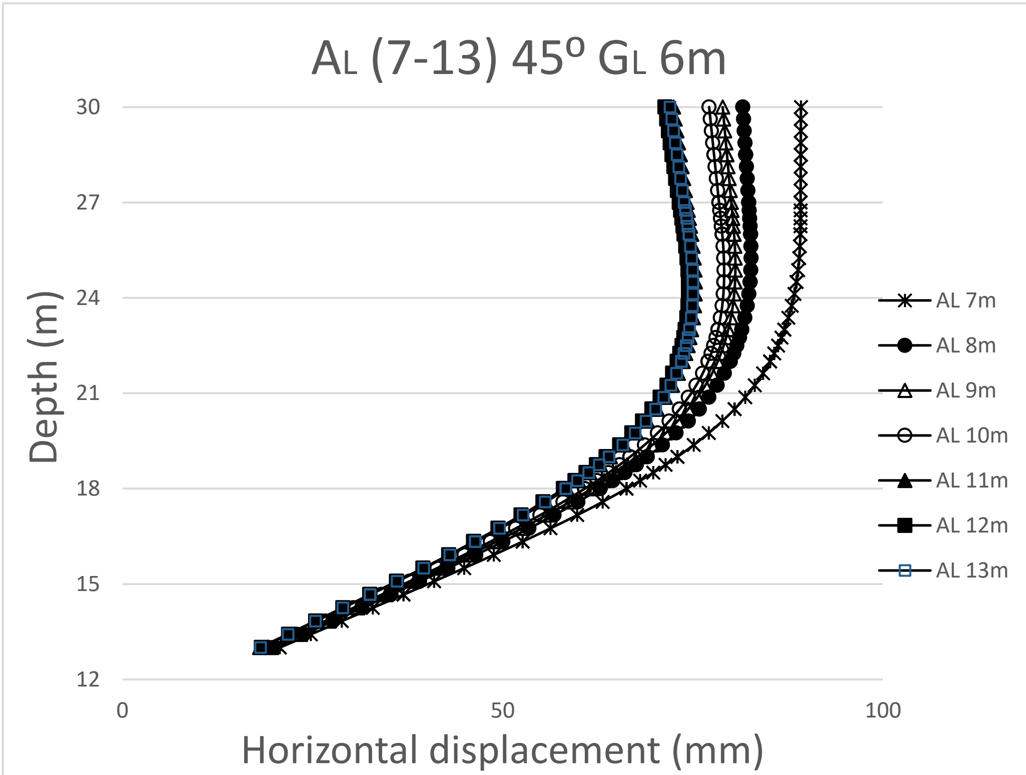


Figure A1 32. Horizontal displacement of the diaphragm wall for GL 6m AA 45 ⁰ Soil 2.

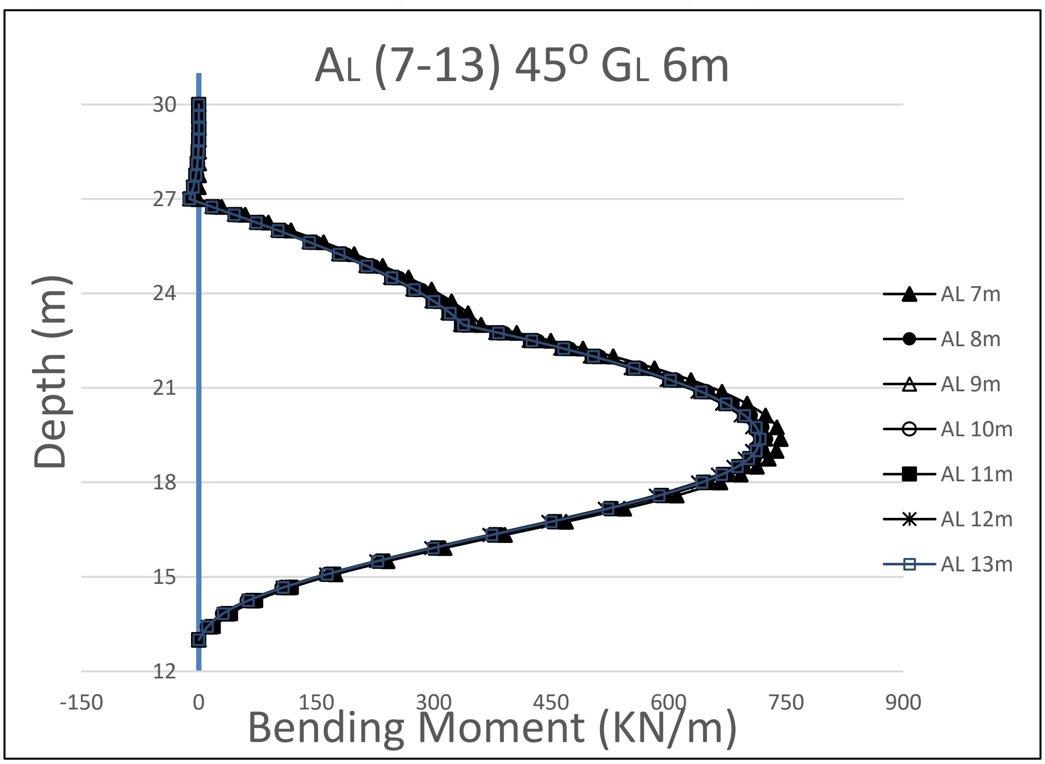


Figure A1 33. Bending moment of the diaphragm wall for GL 6m AA 45⁰ Soil 2.

