STUDY ON THE EFFECT OF DREDGING ON EXISTING SHEET PILE RETAINING STRUCTURES

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Sri Lanka

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Thesis submitted in partial fulfillment of the requirements for the degree Master of Science in Foundation Engineering and Erath Retaining Systems

Department of Civil Engineering

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Sri Lanka

March 2017

DECLARATION

I declare that this is my own work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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The above candidate has carried out research for the Masters under my supervision.

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ABSTRACT

Sheet pile retaining structures are widely used in many purposes in engineering designs. Most common applications are shoring, stabilize excavations, harbor quay wall structures, canal bank protection structures.. etc. This research is mainly focused on the issue of stability of harbor quay wall structure when deepening the harbor basin to cater larger vessels.

Most of the quay wall structures in Srilanka are anchored sheet pile walls. So the effect of dredging on the sheet pile structure is studied. If the same method can be applied for the cantilever sheet pile walls which could be used in drainage improving projects. The study is extended to both anchored and cantilever sheet pile structures to increase the stability while reducing the depth of embedment.

Another improvement sheet pile wall is proposed from the passive side of the existing sheet pile structure to provide an additional support to the main structure. The finite element analysis is used to estimate the effect of the improvement wall on the stability and the deflection of the wall in cohesion less soils. The effect of the distance between the existing structure and the improvement wall and the effect of increasing the depth of embedment of the improvement structure are analyzed through the finite element models. The models are tested for 22 to 38 degrees wide range of friction angles of cohesion less soils

The results of the finite element model are verified by a physical model conducted in a laboratory. The results of the analysis shows that the improvement wall can significantly increase the stability of the existing structure. Compared to anchored sheet pile walls higher improvement can be achieved for cantilever sheet pile walls from this method. Rather than replacing the entire structure , applying a this sort of improvement method will be highly economical as well as less damages to the other structures close to the existing sheet pile wall.

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1. INTRODUCTION

1.1 Introduction to the research

In common practice, the retaining height of cantilever or anchored sheet pile retaining wall is designed to the current requirement. But later on there can be a requirement for the increase of the retaining height of those retaining walls. This situation commonly occurs in the quay walls of the harbors and the canals with sheet pile bank protection. The entire retaining wall has to be reconstructed in most of such cases. It is associated with lot of difficulties such as halting of current operation, damages to existing structures and high cost. The existing depth of harbor basins are not enough mainly due to the increase in size of fishing boats. But harbor basin cannot be dredged as the loss of passive resistance would cause stability issues on the existing quay walls. Currently this issue has arisen in Dikowita fishery harbor and Mutwal fishery harbor. In view of the above, it is a timely study to investigate the alternative economical methods in increasing the depth of basins without causing instability in existing quay walls, propose a general guideline for the same

Same type of issues can occur in drains with sheet pile bank protection. According to the new flood studies canal size and bed levels might need to be changed. So in most of the cases existing sheet pile bank protection totally removed and reconstruct according to the new design. This process involve lot of money as well as it cause damages to the existing structures near by the drain/canal. It create lot of social and environmental issues as well. So it’s better to find a way to increase the depth of drains/canals while keeping the existing bank protection.

In view of the above, the following objectives have been identified.

1.2 Objectives

The objectives of the present study are as follows,

1) Conduct a thorough literature review on the area of study.

2) Experimental investigation on the effect on stability of existing sheet pile walls due to dredging

3) Conducting a parametric study on the effects of dredging on existing sheet pile walls using Finite Element approach.

4) Propose general guidelines on dredging without causing instability of existing sheet pile retaining structure.

1.3 Limitation of the Study

- The study is limited to cohesion less soil mediums to simplify the study area.

  Validation of the results of numerical analysis is carried out using a small model experiment. Field deformation measurement and subsurface characteristics data are required for the validation of the results of numerical analysis.

- Model verification is only done for cantilever retaining walls, due to difficulty of measuring micro meter level displacements

2. LITERATURE REVIEW

2.1 General

This chapter presents concepts, principles and previous studies carried out in analyzing stability of cantilever and anchored retaining walls. Limited number of researches had been carried out to investigate the solutions for the stability improving methods when reducing the depth of embedment.

2.2 Lateral Earth Pressure Theories

Lateral earth pressure is the lateral force exerted by the soil to an adjoining retaining structure. It is dependent on the soil, structure and the interaction of soil with the retaining structure. All of these theories assume plane strain conditions and depend on the theory of plasticity. The classical solutions of lateral earth pressure are Coulomb's (1773) and Rankine's (1857) earth pressure theories. These fundamental solutions still form the basis of earth pressure calculations today. All earth pressure theories now available have their roots in Coulomb and Rankine's work. (Coduto, 2001)

Active pressure (Ka): The soil exerts a pressure against the retaining wall. The wall moves towards the excavation while reducing the horizontal stresses, as the vertical stress remains unchanged. A decompression in the horizontal stress occurs. In a limit situation, a failure wedge is formed, producing a plastic regime.

Passive pressure (Kp): The retaining wall exerts a pressure against the soil. In this case the horizontal stress increases, while the vertical stress remains unchanged. The earth pressure is higher than in the at-rest state. In a limit situation, a failure wedge is formed but compared to the active case its dimensionally large.

2.2.1. Rankine's Earth Pressure Theory

Rankine's method (1857) of evaluating passive pressure is a special case of the conditions considered by Coulomb. Rankine's theory considers the state of stress in a soil mass when the condition of plastic equilibrium has been reached. In particular, Rankine made the following assumptions for the
derivation of earth pressure:

1. The soil mass is homogeneous and semi-infinite.
2. The soil is dry and cohesionless.
3. The ground surface is plane.
4. The back of the wall is smooth and vertical.

Using the Rankine earth pressure coefficients (Eqs. 1, 2) the pressures acting on the active (K_a) and passive (K_p) sides of the wall can be calculated.

$K_a = (1 - \sin\varphi)/(1 + \sin\varphi)$ (1)

$K_p = (1 + \sin\varphi)/(1 - \sin\varphi)$ (2)

2.2.1. Coulomb's Earth Pressure Theory

Coulomb's theory considers the stability of a wedge of soil between a retaining wall and a trial failure plane. The force between the wedge and the wall surface is determined by considering the equilibrium of forces acting on the wedge when it is on the point of sliding up or down the failure plane.

Friction between the wall and the soil is taken into account. The angle of friction between the wall and soil material, denoted by δ and a constant component of shear resistance or wall adhesion are considered.

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

$$K_{pC} = \frac{\sin^2(\alpha-\varphi)}{\sin^2(\alpha)\sin(\alpha+\delta)\left[1-\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2}$$

Where the angles α, β and δ, as seen in Figure 2.1, represent:

α: Inclination of the wall from the horizontal.
β: Inclination of the ground in the back of the wall.
δ: Friction angle between the wall and the soil.

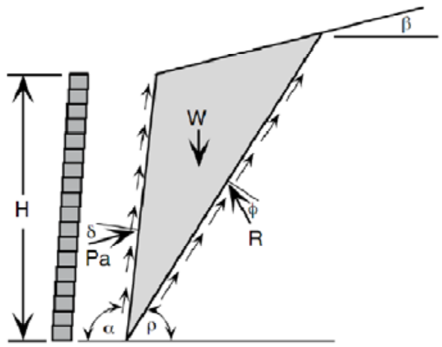

Figure 2.1 Coulomb Wedge Analysis

2.3 Design Methods of Sheet Pile Walls

There are several traditional design methods used to design anchored sheet pile walls; such as: free earth support method and fixed earth support method (Das 2011). The most common methods used in the United States and elsewhere are the free and fixed earth support methods.

2.3.1 Sheet Pile Wall Deformations

The deformations of sheet pile walls are very important, either cantilever or anchored walls. The total deformation of the wall can be due to unloading caused by the excavation area, elastic deformation of the wall, shear deformations of the earth body, and the soil movement below the wall (Smoltczyk 2003).There are no firm guidelines for acceptable deflection in sheet pile walls, and values ranging from 1 to 5 inches are typically considered acceptable. It is recommended that the deflection be limited to 1 to 3 inches for stream restoration and stabilization projects (National Engineering Handbook 2007). There are different methods used to reduce sheet pile wall deformations. while having multiple anchor levels is the most efficient way to reduce deformation in an anchored wall, Installation of larger pile profiles than the structural design requirement can also be very effective. A research showed that 50 percent reduction in anchored wall deformations is observed when using larger pile profile, and also more than 65 percent reduction is obtained when using second level of anchors. The installation of the anchor at 0.25 of the wall height from the top of wall can result in the lowest deformations compared with the deformations if the anchor is installed above or below this level. An increase in the anchor stiffness can also decrease the wall deformations. (Bilgin and Erten 2009)

2.3.2 Effect Of Wall Penetration Depth Of Sheet Pile Walls

Amer et. al. (2013) studied about the effect of wall penetration depth on the behavior of sheet pile walls. The effect of wall penetration depths for varying soil conditions and wall heights on both cantilever and anchored sheet pile walls behavior has been investigated by conducting a parametric study. The finite element method was used to perform numerical modeling and analyses to evaluate the structural response and behavior of the walls. The results show that increasing wall penetration depth can help to reduce wall deformations for both cantilever and anchored walls. Increasing wall penetration depth can also reduce wall bending moments significantly when the walls are anchored.

Maximum wall displacements and bending moments observed against normalized penetration depths for cohesive and cohesiveless soil types in sheet pile retaining walls is decreasing. Increasing wall penetration depth results in higher reduction in maximum wall displacements for relatively shorter cantilever walls compared to higher walls.

Increasing wall penetration depth for anchored sheet pile wall to depths deeper than the ones required by structure design can decrease maximum wall bending moments significantly . There are no benefits of increasing wall penetration depth for relatively high anchored walls in loose, medium dense and dense sand soils to depths more than 0.7, 0.5, and 0.6 of the wall height, respectively. There are no benefits of increasing wall penetration depth for relatively short anchored walls in loose, medium dense and dense sand soils to depths more than 1.0, 0.7, and 0.6 of the wall height. (Amer et. al. 2013)

Anchored sheet pile walls and Cantilever sheet pile walls both show increase of wall displacement when reducing the wall penetration depth. In order to keep the wall displacements within acceptable limits there is a need of a methodology to increase the passive resistance on the retaining wall.

2.3.3 Berms For Stabilizing Earth Retaining Structures

Youssef Gomaa and Youssef Morsi (2003) studied about berms for stabilizing earth retaining structures. This study was limited to cantilever sheet pile retaining walls in cohesion less soil. The finite element program (PLAXIS) is used to estimate the effect of berm on the stability and deflection of the wall in cohesion less soils and Computer programs written in FORTRAN77 code were used for graphical solution and approximate methods. All methods of analysis indicate that the use of a berm has a significant effect on reducing moment and deflection. Mainly two type of berms were studied in the above study.

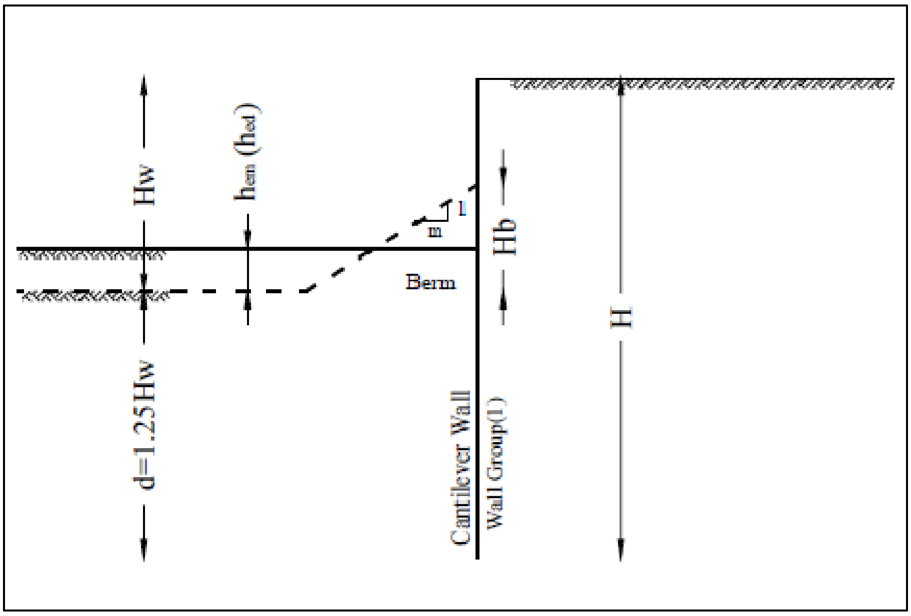

Figure 2.2 Geometry of Bermed Wall with Zero Top Width

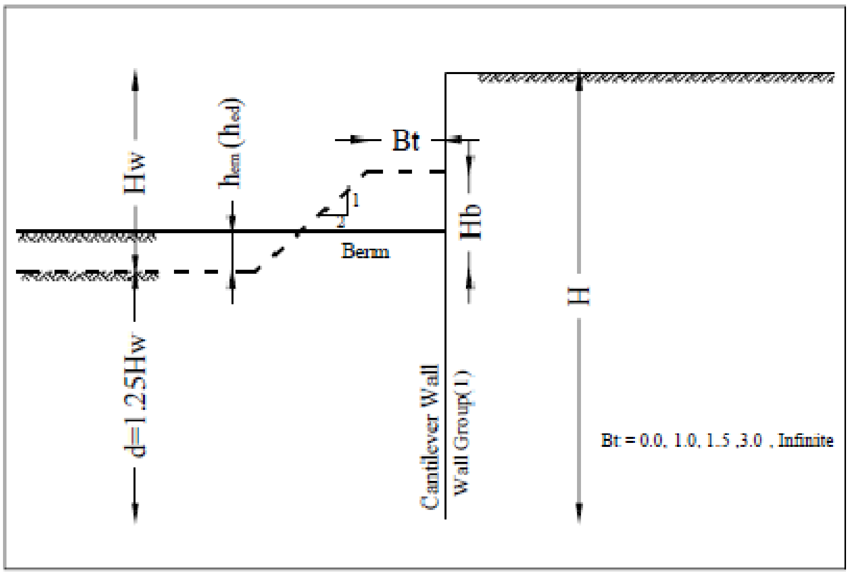

Figure 2.3 Geometry of the Bermed Wall with Different Top Berm Widths

Youssef Gomaa and Youssef Morsi (2003) showed that the berm has a significant effect on the stability of the wall. The most important berms factor influencing stability is the berm height as well as the berm width.. This effect causes reduction in the maximum moment on the wall by about 50% when the ratio of berm to wall height is 0.60. The slope of berm has small effect on the stability especially when slope is more than 1: 4.

2.4 Finite Element Analysis

The finite element method is one of the most powerful approximate solution methods that can be applied to solve a wide range of problems represented by ordinary or partial differential equations. The power of such a method derives from the fact that it can easily accommodate changes in the material stiffness, which is evaluated at element level. It allows different boundary conditions to be applied

PLAXIS is a special purpose two-dimensional finite element computer program used to perform deformation and stability analyses for various types of geo technical applications. It was introduced by technical university of Delft in 1987 as an initiative of the Dutch department of public works and water management. Real situations can be modeled either by using plain strain or axisymmetric model. PLAXIS uses a convenient graphical user interface that enables users to quickly generate a geometry model finite element mesh (Aziz, 2010; “Reference manual”, 2011). Brinkgreve and Broere (as cited in Aziz, 2010) mentioned that PLAXIS is a finite element program for geo technical applications in which soil modes are used to simulate the soil behavior.

Plaxis is mainly a two-dimensional program for statically computing but there are also additional versions of the program which can calculate dynamical models.

Generally advanced constitute models are required to simulate the nonlinear, time dependent and anisotropic behavior of soil and rock. And also special procedures are required to deal with hydrostatic pore – pressure in the soil.

2.4.1 Soil Elements

Plaxis provides 6-node and 15-node triangular elements to model soil, Since the 15 -node triangular element involves 12 stress points and it provides a fourth order interpolation for the displacement. The 15-node triangular elements give higher accuracy than 6-node element. Thus, the 15-node element is used in the analysis.

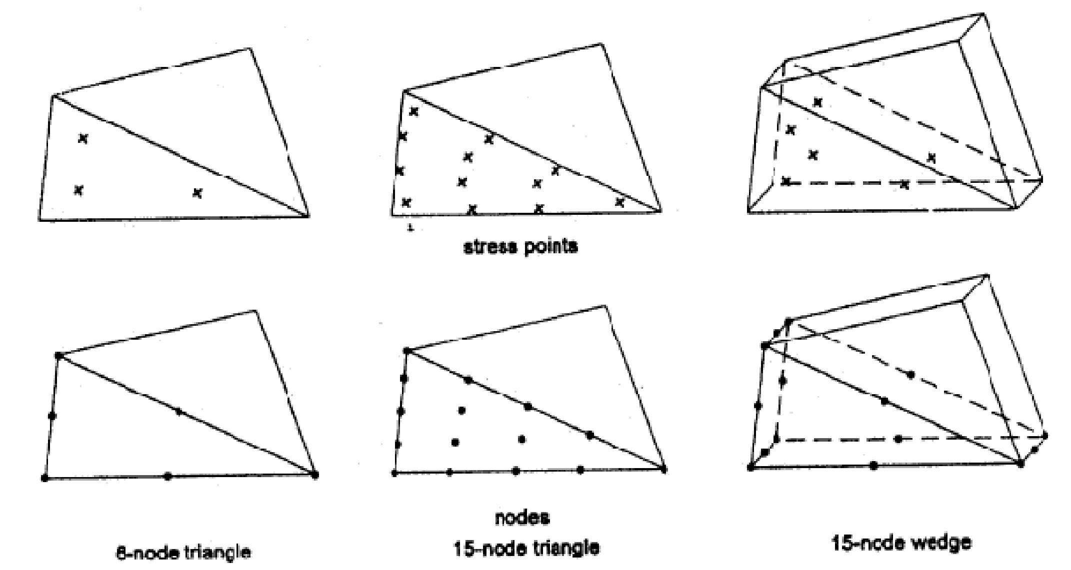

Figure 2.4 Position of Nodes and Stress Points in Soil Elements

2.4.2. Plate Elements

Plate elements are used to model the sheet pile retaining structure. Although plate elements are actually one-dimensional elements, the plates represent real plates in the out-of plane direction and can therefore be used to model the retaining structure such as diaphragm and sheet pile walls. The 5-node beam elements are used to be compatible with the 15-node triangular elements used to modelling the soil. The positions of nodes and stress points in a 5-node beam element are shown in Figure 2.5. The parameters used to define the plate element are the flexural rigidity, (EA), axial stiffness, (EI), Poisson's ratio, (ν) and the weight per unit area, (w).

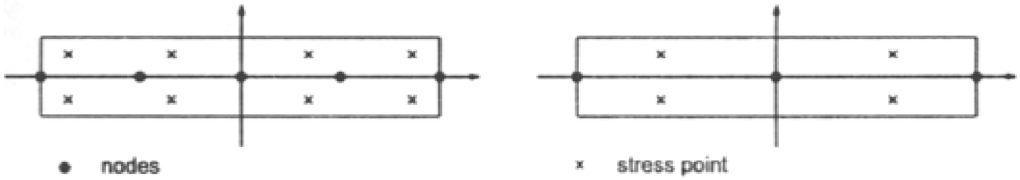

Figure 2.5 Position of Nodes and Stress Points in a 3-node and 5-node Plate Element

2.4.3 Interface Elements

The interaction between the retaining structure and the soil need to be modelled with the interface elements. The interfaces are placed at both sides of the sheet pile structure. The soil structure interaction is modelled by choosing a suitable value for the strength reduction factor. This factor is given base on the wall friction, adhesion and cohesion of the soil. The interface elements are defined by 5-pairs of nodes to connect between the soil element and beam elements. How interface elements connected to the soil elements is shown in Figure 2.6 . The interface elements have zero thickness. The properties of the interface element is defined related to the strength of the soil through the strength reduction factor.

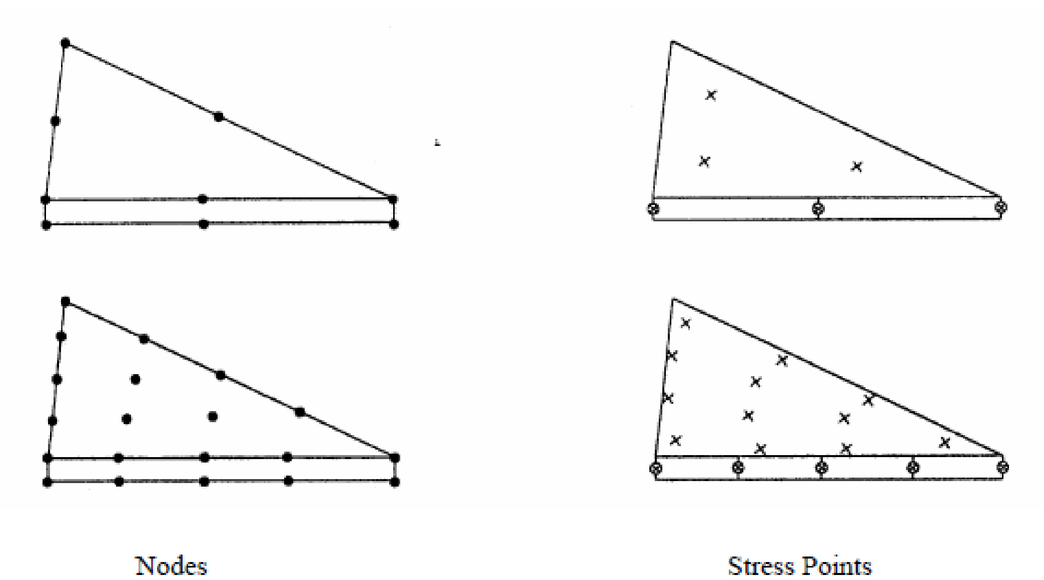

Figure 2.6 Distribution of Nodes and Stress Points in Interface Elements and Connection with Soil Elements

2.4.4 Material Models

Material models are used to represent the soil behavior qualitatively, and model parameters are used to quantify the soil behavior. Material model is a set of mathematical equations that describes the relationship between stress and strain. They are often expressed in a form in which infinitesimal increment of stress are related to infinitesimal increment of strain ("Material model manual v8", 2011).
There are set of material models available in PLAXIS and Mohr-Coulomb material model is selected to model the behavior of the sheet pile in cohesionless soil

2.4.4.1 Mohr-Coulomb model

Mohr-Coulomb model is a simple, robust, nonlinear model and it represents a first order approximation of soil or rock behavior. In PLAXIS Mohr-Coulomb model uses an elastic perfectly plastic constitutive model for three dimensional state of stress. Stiffness behavior before reaching the local shear strength is poorly modeled in the Mohr-Coulomb in PLAXIS, where it assumes the stiffness behavior to be linear elastic below the failure surface. However strength behavior is modeled better in Mohr-Coulomb model (Ehsan, 2013; "Material model manual v8", 2011).

Ehsan (2013) mentioned that the use of effective strength parameters in un-drained analysis of Mohr-Coulomb model may result in an over estimation of the shear strength of the material in un-drained conditions. Pickles (as cited in Ehsan, 2013) found the difference between effective stress paths for Mohr-Coulomb model and real soil as shown in Figure 3.7.

Mohr-Coulomb model basically requires five parameters,

- Young's modulus
- Poisson's ratio
- Friction angle
- Cohesion
- Dilatancy angle

And also Mohr-Coulomb model consists with some advanced parameters. These advanced features comprise of the increase of stiffness and cohesive strength with depth and the use of the tension cut-off. Tension cut-off can be used for the situations where soil has failed due to tension instead of the shear.

