INVESTIGATION OF MECHANISM OF SOIL DEFORMATION AT BERTH 7 & 8 PELABUHAN TANJUNG PELEPAS

AZIZUL HAKIM BIN ISMAIL

CIVIL ENGINEERING
UNIVERSITI TEKNOLOGI PETRONAS

JULY 2009

Investigation Of Mechanism Of Soil Deformation At Berth 7& 8 Pelabuhan Tanjung Pelepas

by

Azizul Hakim Bin Ismail

Dissertation submitted in partial fulfilment of

the requirements for the

Bachelor of Engineering (Hons)

(Civil Engineering)

JULY 2009

Universiti Teknologi PETRONAS
Bandar Seri Iskandar
31750 Tronoh
Perak Darul Ridzuan

TABLE OF CONTENTS

List Of Tables

List Of Figures

CERTIFICATION OF APPROVAL

Investigation Of Mechanism Of Soil Deformation at Berth 7 & 8 Pelabuhan Tanjung Pelepas

by

Azizul Hakim Bin Ismail

A project dissertation submitted to the
Civil Engineering Programme
Universiti Teknologi PETRONAS
in partial fulfilment of the requirement for the
BACHELOR OF ENGINEERING (Hons)
(CIVIL ENGINEERING)

Approved by,

____
(Assoc Prof Dr. Indra Sati Hamonangan Harahap)

UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

July 2009

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

AZIZUL HAKIM BIN ISMAIL

ABSTRACT

Distress on the pavement at Berth 7 & 8 Pelabuhan Tanjung Pelepas (PTP) was noticed around middle of 2006. A couple of soil depression was found on the pavement in consequence of that and it continues to happen on Berth 9 & 10. By observing the soil behavior of the site through modeling, the stress strain reaction of soil at site can be known, evaluated and studied. This research is done to study the deformation mechanism that lead to the depression of the back of wharf pavement structures. Beside it could identify the effect the soil deformation towards the berth structures. The project consists of various disciplines of civil engineering such as geotechnical engineering, structural engineering and oceanography engineering. From the study it is concluded that the soil failed by sliding and the control parameter is used to ensure sound result is produced.

ACKNOWLEDGEMENTS

First and foremost I would like to thank my supervisor Assoc Prod Dr Indra Sati Hamonangan Harahap for his assistance during this project. His kindness and guidance has helped me a lot to complete the whole study of this project.

Second to my family especially my parent and to my beloved Nur Asmah Serakim thanks for the moral support and encouragement to me in order to complete his project.

I would like to express my deepest gratitude to Mr Juraimi Masood, Mdm Mariam Ibrahim, Mr Marzuki and all the Pelabuhan Tanjung Pelepas team for their assistance in gathering the required document for this project.

Not to forget the entire consultant that assisting me in the technical matter and their views is highly appreciated. My deepest appreciation goes to Sepakat Setia Perunding Sdn. Bhd. Engineering and Environmental Consultant Sdn. Bhd. and Dr Nik Associates Sdn. Bhd.

I would like to thank also to my senior Miss Kareen Lee for her guidance and assistance especially when dealing with Plaxis software.

Last for those who are involved directly and indirectly in completion of this projects many thanks for them.

CHAPTER 1: INTRODUCTION

1.1 Background of Study

Pelabuhan Tanjung Pelepas (PTP) is one of the fastest growths of port in Malaysia that requires fast development of its infrastructure and equipment to cater their business demand. It is located at opening of Sungai Pontian near South- West Johore.

The development of PTP started on year 1995 and its first headquarters is located at Kuala Lumpur. The development is divided into 3 phases. Phase 1 development of PTP completed on year 1999 which consists of PTP office, and 6 berths 2.1 km in length and suitable for container vessel up to 120 000 tonne displacement. The extension of PTP requires the construction to be done on larger area. Thus on 2001 until 2004 the land reclamation about 400 acres of area is done. Phase 2 starts on year 2004 on reclamation land where it is an extension of the PTP area. It consists of development 8 new Berth of 2.88 km in length which the berth is designed to cater the vessel size up to 250 000 tonne displacement.

Phase 2 construction of infrastructure was begun in 2004 after the reclamation project completed which started with berth 7 & 8. The berths and the adjacent back of wharf (BOW) area were completed in 2005 and opened for operation later. As comparison in summary the table below will show the difference of berth characteristic.

Characteristic	Phase 1	Phase 2
Max vessel size (water displacement)	120 000 tonne	250 000 tonne
Vessel Berth Length	360 m	425- 450 m
Berth Depth	16.5 CD	18 m CD
Crane Rail Loading	80 tonne/m	100 tonne/ m

Table 1: Comparison Of Berth

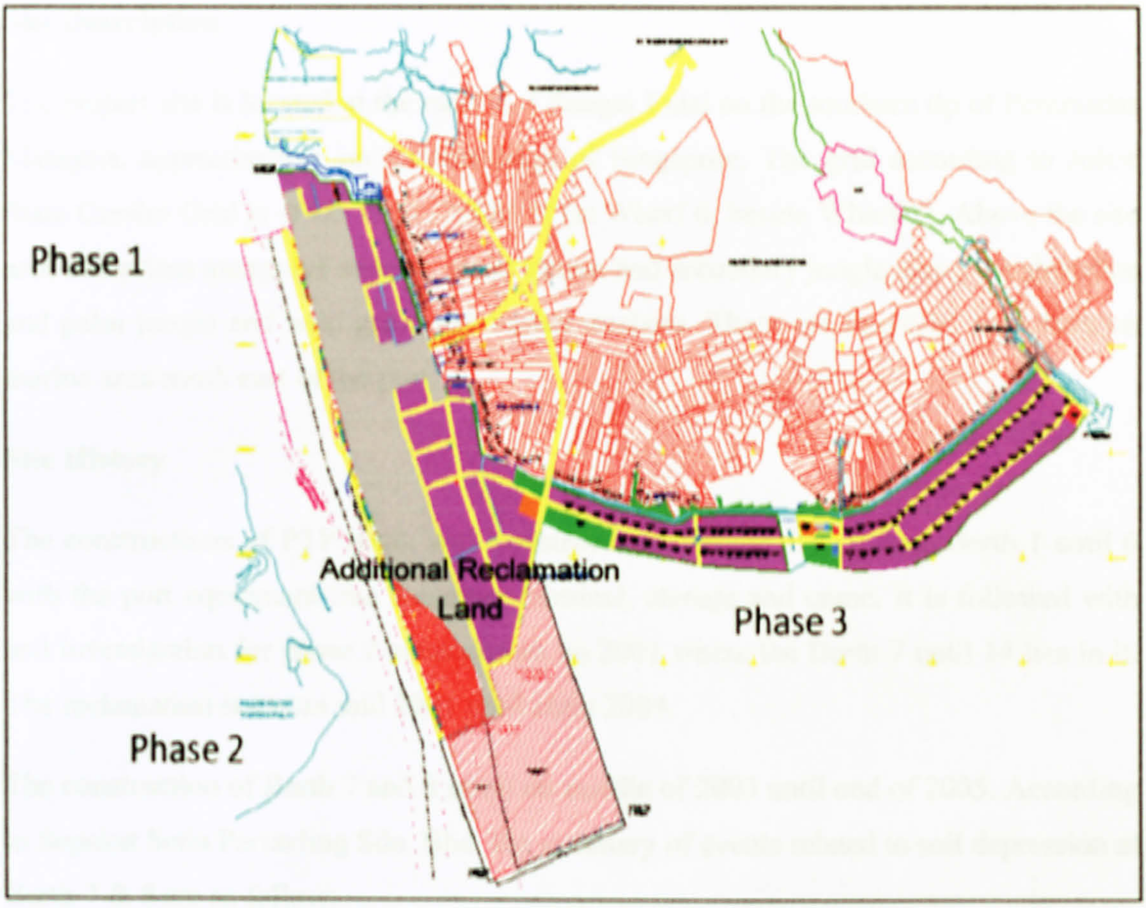


Figure 1: General PTP Layout Plan (Source: PTP Master Plan)

This newly phase 2 berths can accommodate the next generation of vessel up to 250 000 tonne displacement with length 425 to 450 m with 13 000 teu's. Minimum size of vessel that can use this berth is 6000 teu's. The vessel berth depth is 18 m CD.

The port operation is mainly on transshipment business which requires large movement and handling of container. The major equipment and facilities these ports have is various size of ship to shore crane (Quay Crane), Rubber Tyred Gantry (RTG), Prime Mover and Trailer. It operates 7 days a week continuously. It handles more than 6 millions TEU's every year that result in massive container handling in Malaysia.

Site Description

The project site is located at the mouth of Sungai Pulai on the southern tip of Peninsular Malaysia, approximately on western edge of Singapore. The grid according to Johor State Cassini Grid is -1473 m E, -76085 N (at Wharf 6, beside Wharf 7). Above the site area comprises mangrove swamp, small stream and secondary jungle along the shoreline and palm jungle and wild grassland further onshore. There are intertidal mud flats on marine area south east of the port.

Site History

The constructions of PTP begin with construction of Phase 1 PTP from Berth 1 until 6 with the port equipment like container terminal, storage and crane. It is followed with soil investigation for phase 2 construction on 2001 where the Berth 7 until 14 lies in it. The reclamation starts on mid 2001 until early 2004.

The construction of Berth 7 and 8 starts on middle of 2003 until end of 2005. According to Sepakat Setia Perunding Sdn. Bhd. the summary of events related to soil depression at Berth 7 & 8 are as follow:

19 April 2006	Apparent 2 soil depression appeared after heavy rain at rear of Berth 7 GL 7/24 and 7/83
26 April- 31 May 2006	Repair and investigation works were carried out
1 November 2006	Ground Penetration Radar (GPR) scanning trial run was carried out by Digimap
19 December 2006	Two new settlement areas at Berth 7 & 8 Observed
9- 20 January 2007	GPR Scanning Run
22 January 2007	Two more new Soil Depression observed
21 February 2007	Under deck inspection confirmed the leakage under wharf structures due to main water supply
15 March 2007	Remedial Work carried out at site
17 September 2007	New sinkholes identified and remedial works was carried out somewhere in Jan 2008

Figure 2: PTP Berth 7 & 8 Site Top View

1.2 Problem Statement

Problem identification

Distress on the pavement was noticed around middle of year 2006 and a couple of soil depression began to appear along gridline J of the wharf structure on Berth 7 & 8. The soil depressions were backfilled and some ground penetrating radar scanning GPRS survey was carried out. Around September 2007 some depression that is not localized at the pavement of the traffic lanes were found. It is continuing backfilled and repaired until now.

Significant of the project

The project represents the actual condition of problem that happens in engineering field. The need for the larger space requires the area near shore to be reclaimed. The reclamation is using sand replacement method, a proven technique in reclamation project. Place such as Dubai World Island, Hong Kong Disneyland indicates that this method is effective. The sand will squeeze the clay material underneath.

1.3 Objective & Scope of Study

This study is aimed to achieve the intended objectives which are:

- Identify the failure mechanism of soil deformation
- Identify the critical point on the cross sectional layer
- Observe the control parameter to ensure sound result is produced.

Field monitoring through modeling will be reviewed and studied. The study will be emphasized on the geotechnical problems. It will give indication on failure mechanism, critical area, stress strain behavior and able to calculate the depression at specific point. Through this the direction of soil movement and the failure mechanism could be identified.

Because of time constraint and resource availability the study will be only limited to several locations on Berth 7 & 8. Several points will be taken as the point of reference and other is assumed base on the point of reference. The modeling is 2D modeling and it is done on cross section basis.

The assumption made is the soil already achieved sufficient strength for intended construction and this mainly related with reclamation land. Beside other assumption which can be made is the construction work and method is correct and does not give high influence towards the structure.

1.4 Relevancy, Feasibility & Time Frame

The project consists of Soil Engineering, Geotechnical Aspect, Slope Stability and much of civil engineering related. Thus the student can be able to apply all the engineering knowledge and relate the theory with practice by studying the case study mentioned.

This project has certain consideration before selected. The information and data availability, cost, the assumption required and the material and software availability is the consideration criteria.

The Plaxis software is the most suitable method to be used in finite element modeling. It can model the condition of the site with soil parameter input on it. The load also can be included and the whole system stability could be analyzed. Thus it is able to indicate the failure mechanism. The project cost lies between the amounts allocated for the Final Year Project allowance. Beside the data and information required is available. The software required is presence and can be used for investigation.

It is assumed that reclamation process is completed in 6 months where it achieved its intended strength, functions and level required. This is to reduce the parameters involved in analysis.

The task breakdown will be divided into 2 semesters to cover. Semester 1 will focus mainly on literature, data gathering, theory involved and data interpretation. It is followed with analysis work on Plaxis on semester 2. The next table will show the task breakdown and its scope.

Semester	Task
Semester 1	Data gathering Data reduction Data interpretation Plaxis Input Data
Semester 2	Analysis 2D Mohr Coulomb Model- Berth 7 & 8 Determination Of Failure Mechanism

Table 2: Task Breakdown

CHAPTER 2: LITERATURE REVIEW & THEORY

2.1 Overview

Generally the phase 2 design is a semi suspended deck with high revetted slope protection. The form of the wharf is decided on economic assessment of the type of construction and depended on the requirement on the required berth depth, the ground conditions, the loading from quayside cranes and the environmental loads due to the location and degree of exposure of the wharf site. It is chosen because of proven method of construction, cost wise, engineering wise, fast construction, low steel content and durable with minimum maintenance.

The crane rail spacing can be changed from 35 m to 45.5 m without additional cost if this was required. Crane beam at back of wharf structure at the row J is used for the ease of extension of crane spacing later on. It was designed to cater the future development of crane which may have 45.5 m crane spacing due to the future vessel size. Berth 7 & 8 utilize an incomat mattress for its slope protection and as comparison berth 9 & 10 utilize interlocking rock which consists of 2 layers which is 250 kg filter rock and 1800 kg armour rock. Underneath the rock protection is the geotextile layer type KET 22. Note that for this study Berth 7 & 8 will be the main concern.

The berth is divided into 2 which are wharf area which mainly is the deck and the back of wharf area where the interlocking paver block for traffic usage lies. The loading also vary where the main load for wharf structure is combination of traffic loading, ship to shore dual hoist crane, or any container stacker while for back of wharf it mainly comes from traffic loading such as trailer movement. For every berth, it will be equipped with 3 numbers of dual hoist cranes. The crane is about 55 m height which can reach a length of 58 m across the widest section of the vessel. It has capability to reach the far side and make a twin lift of two teu's 20 footer.

