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**GRADUATE STUDIES**  
**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**

**Comparison of Retaining Structures Based on Their Performance**  
**(Case of Addis Ababa)**

By

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A Thesis Submitted to School of Graduate Studies  
For the Partial Fulfillment of the Requirement for Degree of Master of Science  
In  
Geotechnical Engineering

Advisor

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May, 2018

Addis Ababa, Ethiopia

## **Acknowledgments**

I am grateful to my advisor Dr.-Ing Henok Fikre, School of Civil Engineering, Addis Ababa Institute of Technology for his close supervision and constructive suggestions during my research work. I would also want to forward my special thanks to Dr.-Ing Samuel Tadesse and Ato Isayas Getachew who gave me valuable comments on this thesis and provided me with data, and they were very cooperative and welcoming.

I would like to express my deepest gratitude to my family and my colleague Yonathan Alemu for their unlimited support and fulfillment of all my interests.

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## **Abstract**

Deep basement constructions are always anticipated in densely populated and urban areas with limited construction spaces like Addis Ababa. The design of basement retaining walls and their supporting system involves careful analysis, design and monitoring due to the sensitivity of ground movements that can cause damage to the adjacent buildings, roads and other utilities. Therefore, analyses and parametric study are important to better understand the response of the soil to excavation and to predict the magnitude and pattern of ground movement.

In Addis Ababa the most common type of retaining wall for deep excavation is contiguous bored pile wall. This retaining method may not be relatively effective in terms of various parameters like soil type of the area, embedment depth, size of the wall, and construction cost. Thus other alternative retaining systems should be considered and analyzed to select a best option. There are papers written to analyze the performance of different types of retaining structures. The results presented in those papers are for a specific type of retaining structure and lack cost estimation. These two factors limit the flexibility to compare results in an organized manner and have reasonable up-to-date information about the construction cost. Therefore, it is necessary to compare soil retaining structures used for deep excavations by including additional comparison factor, which is cost of construction.

The objective of this thesis is to identify the effect and performance of diaphragm, contiguous and tangent bored pile retaining walls for basement work in terms of bending moment, shear force, lateral wall deflections and ground movements by varying wall types, varying size of the wall and varying wall embedment length during stages excavation using the software package PLAXIS 2D version 8.2 for the chosen case study. Besides that, to determine the appropriate wall type for basement based on economical aspect. Results of these analyses were recorded in terms 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. And those Results from software analysis will be compared with each other to select the best option which is worth to consider based on their performance. And also they will be compared based on the cost which the walls worth's.

# I. Chapter One: Background of the Study

## 1.1. Introduction

Deep excavations are widely used in urban areas for the development of underground space, e.g. basements for high-rise buildings, underground car parks and shopping centers and subway stations. However, the excavation process inevitably alters the stress states in the ground and may cause significant wall deformations and ground movements. Especially when the excavation is close to adjacent infrastructure, e.g. buildings, tunnels, buried pipelines, piled foundations, the excavation-induced ground movements must be carefully monitored and controlled to be within an acceptable range to avoid any potential damage to these classes of infrastructure. The failure of an excavation may have catastrophic consequences, and special care must be taken to avoid such failure. One disaster of this sort is the collapse of a deep excavation adjacent to Nicoll highway in Singapore that occurred on 20 April 2004 (Fig.1), resulting in four casualties and a delay of part of the Circle Line subway project.



**Figure 1: Singapore Nicoll Highway collapse adjacent to an excavation (Yuepeng Dong, 2014)**

The main causes of failure are various, e.g. unexpected soil conditions, rupture of the anchoring or bracing system (e.g. buckling or inadequate connection to the wall), violating the designed construction sequences (e.g. over excavation). In most cases, however, the pre-failure performance is more important, and considerable efforts have been made to understand the characteristics of the soil and structural deformations. To reduce the excavation-induced deformations, an appropriate retaining wall and support system should be designed, as well as employing adequate construction methods. As the excavation becomes deeper and larger in scale, and may be constructed in problematic soils, challenges arise for the research, design, and construction of deep excavations. Therefore, the performance of deep excavations should be better understood through more sophisticated approaches, e.g. real-time monitoring systems, and advanced numerical predictions.

Ground movements around deep excavations critically depend on both the ground conditions (e.g. initial stress states, stiffness and strength properties, and groundwater conditions), retaining schemes (e.g. types of the retaining wall and support system, rigidity of retaining structures), and the methods of construction (e.g. top-down, bottom-up, open-cut, and excavation sequences). Excessive lateral wall displacements are largely due to inadequate support designs (e.g. flexible retaining wall, insufficient strut or anchoring system, and inadequate embedment length), and can also result from construction errors (e.g. excessive excavation). Retaining structures should be designed with respect to their lateral displacements, which can be reduced by adopting a stiffer wall, by reinforcing the anchor or strut system, or by pre-stressing these components. This general stiffening of the retaining system, however, leads to an increase in lateral soil pressure, bending moments in the retaining wall, and forces in the struts or anchors. Braced excavations with a top-down construction method are usually preferred in practice to reduce wall deformations and ground movements.

Numerical analysis can consider both the geotechnical and structural aspects in the deep excavations such as the soil properties, details of structures, and construction sequences, and provide necessary information on the performance of deep excavations for design purpose. It can also be used to predict the behavior of deep excavation and provide guidance for the construction. Field monitoring of the performance of deep excavations during the construction process can provide immediate feedback to engineers to ensure the safety of the project. Measured field data

is a valuable resource to calibrate and verify numerical analyses and facilitate a better understanding of the general performance of deep excavations.

Deep excavations are often necessary in Addis Ababa due to lack of space, very expensive and high level land costs as well as very dense traffic. Open excavation is not possible for a basement work at all. One challenging problem in deep excavation is the need to prevent damage to adjacent structures. To achieve the goal of protecting adjacent buildings, it is necessary to eliminate or reduce excessive settlement caused by the wall deflection to the direction of excavation.

There are many types of available retaining walls. Therefore, the need for and selection of a soil retaining structure should be assessed carefully. Construction method should be fully considered at the design stage, since different construction methods may require different detailed design approaches. The selection of this method must base on several aspects such as the size of excavation, ground conditions, groundwater level, vertical and horizontal displacements of adjacent ground and limitations of various structures, availability of construction cost, speed of work and others. One of the main decisions is the water tightness of wall. Thorough understandings of the parameters that control or alleviate the deformation in a soil model are essential in the modeling of excavation. Sensitivity analyses of soil and structure behavior are important that necessitate performing sanity checks on all design by alternative means. Most commonly used constitutive models coded in geotechnical software such as FLAC, ABAQUS, CRISP and PLAXIS require different input parameters obtained from various soil testing techniques. PLAXIS incorporates constitutive models whose parameters may be obtained directly or correlated from simple tests such as the SPT test. Its user friendly features have made it to be one of the popular engineering software among geotechnical engineers.

## **1.2. Statement of the Problem**

Deep excavations are anticipated when the need to utilize the underground space for different uses where the value of land is expensive. Deep excavation is being extensively practiced in Addis Ababa. Lately, there are some failures related to the retaining wall that are used for deep excavation and basement work. The failures of retaining walls can be catastrophic in addition to affecting the serviceability of adjacent structures. Thus, the selection of the soil retaining walls must be carefully considered during design and construction stages for deep excavation.

In Addis Ababa the most common type of retaining wall for deep excavation is contiguous bored pile wall. This retaining method may not be relatively effective in terms of various parameters like soil type of the area, embedment depth, size of the wall, and construction cost. Thus other alternative retaining systems should be considered and analyzed to select a best option. There are papers written to analyze the performance of different types of retaining structures. The results presented in these papers are for a specific type of retaining structure. These papers also lack cost estimation. These two factors limit the flexibility to compare results in an organized manner and have reasonable up-to-date information about the construction cost. Therefore, it is necessary to compare soil retaining structures used for deep excavations by including an additional comparison factor, which is cost of construction.

## **1.3. Objectives**

### **1.3.1. General Objective**

The general objective of this thesis is to investigate the performance of different soil retaining structures for deep excavations.

### **1.3.2. Specific Objectives**

The specific objectives of the paper are to identify the effect and performance of diaphragm, contiguous and tangent bored pile retaining walls in terms of lateral wall displacement, bending moment and ground settlement for basement work in terms of wall deflections ground movements during stages excavation and roughly estimating cost of construction. Besides that, to determine the appropriate wall type for basement based on economical aspect.

## **1.4. Significance of the Research**

The research presented in this thesis is expected to have practical implications on the design, construction, and research of deep excavations. Deep excavation in urban areas has increasingly become a necessity in different projects from high-rise buildings to subway systems. One challenging problem in deep excavation is the need to prevent damage to adjacent buildings. To achieve the goal of protecting adjacent buildings, it is necessary to eliminate or minimize excessive settlement caused by the wall deflection in excavation. Hence, the selection of the appropriate method, sequence of construction, wall type and support system is important to minimize the ground movement and to achieve economic design and construction. A design which minimize wall dimensions and material use but one which increases construction duration because it is difficult to build may not result in overall economy.

The main significance of this research is to give insight about and recommend the selection of an appropriate and economical soil retaining structure for deep excavations by comparing results of lateral wall deflection, shear force, bending moment, ground settlement and economy by taking an actual site located in Addis Ababa city.

## **1.5. Structure of the Thesis**

The thesis consists of five chapters. The first chapter is about the background of the thesis work. Literature review is presented in the second chapter. And the methodology stated in chapter three. Chapter four outlines the different parameters and results of the analyses carried out. Chapter five closes the whole theme of the thesis by making conclusions and recommendations. In the Appendices, relevant figures, constitutive models of PLAXIS and different cases of parametric study are presented.

## **II. Chapter Two: Literature Review**

### **2.1. Deep Excavations**

Deep excavation is a complex subject in geotechnical engineering and has been studied using various methods, e.g. theoretical and empirical methods, laboratory tests, field measurements, and more sophisticated numerical analysis. However, all these methods have their own limitations, although they have contributed in various degrees to the understanding of the performance of deep excavations. Some of these methods are reviewed and discussed in this chapter. Emphasis, however, is put on the various aspects of observed performance of deep excavations in the field and the capability of finite element analysis to replicate these observed behaviors.

In Addis Ababa the construction of deep excavations is a technically challenging problem. When deep excavation is anticipated, the adjacent building can get damaged if the construction cause excessive ground movement. If damage to the adjacent building occurs, the owner of the building has the right to claim for repair work from the contractor or consultant that is responsible for the construction. Thus, careful considerations during design and construction stages are necessary so that these problems can be minimized.

In recent decades, the demand for underground space, for use as underground parking, railway tunnels/stations, road tunnels, redevelopment of buildings, etc. has increased in many heavily urbanized areas. Deep excavations are required to meet the demand, and, in many cases, excavation sites are in close proximity to existing structures/facilities and problems related to deep excavations and their effects on surrounding structure are growing. Different methodologies or processes are employed to limit the impact of deep excavation on surrounding structures. The objective of this literature review is to assess the progress made in this area in terms of research as well as in the technological developments.

Performance of deep excavations is related to both stability and deformation. Deep excavations are designed to be stable and to limit deformations within acceptable levels. A stable deep excavation is an excavation whose walls do not collapse, whose adjacent soil doesn't settle excessively and whose base does not heave uncontrollably. Ground deformations around

excavations can damage adjacent buildings, streets, and utilities. The severity and extent of damage depends on the magnitude and pattern of ground movements around the excavation. Stability and deformation are related. Thus, prediction of deep excavation performance involves analysis of both stability and deformation. Experience has shown that stability can be evaluated with sufficient accuracy using simple limit equilibrium calculations. Deformations, however, are significantly more difficult to predict, and finite element analyses are often used for this purpose when ground movements are particularly important.

The removal of soil from a deep excavation has two main effects. The first is that the removal of the weight of the excavated soil results in a decrease in the vertical stress in the soil beneath the excavation. The second is that the removal of the soil in the excavation results in a loss of lateral support for the soil around the excavation.

The purpose of a deep excavation support system is to provide lateral support for the soil around an excavation and to limit movement of the surrounding soil. Support systems for deep excavation consist of two main components. The first is a retaining structure. The second component is the support provided for the retaining structure. Many types of supports have been used in deep excavations. The common types of supports are diaphragm wall (structural slurry), sheet pile, soldier piles, tangent piles, and contiguous piles. The principal types of supports are struts (braces), rakers, and tieback anchors.

## **2.2. Theoretical and Empirical Analysis Methods**

Theoretical and empirical methods provide some basic understanding of the performance of deep excavations in a different way, but they also have limitations due to their simplicity and assumptions. Some of these methods are reviewed in the sections to follow.

### **2.2.1. Classical Earth Pressure Theory**

The design of retaining walls requires the evaluation of active earth pressure which is largely based on the classic solutions of lateral earth pressure provided by Coulomb (1776) and Rankine (1857). Coulomb (1776) first studied the earth pressure problem using the limit equilibrium method to consider the stability of a wedge of soil between a retaining wall and the failure plane. It is well

verified for the frictional soil inactive state, but is not the case for either the cohesive soil or for the passive state. The point of application of active thrust is assumed at a distance of one-third of the height of the wall from its base and independent of various parameters such as soil friction angle, angle of wall friction, backfill angle, and wall inclination angle. Rankine (1857) presented a solution for lateral earth pressures in retaining walls based on the plastic equilibrium. He assumed that there is no friction between the retaining wall and the soil, the soil is isotropic and homogenous, the friction resistance is uniform along the failure surface, and both the failure surface and the backfilled surface are planar. The active and passive coefficients were developed for cohesionless soils, but they can be used for evaluating long-term conditions in cohesive soils where complete dissipation of pore water pressure occurs. These classical earth pressure theories and their further development form the basis of earth pressure calculations used today, but they are only applicable under certain conditions to estimate roughly the earth pressures on the wall. Moreover, they do not consider the construction process and give no indications on the wall deformations and ground movements in the more complex braced deep excavations.

### **2.2.2. Stability Analysis**

Stability analysis is important in the design of retaining structures in clay, and is normally conducted using limit equilibrium methods or finite element methods. Limit equilibrium calculations are usually carried out in the design and involve assuming the classical active and passive earth pressure distributions on the back and front of the wall and taking moments about the position of the prop. Terzaghi (1943) suggested a mechanism consisting of a soil column outside the excavation which creates a bearing capacity failure. The failure is resisted by the weight of a corresponding soil column inside the excavation and also by adhesion acting along the vertical edges of the mechanism

### **2.2.3. Stress Path Method**

The soil behavior depends not only on the current stress state, but also on the stress history. The removal of soil in deep excavations mainly results in a decrease of the vertical stress in the soil inside the excavation and a loss of lateral constraint for the soil on the retained side. As the excavation behavior is influenced by the stress state of the soil, understanding the stress paths in

the field during the excavation process is necessary to identify critical elements influencing the shear strength and determine appropriate strength and stiffness parameters through laboratory tests for design and analysis. The stress path method (Lambe 1967) provides a rational approach to understand the variations of effective stress in the soil elements at some typical locations caused by both horizontal and vertical stress relief during the excavation. Hashash and Whittle (2002) used nonlinear finite element analysis to interpret the evolution of lateral earth pressures acting on the well-braced diaphragm walls for deep excavations in clay and explain the soil arching mechanism. It was demonstrated that the stress path experienced by a soil in front of the wall at the final excavation level follows a typical path of plane strain passive mode of shearing, whereas the soil elements behind the wall on the retained side follow more complicated stress paths due to rotations of the principal stress directions and reversal in shear direction caused by the soil arching mechanism.

#### **2.2.4. Empirical Methods**

Empirical methods are used to interpolate the performance of deep excavations from the analysis of previous published field data in different areas of the world and local experiences. Terzaghi (1943) suggested that the average earth pressure is approximately uniform with depth and has small reductions at the top and bottom of the wall based on field measurements. Terzaghi and Peck (1967) proposed the apparent earth pressure envelopes based on field measurements from various locations for predicting maximum strut loads in a braced excavation. However, these diagrams do not represent the real distribution of earth pressures at any vertical section in an excavation, and this method has been evaluated by many different researchers summarized the ground surface settlements from field measurements of deep excavations in different areas of the world and classified the settlement curve into three zones depending on the type of soil and workmanship, and this method was expected for rough estimates of ground surface settlements under various conditions. However, the general description of settlement curves neglects important factors such as soil conditions, wall installation methods, types of retaining structures, and the construction sequence. The maximum lateral wall deflection is evaluated relative to factor of safety against basal heave by Terzaghi (1943) and system stiffness. The derived curves are based on average

condition, good workmanship, and the assumption that cantilever deformation of the wall contributes only a small fraction of the total movement.

## **2.3. Laboratory Tests and Field Measurements**

Braja M. Das (1997) states that the performance of deep excavations has been studied through both laboratory tests and field measurements by a number of researchers, and the main findings are summarized in this section. The advantage of laboratory tests is that factors influencing the results may be controlled quantitatively. The tests can be repeated and continued until failure, which is not possible in the full scale projects. Moreover, the tests are time-efficient and can observe the long-term behavior of a geotechnical construction in soil of low permeability over a reasonably short period of time. However, it should be noted that centrifuge testing has its limitations and the conditions it can model are relatively simple. Field measurement is an effective method, but it is also expensive and takes a long time to obtain the data, and this process is not repeatable.

### **2.3.1. Wall Deformation**

The wall deformation (i.e. lateral deflection and vertical movement) is the major concern in deep excavations and is monitored in most field measurements. The pattern and magnitude of the wall deformation are influenced by a number of factors, e.g. the soil condition (e.g. stiffness, strength, anisotropy, and creep), the support system (e.g. stiffness, connections, and thermal effects), and construction methods (e.g. top-down, bottom-up, and cut and cover). The wall deflections are normally measured with inclinometers, but the readings need to be adjusted to be consistent with the surface survey, because inclinometers usually only record the deflection pattern of the wall by assuming no displacement at the toe of the wall. In practice, however, a non-zero displacement at the toe of the wall is confirmed from both field measurements and numerical analyses.

There are several general trends from the observed wall deflection:

- ✚ The wall deforms as a cantilever deflecting inwards the excavation prior to the installation of the first level of props.
- ✚ The deflection is bulging after the installation of the first row of props as a result of the rotation of the wall about the prop position and of bending deformation, and the magnitude increases as the excavation proceeds.
- ✚ The largest wall deflection occurs around the excavation level.
- ✚ The wall translates horizontally with displacement at the toe of the wall.
- ✚ The maximum wall deflection depends on various factors such as the soil properties (e.g. stiffness, and strength), the type and stiffness of the retaining system (i.e. the retaining wall, and the bracing structure), and the construction method (e.g. top-down, bottom-up, and open-cut).
- ✚ The progressive wall deflection observed during the no construction period may be caused by the dissipation of pore water pressure, but the increment is small compared to that induced by the excavation.
- ✚ The wall deflection is smaller close to the wall corner than that close to the wall center due to the corner effect.

The wall deflections continue after the excavation reached the formation level, but at a reduced rate, probably due to the dissipation of pore water pressure. Large incremental wall deflections occur during the inefficient support period. The wall deflections increase during the structure installation process, which may be attributed to the shrinkage of the bracing concrete floor slabs. Sometimes larger than expected wall deflection were observed, which can be caused by many factors, such as over excavation, long construction duration, or large exposure period without support. The wall movement may increase with time while the excavation depth remains unchanged, due to the soil creep and excess pore water pressure dissipation during a long construction period

### **2.3.2. Ground Movement**

Ground movement is inevitable in deep excavations, and its magnitude depends on various factors such as soil properties, type of the retaining systems, excavation geometry, construction sequence, and workmanship. Excessive ground movement will cause damage to adjacent infrastructure. Therefore, understanding the characteristics of ground movement is essential in the design of deep

excavations, and to take measures to mitigate the adverse impact on adjacent infrastructure. Significant ground movement has been observed during the construction of diaphragm wall panels and bored piles, as well as during the dewatering process. The horizontal ground surface movements (towards the wall) are generally larger than the vertical surface movements and extended further from the wall. The soil lateral displacement close to the soil-wall interface in the slurry trenching process generally decreases with depth, and slightly recovers during concreting process. The ground movements increase in the subsequent excavation process and installation of horizontal support structures. Most field measurements observed settlements at the ground surface in the retained area during the excavation process, and the settlement stabilized quickly following the casting of base slabs. Larger than expected settlements at the ground surface was observed as a result of over excavation. The largest incremental ground movements occurred when the excavation was approximately half completed. On the contrary, ground heave was observed inside the excavation, caused by stress relief due to the soil removal and swelling as negative excessive pore water pressure dissipated and the inward movement of the wall. The distribution of ground movements outside the excavation is actually three dimensional, and is affected by the geometry of the excavation.

### **2.3.3. Earth and Pore Water Pressure**

The understanding of the performance of deep excavations can be improved if more knowledge is known of the original in-situ stresses within the soil, and the changes of earth and pore water pressure during the construction. The horizontal total and effective stress slightly recovered after the wall installation but continued to decrease at the back of the wall in the subsequent excavation process, and these reductions were greater near the surface and close to the wall. Excavation also caused significant reductions in total stress in front of the wall which was entirely due to a drop in pore water pressure, and there was a slight increase in effective horizontal stress, which may be expected from the lateral movement of the wall. When the roof structure was being constructed, the total horizontal stresses on both sides of the wall were virtually constant, but there was a gradual increase in pore water pressure in front of the wall and to a smaller extent at the back of the wall. The initial in situ pore water pressure is often observed close to the hydrostatic state, and

this assumption is normally used in the design. However, the pore water pressure may change during the wall/pile installation and excavation process, as well as the dewatering and drainage.

### **2.3.4. Wall Bending Moments**

Cheng Liang Hsiao (2007) suggested the bending moment ( $M$ ) of the retaining structures can be calculated through the measurements from the reinforcement bar strain gauges, assuming that the variation of stresses over a cross-section of the wall is linear and the neutral axis lies along the center of the wall. Alternatively, the bending moment can be computed from the curvature ( $k$ ) of the wall deflection curve using the equation, where  $EI$  is flexural rigidity. The bending moment computed from the reinforcement bar strain gauges are generally smaller than that from the wall deflection curve, particularly for the location in the neighborhood of the maximum lateral wall deflection, and they explained that the bending moment from the wall deflection curve is computed without considering cracking of concrete, so that the moment of inertia of the wall has not been reduced. The prop loads and wall bending moments gradually increase as the excavation proceeds, and may slightly decrease after the excavation is completed, due to dissipation of excess pore water pressure. The negative wall bending moment (towards the excavation) is also influenced by the prop loads. Increasing the embedment depth of the retaining wall will lead to an increase in wall bending moment and a reduction in bottom prop load, as shown in centrifuge tests. Results also suggested that the prop loads and bending moments are influenced by the in-situ lateral earth pressure, the soil properties, the geometry and depth of the excavation, construction sequence, and the change of water table level behind the retaining wall. Temperature effects on the prop loads are important in propped excavations. Perturbations in the axial prop loads were observed which may be attributed to changes in ambient temperature. However, thermal effects are unlikely to have any significant effect on temporary steel props supporting comparatively low flexibility walls. A seasonal variation of bending moment in the retaining wall has been observed in the field measurements, which is probably a result of the thermal expansion and contraction of floor slabs in summer and winter.

### **2.3.5. Deformation and Damage of Adjacent Infrastructure**

The excavation-induced ground movements may cause deformation and damage in the adjacent infrastructure, e.g. buildings, tunnels, buried pipelines, pile foundations, highways, and bridges. It is important to understand the response of this infrastructure induced by the ground movement, and to assess properly the damage in the infrastructure caused by excavations. Emphasis here is placed on the response of buildings and pile foundations to excavation-induced ground movement, and the criteria to evaluate the building damage.

The building performance during the excavation may be affected by factors such as the type and size of foundation, the geometry of the excavation, and the shape of the settlement profile. A building near a relatively short excavation side may experience smaller inclination than if it is near a long excavation side. Finno and Bryson (2002) found that the settlement of a building outside the excavation follows the development of lateral movement of the soil and the secant pile wall as a result of undrained deformations in saturated clay, and the reduction of wall stiffness and creep also resulted in larger building settlement. Blackburn and Finno (2007) observed tilts of the adjacent buildings on shallow foundations and found that the buildings tilt towards the excavation as a rigid body due to the rigid connection between the wall and underlying strip footing. Only minor diagonal shear and vertical tensile cracks were detected in the external stone and mortar facades of the external bearing wall, and these cracks occurred at locations where the largest distortions were observed. The vertical cracks in the external wall occurred at the transition point between the flat and sloped settlement distribution which also coincided with a change in footing type and footing elevation. The buildings settled unevenly, which may be related to the stiffness of the building and the relative location to the excavation. The settlement is larger close to the center of the diaphragm wall and to assess properly the damage in the infrastructure caused by excavations. A building near a relatively short excavation side may experience smaller inclination than if it is near a long excavation side. They also suggested that information regarding a building's location relative to the settlement influence zone is helpful in planning building protection measures during excavation.

Pile foundations may be adversely affected by nearby deep excavations because the lateral loads imposed by the soil movement induce bending moment and deflection in the piles which may lead to structural distress and failure. The piles essentially deformed with the soil because they are

relatively flexible compared to the soil. Finite element analyses indicated that the actual moments in the piles were not large enough to cause cracking, and the lateral or axial load capacity of the main pile groups was not significantly affected.

## **2.4. Numerical Modelling**

Numerical modelling is an effective way to investigate the soil-structure interaction mechanisms in deep excavations, and has the ability to provide all the required information for design purposes. Some of the numerical modelling processes are described in this section, and the main findings are also summarized.

### **2.4.1. Model Details and Simulation Process**

2D simulations (i.e. plain strain and axisymmetric analysis) have been widely used to approximate real deep excavations in the design process and for research purposes due to the limitation of software capabilities and computational resources available.

The advances in hardware and software nowadays have enabled the application of fully 2D analysis in deep excavations, which can include more geotechnical and structural details (e.g. ground profile, excavation geometry, retaining system, and construction sequence) and deal with large-scale case studies. The geometry and distribution of retaining structures such as the diaphragm wall, the sheet pile wall, joints between diaphragm wall panels, the soldier piles, and horizontal struts, were represented properly in the analyses. The results were promising in these analyses, and the agreement with field measurement was reasonably good considering the uncertainties and complexities involved in the analysis.

In most analyses, the wall installation process was modelled by the Wished-In-Place (WIP) method which is not able to consider the installation effect on the ground movement and subsequent excavation behavior. However, the excavation for diaphragm wall panels or bored piles is certain to result in significant in situ total stress relief which will alter the level of horizontal total stress applied to the retained side of a wall, and can therefore be expected to influence the actual values of prop or anchor forces and of the maximum bending moment in the wall. Substantial ground movement and reduction of in situ lateral stress have been observed in the field measurements

during the construction of embedded retaining walls. The installation effects of bored piles and diaphragm walls have been investigated using numerical analyses in 2D.

The initial stress state in the ground prior to the construction is related to the vertical effective stress and the coefficient of lateral earth pressure at rest. Potts and Fourie (1984) conducted a series of plain strain analyses of a single propped retaining wall at the top in drained condition using two values of (0.5 and 2.0), and found that the value of  $K_0$  has a large influence on the excavation behavior such as the wall displacements and bending moments, the prop force, the ground movements, the stress state and stress path in the soil, and earth pressure on the wall. Potts and Fourie (1984) assumed two totally different types of construction method, excavated and backfilled, and found that the excavation behavior was largely different.

Potts and Fourie (1985) found that the wall stiffness has a large effect on the wall displacements, earth pressures on the back of the wall, wall bending moments, and prop forces. Strut prestressing was found to affect considerably the deflection of the upper portion of the wall while virtually no significant change is evident at the bottom of the wall, and the ground settlement also reduces with the increase in the magnitude of strut pre-stress.

The dewatering and consolidation process is usually neglected in the analyses by assuming undrained conditions, but they can be considered in the analysis straightforwardly. Dewatering of the site can be simulated by controlling the pore pressures at specific locations, and consolidation can be modeled using displacement-pore pressure coupled analysis. In practice, however, the undrained assumption is usually reasonable because the permeability of the soil is small (on the order of  $10^{-8}$  to  $10^{-10}$  m/s) and the construction period is relatively short (e.g. 1 or 2 years). The excess pore water pressures generated during the construction will dissipate after the completion of the construction, and drained conditions are normally assumed to represent a long-term situation in the design and analysis.

Conventional soil models (e.g. Mohr-Coulomb, Soil Hardening model) have been used frequently and extensively in the numerical analyses of deep excavations, but they tend to generate unrealistic ground movements, e.g. upward ground movement rather than settlement in the retained area

outside the excavation, and larger than expected basal heave inside the excavation, because the small-strain stiffness nonlinearity is not considered in these models.

Clough and Hansen (1981) demonstrated that the strong anisotropy of the clay has a large influence on the excavation behavior due to the stress reorientation in the field, whereas the stiffness anisotropy has a relatively smaller influence on the excavation behavior. Finno, Harahap et al. (1991) used both isotropic and anisotropic bounding surface models in the analysis and showed that the isotropic soil model produced smaller sheet-pile wall displacement and ground settlement than the anisotropic model and underestimated the observed behavior. Ng, Leung et al. (2004) investigated the effects of inherent stiffness anisotropy of the soil on the excavation behavior, using a linear elastic model based on a hypothetical plane strain multistage excavation under fully drained conditions, but they found a relatively small difference (1- 2mm) of computed ground settlement and wall deflection between the isotropic and anisotropic analyses. However, it may be worth investigating the anisotropic effects by using a more advanced soil model which can consider the stiffness and strength anisotropy of the soil.

## **2.4.2. Constitutive Models for The Soil, Structures, and the Soil/Structure Interface**

Except for various details in the modeling process, the key issue to be addressed in capturing the main excavation behavior is the material constitutive model for the soil, structures, and the soil-structure interface. Some of these models are summarized in this section.

### **2.4.2.1. Constitutive models for the soil**

In deep excavations, the soil behavior is usually within a relatively small deformation region, so the pre-failure performance is more important than failure conditions. The accuracy of numerical solutions largely depends on the ability of the constitutive model to describe soil behavior in generalized stress and strain conditions. However, considering the complexity of soil behavior, it is not realistic to develop a completely generalized effective stress model for all soils. For a given boundary problem, the complexity of the constitutive model should be closely tied to the major aspects of the problem (e.g., stiffness, strength, deformation, dilation, and anisotropy) to be tackled

and/or the accuracy of solution which is required. Once the soil model is identified, its application in the finite element analysis requires appropriate procedures to derive the model parameters and calibrate the model through laboratory and/or field tests. A number of advanced soil models have been used in the analysis of deep excavations with certain kinds of success, and their characteristics are discussed briefly in this section.

#### **2.4.2.2. Constitutive model for structures**

Structural components in deep excavations are mainly reinforced concrete structures and steel struts. The steel struts can be generally modelled as a linear elastic material, but reinforced concrete components are more complicated due to the imperfections in the concrete such as cracks. The concrete behaves as a linear elastic material in compression until it reaches ultimate strength and subsequently fails in a brittle manner. Under tension conditions, since the failure strength is small, linear elastic model is quite accurate and sufficient to predict the behavior of concrete until failure. However, this simple linear elastic constitutive law is often inappropriate as concrete cracks which is highly nonlinear and inelastic, and in the case of reversal loading.

#### **2.4.2.3. Constitutive model for the soil-structure interface**

The soil-structure interface behavior can be crucial to the overall response of a soil-structure system and should be taken into account properly in the finite element analyses. One way of considering the interface behavior is using a rigid perfectly plastic Coulomb friction model. Another approach is to consider the interface as thin continuum interface elements.

The shear resistance is governed by the Coulomb friction criterion. 2D interface element for the soil-structure contact analysis of a retaining wall. The interface stress is characterized by the normal and shear stresses which are related by a constitutive law to the normal and tangential interface element strains. The constitutive law uses a linear elastic-perfectly plastic model using a Mohr-Coulomb failure criterion as the yield surface. However, they also experienced numerical problems such as ill-conditioning, poor convergence of solution and unstable integration point stresses.

Generally, deep excavation is a complex soil-structure interaction problem, and its performance is influenced by a number of factors such as soil conditions, the type of retaining structures, construction methods, and the workmanship. Several methods in the analysis of deep excavations are reviewed and discussed in this chapter, e.g. theoretical and empirical methods, laboratory tests and field measurements, and numerical analyses.

Theoretical and empirical methods provide some basic understanding of the behavior of deep excavations, but they have limited applications due to their simplicity. Laboratory tests, e.g. centrifuge tests, are useful to investigate some aspects of deep excavations through well-controlled procedures, but it should be noted that centrifuge testing has its limitations and the conditions it can model are relatively simple. Field measurements reflect the real performance of deep excavations, and the field data are valuable to calibrate the numerical analyses. However, field measurements are expensive and take a long time to obtain the data, and the process is not repeatable. Moreover, field measurements can only record the data and are not appropriate for predictions. Numerical modeling is an efficient tool to investigate the behavior of deep excavations and can be used for predictions. However, for more reliable prediction purposes, numerical analyses need to consider appropriately both geotechnical and structural aspects such as the irregular geometries, detailed retaining structures, correct construction sequences, realistic material models, and reliable input parameters.

## **2.5. Types of Retaining Wall**

There are many types of retaining wall that can be adopted in deep excavation as a permanent basement wall. In deep excavation works, the stability of the wall is one of the main important factors that need to be considered. The types of retaining wall are listed below:-

1. Contiguous bored pile
2. Tangent bored pile walls pile
3. Diaphragm wall

### **2.5.1. Contiguous Bored Piles**

Contiguous bored piles wall can be constructed as basement wall in order to avoid excessive bulk excavation and help to control ground movements. The bored pile has various ranges of sizes ranging from 600mm to 2000mm diameter. It is constructed with gaps in between the pile. Thus, the gap in the structural wall has exposed the soil. Hence, this option is suitable where the retained soil is usually firm to stiff (not generally granular). This is the most economical option and normally the fastest method to construct.



**Figure 2 Contiguous Bored Pile Wall**

### **2.5.2. Tangent Bored Piles Walls**

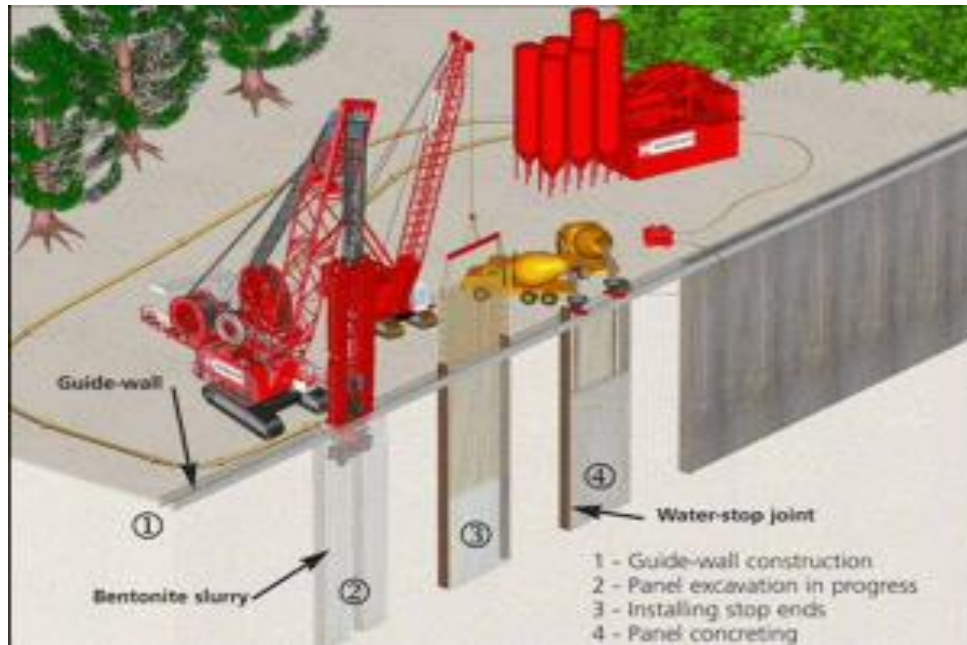
An earth retention system consisting of contiguous (i.e., abutting with small or no gap) vertical piles (e.g., drilled shafts, columns, etc.). A tangent pile wall is constructed by installing concrete or cement/bentonite grout primary vertical piles (with or without steel reinforcement) at prescribed intervals. Secondary vertical piles are then installed in between and typically abutting the primary piles, wherein reinforcing steel is inserted and the secondary piles are filled with the appropriate concrete mix. The reinforcement may be composed of either a steel reinforcement cage or a steel beam section. Typically, the primary and secondary piles are of the same diameter. Wall designs may allow small gaps (1-6 inches) occurring between tangent piles.



**Figure 3 Tangent Bored Pile Wall**

### **2.5.3. Diaphragm Wall**

Diaphragm walling is a technique of constructing a continuous underground wall from the ground level. These reinforced concrete 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 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 for 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. The major disadvantage of diaphragm wall is it requires massive equipment, long construction period, and huge cost.



**Figure 4 Diaphragm Wall**

## **2.6. Excavation and Construction Methods**

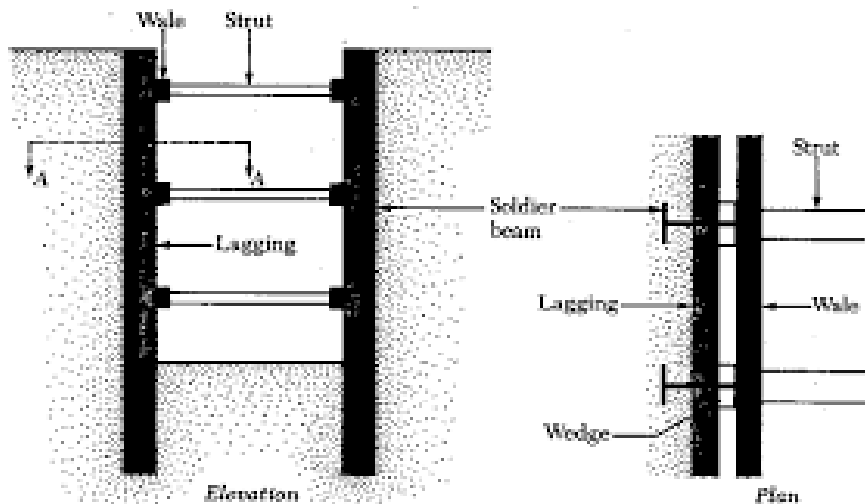
Deep excavation method becomes more complicated when the excavation increases in depth. Thus, the need to properly choose excavation method can minimize the ground movement and also cost of the whole project. Practices engineer nowadays have several options that can be considered for deep excavation work such as the full open cut method, the braced excavation method, the island excavation method, the anchored excavation method, the top-down construction method, and the zoned excavation method. Based on various options available the engineer need to be able to properly choose the type of excavation because it can affect the construction time, available budget and safety of the worker.