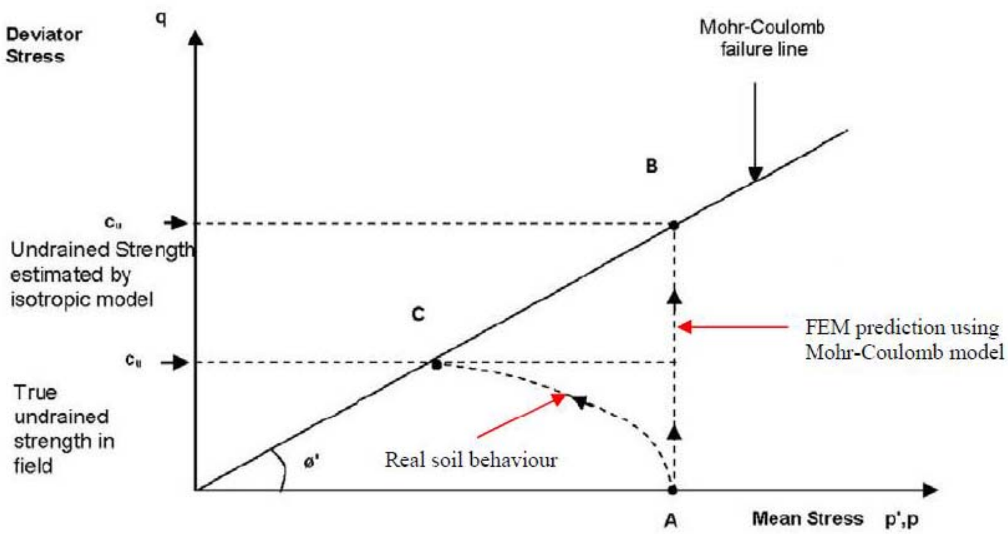

Figure 2.7 Effective stress paths followed in real soil and MC model

Source: Ehsan ,2013

3. METHODOLOGY

This chapter includes the methodology implemented to achieve the objectives of the study.

Previous studies about stability improvement of sheet pile walls were studied. The behavior of the sheet pile quay walls during the operational condition was studied. Current issues associated with sheet pile quay walls and the future requirements of the fishery harbor sector were gathered from the fishery harbor department. When studying the harbor quay walls Dikowita fishery harbor (currently in operation) , Mutwal fishery harbor (currently under some improvements) and Wennappuwa fishery harbor (Currently in detail design stage) were taken in to consideration. The study is limited to the Dikowita fisher harbor since the deepening of the harbor basin is a current requirement to expand the harbor operation.

Mainly there was a requirement of deepening existing harbor basins to cater newly built fishing multiday boats. Currently in Dikkowita fishery harbor there are jetties with 3.5 m depth and 4.5 m depth. All main jetties have anchored sheet pile quay walls which were designed for the requirement at construction stage. But newly built multiday boats and dredgers require more draft to navigate safely. Deepening the harbor basin means that reducing the passive pressure of existing quay walls, So It is required to re check the stability of existing structures and need to proposed a solution to minimize the risk of failure and control the deflection in the existing structures. It is proposed to drive another sheet pile raw in front of the existing anchored sheet pile wall. Which will increase the stability and reduce the deflection after deepening the harbor basin.

Cantilever sheet pile canal bank protection is a common application in Srilanka. Due to increasing flooding threat in many areas there is a requirement of widening and deepening the existing canals. Drain improvements proposed in Galle flood mitigation project was studied with existing sheet pile bank protection. Galle municipal council area is highly sensitive to floods due to low elevation in the terrain and filling of existing wetlands due to high urbanisation. According to the new flood studies carried out by Lanka hydraulic institute it was recommended to improve the existing canal system as well as to introduce new drains. Major issue of improving existing drains in urbanize area is damages which can cause to adjacent structures when removing the existing bank protection. So the possibility of improving the drains without damaging the existing bank protection is studied under this study. But it is limited to the drains with cantilever bank protection and the required drain improvement is only the deepening of the canal.

The research is not based on a specific case but to propose a general guild to the application of this improvement method. The behavior of the cantilevered and anchored sheet pile walls need to be studied when reducing the passive side soil support. Then the effect of introducing new improvement to those cases studied varying soil parameters as well as the location of the improvement wall. All these cases are studied through a numerical model of finite element analysis. So it is necessary to validate the finite element model for a similar case.

A physical model of a cantilevered sheet pile wall is conducted to validate the finite element analysis of the study. A particular cohesion less type of soil is selected and laboratory tests are performed to find the soil parameters. The physical model is simulated in the finite element model with the measured parameters to observe the validity.

The developed models of cantilevered or anchored sheet piles walls can be used for any application and since the modeling has been done for wide range of soil parameters results can be used as a general guild for designing an improvement to a existing structure.

General procedure of the research can be summarized in to a chart as shown in Figure 3.1

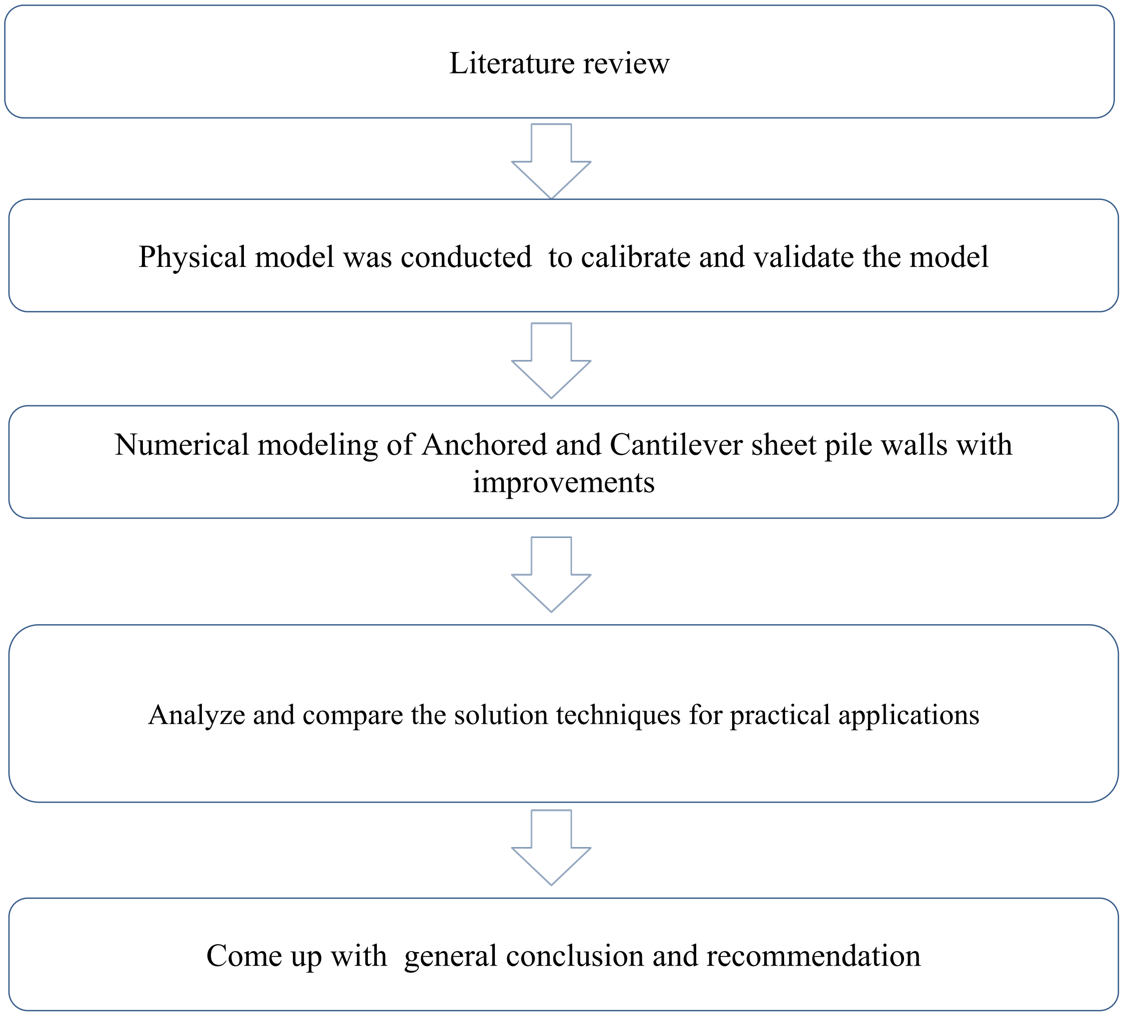

Figure 3.1 Methodology

4. PHYSICAL MODEL STUDY

4.1 Introduction

Physical model study is performed for the comparison of the finite element analysis done for sheet pile walls. This study is conducted to compare the wall deflection values observed from the physical model and deflection values obtained from numerical analysis. 0.8 m wide and 1.0 m height physical wave model testing flume is used as the testing tank. Displacements were measured using sensitive dial gauges and using image analysis.

The cantilever sheet pile is modeled in the flume for one type of sand. The physical and mechanical properties of the sand are tested in the laboratory before the physical model.

4.2 Cantilever Wall Model

Specially prepared plywood sheet is used as the sheet pile in the physical model.

* Thickness of the sheet pile is 15 mm
* Length of the Sheet pile wall is 400 mm
* Width of the Sheet pile wall is 800 mm
* 0.8 m wide wave flume is used for the model

4.3 Physical and Mechanical Properties of the Sand

Since the wave flume is very long, sand is filled few meters to both sides of the sheet pile wall to minimize the boundary effect. So large amount of uniform type of sand is needed for this study. Sea sand is used after removing seashells and large particles for the physical model.

4.3.1 Grain Size Distribution

This test is performed to determine the percentage of different grain sizes contained within the soil. Mechanical analysis is performed according to the Standard Reference: ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils by dry sieving. The gradation curve of the representative sample is shown in Figure 4.1

Tested sample is fine to medium sand with mean average grain size, D50 = 0.2 mm and having a mean coefficient of uniformity, D60/D10 = 1.7. So the sand is considered to be uniform sand.

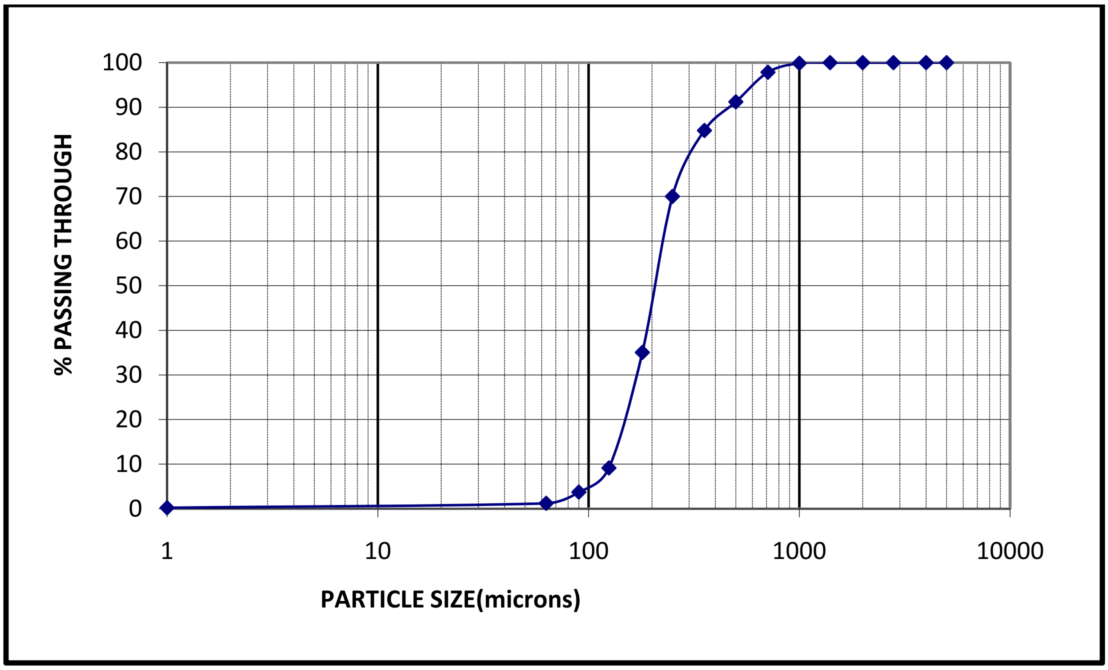

Figure 4.1 The Gradation Curve

4.3.2 Specific gravity, Dry Unit Weight and Saturated Unit Weight of Sand

Six samples were taken from the testing tank to determine the specific gravity and bulk density of the sand. Average specific gravity observed is 2.64.The average bulk density calculated of the sand was 15.52kN/m^3 and saturated unit weight is 17.01 kN/m^3.

4.3.3 Shear Strength of Sand

The shear strength of the sand need to be determined to used in numerical analysis. So the direct shear test is conducted to obtain the angle of friction of the sand.

4.3.3.1 The Direct Shear Tests

The 100mm X 100mm squire standard shear box is used for these tests. The shearing force is applied under four different normal pressures and the standard procedures followed to determine the friction angle of soil. Figure 4.2 shows the test results of the four tests.

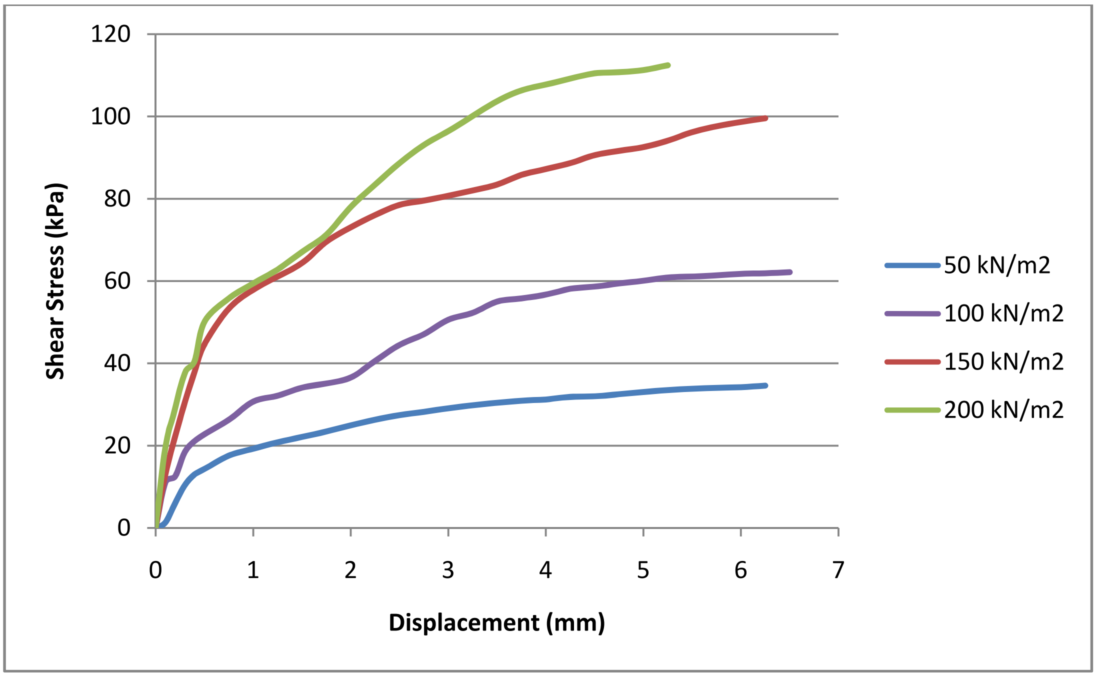

Figure 4.2 Displacement Vs Shear stress

Shear stress Vs diagram is shown in Figure 4.3

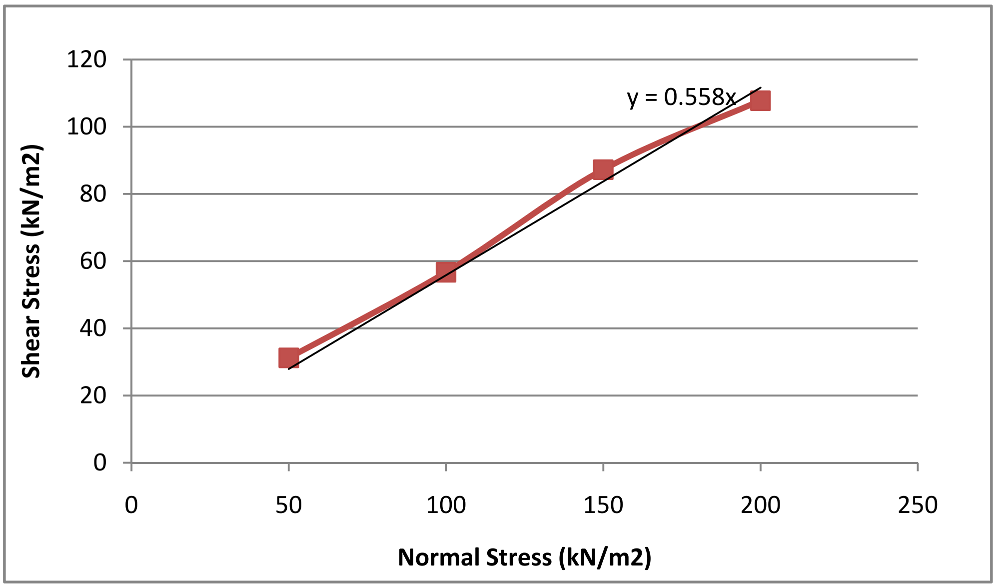

Figure 4.3 Normal stress Vs Shear stress

According to the test results , the internal friction angle of the soil is 29.2 degrees. Stiffness of the soil is estimated from the direct shear test assuming poison's ratio of 0.2 .

$G = \frac{E}{2(1+\gamma)}$

G - Shear modulus

E - Elastic modulus

γ - Poison's ratio

Average stiffness of the soil is taken as 5500 kN/m^2

4.4 Physical Model Arrangement

The physical model was conducted in a 0.8 m wide wave flume in Lanka Hydraulic Institute.

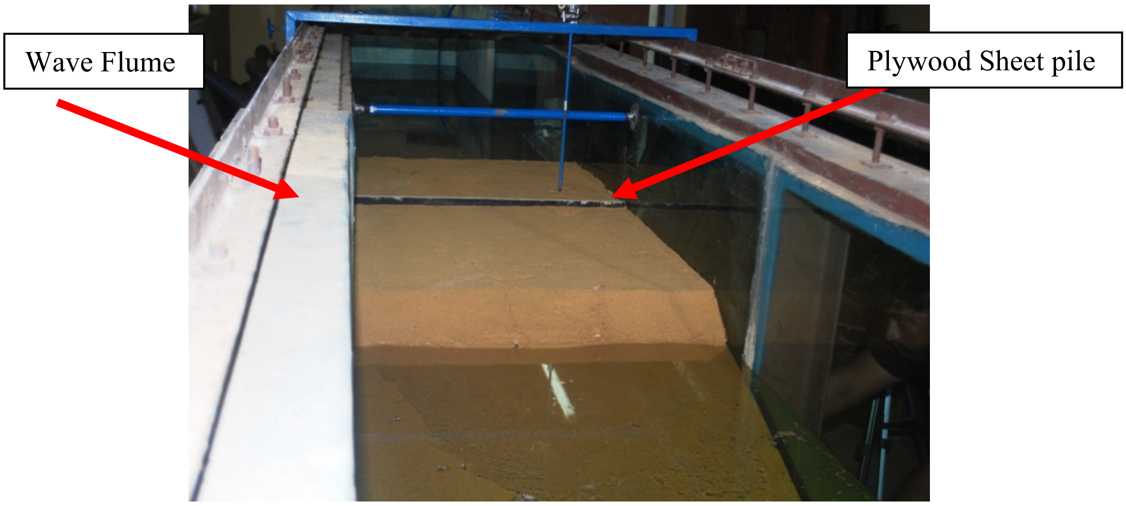

Figure 4.4 Physical Model Arrangement

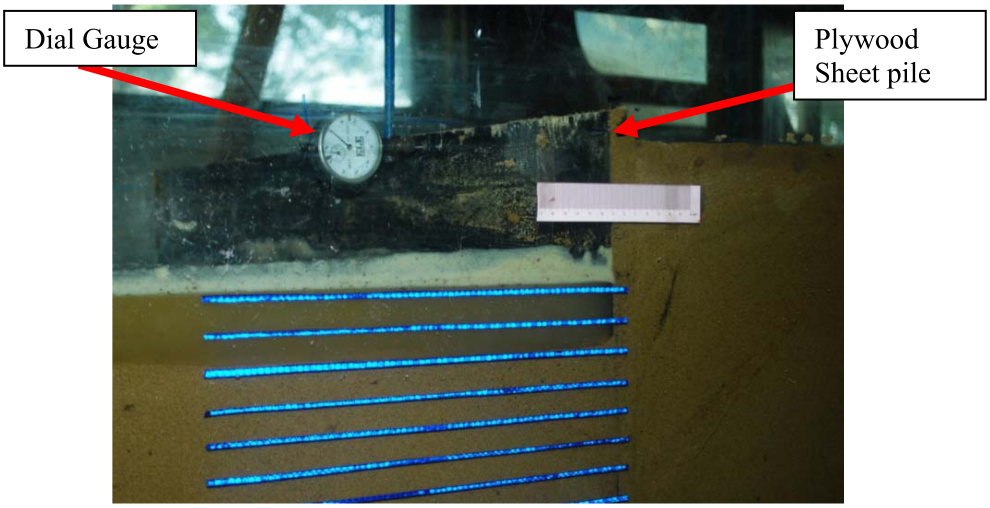

Figure 4.5 Dial Gauge Placement And Soil Layers To Be Removed

4.5 Adopted Methodology

The physical model test is preformed on one type of sand which was initially tested in the laboratory. A loose packing of sand could be obtained by depositing the sand in to the flume in horizontal layers of 100 mm thick each. The sand is deposited from constant height of 400 mm. Static water level at both sides of the sheet pile wall is maintained at 10 cm below from the top level of the sheet pile wall.

A dial gauge is set as shown in the Figure 4.5 and cameras are set with a scale to get photometric measurements to verify the dial gauge readings. The horizontal deflection of the top point of the wall is measured after removal of one layer (20 mm) of sand. Measurements are taken after few minutes from the finishing of the excavation. Sand removal is proceeded until sheet pile structure fails and measurements are recorded from 10 cm excavation depth. recorded dial gauge reading values are checked again with the photometric values and plotted against the excavation depth.

4.6 Results Observed

Observed results of the physical model is shown in Figure 4.6

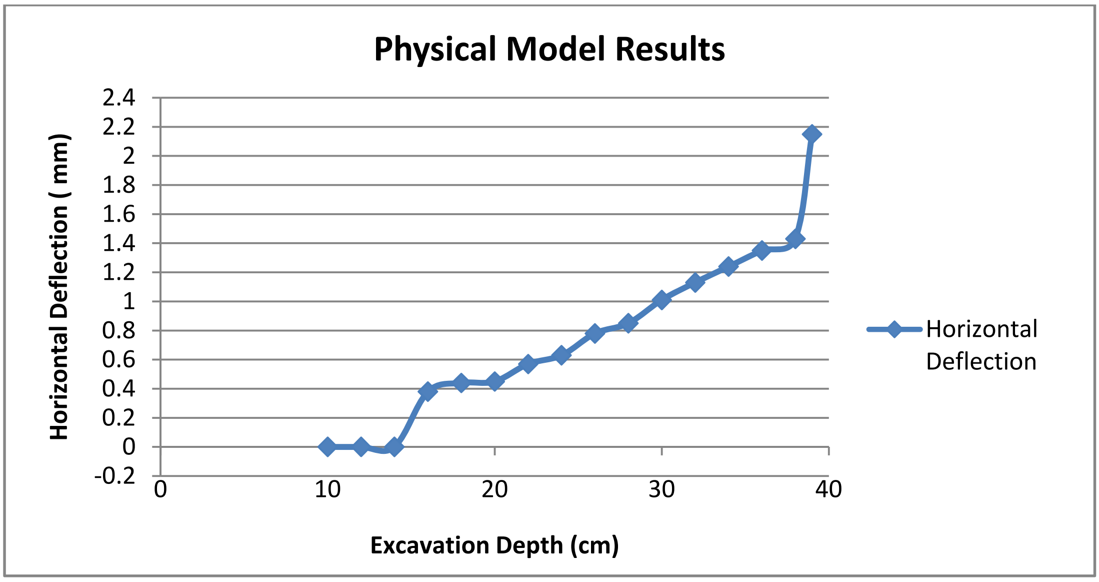

Figure 4.6 Excavation Depth Vs Horizontal Deflection

5. FINITE ELEMENT ANALYSIS

5.1 Numerical Modeling of Sheet pile walls

The behavior of the sheet pile walls are studied through numerical models. A software call PLAXIS which use finite element analysis method is used to model those scenarios.