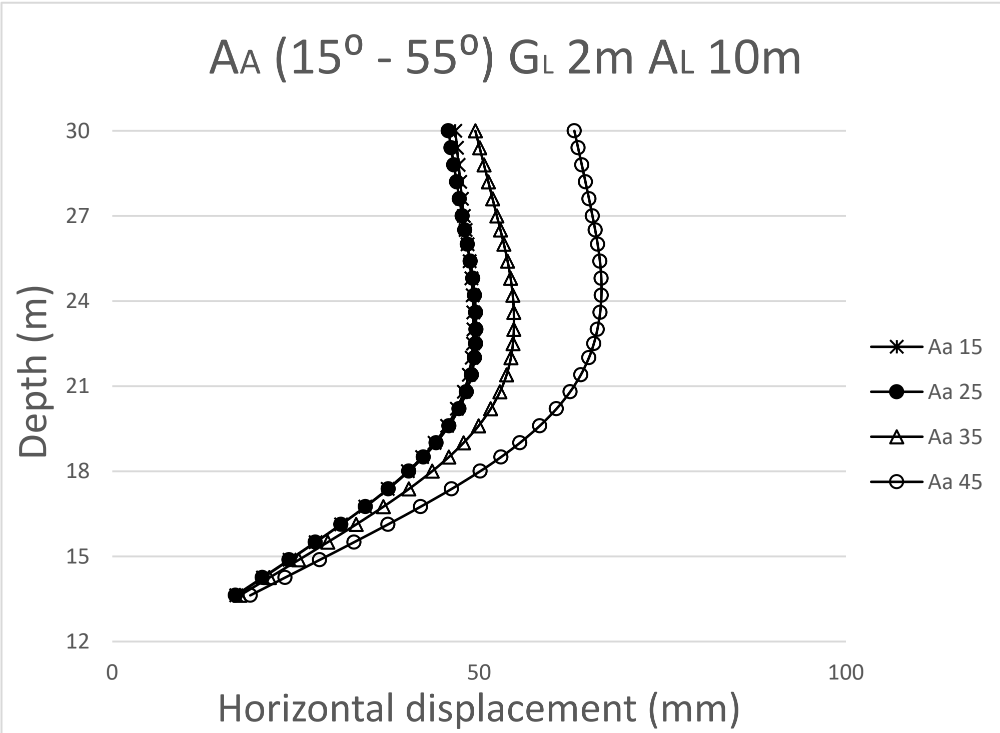


Figure A1 34. Horizontal displacement diagram for AA (15 ⁰ - 55 ⁰) GL 2m AL 10m Soil 1.

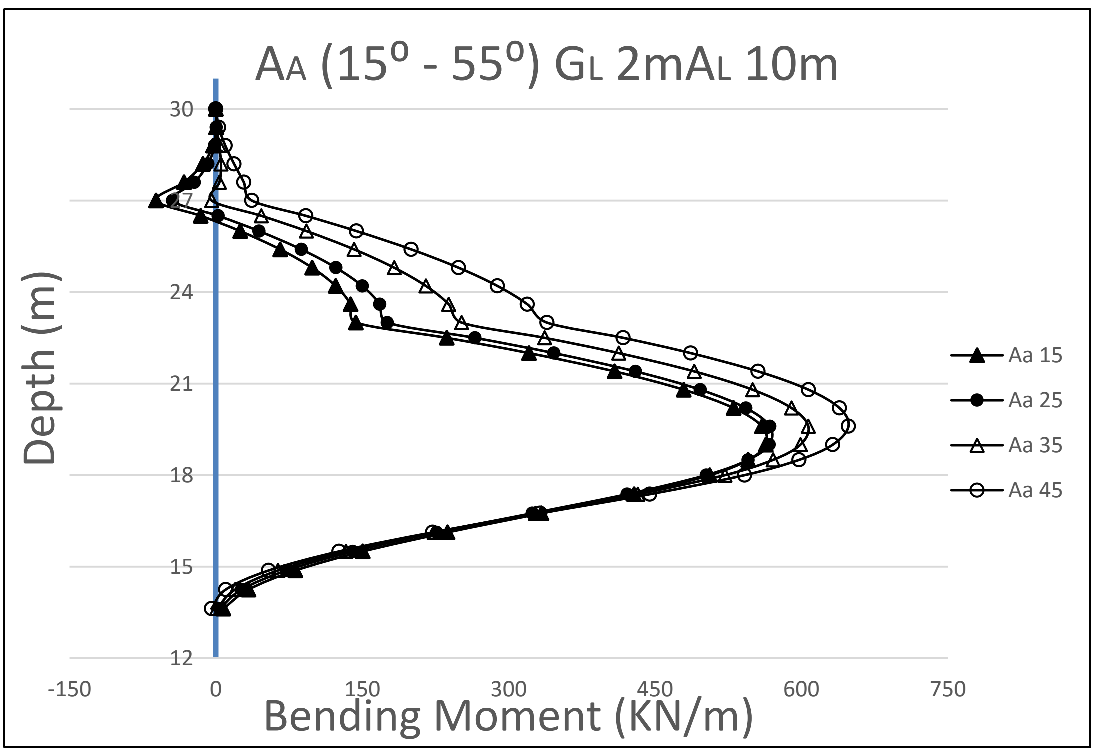


Figure A1 35. Maximum Bending moment for AA (15 ⁰ - 55 ⁰) GL 2m AL 10m Soil 1.