In Addis Ababa, the available land for construction is very limited. Normally full open cut method cannot be considered because this method does not use retaining walls or struts but only make slope cut. Thus, it requires a lot of space for deep excavation work because the slope may be very gentle, the amount of excavated soil is tremendous and a great amount of soil will be needed to backfill after the construction is finished.

### 2.6.1. Braced Excavation Methods

In this method, horizontal strut is installed in front of retaining walls to resist the earth pressure on the backs of walls. There are several components in braced excavation method which are struts, wales, end braces, corner braces, and center posts. The purpose of wales is to transfer the earth pressure on the back of retaining walls on to horizontal struts. End or corner braces can help shorten the span of wales without increasing the number of struts.

For narrow excavations, internal struts are most appropriate. Before struts are installed, a horizontal member called waler is placed against the soil support. Intermediate struts are then installed from waler to waler across the excavation.



**Figure 5 Braced Excavation Method a) Profile and b) Plan**

For very wide excavations, raker bracing is used. The supports for the rakers are installed at the bottom of the excavation as shown in the Figure 6 below.



**Figure 6 Support for Raker**

### **2.6.2. Anchored Excavation Methods**

This method substitutes anchors for struts to stabilize the lateral earth pressure. An anchored retaining wall can be constructed in any of the aforesaid styles but also includes additional strength using cables or other stays anchored in the rock or soil behind it. Anchor usually is driven into the material with boring either by mechanical means or often by injecting pressurized concrete. This method is very beneficial where high loads are expected, or where the wall itself has to be slender and would then be too weak.



**Figure 7 Anchored Excavation Method**



### 2.6.4. Bottom-Up Construction Methods

In this method, the earth is excavated to the required depth with retaining walls and struts supporting the soil at the sides. Upon the completion of excavation to the required depth, the base slab of the underground structure is cast at the bottommost level, followed by the side walls. Casting of concrete progresses upwards, level by level till the roof of the structure is completed. Ground is then backfilled and reinstated.

The safety issues are the risks to the workforce during the installation and removal of these elements together with the risk of accidental damage or unforeseen loading on the props during the excavation and construction operations. The sequential construction of the superstructure and the substructure minimizes conflicts between different operations on site and normally results in a less congested critical path compared to a top down sequence.

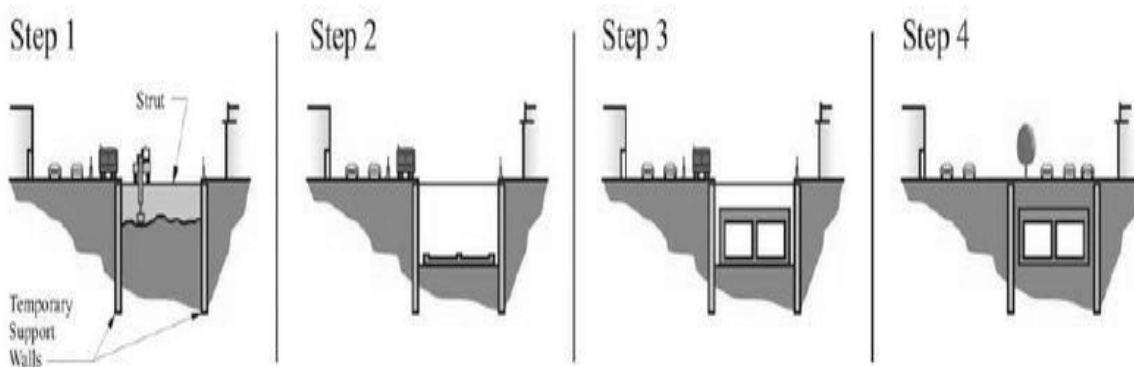


Figure 10 Bottom-Up Construction Sequence

### 2.7. Analysis

Construction of basements for new buildings is a normal method being adopted in Addis Ababa, where the land is very expensive. An adequate support system is a must in order to minimize the amount of excavation and to ensure that adjacent property is not damaged. Thus, the analysis of such an excavation should pay particular attention to eliminate all possibility of the collapse of the retaining structures. Since unexpected ground conditions are common, the engineer must be able

to know the important factor that needs to be considered in the analysis. In practice, a wrong analysis may lead to an uneconomical or even unsafe design.

### **2.7.1. Ground Movement**

Gordon Tung-Chin Kung (2007) states that in deep excavation work, excessive ground movement tends to cause problems with the adjacent properties such as buildings, roads, utilities. When this problem occur the cost for remedial work is high. Hence, the decision to set particular wall deflection and ground movement limits can be of significant economic importance. The setting of appropriate limits should be carefully considered.

### **2.7.2. Sources of Ground Movements**

Ground movement is induced due to the construction of the wall, excavation in front of the wall and flow of water causing loss of ground and consolidation caused by changes in water pressures due to seepage through and/or around the wall. Each of these is discussed separately below.

#### **2.7.2.1. Effect of Wall Installation**

The process of installing a diaphragm or bored pile retaining wall may potentially be important in three respects:

1. Wall installation by boring or excavating panels may reduce the horizontal effective stresses close to the wall to below their in situ values. Wall installation by ground displacement methods (e.g. driving) may increase the horizontal effective stresses close to the wall.
2. During wall installation, the surrounding soil may be subjected to various stress paths involving lateral unloading and reloading. These define the recent stress history of the soil, which may influence the soil stiffness during bulk excavation in front of the wall (Powrie et al, 1998).
3. Ground movements during wall installation may require consideration in their own right. Experience (Thompson, 1991; Powrie and Kantartzi, 1996) indicates that ground movements due to the installation of a cast-in-place wall under good workmanship conditions, in the stiff

ground where the water table is low are unlikely to be significant. Ground movements arising from wall installation where the ground is very soft and/or the water table is high, or workmanship is poor or local construction difficulties (e.g. obstructions in the ground) are encountered can be significant. For driven walls in coarse-grained deposits, vibration induced settlement can also be vital.

When the soil is removed from in front of an embedded retaining wall, the wall will usually move into the excavation. This will result in a reduction of the lateral stress in the soil behind the wall and eventually bring it to the active condition in which the soil is at failure with the horizontal effective stress as small as it can be for the effective overburden pressure. In the soil that remains in front of the wall below formation level, the lateral stress increases until eventually a state of passive failure is reached, in which the horizontal effective stress is as large as it can be for the effective overburden pressure.

#### **2.7.2.2. Wall Friction**

The interface between the soil and the wall is not frictionless. As a result of wall friction, the resultant force between the wall and the soil is inclined rather than normal to the wall.

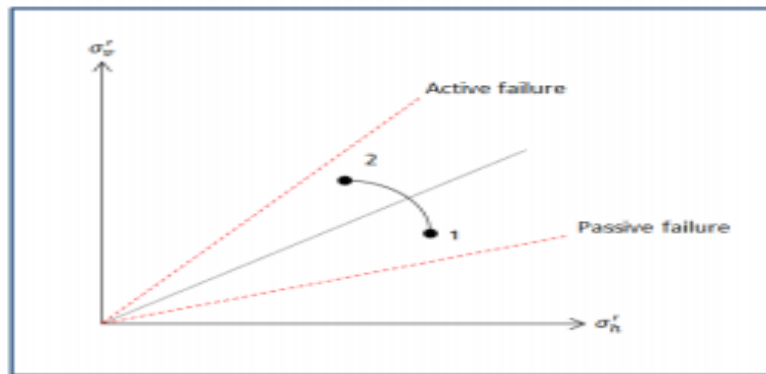
Measurements of wall deflections, ground movements and the use of numerical analysis over recent years have allowed a better understanding of ground behavior. The assessment of ground movements is not straightforward and more experience is required to make appropriate use of numerical analysis. It is therefore essential that optimum use is made of precedent in comparable conditions through the use of case history data.

#### **2.7.2.3. Excavation in Front of a Wall**

During the deep excavation work in the construction of underground car park, the stability and deformation characteristic is very crucial. Excessive ground movement induced by the excavation may cause failure to the retaining wall and also can cause damage to the adjacent building, roads or utilities. Thus, the need to have some basic knowledge of the stress path of the soil is very important in order to prevent failures. In deep excavation, there are two possible failures that can

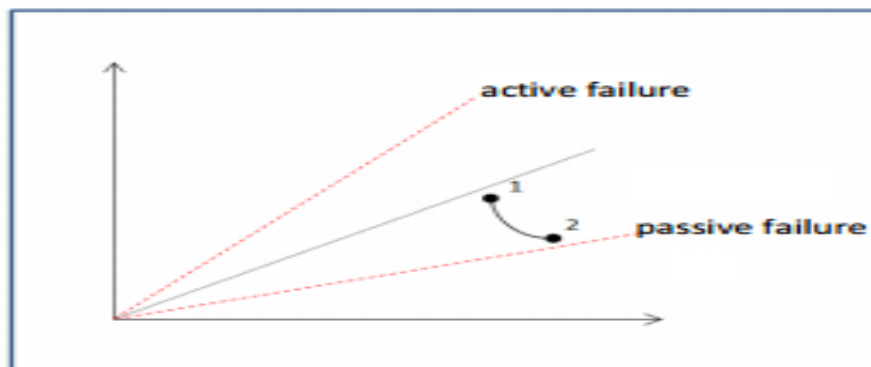
occur, which are active failure and passive failure. In order to understand more about how to avoid these two failures during excavation work, first let's assume the location of the stress path element.

During excavation in front of the wall, the wall is likely to move forward into the excavation area. This can cause a reduction in horizontal total stress for the soil element (a) which is located behind the wall. The pore water pressure will also decrease behind the wall. Thus based on the Figure 13, the stress path for element A is shown as moving from point 1 to 2. Hence, the stress path is approaching to the active failure.



**Figure 11 Location of Assumed Stress Path**

During excavation, the soil of element B will experience a large reduction in vertical total stress, which will result in large reduction of pore water pressure. Meanwhile, the movement at the wall below formation level into the soil in front tends to increase the horizontal stress. These changes will result in increasing horizontal effective stress and reducing in vertical effective stress during excavation. The stress path is shown on the Figure 14 at which the stress is approaching the passive failure line.



**Figure 12 Stress Path of Soil Element during Excavation**

The stress path can assist engineer to have general idea on how to prevent the failure during deep excavation work. The retaining structure is built to provide some restraint to the element which tends to experience reduction of lateral pressure. Therefore, an adequate support system is required to avoid excessive lateral wall movement.

## **2.8. Excavation and Protection of Adjacent Buildings**

The influence of excavation on the surrounding environment is not great when shallow excavation is anticipated. However, with the increase of excavation depth and scale in urban areas, the magnitude and extent of ground settlement increases, which can cause damages the structures surrounding it. Once the problem the influence of excavation on the surrounding environment is not great when shallow excavation is anticipated. However, with the increase of excavation depth and scale in urban areas, the magnitude and extent of ground settlement increases, which can cause damages the structures surrounding it. Once the problem occurs, the adjacent building will get affected and the responsible person that in charge for the project must do the remedial work?

## **2.9. Stability Analysis**

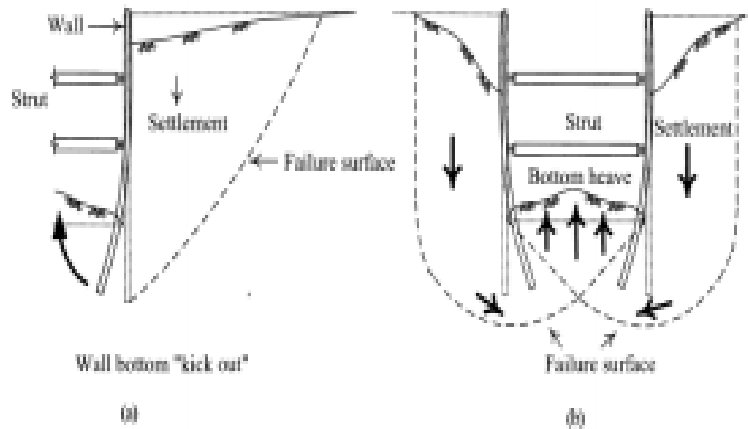
K.R Arora (2003) explained Failures or collapses of excavations are disastrous at excavation sites. Life of workers at the site is exposed to the risk of collapsing excavation. Furthermore, the failure also can be catastrophic and can cause the collapse of the adjacent building. Since the ground movements that can cause failures are so crucially influential, the failures or collapses of excavation need to be avoided. Hence, stability analyses are therefore required. Failure of an excavation can be from the stress on the support system exceeding the strength of its materials for instance when the strut load exceeds the buckling load of struts or the bending moment of the retaining wall exceeds the limiting bending moment. Failure can be also due to shear stress in soil exceeding the shear strength. The methods of analyzing whether the soils at the excavation site are able to bear the stress generated by excavation are called stability analyses. Stability analyses include overall shear failure analysis and upheaval analysis. The overall shear failure analysis can be further divided into push-in and basal heave failure analyses.

### **2.9.1.1. Overall Shear Failure**

Arora (2003) the failure state or limiting state is said to happen when the shear stress at a point in soil exceeds or equals the shear strength of soil at the point. When many failure points connect up into continuities and form a plane, the failure surface is thus produced. Once the failure surface is produced, the excavation failure or collapse will occur. This is called the overall shear failure. There are two main overall failure modes of excavation which are push-in and the basal heave failure. The push-in is a failure is caused by the earth pressures, reaching the limiting state, on both sides of the retaining wall, which is thereby moved, a large distance, toward the excavation zone until reaching the full-zone failure. The analysis views the retaining wall as a free body and the external forces on the wall and internal forces of the wall are in equilibrium. When push-in is caused, with different extents of movement of the embedded part of the retaining wall, the earth pressure on the retaining wall varies.

The basal heave happens when the weight of soil outside the excavation zone exceeding the bearing capacity of soil below the excavation bottom. This has caused the soil to move and the excavation bottom to heave so much that the whole excavation collapses. Figure 15 is a possible form of basal heave. When analyzing the basal heave, we should assume several possible basal heave failure surfaces and find their corresponding factors of safety according to mechanics. The surface having the smallest factor of safety is the most likely potential failure surface or critical failure surface. With the variable forms of critical failure surfaces, there exist many analyzing methods

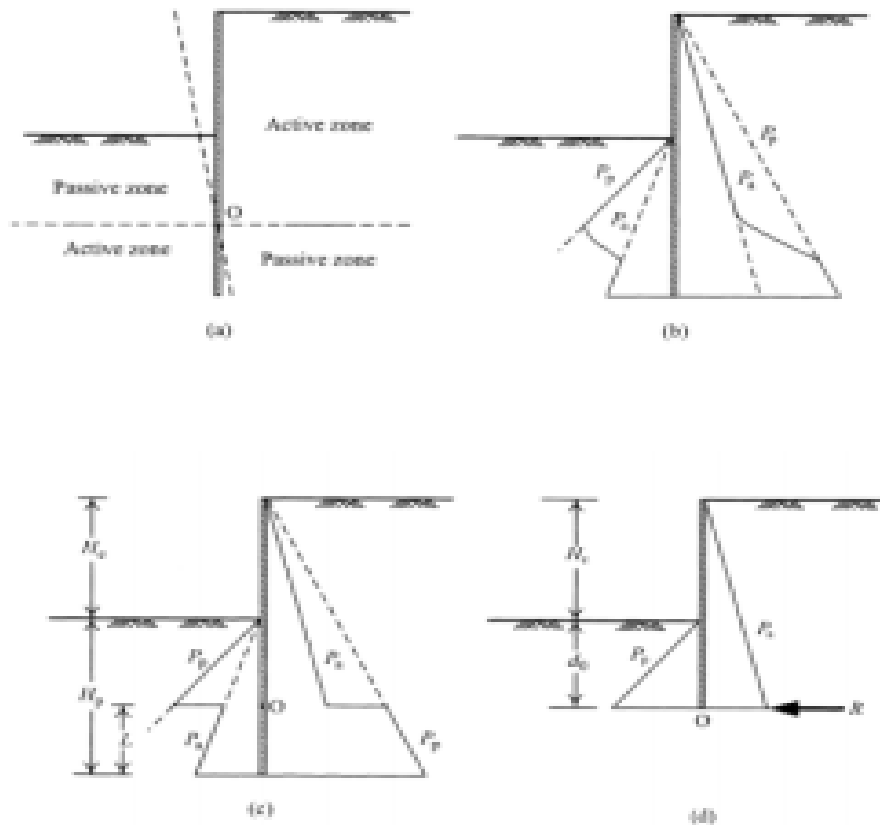
As mentioned above, the mechanisms of push-in and basal heave are different. Basically, push-in refers to the stability of the retaining wall. Push-in also causes soil near the wall to heave. As to basal heave, it refers to the stability of the soil below the excavation bottom and its failure surface may pass through the bottom of the retaining wall or through the soil below the bottom of the retaining wall. When basal heave occurs, the soil around the excavation bottom will mostly heave. Nevertheless, when it occurs to a soft clay ground, the earth pressure on both sides of the wall may also reach the limiting state, from which it follows that a push-in failure is also possible.



**Figure 13 Overall Shear Failure Modes: (a) Push-In and (b) Basal Heave**

### **2.9.2. Overall Shear Failure of Cantilever Wall**

Based on theory, the overall shear failure analysis of a cantilever wall should include analyses of push-in failure and basal heave failure. Though, the stability of the cantilever wall is rather weak and its application is usually confined to sand, gravelly soils or stiff clays. On soft clay, the cantilever wall is not reliable enough to be adopted. Since no basal heave failure is found happening in sandy gravel soils or stiff clays, as far as the cantilever wall is concerned, only analysis of push-in failure is required. The stability of a cantilever wall counts on the soil reaction at a specific fixed point. Figure 14 illustrates a cantilever wall rotating about point O in a limiting state. Figure 14 shows the earth pressure on the retaining wall. For the simplification of analysis, assume the active and passive earth pressures above and below cantilever point O are fully mobilized and therefore the earth pressure distribution is discontinuous around point O as shown in Figure 14.



**Figure 14 Analysis of a Cantilever Wall by Gross Pressure Method: (a) Deformation of the Retaining Wall, (b) Real Distribution of Lateral Earth Pressure, (c) Idealized Distribution of Lateral Earth Pressure, and (d) Simplified Analysis Method**

## 2.10. Stress and Deformation Analysis

There are many different ways to analyse earth retaining structures. Computer software has become more important in order to analyse complex analysis so that more accurate and faster result can be obtained. Finite element and finite difference formulation methods is common tool to calculate the nonlinear analysis nowadays. A complex soil constitutive behavior, actual construction sequences, structural and support details, and consolidation and groundwater effects can be modelled by using a finite element method. Ground movements, wall movements, bending moments and prop loads are well calculated but may be of limited value unless a well-developed

soil constitutive model has been used and the results "calibrated" against reliable measurements of well-monitored comparable excavations and wall systems.

Nowadays, there are many powerful computer programs which are able to calculate distributions of active and passive pressures, given basic data such as strata thickness and soil parameters. One of the good geotechnical software that available in the market at the moment is Plaxis2D. PLAXIS 2D is a finite element package that is able predicts deformation and stability in geotechnical engineering projects. The simple graphical input procedures enable a quick generation of complex finite element models, and the enhanced output facilities provide a detailed presentation of computational results. The calculation itself is fully automated and based on robust numerical procedures. This concept enables new users to work with the package after only a few hours of training.

PLAXIS is a finite element package intended for analysis of deformation, stability and groundwater flow problems in geotechnical engineering (Brinkgreve et al. 2004). It is widely used in practice because of its simplicity, user-friendliness and reliability. PLAXIS is equipped with features to deal with deep excavation supported with diaphragm wall. A brief summary of some important features of PLAXIS is given below:

- The input parameters and the boundary conditions of the geometry can be drawn based on computer-aided drawing (CAD) procedures. A 2-D finite element mesh can also be generated easily
- It allows automatic generation of unstructured triangular 2-D finite element meshes with options for global and local mesh refinements.
- 6-node and 15-node triangular elements are available to model stresses and deformations in the soil
- Special beam elements (designated as plates) are used to model structural elements such as diaphragm walls, tunnel linings, shells, and other slender structures.
- Elasto-plastic spring elements are used to model anchors and struts.
- The presence of geo-synthetic reinforcements (e.g. a geotextile or a geogrid) can be simulated by the use of special tension elements
- Steady-state pore pressure can be generated using either phreatic levels or groundwater flow calculation. Excess pore

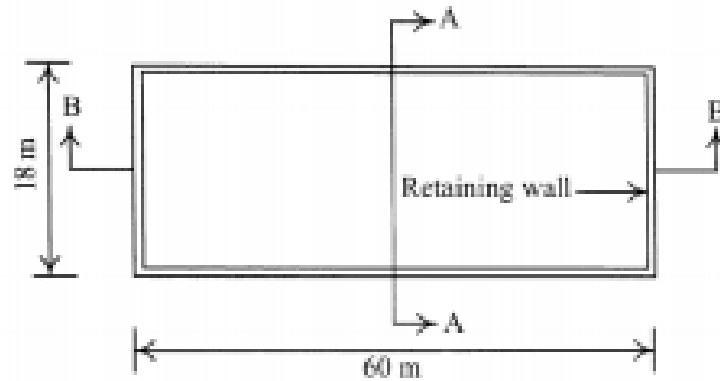
pressures are computed during plastic calculations when undrained soil layers are subjected to loads.

- Automatic load stepping avoids the need for the user to select suitable load increments for plastic calculations, thus ensuring an efficient and robust calculation process.
- Stage construction feature enables a realistic simulation of construction and excavation processes by activating and deactivating clusters of elements, application of loads, changing of water tables, etc.
- The change in excess pore pressures with time can be computed using a consolidation analysis. It incorporates simple linear, isotropic Mohr-Coulomb Soil model as well as more complex and non-linear models such as Hardening Soil model, Jointed Rock model, Soft-Soil-Creep model, Soft Soil model and Modified Cam-Clay model.

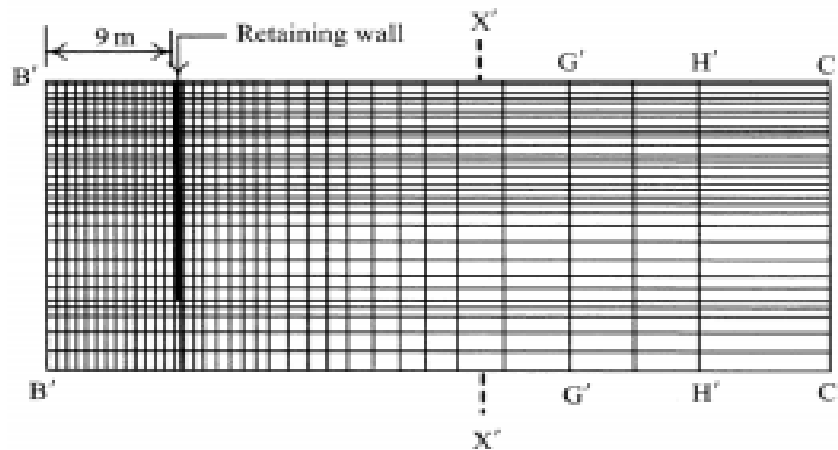
In major deep excavation projects, the Mohr-Coulomb (MC) model and the Hardening Soil (HS) model are mostly used to simulate soil behavior. The details of the Mohr-Coulomb model can be found in any geotechnical engineering text (e.g. Budhu 2000). A comprehensive description of the Hardening Soil model can be found in Schanz et al. (1999).

### **2.10.1. Basic Principles**

In this section the basic principles of the finite element method will be explained. Based on the Figure 15a, the central section in most cases is always chosen. Then take the profile of this section and divide the soils and structures within the excavation influence range into many meshes, each of which is called an element.



(a)



(b)

**Figure 15 Finite Element Analysis of an Excavation: (a) Plan and (b) Meshes of the Section A-A**

### 2.10.2. Plane Strain Elements

In terms of shape, plane strain elements are usually categorized into triangular elements and quadrilateral elements as shown in Figure 18. For Plaxis2D, there are two options available which

either to use 6 nodes or 15 nodes. Element with 15-nodes can provide more accurate calculation of stresses and failure loads. However, 6-node triangles can offer more quick calculation.

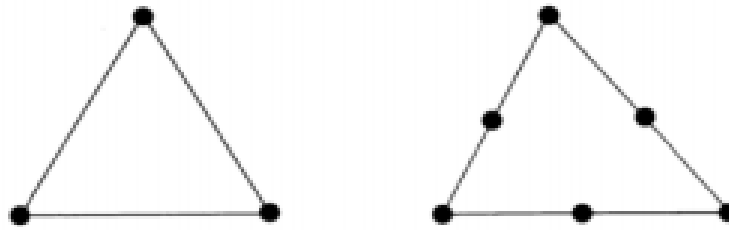


Figure 16 Nodes Element

### 2.10.3. Bar Elements

Bar elements in Plaxis2D can be used to simulate struts, anchors, or other members bearing only axial stress, as shown in Figure 17. Each node of a bar element has only one degree of freedom. A bar element of low order has two nodes while one of high order has three nodes. In general excavation analyses, two node bar elements can achieve good accuracy.

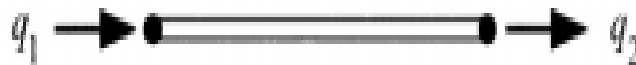


Figure 17 Two-Node Bar Element

### 2.10.4. Beam Elements

Members subjected to moment can be simulated by using beam elements, or also known as flexural elements, as shown in Figure 18. Each node of a beam element has two degrees of freedom. A beam element of low order has two nodes while one of high order has three nodes. In general excavation analyses, the two node beam element can achieve good accuracy.

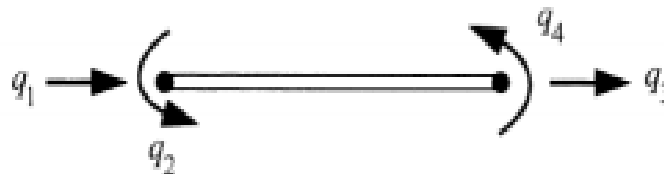
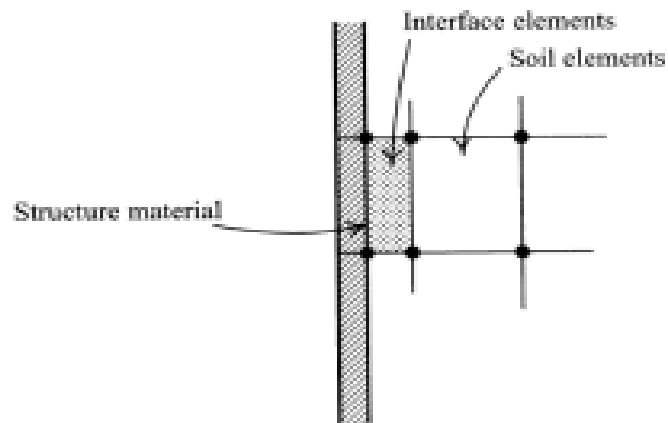


Figure 18 Two-Node Beam Element

### 2.10.5. Interface Elements

Retaining walls that used in excavation are stiff while the adjacent material which is soil is relatively soft. When the retaining wall deforms, relative displacement may be generated between the soil and the wall. To simulate the relative displacement between soil and structures during excavation, interface elements are sometimes used in analysis. As shown in Figure 19, an interface element is an element connecting structures and soil, with or without thickness, which has a quite large normal stiffness but relatively small shear stiffness so that it can simulate the relative displacement between soil and structures.

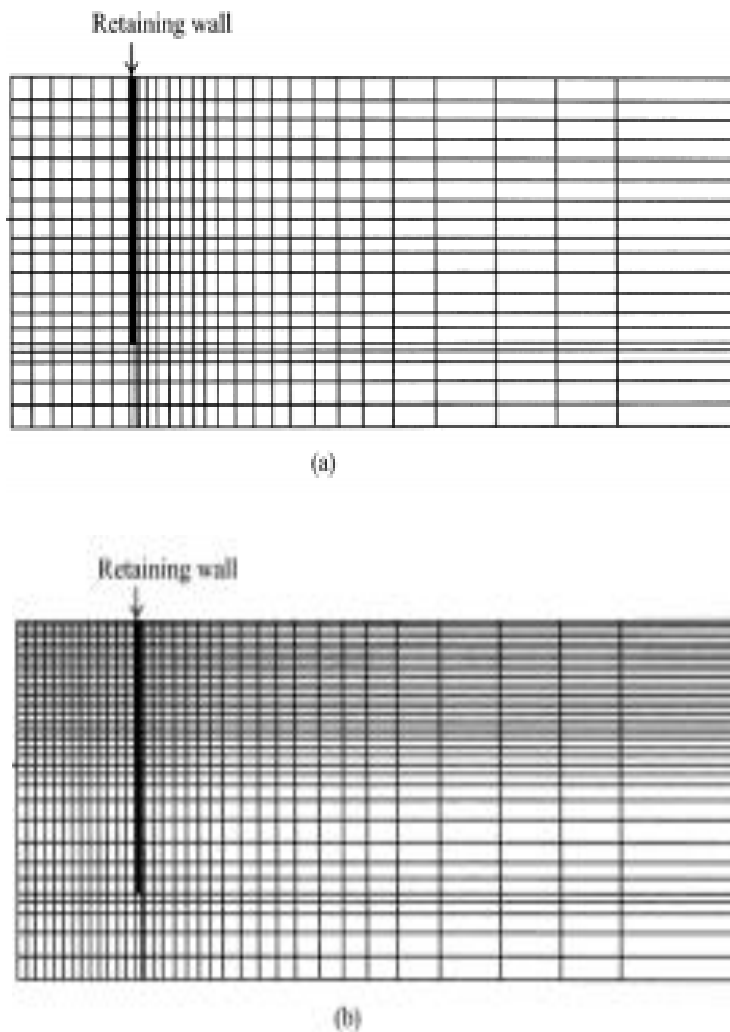
Though interface elements can rationally simulate the relative displacement between soil and structures, since extra parameters, which are not easily obtained from conventional soil tests, will be introduced, and numerical instability during analysis often occurs, they have to be used especially carefully. If interface elements are not being adopted, the soil in the vicinity of the structure can be considered to divide into fine elements. When the retaining wall is deformed, these fine soil elements can easily attain the plastic state, which will then produce larger deformation. Therefore, a rational analysis result is also attainable.



**Figure 19 Interface Element**

### 2.10.6. Mesh Generation

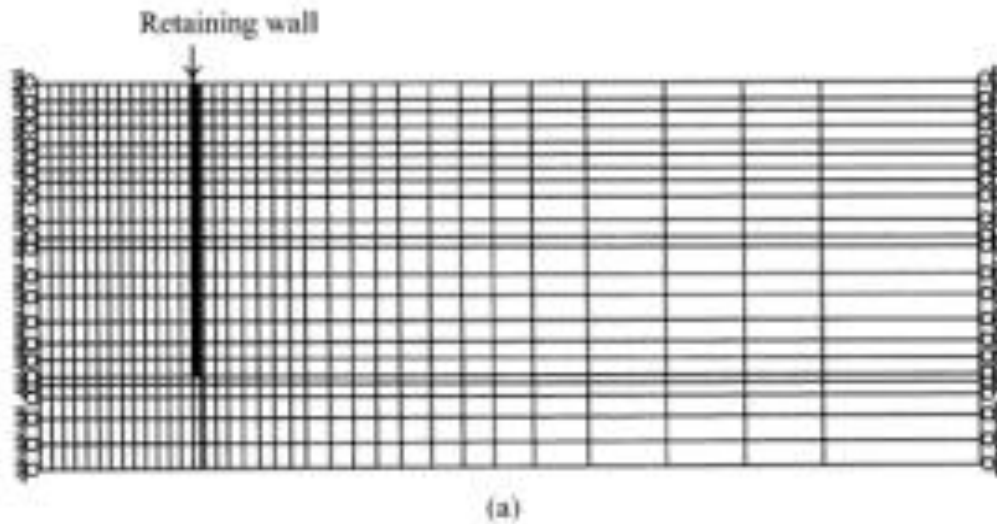
The shape of the element used in an analysis can strongly affect the results obtained. Density of mesh is one of the main factors that can affect the result. In deep excavation work, the mesh in the area of stress concentration, of rapid strain changing, the crucial areas, and the object zones should be finer. Density of mesh for soil closest to the retaining wall should be finer than the soil further away from the retaining wall.



**Figure 20 Finite Element Meshes: (a) Bad Mesh and (b) Good Mesh**

### 2.10.7. Boundary Condition

If considering the symmetry of an excavation and taking a half for analysis as shown in Figure 21, the symmetric boundary should be equipped with rollers to restrain the lateral displacement and allow vertical displacement. According to Ouand Shiau (1998), to analyze movement in an excavation, rollers will be more efficient than hinges placed.



**Figure 21 Boundary Condition: (a) Allowed With Rollers and (b) Allocated With Hinges**

## **2.11. Retaining Wall Construction Cost and Production Rates**

Fernando Pizarro (2014) forecast of the construction durations and costs result of key importance when planning, bidding, managing and controlling the construction projects. Although the scheduling and estimating knowledge and tools have evolved greatly in the last decades, avoidable social costs, delays and budget overruns are not uncommon in current construction. In order to develop an accurate set of unit costs and production rates phases, materials and equipment involved in the construction of retaining walls, it is necessary to investigate the availability of those factors.

The unit costs and production rates associated to the construction of any retaining wall is very complex in nature. Not only do these rates involve several different materials but also equipment, labor, subcontractors and overhead. Under some circumstances, three walls for a certain location will be ranked just by the comparison of their unit costs and production rates. The consequences of an improper estimation of project costs and durations can have not only economic implications but also social, political, security or even legal repercussions experiences and literature.

Several factors can have an impact on the production rates of the construction of each types of wall. These factors are categorized by:

- Uniqueness of the project
- Labor
- Varied location
- Dependence of the economy
- Weather and seasonality
- Risk of worker accidents
- Disruptions and material supply
- Traffic and accessibility
- Advancement in technology

## **2.12. Earlier Works**

Mana and Clough (1981) carried out numerical parametric study to investigate the effects of wall stiffness, prop spacing, prop stiffness, pre-stress and elastic soil stiffness on ground deformations due to excavation. It was revealed that ground deformations decreases with increased wall bending stiffness or decreased prop spacing. This effect is more significant when factor of safety against basal heave is relatively low. Ground deformations decrease with increased prop stiffness, with a decreasing rate at high stiffness. Moreover, ground deformations are also significantly affected by soil modulus. Higher modulus leads to smaller movement.

Mana and Clough (1981) conducted a two-dimensional numerical parametric study to investigate effects of excavation geometry such as excavation width and depth of the underlying firm layer on ground deformations due to excavation. It was revealed that ground deformations increase with excavation width and depth to an underlying firm layer.

Clough and O'Rourke (1990) conducted a series of finite element parametric studies on excavations in stiff clay. The computed results show that parameters such as wall stiffness and prop spacing have only a small influence on deformations around excavations in stiff clay. This is because the modeled soils are stiff enough to minimize the need for stiff retaining systems. It was found that soil modulus and coefficient of lateral earth pressure have a more significant impact on the ground movements, compared to stiffness of retaining systems. It was also found that base instability is usually not concern for excavations in stiff clays.

Hashash and Whittle (1996) carried out a series of two-dimensional numerical parametric studies to study effects of wall embedment depth and prop spacing on ground deformations due to multi propped excavations. Constitutive model adopted in the numerical analyses is capable of considering anisotropic stress-strain relationship, stress path dependency and strain dependency of clay. Computed results show that wall length has a minimal effect on the pre-failure deformation for excavations in deep layers of clay, but does have a major effect on the location of failure within the soil. Use of very depth wall can improve base stability. However, large bending moment can be resulted and may cause flexure failure of the retaining wall. Not only is basal affected by final excavation depth, but also influenced by vertical prop spacing. Larger vertical prop spacing can result in additional basal heave.

Ou et al. (1998) presented field performance of an excavation constructed by top-down method in Taipei soft clay. The field measurement include lateral total earth pressure and pore water pressure on both sides of the wall, prop load, lateral wall displacement, ground settlement behind the wall and basal heave inside excavation.

Comprehensive model tests by Tschebotarioff (1948) and Rowe (1952) led to the first quantitative evaluation of the effect of the wall flexibility on bending moment. Theoretical studies by Terzaghi (1953), has demonstrated that the maximum bending moment in anchored sheet pile retaining wall is dependent on the stiffness of the wall. Rowe (1957) performed approximately 900 small scale model tests on anchored sheet pile wall. He conducted two types of tests denoted as pressure test and flexibility test. From the flexibility test, Rowe established a relationship between the degree of sheet pile and reduction in bending moment.

Tewodros Fekadu (2010) made a research entitled Analysis and parametric study of deep excavation with diaphragm wall using finite element based software. He examined that the effect of the change in thickness of diaphragm wall is not as prominent as the change in soil type, depth of excavation and depth wall embedment. It is interesting to note that the maximum bending moment in the diaphragm wall increases but the horizontal displacement of the diaphragm wall decreases when the thickness of diaphragm wall is increased. Clearly, a thicker diaphragm wall is able to resist horizontal deformation better but at the expense of induced greater bending moment. In other words, a stiffer diaphragm wall also needs greater bending strength. It is also worth noting that the horizontal displacement of the diaphragm wall does not reduce appreciably but the maximum bending moment continues to increase when the thickness of the diaphragm wall is increased. This implies that it is not useful to increase the stiffness of the diaphragm wall beyond a certain maximum value.

## **III. Chapter Three: Modeling**

### **3.1. Overview**

This Thesis was conducted in order to obtain the suitable wall system through a comparative study of retaining wall in deep excavation and basement work. This study used the data obtained from site investigation works for the 4B+G+30 United Bank project. The case was selected because most of the data needed for the analysis such as soil profiles are available.

Analysis of walls was conducted to obtain the most suitable wall to be adopted at the site. The walls must have small deflections to minimize the ground surface settlement. They should also be cost efficient.

A finite element software was used to determine the different parameters that play an important role in the design of walls and parametric study is then conducted to establish the significance of each parameter.

