BACK CALCULATION FROM GEOTECHNICAL STRUCTURES RESPONSE AROUND KUALA LUMPUR

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the requirements for the
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CERTIFICATION OF APPROVAL

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A project dissertation submitted to the
Civil Engineering Programme
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TRONOH, PERAK

January 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.

KAREEN LEE EE KENG

ABSTRACT

It is well recognized that uncertainties are abundant in geotechnical engineering which may result in failure of geotechnical structures. Achieving an idealized parameters' literature has become a form of approach analysis but the accurate determination of soil parameters is rather difficult. The main focus of this study is to evaluate aspects of computational geotechnics through inverse analysis to deduce design parameters through correlation with field monitoring data of various high rise residential developments around Kuala Lumpur, mainly focusing on soil-structure interaction during Simulation of Basement Excavation and Construction.

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CHAPTER 1

INTRODUCTION

1.1 Background Study

Geotechnical structures are substructures or that are in contact with rock and soil. These substructures are designed and constructed for any superstructure of varying sizes, to either transmit large superstructure loads to the soil or to retain soil mass (Bowles, 1982). The primary concerns while designing and evaluating the performances of substructures are bearing capacity, settlement and ground movements surrounding the substructures.

Design codes available such as British Standard and Eurocode specify basic geotechnical design parameters which only serve as a guide for simple conditions but not necessary applicable to all types of site. In many cases, considerations for geotechnical design are also based on past experiences as soil properties vary with location and time.

In urban areas like Kuala Lumpur, many high rise projects are developed to make full advantages of limited and expensive price of land per square feet. These high rise projects require construction of basement for additional space. In view of the soil condition, the soil conditions in Kuala Lumpur are variable even within short distances.

Using Observational Method, Inverse Analysis application had been around for geotechnical problems since the 1980's to evaluate performance of geotechnical structures and also to identify soil parameters for future design. (C. Rechea a, 2008)

1.2 Problem Statement

The design of geotechnical structures requires a lot consideration criterion in analysis and design. In many cases, soil properties are not accurately known and wrongly predicted. Also, many designs are not done in a more conservative approach, which failed to compensate for additional magnitude of settlement or bearing capacity, which technically lead to failure of geotechnical structures. (Y.C. Tan, 2008)

PLAXIS, a finite element modelling (FEM) software for soil and rock analysis had been used for design and simulation of basement excavation and construction by many geotechnical engineers in consultancy firms nowadays. FEM is theoretically complex and any small neglect is likely to quantify unreasonable results. The engineer or analyst must be well-verse with geotechnical knowledge and experience but at times, they may be relatively new to the field or software.

1.3 Objectives

This study is aimed to deduce representative values for design parameters of geotechnical structures around Kuala Lumpur using Observational Method by correlation of different types of field monitoring instrumentation tests results to achieve the following:

1. Identify suitable subsoil parameters for PLAXIS input in simulation of basement excavation and construction

1.4 Scope of Study

Field monitoring from three high rise projects in the vicinity of Kuala Lumpur City Centre was provided by Web Structures Pte Ltd, a civil structural and geotechnical consulting firm and reviewed. In order to achieve the objective, this project focused on simulation of basement excavation and construction. Back analysis is conducted to achieve correlation between inclinometer reading results.

CHAPTER 2

LITERATURE REVIEW AND THEORY

2.1 Observational Method

Inverse Analysis is a process of evaluating the correct soil parameters deduced from field observations. In geotechnical term, it is known as "Observational Method". It was proposed by Karl Terzaghi and followed up by Ralph B.Peck in 1969 (Wikipedia, 2009) where observation during construction stages will be used to evaluate assumptions derived during design stage. The key feature for this method is a standardised, planned and systematic approach to objectively observe and record behaviour of field performance. The shortcomings in the implementation of Observational Method are due to the lack of general understanding on the principles of Observational Method. (Patel, 2005)

Despite the shortcoming, Observational Method had improved confidence in geotechnical design to reduce risk on accidents related to geotechnical construction. Contrary to the traditional ground engineering projects, Observational Method involved the active role of instrumentation and monitoring to check original predictions during design with the actual results during construction to allow any modification to be carried out when necessary.

The eight (8) steps involved in Observational Method can be listed as follow:

1. Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
2. Assessment of the most probable conditions and the most unfavourable conceivable deviations from these conditions. Geology plays a major role.
3. Establishment of the design based on a working hypothesis of behaviour anticipated under the most probable conditions.
4. Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.