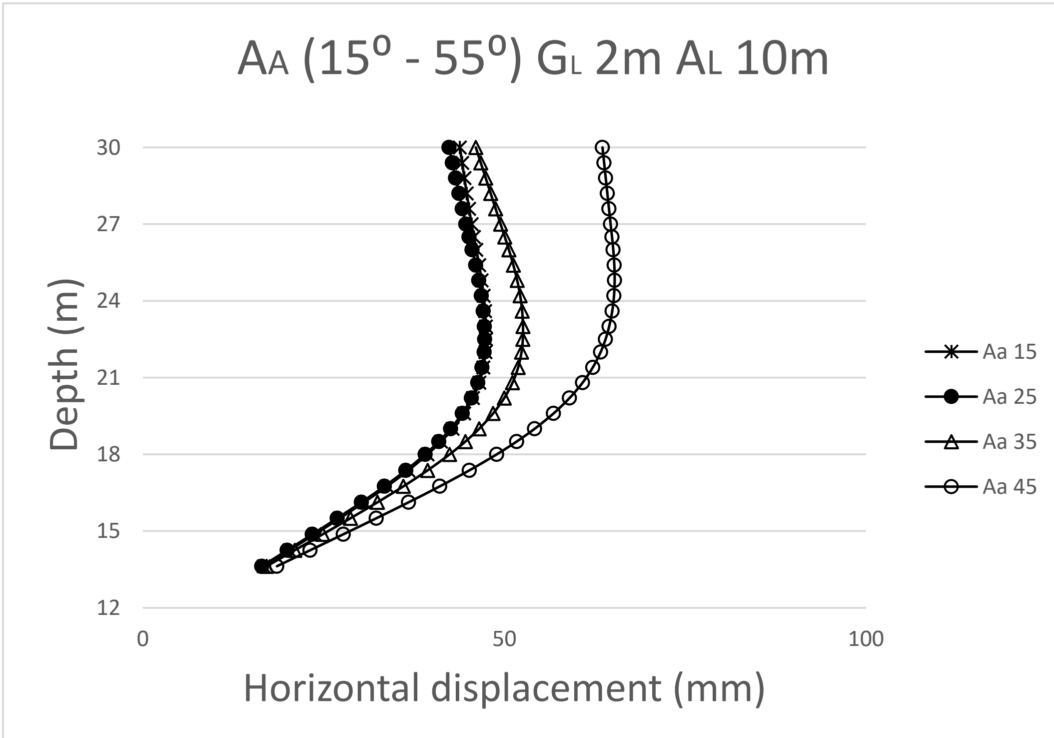


Figure A1 36. Horizontal displacement diagram for AA (15 º - 55 º) GL 2m AL 10m Soil 2.

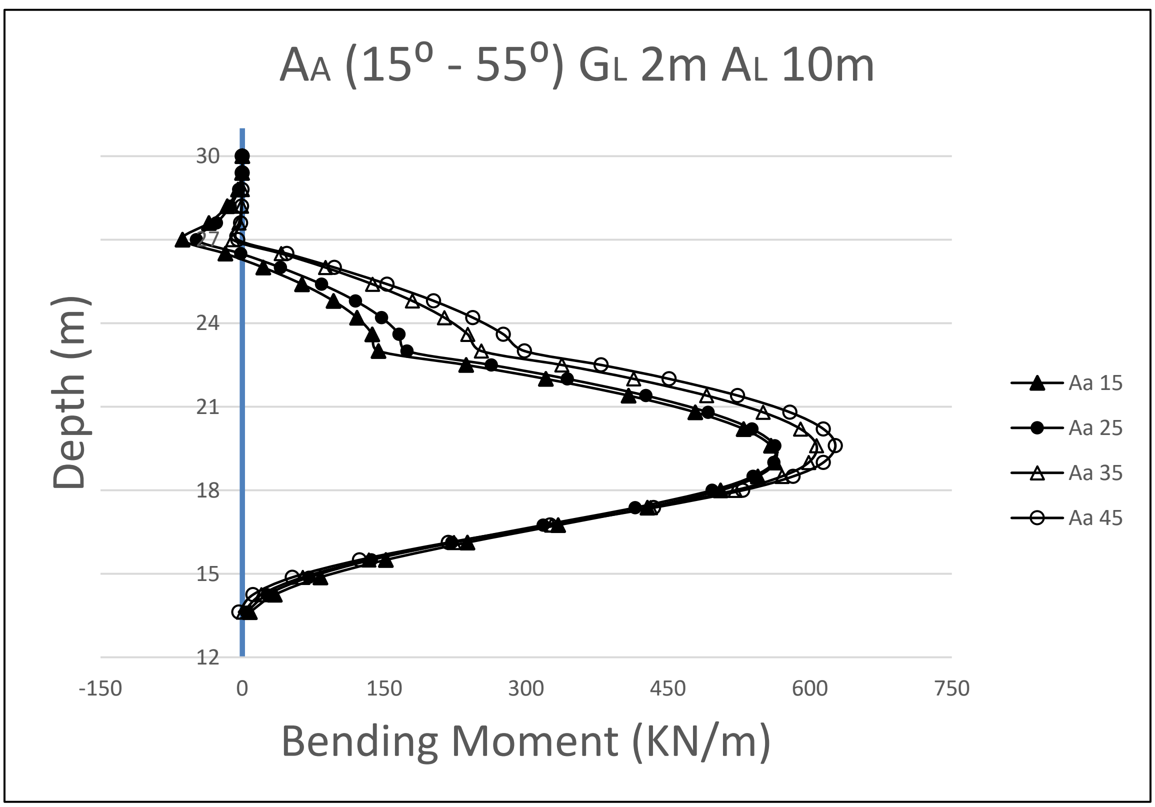


Figure A1 37. Maximum Bending moment for AA (15 º - 55 º) GL 2m AL 10m Soil 2.