5.1.1 Finite Element Analysis of Anchored Sheet pile walls

The existing case and the proposed solutions are modelled using finite element method. The model is used to evaluate the effect of driving another sheet pile row on the passive side of the existing sheet pile wall. The effect is evaluated for different types of cohesion less sand while varying the distance from the existing sheet pile wall to the proposed new sheet pile row. It is assumed that the new sheet pile used for the improvement wall is same as the sheet pile type used in the existing sheet pile wall and the possible sheet pile driving depth is also the same.

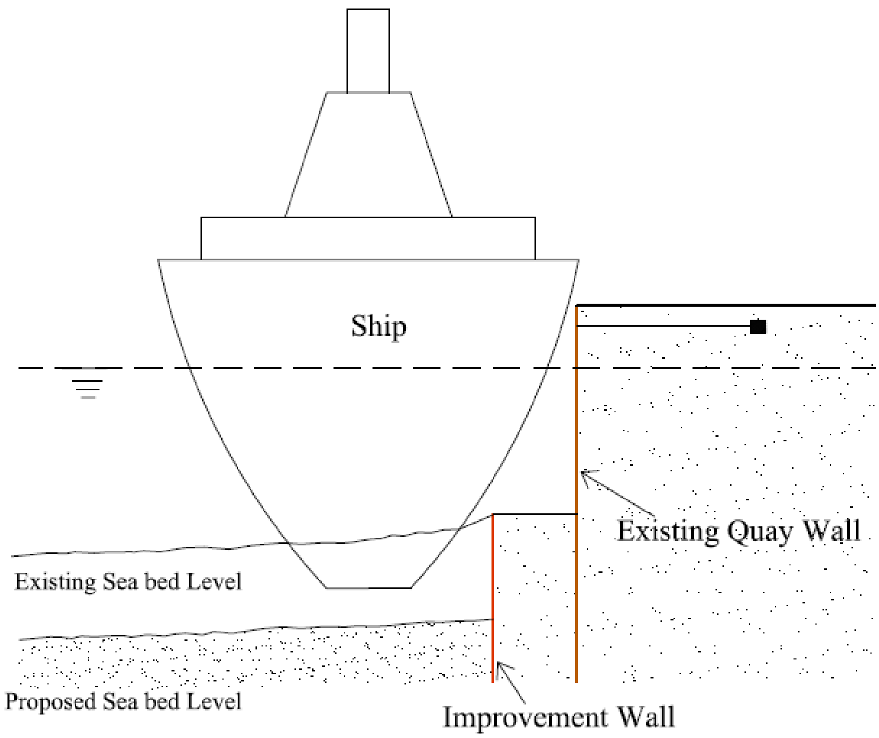

Figure 5.1 Proposed Improvement in the Quay Wall Structure

Anchored sheet pile wall is modelled based on the typical quay wall structure type used in fishery harbours in Srilanka. So water level is kept 1.5 m (Hw) below the top level of the sheet pile wall. The maximum distance (X_d) from the existing sheet pile wall to the improvement wall is limited to 3.0 m to facilitate the boats or ships to reach the quay wall without making it an obstacle to the navigation. During the first series of models it is observed that the composite structure fails due to the displacement of improvement sheet pile wall. So the few cases are further analyzed by increasing the embedment depth of the improvement sheet pile wall.

5.1.1.1 Dimensions of Anchored Sheet pile walls

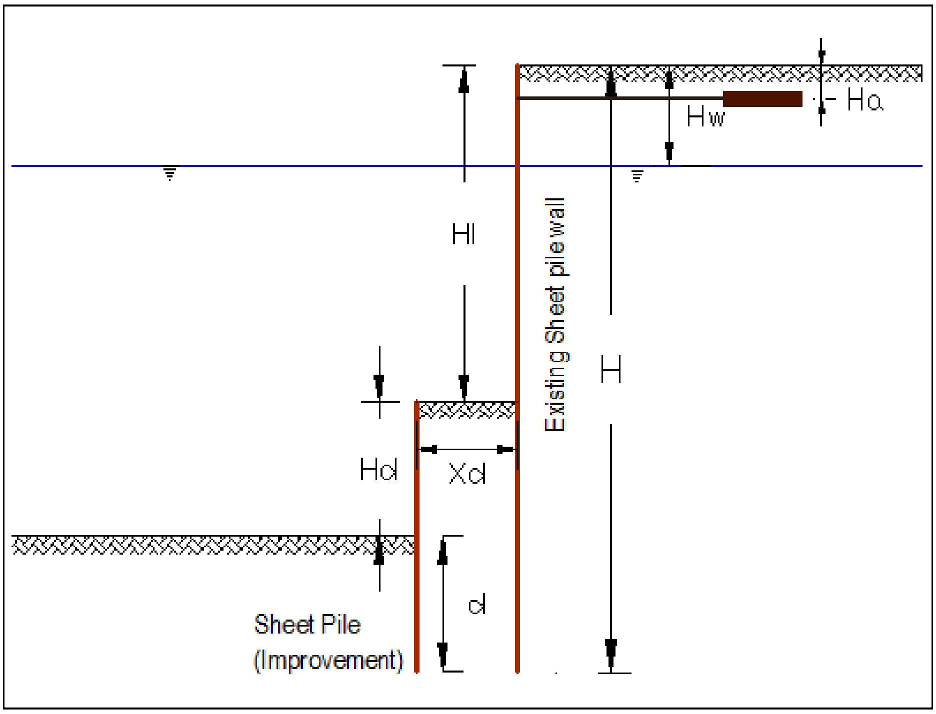

Figure 5.2 Anchored Sheet Pile wall Scenario

H - Total height of the Retaining Wall

Hi - Retaining height of Existing Retaining Wall

Hd - Additional Retaining height after Improvement

Hw - Depth to the water table

Ha - Depth to the Anchor

Xd - Distance between Existing wall and Improvement Sheet pile wall

d - Depth of embedment of sheet pile wall

5.1.2 Finite Element Analysis of Cantilevered Sheet pile walls

The existing case and the proposed solutions are modelled using Finite Element Method same as the anchored sheet pile wall. The model is used to evaluate the effect of driving another sheet pile row on the passive side of the existing sheet pile wall. When it is done for the both banks middle part can be further dredge and use as a dry flow canal. The effect is evaluated with the distance (Xd) from existing sheet pile wall to the proposed new sheet pile row. It is assumed that the new sheet pile type also same as the existing type as well as the possible driving depth also the same

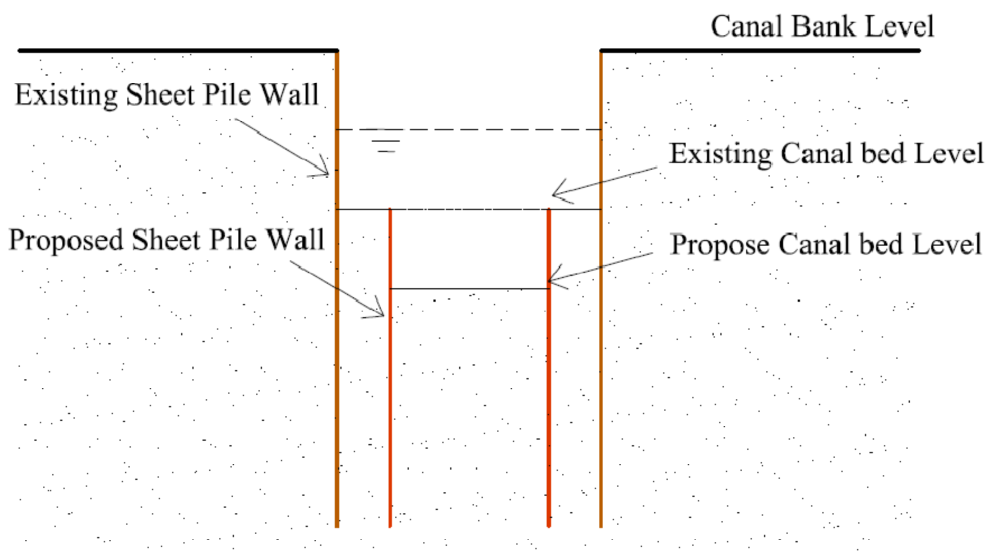

Figure 5.3 Proposed Improvement in the Canal Bank Protection

Cantilever sheet pile model is modelled based on the bank protection wall of canals. So the same concept is applied for this case to proceed dredging without disturbing the existing structure. So the ground water level is kept 1.5m (Hw) below the top of the sheet pile wall. The maximum distance from the existing sheet pile wall to the improvement wall is limited to 3.0 m (Xd), same as the previous case. Further analyses are done for the cantilever sheet pile model as well by increasing the depth of embedment (d).

5.1.2.1 Dimensions of Cantilevered Sheet pile walls

H - Total height of the Retaining Wall

Hi - Retaining height of Existing Retaining Wall

Hd - Additional Retaining height after Improvement

Hw - Depth to the water table

Ha - Depth to the Anchor

Xd - Distance between Existing wall and Improvement Sheet pile wall

d - Depth of embedment of sheet pile wall

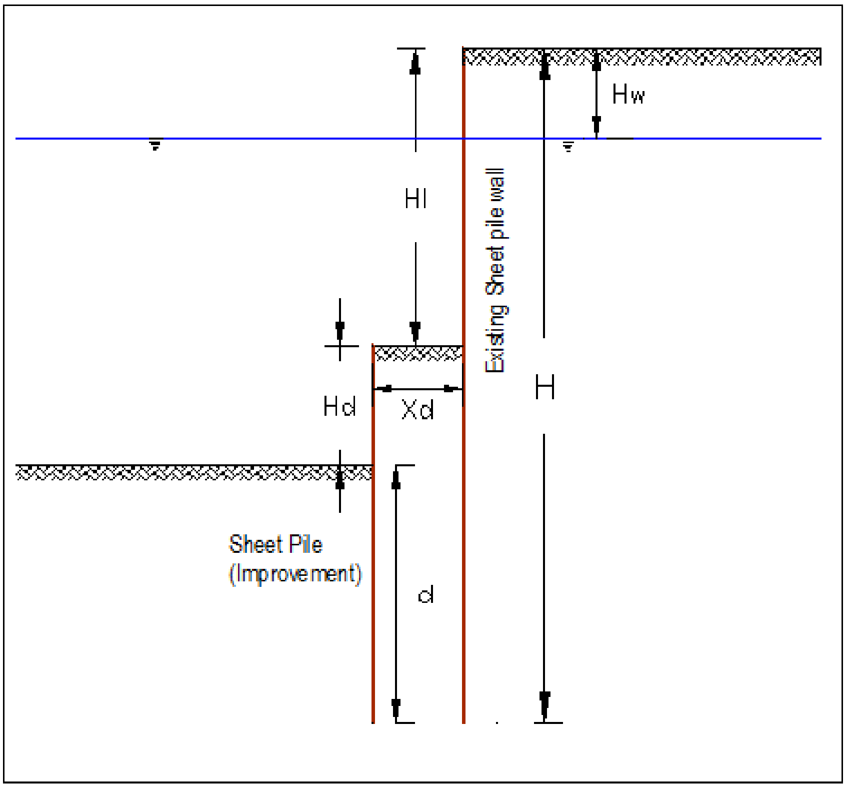

Figure 5.4 Cantilever Sheet Pile Wall Scenario

5.2 Dimensions of the Finite Element Mesh

A finite element mesh generated in analyzing the situation must always have dimensions that are sufficient for representing the problem and reduce the boundary effect for the model.

For the analysis of the sheet pile wall, the dimensions of the mesh must be taken as in Figure 5.5 as given by Aziz, F., (1999). During the modelling boundary distances were maintained to avoid the boundary effect to the model results.

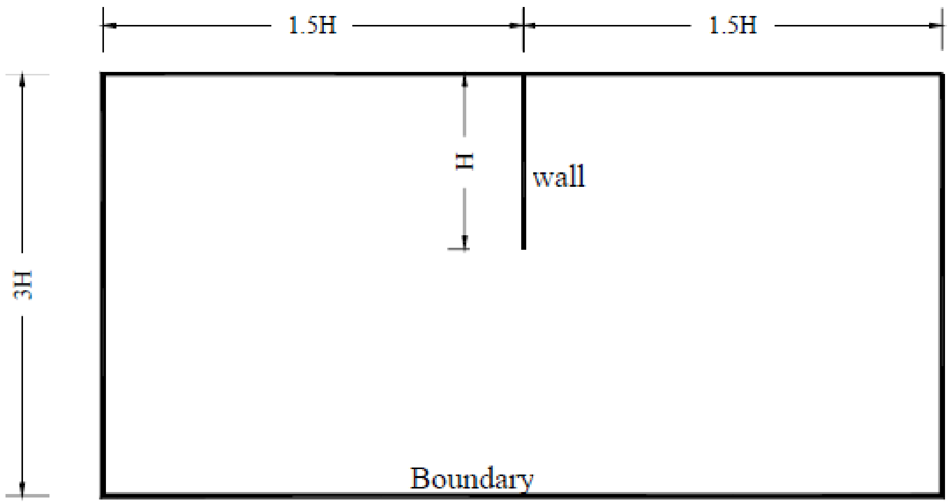

Figure 5.5 Proposed Model boundaries by Aziz, F., (1999)

5.3 Configuration of the Finite Element Mesh

PLAXIS allows for a fully automatic mesh generation of finite element meshes. The mesh generator is a special version of the triangle mesh generator developed by Sepia1. The generation of the mesh is based on a robust triangulation procedure. which results in ‘unstructured meshes. These meshes may look disorderly, but the numerical performance of such meshes is usually better than for regular (structured) meshes.

Distinction is made between five levels of global coarseness Very coarse. Coarse, Medium. Fine. and Very fine. The average element size and the number of generated triangular elements depends on this global coarseness. A rough estimate is given below

- *Very coarse:* Around 50 elements
- *Coarse:* Around 100 elements
- *Medium:* Around 250 elements
- *Fine:* Around 500 elements

- *Very fine:* Around 1000 elements

The number of elements in the finite element mesh has a significant effect on the accuracy of the analysis. Because of that the degree of coarseness of the mesh must be selected in a systematic way. The selection of the degree of coarseness for this model was done after performing a sensitivity analysis using different degree of coarseness. The medium degree of coarseness was found to be the most suitable for this study. previous similar researches also confirmed the medium degree of coarseness gives the reasonable results.

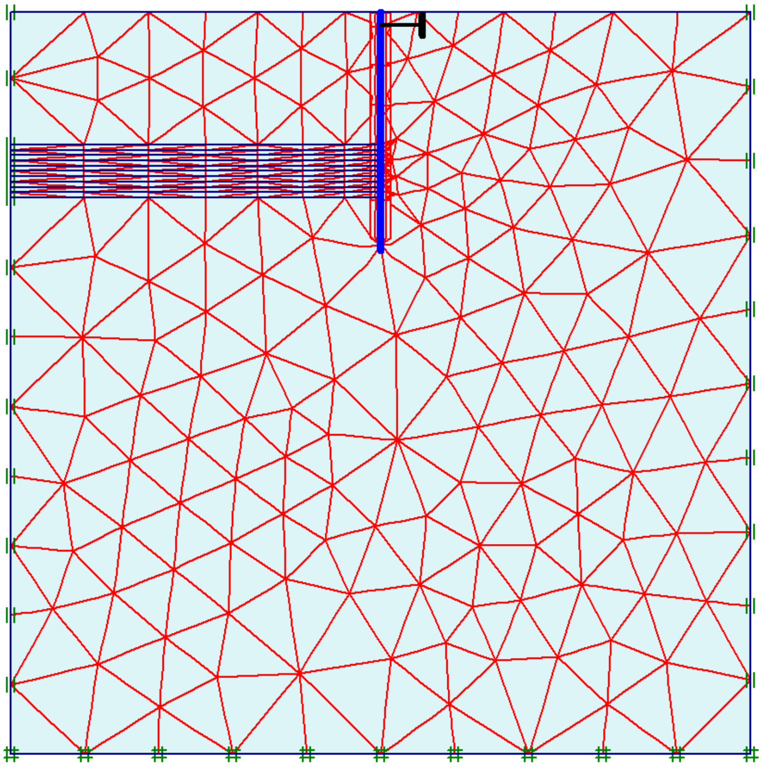

Figure 5.6 The Finite Element mesh (Anchored Sheet Pile)

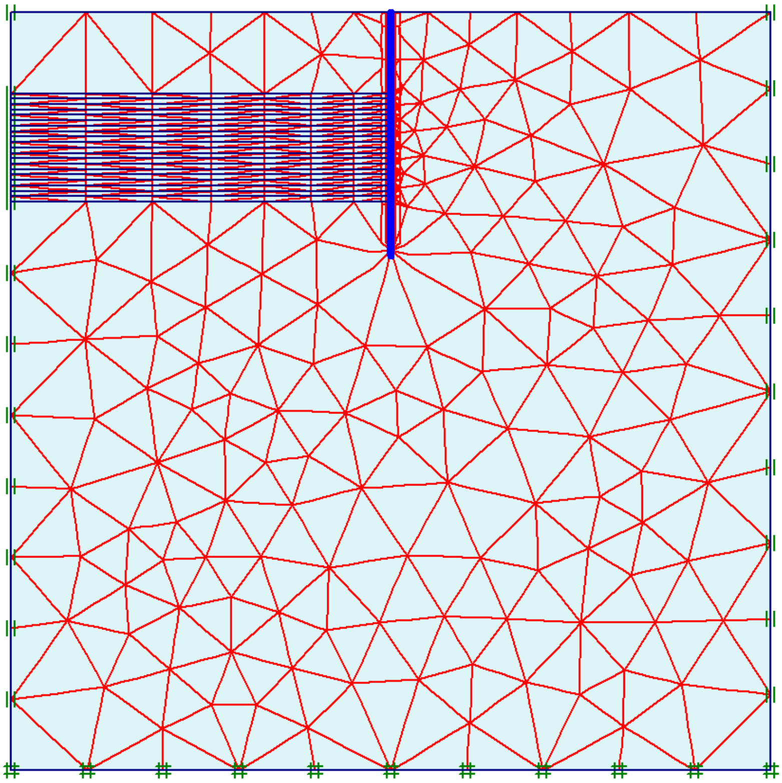

Figure 5.7 The Finite Element mesh (Cantilever Sheet Pile)

5.4 Properties of Soil Elements

The finite element analysis is performed only for cohesion less soils to get better understanding and narrow the scope of study. The shear strength of soil is given by the ultimate angle of friction, φ, and the stress-strain behaviour of soil is incorporated by the modulus of elasticity, E. A uniform subsurface is assumed in the analysis. A parametric study was conducted by varying the characteristics of the subsurface.

The soil is categorized based on the shear strength and behaviour is studied in the range of 22 to 38 degrees of angle of friction values. The properties of the different soil groups are shown in Table (5.1)

φ = Angle of shear friction.

E = Stiffness of soil.

γd = Unsaturated unit weight of soil

Ψ = Angle of dilatation.

ν = Poisson's ratio of soil

Table 5.1 Properties of Soil Groups

Soil Group	φ Degree	E (kN/m^2)	γ_{bulk} (kN/m3)	γ_{sat} (kN/m3)	ν	ψ
1	22	8000	17	19	0.3	0
2	24	10000	17	19	0.3	0
3	26	15000	17	19	0.3	0
4	28	20000	17	19	0.3	0
5	30	25000	17	19	0.3	0
6	32	30000	17	19	0.3	2
7	34	35000	17	19	0.3	4
8	36	40000	17	19	0.3	6
9	38	45000	17	19	0.3	8

5.5 Properties of Sheet Pile and Anchor Elements

The properties of the sheet pile walls are selected based on the types used in practical quay wall projects in Srilanka. After studying the sheet pile quay wall designs of Dikkovita , Oluvil and Codbay fishery harbours ,One common type of sheet pile is selected to the entire research to narrow down the scope and to get better understanding about the behaviour of soil.

The behaviour of the sheet pile wall is considered as elastic nature in the study. So wall is modelled as a plate element. It is assumed that the existing anchors are strong enough to take the additional loads So the tensile forces generated in the anchors are monitored in the study. Properties assigned in the model is tabulated
in table 5.2

Table 5.2 Properties of Sheet Pile and Anchor Elements

No	Property	Sheet Pile	Anchor
1	Flexural rigidity (kN/m)	7.938E+04	-
2	Axial stiffness (kN.m²/m)	2.923E+06	3.0E5
3	Unit weight (kN/m)	0.750	
4	Poisson's Ratio	0.3	
5	Spacing (m)	-	1.00

5.6 Properties of Soil - Structure Interface

Properties of the soil structure interface need to be defined properly to model the soil structure interaction . Interfaces are marked around the elements and the behaviour of the interface is calculated based on the soil properties. Stress reduction factor ;R is taken as 0.67. Angle of friction between soil and the wall is calculated as show below.

$$\tan(\delta) = R \cdot \tan(\varphi)$$

Where:
δ = Angle of friction between soil and the wall.
R = Shear strength reduction factor.
φ = Angle of internal friction of soil

5.7 Dimensions of Finite Element Model

The model dimensions are selected to avoid the boundary effect interfere on the results of the analysis. These dimensions of the domain are selected considering the previous researches and few trials are done increasing the dimensions until the size has no effect on the results. The selected dimensions for Anchored and Cantilever walls kept same and height of the wall also 9 m in both cases. it's shown in figure 5.8

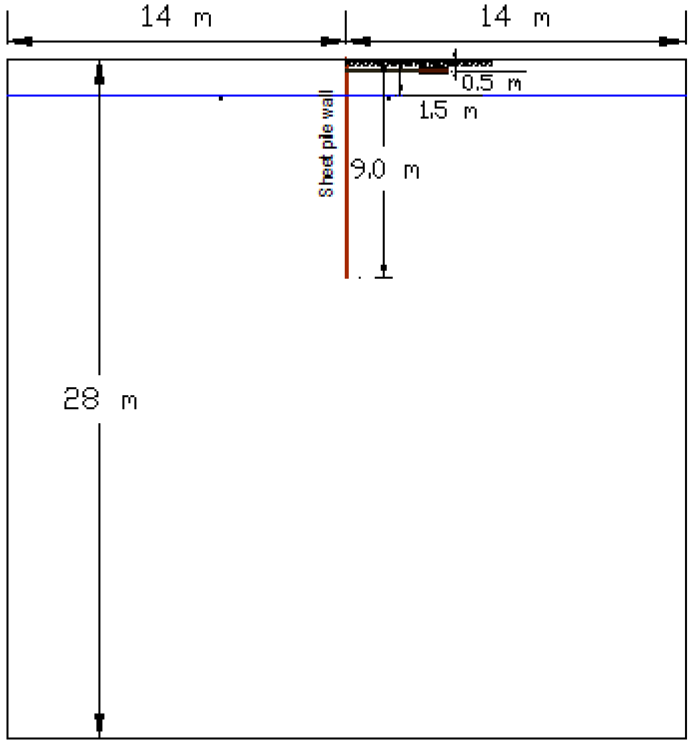

Figure 5.8 Selected Dimensions for Finite Element Model

5.8 Verification of The Numerical Model

All conditions of the physical model are simulated in the finite element model to verify the behaviour of the finite element model for this type of modelling work

5.8.1 Finite Element Analysis of the Physical model

The same model which was tested in the flume is modeled in the finite element software to check the validity of the finite element analysis. Soil parameters which were taken from the laboratory tests are used in the finite element model.