Then the load from the quay crane will be taken by the crane rail thus transfer it into crane beam. Crane beam will transfer it to the pile underneath at row A & G. The load from the deck also comes from the prime mover and trailer movement to carry the container. Then the load is then transferred to the deck which later will further it to the crosshead beam. Crosshead beam will distribute the load imposed to the pile underneath.

Characteristic	Berth 7 &8
Max vessel size	250 000 tonne
Vessel's Berth Length	425- 450 m
Vessel's Berth Depth	18 m CD
Crane Rail Loading	100 tonne/ m
Length Of each Berth	360 m
No of Quay Crane/ Berth	4 for single hoist/ 3 for dual hoist

Table 3: Characteristic Of Berth

The top portion of back of wharf consists of square interlocking paver block, and then followed with lean concrete. Underneath is the compacted sand that is done during construction. The next layer is the reclamation land which said to gain enough strength before the construction begins. After that layer is the existing ground soil. The distress in the pavement is at the back of wharf.

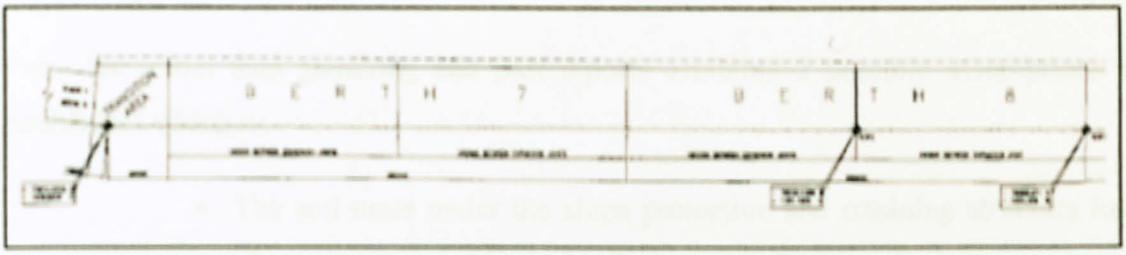


Figure 3: Berth 7- 8 Top Drawing View. (Source: B 7 & 8 Const. Dra.)

Much of the load on back of wharf will be transferred to the ground. The ground consists of the soil that gives the effect to the slope. The slope gradient is 1: 2.4 which is steeper. The slope protection used is incomatt mattress.

For 7 & 8 the mattress that used for slope protection at berth 7 & 8 is divided into 2 phase of installation part A & B. It. It is fabricated by following the pile design layout. Then it will be lowered down via divers and is inflated with non shrink grout that helps the structure to counter weight the slope soil mass. Because of the weight given by the concrete (non shrink grout concrete) it is tightened through the pile. Later the rope nose will be tightened as final stage and the slope protection complete.

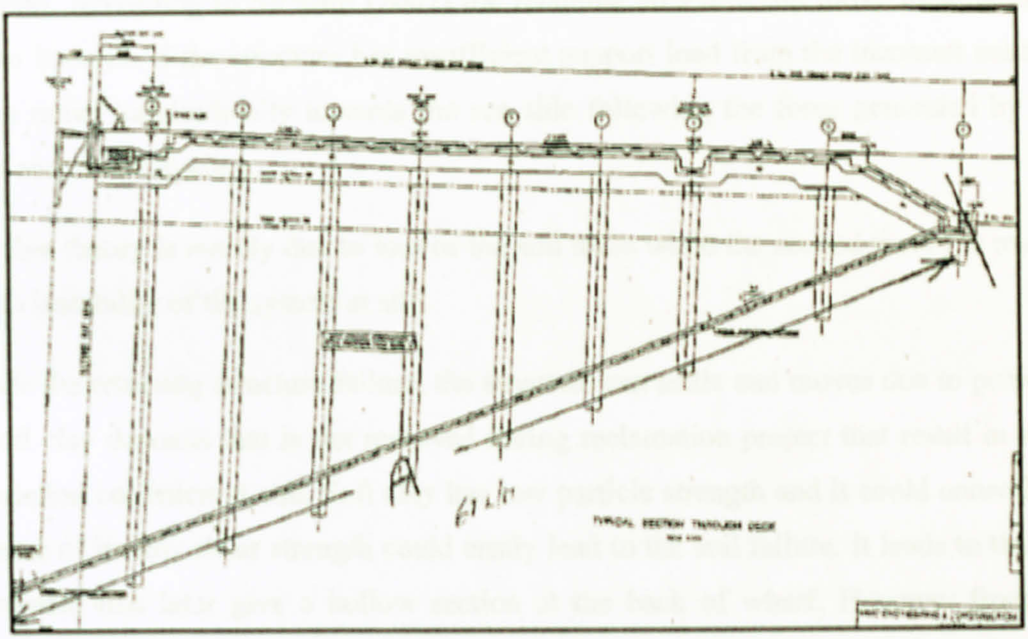


Figure 4: Cross Section Berth 7 & 8 (Source: B 7 & 8 Const. Dra.)

2.2 Theory

From the initial data gathering and past reports available 2 possible assumptions is established which is:

- The soil mass under the slope protection and retaining structure loss due to erosion and loss of soil mass near slope
- The structure settles and moves by sliding due to instability of the system that result a failure to the soil at site.

The soils can loss from the sand trespass the slope protection (e.g the incomat mattress failed to retain the soil mass at its origin). The loss might come from the erosion of soil due to the wave action and the propeller wash that comes from the ship.

Another theory established the structure also can become instable due to the failure of soil at site. The Uniform Distributed Load that comes from the traffic or could be said as traffic loading with the soil carries a mass, that generates force towards the incomatt mattress. According to Steward (2007) the retaining structure can move longitudinally due to backfill. If the structure has insufficient support load from the incomatt mattress it can move longitudinally towards the sea side following the force generated by soil mass and traffic loading.

The first theory is mainly due to loss of the soil mass while the second theory is mainly due to instability of the system at site.

Beside the retaining structure failure, the structure can settle and moves due to presence of soft clay deposits that is not removed during reclamation project that result in weak foundation condition at site. Soft clay has low particle strength and it could consolidate because of its low shear strength could easily lead to the soil failure. It leads to the soil movement that later give a hollow section at the back of wharf. However from the previous study done by Dr Nik Associates Sdn. Bhd. it is confirmed that the soil at site has achieved its desired strength and enough settlement before the construction begin. Thus this possibility is eliminated.

The mode of failure that can be expected from theory 1 is there is no significant sliding from the upper soil layer downward. Larger stress could be expected near the slope protection at marginally 1 to 10 m depth. However if it is related with theory 2 condition there is significant movement could be expected from upper soil layer downward in sliding motion and the stress is not concentrated at the incomatt mattress (the slope protection) but disperse more than 10 m depth and varies in location.

2.3 Geology

The general geological sequence encountered at the site comprises drift geology, marine and fluvial deposits, overlying the solid geology of the Jurong Formation (Gobbett, 1973). This sequence can be observed base on the Soil Investigation Report done by Fugro Geoscience (M) Sdn. Bhd. **(Fugro)** on 2001.

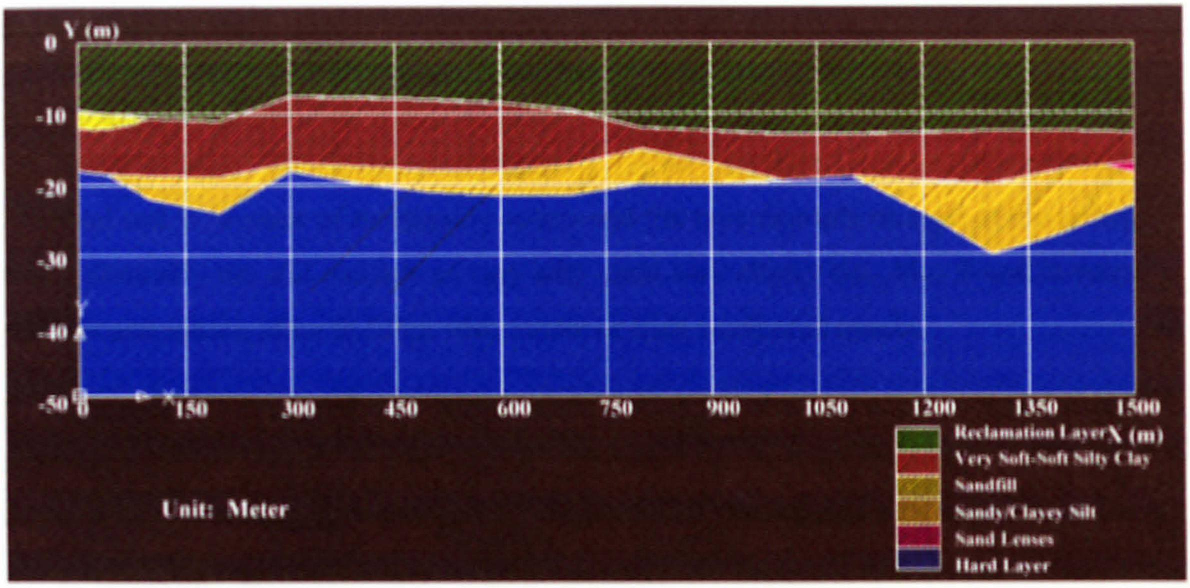


Figure 5: Summary Of Soil Cross Sectional Layer

It consists of very soft soil/ silty clay which is marine deposit, sandy/ clayey silt/ silty clay which is fluvial deposit, and firm to hard sandy silt/silty clay (Jurong Formation). Near Berth 6 it also has a sand fill for the base of construction for berth 6. Above the existing ground is where the reclamation layer takes place.

Marine deposits

The marine deposits are composed of very soft clay to silty clay. The unit found in elevation – 8 m to -20 m from the surface. The clay is normally consolidated clay and of high plasticity. It contains large silt content. Numerous thin sand lenses and pockets also occur at the layer. During reclamation, very soft clay (thickness average 2 m) is removed and dumped to the Long Bank, Malacca Straits. The remaining marine deposits are leaved at site and allow consolidating by using squeeze technique, which the surcharge is used to assist in consolidation of soils.

Fluvial Deposits

It comprise sot to stiff grey brown clays frequently sandy and with organic material occasionally very peaty. Numerous medium dense brown clayey sand lenses appear within this unit. It has varying topography and thickness from -15 m to -30 m elevation.

Jurong Formation

It is divided into 2 portions where the upper layer can be classified as residual soil and the lower is rock. The difference is because of the weathered state of the material. Residual soil is because of weathering action and the type depends on type of the parent rock. Normally the soil consists of very silty clay/ very clayey silt. Also found is veins and discontinuities, often in filled with calcite or high weathered quartz.

The bedrock encountered comprised fresh to moderate weathered siltstone and mudstone. Fine grained sandstone also presence at bedrock. Most of the rock encountered is completely fractured indicate the faulty of the bedrock.

2.4 Material Model

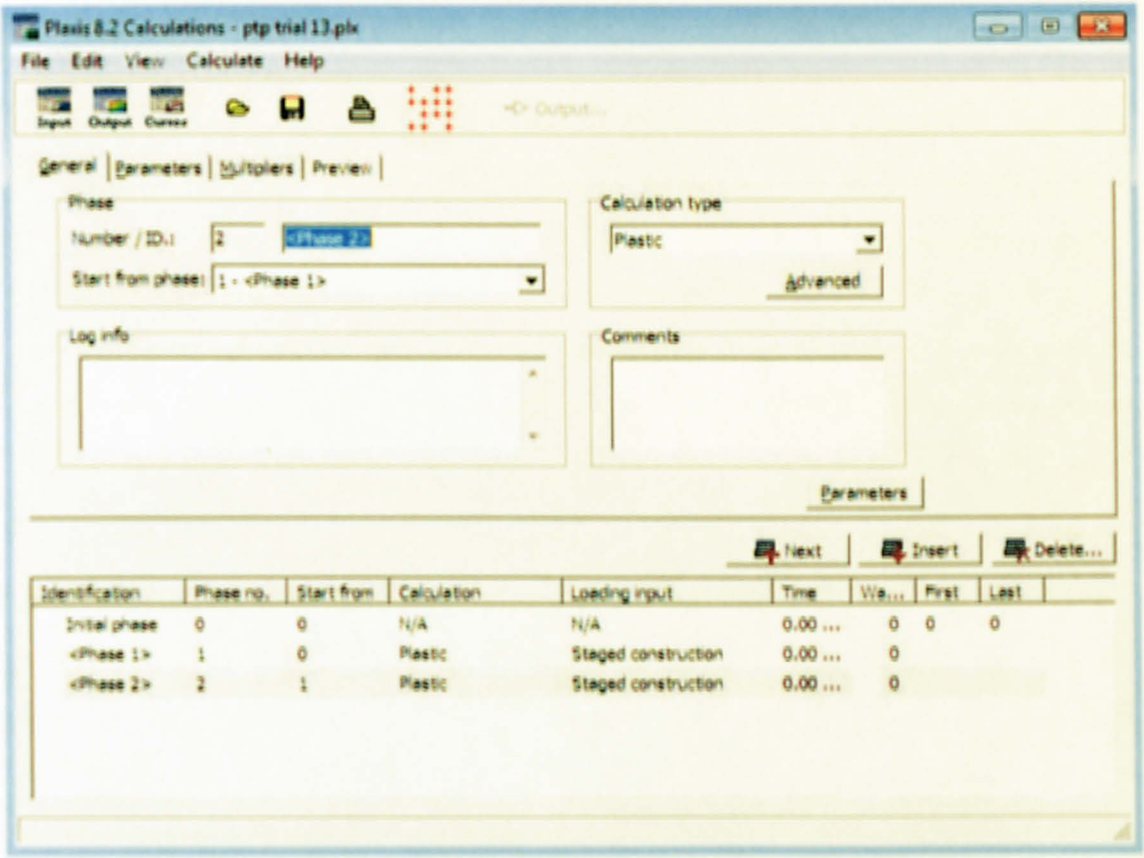


Figure 6: Plaxis Calculation Window

Mohr Coulumb selection model will be used for the analysis because it is least parameter dependant. It has soil elasticity, soil plasticity and angle of dilatancy for parameters that need to be considered. This model represents a first order approximation of soil behavior.

Staged construction allows the calculation to be done according to phase. Indicated at calculation window is phased specified. Before the previous phase is calculated, the current phase cannot be executed. This allow time dependant, and staged construction dependant problem to be studied briefly. Effect on each stage can be known towards the soil model.

CHAPTER 3: METHODOLOGY & PROJECT WORKS

3.1 Background Works

This field study will take mainly 2 semesters to cover. First semester will be mainly involves on data gathering, data computation and paperwork and literature of the study. The second semester will be mainly on the modeling.

3.2 Assumption

Certain assumption should be made for any data that not available, such as the reclamation is assumed to gained its strength required to carry out with the construction. The data presence is assumed to be accurate and the method of construction is appropriate.