### **3.2. Examples From Literatures**

#### **3.2.1. Optimization Of Retaining Wall In Deep Excavation For Basement Work**

Fahmi (2012) studied an actual site in Kuala Lumpur. The site is located in Kenny Hill formation which is basically a completely decomposed rock and generally has the consistency of a clayey SILT soil. Diaphragm wall and contiguous bored pile has been analyzed using Plaxis2D and as plain strain models. However, geometry conversion is needed for CBP so that it can be modelled as plain strain instead of asymmetry model. The Hardening soil model is found to be suitable for analysis after making comparison between Mohr Coulomb model which overestimates the shear strength of soil. Result of lateral wall deflection from both types of walls shows the relationship between system stiffness and deflection of wall. A higher stiffness wall will result in less deflection. Thus, the main factor to select the suitable wall is not the system stiffness. However it depends on working space for construction, cost, embedment length etc.

### **3.2.1.1. Lateral Earth Support System**

The permanent lateral earth support system comprising an 800mm-1400mm thick perimeter slurry wall and Contiguous wall. The tieback anchors were installed through steel sleeves cast into the slurry wall. Each anchor was inclined at 45° with minimum fixed anchor lengths of 6m in the bedrock. Horizontal spacing of the anchors ranged from 2.5m and each tendon comprised from 9 to 16 strands of 1.5cm diameter high tensile strength steel.

### **3.2.1.2. Finite Element Simulation**

A series of finite element simulations have been carried out to obtain better insight into the performance of the excavation support system for the selected site. The calculations have been carried out using the PLAXIS finite element code (2002), plane strain models. Each of the soil layers has been simulated using the Mohr Coulomb model as it enables a realistic description of the stiffness of the retained soil relative to the excavated material with minimal additional parameters. The perimeter slurry wall was modeled using elastic beam elements (with axial and bending stiffnesses;  $EA = 2.52 \times 10^7 \text{ kN/m}$  and  $EI = 1.7 \times 10^6 \text{ kNm}^2/\text{m}$ , respectively), while elastic properties and prestress loads for the rock anchors are  $EA = 1.0 \times 10^5 \text{ kN/m}$

### **3.2.1.3. Computation Results**

Various sizes of wall diameter for bored pile wall and thickness for diaphragm wall have been adopted for this project in order to see the effect of wall stiffness towards the lateral deflection of the wall. In theory, the deformation of a retaining wall/basement wall with the increase of its stiffness will decrease. Stiffness of the wall is depending on the thickness and diameter of the wall.

As it can be seen from figure 25, the lateral wall deflection predicted by PLAXIS 2D matches the theory. Figure 25 shows the relationship between flexural stiffness of wall (EI) and lateral deflection of wall. The plot clearly shows us the trend to decrease in wall deflection as the wall stiffness increases.

Figure 26 shows the distribution of bending moment of different wall stiffness. The magnitude of maximum bending moments indicates that, increasing of wall stiffness (wall size) will results in

higher bending moment. Thus, the increase of wall stiffness to reduce wall deformation is certainly effective, but only to a certain extent. Hence, to decrease the deformation by way of increasing the thickness of the retaining wall will not be very effective due to number of reinforcement needed to cater the bending moments induced on the wall.

Figure 24 can be used to show the results of wall deflection in another way. CBP and DW of different sizes have shown the same trend in wall deflection when increasing the wall size. The deflection is significantly reduced from small size of wall towards bigger size of wall. However, the deflection is no more significant when the size of the wall is big enough.

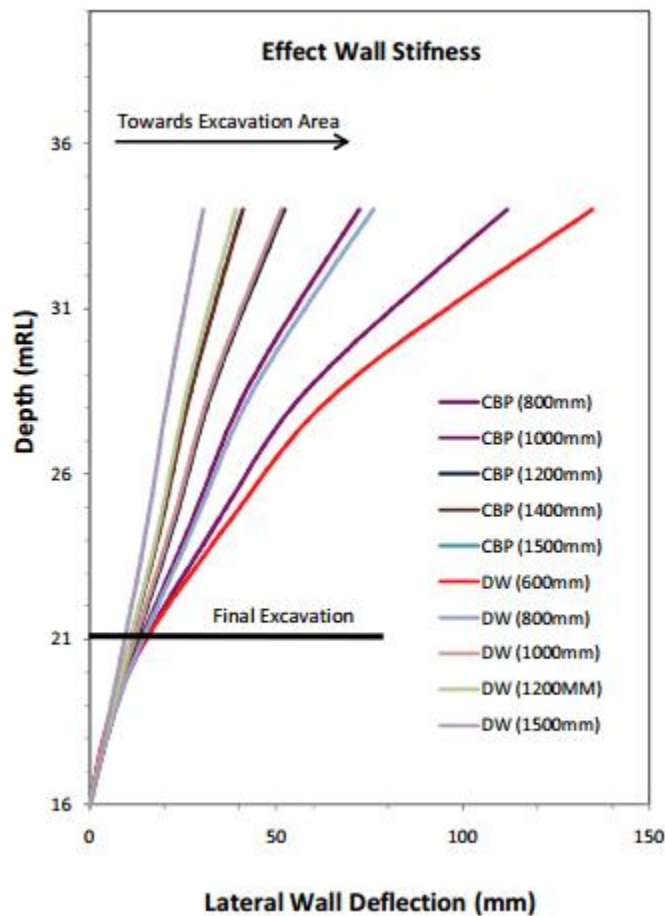


Figure 22 Effect of different wall sizes towards deflection of the wall

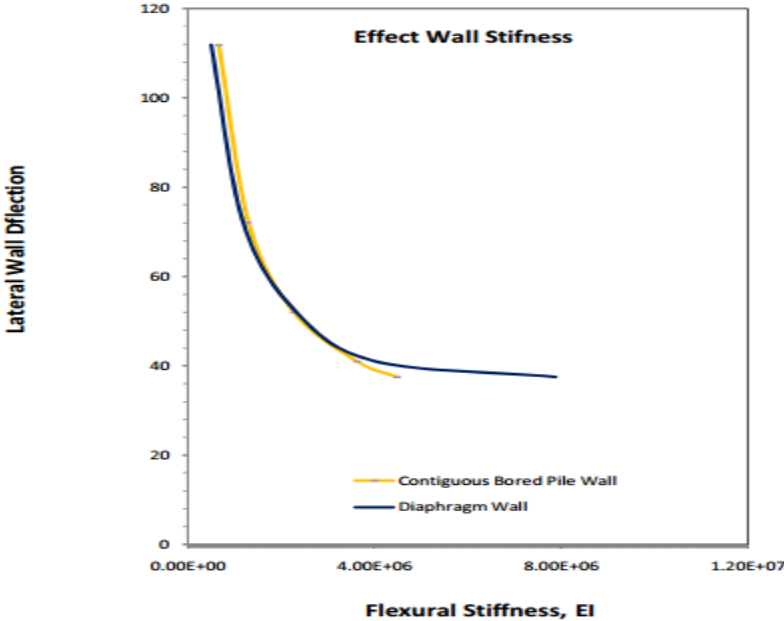


Figure 23 Effect of different wall sizes towards deflection of the wall

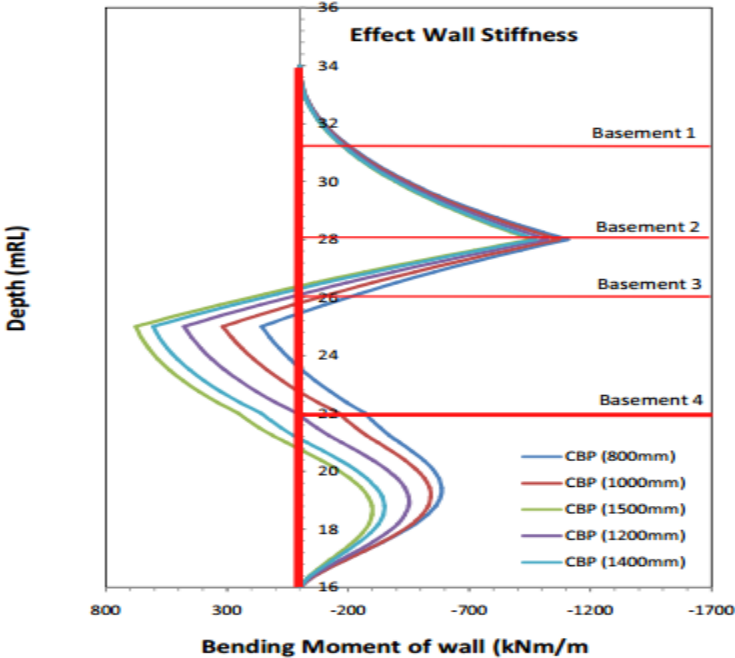


Figure 24 Effect of different wall sizes towards deflection of the wall

### **3.3. Data Collection**

#### **3.3.1. Ground Conditions**

##### **3.3.1.1. Interpreted Sub Soil and Geological Profile**

Topographically, the site is undulating and partly flat with an elevation difference of about 4m. The highest points are around BH7 and BH8. The surface undulations are the result of demolished garbage materials. Adjacent to the north eastern boundary of the site is Biftu building (10 story building). Precautions should be taken while basement excavation begins in order not to disturb the stability of the aforementioned buildings.

A total of eight boreholes were drilled to explore the subsurface geology of the project site. Borehole BH2 & BH5 were drilled to a depth of 50m. Borehole BH1, BH3 & BH4 were drilled to a depth of 45m and the rest of boreholes (BH6, BH7 & BH8) were drilled to 30m depth. The drilling result showed that the project site is constituted by highly weathered basalt intercalated with over consolidated clay.

The various geotechnical units that make up the project site can be treated as five major geological layers, described below.

#### **3.3.2. Description of the Geological Layers**

##### **Layer 1**

This is the top firm, expansive clay layer which original extends to an average depth of 3.6m below the natural ground level and has an average thickness of 1.5m considering an average of 2.25m fill material overlying the expansive clay layer. The standard penetration test (SPT) performed gave N-value of 8 to 11 blows for 30cm penetration. The average N-value of the layer is 10. It is surficial covered by about 1.2m to 3.3m thick fill and demolished materials.

##### **Layer 2**

This is a thick dense to very dense, pinkish to grayish sandy gravel layer with occasional rock core. It is derived from weathering of both aphanitic and vesicular basalts. The layer is found within average depth interval of 3.6m to 19.50m below the ground level and has an average thickness of 16m. The standard penetration test conducted in the layer resulted in a range of 24 to greater than 50 blows for 30cm depth penetration. The average N-value is 48 blows for 30cm penetration.

### **Layer 3**

This very stiff to hard, consolidated silty clay soil layer found below the top highly weathered rock layer and averagely extends from 19.50m to 28.50m below the ground level and has an average thickness of 9m. It is very hard soil layer with an average standard penetration N- value of 45 blows for 30cm depth penetration. The scatter N-value of the unit ranges from 19 to greater than 50 blows for 30cm depth penetration. This layer is a competent soil layer found underneath the basement floor level.it is underlain by very dense highly weathered basalt (sandy gravel material). It doesn't appear in BH7 till the end of drilling (30m beneath the natural ground).

### **Layer 4**

This is the second highly weathered rock (sandy gravel) layer with very stiff reddish silty clay interbedded. It is found within an average depth interval of 28.50m to 41.80m below the ground level and has an average thickness of 13m. The standard penetration test conducted inside the layer is in the range of 39 to greater than 50 blows for 30cm depth penetration and the average N-value is 48.

### **Layer 5**

This is moderately weathered, intensively fractured and fragmented rock layer found at variable depth intervals.in boreholes BH2 & BH5,it is encountered beneath layer 4, where as in BH7 & BH8, it appears overlying the hard reddish brown silty clay layer. In general, the layer is weak to medium strong rock with very poor RQD value (0 to 19%). The range of the SPT value is generally from 19 to greater than 50 and the average N-value is 45.

### **3.3.3. Ground Water Profile**

During the soil investigation field works, water level was monitored daily in order to examine the groundwater profile. Static water level measurement was carried out in all boreholes and it was observed in all boreholes at a relatively shallow depth (6.00m in BH2 to 9.90m in BH5). The static water level in the boreholes was recorded daily and is shown on the log sheets.

In order to monitor the groundwater level fluctuation of the project site, standpipe piezometer has been installed in borehole BH1. The standpipe consists of a perforated or slotted plastic pipe inserted into a borehole with suitable granular packing or filter material to prevent fine grains of soil from blocking the pipe openings. Water level measured by a dip meter. This usually consists

of twin electric cable to take measurements of the site static groundwater level over longer periods of time.

### **3.3.4. Soil Strength**

The soil strength was determined through in-situ SPT 'N' values with depth below existing ground level. Details of soil strength are shown above.

The soil strength/relative density was determined from laboratory tests on undisturbed samples together with empirically related to the SPT-N blow counts and hence, geotechnical designs can be provided.

### 3.3.5. Design Parameters

For geotechnical design, the design parameters are correlated and established. Based on the soil investigation carried out, the design parameters recommended for the designs are tabulated.

**Table 1 Design soil parameters**

Parameter	Symbol	Unit	Soil Description				
			Layer-1	Layer-2	Layer-3	Layer-4	Layer-5
Bulk Density	$\gamma_b$	kN/m <sup>3</sup>	15.4	16.95	18.41	19.5	20.84
Apparent cohesion	$c'$	kN/m <sup>2</sup>	21.4	15.5	23.83	20.26	22
Angle of friction	$\phi'$	Degree	22.1	24.89	23.95	31.5	35
Poisson's Ratio	$\nu'$	-	0.35	0.35	0.3	0.3	0.4
SPT-N	N	blows/30 cm	8-11	24-50	19-50	30-50	19-50
Young's modulus	$E'$	kN/m <sup>2</sup>	12,500	30,000	40,000	60,000	85,000
Depth	D	m	3.6	3.6-19.5	19.5-28.5	28.5-41.5	>41.5
Moisture content	$W$	-	0.3	0.3023	0.232	0.285	0.34
Initial void ratio	$e_o$	-	0.8	0.82	0.961	0.897	0.883
Specific gravity	$G_s$	-	2.5	2.4	2.55	2.6	2.64
Saturated unit weight	$\gamma_{sat}$	kN/m <sup>3</sup>	17.31	17.36	17.56	18.08	18.41

### 3.3.6. Boundary Conditions

The region to be analyzed in deep excavations is of relatively small dimensions compared with those of the surrounding medium. In finite element analysis, the normal practice is to extend the mesh to some distance away from the zone of interest, and apply fixed displacement boundary conditions there.

### 3.4. Anchor Properties

Three anchors and two strands will be used as an anchoring system. The data for anchor design must be derived from geo mechanical analysis of the shear characteristics of the soil and the strength of the tendon. Bauer’s structural engineering have many years of experience in design and can also draw on the results of numerous anchor load tests to aid in correct dimensioning.

In addition, the optimum number, distance length and angle of anchor for any individual site can be determined by continuously updated software packages

**Table 2 Recommended Maximum Allowable Anchor Forces**

<b>Maximum allowable anchor forces</b>					
Anchor type	Steel quality	Steel diameter (mm)	Allowable anchor force acc. DIN 4125		Elastic stretch per kN and m of free length (mm/kNxm)
	(N/mm <sup>2</sup> )		Normal earth pressure (kN)	At rest (kN)	
Strand anchor	St 1570/ 1770	2 x 0,6"	251	330	0,018315
		3 x 0,6"	377	496	0,012210
		4 x 0,6"	502	661	0,009158
		5 x 0,6"	628	826	0,007326
		6 x 0,6"	754	992	0,006105
		7 x 0,6"	879	1 157	0,005233
		8 x 0,6"	1 005	1 300	0,004579
		9 x 0,6"	1 130	-	0,004070
		10 x 0,6"	1 256	-	0,003663
		11 x 0,6"	1 300	-	0,003330
Single bar anchor	St 835/ 1030	32	384	473	0,00604
		36	485	599	0,00477
	St 1080/ 1230	26,5	340	388	0,00880
		32	496	563	0,00604
		36	628	723	0,00477

Steel diameter = 2\*0.6mm

Allowable anchor force DIN 4125

At normal earth pressure = 251 KN

At rest earth pressure = 330 KN

Elastic strength per KN and m of free length = 0.018315 mm/kNm

Total length of anchor = 13m

Fixed length = 5m

Free length = 8m

Anchor angle = 45°

### 3.5. Wall Properties

In PLAXIS 2D, real situation can be modelled either by a plane strain or an axisymmetric model. In order to predict the field behavior of contiguous and tangent walls, the plane strain idealization of continuous wall is reasonable for the purpose of comparative study with diaphragm wall and sheet pile walls which has been modelled as plane strain.

It is required to idealize the equivalent of plane strain for CBP and TBP so that Plaxis2D analysis will be more accurate and represent the actual site condition. Based on the table below, equivalent thickness has been reduced to account for the spacing of the piles.

#### 3.5.1. Contiguous Bored Pile

**Table 3 Contiguous bored pile wall parameter**

<b>d(mm)</b>	<b>Equivalent Thickness</b>	<b><math>f_{cu}</math> (N/mm<sup>2</sup>)</b>	<b>E (KN/mm<sup>2</sup>)</b>	<b>A (mm<sup>2</sup>)</b>	<b>Bending Stiffness, EA (KN/m)</b>	<b>I (mm<sup>4</sup>)</b>	<b>Flexural Stiffness, EI (KNmm<sup>2</sup>/m)</b>
800	511.68	40	30	511,680	15,350,400	11.6*10 <sup>9</sup>	3.48*10 <sup>11</sup>
1000	665.222	40	30	665,222	19,956,660	24.5*10 <sup>9</sup>	7.35*10 <sup>11</sup>
1200	821.76	40	30	821,760	24,652,800	46.24*10 <sup>9</sup>	1.39*10 <sup>12</sup>
1400	980.417	40	30	980,417	29,412,510	78.53*10 <sup>9</sup>	2.36*10 <sup>12</sup>

### 3.5.2. Tangent Bored Pile

**Table 4 Tangent bored pile wall parameters**

<b>d(mm)</b>	<b>Equivalent Thickness</b>	<b>fcu(N/mm<sup>2</sup>)</b>	<b>E(KN/mm<sup>2</sup>)</b>	<b>A(mm<sup>2</sup>)</b>	<b>Bending Stiffness, EA(KN/m)</b>	<b>I(mm<sup>4</sup>)</b>	<b>Flexural Stiffness, EI(KNmm<sup>2</sup>/m)</b>
800	670.5	40	30	670500	20115000	25.12*10 <sup>9</sup>	75.36*10 <sup>10</sup>
1000	838.1	40	30	838100	25143000	49.06*10 <sup>9</sup>	14.718*10 <sup>11</sup>
1200	1005.7	40	30	1005700	30171000	84.78*10 <sup>9</sup>	25.43*10 <sup>11</sup>
1400	1173.4	40	30	1173400	35202000	13.46*10 <sup>10</sup>	40.38*10 <sup>11</sup>

### 3.5.3. Diaphragm Wall

**Table 5 Diaphragm wall parameters**

<b>t(mm)</b>	<b>length(mm)</b>	<b>fcu(N/mm<sup>2</sup>)</b>	<b>E(KN/mm<sup>2</sup>)</b>	<b>A(mm<sup>2</sup>)</b>	<b>Bending Stiffness, EA(KN/m)</b>	<b>I(mm<sup>4</sup>)</b>	<b>Flexural Stiffness, EI(KNmm<sup>2</sup>/m)</b>
800	1000	40	30	800000	24000000	4.3*10 <sup>10</sup>	1.28*10 <sup>12</sup>
1000	1000	40	30	1000000	30000000	8.3*10 <sup>10</sup>	2.5*10 <sup>12</sup>
1200	1000	40	30	1200000	36000000	1.44*10 <sup>11</sup>	4.32*10 <sup>12</sup>
1400	1000	40	30	1400000	42000000	2.29*10 <sup>11</sup>	6.86*10 <sup>12</sup>

### 3.5.4. Surcharge

The surcharge value should take into account the site conditions and control at site. Site conditions such as loadings from adjacent buildings; vehicles, services, etc. should be taken into consideration in the design. It is prudent to incorporate a minimum surcharge of 10kPa to cater for construction loads and unforeseen circumstances.

### **3.6. Proposed Scheme**

Based on the selection processes outlined in the preceding sections, diaphragm wall, sheet pile wall, tangent bored wall and contiguous bored pile wall are the most feasible options to be adopted for the project site.

The anticipated excavation for construction of deep basement is approximately 22m below ground. With the proposed scheme, wall is designed to be embedded in the firm to stiff layer. As highlighted earlier, the need for adequate support, embedment and wall stiffness will govern the wall design, as the proposed structure is In order to make a comparative study and to find the most feasible option, the followings are envisaged to be adopted: -

- Diaphragm wall thickness of 600mm to 1500mm, and typically, the steel reinforcement between 0.5% to 2%, and
- Contiguous and tangent bored pile wall piles of diameter 800mm to 1500mm may be adopted; Steel reinforcement may be as per the diaphragm wall.
- The anchorage system will have 3 anchors and each anchor have 2\*0.6 cable stands.

### **3.7. Base Model Generation and Construction Sequence**

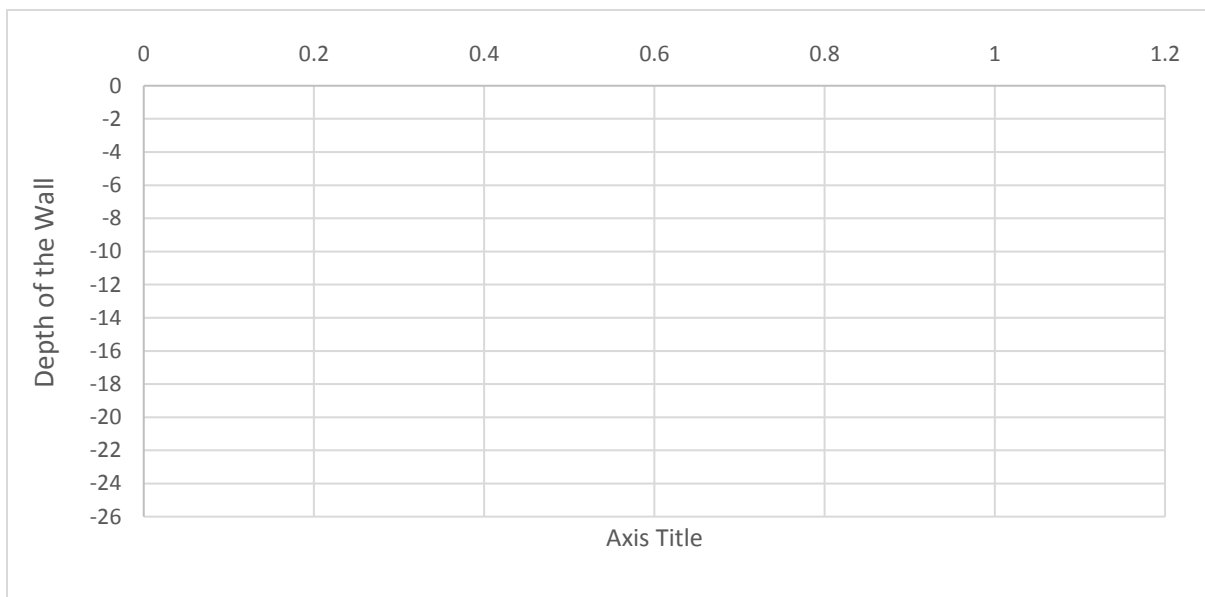
Typical base model geometry and general dimensions for the study is shown in Figure. The model is simulated as a symmetrical plain strain finite element employing 15-noded triangular elements and half of the geometry with a width of 100 meters and depth of 50 meters.

**Table 6 Construction sequence**

Stage No.	Construction Activity
0	Initial condition
1	10 kPa loading
2	Construction of the wall
3	Excavate to -3.00 depth
4	Excavate to -8.00 depth
6	Excavate to -13.00 depth
7	Excavate to -18.00 depth
8	Excavate to -22.00 depth

### 3.8. Justification for all PLAXIS figure output results

The figures below are made for the mere purpose of illustrating the conversation of the coordinate axis of the PLAXIS figure output results into a more conventional type of coordinate axis for the intent visualizing the PLAXIS figure out put results.



**Figure 25 Sample co-ordinate axis for all PLAXIS figure output results (4m embedment depth)**

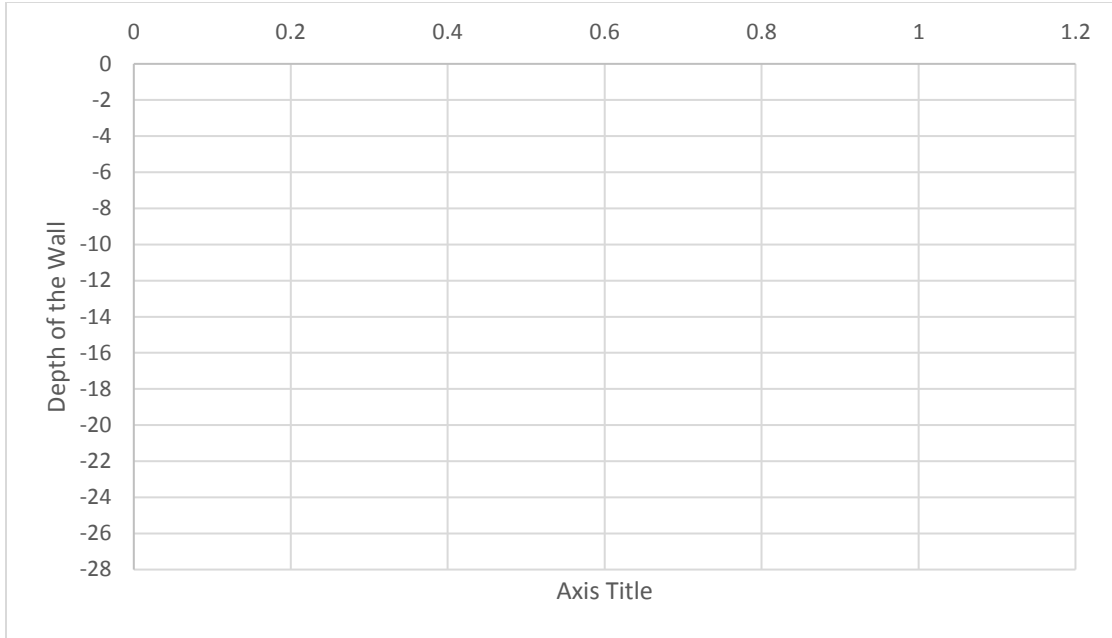


Figure 26 Sample co-ordinate axis for all PLAXIS figure output results (6m embedment depth)

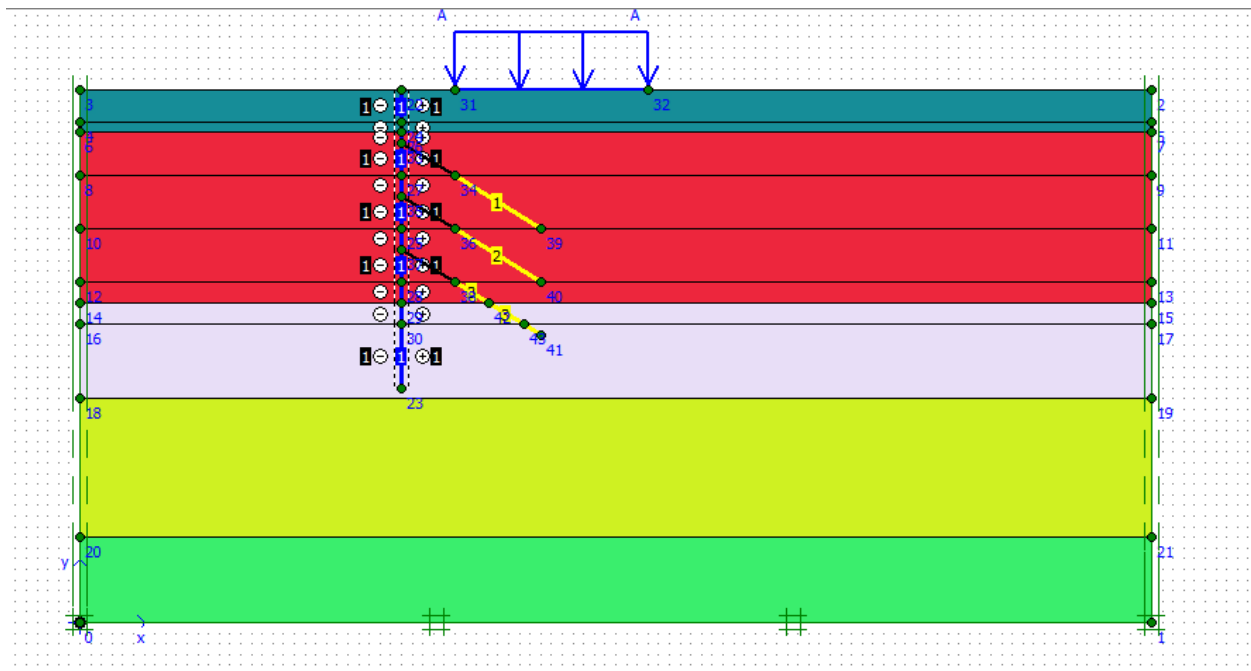


Figure 27 FE Base Model

## **IV. Chapter Four: Parametric Study**

### **4.1. Results and Discussion**

#### **4.1.1. Effect of Wall Stiffness**

Various sizes of wall diameter (800mm-1400mm) for bored pile walls and thickness for retaining structures have been adopted for this paper in order to see the effect of wall stiffness on the lateral wall deflection. In theory, the deformation of a retaining wall/basement wall with the increase of its stiffness will decrease. Stiffness of the wall is depends on the thickness or diameter of the wall.

In this part of the analysis, the paper concentrates on how change in stiffness of Contiguous bored pile, Tangent bored pile and diaphragm wall affects the performance of deep excavation for 4m embedment length. The output from this analysis is presented below.

##### **4.1.1.1. Shear Force**

For Diaphragm walls, a significant increase in the shear force is observed when the wall thickness of the wall is increased from 800mm to 1400mm. The shear force value become 235 KN when the thickness of the wall is 800mm. when the thickness becomes 1000mm it increased from 235 kN to 280.76 kN which is a 19.47% increase. Similarly, the shear force showed an increase of 26.8% and 31.74% increase in the 1200mm and 1400mm diaphragm walls respectively.

The Tangent bored pile walls also showed the same trend of shear force increase as the thickness of the wall is increased from 800mm to 1400mm. As the thickness of the wall increased, the shear force increased from 229.43 kN to 259.34 kN which implies an increment of 13.03%. Similarly, the shear force increased by 22.74% and 29.31% for the 1200mm and 1400mm tangent bored pile walls respectively.

As it can be seen from figure 29, the shear force in the Contiguous bored piles increases for thickness of the wall increased from 800mm to 1400mm for each embedment length. For increasing of thickness from 800mm to 1000mm, the shear force increased from 195.31 kN to

227.81 kN which is a 16.64% increase. Similarly, the shear force showed an increase of 31.43% and 42.65% increase in the 1200mm and 1400mm contiguous bored pile walls respectively.

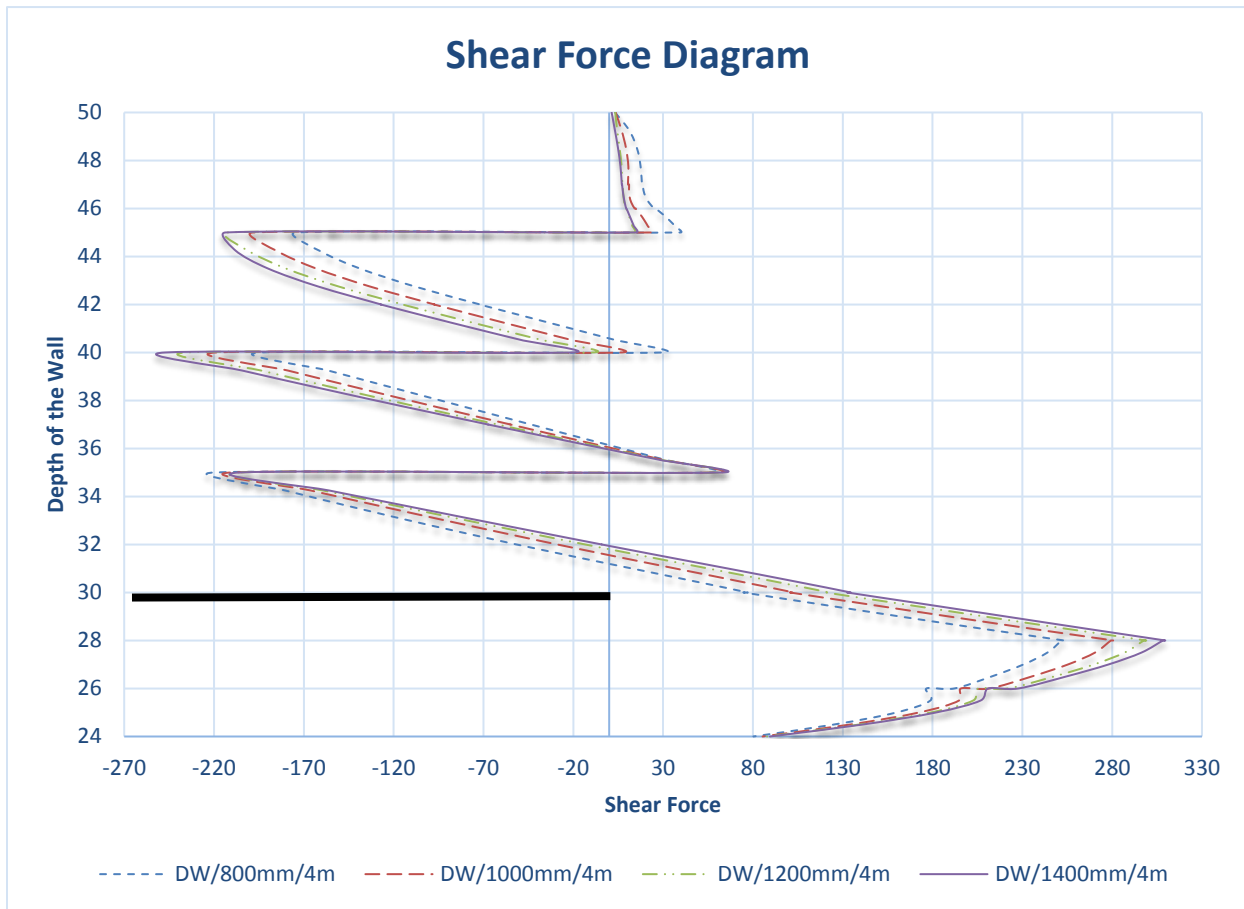


Figure 28 Shear Force Diagram of Diaphragm Wall for 4m Embedment Length

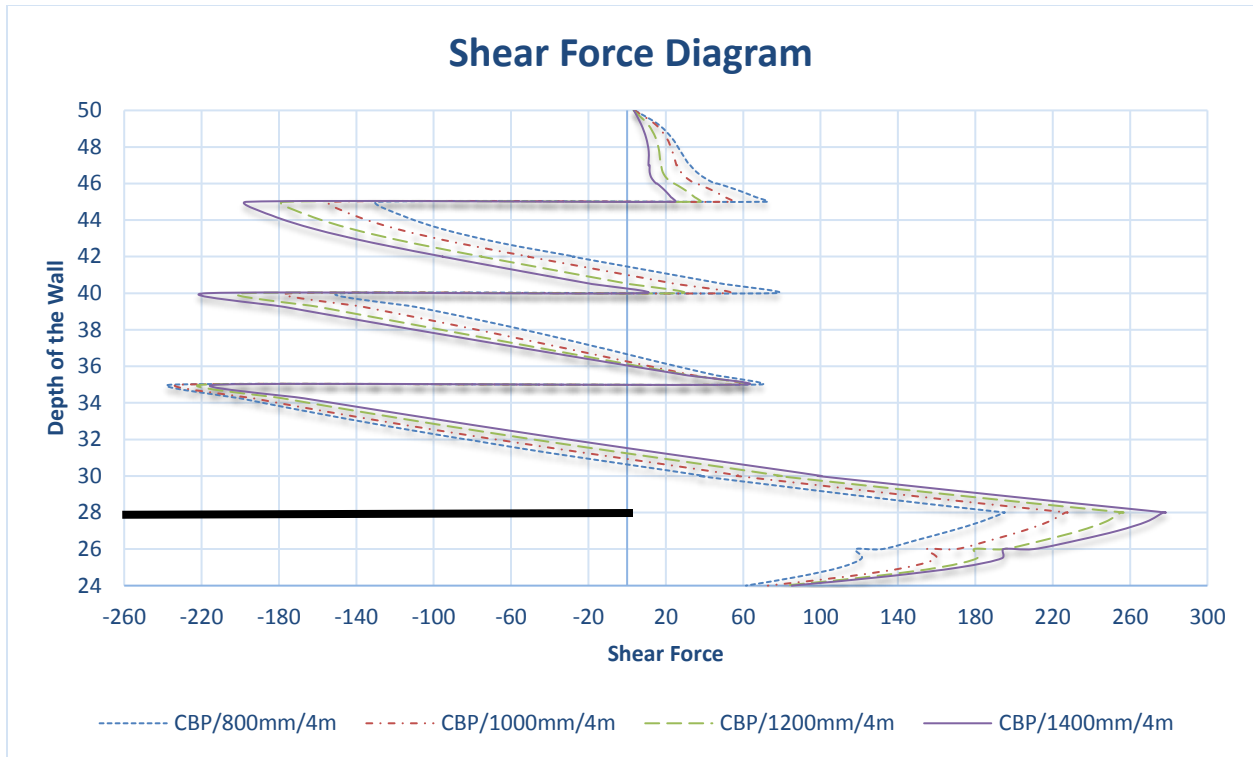


Figure 29 Shear Force Diagram of Contiguous Bored Pile Wall for 4m Embedment Length

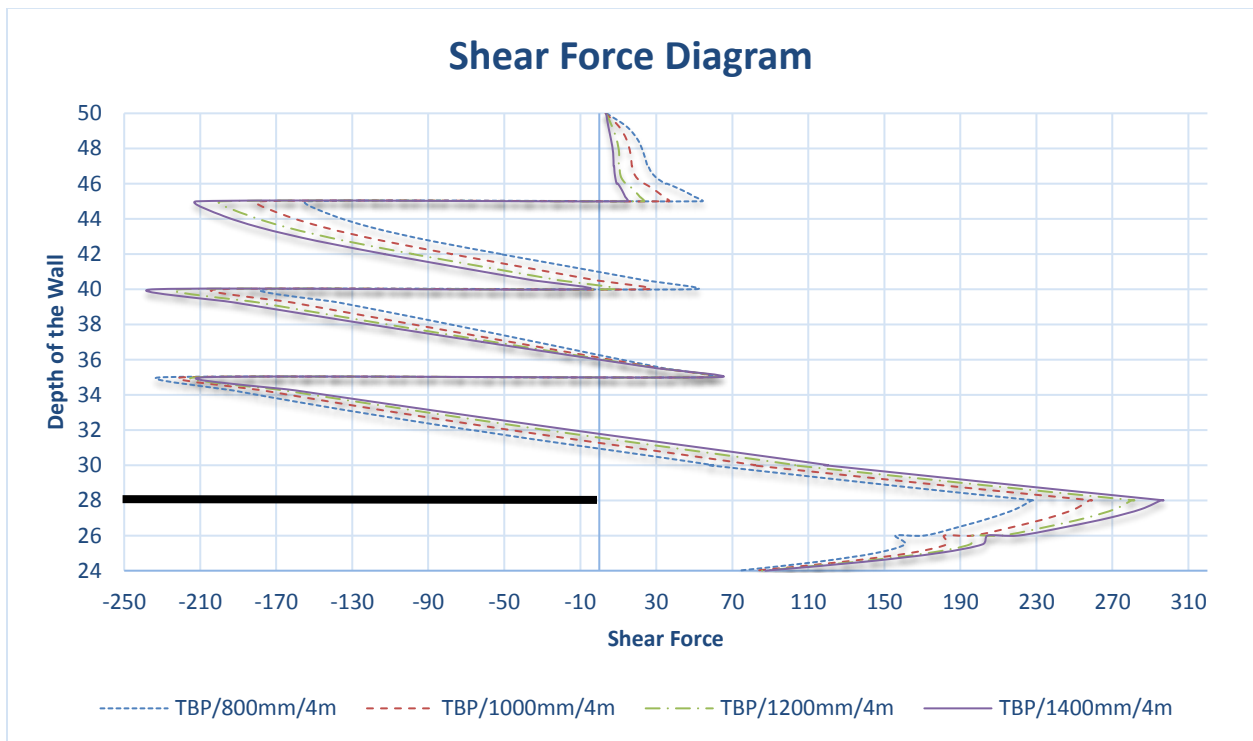


Figure 30 Shear Force Diagram of Tangent Bored Pile Wall for 4m Embedment Length

#### **4.1.1.2. Bending Moment**

The maximum bending moment in the walls also increases in response to increase stiffness of the wall in the same manner as the shear force. For the change of thickness from 800mm to 1000, 1200mm and 1400mm of diaphragm wall, it is shown that a 14.77%, 24.56%, and 30.78% increment in maximum bending moment from the values for 4m embedment depth.