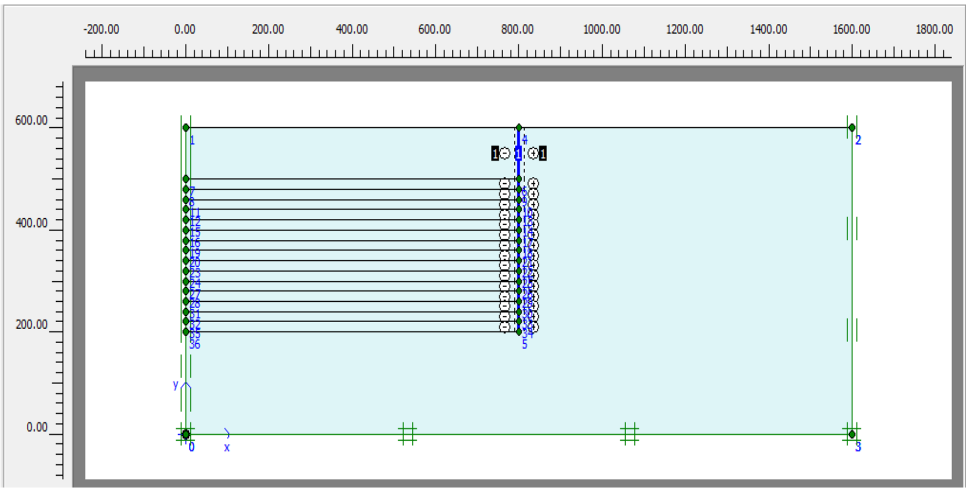

Figure 5.9 Numerical Model Setup of the Physical Model

Table 5.3 Model parameters used for Numerical Modeling

No	Property	Sheet Pile	Soil
1	Flexural rigidity (kN/mm)	180	
2	Axial stiffness (kN.mm²/mm)	3375	
3	Unit weight (kN/m)	0.10	
4	Poisson’s Ratio	0.3	0.3
5	Angle of shear friction.	-	29
6	Stiffness of soil.(kN/m^2)		5500
7	UnSat. Density (kN/m^3)		15.5
8	Sat. Density (kN/m^3)		17

Numerical model also performed same as the physical model removing 20mm layer of sand at a time starting from 10 cm initial excavation. The horizontal deformation of the wall is taken from the numerical model is compared with the physical model results in Figure 5.10

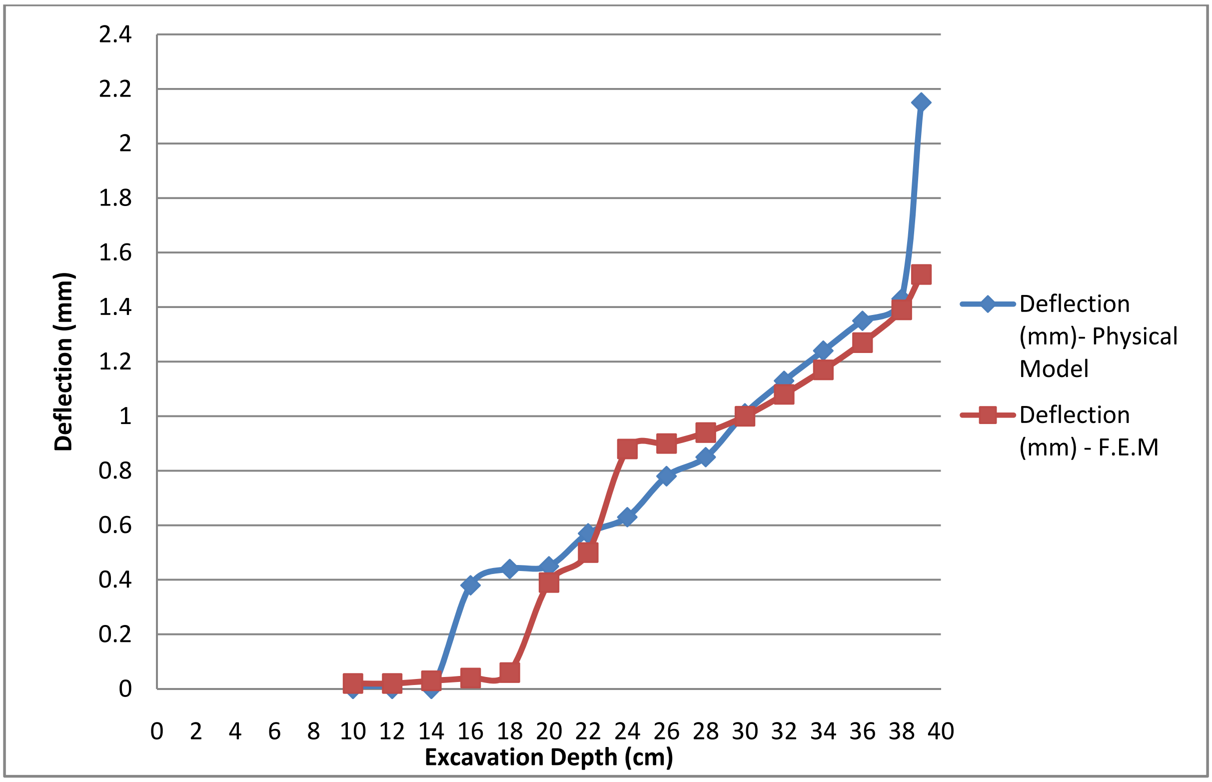

Figure 5.10 Horizontal Deformation of The Finite Element Model and The Physical Model

6. RESULTS AND DISCUSSION

6.1 Validation of the Finite Element Model

The comparison of horizontal deflection values of the physical model and the Numerical model shown in the figure 5.10 . Those results shows that the deviation of most of the numerical model values are less than 0.1 mm from the measured physical model values . So the results obtained from the finite element analysis can be validate with the actual behaviour of soil and the structure.

Finite element analysis is done for two main scenarios as explained before. First ,anchored sheet pile wall is modeled idealizing the harbor quay wall structure. Proposed sheet pile wall improvements are introduced starting from 1.0m from existing sheet pile wall and every 0.5m interval till 3.0 m away from existing wall. It is assumed that the existing sheet pile wall was designed for D/H ratio is 0.8. So initial retaining height is 5.0m for 9.0m sheet pile wall. Additional excavation requirement is taken as 2.0m. All the analysis have been performed for nine soil groups as per shown in table 3.1

Second, Cantilever sheet pile wall is modeled and improvements are introduced same as in anchored sheet pile wall. Its assumed as the existing sheet pile wall was designed for D/H ratio is 2 . As a thumb rule 2/3 of sheet pile length use as depth of embedment in designing cantilevered retaining walls. (ÁRNI JÓNSSON etal, 2006) So initial retaining height is 3.0m for 9.0m sheet pile wall. Additional excavation requirement is taken as 2.0m in this scenario as well.

The analysis for cantilever sheet pile also have been performed for all nine soil groups varying the distance between existing wall and improvement sheet pile wall. The total length of the improvement sheet pile wall is referred to differentiate the extended improvement wall models.

6.2 The Anchored Sheet Pile Wall Model

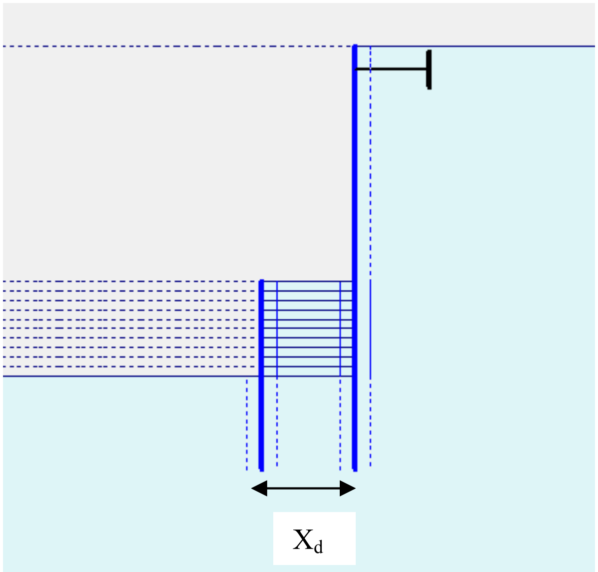

Figure 6.1 The Anchored Sheet Pile Model

The cases are defined based on the distance to the improvement sheet pile wall from existing sheet pile wall. That distance is X_d as shown in figure 6.1 is varied from 1.0m to 3.0 m at 0.5m interval. significant improvement could not be observed below 1.0 m distance and practical applicability is less beyond 3.0m distance. The cases are defined as shown below

Table 6.1 Numerical Model Cases

Case	X_d (m)
Case 1	Existing Wall Only
Case 2	1.0
Case 3	1.5
Case 4	2.0
Case 5	2.5
Case 6	3.0

After analyzing all these cases results shows significant stability improvement with the proposed solution. The analysis results of dredging with existing sheet pile wall (Case 1) and most stable solution (Case 6) are tabulated in Table 6.2 and Table 6.3

Table 6.2 Results Of Case 1 Analysis - Anchored Retaining Wall (4m Improvement Wall)

Case 1

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	204.51	190.93	115.6	6.8*
24	32.16	146.62	85.74	6.8*
26	45.17	153.91	97.38	7
28	10.17	117.14	66.38	7
30	7.23	98.98	57.93	7
32	5.83	82.89	52.06	7
34	4.93	71.36	47.37	7
36	4.05	60	42.33	7
38	3.33	50.51	37.75	7

Table 6.3 Results of Case 6 Analysis - Anchored Retaining Wall (4m Improvement Wall)

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth
22	41.53	143.53	84.31	6.8*
24	15.27	117.76	64.65	7*
26	6.13	91.99	51.87	7
28	4.15	73.25	45.64	7
30	2.97	57.77	40.09	7
32	2.01	44.27	33.43	7
34	1.39	34.93	28.4	7
36	0.945	27.77	24.16	7
38	0.607	22.48	20.37	7

* soil body collapses in numerical model

Dredging depth can be increased with the improvement and there is a reduction in horizontal deflection as well as in bending moment and anchor force. The horizontal deflection variation when increasing the retaining height for all cases are plotted in figure 6.2 to 6.8

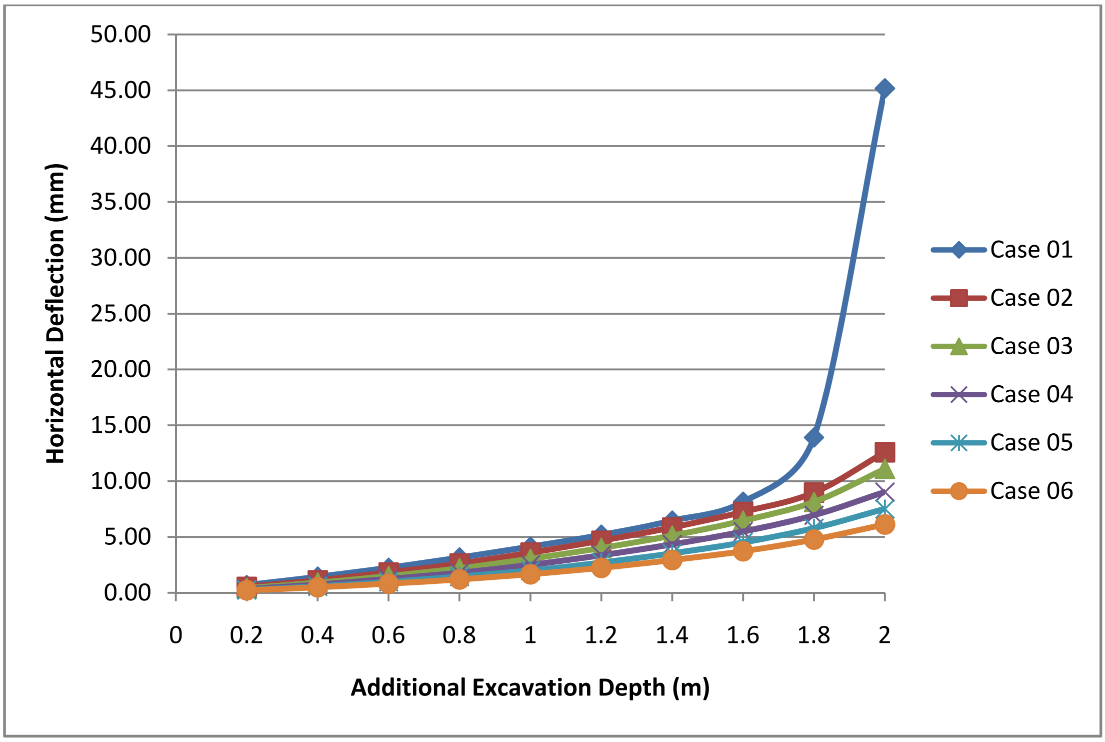

Figure 6.2 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 26 degrees (Anchored Retaining Wall)

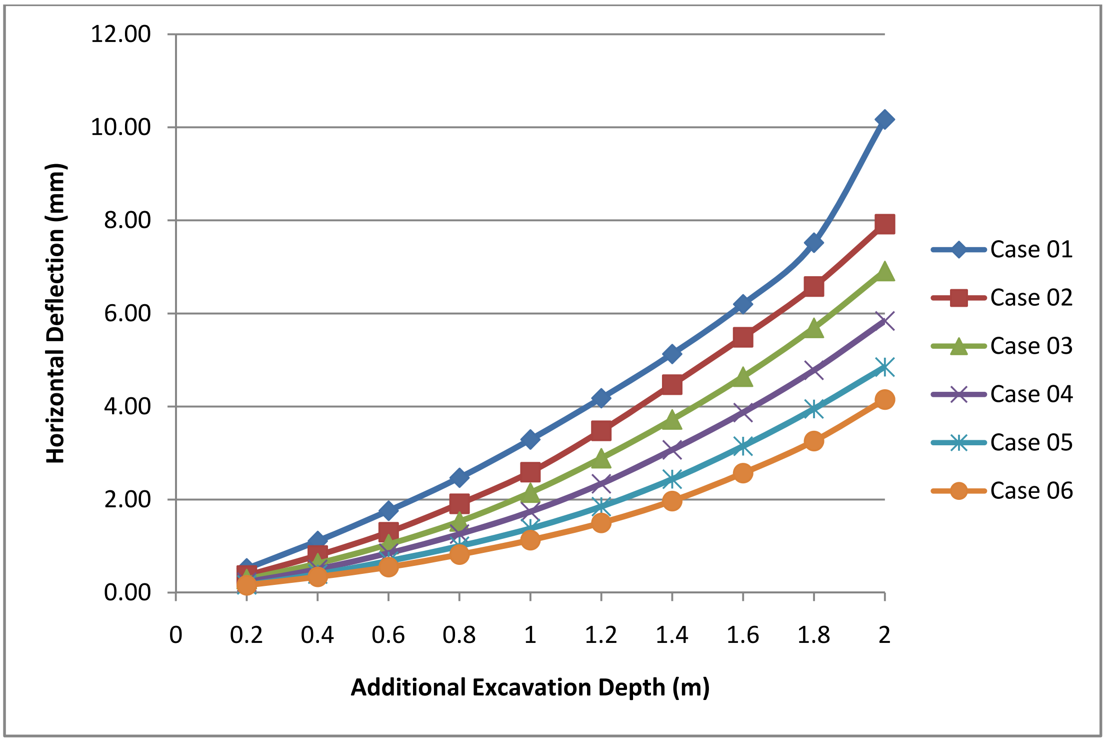

Figure 6.3 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 28 degrees (Anchored Retaining Wall)

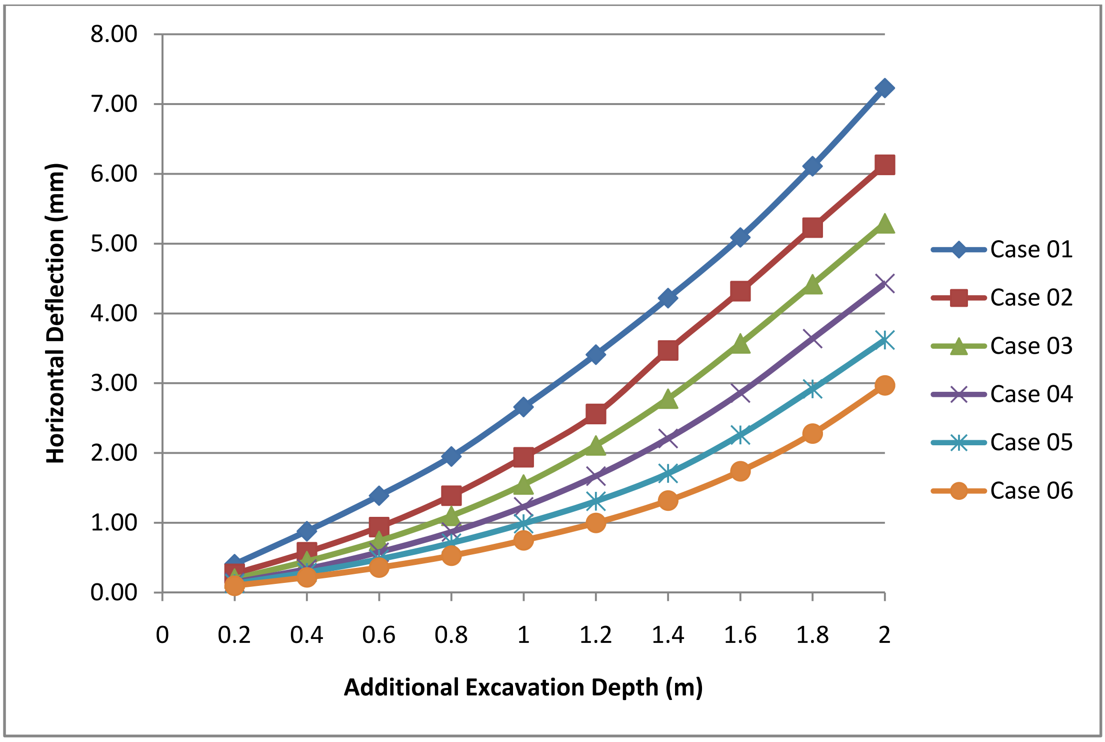

Figure 6.4 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 30 degrees (Anchored Retaining Wall)

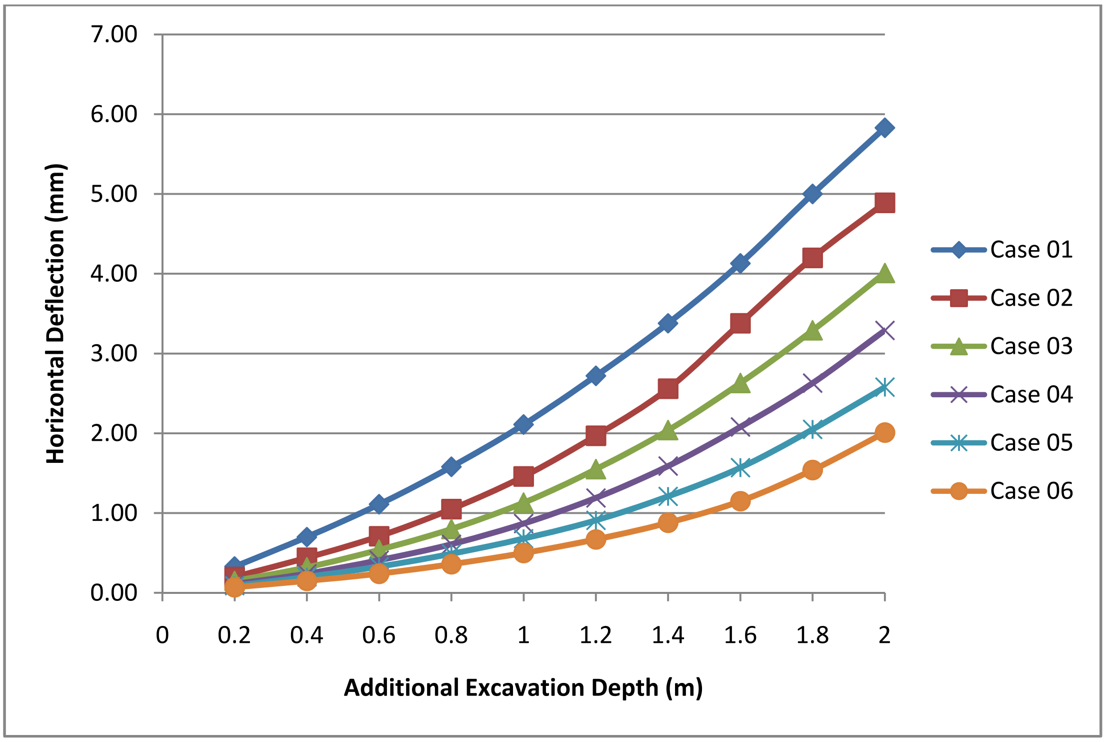

Figure 6.5 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 32 degrees (Anchored Retaining Wall)

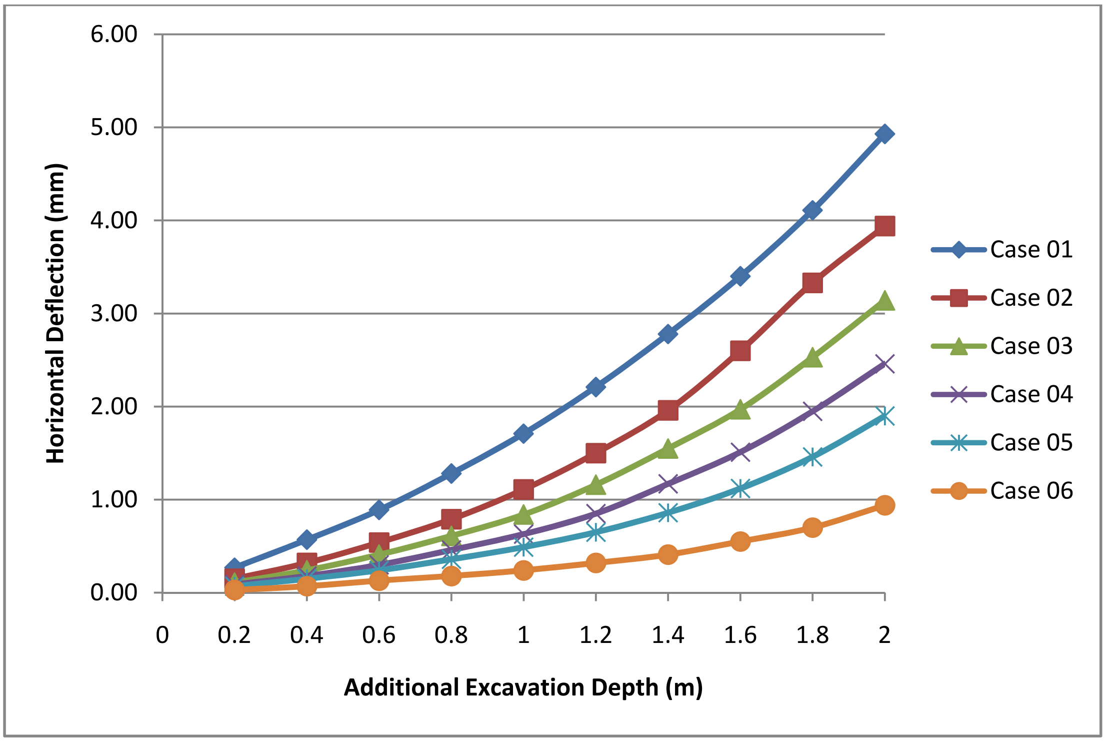

Figure 6.6 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 34 degrees (Anchored Retaining Wall)

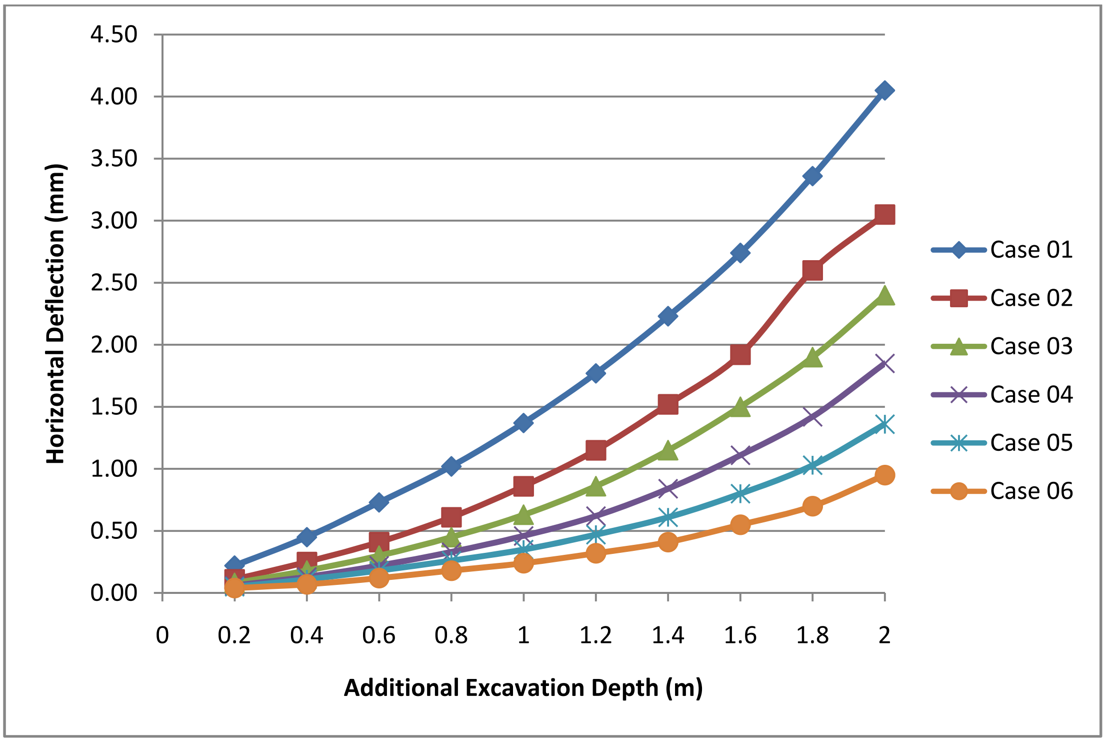

Figure 6.7 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 36 degrees (Anchored Retaining Wall)

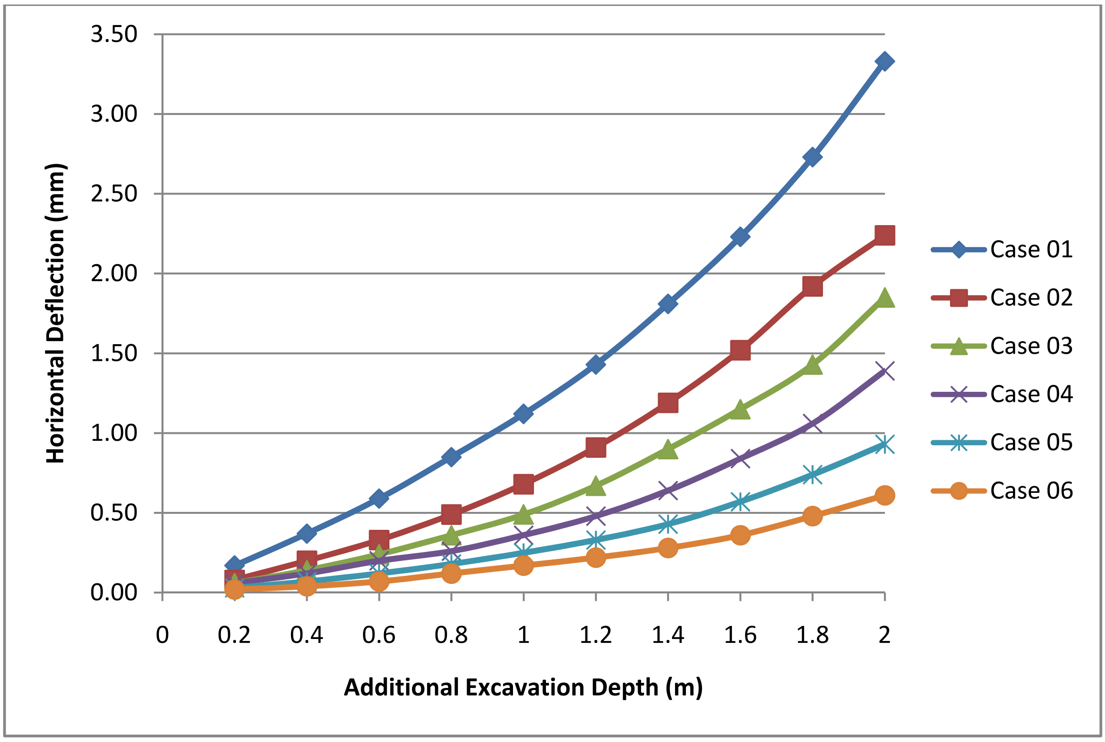

Figure 6.8 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 38 degrees (Anchored Retaining Wall)

The internal friction angle beyond 24 degrees seems to be much stable in the range of 5m to 7m dredging. but it is observed that with the improvement wall there is a reduction of wall deflection, bending moment and the anchor force.