3.3 Methodology

The data reduction involves comparing all the data available. It is gathered by putting all the related information into meaningful figures and computes a cross sectional layer using autocad. The data required to be studied in depth are soil investigation report, reclamation soil profile, specification and drawing that give information about in situ compacted soil, the report that give the loading which is used during design stage. Certain location of very affected point will be taken as the subject model for the ease of 2D modeling since it involves large area. Note that the location identified is 40 at berth 7 (identified as 7/40).

3.4 Material & Equipment

The material and equipment required for this project is shown below:

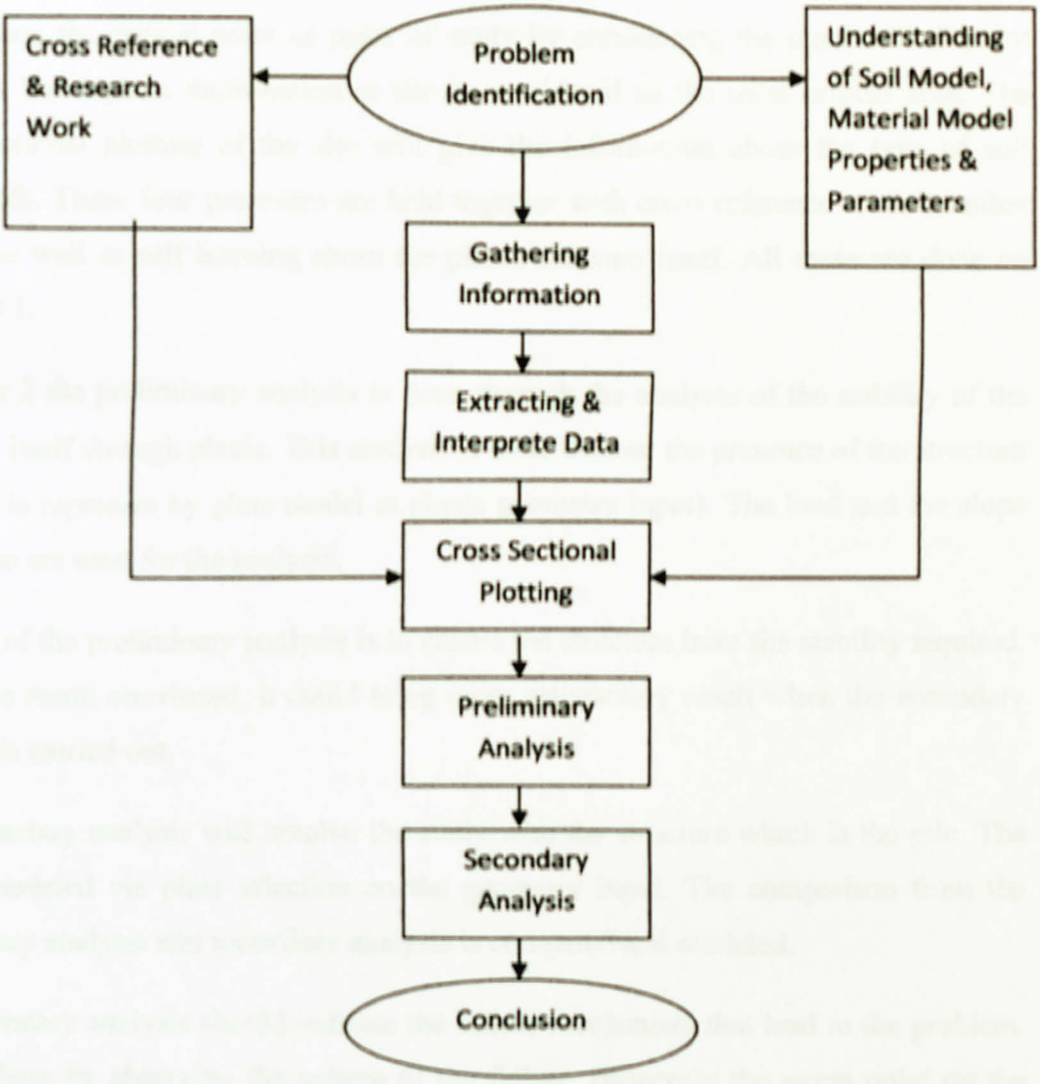
Material

1. Specification and Drawing for Berth
2. Soil Investigation report of the site
3. Reclamation Report of the site
4. Catalogue and Method Statement for installation works

Equipment

1. Plaxis v 8.0 Professional
2. Autodesk Autocad 2007
3. Microsoft Excel

3.5 Progress Flow Chart



The progress flow chart before explain the progress and the process required in this study. The problem identification process involves preliminary analysis on the site. The site visit is carried on at site and possible assumption is established. Then it is followed by gathering the required information available such as soil investigation report, technical specification of the structures, design criteria on the structures, environmental condition and load, progress report, and any person in charge views for the project. Later it is followed with extracting and interprets all the data available. Thus the process of selecting the critical point as point of study by considering the most critical point affected. The highest deformation at site is considered as the most critical area. The cross sectional plotting of the site will give the information about the type of soil underneath. These four processes are held together with cross reference work on other project as well as self learning about the plaxis software itself. All these are done on semester 1.

Semester 2 the preliminary analysis is done through the analysis of the stability of the structure itself through plaxis. This analysis is done without the presence of the structure (the pile is represent by plate model at plaxis geometry input). The load and the slope protection are used for the analysis.

The aim of the preliminary analysis is to ensure the structure have the stability required. When the result convinced, it could bring more satisfactory result when the secondary analysis is carried out.

The secondary analysis will involve the study with the structure which is the pile. The pile is modeled via plate selection on the geometry input. The comparison from the preliminary analysis and secondary analysis is compared and modeled.

The secondary analysis should indicate the failure mechanism that lead to the problem. This is done by observing the pattern of the failure, observing the stress point on the model and observes the failure on structure and the slope protection.

CHAPTER 4: RESULT AND DISCUSSION

4.1 Preliminary Analysis

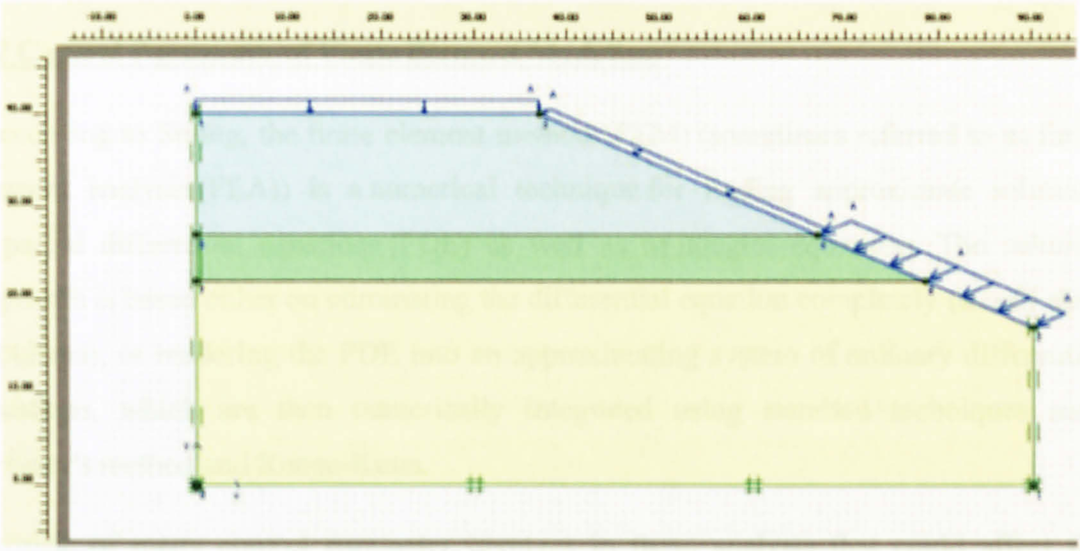


Figure 7: Geometry Input, Preliminary Analysis.

The input of preliminary analysis involves a range of 40 m depth and 90 m wide. It does not contain the structures, only the distributive load on the upper portion and incomatt mattress load on the slope.

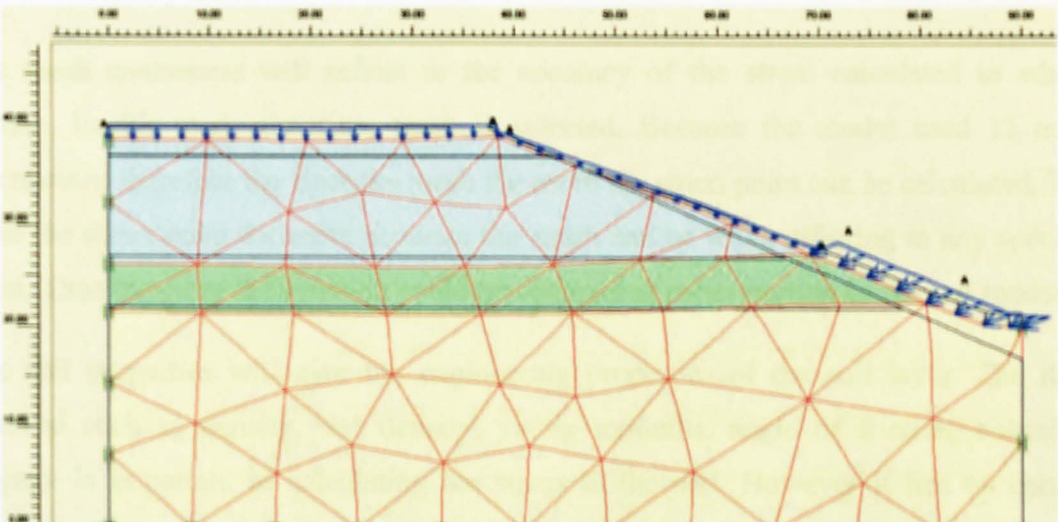


Figure 8: Deformed Mesh, Preliminary Analysis

From the deformed mesh figure before it indicates that the settlements do happen and the deformation will occur. But in comparison it is marginally acceptable. Therefore the secondary analysis can be carried out.

4.2 Control Parameter of Finite Element Modeling

According to Strang, the finite element method (FEM) (sometimes referred to as finite element analysis (FEA)) is a numerical technique for finding approximate solutions of partial differential equations (PDE) as well as of integral equations. The solution approach is based either on eliminating the differential equation completely (steady state problems), or rendering the PDE into an approximating system of ordinary differential equations, which are then numerically integrated using standard techniques such as Euler's method and Runge-Kutta.

Because of many control parameter involves in these analysis that could affect the result, therefore it is essential to reduce the dependencies of the data. The control parameter is

a) Mesh coarseness
b) Upper and lower soil properties
c) Boundary area.

The mesh coarseness will reflect to the accuracy of the stress calculated in whole system. In this study the finer mesh is selected. Because the model used 15 node calculations therefore the finer the mesh the more the stress point can be calculated. The more the stress point the more accurate the result can be when referring to any specific point. Thus the more deformation could be obtained at upper portion of the soil model.

The soil properties will give the engineering properties of the soil layer. The item required such as density, wet density, young modulus, angle of friction, cohesive, seepage is important in calculating the stress in the soil. However it lies on certain range. In these study 3 different results is produced by using upper soil properties, lower soil properties and medium. The upper soil property is using upper value of the soil properties and vice versa for the lower. It represent as control measures that later will be compared with the medium parameter. The medium parameter is taken as the subject to refer on this analysis.

Boundary is another factor that will give significant influence in the result. Therefore to reduce the influence that might come from lack of effective area, the boundary is extended 20 m deep and 40 m wide from its original boundary and become 60 m deep and 130 m wide.

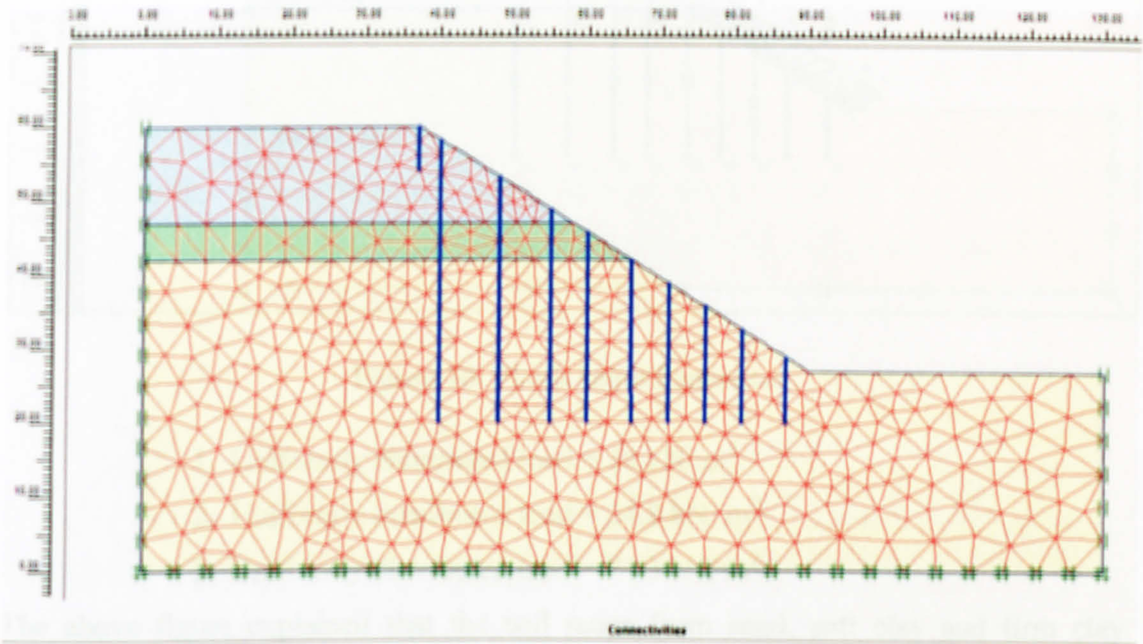


Figure 9: Mesh Coarseness And The Boundary

Method to determine the correct coarseness and boundary

The boundary is extended until it gives no significant influence when the program is executed. At depth 60 m and wide 130 m it is said that the influence of the boundary towards the result is minimal and neglected.

This is same in the mesh coarseness, the finer the mesh the accurate the result. As the mesh changes from very coarse to very fine mesh, the changes become minimal on each selection until the finest mesh. Thus the smallest difference is taken into account and the mesh that next to it become the selection to be used for this project.