Figure 32 shows the maximum bending moment in the contiguous bored piles. It can be seen that the maximum bending moment increased by 25.27%, 46.15%, and 62.82% as the thickness of the wall was increased from 800mm to 1000mm, 1200mm and 1400mm contiguous bored pile walls respectively for 4m embedment depth.

The tangent bored pile walls also showed the same trend of maximum bending moment increase as the thickness of the wall is increased. As the wall thickness increased from 800mm, 1000mm, 1200mm and 1400mm, the maximum bending moment increased by 17.26%, 30.4% and 39.5% for the 800mm to 1000mm, 1200mm and 1400mm tangent bored pile walls respectively for 4m embedment depth.

The magnitude of maximum bending moments indicates that, increasing of wall stiffness or in other words, an increase in wall size results in higher bending moment. Thus, the increase of wall stiffness to reduce wall deformation is certainly effective, but only to a certain extent. Hence, to decrease the deformation by way of increasing the thickness of the retaining wall will not be very effective due to number of reinforcement needed to accommodate the bending moments induced on the wall.

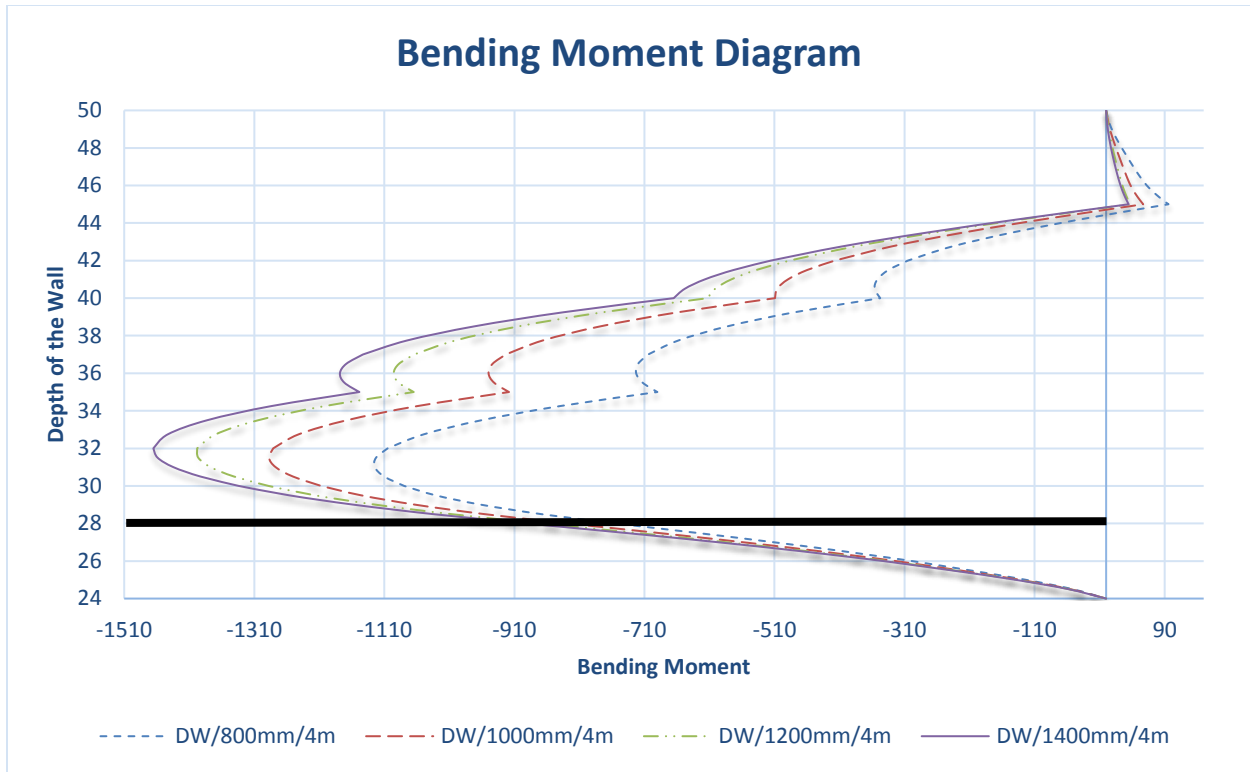


Figure 31 Bending Moment Diagram of Diaphragm Wall for 4m Embedment Length

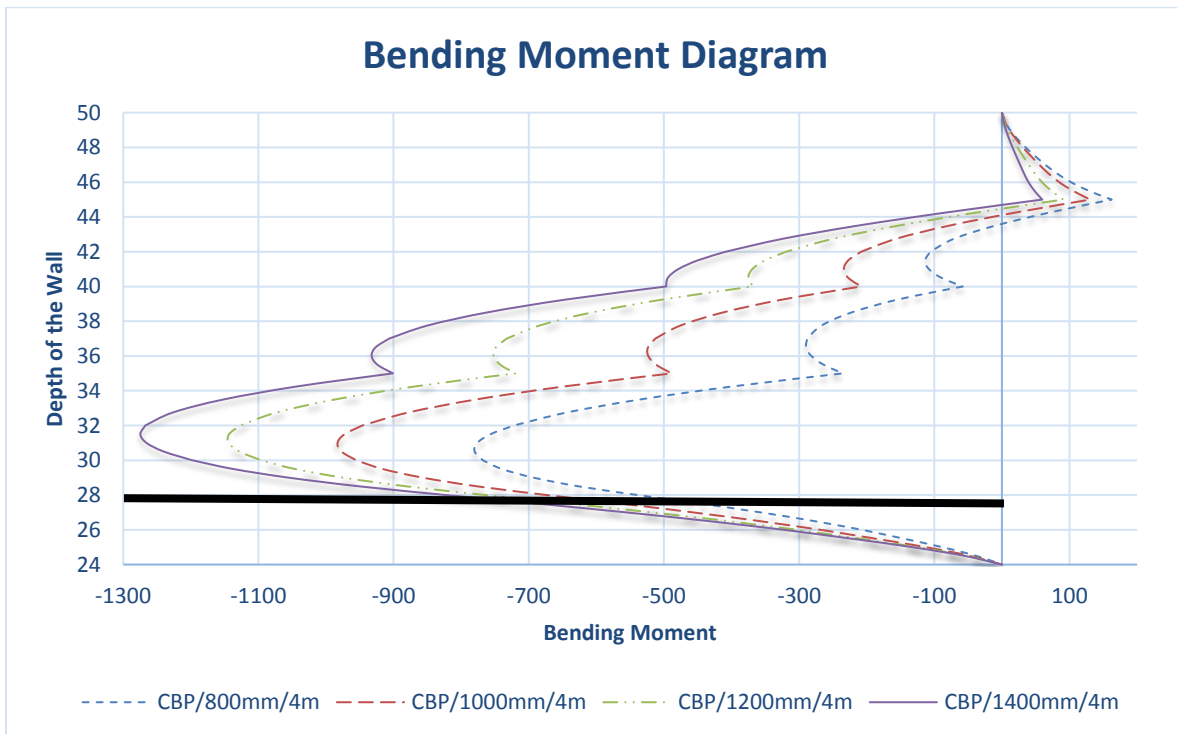
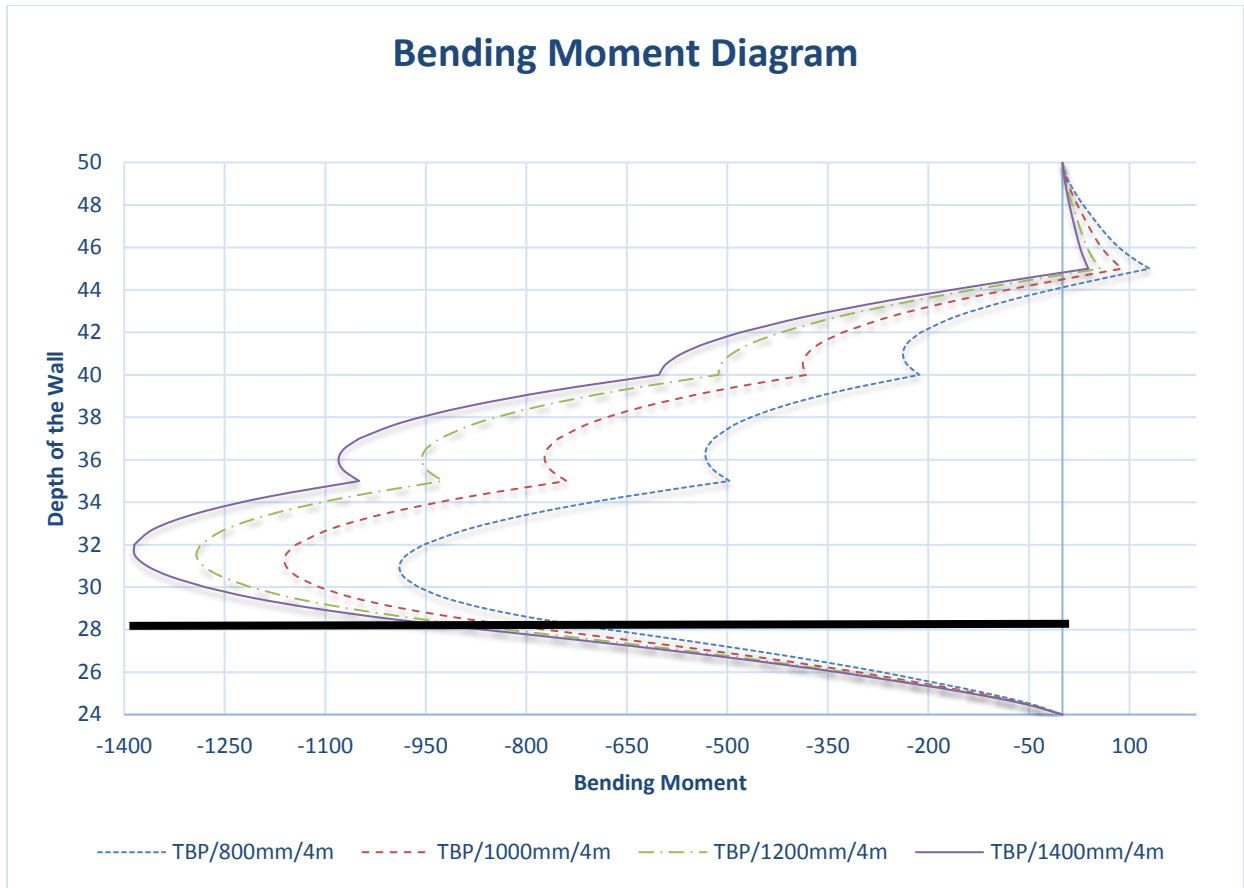


Figure 32 Bending Moment Diagram of Contiguous Bored Pile Wall for 4m Embedment Length



**Figure 33 Bending Moment Diagram of Tangent Bored Pile Wall for 4m Embedment Length**

#### **4.1.1.3. Horizontal Displacement of the Wall**

The horizontal displacement of the wall increased as the thickness of walls was increased from 800mm to 1400mm. The increment of wall thickness of the diaphragm wall from 800mm to 1000mm, 1200mm and 1400mm resulted in an increase of 5.46%, 9.56%, and 11.95% of lateral wall respectively for 4m embedment depth but it gets more straighten behavior as a result of increasingly of its rigidity. However, this works the depth of anchor provision.

Similar to the diaphragm walls, the horizontal displacement in the contiguous bored piles shows an increase in response to the increase of wall thickness from 800mm to 1400mm to the depth of the first anchor. It can be seen in figure 35 that horizontal displacement increased by 8.23%,

15.29%, and 20.78% as the thickness of wall was increased from 800mm to 1000mm, 1200mm and 1400mm contiguous bored pile walls respectively.

The behavior of increased horizontal displacement is also the same for the tangent bored pile walls. As the thickness of the wall is increased from 800mm to 1000mm, 1200mm and 1400mm, their horizontal displacement increased by 6.86%, 11.91% and 15.52% respectively. However, the deflection is no more significant when the size of the wall is big enough.

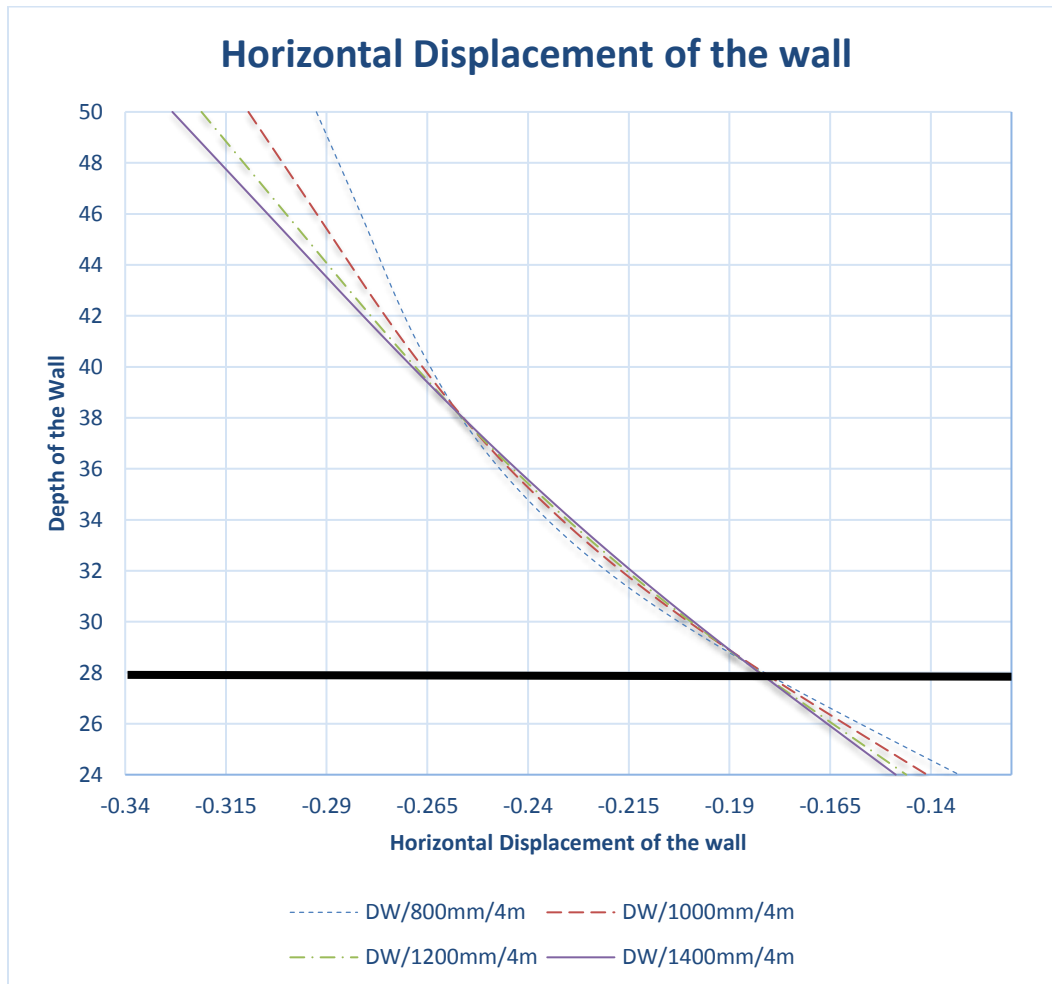


Figure 34 Horizontal Displacement Diagram of Diaphragm Wall for 4m Embedment Length

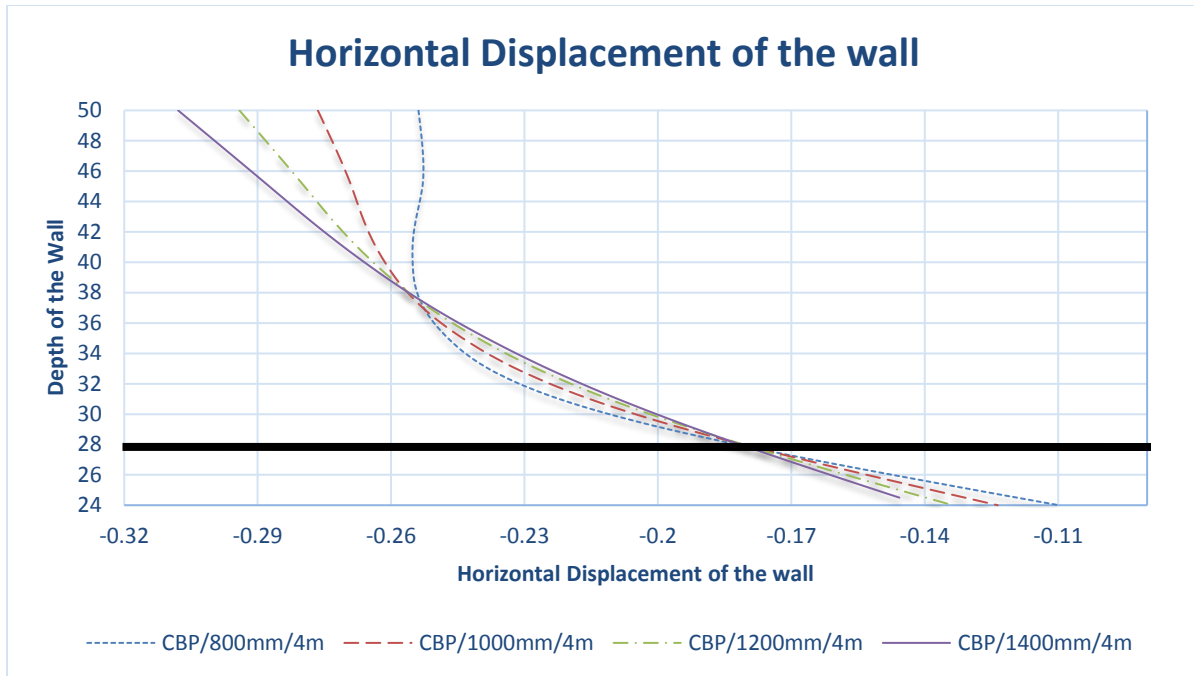


Figure 35 Horizontal Displacement Diagram of Contiguous Bored Pile Wall for 4m Embedment Length

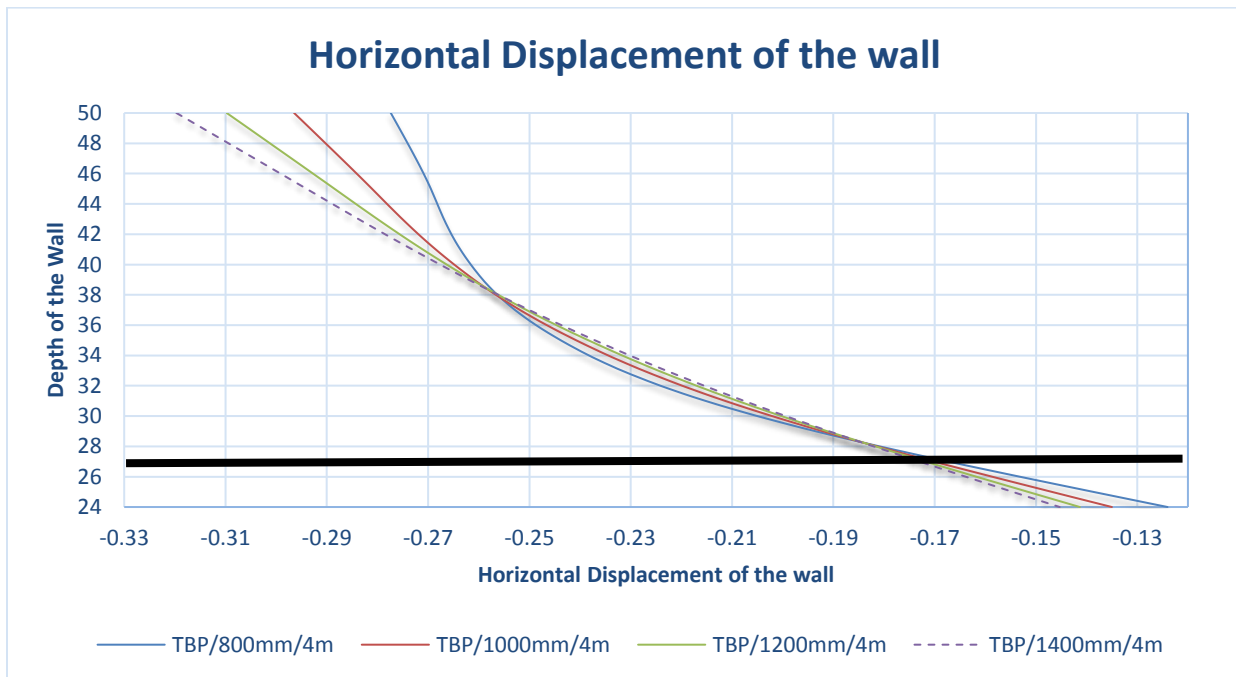


Figure 36 horizontal displacement Diagram of Tangent Bored Pile Wall for 4m Embedment Length

#### 4.1.1.4. Ground Settlement

The horizontal displacement of the wall and the ground settlement behind retaining structures shows contrasting fashions with the change in thickness of the retaining structures. The horizontal displacement of the retaining structures decrease but the ground settlement behind the retaining structures increase as the thicknesses of the retaining structures become increased. In case of stiffer retaining structures, the ground is prevented from spreading horizontally. The deformation of the ground, therefore, occurs mainly in the vertical direction, resulting in increase in ground settlement behind the retaining structures. In other words, the consolidation of the ground behind stiff retaining structures is mainly one dimensional. For a relatively flexible retaining structures (smaller thickness), there is increase in horizontal displacement and decrease in ground settlement. In appendix 1, additional figures that show effect of change in diaphragm wall on the performance of deep excavation is presented.

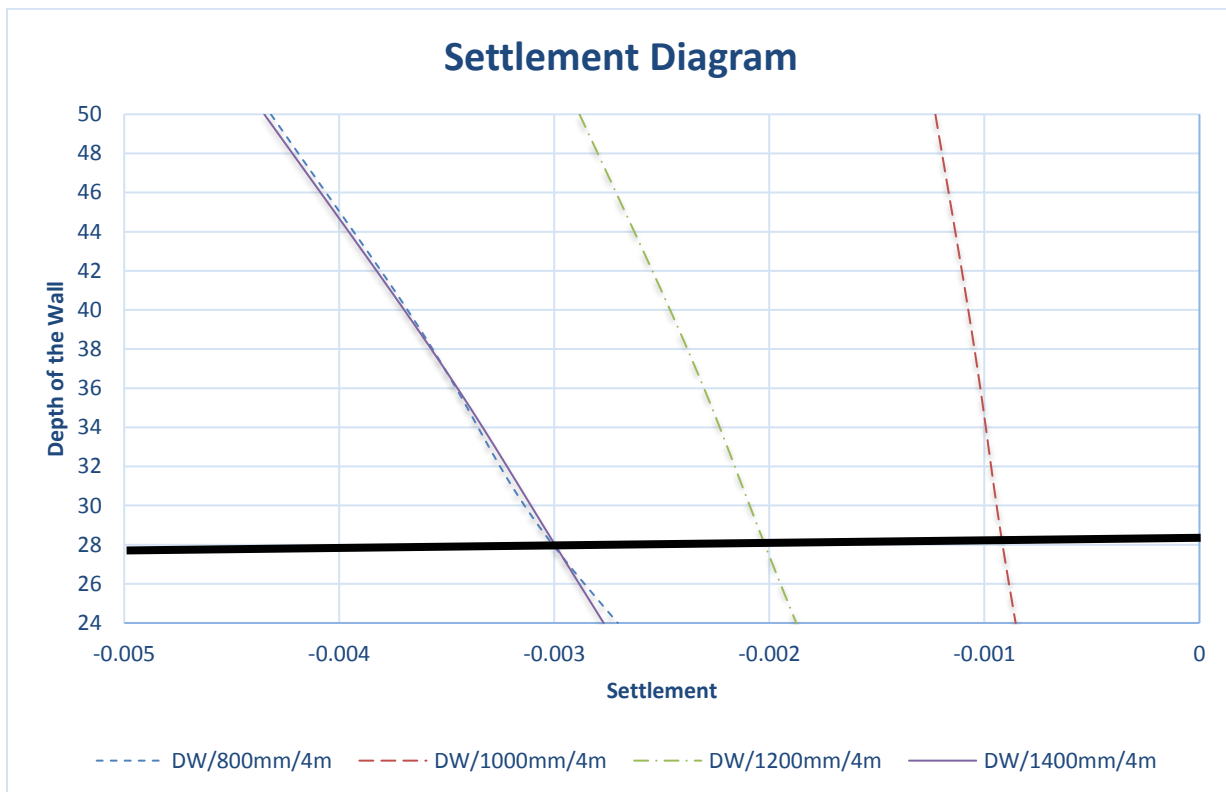


Figure 37 Settlement Diagram of Diaphragm Wall for 4m Embedment Length

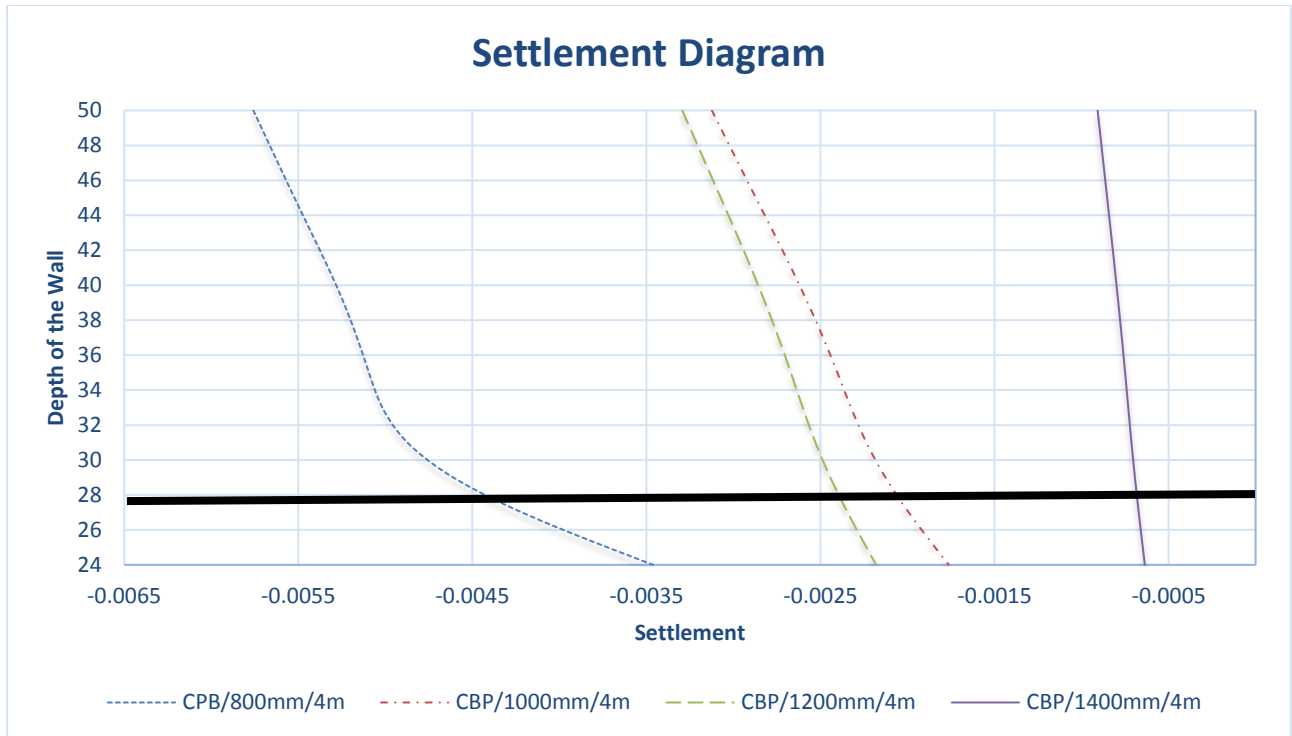


Figure 38 Settlement Diagram of Contiguous Bored Pile Wall for 4m Embedment Length

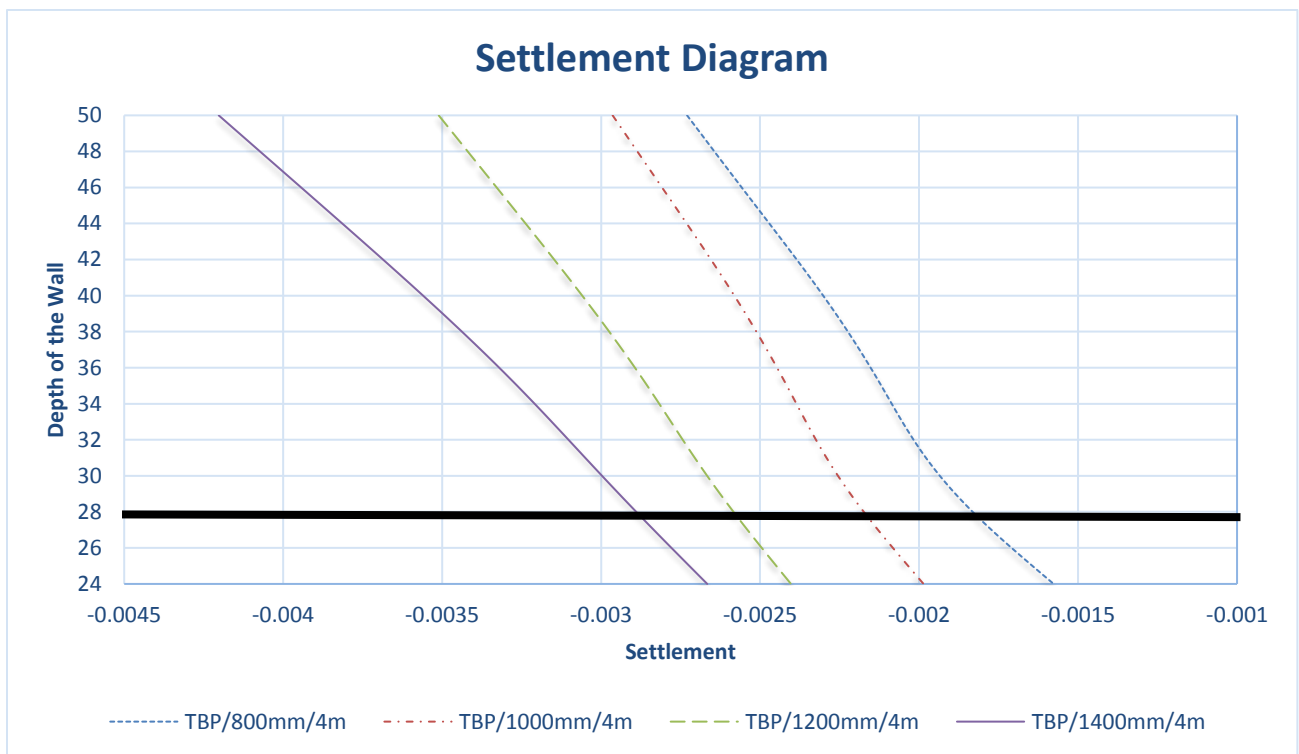


Figure 39 Settlement Diagram of Tangent Bored Pile Wall for 4m Embedment Length

## **4.1.2. Effect of Wall Embedment Length**

In this section, the effect of change in depth of wall embedment on the performance of deep excavations supported by Contiguous bored pile, Tangent bored pile and Diaphragm wall for 4m and 6m embedment lengths is presented.

### **4.1.2.1. Shear Force**

For all types of walls, a significant increase in the shear force is observed when the wall embedment depth is increased from 4m to 6m. For the 800mm thick diaphragm wall, the shear force increased from 235.00 kN to 260.04 kN which is a 10.66% increase. Similarly, the shear force showed an increase of 6.76% 10.82% and 14.04% increase in the 1000mm, 1200mm and 1400mm diaphragm walls respectively.

The tangent bored pile walls also showed the same trend of shear force increase as the depth of embedment is increased from 4m to 6m. As the embedment depth increased, the shear force increased from 229.43 kN to 233.10 kN which implies an increment of 1.60. Similarly, the shear force increased by 3.20%, 6.80% and 10.34% for the 1000mm, 1200mm and 1400mm tangent bored pile walls respectively.

As it can be seen from figure 49, the shear force in the contiguous bored piles increases for each wall thickness as the embedment length was increased from 4m to 6m. For the 800mm thick contiguous bored pile wall, the shear force increased from 195.31 kN to 214.54 kN which is a 9.5% increase. Similarly, the shear force showed an increase of 1.75%, 2.20% and 6.30% increase in the 1000mm, 1200mm and 1400mm contiguous bored pile walls respectively.