Further it can be observed that the combined structure fails due to the instability of the improvement sheet pile wall. The horizontal displacement and the bending moment of the improvement wall at failure depth or maximum excavation depth is tabulated in Appendix B

So the depth of embedment of the improvement sheet pile wall is extended by 2.0m and dredging is further extended up to 8m retaining height to observe the behavior of the wall and the significance of the improvement. The cases are tested the distance between the existing sheet pile wall and the improvement wall (X_d) at 1.0m interval only for case 2, 4 and 6.

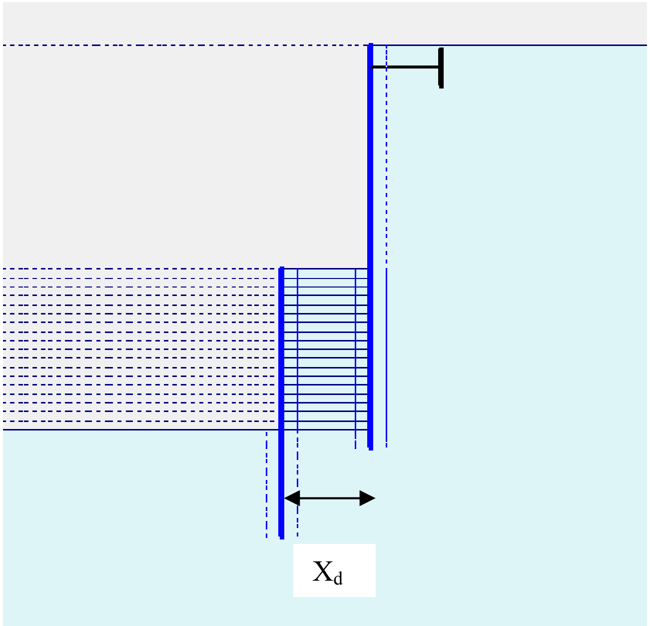

Figure 6.9 The Anchored Sheet Pile Model in Plaxis
(Extended Depth of Embedment of Improvement Wall)

The analysis results of dredging with existing sheet pile wall (Case 1) and most stable solution (Case 6) are tabulated in Table 6.4 and Table 6.5

Table 6.4 Results of case 1 analysis - anchored retaining wall (6m improvement wall)

Case 1

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	204.51	190.93	115.6	6.6*
24	32.16	146.62	85.74	6.8*
26	30.26	149.23	92.49	7.2*
28	18.72	136.33	82.52	7.4*
30	19.07	132.51	87.73	7.6*
32	15.57	120.98	82.37	7.8*
34	8.81	97.26	62.32	7.8*
36	9.36	93.12	64.43	8*
38	8.42	85.03	61.7	8

Table 6.5 Results of case 6 analysis - anchored retaining wall (6m improvement wall)

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth
22	87.41	152.69	103.5	7.2*
24	90.47	149.27	106.9	7.6*
26	26.62	113.83	84.27	7.8*
28	9.38	82.71	60.08	7.8*
30	9.43	78.43	63.65	8
32	4.94	58.51	44.69	8
34	3.19	45.01	36.38	8
36	2.29	35.76	31	8
38	1.72	29.3	26.88	8

* soil body collapses in the numerical model

The horizontal deflection variation when increasing the retaining height for Case 1, 2, 4 and 6 are plotted in figure 6.10 to 6.16

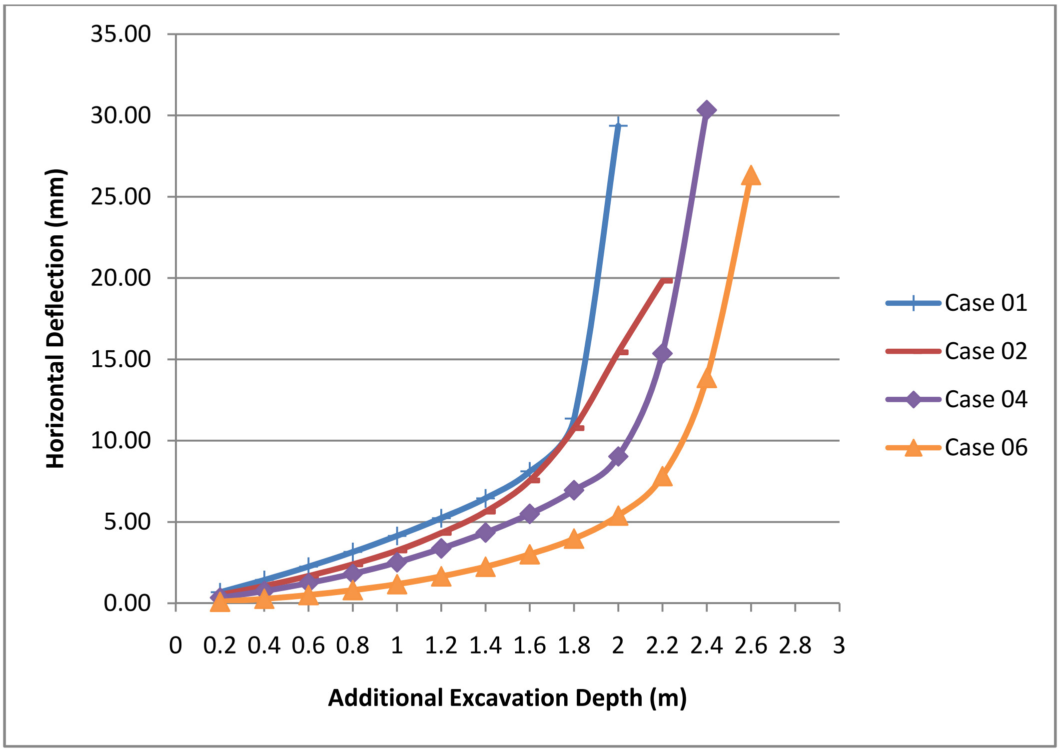

Figure 6.10 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 26 degrees - Anchored Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

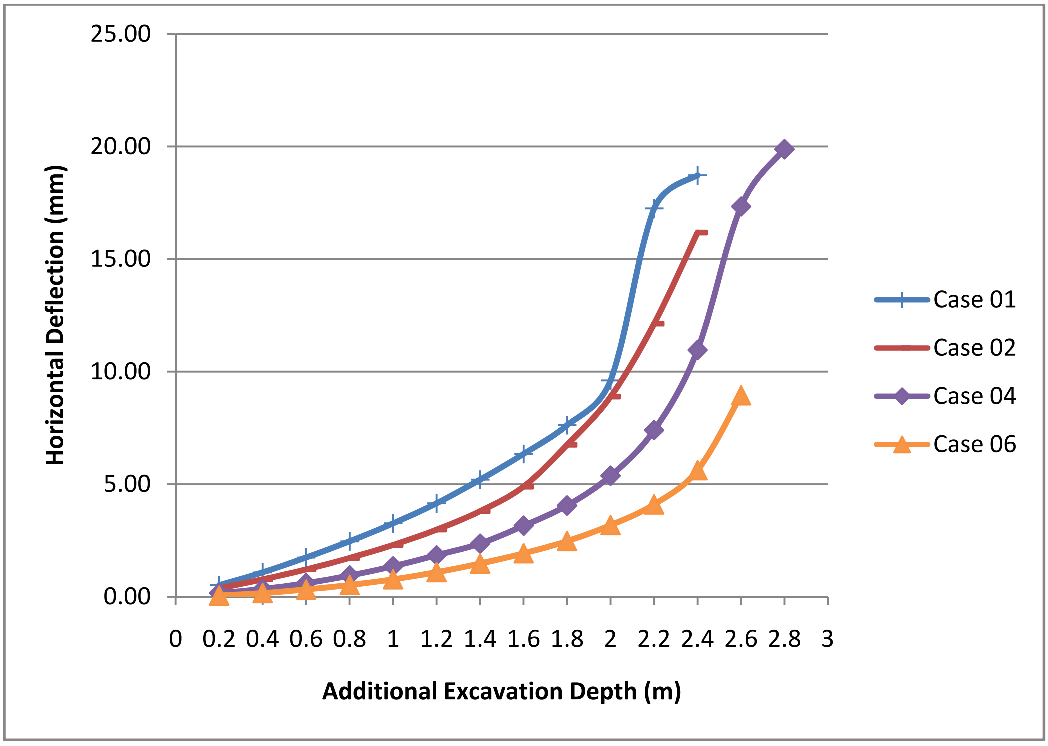

Figure 6.11 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 28 degrees - Anchored Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

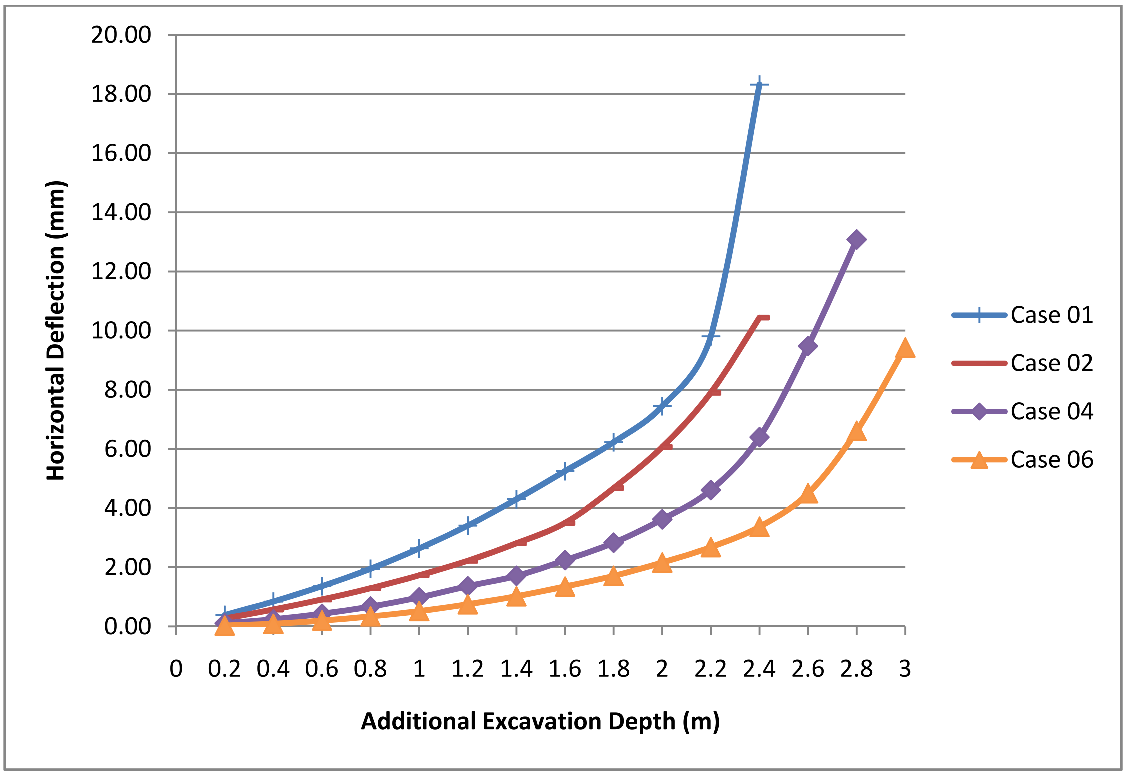

Figure 6.12 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 30 degrees - Anchored Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

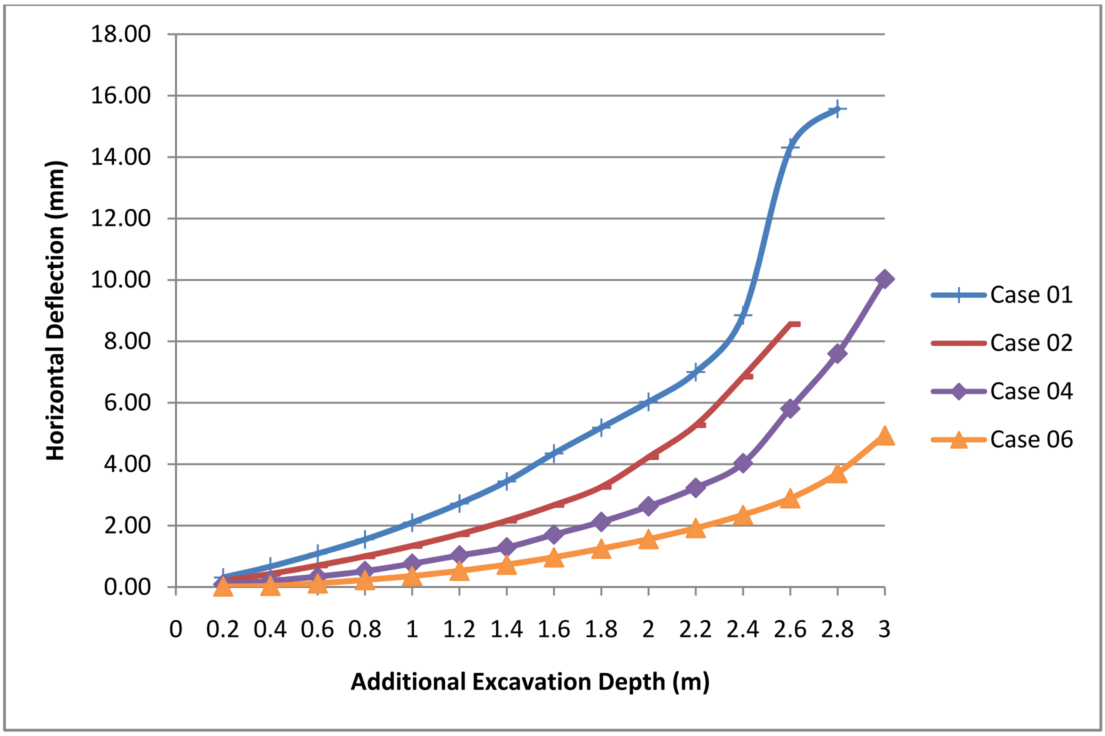

Figure 6.13 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 32 degrees - Anchored Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

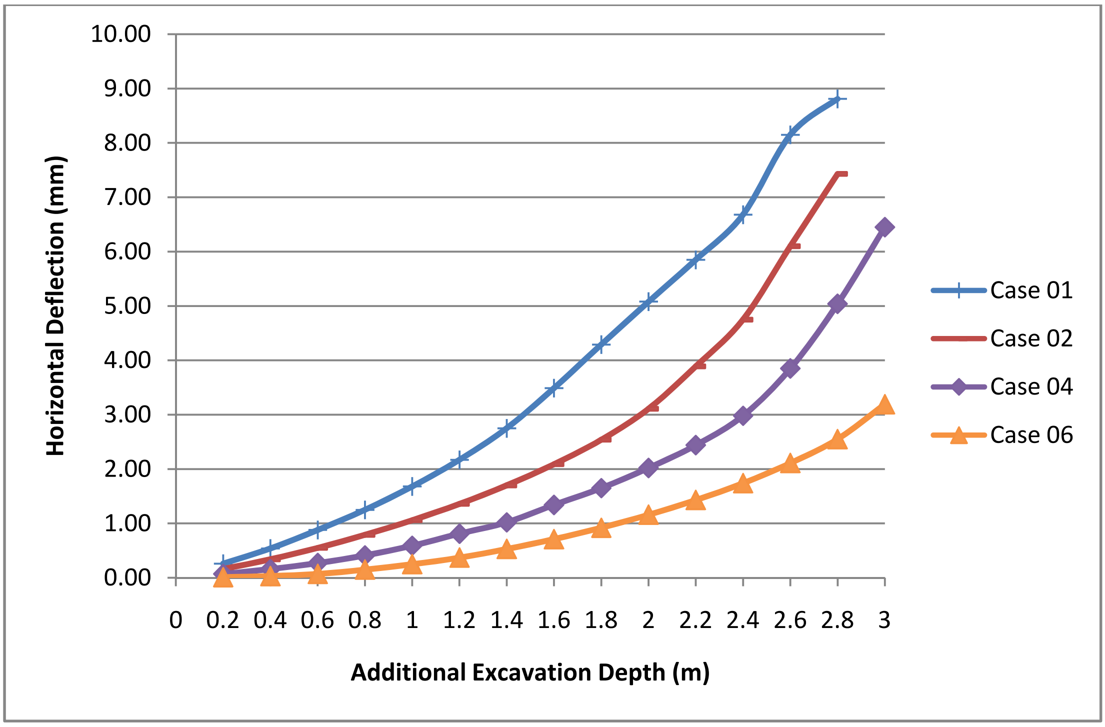

Figure 6.14 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 34 degrees - Anchored Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

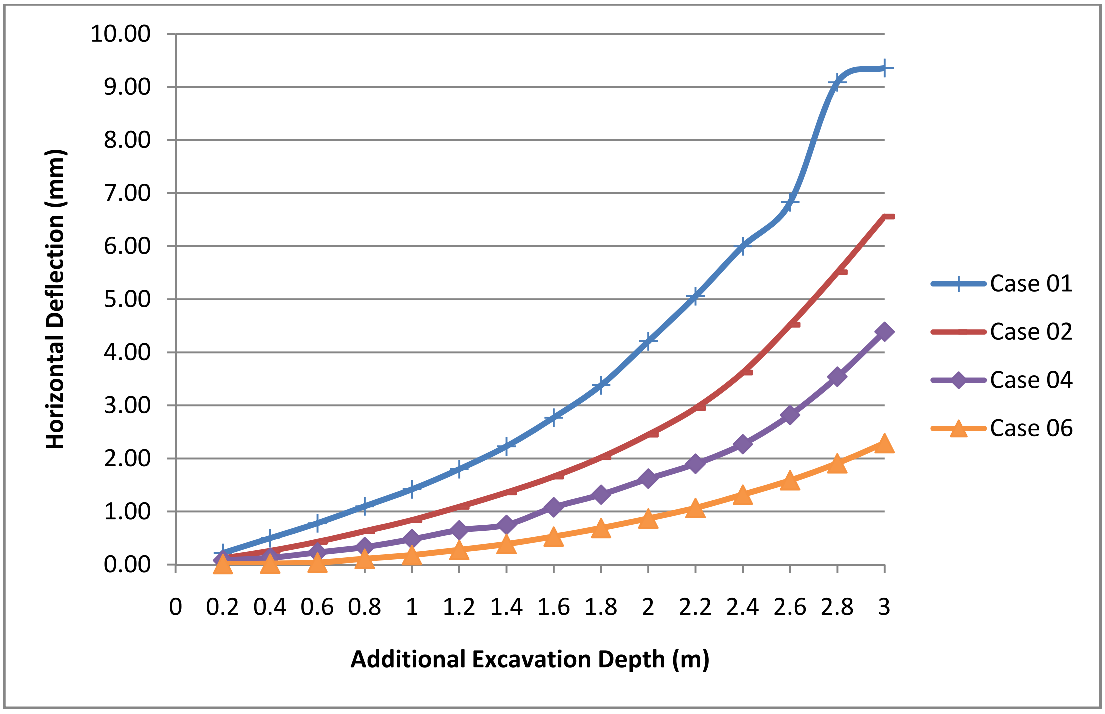

Figure 6.15 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 36 degrees - Anchored Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

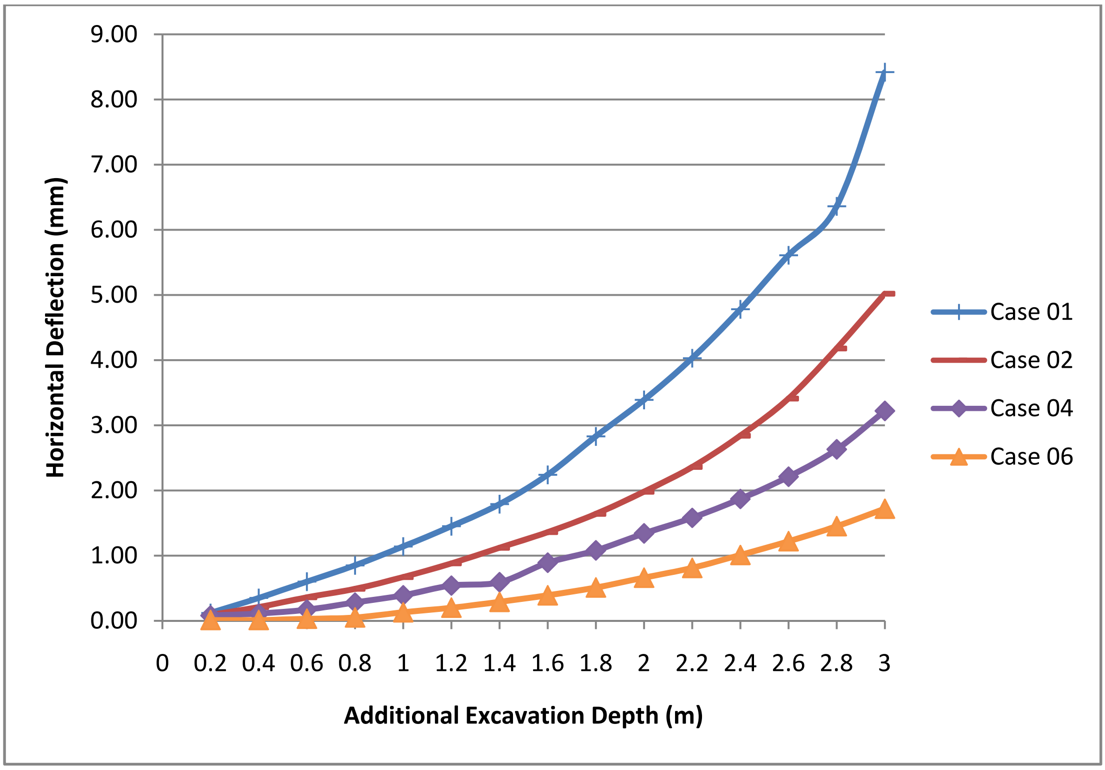

Figure 6.16 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 38 degrees - Anchored Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

The existing case results shows that only for internal friction angle more than 36 degrees sand can be dredged up to 8.0m retaining height. But with the improvement it can be done even in sand with friction angle of 30 degrees. So it is a significant improvement in the stability with the proposed solution technique.