4.3 Plaxis Input Parameter

Geometry Input

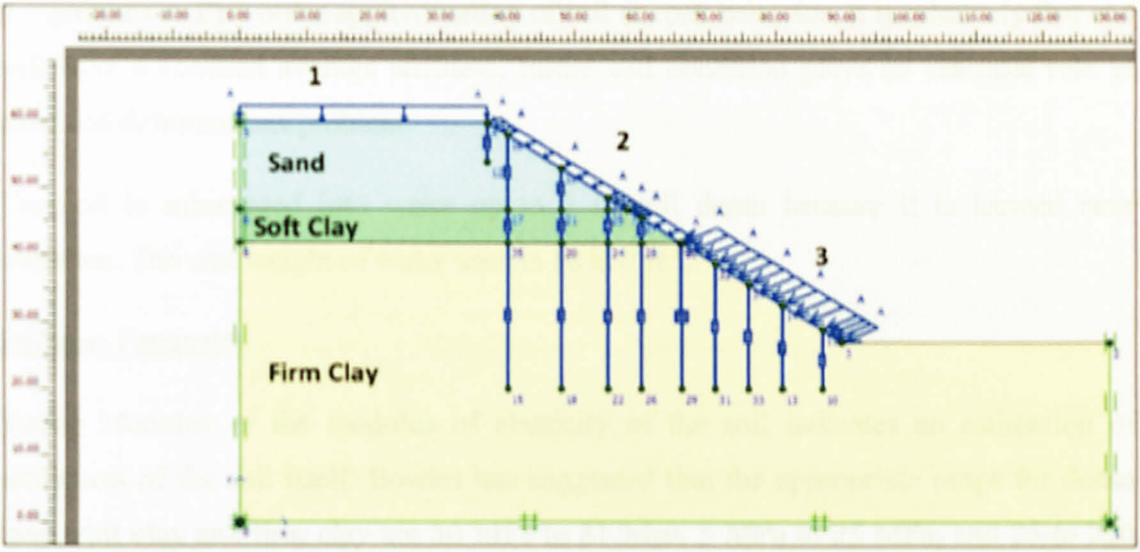


Figure 10: Plaxis Input Parameter

1. Uniformly distributed load = 30 kN/ m^2
2. Uniformly distributed load = 24.8 kN/ m^2
3. Uniformly distributed load = 74.3 kN/ m^2

The above figure explained that the soil range from sand, soft clay and firm clay. Preliminary analysis involves studying of equilibrium state of the main forces acting on the soil model. 1 is a UDL load from design axle load for PTP traffic operation. The traffic mainly comes from the prime mover and trailers. The function it is to transport the container from the ship into container storage area and vice versa. 2 & 3 is the incomatt mattress that used to protect the slope and counter masses the UDL from back of wharf (1).

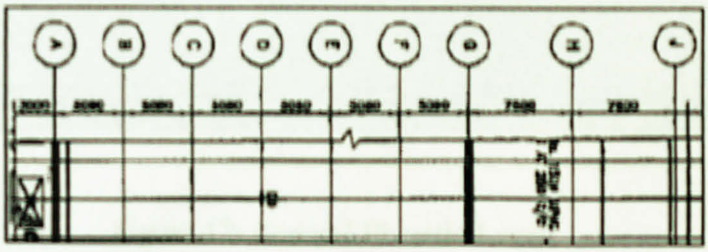


Figure 11: Pile position according to horizontal gridline.

Constitutive Model

The Mohr- Coulomb model involves soil elasticity, soil plasticity and angle of dilatancy. It represents as first-order approximation of soil or rock behavior. It is relatively fast and will have a constant average stiffness. Initial soil condition plays an essential role in most soil deformations problem.

The soil is submerged into water up to 3 m soil depth because it is located near shoreline. The unit weight of water used is 10 kN/ m^3.

Stiffness Parameter

Young Modulus or the modulus of elasticity of the soil indicates an estimation of settlement of the soil itself. Bowles has suggested that the appropriate range for dense sand, soft clay and firm clay are 50 MPa to 81 Mpa, 5 MPa to 25 MPa, and 25 to 250 MPa.

General Phreatic Level

The assumption of the phreatic level is 3 m. The depth is in accordance with the Mean Sea Level (the back of wharf is located at 3 m above the sea level).

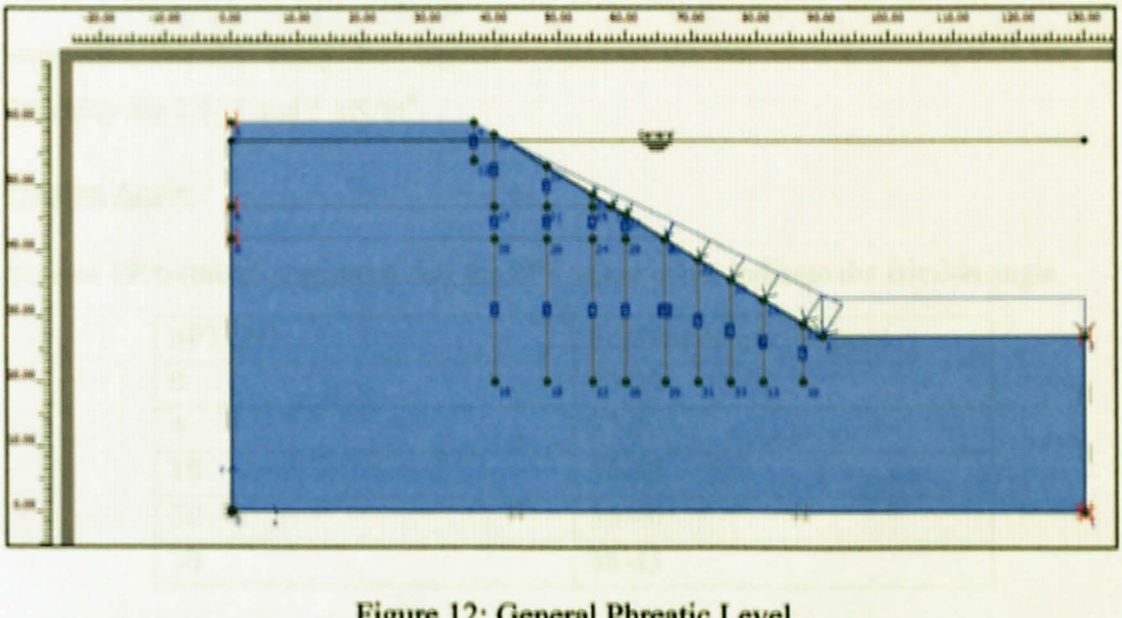


Figure 12: General Phreatic Level

Hydraulic Conductivity Of Soil

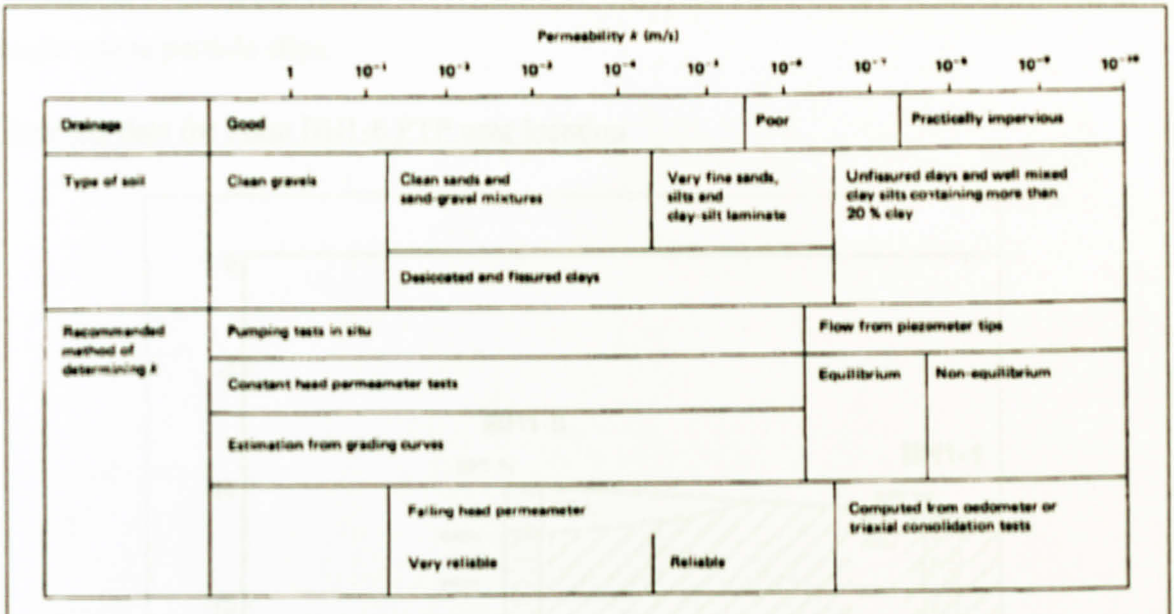


Figure 6 — Permeability and drainage characteristics of soils

Figure 13: Permeability And Drainage Characteristic, BS 8004

Cohesion

The cohesion is force that hold together molecules and particle within the soil. Cohesive soil is a clay type of soil. The higher the cohesion the higher the soil holds the clay properties. For this study the respective value of the cohesion for sand, soft clay and firm clay are 1.5, 3 and 5 kN/m^2.

Friction Angle

Bowles 1996 (table) suggested that the SPT value could indicate the friction angle.

SPT (N)	Friction Angle (degree)
0	25-30
4	27-32
10	30-35
30	35-40
50	38-43

Table 4: General Empirical Values For Friction Angle Base On The SPT

Angle of internal friction is an indication of the shear strength and effective shear strength at which shear failure occur on Mohr Circle. In other word it is an indication of angle where particle slips.

<u>Bore log data for Point BH1-6 PTP near location</u>

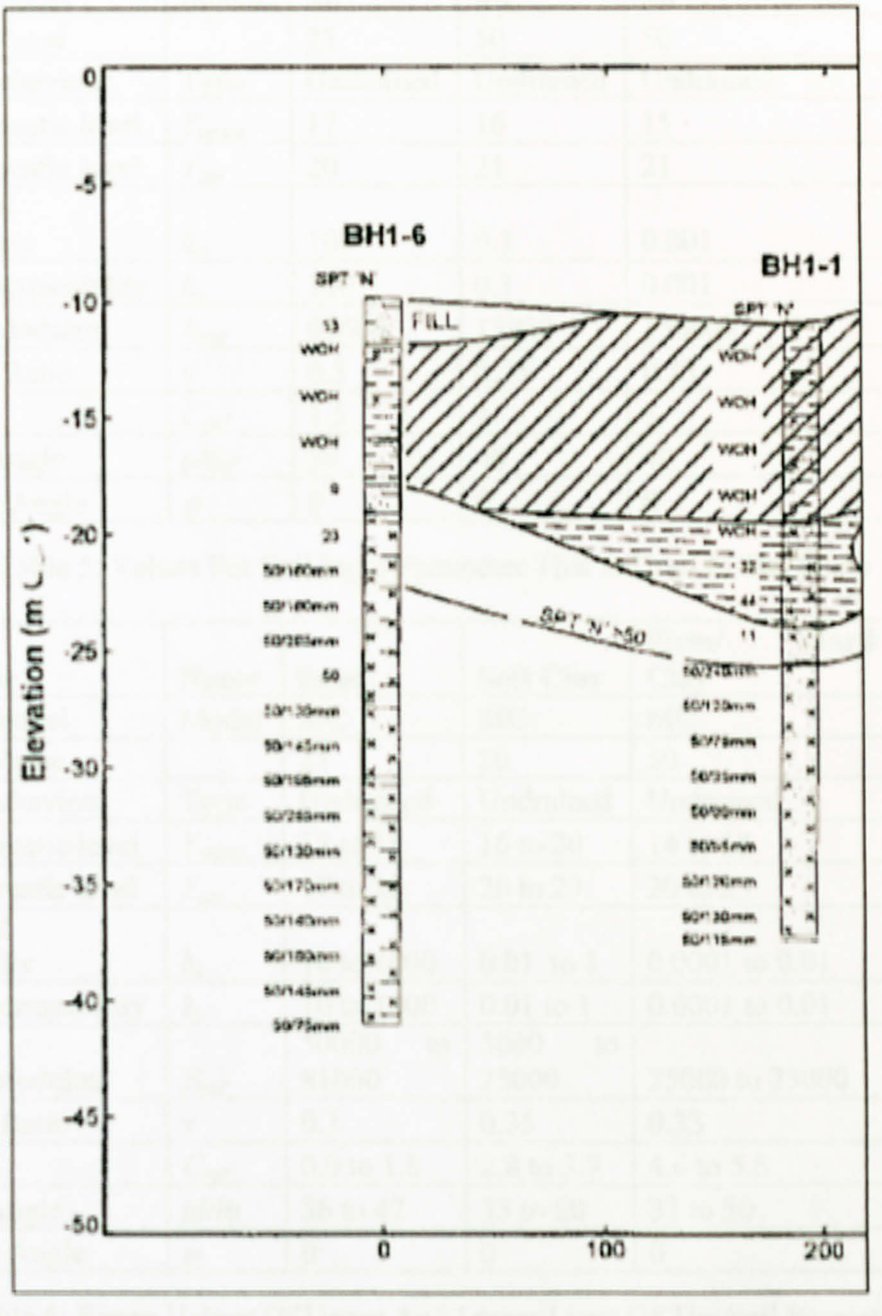


Figure 14: Bore Log Summary, SI Report PTP 1999

Soil Parameter Input

The tables show the parameter used and the input values for material soil model.