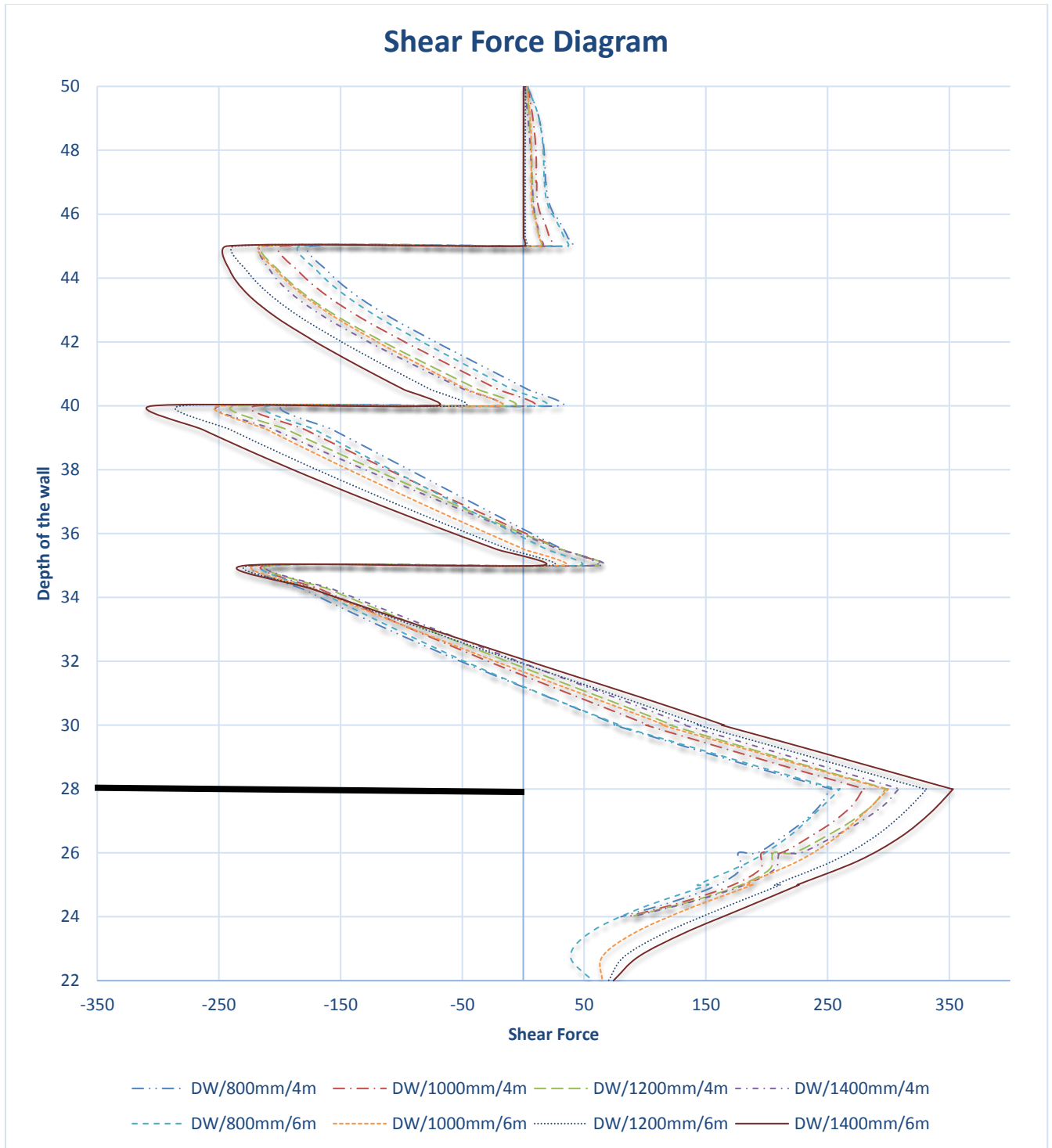


Figure 40 Shear Force Diagram of DW for 4m and 6m Embedment Length

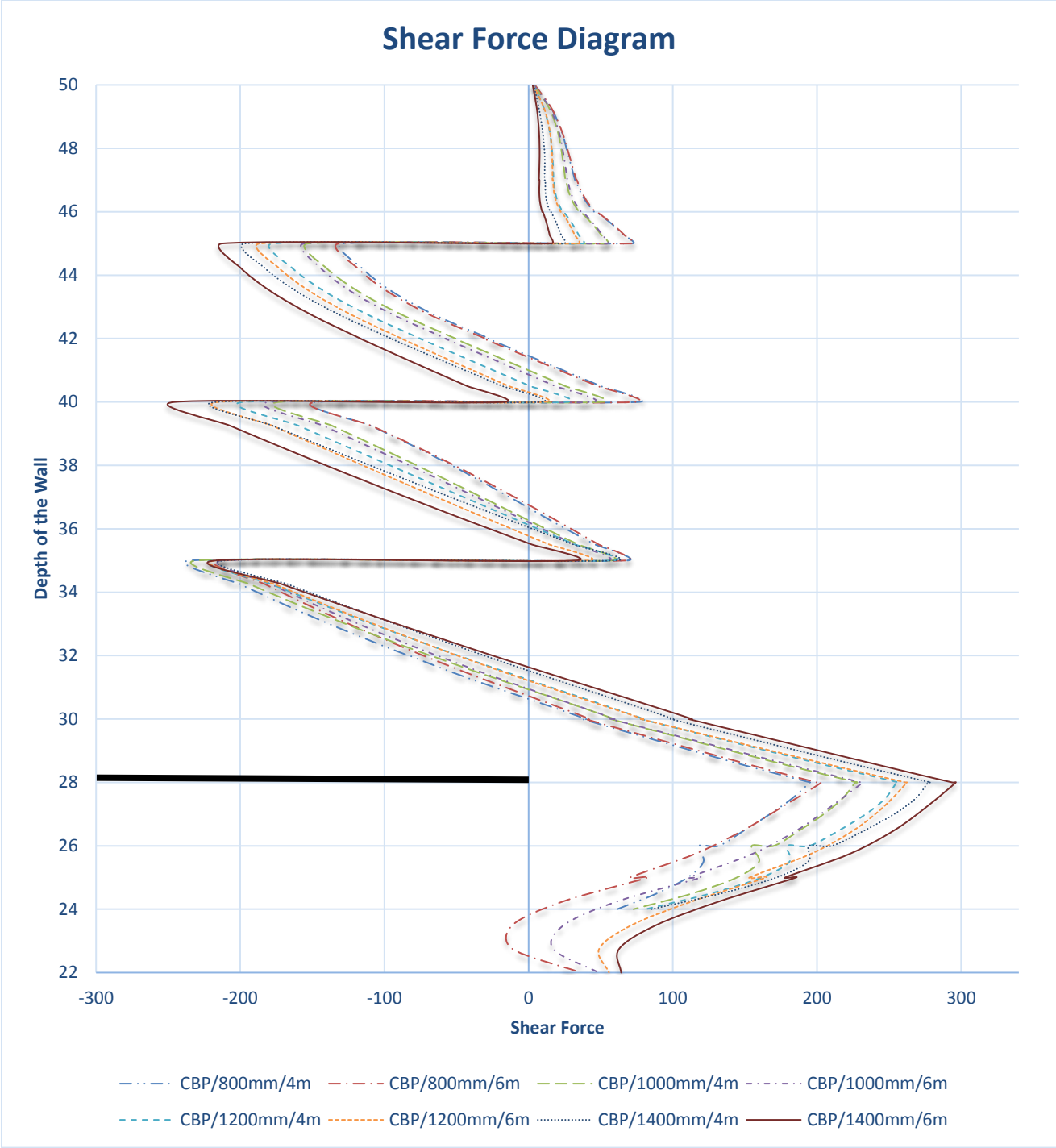


Figure 41 Shear Force Diagram of CBP for 4m and 6m Embedment Length

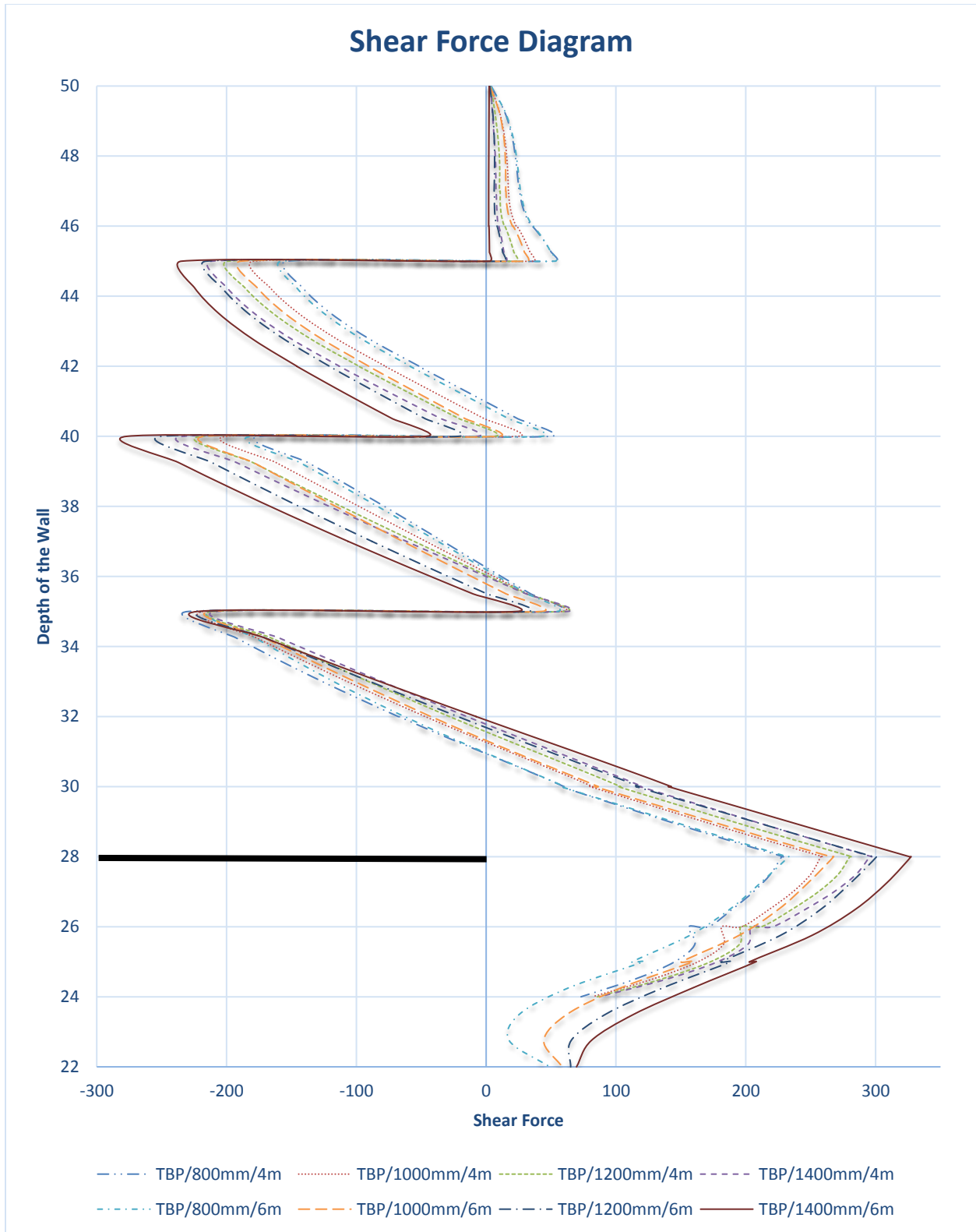


Figure 42 Shear Force Diagram of TBP for 4m and 6m Embedment Length

### 4.1.2.2. Maximum Bending Moment

The maximum bending moment in the walls also increases in response to increase in depth of wall embedment in the same manner as the shear force. For the change of embedment depth from 4m to 6m of diaphragm wall, it is shown that a 9.43%, 21.70%, 30.71%, and 36.73% increment in a maximum bending moment from the values for 4m embedment depth were obtained in the 800mm, 1000mm, 1200mm and 1400mm.

Figure 44 shows the maximum bending moment in the contiguous bored piles. It can be seen that the maximum bending moment increased by 7.82%, 1.58%, 14.03%, and 21.25% as the embedment depth was increased from 4m to 6m in the 800mm, 1000mm, 1200mm and 1400mm contiguous bored pile walls respectively.

The tangent bored pile walls also showed the same trend of maximum bending moment increase as the depth of embedment is increased from 4m to 6m. As the embedment depth increased, the maximum bending moment increased by 1.08%, 12.06%, 22.48% and 30.43% for the 800mm, 1000mm, 1200mm and 1400mm tangent bored pile walls respectively.

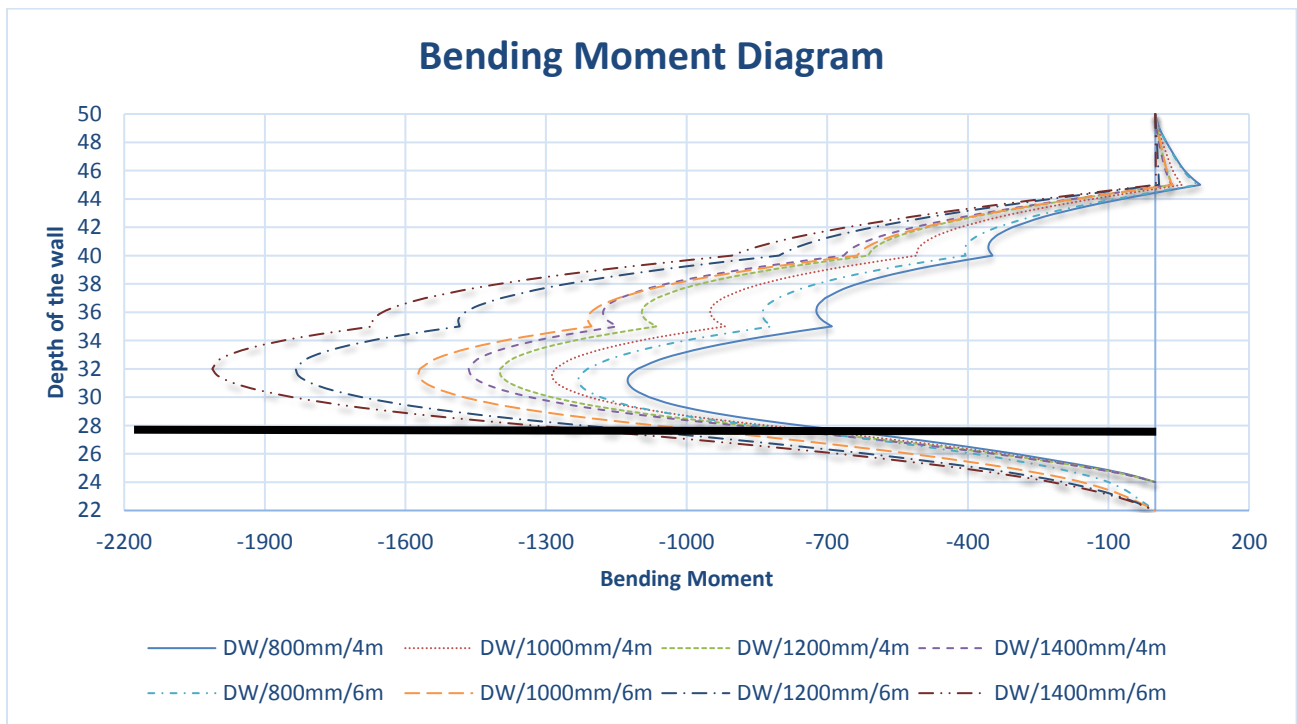


Figure 43 Bending Moment Diagram of DW for 4m and 6m Embedment Length

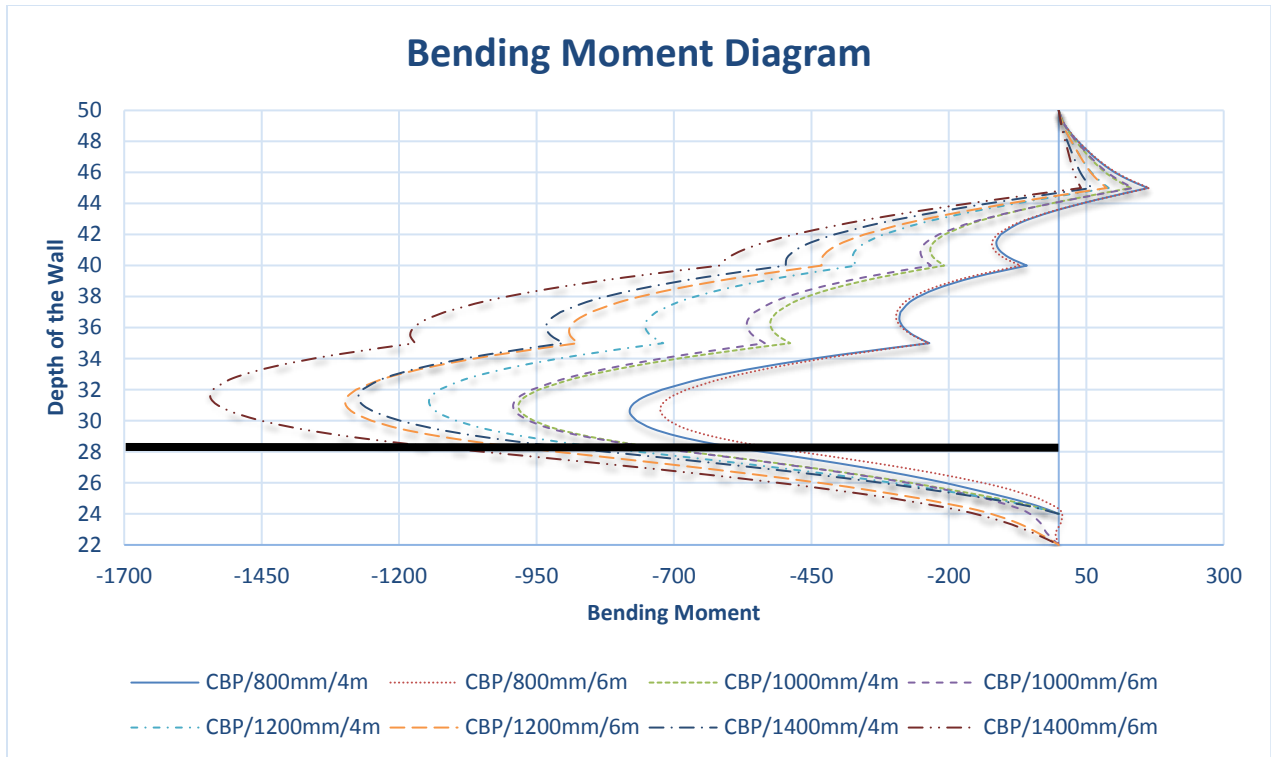


Figure 44 Bending Moment Diagram of CBP for 4m and 6m Embedment Length

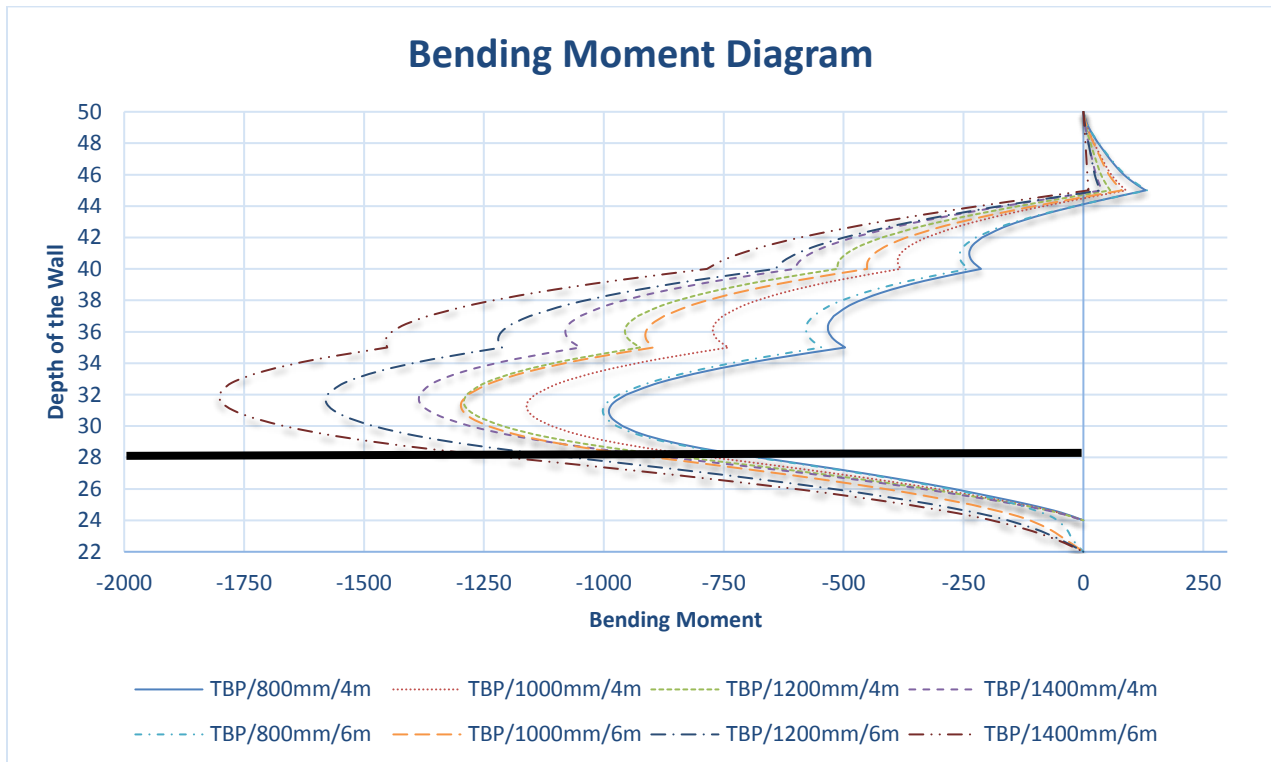


Figure 45 Bending Moment Diagram of TBP for 4m and 6m Embedment Length

### 4.1.2.3. Horizontal Displacement of the Wall

Unlike the shear force and maximum bending moment, the horizontal displacement of the walls decreased as the embedment depth of walls was increased from 4m to 6m. The increment of embedment depth of the diaphragm wall resulted in a decrease of 33.00%, 33.76%, 32.64%, and 31.20% of lateral wall deflection of 800mm, 1000mm, 1200mm and 1400mm walls respectively.

Similar to the diaphragm walls, the horizontal displacement in the contiguous bored piles shows a decrease in response to the increase of wall embedment depth from 4m to 6m. It can be seen in figure 47 that horizontal displacement decreased by 12.33%, 27.77%, 27.82%, and 33.91% as the embedment depth was increased from 4m to 6m in the 800mm, 1000mm, 1200mm and 1400mm contiguous bored pile walls respectively.

The behavior of decreased horizontal displacement of the wall is also the same for the tangent bored pile walls. As the depth of embedment is increased from 4m to 6m, their lateral wall deflection decreased by 28.24%, 32.73%, 34.19% and 32.78% for the 800mm, 1000mm, 1200mm and 1400mm tangent bored pile walls respectively.

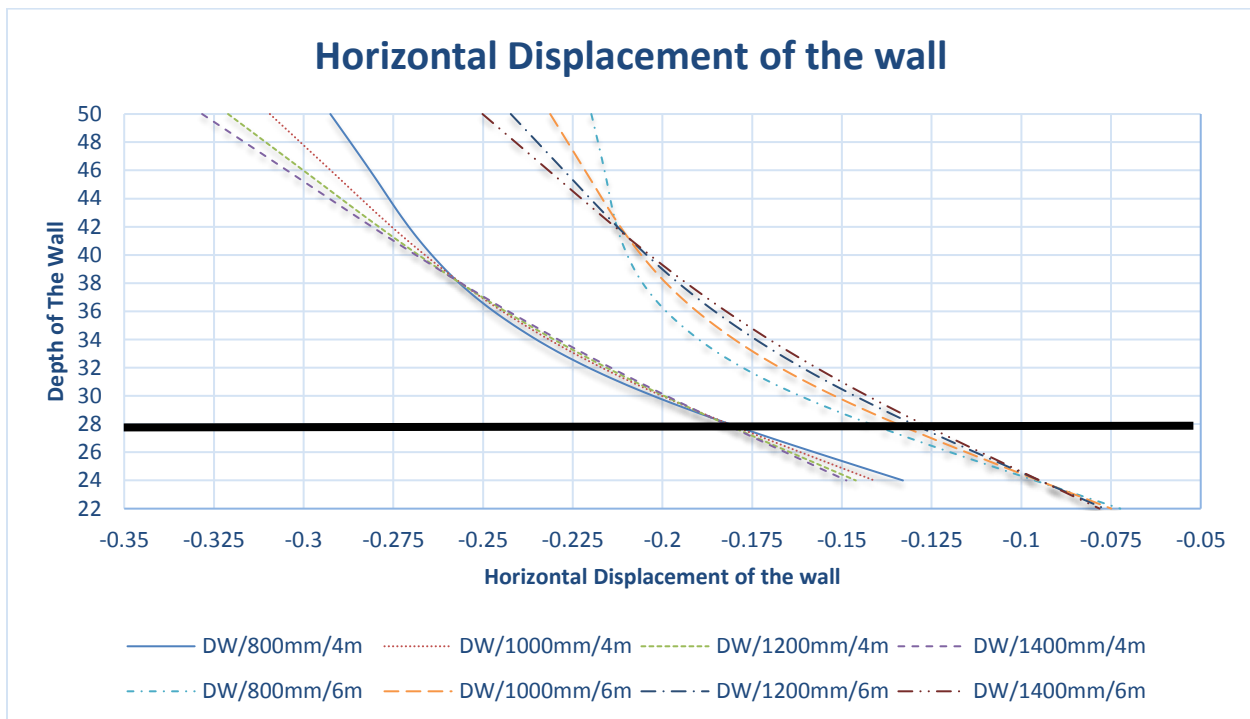


Figure 46 Horizontal Displacement Diagram of DW for 4m and 6m Embedment Length

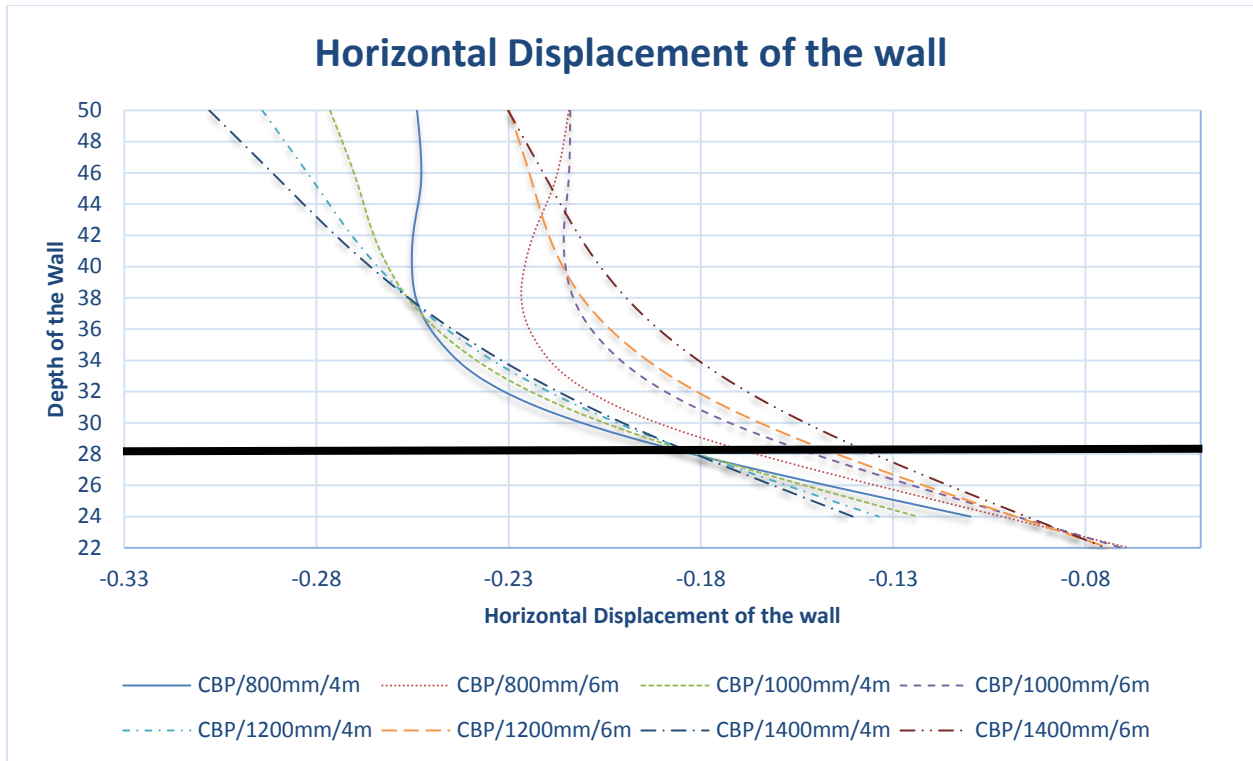


Figure 47 Horizontal Displacement Diagram of CBP for 4m and 6m Embedment Length

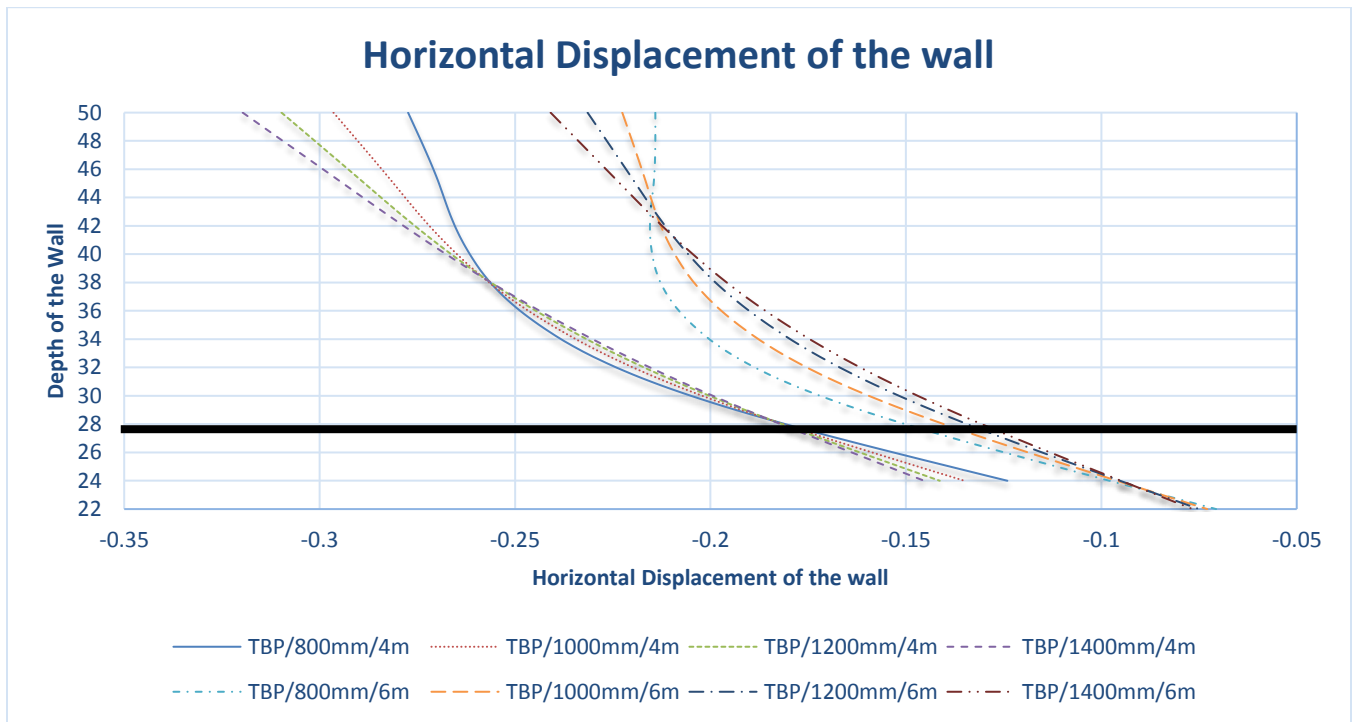


Figure 48 Horizontal Displacement Diagram of TBP for 4m and 6m Embedment Length

#### 4.1.2.4. Ground Settlement

For all types of walls, a significant decrease in the ground settlement is observed when the wall embedment depth is increased from 4m to 6m. For the 800mm thick diaphragm wall, the ground settlement decreased from 4.318mm to 1.601mm which is a 69% decrease.

The tangent bored pile walls also showed the same trend of ground settlement decrease as the depth of embedment is increased from 4m to 6m. As the embedment depth increased, the ground settlement decreased from 2.730mm to 1.310mm which implies an increment of 187%.

As it can be seen from figure 50, the ground settlement in the contiguous bored piles increases for each wall thickness as the embedment length was increased from 4m to 6m. For the 800mm thick contiguous bored pile wall, the ground settlement decreased from 5.758mm to 0.411mm.

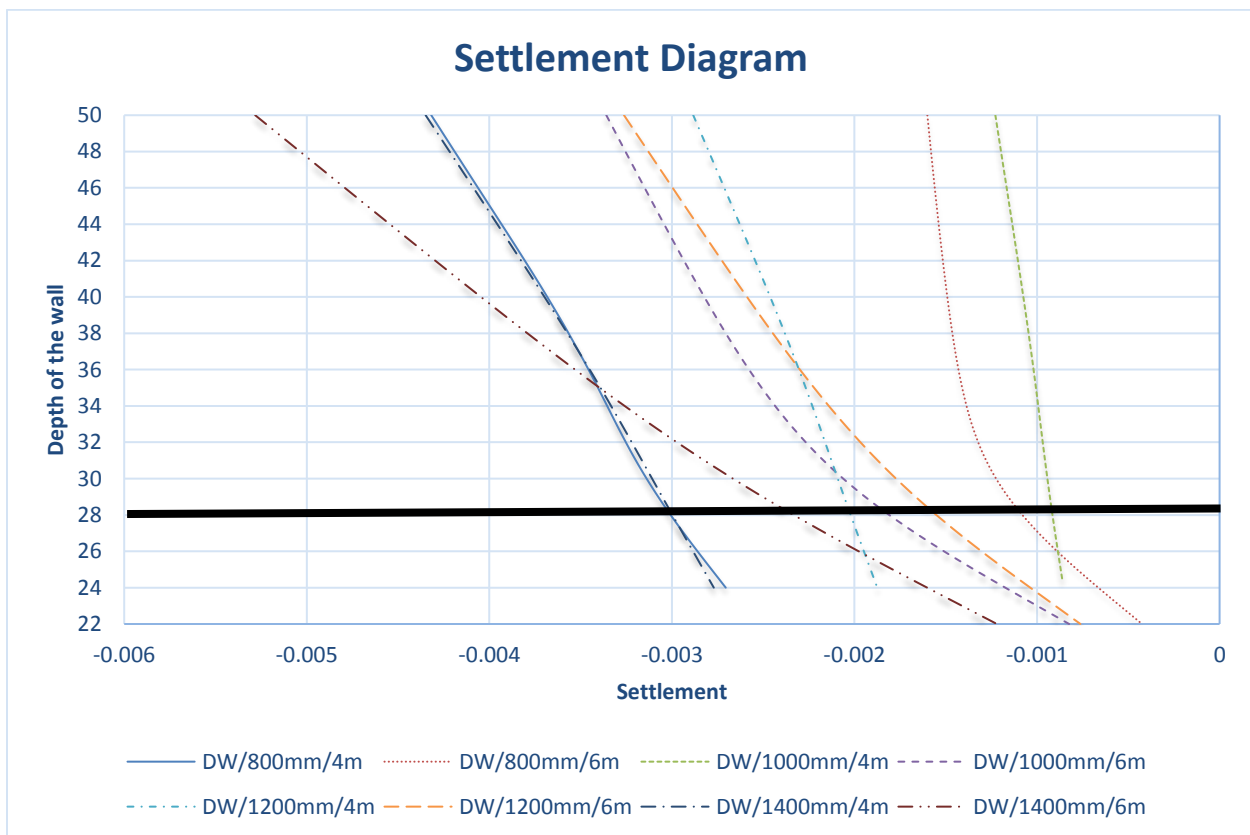


Figure 49 Settlement Diagram of DW for 4m and 6m Embedment Length

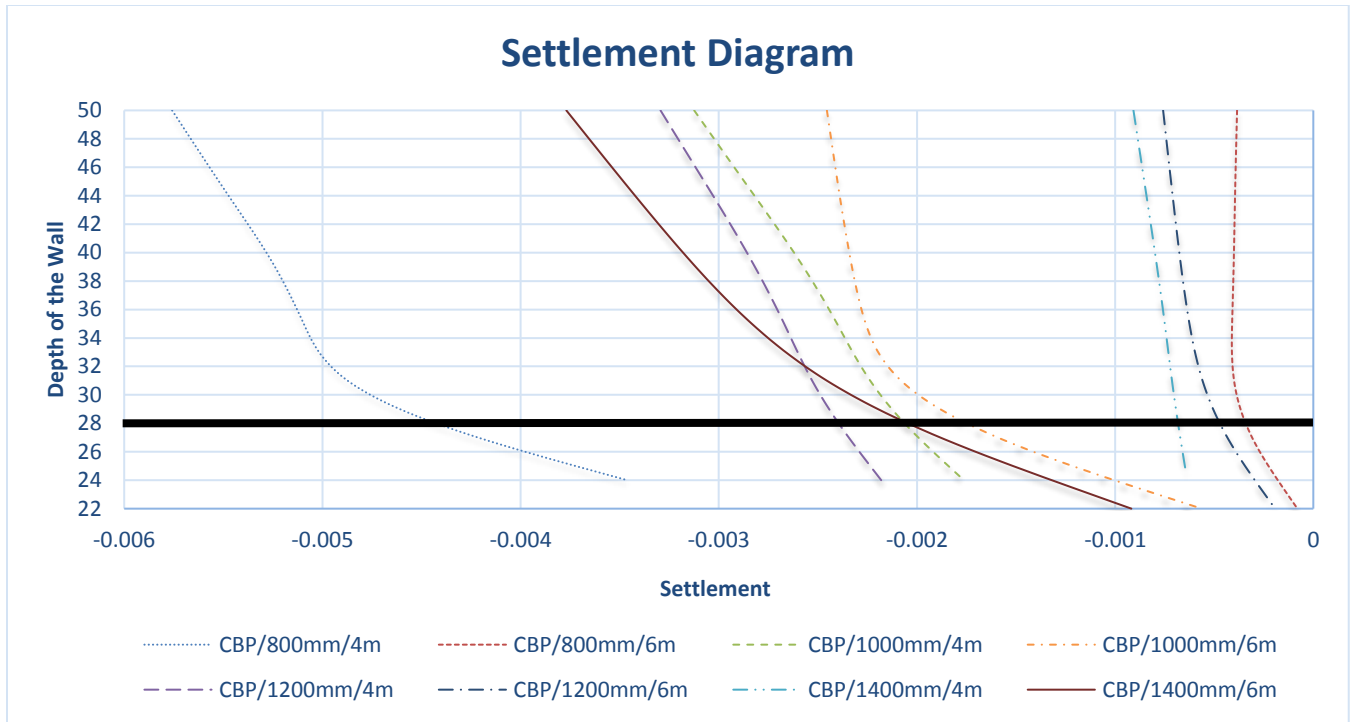


Figure 50 Settlement Diagram of CBP for 4m and 6m Embedment Length

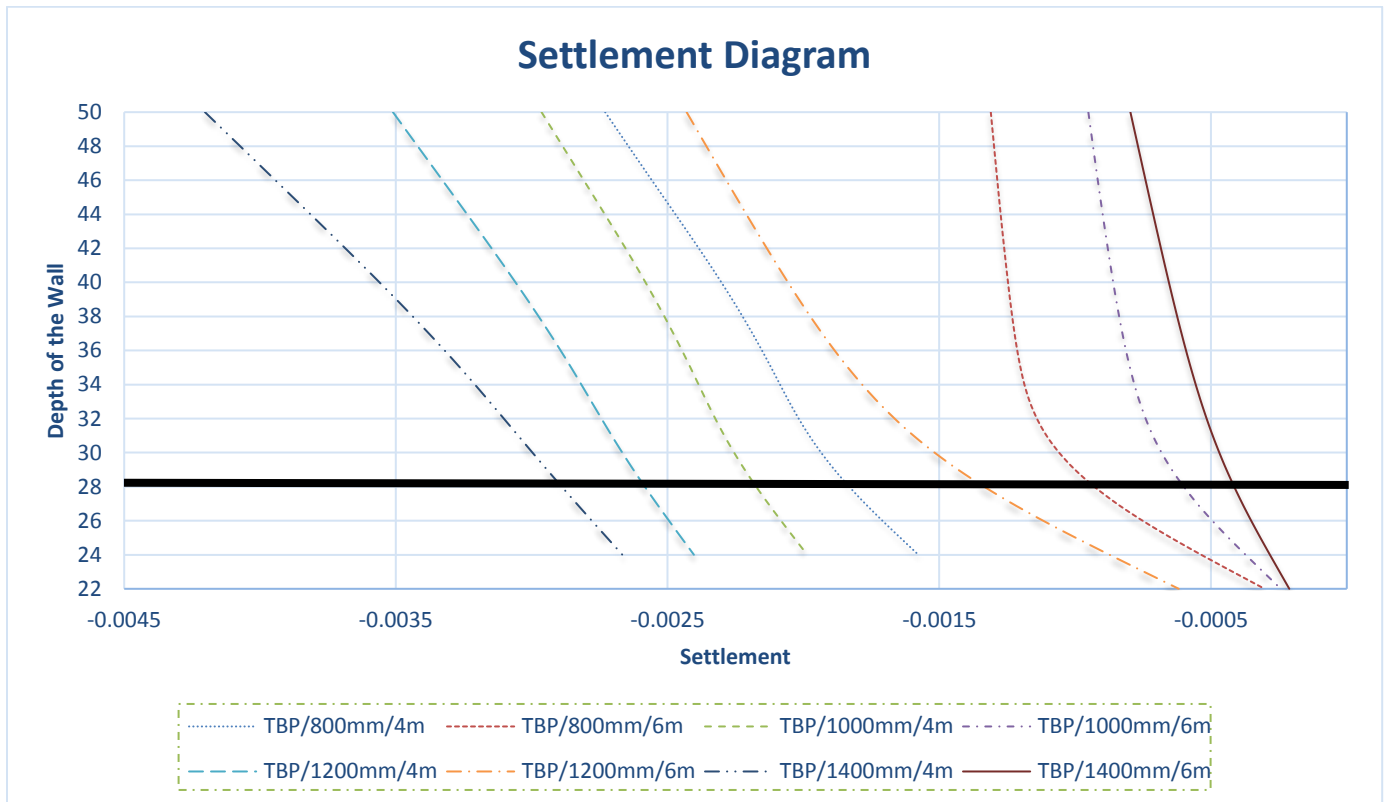


Figure 51 Settlement Diagram of TBP for 4m and 6m Embedment Length

The solution to minimize the ground surface settlement without increasing the wall size is by extending the embedment length of the wall. Hence, the wall deflection is significantly reduced when the embedment length is extended just a few meters. In addition to that, the ground surface settlement also significantly reduced. Thus, the foremost important factor in reducing the wall deflection and also ground surface settlement is by having sufficient embedment wall length. This factor also leads to safer designs and at the same time helps to reduce the cost of the overall project

### 4.1.3. Effect of Wall Type

In this part of the analysis, the paper focuses on how change in type's different retaining structures (Contiguous bored pile, Tangent bored pile and Diaphragm wall) affects the performance of deep excavation. The results from this analysis are presented below.