6.3 The Cantilever Sheet Pile Wall Model

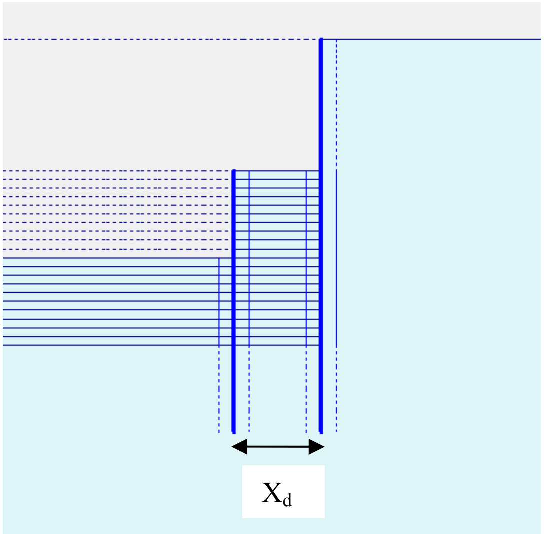

Figure 6.17 The Cantilever Sheet Pile Model in Plaxis

The same six cases of models are tested for all nine soil groups as it is done for anchored sheet pile wall. The cases are defined based on the distance X_d as shown in Figure 6.17 is varied from 1.0m to 3.0 m at 0.5m interval. Even in cantilever sheet pile wall , significant improvement could not be observed below 1.0 m distance and the practical applicability is less beyond 3.0m distance. The results of the case 1 and case 6 are tabulated below.

Table 6.6 Results Of Case 1 Analysis - Cantilever Retaining Wall (6m Improvement Wall)

Case 1

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	297.42	149.57	3.8*
24	198.73	145.7	4*
26	114.81	140.35	4.4*
28	46.59	117.76	4.4*
30	80.66	131.63	4.8*
32	42.99	110.56	4.8*
34	58.46	116.09	5
36	42.3	102.65	5
38	30.26	88.28	5

Table 6.7 Results Of Case 6 Analysis - Cantilever Retaining Wall (6m Improvement Wall)

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	264.56	100.16	4.4*
24	193.87	89.19	4.8*
26	21.53	67.52	5
28	29	59.58	5
30	14.08	50.44	5
32	7.85	40.47	5
34	4.83	33.73	5
36	3.08	29.34	5
38	1.83	24.83	5

* soil body collapses in the numerical model

The horizontal deflection variation when increasing the retaining height for all cases are plotted in figure 6.18 to 6.24

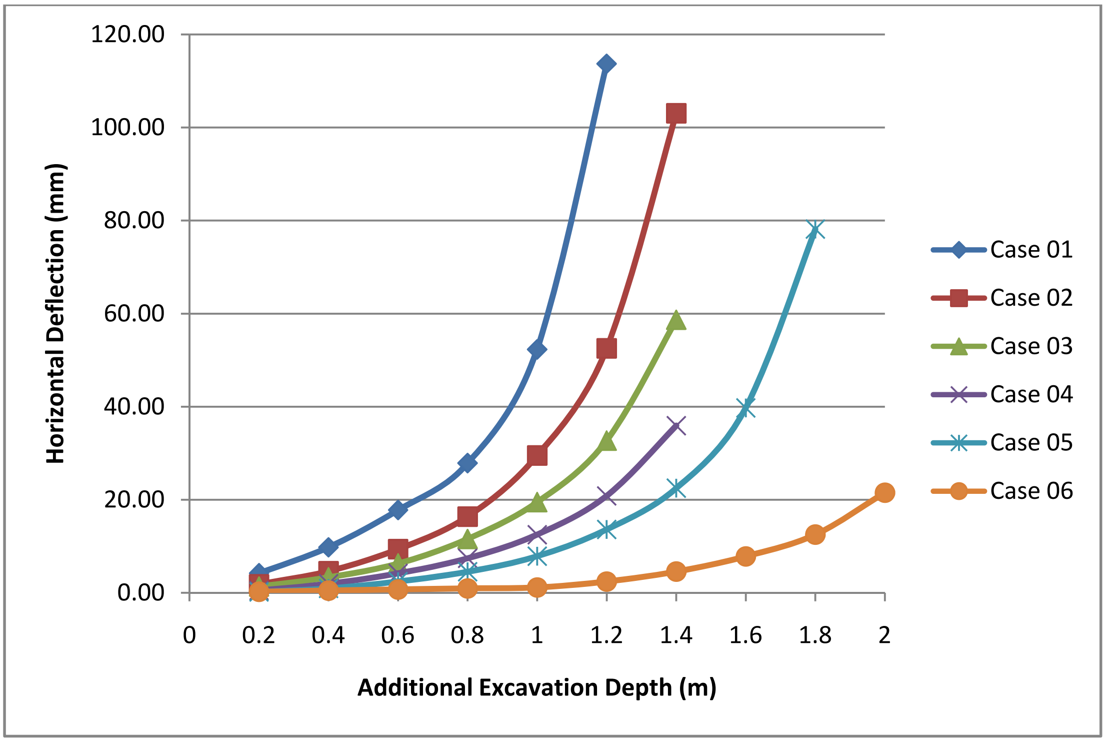

Figure 6.18 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 26 degrees (Cantilever Retaining Wall)

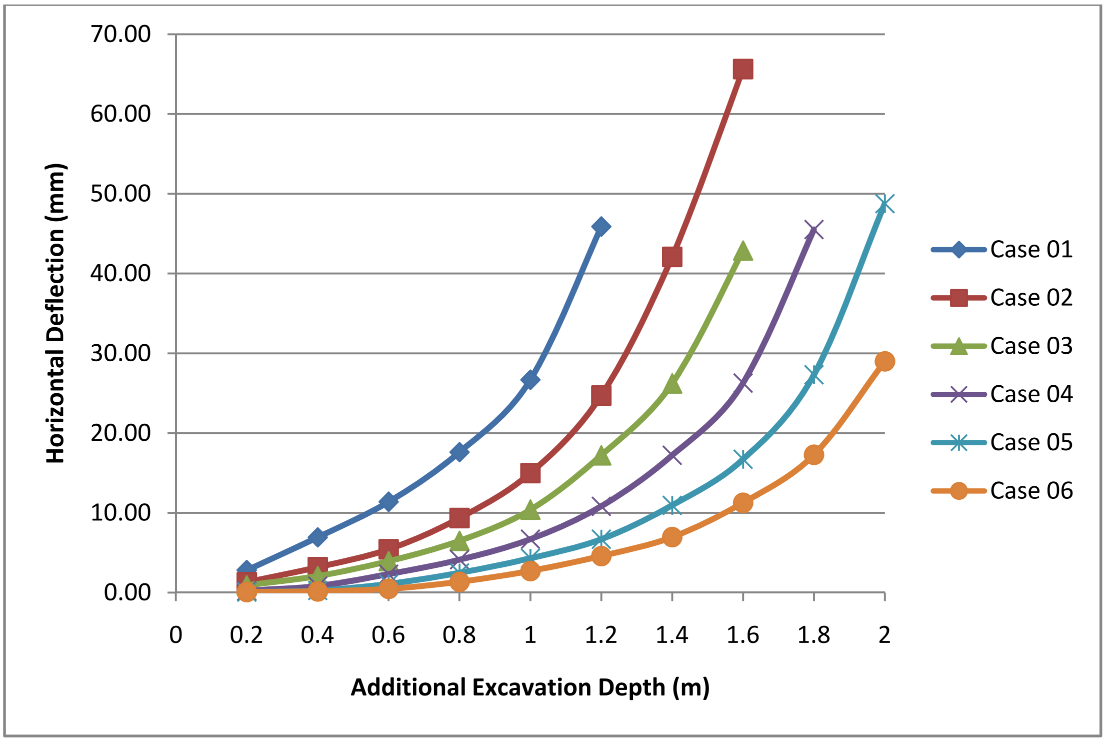

Figure 6.19 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 28 degrees (Cantilever Retaining Wall)

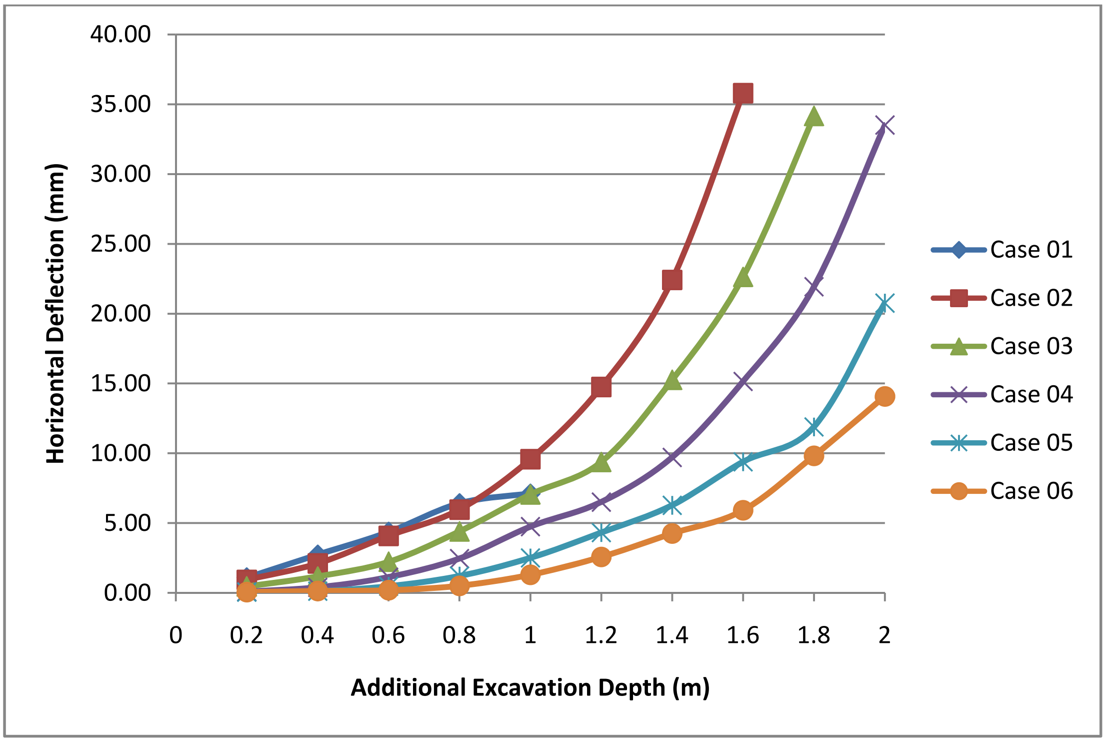

Figure 6.20 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 30 degrees (Cantilever Retaining Wall)

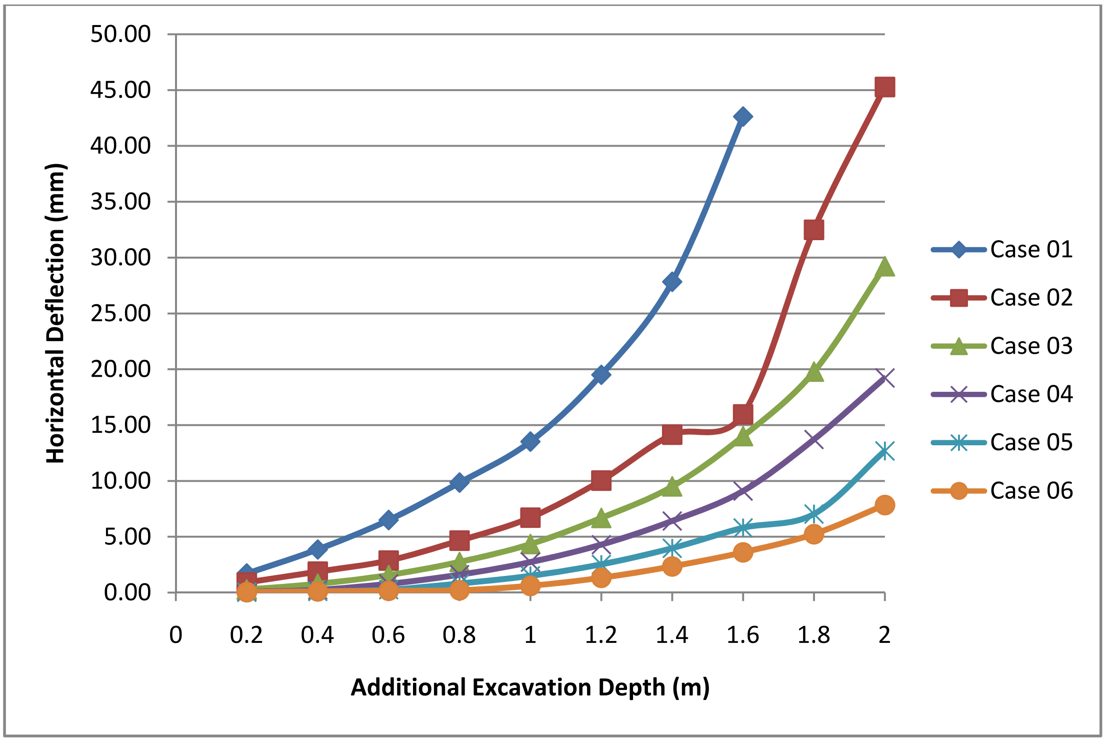

Figure 6.21 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 32 degrees (Cantilever Retaining Wall)

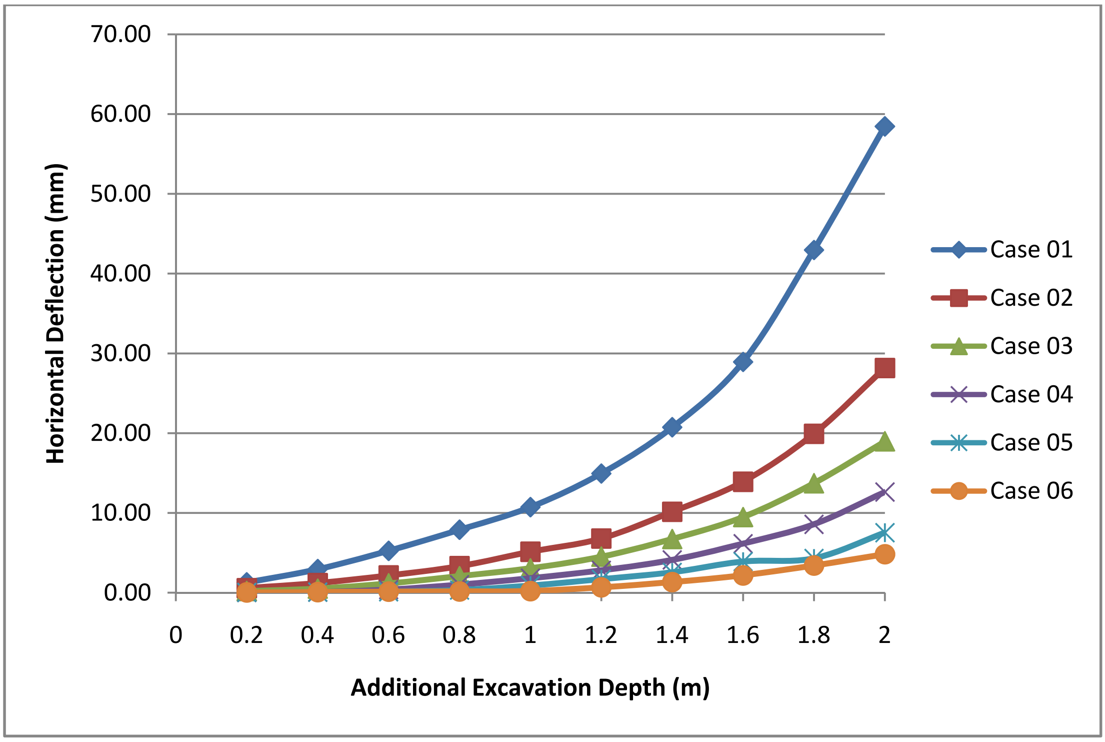

Figure 6.22 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 34 degrees (Cantilever Retaining Wall)

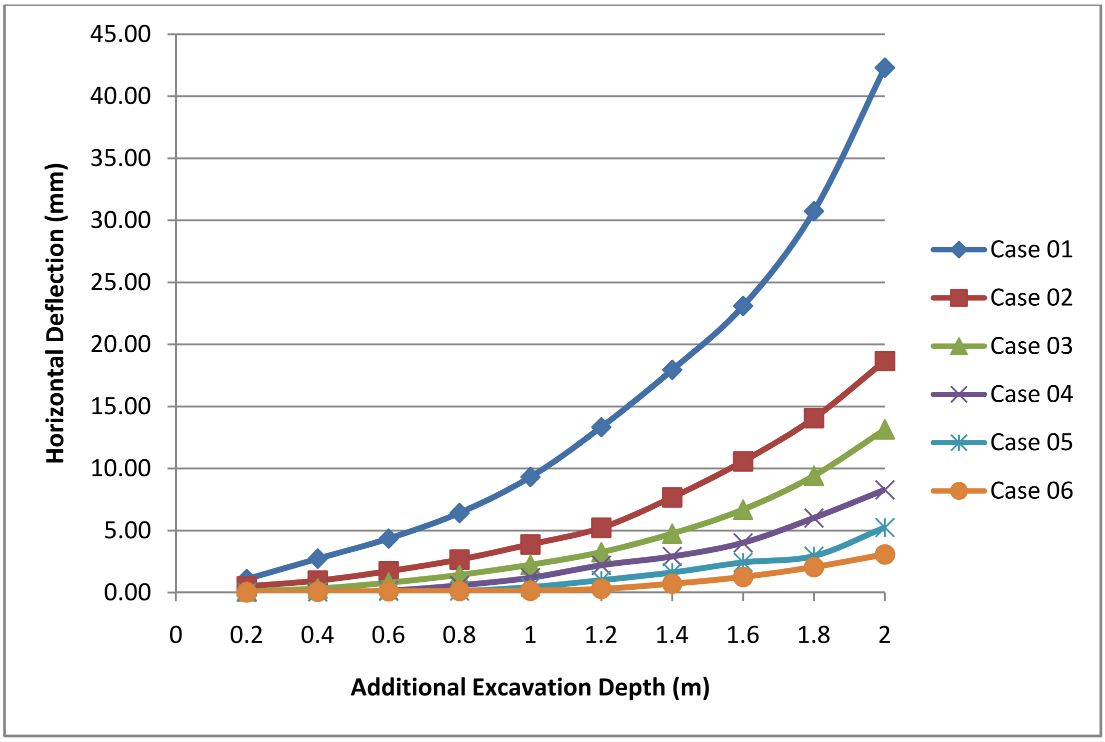

Figure 6.23 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 36 degrees (Cantilever Retaining Wall)

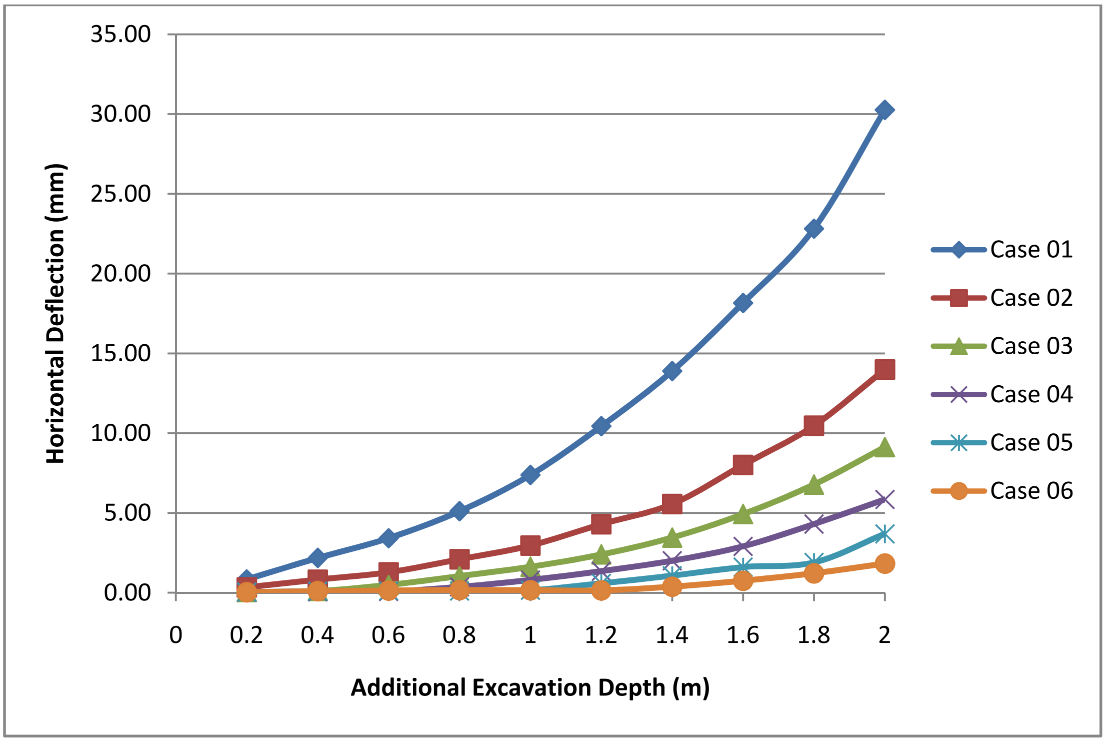

Figure 6.24 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 38 degrees (Cantilever Retaining Wall)

The results shows that the significant improvement can be achieved from proposed improvement method for cantilever retaining walls as well.

The existing case results shows that only for internal friction angle more than 32 degrees sand can be dredged up to 5.0m retaining height. But with the improvement it can be done even in sand with friction angle of 30 degrees. So it is a significant improvement in the stability with the proposed solution technique.

Further it can be observed that the combined structure fails due to the instability of the improvement sheet pile wall. The horizontal displacement and the bending moment of the improvement wall at failure depth or maximum excavation depth is tabulated in Appendix B

So the depth of embedment of the improvement sheet pile wall is extended by 2.0m and dredging is further extended up to 6m retaining height to observe the behavior of the wall and the significance of the improvement.

The cases are tested the distance between the existing sheet pile wall and the improvement wall (X_d) varying at 1.0m interval only for case 2,4 and 6.