Parameter	Name	Sand	Soft Clay	Firm/ Hard Clay	Unit
Material model	Model	MC	MC	MC	
SPT 'N' Value		23	50	50	
Type of behaviour	Type	Undrained	Undrained	Undrained	
Above phreatic level	Y_{unsat}	17	16	15	KN/m^3
Below phreatic level	Y_{sat}	20	21	21	KN/m^3
Horizontal permeability	k_x	100	0.1	0.001	m/day
Vertical permeability	k_y	100	0.1	0.001	m/day
Young's Modulus	E_{ref}	69000	15000	150000	Kn/m^2
Poisson's Ratio	v	0.3	0.35	0.33	
Cohesion	C_{ref}	1.5	3	5	Kn/m^2
Friction Angle	$phi\varphi$	39	40	40	degree
Dilatancy Angle	ψ	0	0	0	degree

Table 5: Values For Soil Input Parameter That Is Used In The Study

Parameter	Name	Sand	Soft Clay	Firm/ Hard Clay	Unit
Material model	Model	MC	MC	MC	
SPT 'N' Value		23	50	50	
Type of behaviour	Type	Undrained	Undrained	Undrained	
Above phreatic level	Y_{unsat}	15 to 18	16 to 20	14 to 18	KN/m^3
Below phreatic level	Y_{sat}	17 to 22	20 to 23	20 to 23	KN/m^3
Horizontal permeability	k_x	10 to 1000	0.01 to 1	0.0001 to 0.01	m/day
Vertical permeability	k_y	10 to 1000	0.01 to 1	0.0001 to 0.01	m/day
Young's Modulus	E_{ref}	50000 to 81000	5000 to 25000	25000 to 25000	Kn/m^2
Poisson's Ratio	v	0.3	0.35	0.33	
Cohesion	C_{ref}	0.9 to 1.8	2.8 to 3.7	4.6 to 5.5	Kn/m^2
Friction Angle	$phi\varphi$	36 to 42	33 to 50	33 to 50	degree
Dilatancy Angle	ψ	0	0	0	degree

Table 6: Range Values Of Upper And Lower Limit Of The Soil Properties

Material Parameter Input

EI & EA of concrete piles

A= normal area of pile =0.31447 m^2

$E_{concrete}$= 4700 (fc)^0.5-= 4700(80)^0.5= 42.0381 x 10^6 kN/m^2

Note that F_c = concrete 28 days compressive strength, MPa

I= polar moment of inertia of pile = phi() d^4/32 =((22/7)/32) (0.9^4-$(0.9-0.13x2)^4$)

= 0.04794 m^4

EI =2 015 306.5 kNm^2/m

EA = 1 3219 721.31 kN/m

w = pile unit weight= 6374.6 kN/m/m

Parameter	Unit	Value
EA	kN/m	2 015 306.5
EI	kNm^2/m	1 3219 721.31
d =EA/EI	m	automatic
w	kN/m/m	6374.6
v	-	Assume 0.00

Table 7: Material Properties Parameter For The Pile

4.4 Plaxis Calculation

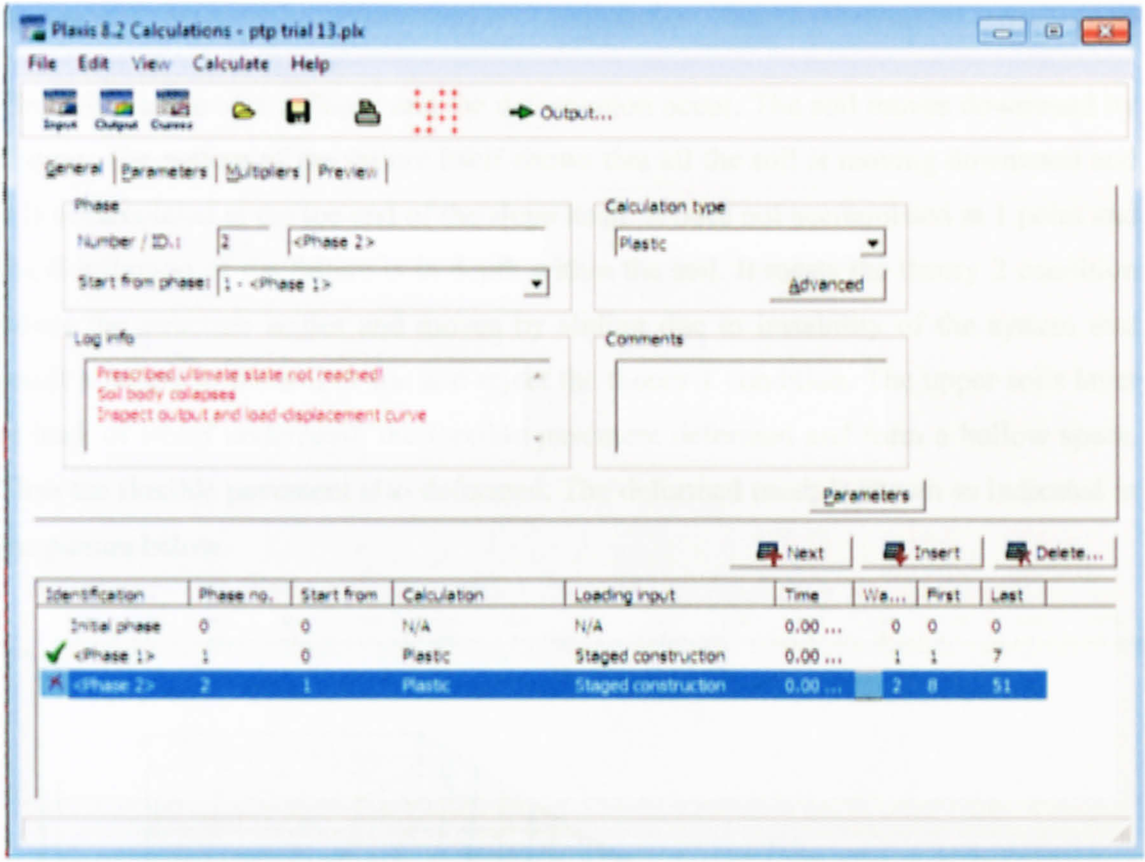


Figure 15: Plaxis Calculation Window

The picture indicated calculation window. This is used to calculate the total reaction of the model. The calculation involves 3 phases. Initial phase is when the initial condition is set up. Phase 1 involve influence of Phreatic level to the soil model. Phase 2 involve influence of forces (UDL and Incomatt mattress) towards the soil model.

The results shown prescribed ultimate state not reached. Soil body collapses. This means the static analysis of the model is not equilibrium by using mohr coulomb. The calculation will involve calculation at each nodal point on the mesh and the sum of the stress is defined as the mstage. The soil collapse when it reaches mstage more than 1. Later the graph of mstage vs deformation at each point could be indicated. Here in this analysis the pattern of the collapse is by sliding.

4.5 Plaxis Result

Result

The soil mass model collapse and the deformation occur. The soil moves downward by sliding. The pattern of the failure itself shows that all the soil is moving downward and it is accumulated at the toe end of the slope itself. It does not accumulated at 1 point and the distribution of the failure is in depth within the soil. It meets the theory 2 condition where the structure settles and moves by sliding due to instability of the system that result a failure to the soil at site and reject the theory 1 condition. The upper soils layer at back of wharf underneath the flexible pavement deformed and form a hollow space. Thus the flexible pavement also deformed. The deformed mesh is shown as indicated in the picture below.

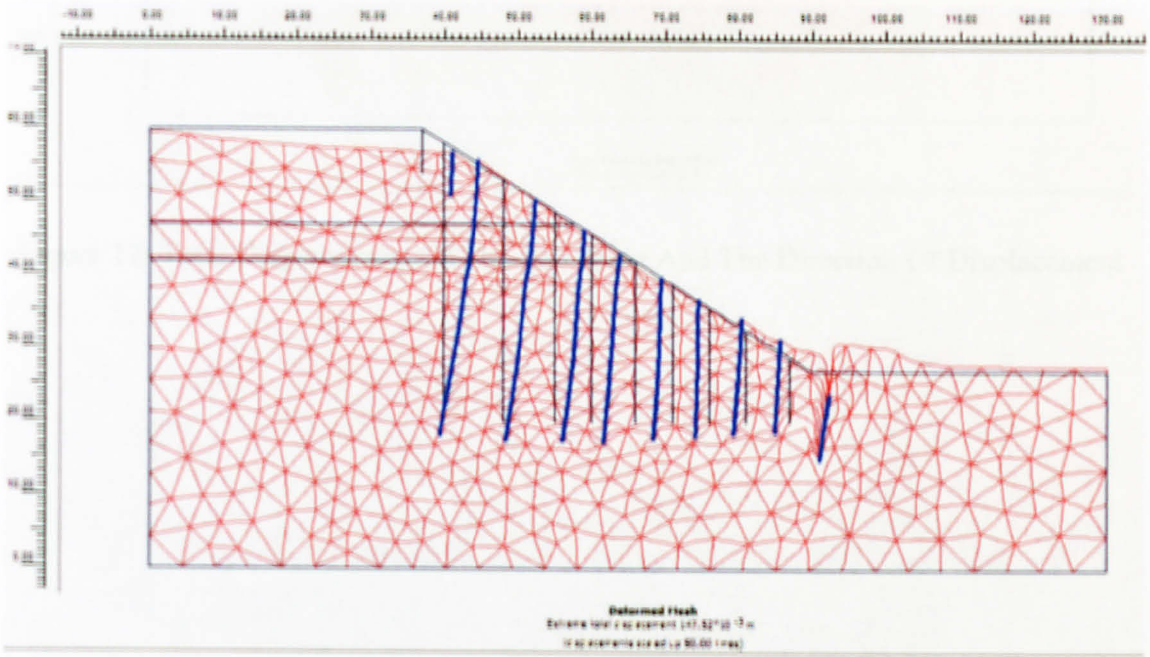


Figure 16: Deformed Mesh Of The Soil At Site

The deformed soil mass pushed the pile downward and cause failure to the pile. The incomatt mattress failed to retain the soil mass into its position. From the deformed mesh it can be said that the soil mass moves from the middle of the back of wharf towards the sea is the severe area that undergo soil deformation.

The next picture indicates the critical area at the soil model. The higher the red color the critical the displacement occur. The picture also indicates the critical area for soil movement. Note that near the slope the color is yellow to red, indicates higher soil deformation at this area.