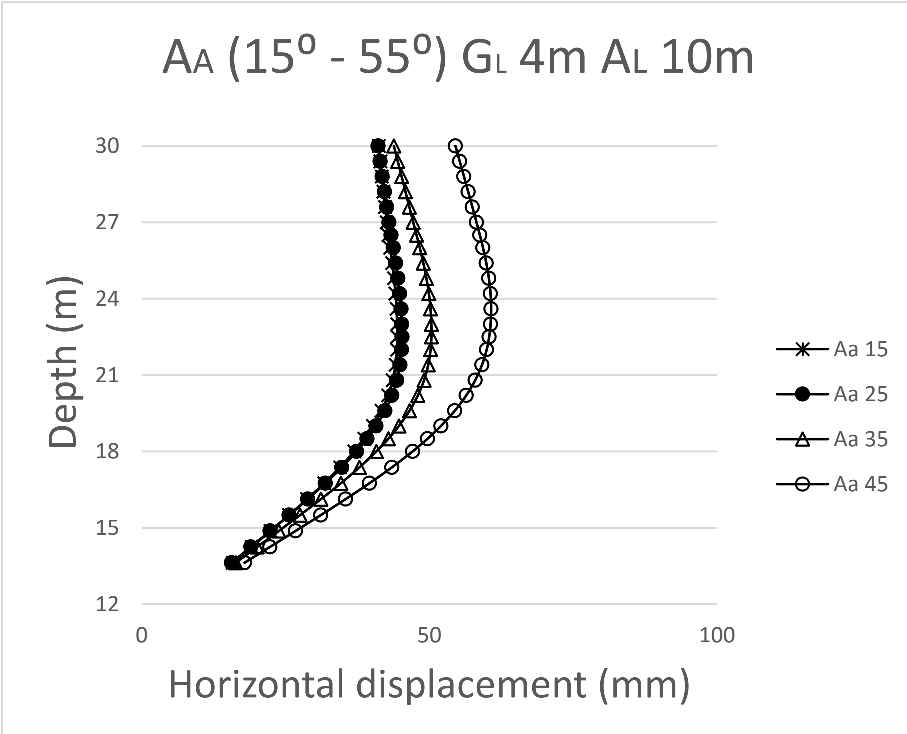


Figure A1 38. Horizontal displacement diagram for AA (15⁰ - 55⁰) GL 4m AL 10m Soil 1.

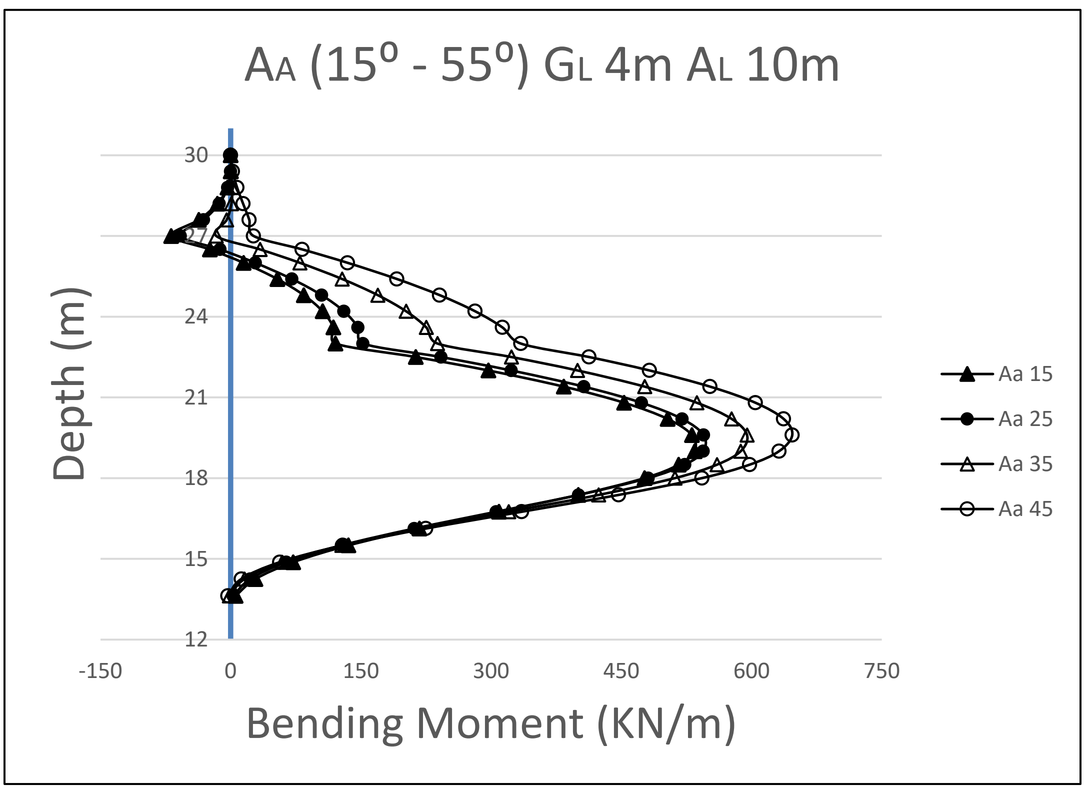


Figure A1 39. Maximum Bending moment for AA (15⁰ - 55⁰) GL 4m AL 10m Soil 1.