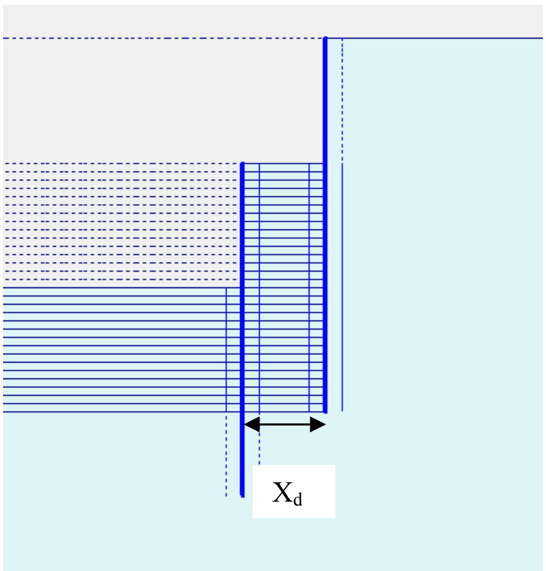

Figure 6.25 The Cantilever Sheet Pile Model in Plaxis (Extended Depth of Embedment of Improvement Wall)

The results of the case 1 and case 6 are tabulated in table (6.8) and table (6.9)

Table 6.8 Results of case 1 analysis - cantilever retaining wall (8m improvement wall)

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	297.42	149.57	3.8*
24	198.73	145.7	4*
26	114.81	140.35	4.4*
28	46.59	117.76	4.4*
30	80.66	131.63	4.8*
32	42.99	110.56	4.8*
34	103.16	132.33	5.4*
36	62.42	114.91	5.4*
38	47.24	102.95	5.4*

Table 6.9 Results of case 6 analysis - cantilever retaining wall (8m improvement wall)

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	251.13	89.66	5*
24	33.69	74.37	4.6*
26	39.32	64.7	5.2*
28	89.68	56.58	6*
30	51.6	53.51	6*
32	33.61	45.13	6
34	20.72	36.96	6
36	13.97	31.85	6
38	9.71	26.93	6

* soil body collapses in the numerical model

Above results shows that with the increased depth of embedment of the improvement sheet pile wall , stability of the composite structure has been increased. Even in the sand with 38 degrees of internal friction angle It's not possible to excavate beyond 5.4 m. But with the improvement sheet pile wall it's possible to excavate up to 6 m retaining height in the sand with internal friction angle higher than 32 degrees. So it also proves that a significant improvement can be achieved from installing an improvement sheet pile row.

The horizontal deflection variation when increasing the retaining height for Case 1, 2, 4 and 6 are plotted in Figure 6.26 to 6.32

Appendix A shows the results of the other cases of the finite element analysis.

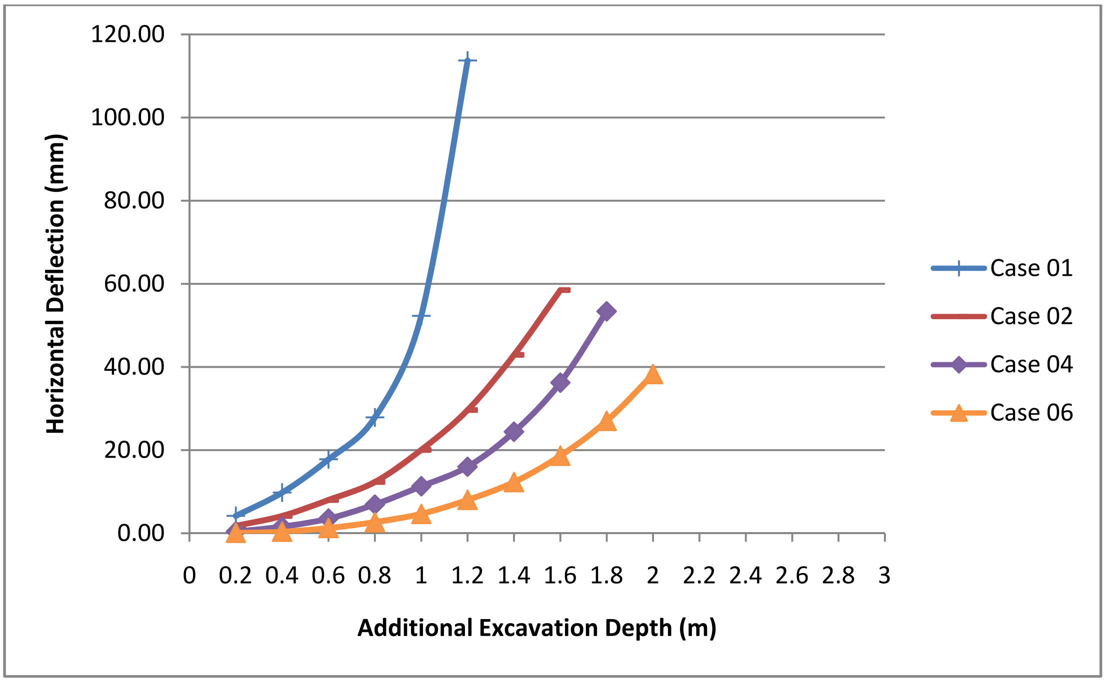

Figure 6.26 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 26 degrees - Cantilever Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

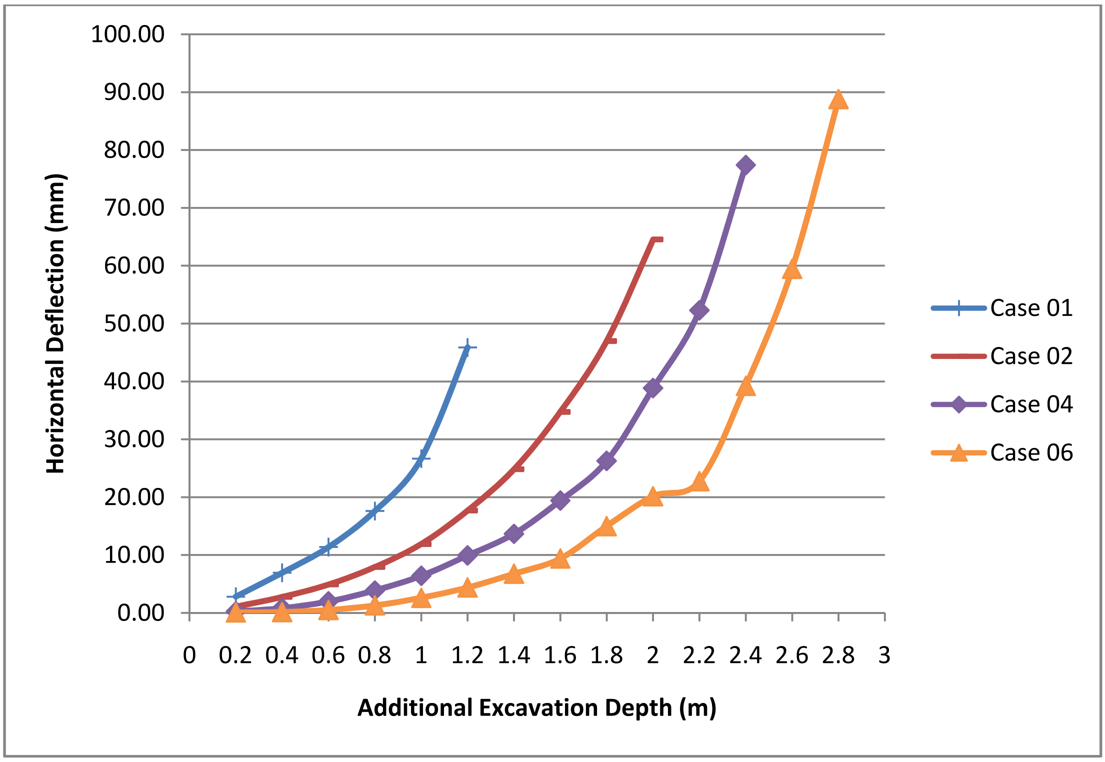

Figure 6.27 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 28 degrees - Cantilever Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

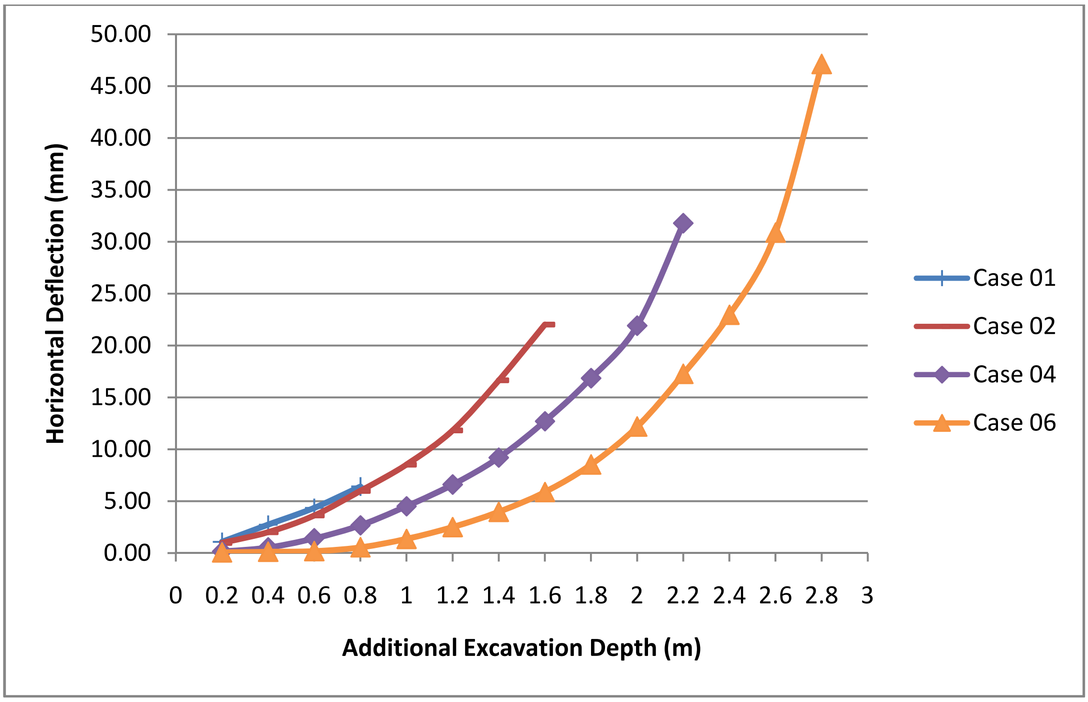

Figure 6.28 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 30 degrees - Cantilever Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

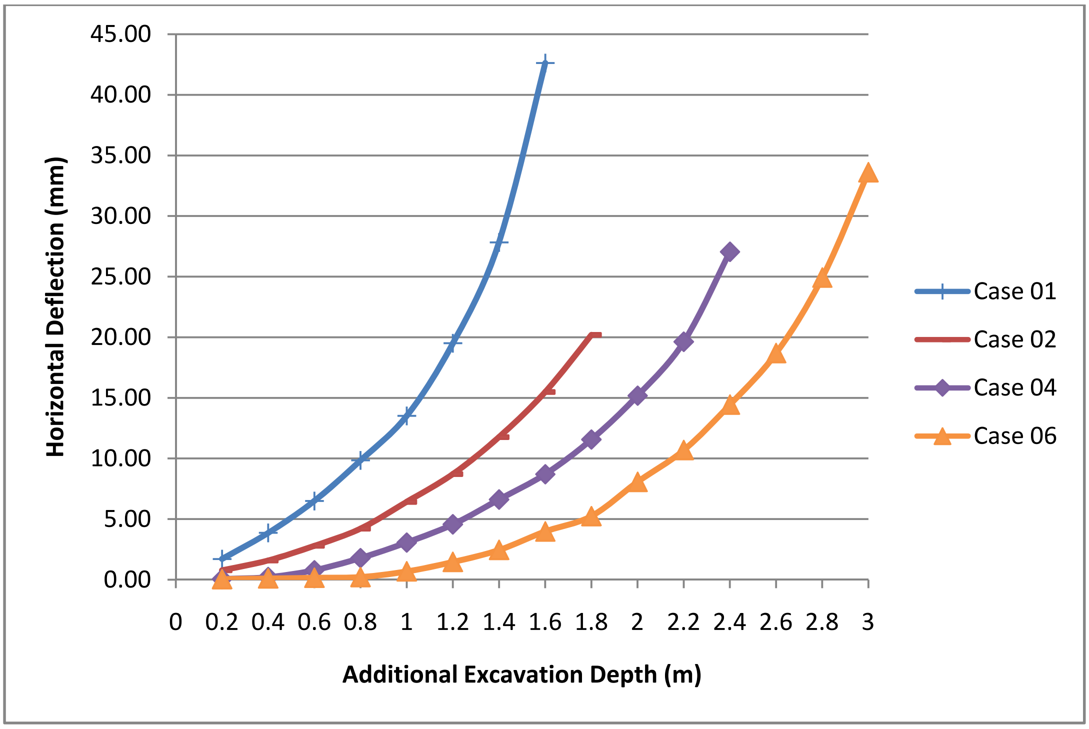

Figure 6.29 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 32 degrees - Cantilever Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

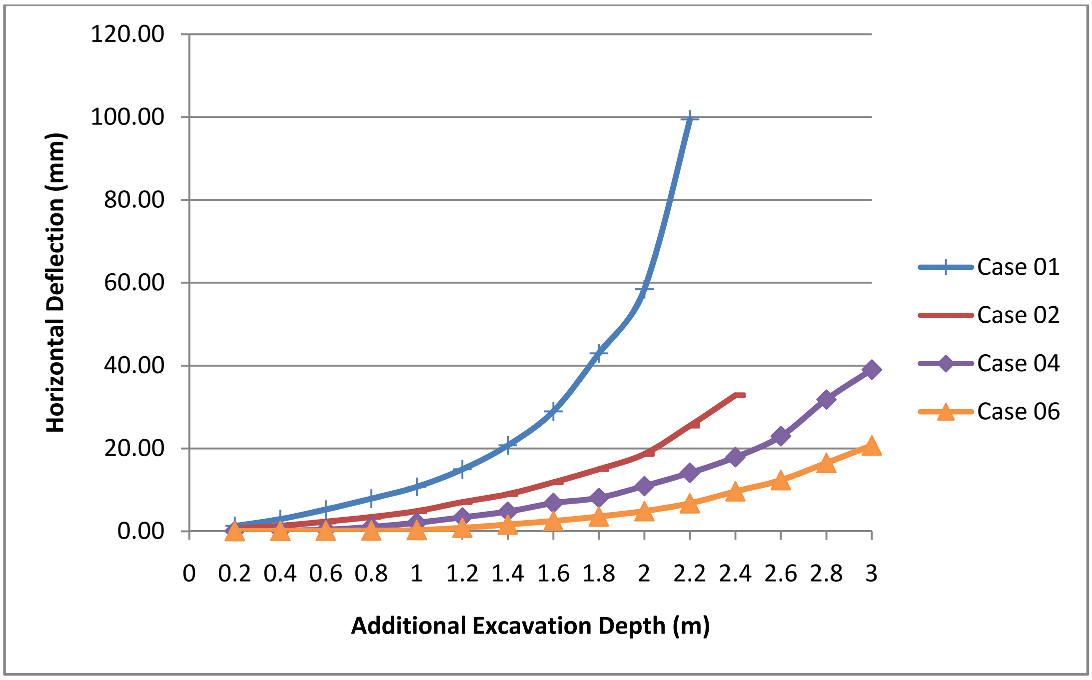

Figure 6.30 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 34 degrees - Cantilever Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

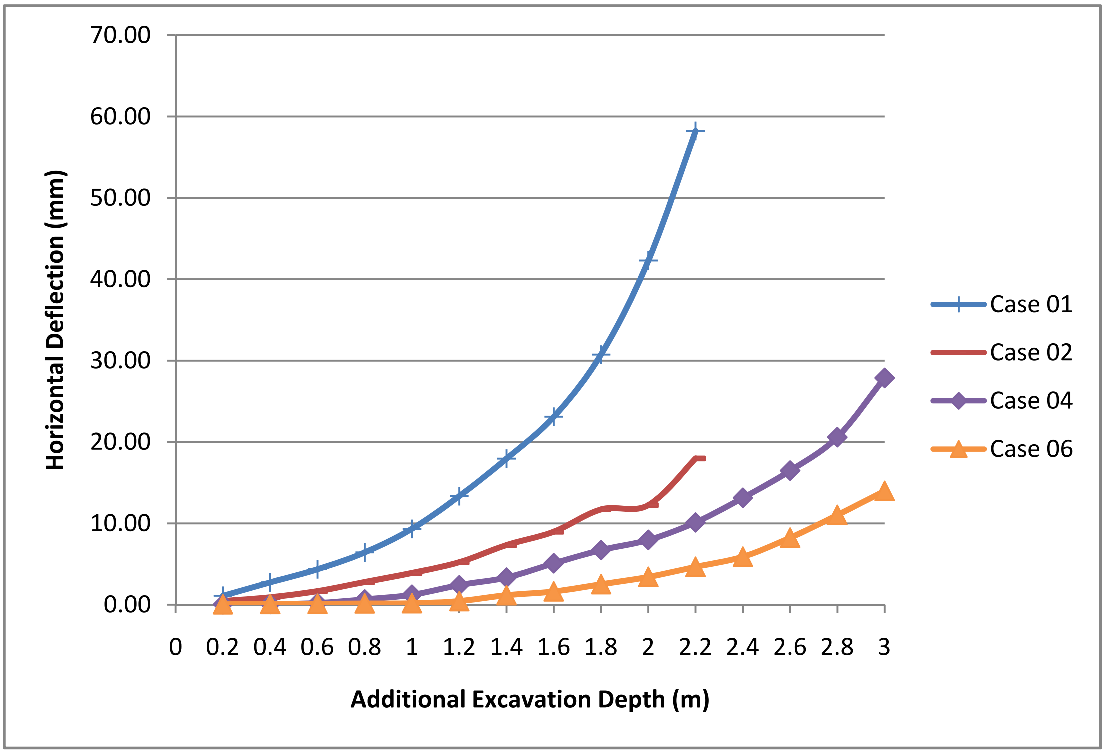

Figure 6.31 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 36 degrees - Cantilever Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

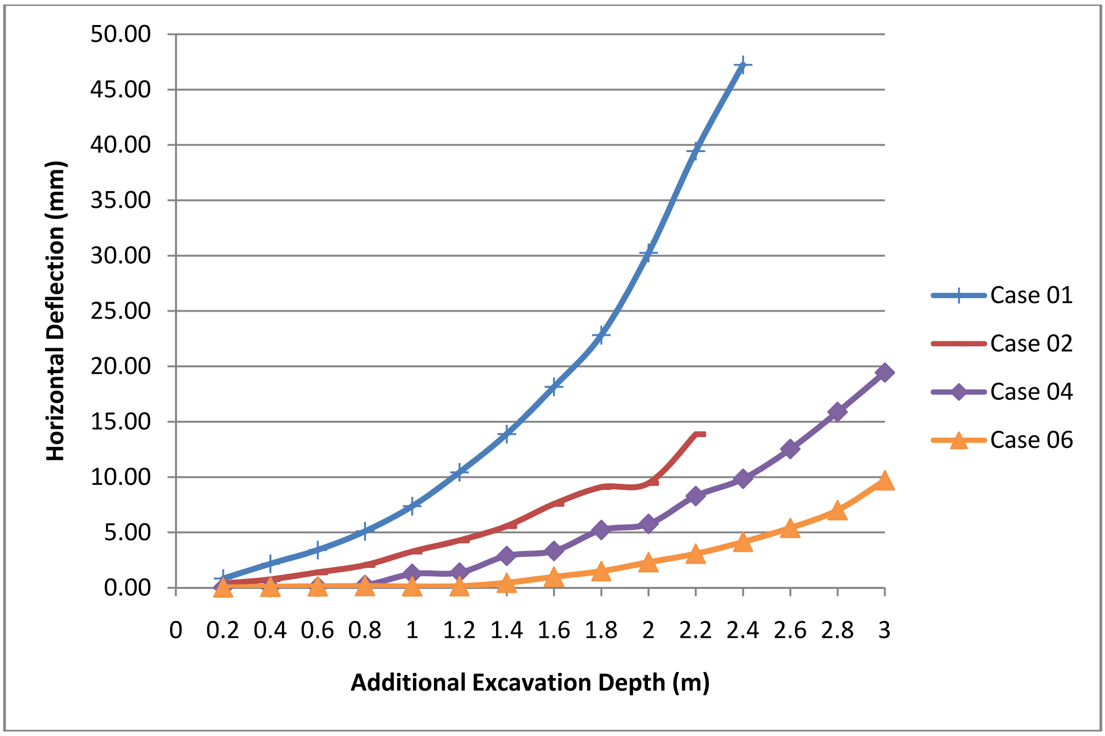

Figure 6.32 Additional Excavation Depth Vs Horizontal Deflection for Internal Friction Angle of 38 degrees - Cantilever Retaining Wall

(Extended Depth of Embedment of Improvement Wall)

7. CONCLUSION AND RECOMMENDATION

Based on the field study done during the research it's observed that there is a requirement of dredging most of existing fishery harbours to facilitate newly build fishing boats with higher draft. But the major issue of further dredging of the harbour basin is the stability of the existing quay wall. Re construction of quay wall and nearby onshore structures cost a lot of money and resources. So if it is possible to dredge further without damaging the existing quay wall, it saves lot of additional work.

Canal and drain system improvement without damaging existing bank protection also save lot of money and avoid many social issues in urban areas. If it is need to dredge more than the design limit in sheet pile wall , either more anchors need to be placed or remaining passive side soil need to be improved. It's very difficult to place new anchors in already developed site and soil improvement in passive side is also not an easy task with the available technology.

Based on the results observed through finite element analysis a significant improvement can be achieved from the proposed technique. Two main scenarios are modelled for sheet pile retaining walls based on an assumption made. Which was the possible driving depth of the improvement sheet pile wall is the same depth of embedment of the existing sheet pile wall. . Only the soil with internal friction angle higher than 26 degrees could be excavate up to 7m retaining height and which shows a horizontal deflection of 45 mm. But after the improvement proposed under case 6 , 45mm deflection could be reduced to 6 mm. Under that assumption the improvement which could achieve in the anchored sheet pile wall is not much significant. Because of the less depth of embedment of the improvement wall , improvement wall fails first. So modelling was extended beyond the assumption and depth of embedment of improvement retaining wall is extended. Then the significant improvement could be obtained from improvement wall as it is shown in Figure 6.10 to Figure 6.16

Finite element analysis done for the cantilever sheet pile wall under the previous assumption shows significant improvement in the existing sheet pile wall. Only the soil with internal friction angle higher than 36 degrees could be excavate up to 5m retaining height and which shows a horizontal deflection of 58 mm. But after the improvement proposed under case 6 , 58mm deflection could be reduced to 3 mm.

As it is done for the anchored sheet pile wall , the depth of embedment of the improvement sheet pile wall is increased and stability improvement of the existing sheet pile wall become significantly higher as shown in Figure 6.26 to Figure 6.32

The improvement sheet pile wall has a significant effect on the stability of the existing sheet pile wall. The effect causes reduction in the maximum bending moment on the wall , reduction in the anchor force in anchored sheet pile walls as well as reduction in horizontal deflection of the wall. So This technique can be applied where it is need to be done further dredging or excavation beyond it 's design limit of a sheet pile wall. In cantilever sheet pile walls more improvement can be expect from this method compared to anchored sheet pile wall. It is recommended to check the ability to use this improvement sheet pile wall technique whenever there is a requirement to dredge or excavate small amount beyond its original design limit. which will reduce waste of money and resources as well as social and environmental issues which could have arisen in removing and reconstruction process.

7.1 Recommendation for Future Research

The studies done in this research is limited to cohesion less soil and few scenarios of dredging. This study may extended to cover the following suggested points

1. This analysis can be done for sheet pile walls with different driven depths as well as different retaining heights
2. This study can be extend for cohesive soils and layered different soil mediums
3. Under this study only one type of sheet pile is used for existing and improvement walls , that can be change and specially more stiff sheet pile can be used for the improvement wall
4. The finite element analysis can be done using other soil models and compare with this study as well as the physical model.
5. Physical modeling can be improved in an advance laboratory and validation can be extended for different cases.
6. A method like stone columns/piles can be use alone or together with this method to improve the passive side soil.