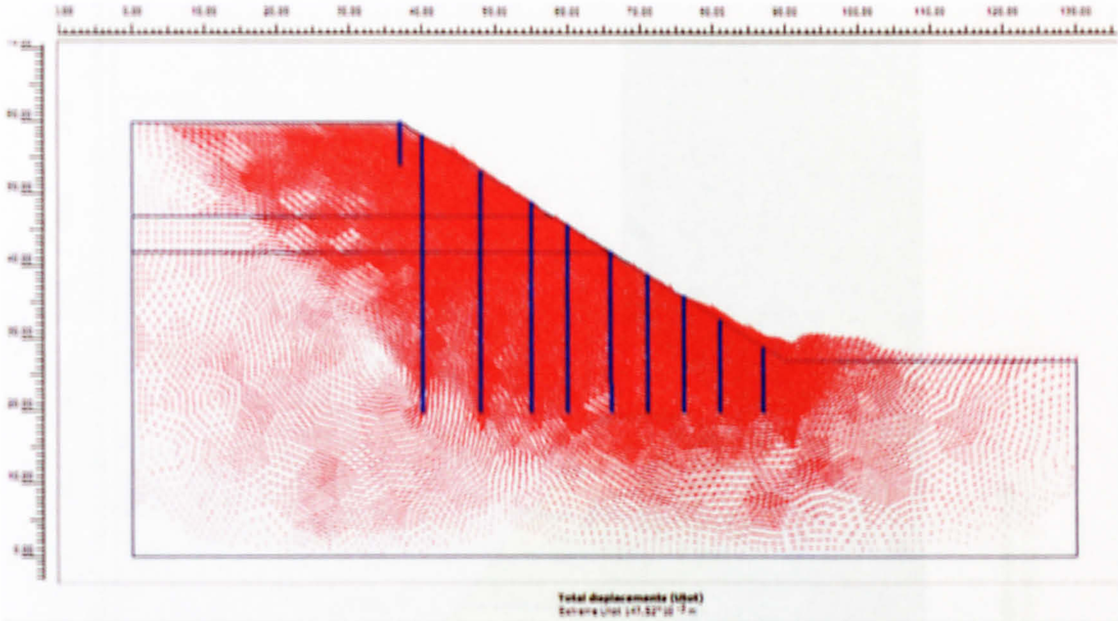


Figure 17: Total Displacement Of The Soil Mass And The Direction Of Displacement

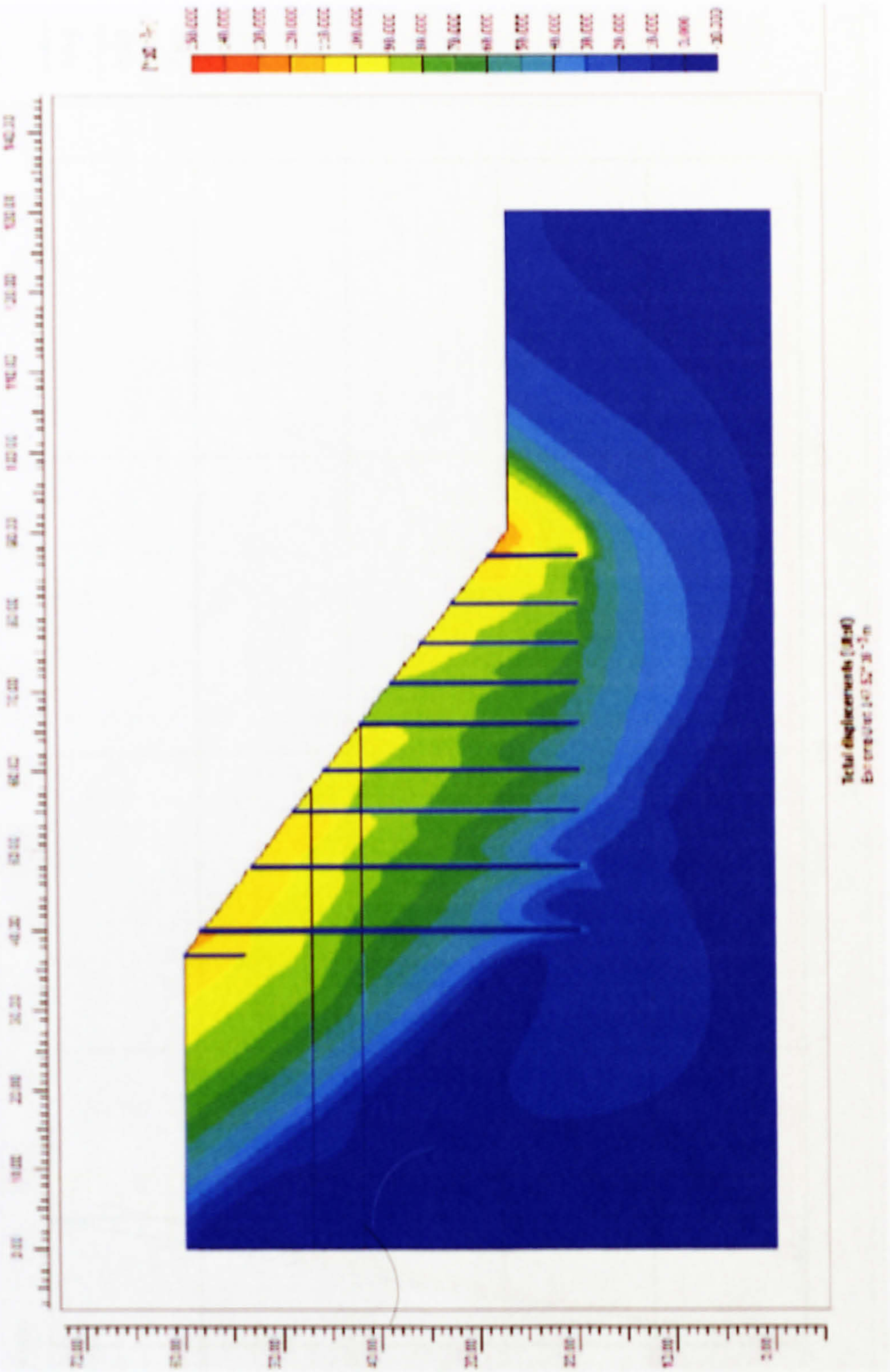


Figure 18: Critical Point On The Model

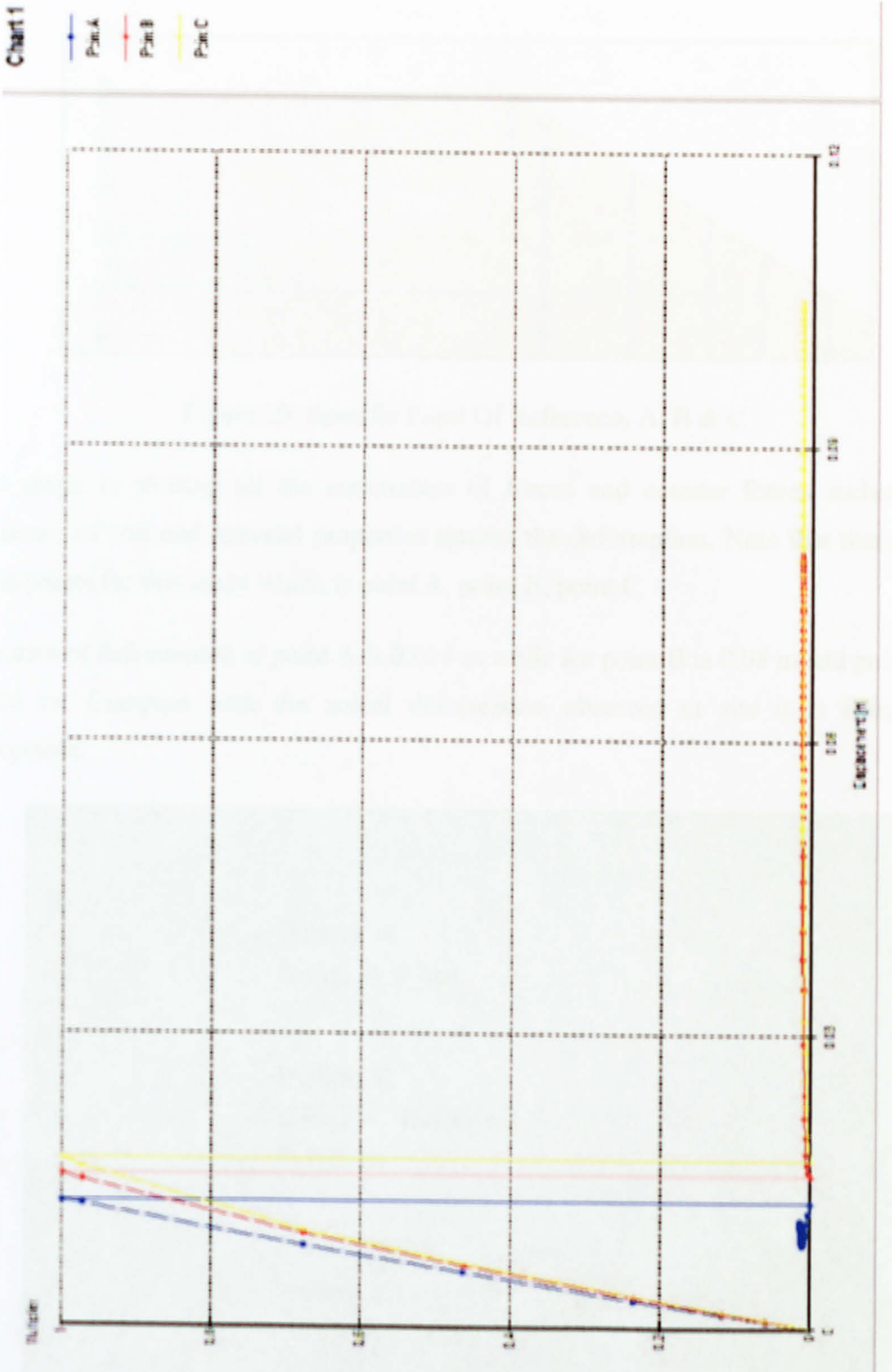


Figure 19: The Deformation Of The Soil Model

The graph show the deformation depth in meter at specific location

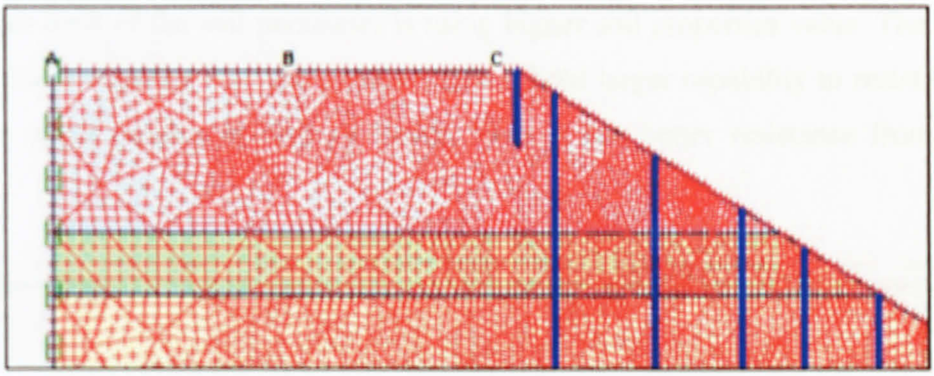


Figure 20: Specific Point Of Reference, A, B & C

The graph is plotting all the summation of forces and counter forces inclusive of influence of soil and material properties against the deformation. Note that there are 3 main points for this study which is point A, point B, point C.

The sum of deformation at point A is 0.014 m while for point B is 0.08 m and point C is 0.105 m. Compare with the actual deformation observed at site it is marginally acceptable.

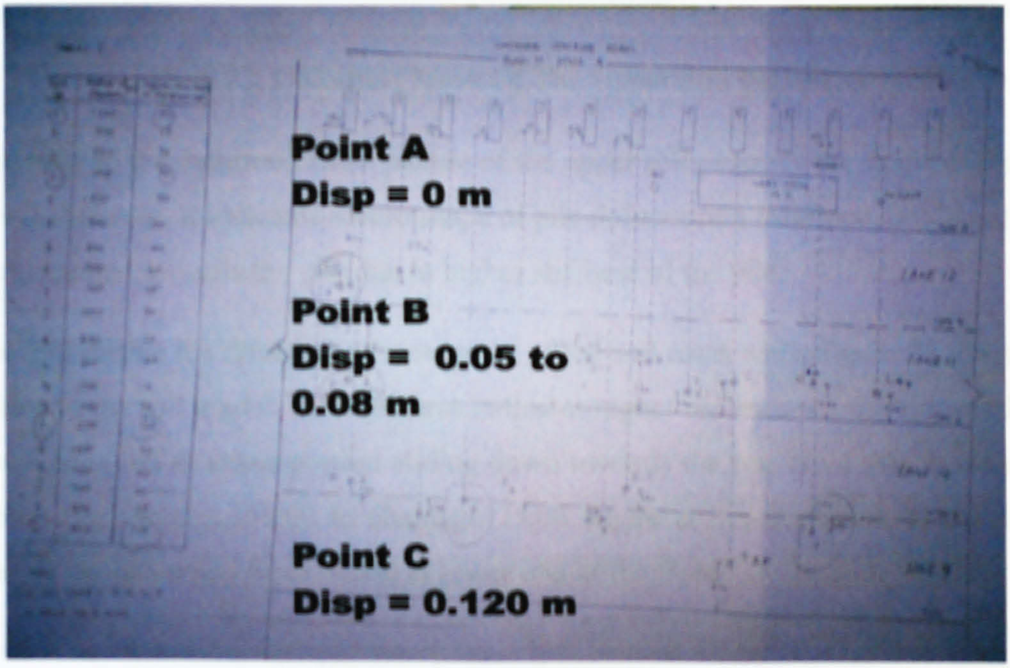


Figure 21: Actual Soil Deformation Observed At Site

Upper Soil Property Parameter

This upper limit of the soil parameter is using higher soil properties value. The soil has more stiffness capacity than the previous thus it hold larger capability to resist the load from the upper portion at back of wharf. Thus it has better resistance from sliding failure.

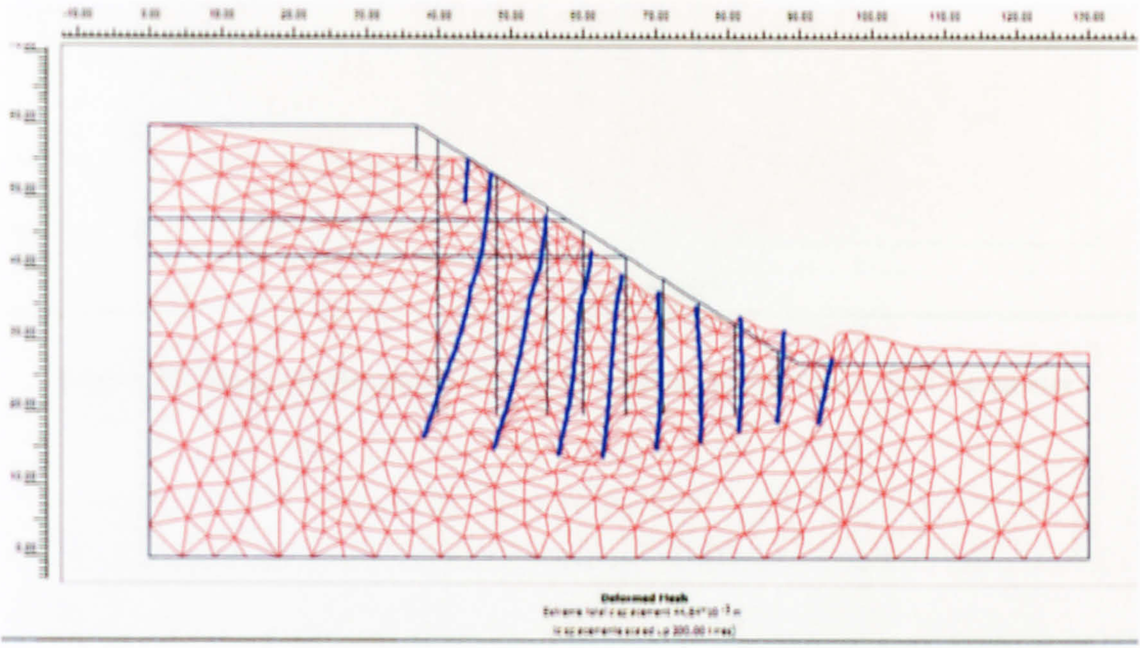


Figure 22: Deformed Mesh Of The Upper Soil Parameter

By referring to the deformed mesh picture of the upper soil property the shape of the failure is different. It indicates severe shape of pile deformation informed that larger stress is applied towards the pile due to higher stiffness of the pile.

Figure 23 indicate the direction of movement of the soil mass while figure 24 illustrates the stress at the soil model. Both pictures indicates higher movement from the middle of the back of wharf at upper portion sliding down towards the sea. Note that the stress is higher at the chainage 40.00 to chainage 75.00 where at upper portion of the slope. Higher stress also could be expected at lower end of the slope.