#### 4.1.3.1. Shear Force

As it can be seen from figure 52 the shear force in the 800mm thick and 4m embedment depth the diaphragm wall has a higher value than that of Tangent bored pile wall and contiguous bored pile wall which is increased by 2.43% and 20% respectively.

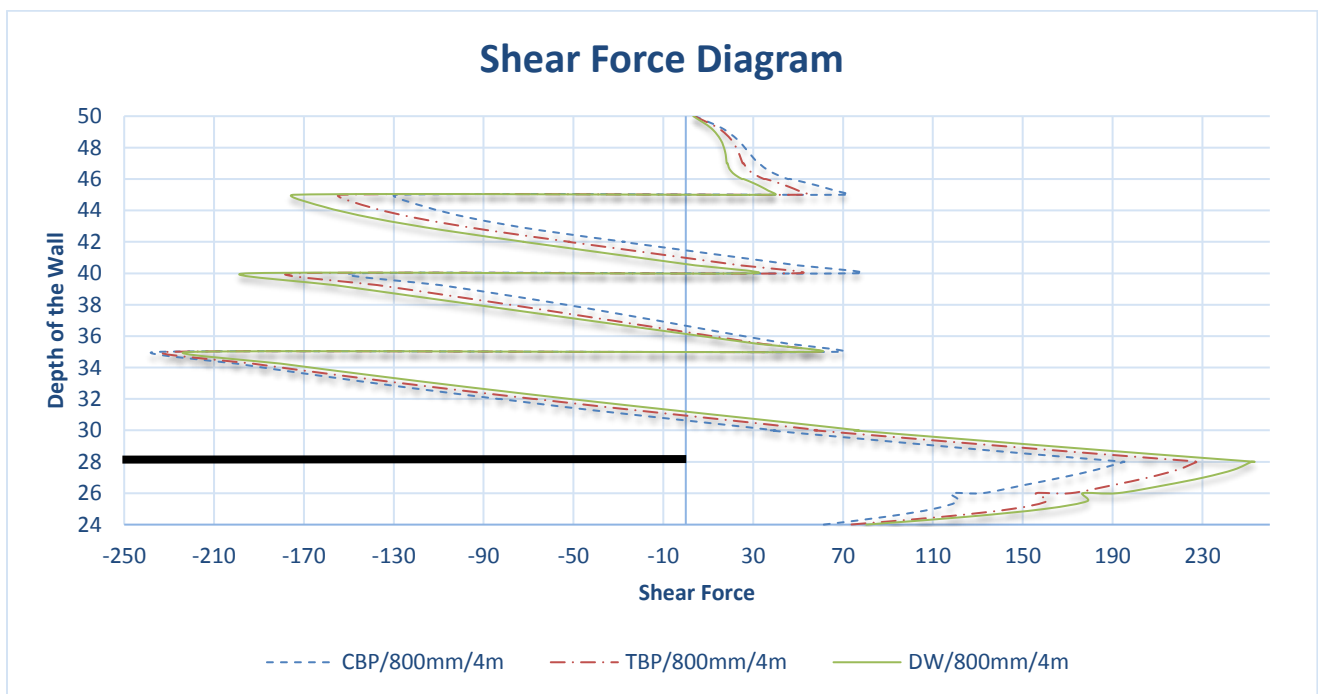


Figure 52 Shear Force Diagram of CBP, TBP and DW 800mm/4m

### 4.1.3.2. Bending Moment

Figure 53 shows the maximum bending moment in the Diaphragm wall increased by 13.61%, and 44.1% Tangent bored pile wall and Contiguous bored pile wall respectively for the case of 800mm thick and 4m embedment depth.

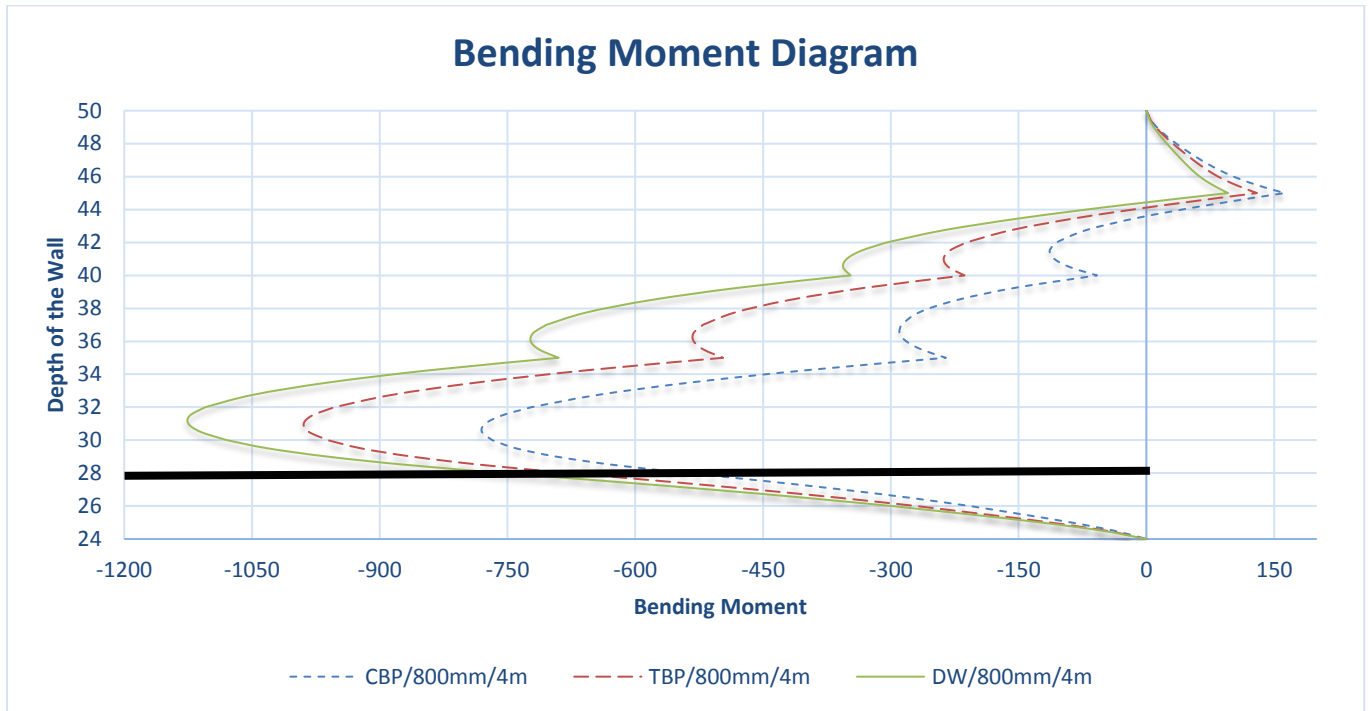
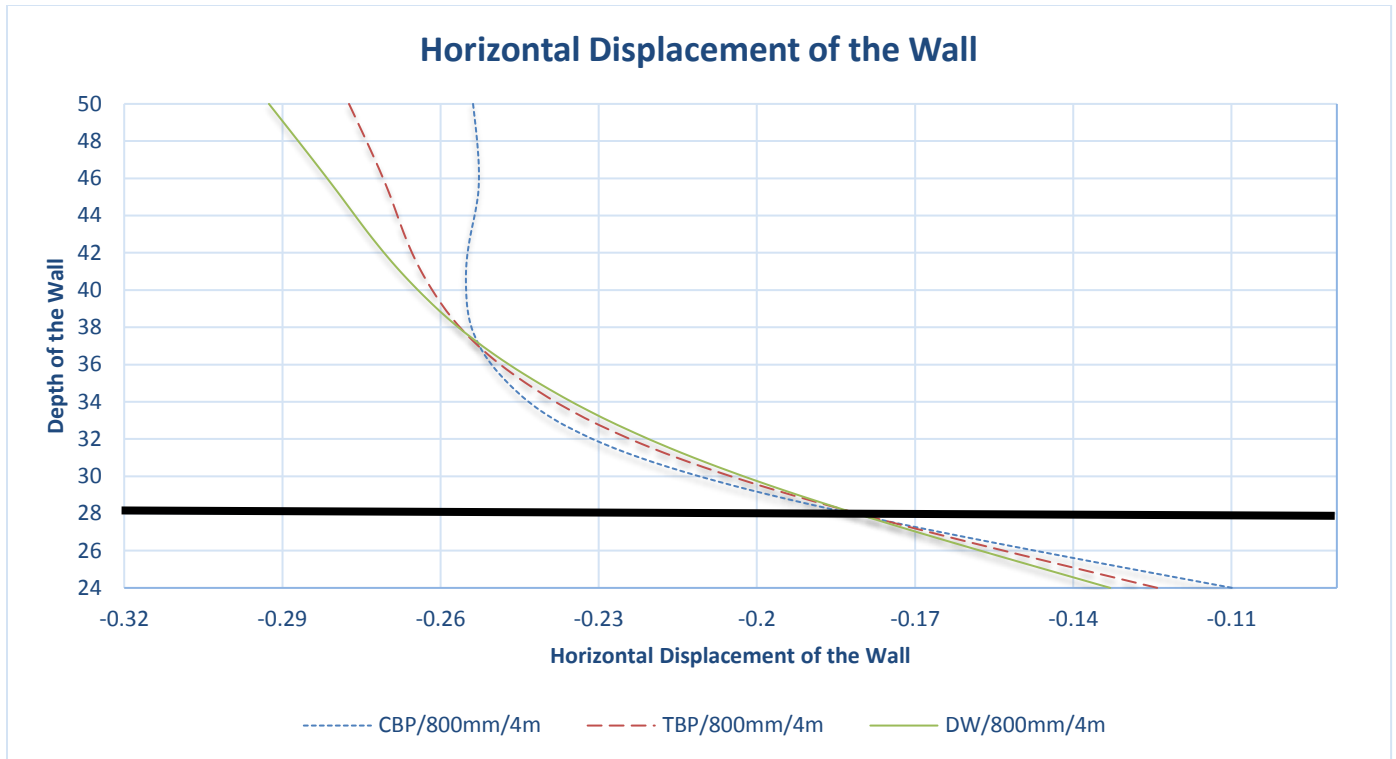


Figure 53 Bending Moment Diagram of CBP, TBP and DW 800mm/4m

### 4.1.3.3. Horizontal Displacement of the Wall

Unlike the shear force and maximum bending moment, horizontal displacement of the Diaphragm wall increased by 5.77%, and 14.9% that of Tangent bored pile wall and Contiguous bored pile wall respectively for the case of 800mm thick and 4m embedment depth. But as it can be seen from the figure below the shape of the diaphragm wall is straighter than the others because of its rigidity. In addition to that when the thickness of the wall become increased the values of lateral wall deflection of the diaphragm, contiguous bored pile and tangent pile walls wall as a result of the increase in rigidity.



**Figure 54 Horizontal Displacement Diagram of CBP, TBP and DW 800mm/4m**

#### 4.1.3.4. Ground Settlement

Similar the lateral wall deflection, the Tangent bored pile wall shows a decrease in ground settlement for the case of 800mm thick and 4m embedment depth. It decreased by 110.92% and 58.17% contiguous bored pile wall and Diaphragm wall respectively. In case of a stiffer wall, the ground is prevented from spreading horizontally. The deformation of the ground, therefore, occurs mainly in the vertical direction, resulting in an increase in ground settlement behind the diaphragm wall. In other words, the consolidation of the ground behind a stiff diaphragm wall is mainly one dimensional. For a relatively flexible diaphragm wall (smaller thickness), there is an increase in horizontal displacement and decrease in the ground settlement.

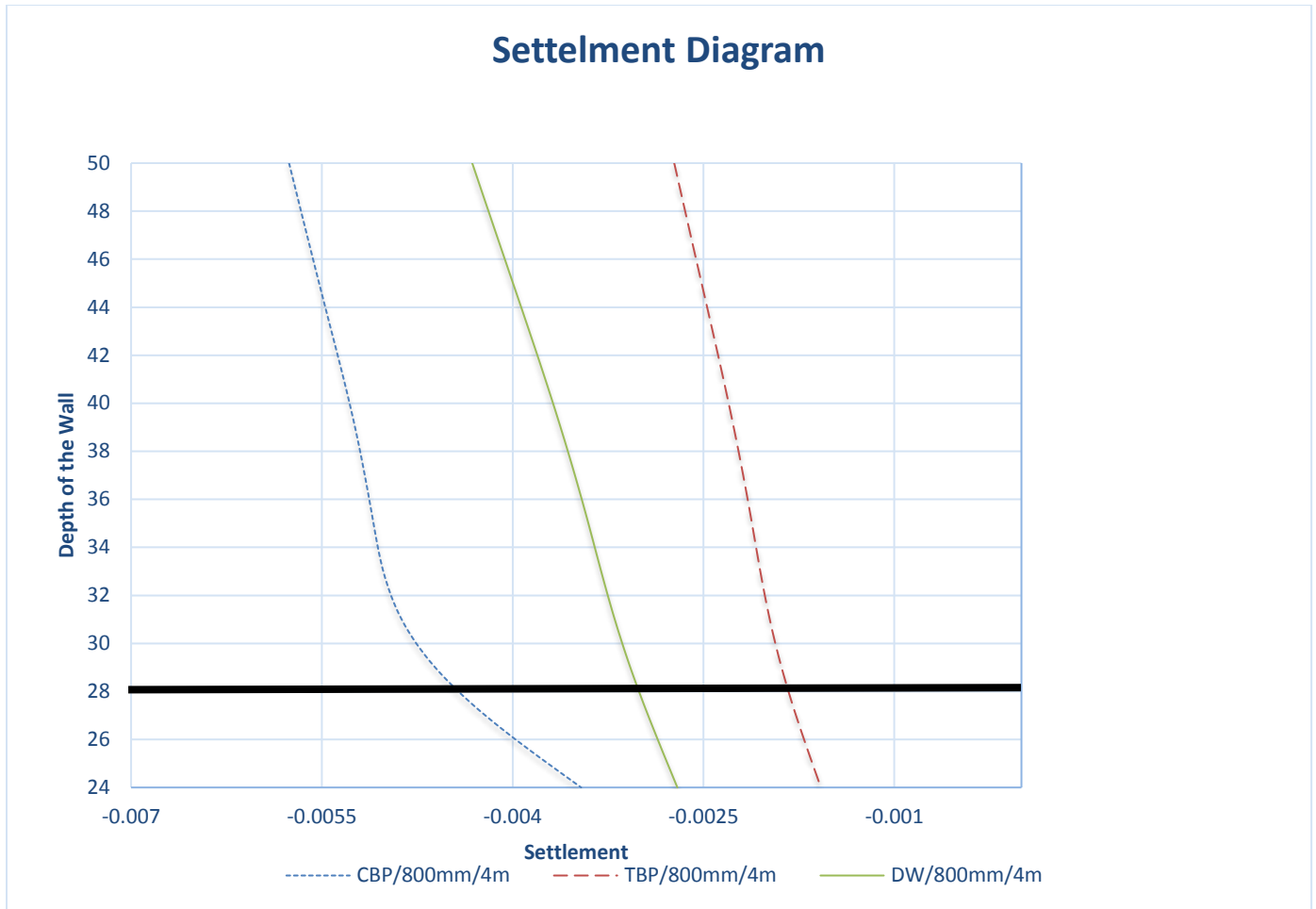


Figure 55 Settlement Diagram of CBP, TBP and DW 800mm/4m

#### 4.1.4. Cost Analysis

As it can be seen from tables 7-9, the total cost per meter of the tangent bored pile is higher than that of the contiguous bored pile and the diaphragm wall being the cheapest among the alternatives because it does not require shotcrete works.

The cost analysis (unit rate) of the walls is prepared per meter. The unit rate for contiguous, diaphragm and tangent bored pile walls is taken from the current Ethiopian construction condition whereas drilling unit rate of the diaphragm wall, due to local unavailability, is adopted from the international market.

N.B.: All prices are expressed hereafter are in **ETB (Ethiopian Birr)** and are net prices which are subject to 15% VAT.

## Contiguous Bored Pile Bill of Quantity Shoring Works

**Table 7 Contiguous Bored Pile Bill of Quantities Shoring Works**

Item No.	Description	unit	Total Quantity	Unit Price	Total Price
1	Drilling Equipment Mobilization	Lump sum	1.00	156,329.16	156,329.16
2	Drilling Equipment Demobilization	Lump sum	1.00	78,704.16	78,704.16
3	Anchoring, shotcreting and grouting machine mobilization	Lump sum	1.00	26,869.87	26,869.87
4	Anchoring shotcreting and grouting machine demobilization	Lump sum	1.00	26,869.87	26,869.87
5	Drilling of piles apex. Diam. 800mm piles	m	26.00	3,846.37	100,005.63
6	Reinforcement cage installation	Kg	1,283.62	65.00	83,435.30
7	Piling Concrete Installation	m <sup>3</sup>	13.06	3,837.16	50,113.30
8	Installation of Temporary Grouted Anchors	m	39.00	2,474.93	96,522.13
9	Stressing and Proof testing of Anchors	No.	3.00	2,197.89	6,593.67
10	Shotcreting Reinforcement cage installation	Kg	111.36	65.00	7,238.40
11	Shotcreting (15cm thick)	m <sup>2</sup>	22.00	1,762.71	38,779.59
<b>TOTAL</b>					<b>671,461.07</b>
<b>VAT (15%)</b>					<b>100,719.16</b>
<b>GRAND TOTAL</b>					<b>772,180.24</b>

## Tangent Bored Pile Bill of Quantities Shoring Works

**Table 8 Tangent Bored Pile Bill of Quantities Shoring Works**

<b>Item No.</b>	<b>Description</b>	<b>unit</b>	<b>Total Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
1	Drilling Equipment Mobilization	Lump sum	1.00	156,329.16	156,329.16
2	Drilling Equipment Demobilization	Lump sum	1.00	78,704.16	78,704.16
3	Anchoring, shotcreting and grouting machine mobilization	Lump sum	1.00	26,869.87	26,869.87
4	Anchoring shotcreting and grouting machine demobilization	Lump sum	1.00	26,869.87	26,869.87
5	Drilling of piles apex. Diam. 800mm piles	m	32.50	3,846.37	125,007.04
6	Reinforcement cage installation	Kg	1,565.08	65.00	101,730.20
7	Piling Concrete Installation	m <sup>3</sup>	15.66	3,837.16	60,089.91
8	Installation of Temporary Grouted Anchors	m	39.00	2,474.93	96,522.13
9	Stressing and Proof testing of Anchors	No.	3.00	2,197.89	6,593.67
<b>TOTAL</b>					<b>678,716.01</b>
<b>VAT (15%)</b>					<b>101,807.40</b>
<b>GRAND TOTAL</b>					<b>780,523.41</b>

## Diaphragm Wall Bill of Quantities Shoring Works

**Table 9 Diaphragm Wall Bill of Quantities Shoring Works**

Item No.	Description	unit	Total Quantity	Unit Price	Total Price
1	Drilling Equipment Mobilization	Lump sum	1.00	171,962.08	171,962.08
2	Drilling Equipment Demobilization	Lump sum	1.00	86,574.58	86,574.58
3	Anchoring, shotcreting and grouting machine mobilization	Lump sum	1.00	26,869.87	26,869.87
4	Anchoring shotcreting and grouting machine demobilization	Lump sum	1.00	26,869.87	26,869.87
5	Drilling cost	m <sup>3</sup>	20.08	954.88	19,173.89
6	Reinforcement cage installation	Kg	1,709.77	65.00	111,135.05
7	Piling Concrete Installation	m <sup>3</sup>	20.80	3,837.16	79,812.91
8	Installation of Temporary Grouted Anchors	m	39.00	2,474.93	96,522.13
9	Stressing and Proof testing of Anchors	No.	3.00	2,197.89	6,593.67
10	Bentonite and Cement	m <sup>2</sup>	468.00	42.00	19,656.00
<b>TOTAL</b>					<b>645,170.04</b>
<b>VAT (15%)</b>					<b>96,775.51</b>
<b>GRAND TOTAL</b>					<b>741,945.54</b>

## **V. Chapter Five: Conclusions and Recommendations**

### **5.1. Conclusions**

Based on the finite element analyses made, it can be concluded that the ground movement can be reduced if using higher wall stiffness. It is true that by increasing the wall stiffness ( $EI_w$ ) will result in less horizontal displacement of the wall only to some extent because stiffer walls will be subjected to larger bending moments resulting in the increment of reinforcement needed which entails uneconomical design. Extending the embedment depth of the wall portrays, a significant reduction in the lateral wall displacement and the ground surface movement at the retained side. Besides that, more economical design can be achieved by extending the embedment wall depth.

A stiff retaining wall is required for controlling horizontal ground movements resulting from deep excavation. When choosing a support system for the deep excavation, it should be kept in mind that even the stiffest of the wall will result in some horizontal displacement on the soil. Selecting a stiff wall alone does not eliminate all the horizontal ground movements.

The use of a stiff wall results in the reduction of the horizontal movement of the soil stratum, at same time reducing the settlement of the ground behind the diaphragm wall. In fact, it appears to increase the settlements. If the major concern is to limit the ground settlement behind the diaphragm wall, it is better to use a slightly flexible diaphragm wall system. Using slightly flexible wall with tie-back supports is better in reducing ground deformation around deep excavation in comparison with that of using stiffer retaining wall without tie-back supports.

### **5.2. Recommendation**

As mentioned earlier, in our country especially in Addis Ababa the most common type of retaining wall for deep excavation is contiguous bored pile wall. Based on the results of the study diaphragm wall is the most effective retaining structure than the contiguous bored pile and tangent bored pile walls in terms of performance and cost. In addition to these, diaphragm wall has many advantages like its performance as a retaining structure for very deep excavations as it can be designed to take very high structural loads, its reduced number of joints which ultimately improve the water tightness to be used as a permanent structural wall, its work may be carried out right against

existing structures and the line of wall may be adjusted to any shape in plan, and its relatively minimum noise and vibration levels make it suitable for construction in urban areas.

### **5.2.1. Recommendation for further studies**

All the analysis performed in this study was assumed that there is no effect of wall installation. Thus, to simulate an actual condition at the site and to get a more accurate result in term of ground movement the wall installation effect needs to be considered.

Besides that, all the analysis in this study also was performed by using Plaxis2D. For further study, Plaxis3D can be used in order to help researcher get a more accurate result and reliable. The sophisticated software also can help the researcher to understand and explain very well the behavior of ground movement induced by deep excavation work.

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## VII. Appendix – One

### Additional Results

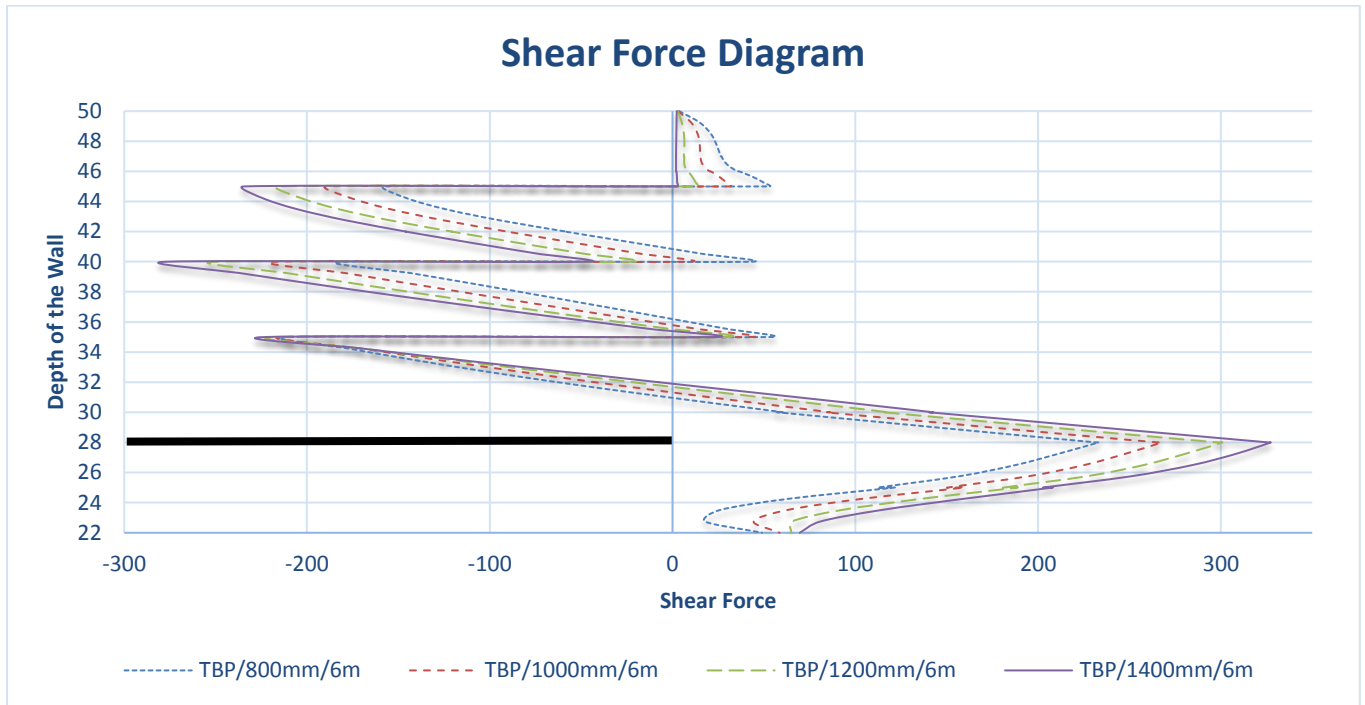


Figure 56 Shear Force Diagram of TBP for 6m Embedment Length

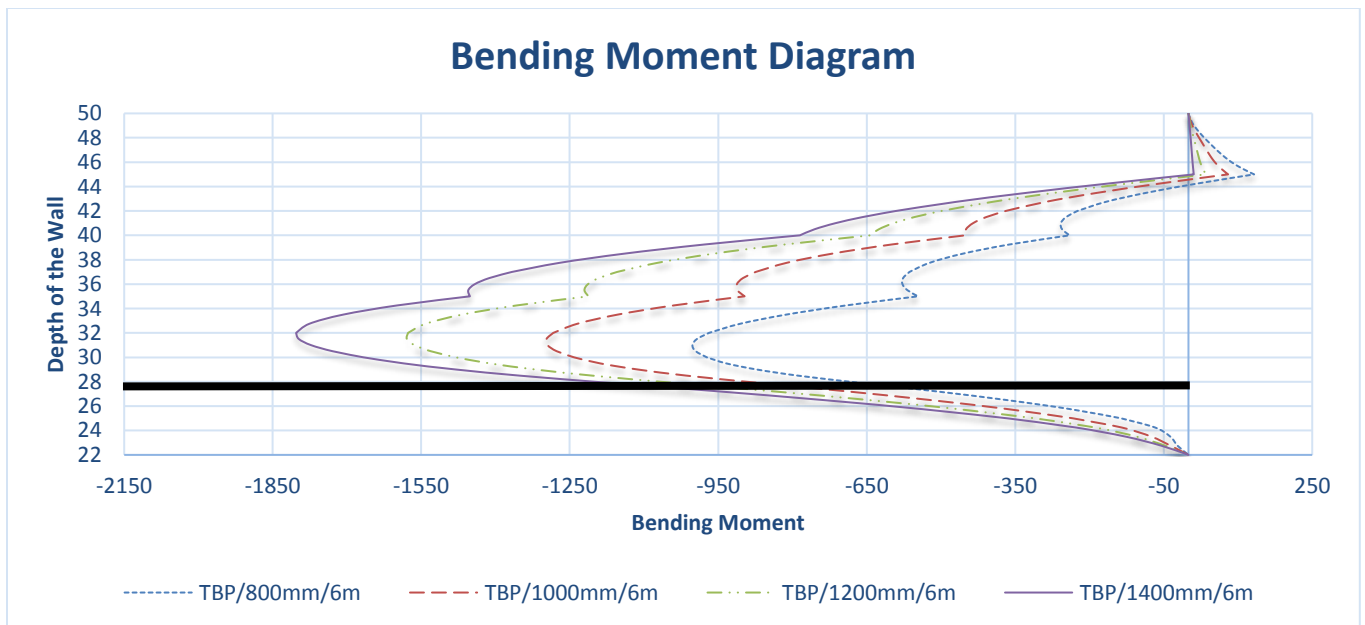


Figure 57 Bending Moment Diagram of TBP for 6m Embedment Length

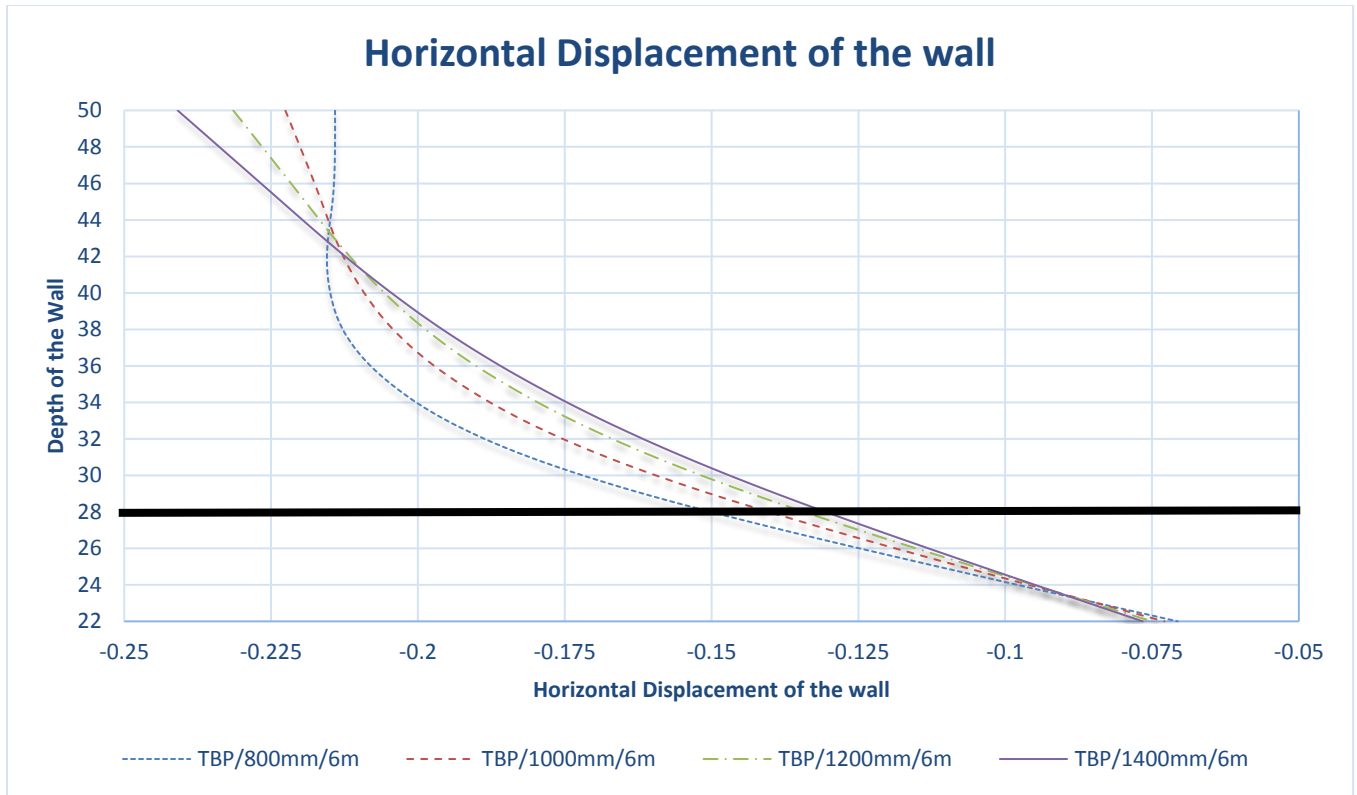


Figure 58 Horizontal Displacement Diagram of TBP for 6m Embedment Length

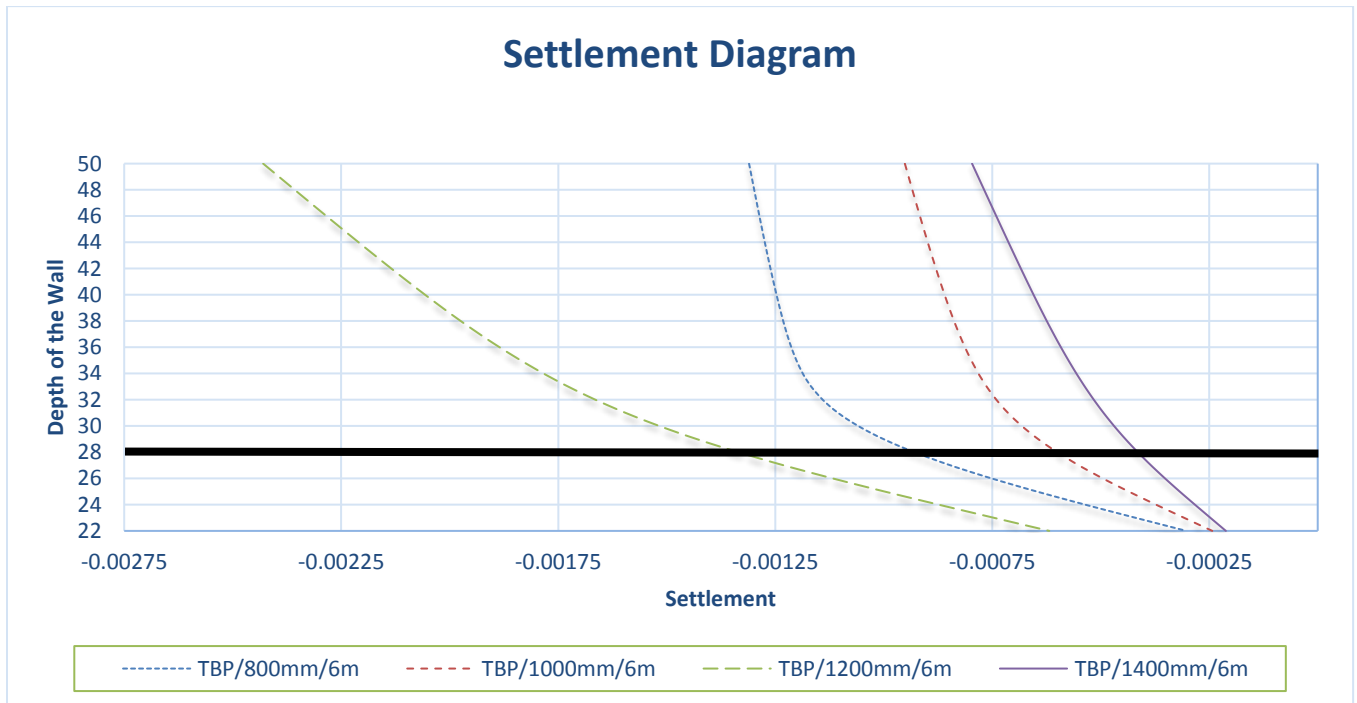
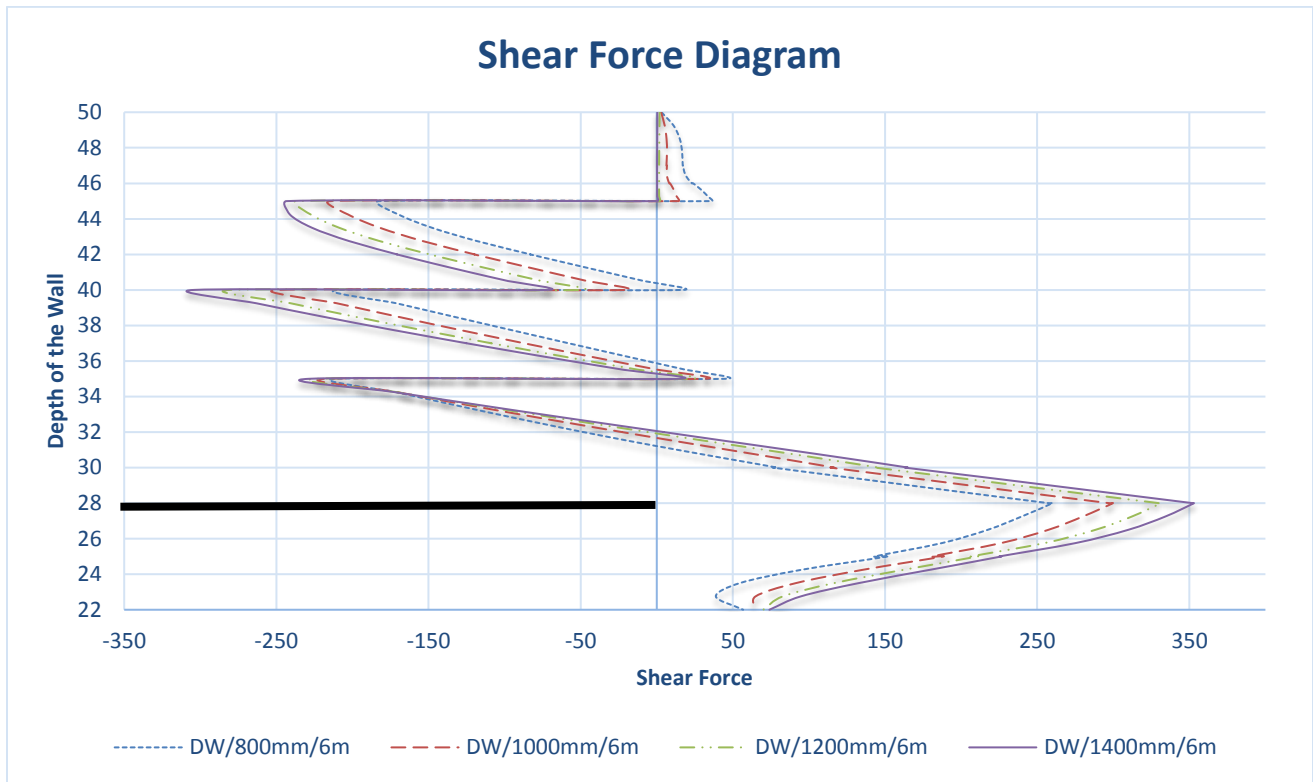
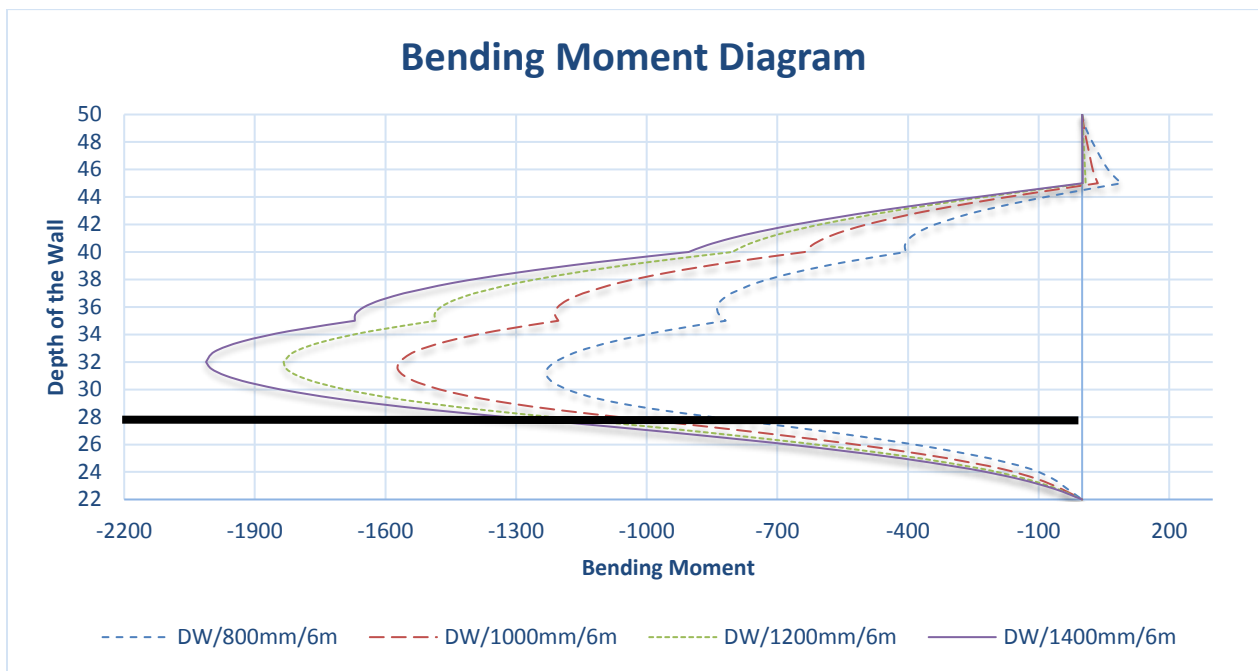