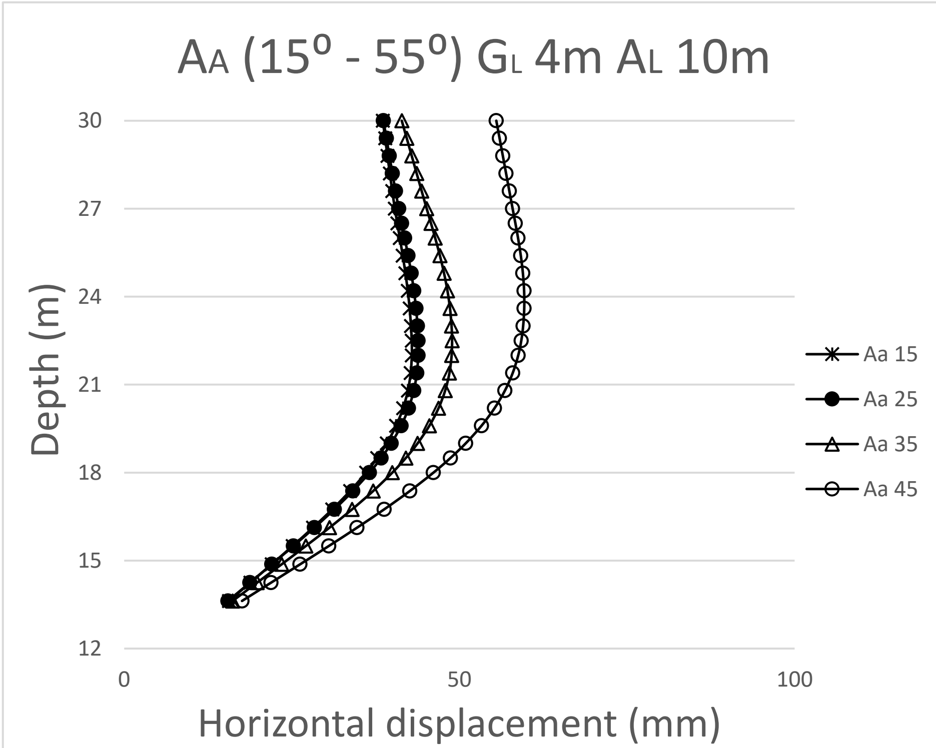


Figure A1 40. Horizontal displacement diagram for AA (15 ⁰ - 55 ⁰) GL 4m AL 10m Soil 2.

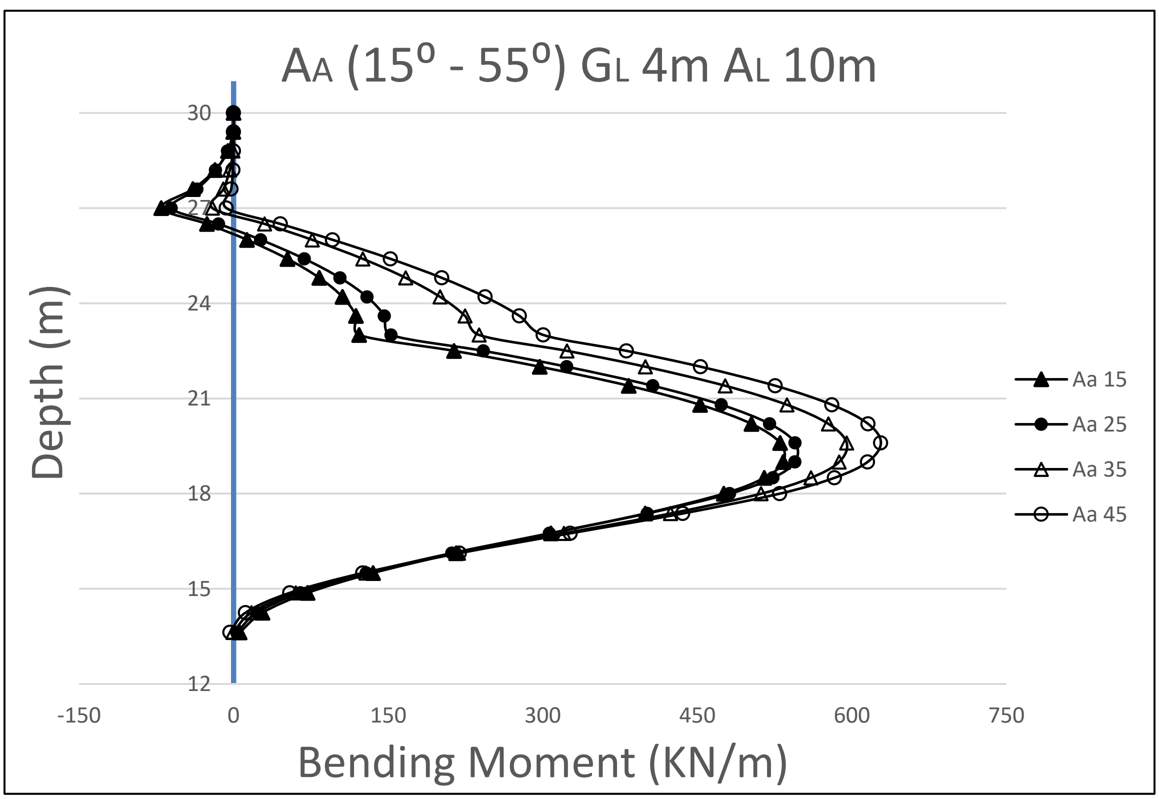


Figure A1 41. Maximum Bending moment for AA (15 ⁰ - 55 ⁰) GL 4m AL 10m Soil 2.