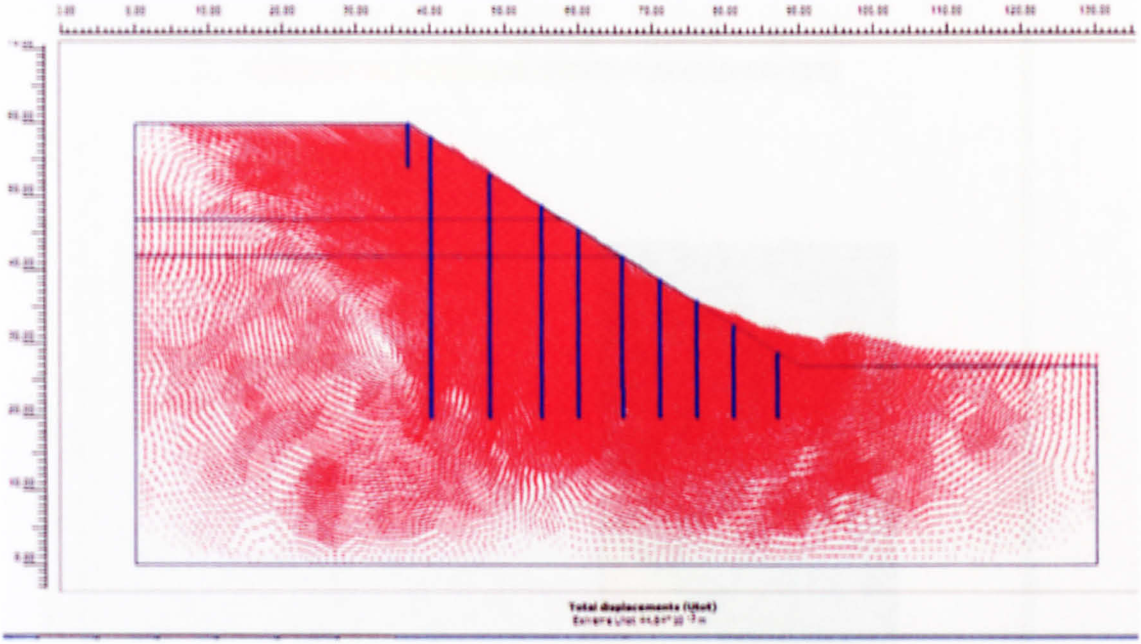


Figure 23: Total Displacement Of The Soil Mass Of The Upper Soil Parameter

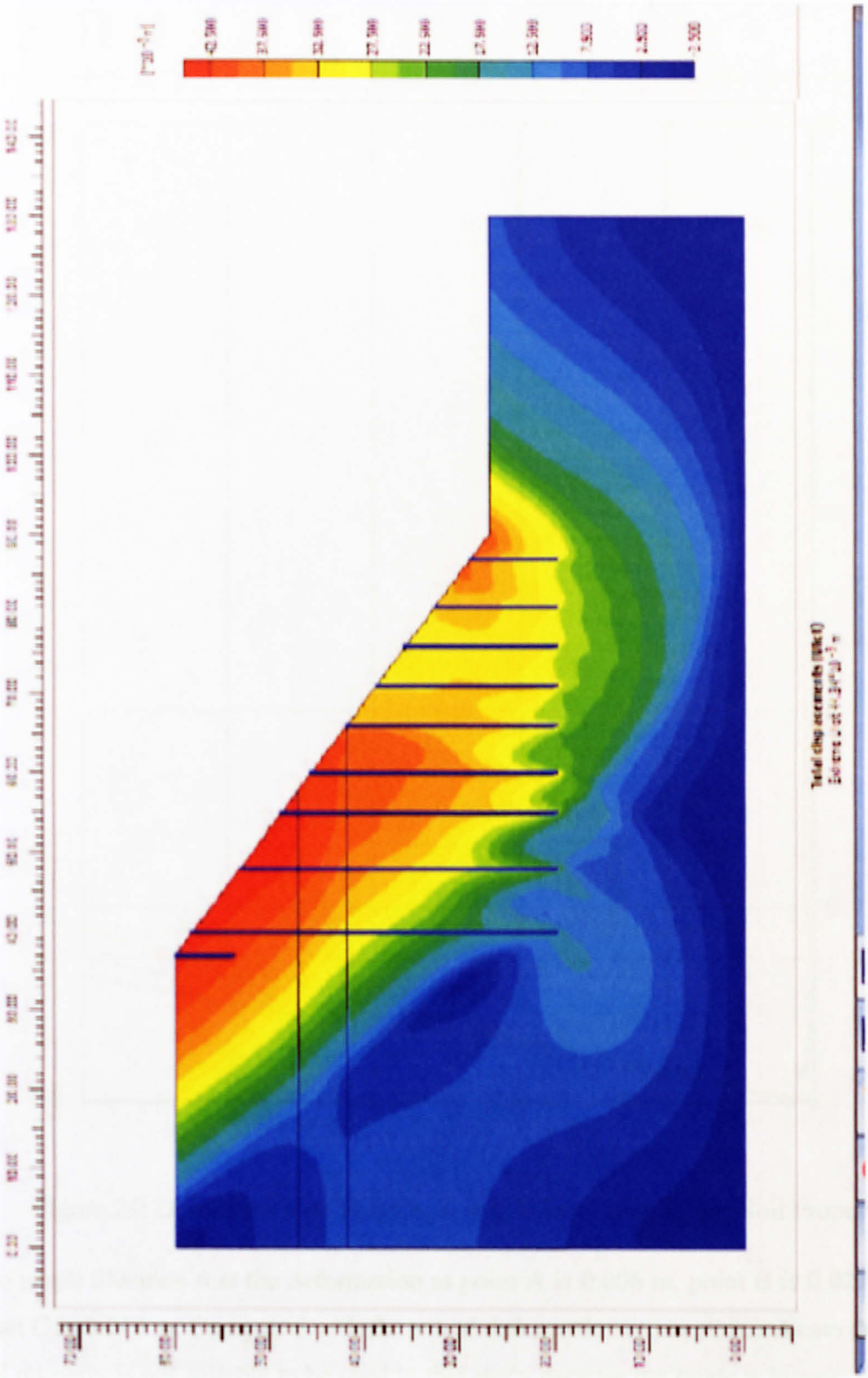


Figure 24: The Critical Point Of The Upper Soil Parameter

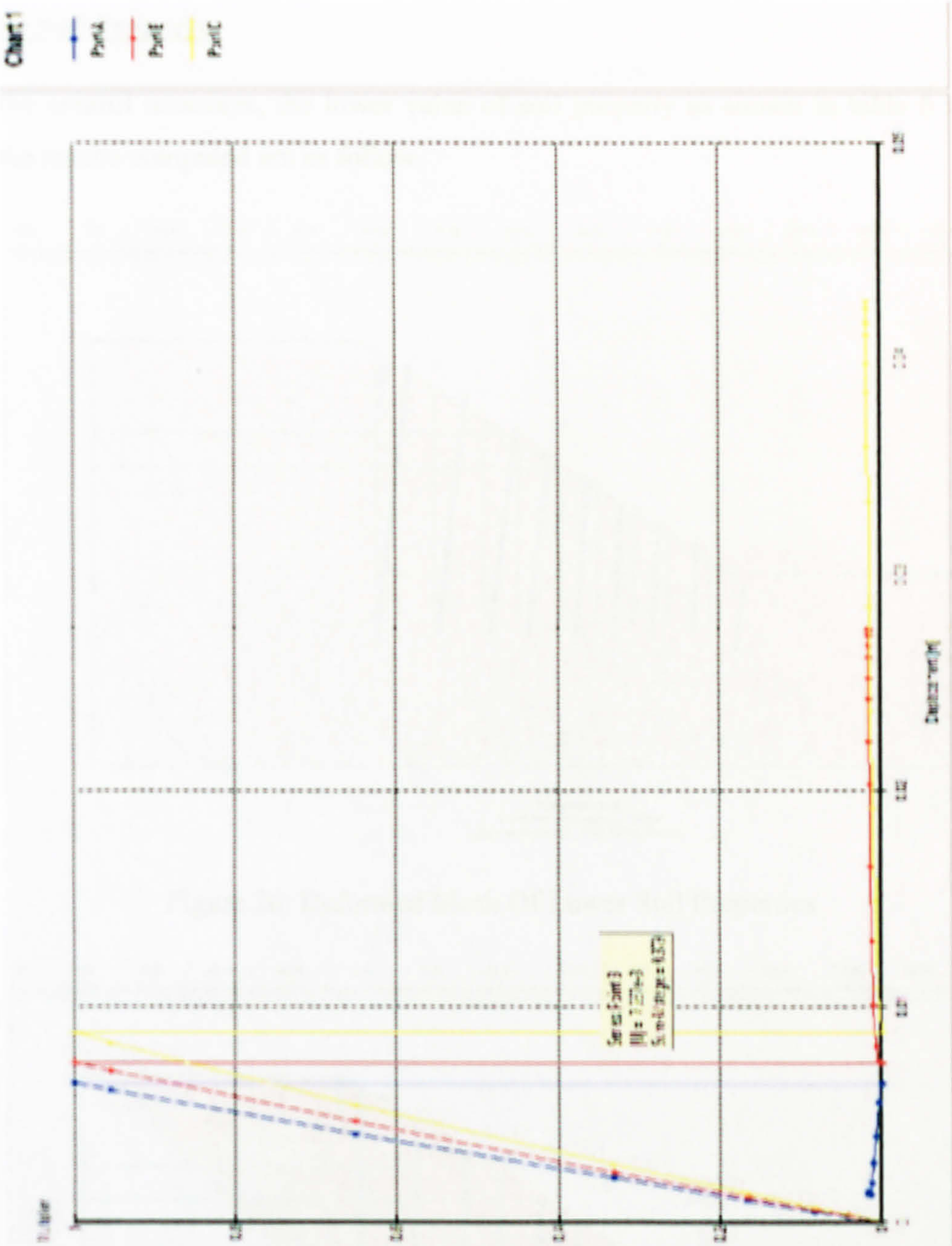


Figure 25: Deformation At Point A, B & C When Using Upper Soil Property

The graph illustrate that the deformation at point A is 0.006 m, point B is 0.027 m, and point C is 0.043 m. Compared with the actual deformation at site, this indicate the upper soil property is not suitable to be used in this study because the range is bigger.

Lower Soil Property

For the control measures, the lower value of soil property as shown in table 6 is used and the results computed are as follow.

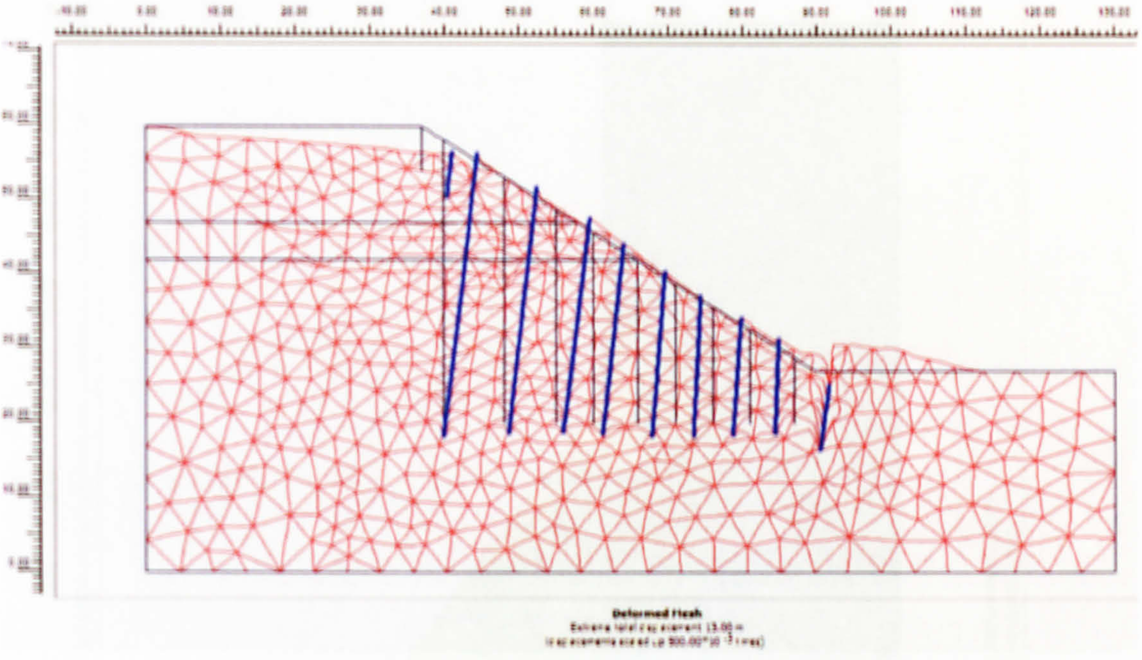


Figure 26: Deformed Mesh Of Lower Soil Properties

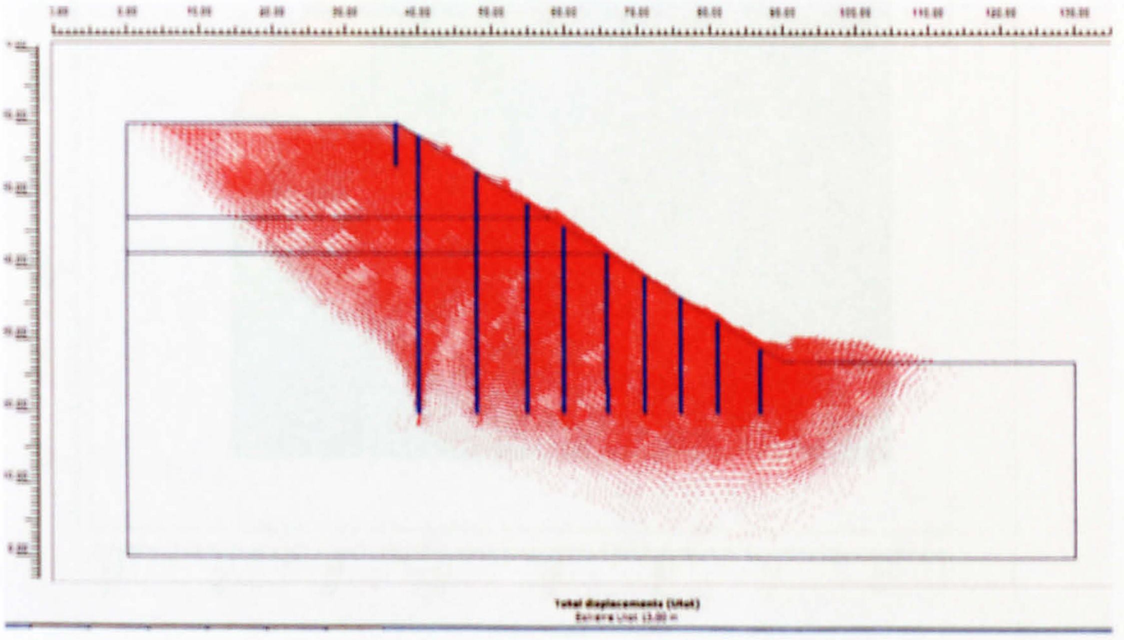


Figure 27: Total Displacement Of Lower Soil Property

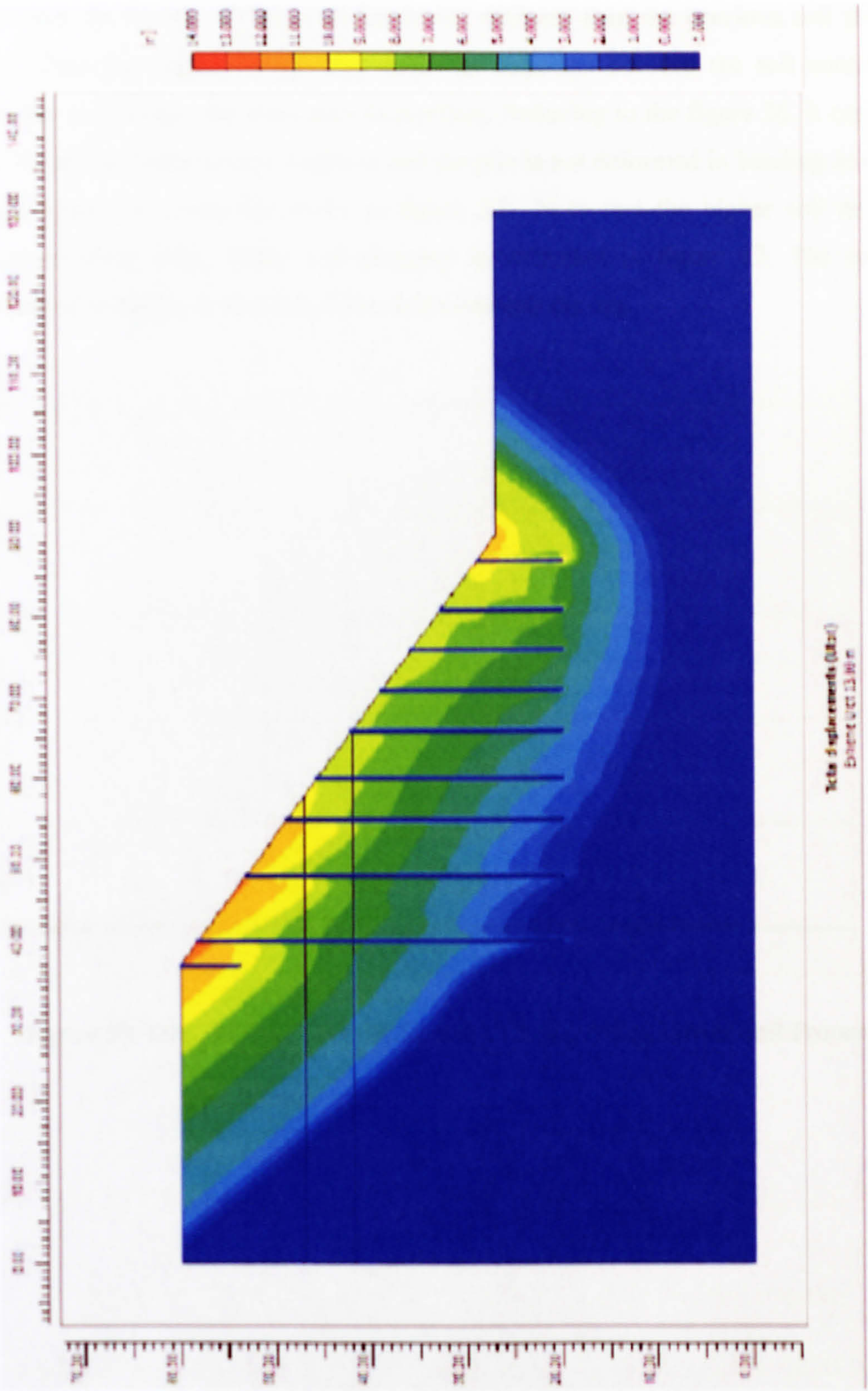


Figure 28: Critical Point Of Lower Soil Properties

Note that the lower soil property has lower stiffness than the previous soil parameter used. Thus the higher deformation could be expected because the soil contains less capacity to hold the soil mass into its position. Referring to the figure 26, it can be said that the deformation is more uniform and the pile is not deformed in bending as much as using upper soil properties (refer to figure 22). Note that the higher soil movement occurred when using lower soil property as indicated in figure 27. The soil mass movement is higher at the start of the slope towards the sea.