Figure 59 Settlement Diagram of TBP for 6m Embedment Length



**Figure 60 Shear Force Diagram of DW for 6m Embedment Length**



**Figure 61 Bending Moment Diagram of DW for 6m Embedment Length**

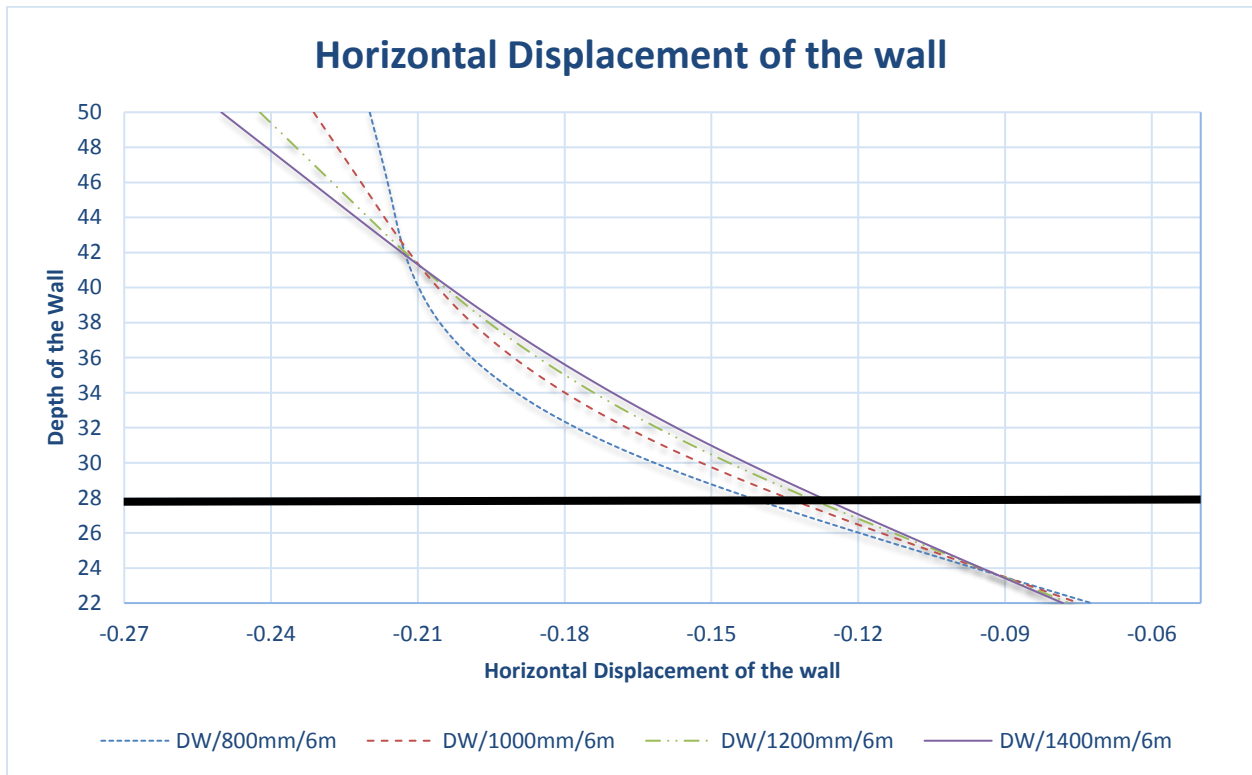


Figure 62 Horizontal Displacement Diagram of DW for 6m Embedment Length

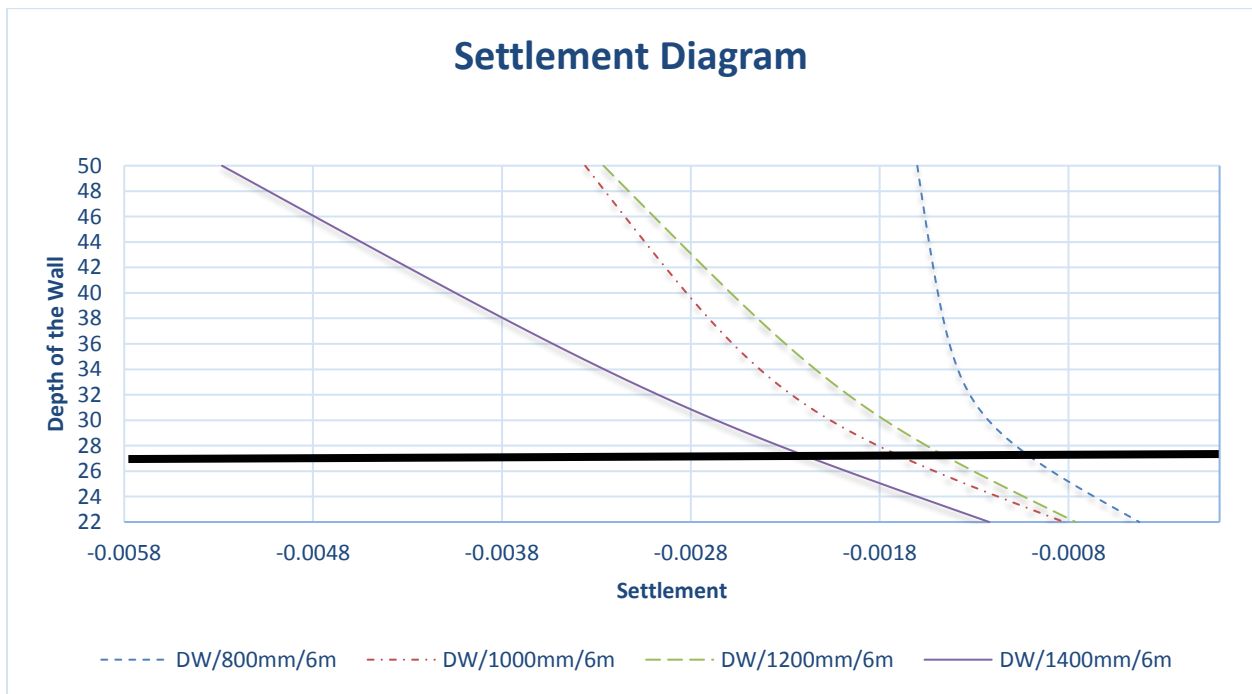
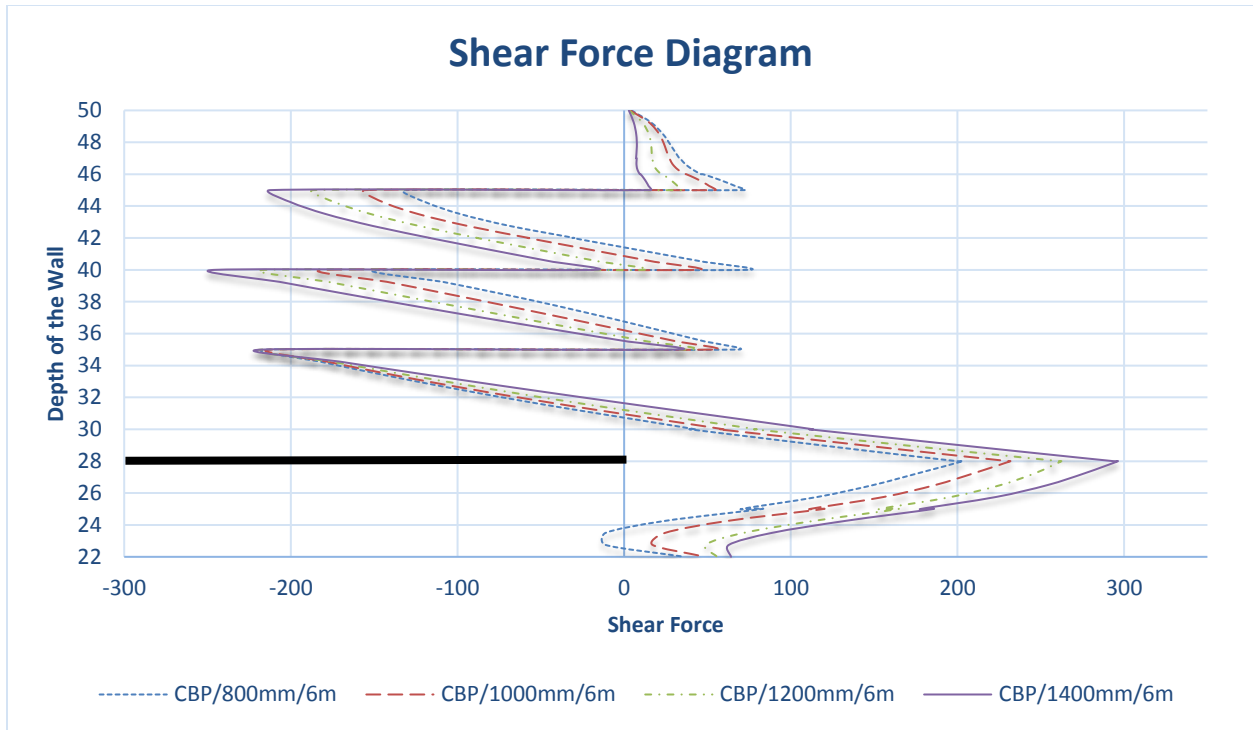
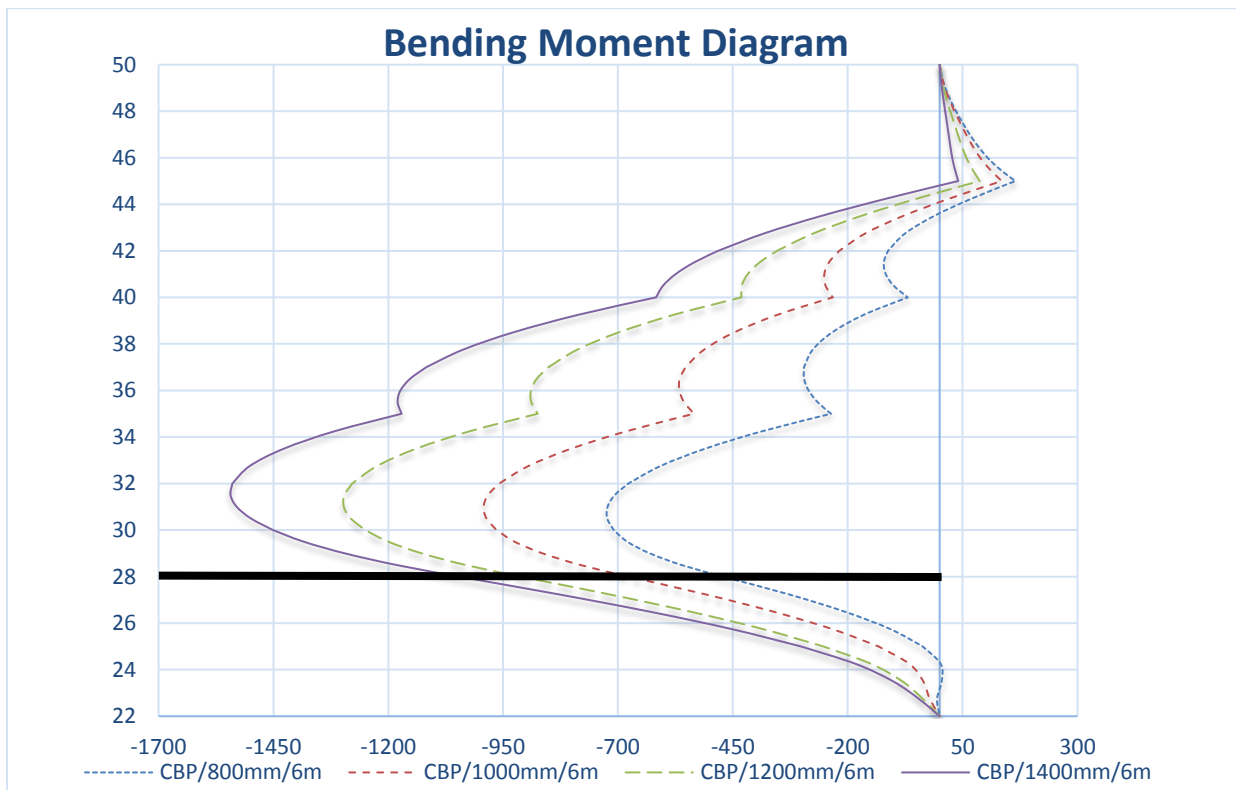