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APPENDIX A

Finite element model results
(Parameters of the main retaining wall)

1.1 Anchored Sheet Pile Wall Model Results - (4m Improvement wall)

Case 1

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	204.51	190.93	115.6	6.8*
24	32.16	146.62	85.74	6.8*
26	45.17	153.91	97.38	7
28	10.17	117.14	66.38	7
30	7.23	98.98	57.93	7
32	5.83	82.89	52.06	7
34	4.93	71.36	47.37	7
36	4.05	60	42.33	7
38	3.33	50.51	37.75	7

Case 2

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	20.48	139.73	73.1	6.6*
24	22.1	140.13	77.37	7*
26	12.59	124	68.26	7
28	7.92	101.41	57.22	7
30	6.13	85.66	52.03	7
32	4.89	72.14	46.58	7
34	3.94	60.62	41.83	7
36	3.05	49.3	36.78	7
38	2.24	38.18	31.12	7

Case 3

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	37.14	149.88	83.08	6.8*
24	17.6	129.73	69.58	7*
26	11.1	113.84	63.21	7
28	6.91	91.9	54.25	7
30	5.29	78.26	49.21	7
32	4.01	63.4	43.37	7
34	3.14	52.61	38.28	7
36	2.4	42.53	33.59	7
38	1.85	34.61	29.02	7

Case 4

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	31.44	144.39	78.97	6.8*
24	14.23	119.69	63.94	7*
26	9.03	104.01	57.82	7
28	5.84	84.25	50.88	7
30	4.43	71.06	46.06	7
32	3.29	56.71	40.28	7
34	2.46	45.53	34.82	7
36	1.85	36.96	30.46	7
38	1.39	29.92	26.17	7

Case 5

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	52.37	148.59	89.41	6.8*
24	20.62	125.08	71.93	7*
26	7.52	97.52	54.71	7
28	4.85	77.85	47.88	7
30	3.62	63.57	42.9	7
32	2.58	49.9	36.96	7
34	1.9	39.91	31.82	7
36	1.36	31.95	27.09	7
38	0.933	25.31	22.79	7

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	41.53	143.53	84.31	6.8*
24	15.27	117.76	64.65	7*
26	6.13	91.99	51.87	7
28	4.15	73.25	45.64	7
30	2.97	57.77	40.09	7
32	2.01	44.27	33.43	7
34	1.39	34.93	28.4	7
36	0.945	27.77	24.16	7
38	0.607	22.48	20.37	7

* soil body collapses in the numerical model

1.2 Anchored Sheet Pile Wall Model Results - (6m Improvement wall)

Case 1

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	204.51	190.93	115.6	6.6*
24	32.16	146.62	85.74	6.8*
26	30.26	149.23	92.49	7.2*
28	18.72	136.33	82.52	7.4*
30	19.07	132.51	87.73	7.6*
32	15.57	120.98	82.37	7.8*
34	8.81	97.26	62.32	7.8*
36	9.36	93.12	64.43	8*
38	8.42	85.03	61.7	8

Case 2

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	42.32	147.96	90.74	6.8*
24	29.47	132.97	83.63	7*
26	19.84	119.23	80.46	7.2*
28	16.18	108.01	78.19	7.4*
30	10.52	90.39	66.3	7.6*
32	8.63	80.28	61.83	7.8*
34	7.48	71.82	57.83	8*
36	6.56	63.79	54.11	8
38	5.02	53.7	44.91	8

Case 4

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	29.55	130.41	79.23	6.8*
24	16.34	110.18	66.94	7*
26	30.7	123.22	91.17	7.6*
28	19.88	109.01	84.14	7.8*
30	13.16	93.85	73.83	8*
32	10.03	79.73	65.92	8
34	6.45	62.47	52.56	8
36	4.39	49.21	40.94	8
38	3.22	39.42	33.83	8

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Anchor Force (kN)	Excavation Depth (m)
22	41.53	143.53	84.31	6.8*
24	15.27	117.76	64.65	6.8*
26	6.13	91.99	51.87	7
28	4.15	73.25	45.64	7
30	2.97	57.77	40.09	7
32	2.01	44.27	33.43	7
34	1.39	34.93	28.4	7
36	0.945	27.77	24.16	7
38	0.607	22.48	20.37	7

* soil body collapses in the numerical model

1.3 Cantilever Sheet Pile Wall Model Results - (6m Improvement wall)

Case 1

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	297.42	149.57	3.8*
24	198.73	145.7	4*
26	114.81	140.35	4.4*
28	46.59	117.76	4.4*
30	80.66	131.63	4.8*
32	42.99	110.56	4.8*
34	58.46	116.09	5
36	42.3	102.65	5
38	30.26	88.28	5

Case 2

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	135.34	93.42	4*
24	44.15	77.54	4*
26	104.09	78.15	4.6*
28	66.27	68.72	4.8*
30	36.48	56.81	4.8*
32	45.28	55	5*
34	28.18	44.89	5
36	18.67	38.3	5
38	13.99	33.78	5

Case 3

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	201.46	96.53	4.2*
24	51.63	78.08	4.2*
26	98.65	77.72	4.6*
28	44.21	63.57	4.8*
30	34.7	55.03	5*
32	29.22	47.99	5
34	18.97	40.48	5
36	13.15	33.31	5
38	9.13	28.22	5

Case 4

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	106.97	94.29	4.2*
24	130.6	88.04	4.6*
26	37.08	67.98	4.6*
28	46.09	63.92	5*
30	33.52	52.89	5
32	19.23	43.93	5
34	12.63	36.47	5
36	8.29	30.61	5
38	5.85	26.1	5

Case 5

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	163.37	96.44	4.4*
24	184.38	86.52	4.8*
26	79.01	75.13	5*
28	48.77	64.22	5
30	20.76	49.43	5
32	12.69	41.84	5
34	7.53	34.15	5
36	5.25	29.2	5
38	3.69	25.77	5

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	264.56	100.16	4.4*
24	193.87	89.19	4.8*
26	21.53	67.52	5
28	29	59.58	5
30	14.08	50.44	5
32	7.85	40.47	5
34	4.83	33.73	5
36	3.08	29.34	5
38	1.83	24.83	5

* soil body collapses in the numerical model

1.4 Cantilever Sheet Pile Wall Model Results - (8m Improvement wall)

Case 1

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	297.42	149.57	3.8*
24	198.73	145.7	4*
26	114.81	140.35	4.4*
28	46.59	117.76	4.4*
30	80.66	131.63	4.8*
32	42.99	110.56	4.8*
34	103.16	132.33	5.4*
36	62.42	114.91	5.4*
38	47.24	102.95	5.4*

Case 2

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	134.63	84.85	4.4*
24	227.7	72.55	5*
26	60.23	71.44	4.8*
28	65.05	67.06	5.2*
30	22.6	55.33	4.8*
32	60.96	48.87	5*
34	33.15	47.36	5.6*
36	18.21	38.07	5.4*
38	14.37	33.14	5.4*

Case 4

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	37.03	87.27	4*
24	39.67	75.19	4.4*
26	54.56	68.36	5*
28	78.78	59.74	5.4*
30	32.85	50.16	5.4*
32	27.83	42.44	5.6*
34	39	36.99	6
36	27.87	31.56	6
38	26.04	27.45	6

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	251.13	89.66	5*
24	33.69	74.37	4.6*
26	39.32	64.7	5.2*
28	89.68	56.58	6*
30	51.6	53.51	6*
32	33.61	45.13	6
34	20.72	36.96	6
36	13.97	31.85	6
38	9.71	26.93	6

* soil body collapses in the numerical model

APPENDIX B

Finite element model results
(Parameters of the improvement wall)

2.1 Anchored sheet pile wall - Parameters of 4m improvement wall

Case 2

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Retaining Height (m)
22	42.56	4.84	6.6
24	56.61	6.65	7
26	35.09	7.39	7
28	17.37	7.94	7
30	11.89	8.97	7
32	8.71	9.94	7
34	6.62	10.66	7
36	4.93	11.24	7
38	3.59	11.94	7

Case 3

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Retaining Height (m)
22	81.85	5.77	6.8
24	50.96	6.65	7
26	32.37	7.62	7
28	16.23	8.51	7
30	11.01	9.72	7
32	7.83	10.54	7
34	5.8	11.01	7
36	4.25	11.05	7
38	3.19	11.08	7

Case 4

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth
22	87.46	6.56	6.8
24	44.18	6.68	7
26	32.37	7.62	7
28	14.57	8.68	7
30	10.59	9.43	7
32	7.61	9.98	7
34	4.89	10.43	7
36	3.58	10.41	7
38	2.61	10.63	7

Case 5

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth
22	163.84	7.46	6.8
24	90.61	8.44	7
26	24.36	8.26	7
28	12.63	8.7	7
30	8.3	9.09	7
32	5.52	9.14	7
34	3.91	9.19	7
36	2.82	9.34	7
38	1.93	9.71	7

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth
22	159.14	7.93	7
24	79.73	9.15	7
26	20.89	8.51	7
28	11.46	8.71	7
30	7.26	8.91	7
32	4.71	8.84	7
34	3.27	8.61	7
36	2.18	8.58	7
38	1.43	8.67	7

* soil body collapses in the numerical model

2.2 Anchored sheet pile wall - Parameters of 6m improvement wall

Case 2

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth
22	72.82	18.81	6.6*
24	53.4	19.64	7*
26	37.3	20.43	7.2*
28	30.33	21.13	7.4*
30	17.69	21.91	7.6*
32	14.2	22.13	7.8*
34	12.2	21.68	8*
36	10.72	20.85	8
38	7.52	21.5	8

Case 4

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth
22	49.43	22.98	6.8*
24	29.85	22.05	7*
26	62.01	23.97	7.6*
28	42.93	22.53	7.8*
30	26.83	21.71	8*
32	20.15	21.79	8
34	11.54	22.61	8
36	7.29	22.53	8
38	5.14	21.26	8

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth
22	149.82	25.66	7.2*
24	176.82	24.87	7.6*
26	56.71	24.39	7.8*
28	19.5	25.67	7.8*
30	21.1	25.27	8
32	10.26	25.52	8
34	6.39	23.7	8
36	4.48	21.37	8
38	3.32	18.99	8

* soil body collapses in the numerical model

2.3 Cantilever sheet pile wall - Parameters of 6m improvement wall

Case 2

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	83.14	43.36	4*
24	26.5	32.78	4*
26	65.1	55.88	4.6*
28	41.21	54.95	4.8*
30	22.26	48.14	4.8*
32	27.91	55.83	5
34	17.01	47.71	5
36	10.97	40.79	5
38	8.03	36.02	5

Case 3

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	118.98	47.25	4.2*
24	30.58	36.59	4.2*
26	60.44	52.31	4.6*
28	26.88	47.64	4.8*
30	21.18	46.97	5*
32	17.83	46.9	5
34	11.27	40.45	5
36	7.77	34.51	5
38	5.36	28.77	5

Case 4

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	58.81	38.75	4.2*
24	75.6	45.46	4.6*
26	22.1	39.79	4.6*
28	28.28	47.61	5*
30	20.89	46.45	5
32	11.77	40.1	5
34	7.7	33.73	5
36	5.05	27.57	5
38	3.57	23.11	5

Case 5

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	86.61	42.89	4.4*
24	106.6	47.52	4.8*
26	47.27	46.83	5*
28	29.53	45.59	5
30	12.8	38.05	5
32	7.77	32.35	5
34	4.71	25.92	5
36	3.3	21.5	5
38	2.28	17.74	5

Case 6

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	137.06	43.96	4.4*
24	109.92	45.08	4.8*
26	15.69	51.64	5
28	17.56	38.34	5
30	8.44	31.54	5
32	4.89	25.1	5
34	3.05	19.96	5
36	1.97	15.95	5
38	1.24	12.64	5

* soil body collapses in the numerical model

2.4 Cantilever sheet pile wall - Parameters of 8m improvement wall

Case 2

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	76.73	126.55	4.4*
24	131.69	179.83	5*
26	35.78	98.92	4.8*
28	39.13	114.91	5.2*
30	13.28	54.12	4.6*
32	12.15	53.35	5*
34	20.26	80.58	5.6*
36	10.8	51.68	5.4*
38	8.39	43.81	5.4*

Case 4

Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	18.59	39.43	4*
24	21.66	53.89	4.6*
26	30.66	81.77	5*
28	48.07	119.06	5.6*
30	20.72	75.31	5.4*
32	17.8	70.28	5.6*
34	25.19	87.12	6
36	18.05	70.89	6
38	12.53	55.66	6

Case 6

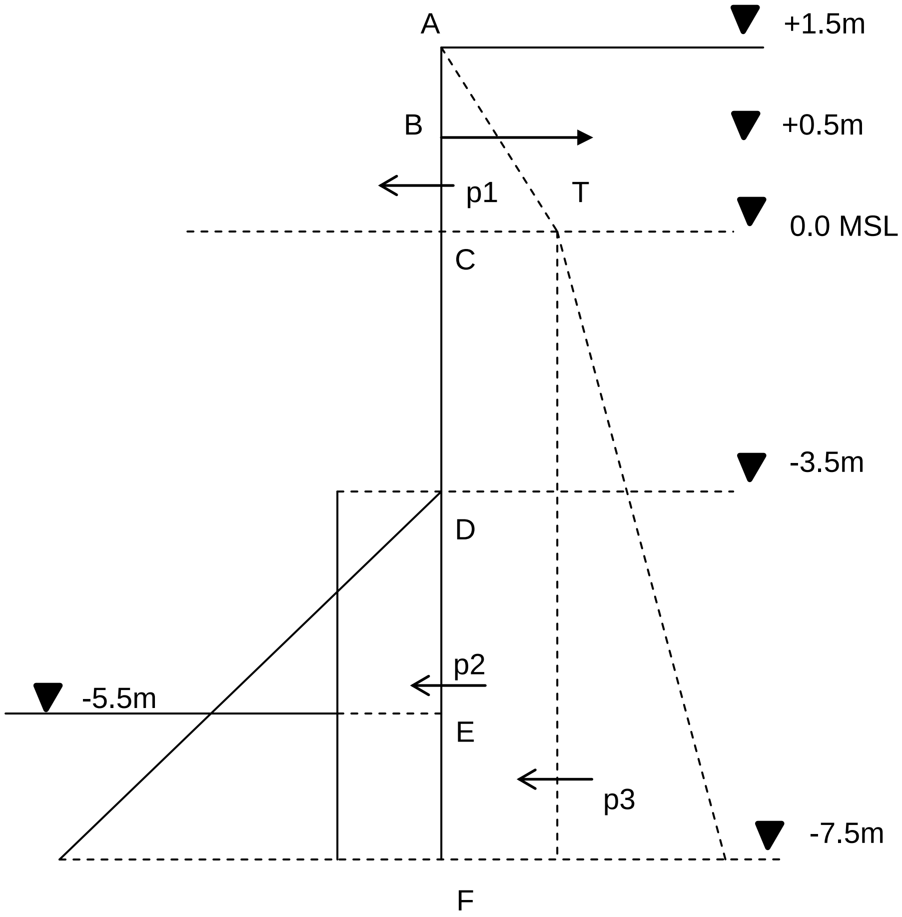
Int. Fric.Angle	Wall Hori. Def (mm)	Bending Mom. (kNm)	Excavation Depth (m)
22	114	118.13	5*
24	18.18	48.52	4.6*
26	22.8	66.52	5.2*
28	52.29	109.98	6*
30	30	87.06	6*
32	20.15	70.01	6
34	12.65	52.57	6
36	8.56	41.1	6
38	6.07	32.91	6

* soil body collapses in the numerical model

APPENDIX C

Sample Calculation

REF	CALCULATION	OUTPUT
	Anchored Sheet Pile Wall	



<u>Active side</u>

Ka=1/3

Kp=3

σ_A = 0kN/m

$\sigma_C = k_a \gamma Z$
=1/3 × 17 × 1.5
=8.5kN/m^2

$\sigma_F = k_a[\gamma z_1 + (\gamma - \gamma_w) z_2]$
= 8.5+1/3 x 9 x 7.5
= 19kN/m^2

P_1=1/2 x 8.5 x 1.5

=6.375kN

P_2=8.5 x 7.5

=63.75kN

P_3=1/2 x7.5 x 22.5

=84.375kN

<u>Passive side</u>

$\sigma_{F,ult}$ = kp x γ x z

= 3 x 9 x 4

= 108kN/m^2

$Fp_{,ult}$ =1/2 x 108 x 4

= 216 kN

F , take moments around point F

0 = P_1 * 0.5 + P_2 * 4.75 + P3 x 6 - Fp * 7.17

Fp, = 113.28 kN

FOS = 216/113.28 =1.91

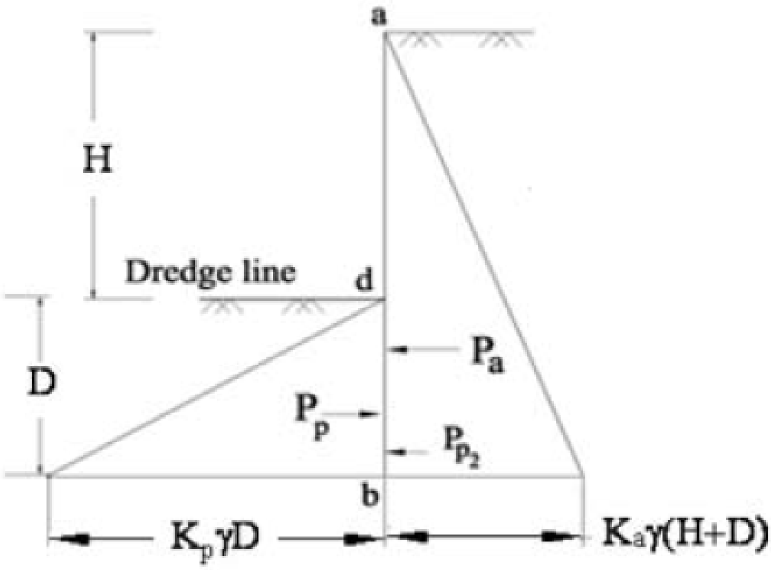
→ Horizontal force equilibrium

P_1+ P_2 + P_3 = Fp + T

T = 41.22 kN

Anchor Force = 41.22 kN

Improvement Sheet Pile wall - Anchored Sheet Pile Structure



H = 3.0 m

φ = 30̊

γ = 19 kN/m^3

Taking moments about b;

$P_p \times \frac{D}{3} - P_a \times \frac{(H+D)}{3} = 0$

$$K_p\gamma_s D^3 - K_a\gamma_s(H+D)^3 = 0$$

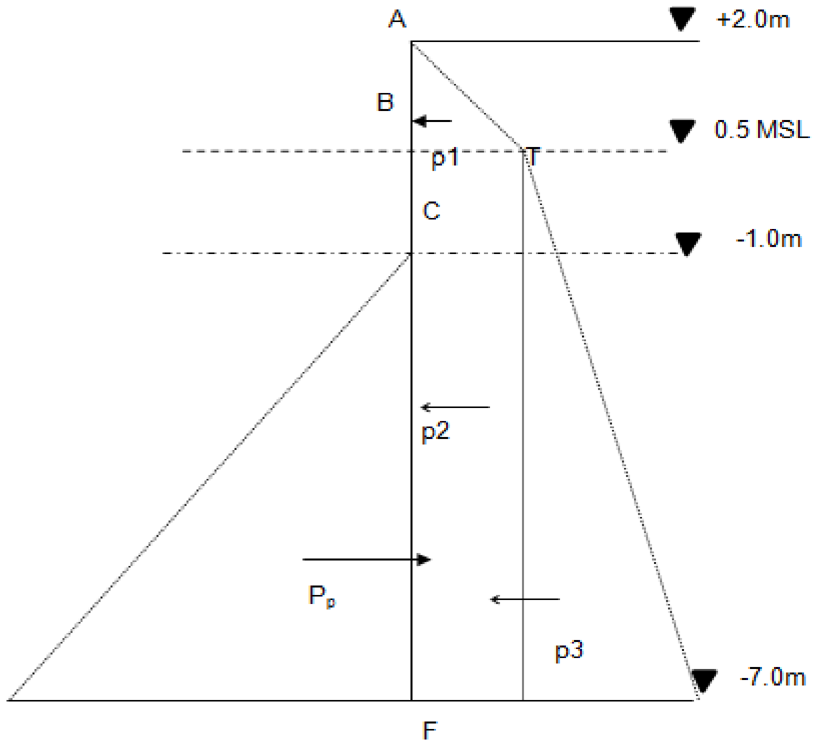
D = 1.85 m

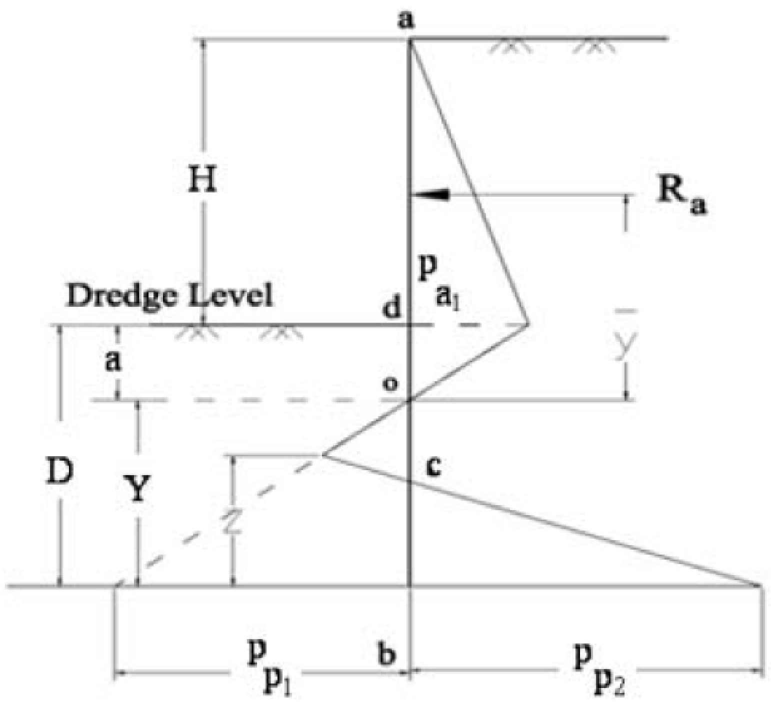
D_{actual} = 2.0 m

D_{actual} = D_{design} x 8%

(20% - 40% recommended)

Cantilever Sheet Pile Wall





At point o ;

$K_p \gamma a - (P_{a1} + K_a \gamma a) = 0$

a = 0.54 m

$P_{p1} = (K_p - K_a) \gamma Y$

$P_{p2} = K_p \gamma (H + a + Y) - Ka \gamma (a + Y)$

From the pressure diagram ;

$\bar{Y} = 1.44$ m
Ra = 26.01 kN

$\Sigma H = 0$

$Ra + 1/2 (Pp1 + Pp2) Z - 1/2\ Pp1\ Y = 0$

taking moments about base b ; $\Sigma M = 0$

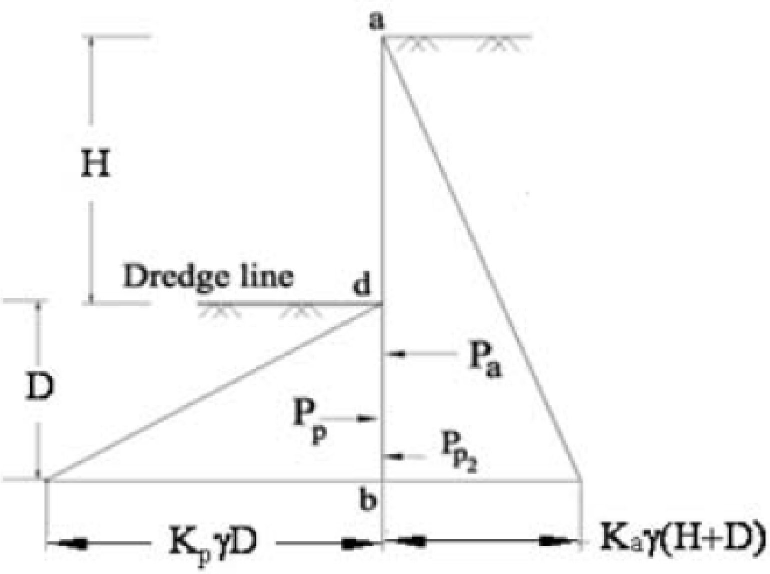
$Ra (\bar{Y} + Y) + 1/2 (Pp1 + Pp2) Z. \frac{Z}{3} - 1/2\ Pp1\ Y. \frac{Y}{3} = 0$

Y = 4.43 m
D = Y + a
D = 4.97 ~ 5 m

D design = D + D x 20%

= 6.0 m

Improvement Sheet Pile wall - Cantilever Sheet Pile Structure



H = 2.0 m

φ = 30̊

γ = 19 kN/m^3

Taking moments about b;

$P_p \times \frac{D}{3} - P_a \times \frac{(H+D)}{3} = 0$

$$K_p\gamma_s D^3 - K_a\gamma_s (H+D)^3 = 0$$

D = 1.85 m

D_{actual} = 3.0 m

D_{actual} = D_{design} x 62%

(20% - 40% recommended)