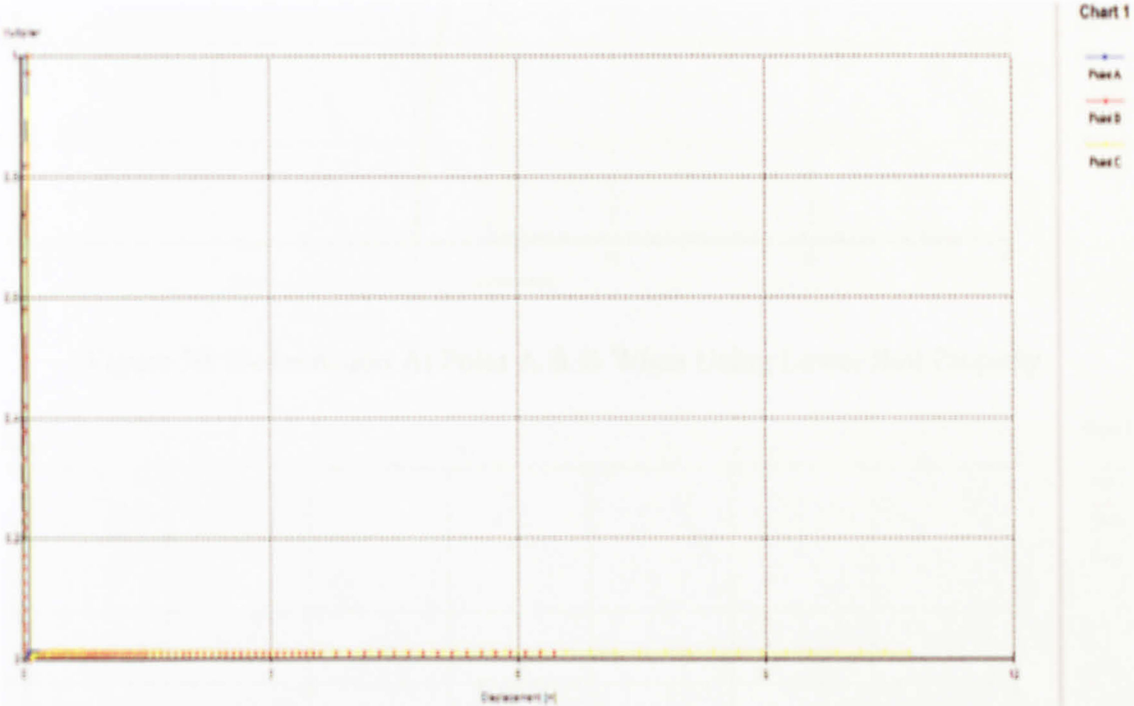


Figure 29: Deformation At Point A, B & C When Using Lower Soil Properties

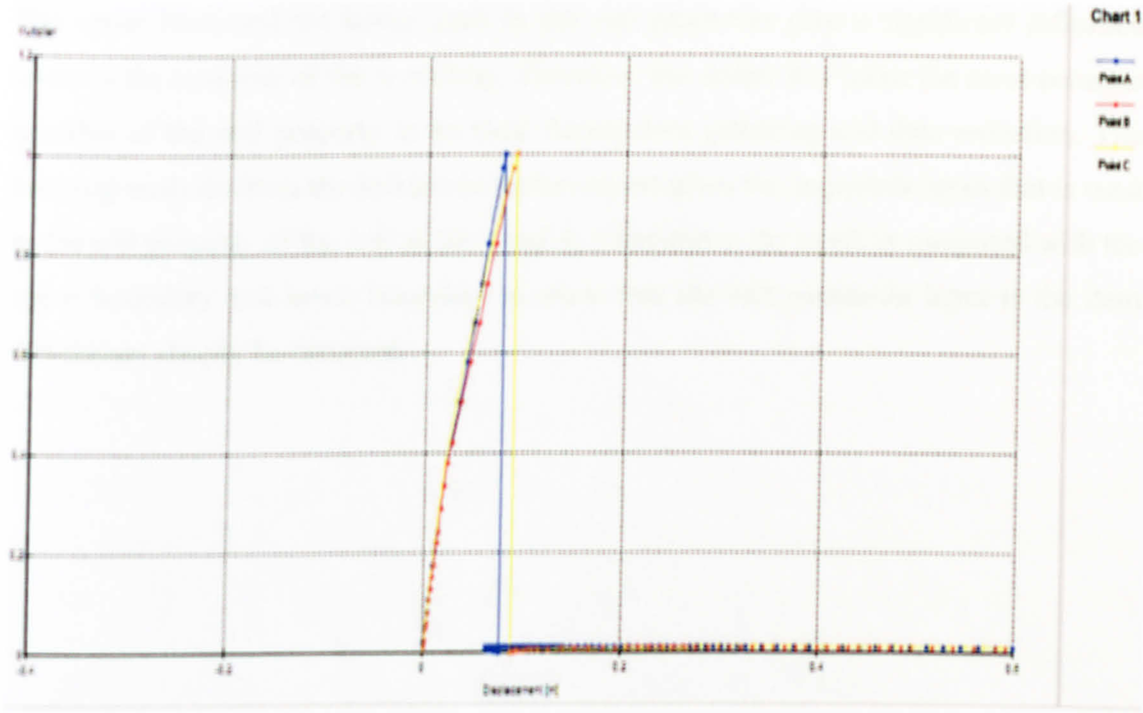


Figure 30: Deformation At Point A & B When Using Lower Soil Property

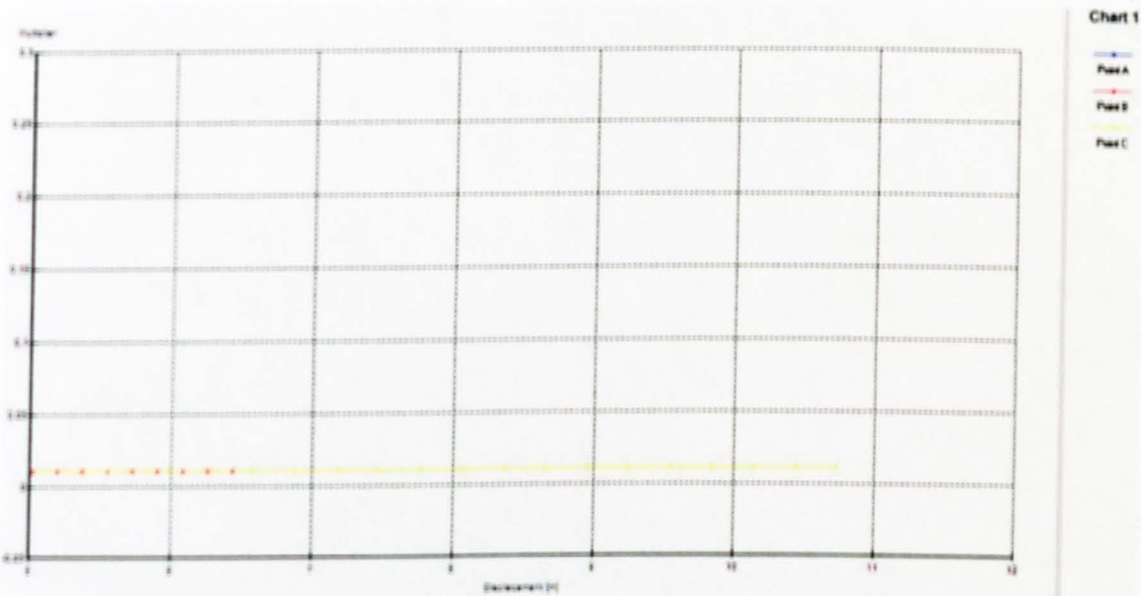


Figure 31: Deformation At Point C When Using Lower Soil Property

From the graph it could be said that the deformation at point A is 1.5 m, point B is 6.5 m and point C is 10.75 m. therefore it is not suitable to be used in this study.

The upper limit and the lower limit in the soil properties give a significant influence towards the outcome of the modeling. Therefore the author has taken the most accurate possible of the soil property to be used during data gathering and data reduction. The bore log analysis from the soil investigation report gives the important input that is used as the soil property of the soil at site. And as comparison the result is compared with the upper boundary and lower boundary to show that the soil parameter input is the item that cannot simply be assumed.

CHAPTER 5: CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The objective of this study is to identify the failure mechanism that lead to the soil deformation at site. By studying the critical point established at the plaxis the appropriate solution could be suggested for the next investigation. The last objective for this project to study the elements that contributes to inaccuracy of the study. Thus the control parameter can be established to ensure accuracy of the result.

From the study that carried out it can be said that it meets the objective. The outcome of the modeling meet the theory two condition where the structure settles and moves by sliding due to the instability of the system. This instability condition related much by the load imposed at the back of wharf and the load used to retain the soil mass at the slope.

The critical point at the soil model also can be observed and compared and the author successfully establish the control parameter to ensure that the sound result is produced.

The soil at site comprise of marine deposits, fluvial deposit and Jurong Formation comprise of residual soil and hard layer. Each of it has several ranges of soil properties to be controlled.

There are two assumptions made, the soil mass under the slope protection and retaining structure loss due to erosion and loss of soil mass near slope. Another theory established is the structure settles and moves by sliding due to instability of the system that result a failure to the soil at site.

Findings and conclusion of the findings

In order to obtain accurate result, several control measures is used. These inclusive:

a) Extend the boundary limit from 40 m depth and 90 m wide to 60 m depth and 130 m wide
b) Use finer mesh to ensure more point calculated so that the stress is more precise and accurate.

c) Use upper and lower soil property as comparison with the subject studies that later will be compared to support the usage of the middle soil property for the study.

Each of the control measures gives significant impact in the accuracy of the project. The extended boundary gives lower influence of water pressure and the stiffness at certain point is reduced. This minimizes the disturbance of the data itself. The finer the mesh the more the calculation could be done in the geometry boundary for the soil model. The upper soil property parameter if to use will have greater stiffness. Thus it reflects with lower deformation of soil achieved. The lower soil property if to be used will gives higher soil deformation as a result of lower stiffness. The information obtained from the soil investigation report helps a lot in determining the suitable soil property to be used.

From the result of the experiment using plaxis, the simulation indicates that the soil failed by sliding and proves that the failure is not from the erosion of the soil mass. The deformation observed is at point A is 0.014 m while for point B is 0.08 m and point C is 0.105 m. Compare with the actual deformation observed at site it is marginally acceptable.

5.2 Recommendation

The finite element modeling such as plaxis depend on the input criteria the user define into it. Thus it give the influence on the accuracy should the user not correctly define the input. Further analysis can be done to establish additional control parameter than what is introduced by the author. The things that could be done is more soil boundary and modeling with changes of soil parameter until the soil parameter does not contribute to the significant changes in the result. The higher the soil boundary the lesser the effect influence the result. However when the soil boundary reach a certain limit as suggested by the author (note that the soil boundary is 60 m depth and 130 m wide) it give no further influence.

Else more in depth study could be done to carry on the actual soil investigation at site with the current structures on it. If there is any change in the soil layer it could be obtained.

This modeling could be combined with other type of analysis such as resistivity study to confirm on the findings of the plaxis itself. It could model if there is any hollow or presence of soil failure at site since resistivity study reflects with the conductivity of the soil itself.

As an additional other type of soil model could be use to further simulate the study. The mohr coulomb is a very simple model and it does not reflect with time. Further analysis by using soft- soil creep model could be used where the influence of time dependencies (consolidation over time) could be simulated and investigated.

REFERENCES

1. 2001, *Option Study for Dredging & Reclamation,* Engineering & Environmental Consultant Sdn. Bhd.

2. 1999, *Report On Design Criteria For Wharf Structures,* Engineering & Environmental Consultant Sdn. Bhd.

3. 1999, *Report On Option Study For Wharf,* Engineering & Environmental Consultant Sdn. Bhd.

4. 1996, *Material Model Manual Plaxis* v 8.0, Netherlands, AA Balkema Publishers.

5. 1996, *Tutorial Model Manual Plaxis* v 8.0, Netherlands, AA Balkema Publishers.

6. 2008, *Report On Pavement Defects At Wharf Side Road At Berth 7 & 8*, Sepakat Setia Perunding Sdn. Bhd.

7. 2001, *Phase 2 Soil Investigation Report*, Fugro Geoscience Malaysia Sdn. Bhd.

8. Gobbett, D.J. and Hutchison, CS 1973, "*Geology of the Malay Peninsula*" (West Malaysia and Singapore).

9. Jonathan, PS, 2007, *Full Scale Cyclic Testing On Foundation Support Systems For Highway Bridges Part II: Abutement Brickwalls.*

10. 2003, *Construction Drawing of Berth 7 & 8,* Engineering & Enxvironmental Consultant Sdn. Bhd.

11. 2008, *PTP Master Plan*, Pelabuhan Tanjung Pelepas Sdn. Bhd.

12. Strang, G. and Fix. G, 1973. *An Analysis of The Finite Element Method.* Prentice Hall