Figure 63 Settlement Diagram of DW for 6m Embedment Length



**Figure 64 Shear Force Diagram of CBP for 6m Embedment Length**



**Figure 65 Bending Moment Diagram of CBP for 6m Embedment Length**

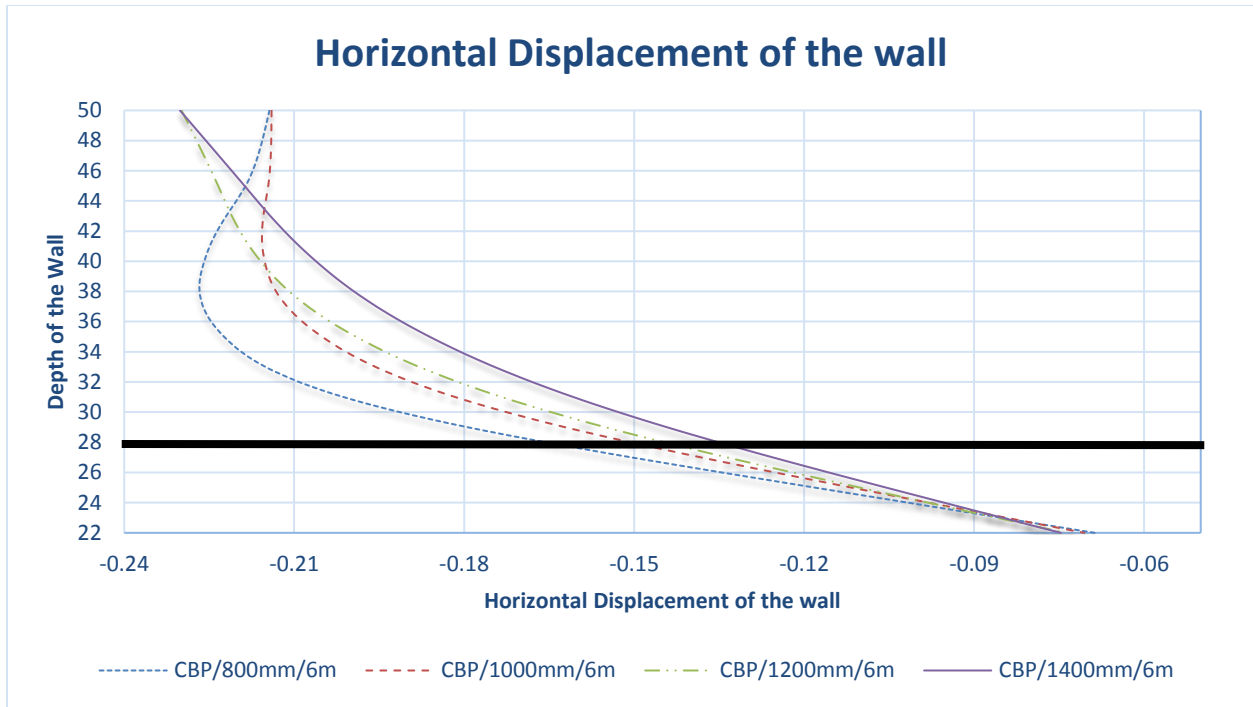


Figure 66 Horizontal Displacement Diagram of CBP for 6m Embedment Length

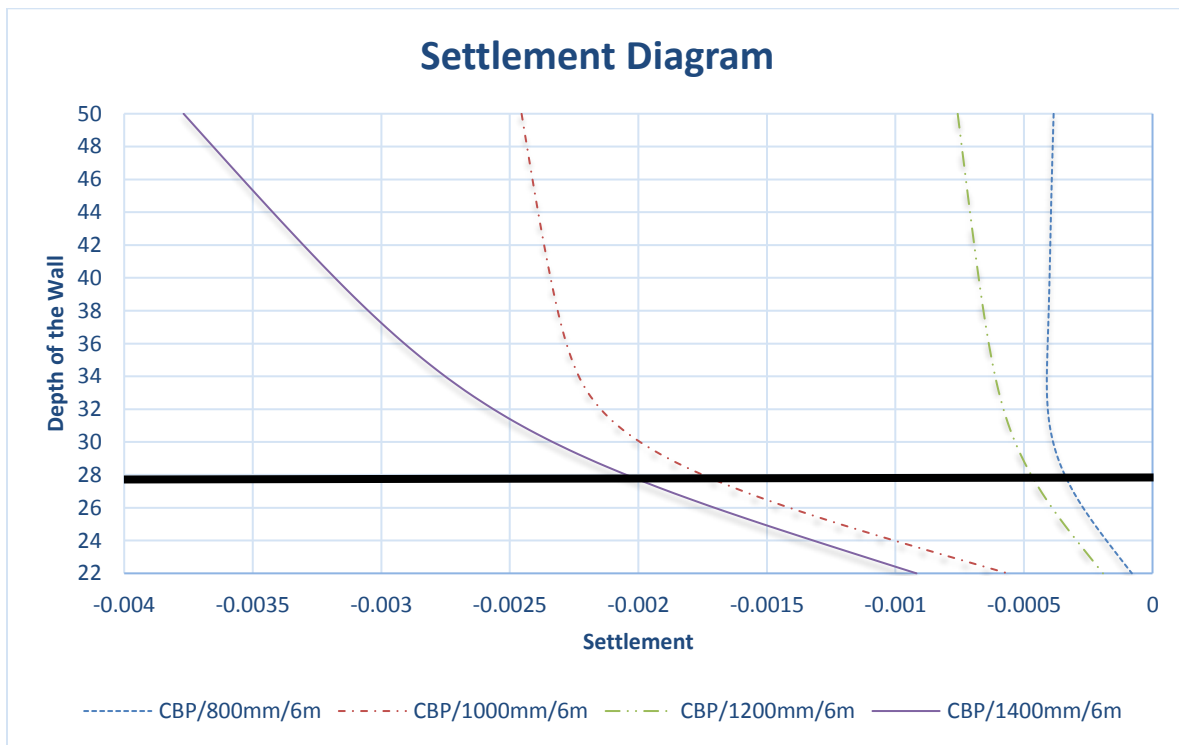


Figure 67 Settlement Diagram of CBP for 6m Embedment Length

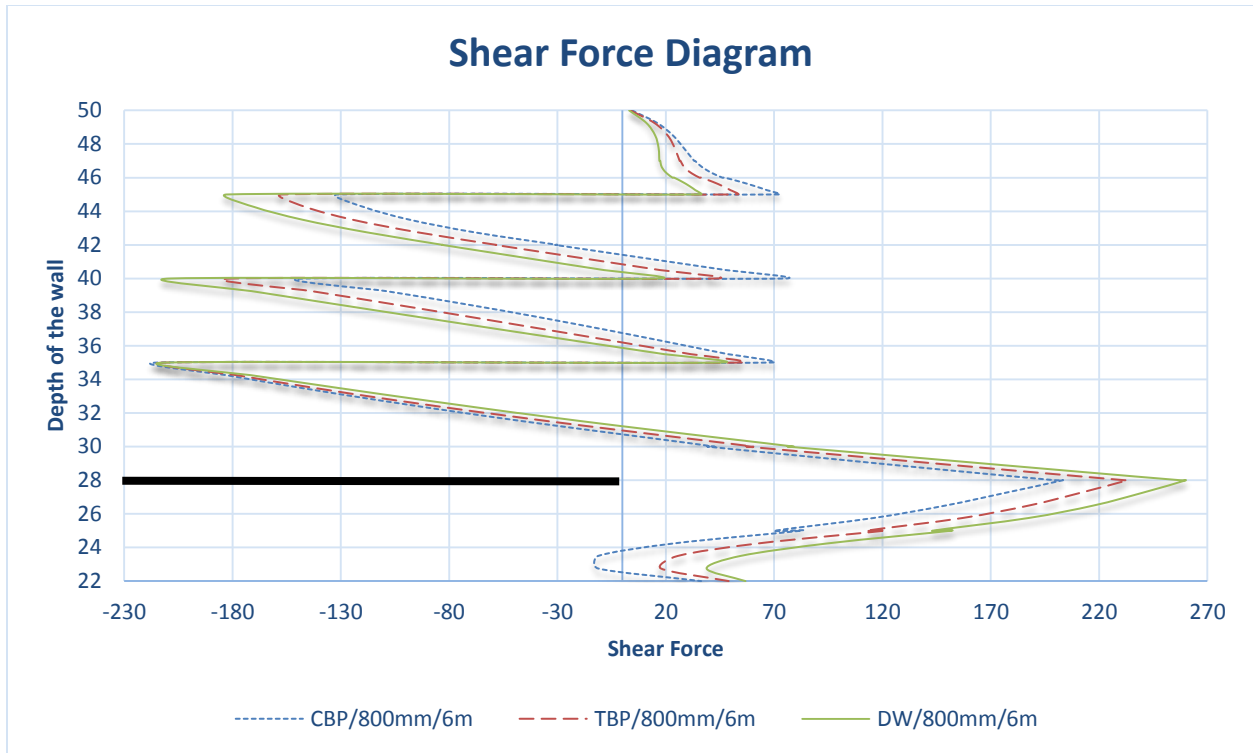


Figure 68 Shear Force Diagram of CBP, TBP and DW 800mm/6m

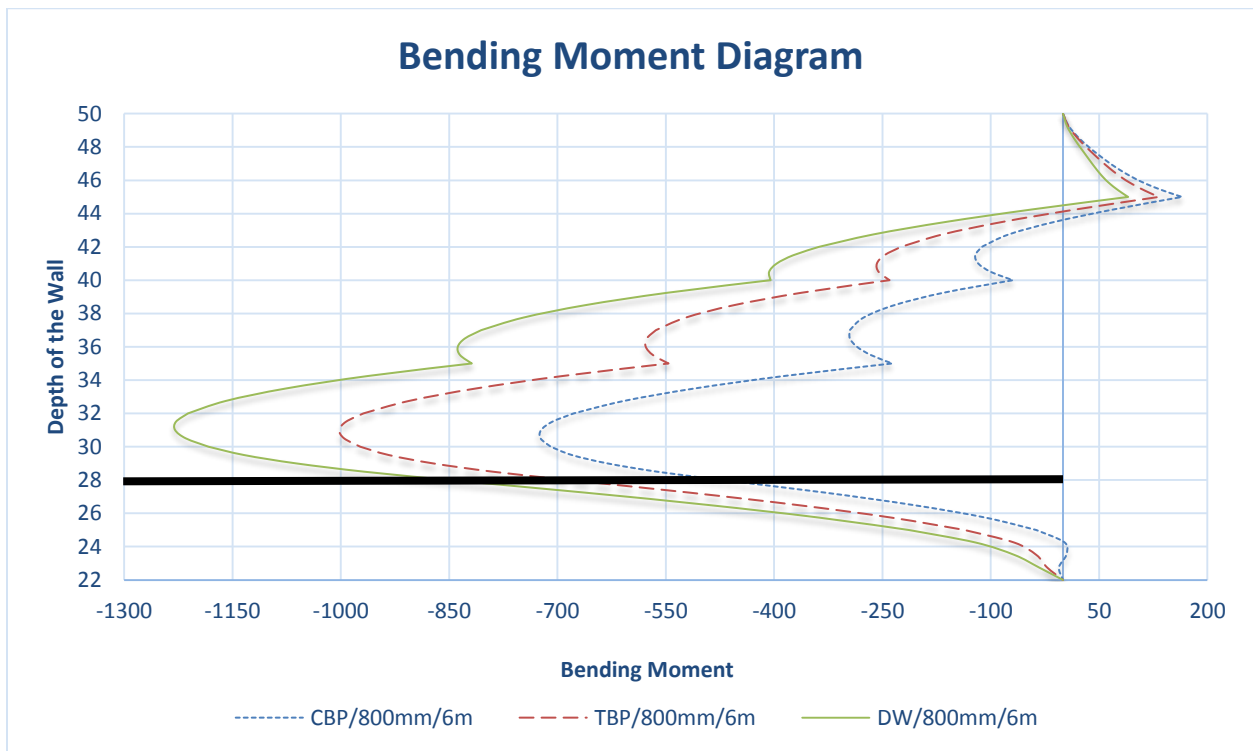


Figure 69 Bending Moment Diagram of CBP, TBP and DW 800mm/6m

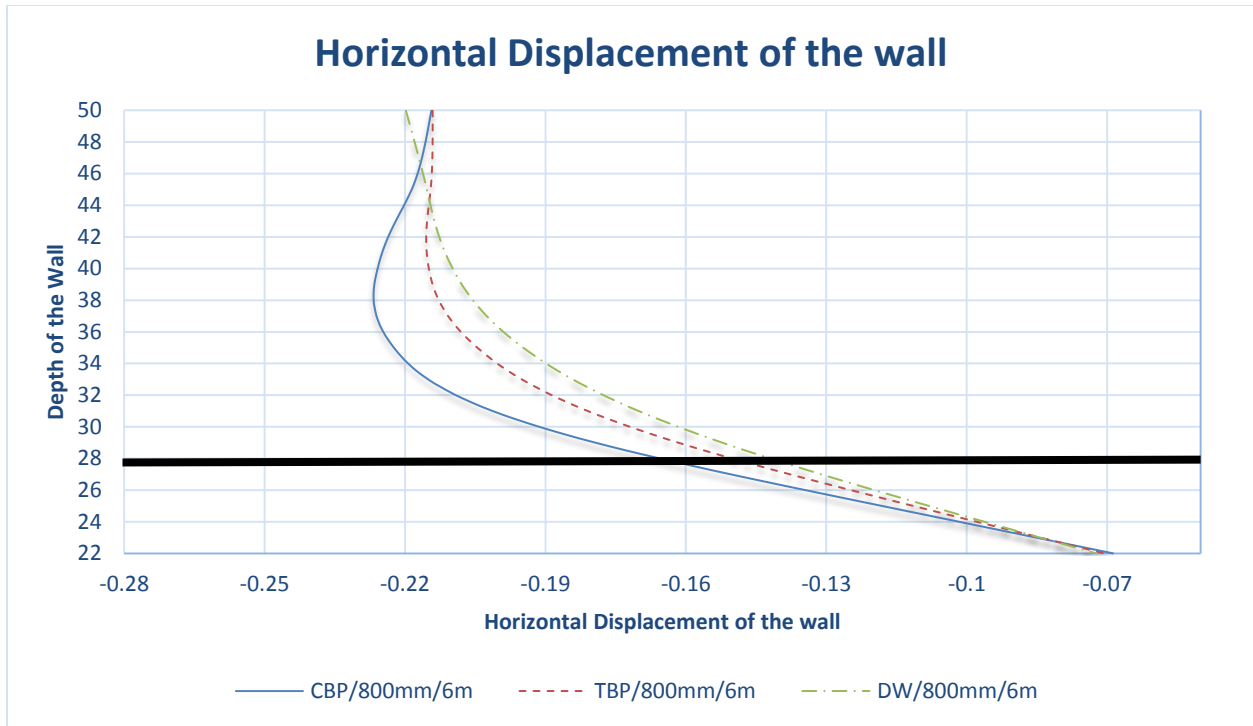


Figure 70 Horizontal Displacement Diagram of CBP, TBP and DW 800mm/6m

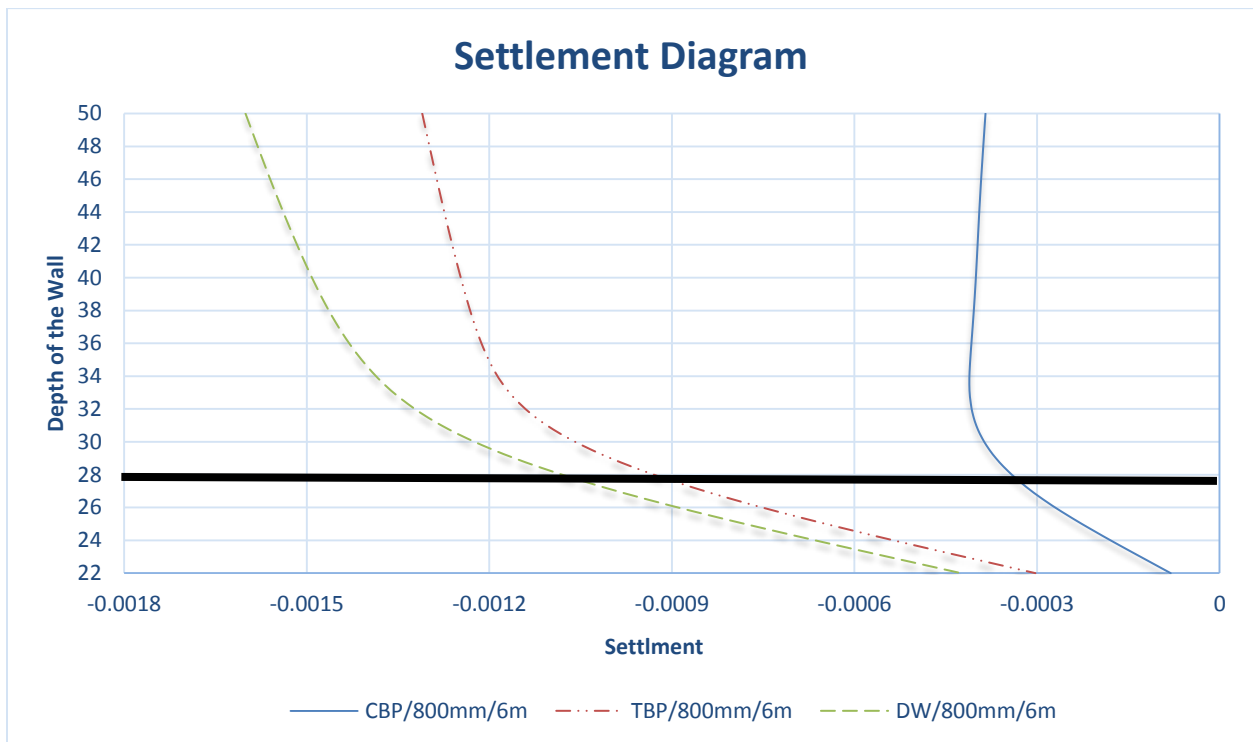


Figure 71 Settlement Diagram of CBP, TBP and DW 800mm/6m

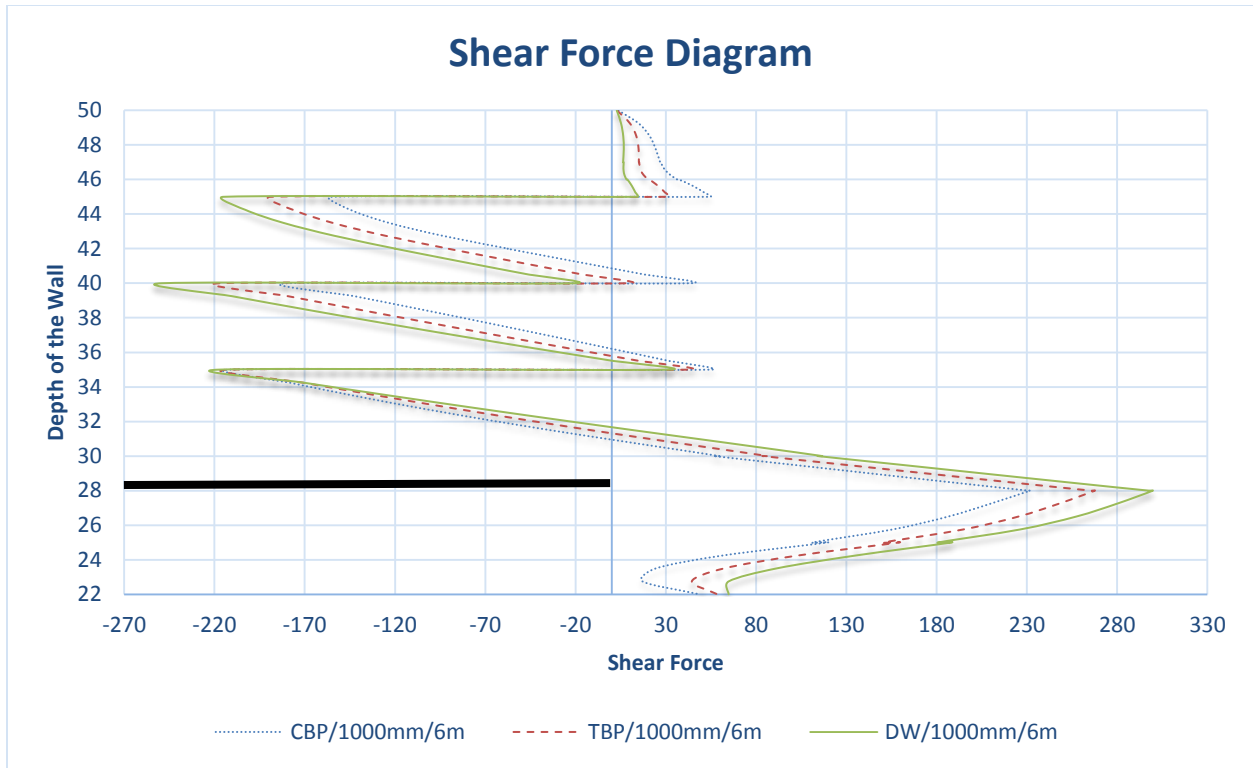


Figure 72 Shear Force Diagram of CBP, TBP and DW 1000mm/6m

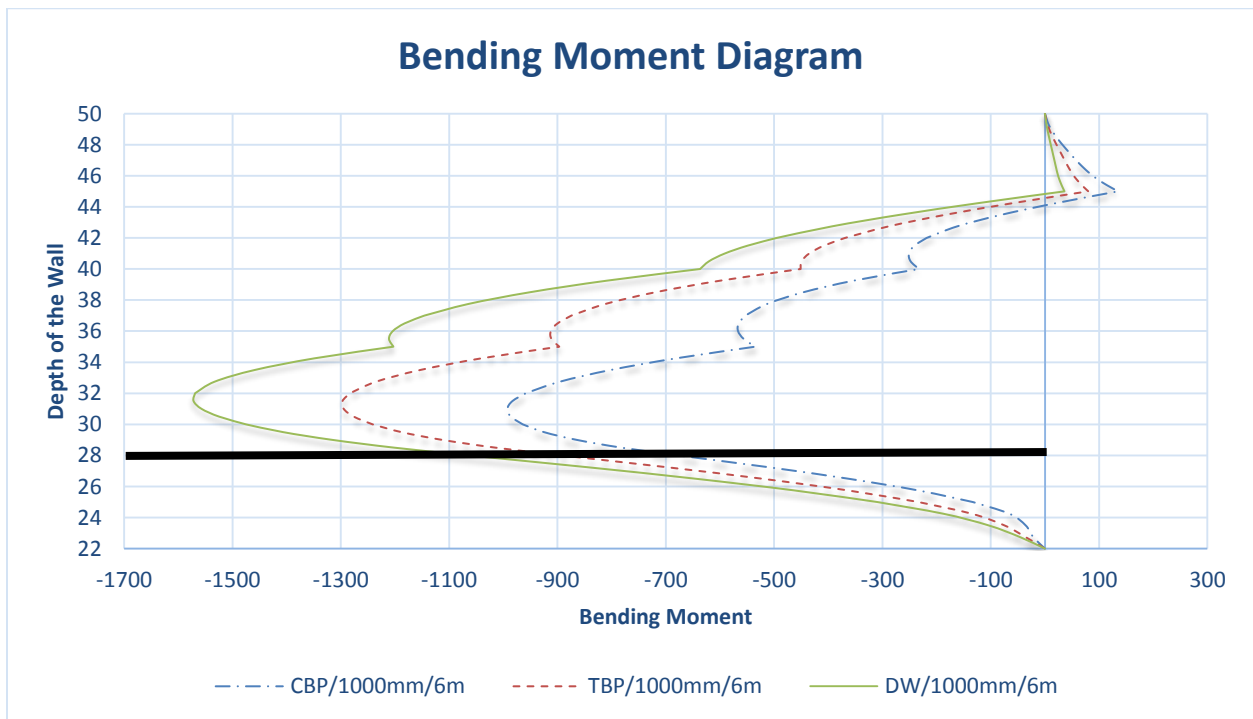


Figure 73 Bending Moment Diagram of CBP, TBP and DW 1000mm/6m

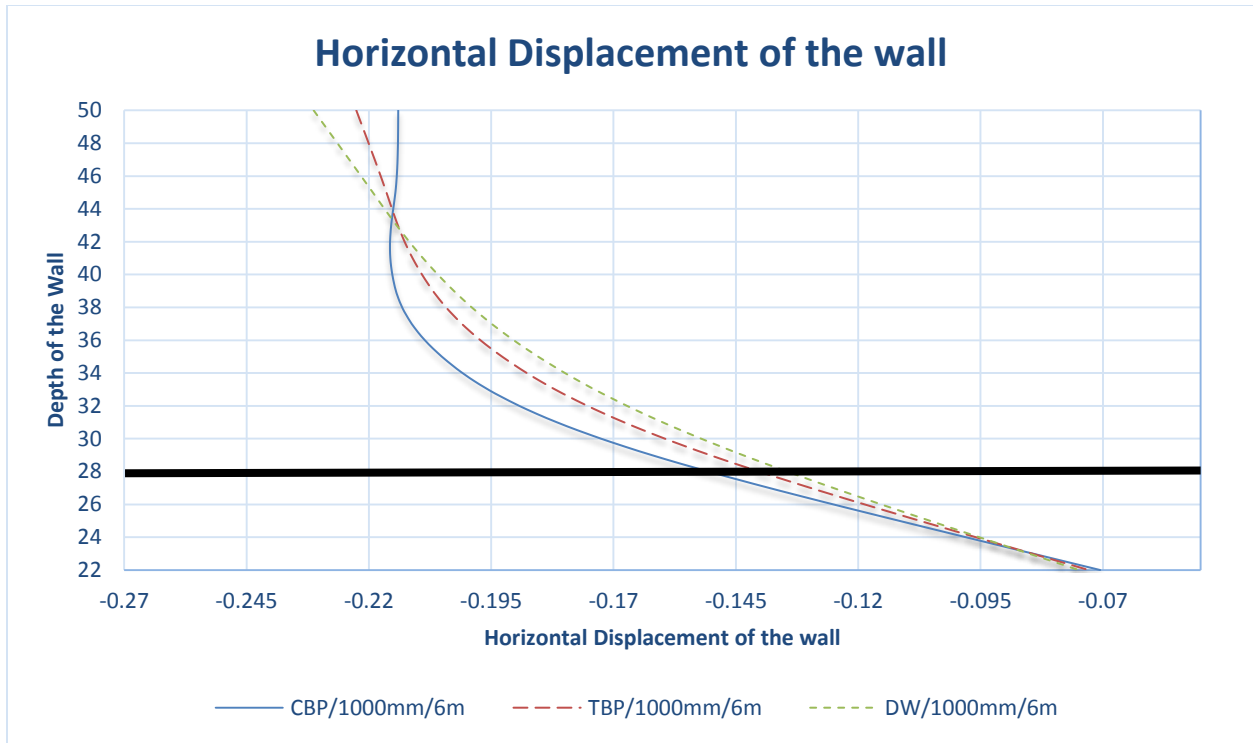


Figure 74 Horizontal Displacement Diagram of CBP, TBP and DW 100mm/6m

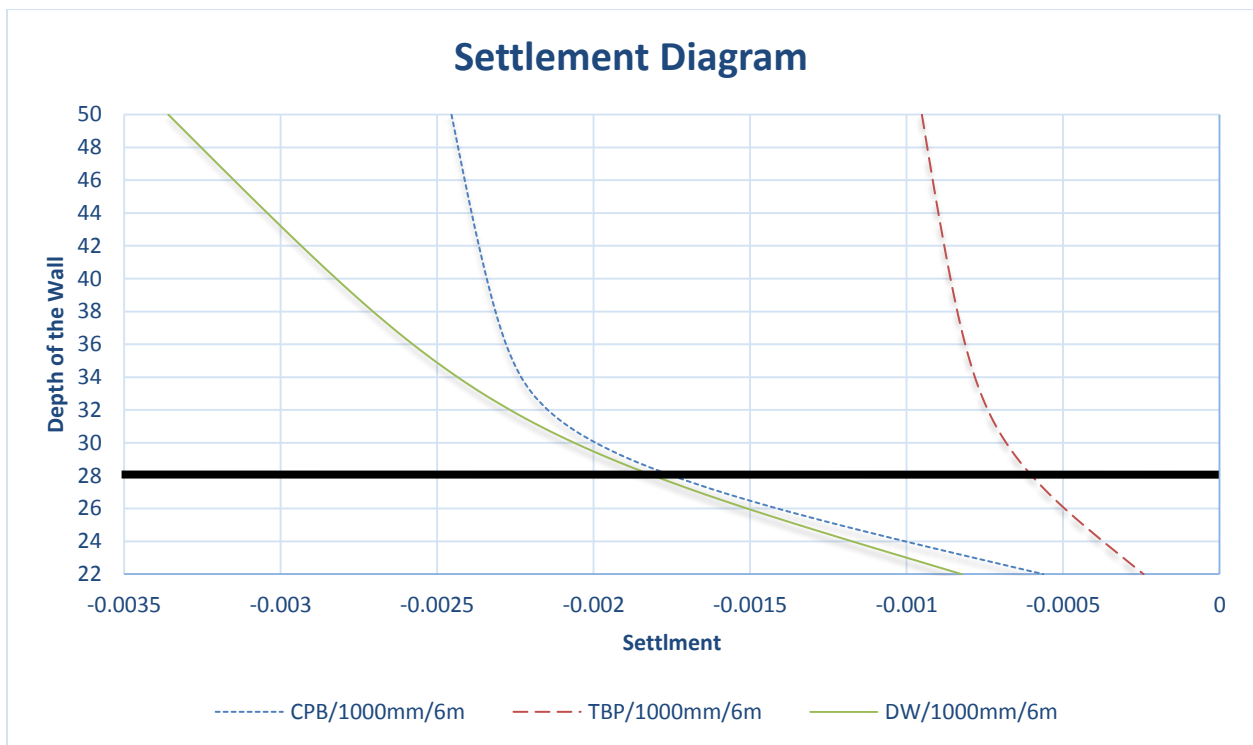


Figure 75 Settlement Diagram of CBP, TBP and DW 100mm/6m

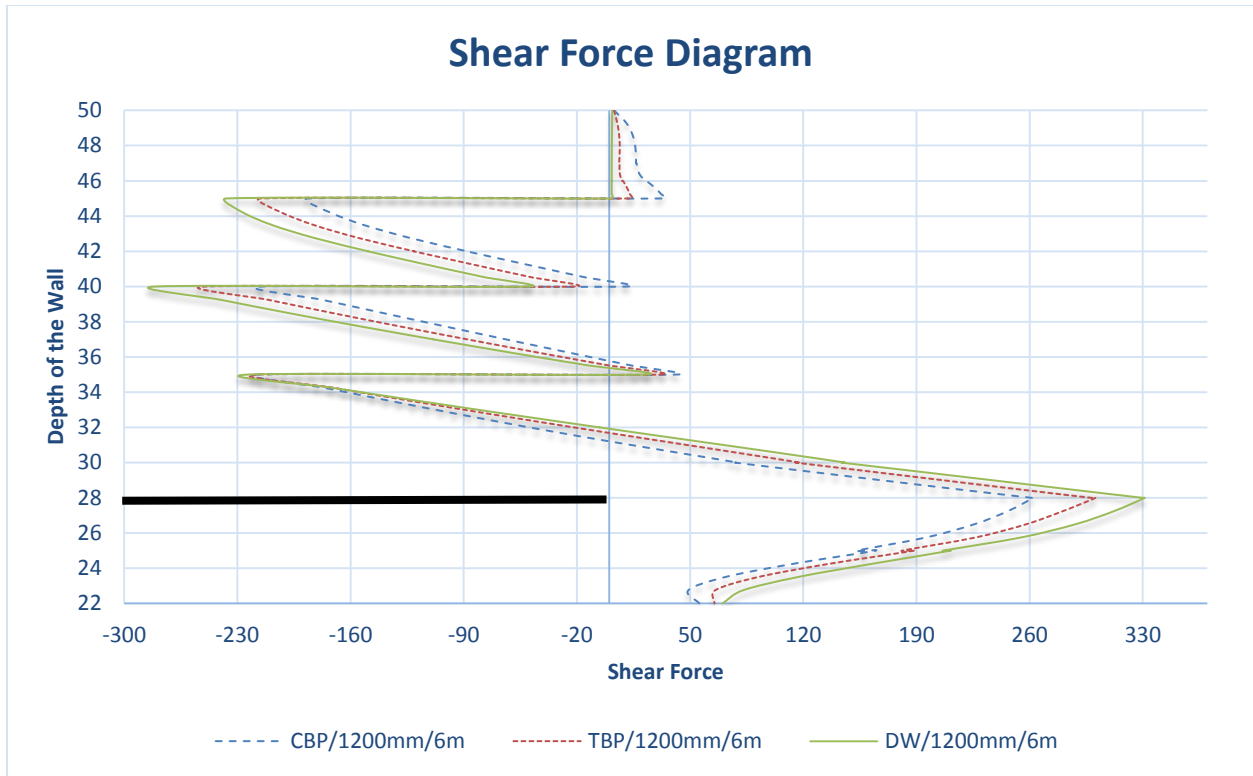


Figure 76 Shear Force Diagram of CBP, TBP and DW 1200mm/6m

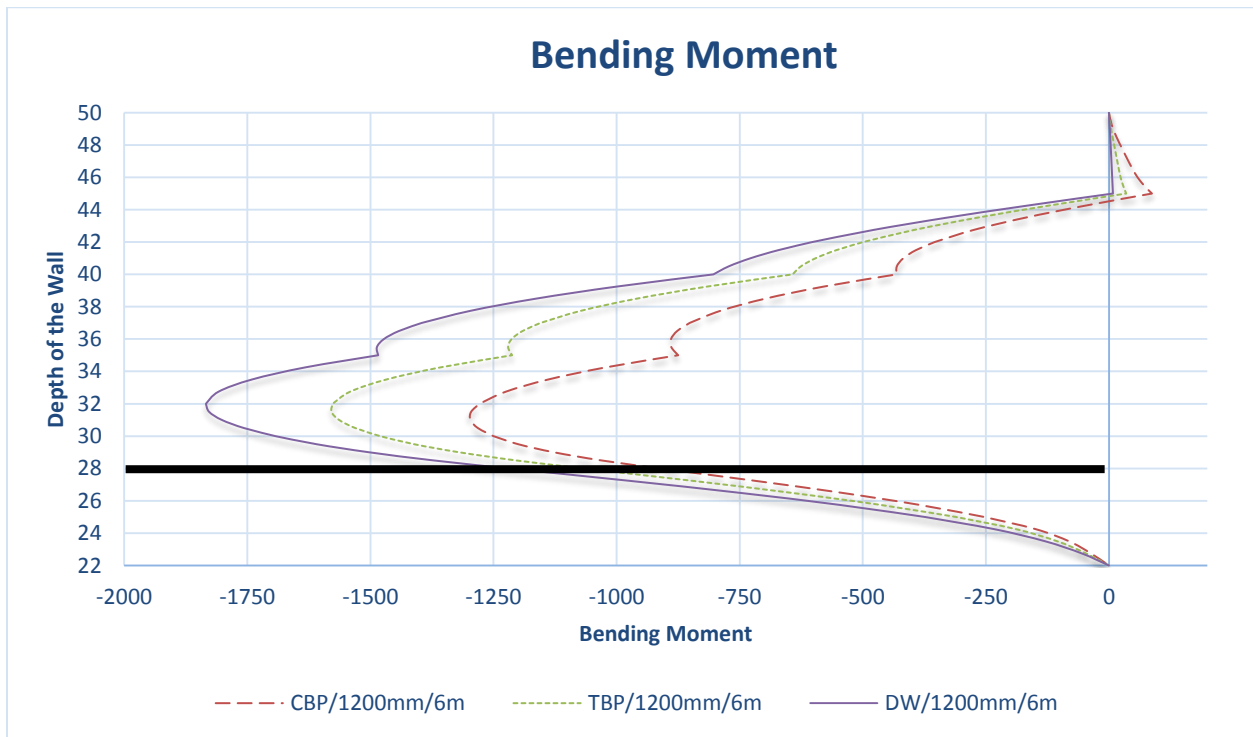


Figure 77 Bending Moment Diagram of CBP, TBP and DW 1200mm/6m

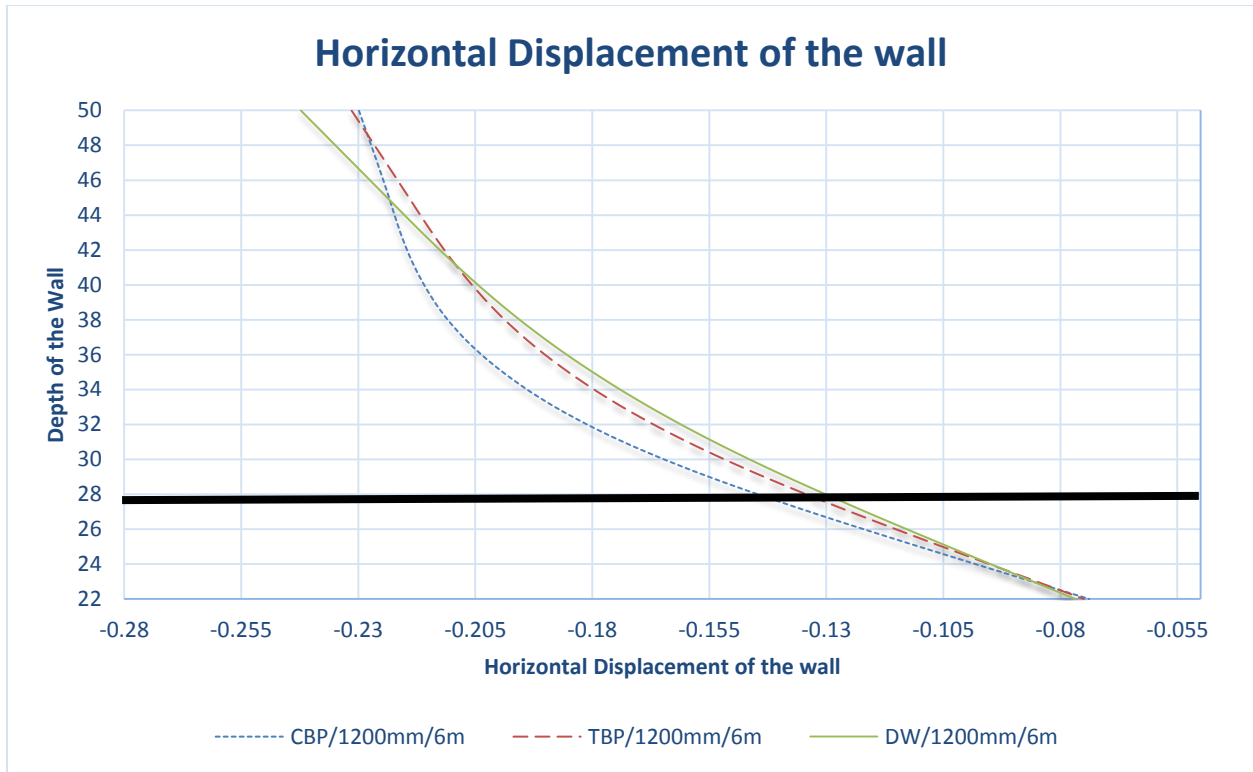


Figure 78 Horizontal Displacement Diagram of CBP, TBP and DW 1200mm/6m

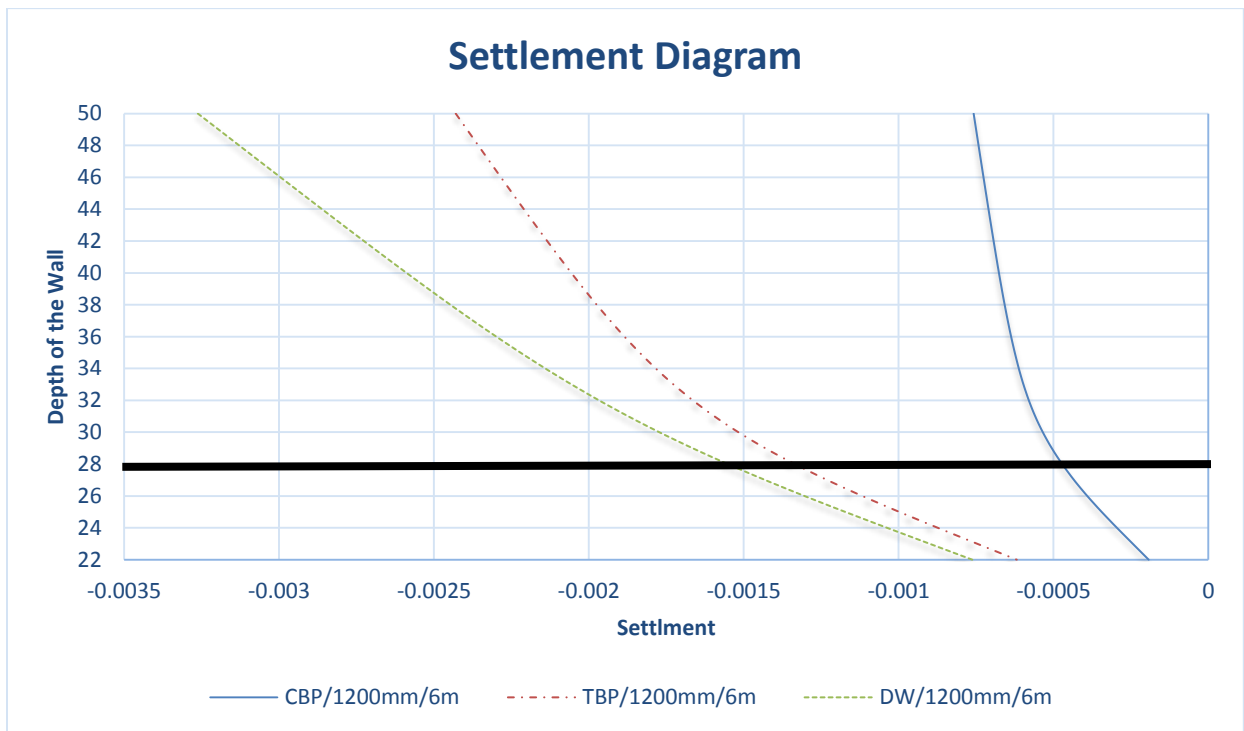


Figure 79 Settlement Diagram of CBP, TBP and DW 1200mm/6m

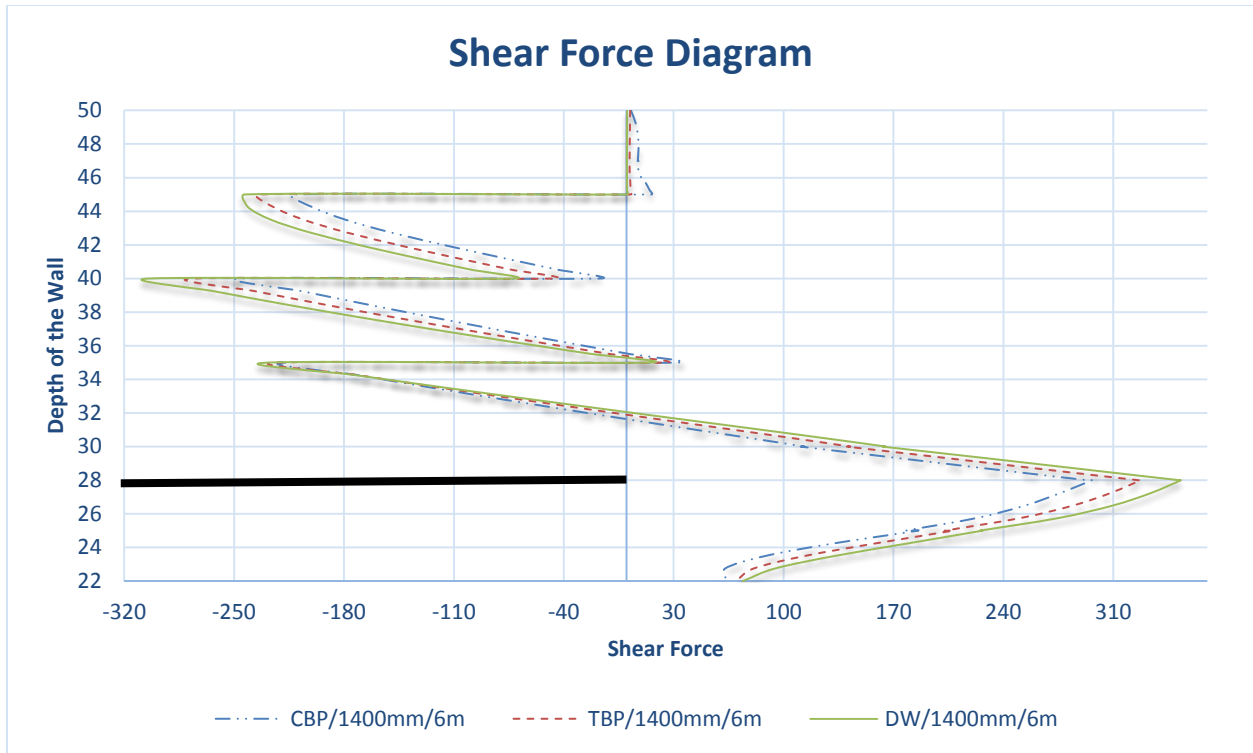


Figure 80 Shear Force Diagram of CBP, TBP and DW 1400mm/6m

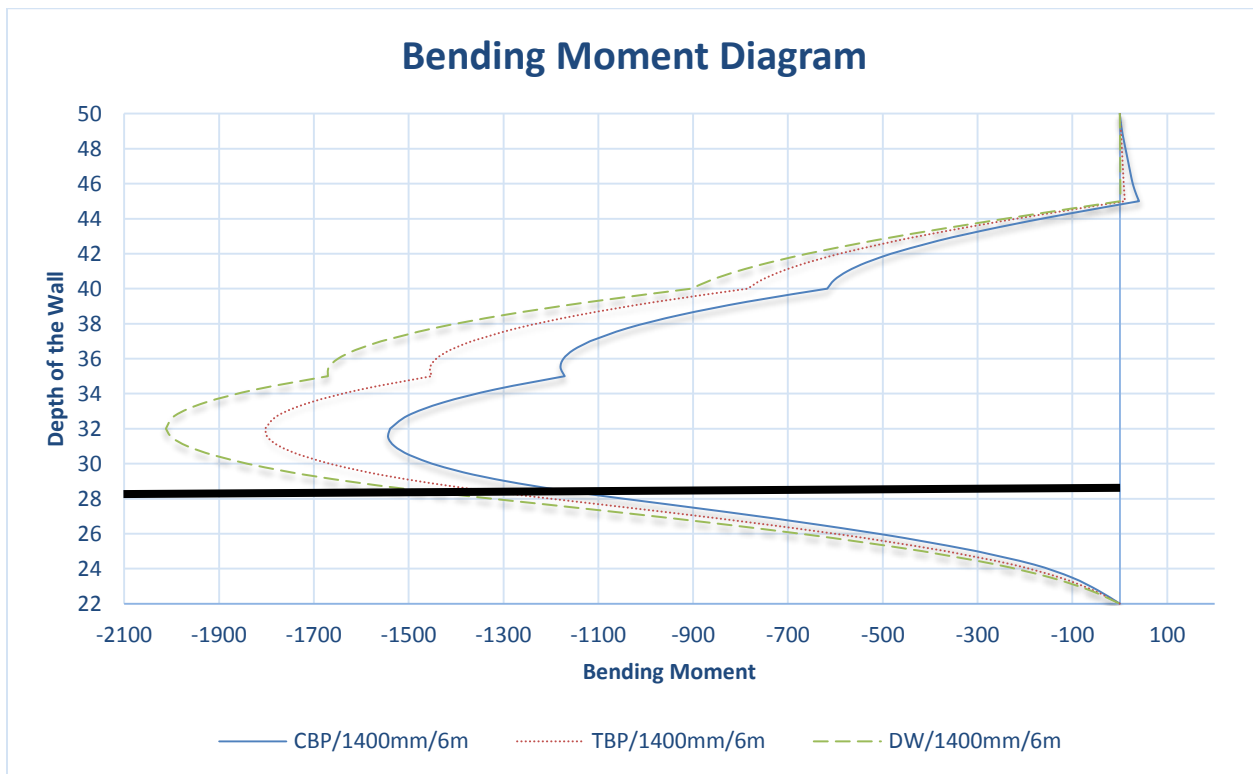


Figure 81 Bending Moment Diagram of CBP, TBP and DW 1400mm/6m

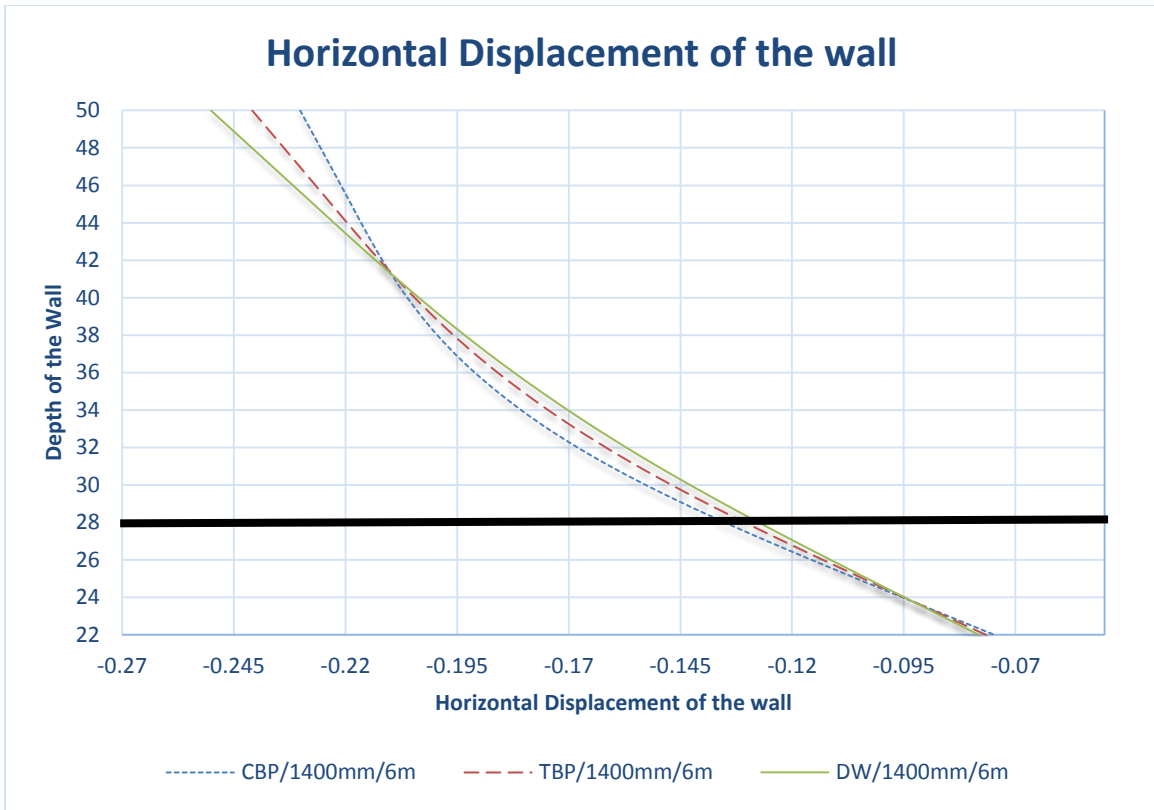


Figure 82 Horizontal Displacement Diagram of CBP, TBP and DW 1400mm/6m

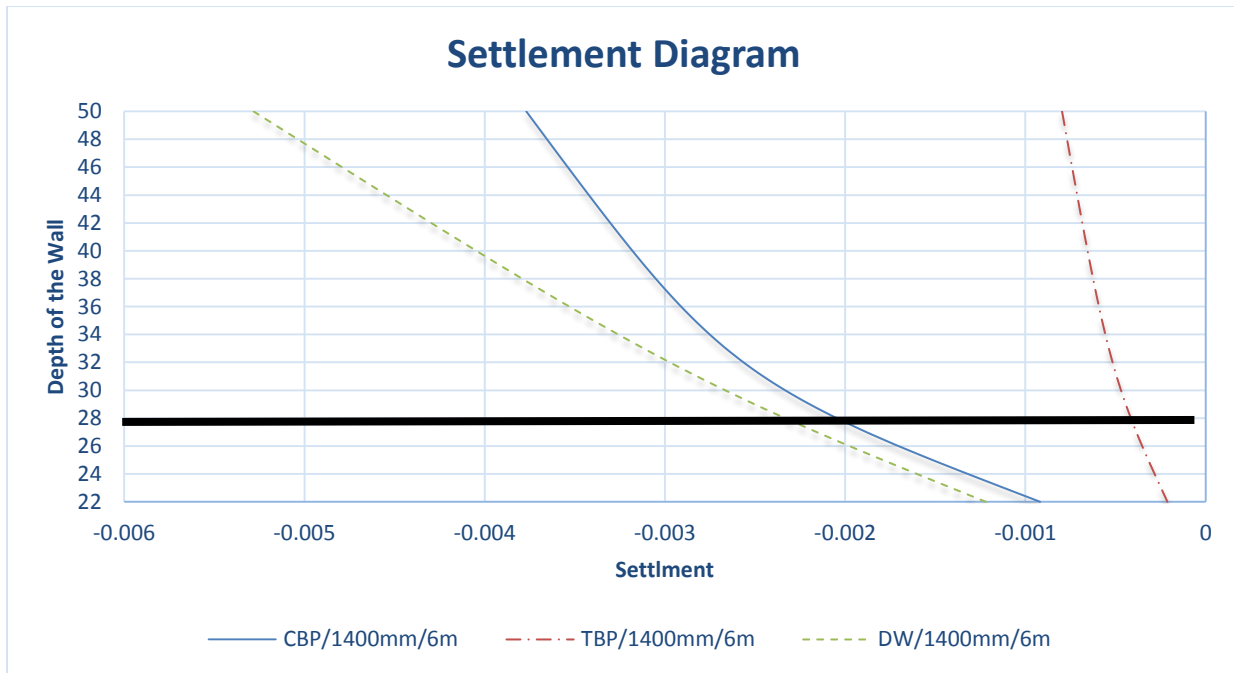


Figure 83 Settlement Diagram of CBP, TBP and DW 1400mm/6m

## VIII. Appendix – Two

### Unit Rate Break Down For Cost Analysis

In this section, additional pertinent Tables related to unit rate (cost break down) for those different walls (CBP, TBP and DW) are presented in details in order to the cost estimation that each walls worth.

### Concrete Take of Sheet

**Table 10 Contiguous Pile Shoring Pile Concrete Take-Off**

Pile No.	Pie Diameter , m	Concreted Length, m	Concrete Volume (m <sup>3</sup> )
1	0.80	26.00	13.06
	Total		<b>13.06</b>

**Table 11 Tangent Pile shoring Pile Concrete Take-Off**


Pile No.	Pie Diameter , m	Pie Area, m <sup>2</sup>	Concreted Length, m	Concrete Volume (m <sup>3</sup> )
2	0.80	0.50	26.00	13.06
1 & 3	-	0.10	26.00	2.60
	Total			<b>15.66</b>

**Table 12 Diaphragm Wall Shoring Pile Concrete Take-Off**


Pile No.	Pie Width , m	Pie Depth, m	Concreted Length, m	Concrete Volume (m <sup>3</sup> )
1	1.00	0.80	26.00	20.80
	Total			<b>20.80</b>

## Pile Rebar Take-Off

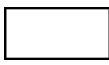
**Table 13 Contiguous Pile Shoring Pile Rebar Take-Off**

Concreted Length, m				Ø10mm (Transverse Bar)	
	Number	Length (m)	Total Length(m)	shape	Total Length(m)
22.00	19	22.80	433.20		346.89
			433.20		346.89
			2.469		0.617
			<b>1,069.58</b>		<b>214.04</b>

**Table 14 Tangent Pile Shoring Pile Rebar Take-Off**

Concreted Length, m				Ø10mm (Transverse Bar)	
	Number	Length (m)	Total Length(m)	shape	Total Length(m)
22.00	24	22.80	547.20		346.89
			547.20		346.89
			2.469		0.617
			<b>1,351.04</b>		<b>214.04</b>

**Table 15 Diaphragm Wall Shoring Pile Rebar Take-Off**

Concreted Length, m				Ø10mm (Transverse Bar)	
	Number	Length (m)	Total Length(m)	shape	Total Length(m)
22.00	24	22.80	547.20		581.40
			547.20		581.40
			2.469		0.617

	<b>1,351.04</b>	<b>358.73</b>
--	-----------------	---------------

## Anchor

**Table 16 Contiguous Pile Anchor Installation Take-Off Sheet**

No.	Anchor No.	Inclination, degree	Length, m
1	18	45°	13.00
<b>Total</b>			<b>13.00</b>

**Table 17 Tangent Pile Anchor Installation Take-off Sheet**

No.	Anchor No.	Inclination, degree	Length, m
1	18	45°	13.00
<b>Total</b>			<b>13.00</b>

**Table 18 Diaphragm Wall Anchor Installation Take-off Sheet**

No.	Anchor No.	Inclination, degree	Length, m
1	18	45°	13.00
<b>Total</b>			<b>13.00</b>

## Shotcrete

**Table 19 Contiguous Shotcrete Installation Take-off Sheet**

No.	Between Piles	Layer	Bay Height (m)	Bay Width (m)	Area (m <sup>2</sup> )
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Comparison of Retaining Structures Based on Their Performance (Case of Addis Ababa)

1	01	02	First	22.0	1.00	22.00
<b>Total</b>						<b>22.00</b>

### Shotcrete Rebar

**Table 20 Contiguous Pile Shotcrete Rebar Installation Take-Off Sheet**

No.	Between Piles		Bay width	Bay height	Horizontal			Vertical		
					L(m)	No. of bars	Tot. Length(m)	L (m)	No. of bars	Tot. Length(m)
1	01	02	1.00	22.0	1.00	148	148.00	22.3	6	133.92
Total							148.00			133.92
<b>Unit weight (Kg/m)</b>							<b>0.395</b>			<b>0.395</b>

## IX. Appendix - Three

### Summary of results for CBP, TBP and DW

In this section, additional relevant Table is incorporated to show how change in stiffness, the Embedment depth and wall type affect the performance of deep excavations in terms of Shear Force, Bending Moment and Lateral Wall Deflection in summarized way.

**Table 21 Summary of results**

Wall Type	Embedment Depth (m)	Thickness(mm)	Max. Shear Force (kN/m)	Max. Bending Moment (kN-m/m)	Max. Deflection (m)	Settlement (m)
CPB	4	800	195.31	780.00	-0.255	-0.005758
		1000	227.81	977.08	-0.276	-0.003124
		1200	256.71	1140.00	-0.294	-0.003294
		1400	278.61	1270.00	-0.308	-0.000908
	6	800	214.54	723.43	-0.227	-0.000411
		1000	231.80	992.50	-0.216	-0.002454
		1200	262.39	1300.00	-0.230	-0.000758
		1400	296.29	1540.00	-0.230	-0.003769
TPB	4	800	229.43	989.29	-0.277	-0.002730
		1000	259.34	1160.00	-0.296	-0.002964
		1200	281.60	1290.00	-0.310	-0.003511
		1400	296.67	1380.00	-0.320	-0.004203
	6	800	233.10	1000.00	-0.216	-0.001310
		1000	267.67	1300.00	-0.223	-0.000951
		1200	300.80	1580.00	-0.231	-0.002430
		1400	327.35	1800.00	-0.241	-0.000796
DW	4	800	235.00	1124.00	-0.293	-0.004318
		1000	280.76	1290.00	-0.309	-0.001228
		1200	298.98	1400.00	-0.321	-0.002883
		1400	309.59	1470.00	-0.328	-0.004349
	6	800	260.04	1230.00	-0.220	-0.001601
		1000	299.76	1570.00	-0.231	-0.003359
		1200	331.32	1830.00	-0.242	-0.003262
		1400	353.07	2010.00	-0.250	-0.005282