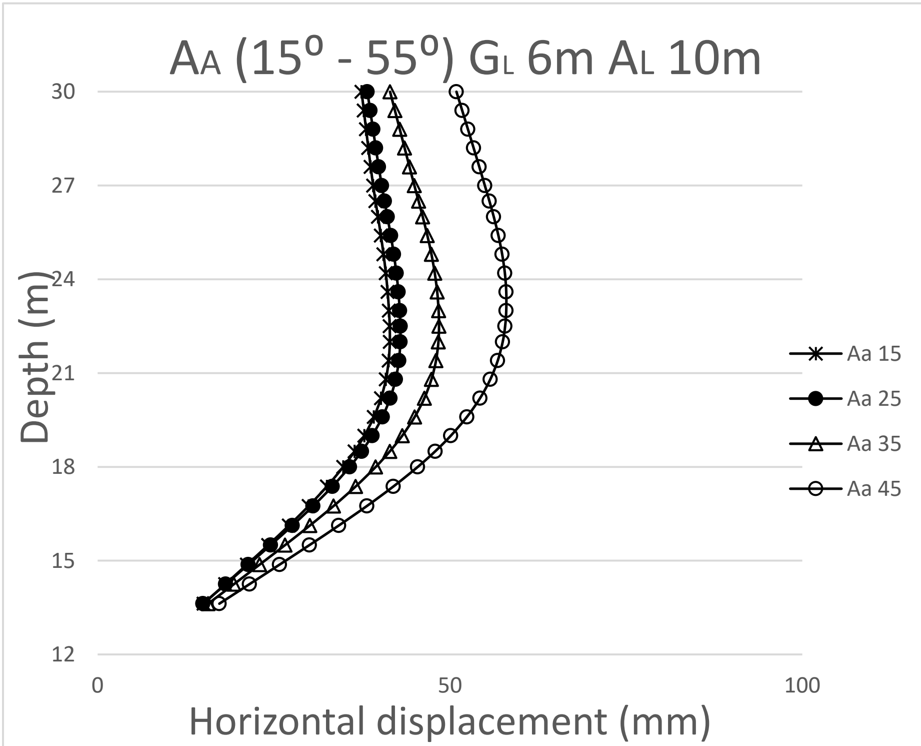


Figure A1 42. Horizontal displacement diagram for AA (15 ^{0} - 55 ^{0}) GL 6m AL 10m Soil 1.

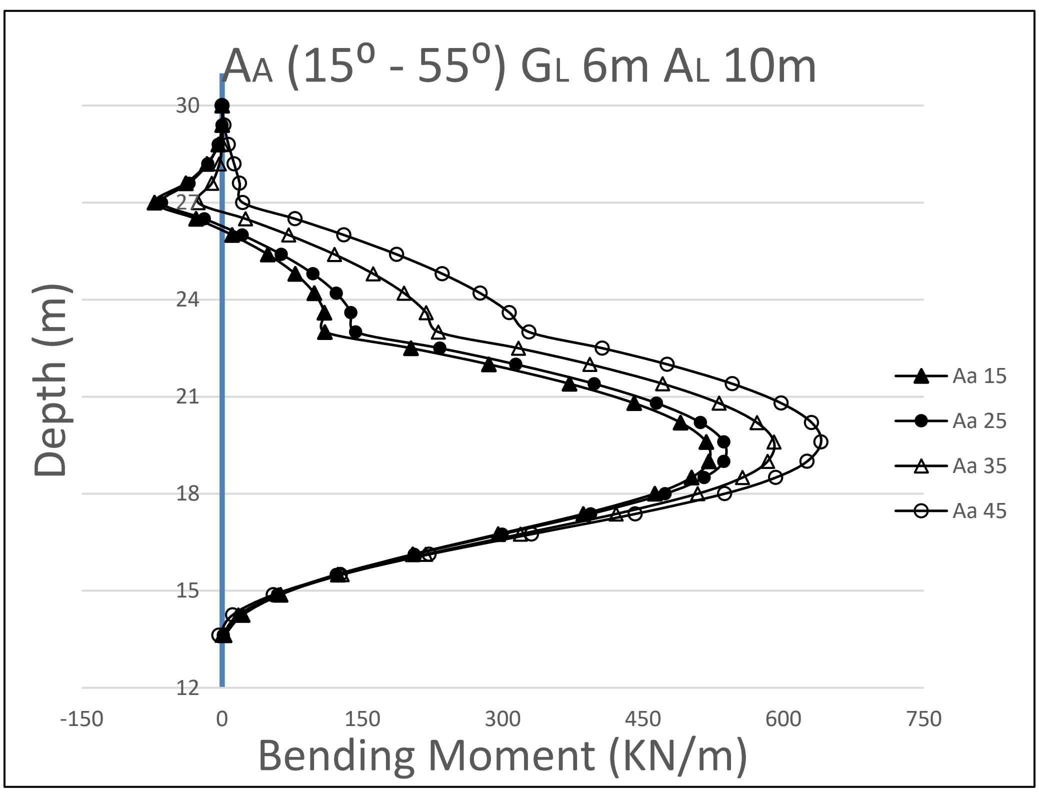


Figure A1 43. Maximum Bending moment for AA (15 ^{0} - 55 ^{0}) GL 6m AL 10m Soil 1.

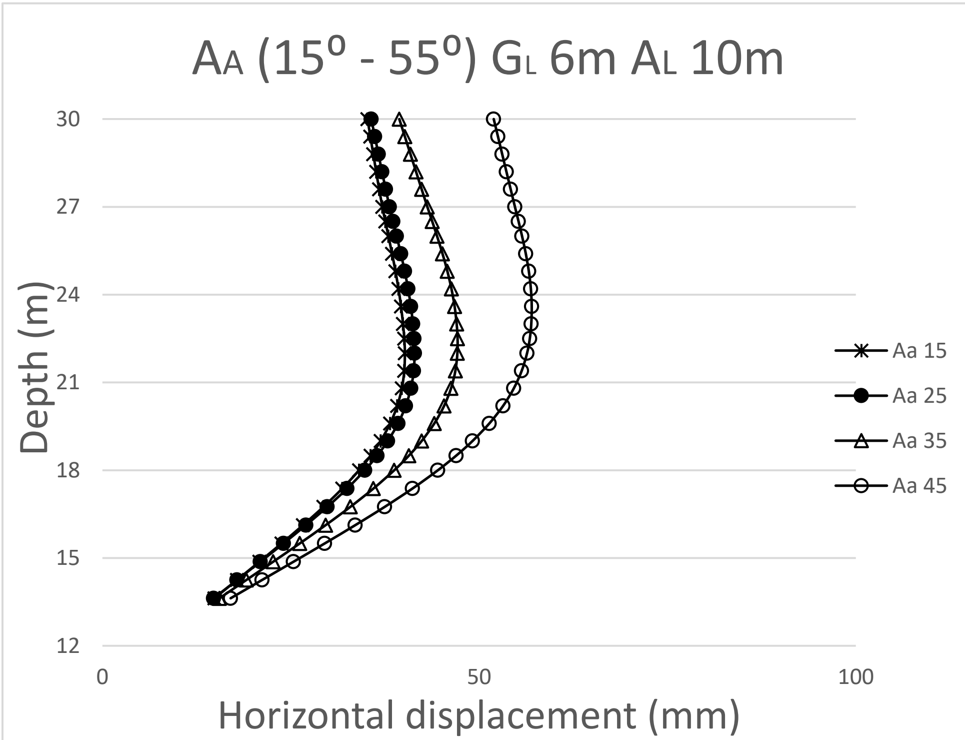


Figure A1 44. Horizontal displacement diagram for AA (15 ^{0} - 55 ^{0}) GL 6m AL 10m Soil 2.

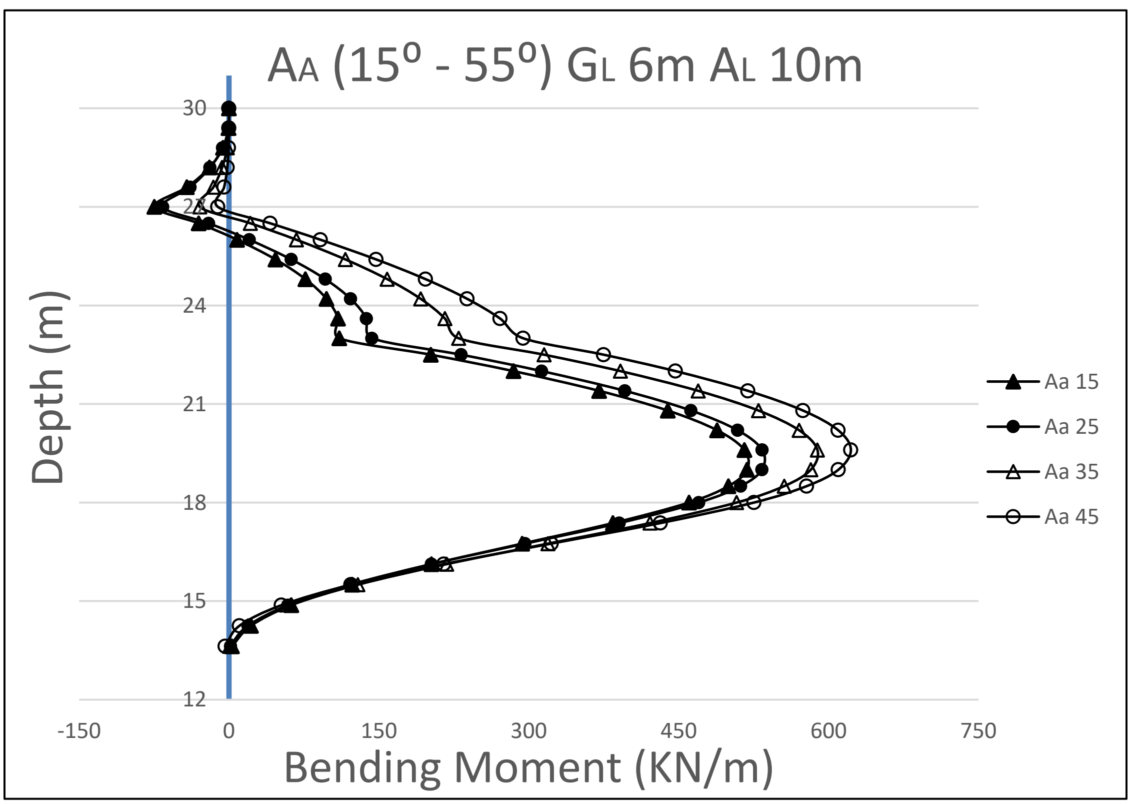
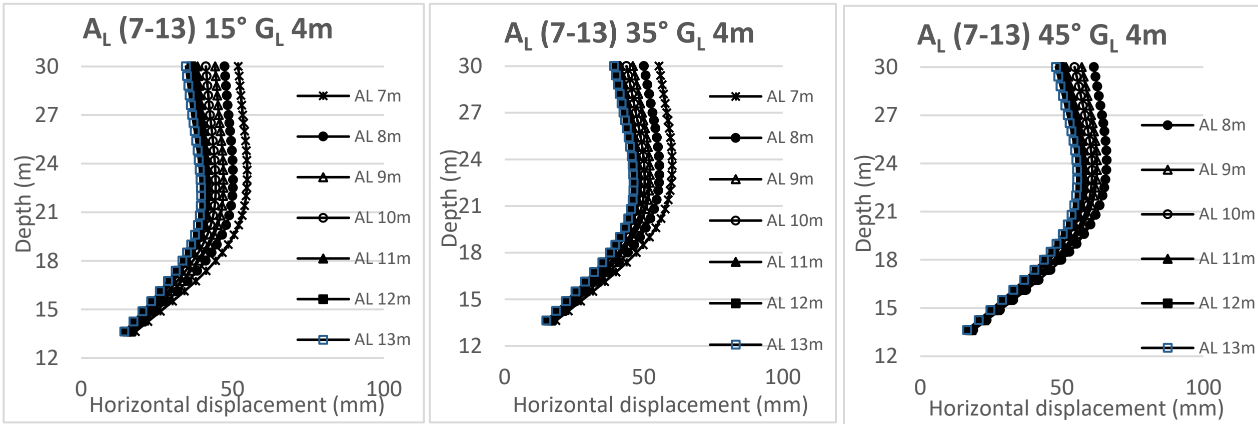
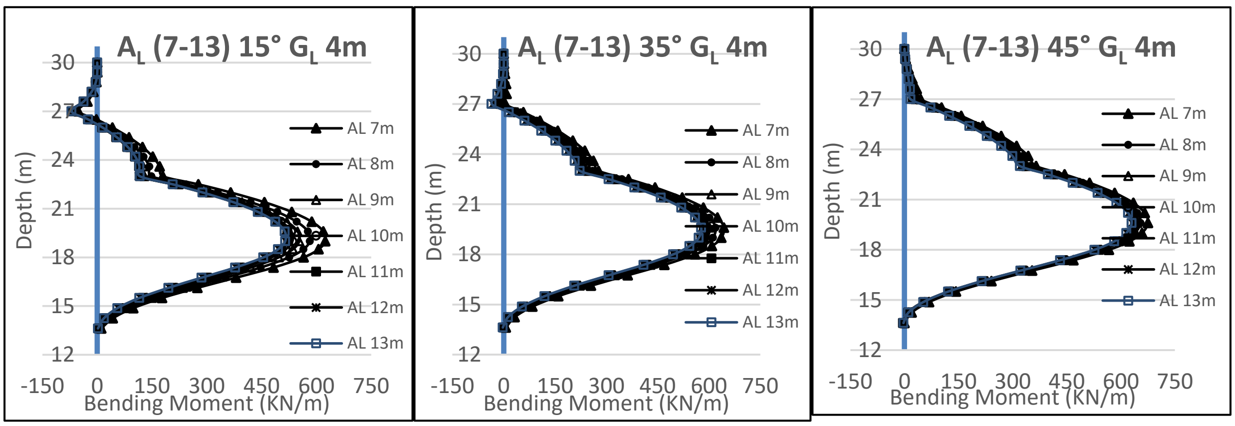


Figure A1 45. Maximum Bending moment for AA (15 ^{0} - 55 ^{0}) GL 6m AL 10m Soil 2.



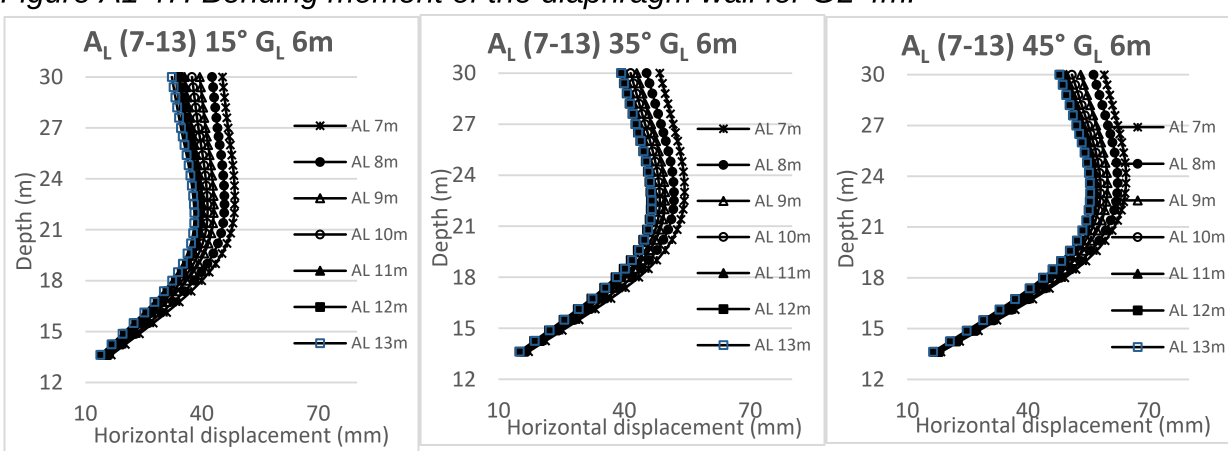
a) Expansive soil A$_A$ 15º **b) Expansive soil A$_A$ 35º** **c) Expansive soil A$_A$ 45º**

Figure A1 46. Horizontal displacement of the diaphragm wall for GL 4m.



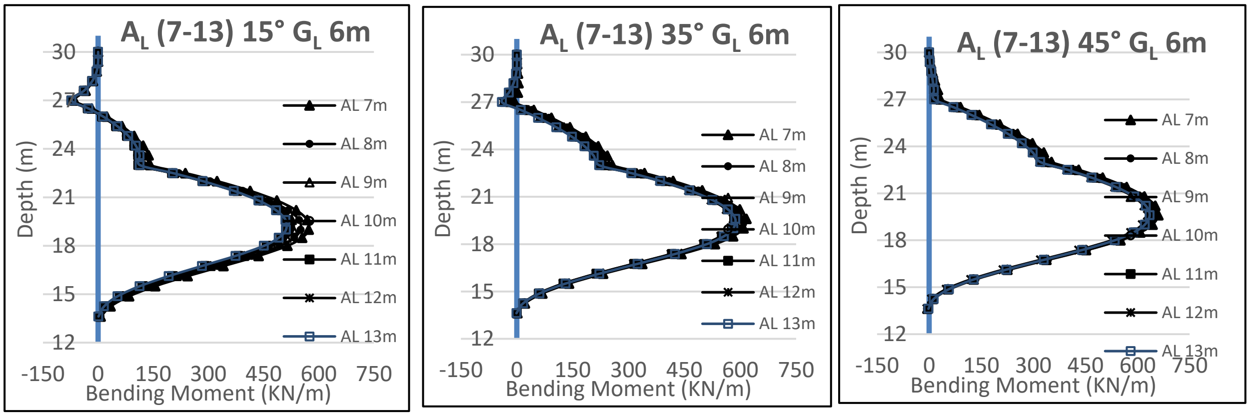
a)Expansive soil A$_A$ 15º **b)Expansive soil A$_A$ 35º** **c) Expansive soil A$_A$ 45º**

Figure A1 47. Bending moment of the diaphragm wall for GL 4m.



a) Expansive soil A$_A$ 15º **b) Expansive soil A$_A$ 35º** **c) Expansive soil A$_A$ 45º**

Figure A1 48. Horizontal displacement of the diaphragm wall for GL 6m.



a)Expansive soil A_A 15º **b) Expansive soil A_A 35º** **c) Expansive soil A_A 45º**

Figure A1 49. Bending moment of the diaphragm wall for GL 6m.