5. Calculation of values of the same quantities under the most unfavourable conditions compatible with the available data concerning the subsurface conditions.
6. Selection in advanced of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.
7. Measurement of quantities to be observed and evaluation of actual conditions.
8. Modification of design to meet actual conditions.

2.2 Simulation of Basement Excavation and Construction

2.2.1 Finite Element Modelling using PLAXIS

Finite Element Modelling is a numerical approximation of mathematical equations by taking a continuum approach to geotechnical systems and solving the governing equations analytically which is divided into one-dimensional and two-dimensional problems. According to Scott *et. al* (1999) and Schlosser (1999), Finite Element had been proven to analyse lateral deformation during basement excavation at various construction stages. This study involved the simulation of basement excavation of the case studies investigated using the finite element program for soil and rock analysis, PLAXIS.

2.2.2 Selection of Constitutive Model and Limitations

The 5 constitutive models in finite element modelling are:

- The Mohr-Coulomb Model (Perfect-Plasticity)
- The Jointed Rock Model (Anisotropy)
- The Hardening-Soil Model (Isotropic Hardening)
- The Soft-Soil-Creep (Time-Dependent Behaviour)
- The Soft-Soil Model

Analysis performed by Ng H,H 2007) and Liew *et. al* (2007) are quite similar, where both had recommended Hardening Soil Model compared to Mohr Coulomb Model as it is the more precise and advanced constitutive model for prediction of soil deformation. Mohr Coulomb is suitable if there are insufficient soil data and simple, but had the tendency of producing misleading result as the stress-dependency of stiffness is not taken into account. This means that all stiffness do not increase with pressure which is not probable in real life.

Meanwhile, Soft Soil Model is not recommended for use in excavation problems as it is a Cam Clay type model meant for primary compression of near normally-consolidated clay-type soils. Jointed Rock Model is meant to stimulate the behaviour of rock layers involving stratification and particular fault directions.

Ng H.H (2007) had also recommended that more back analysis should be carried out to establish the correlation between normal consolidated soil and highly consolidated soil. He also had suggested that a study on the ratio between loading and unloading stiffness as input parameter in modelling excavation using Hardening Soil Model is important for better understanding of the relationship of in-situ soils.

In the study by Liew *et. al* (2007), parameters such as wall interface, Rinter of 0.8 had been adopted for diaphragm wall to optimize design. Rinter affects significantly the soil-structure interaction and may give a close estimate on the wall displacement profile. Also, normally in a conservative PLAXIS design, The author did not considered the surcharge load during the back analyses to stimulate the condition at site but had mentioned that undrained condition is able to give good prediction on wall lateral displacement.

2.2.3 Selection of Excavation Method

In PLAXIS, it is important to select whether the analysis method is drained or undrained. Each method had distinctive differences and should be selected based on the nature of the project and the characteristics of the subsoil. Drained analysis should be done with the effective stress while undrained analysis can be carried out with either the total stress or effective stress (Yu-Ou, 1996)

2.2.4 Excavation Process

Excavations depend on several factors such as depth of water table, nature of soil, height of excavation and the existing structures in the vicinity of the excavation (Schlosser 1999). In the case studies investigated, top-down construction method is selected as the excavation method. Several pre-stressed struts are progressively installed during excavation of soil. The struts are then removed floor by floor and floor slabs are built accordingly.

In top-down construction method, the floor slabs are permanent structures, which replace temporary steel struts in braced excavation method to counteract earth pressure from the back of retaining wall. The basement structure is finished when the excavation process is competed. As for the retaining system, diaphragm wall are selected as it can be used as a permanent structure. It has low vibration and noise which makes it suitable for construction in Kuala Lumpur. The thickness and depth of the wall are adjustable and has good watertight capability.

In the first phase of diaphragm wall construction, the perimeter length of the building is divided into several panels according to construction conditions. Guided wall are first constructed, followed by the excavation of trenches. After excavating the trenches, mud in the trench must be cleared away. Then steel cages reinforcement is placed into the trench. Finally, concrete will be poured into the excavated trench using Tremie pipe.

2.2.5 Monitoring Systems for Lateral Deformation

This study utilizes Observational Method principles by comparing results with monitoring data from field observations. The geotechnical measuring instruments for monitoring reports involved are:

- Vibrating Wire Strain Gauges
- Inclinometer
- Observation Well
- Precise Levelling

Vibrating wire strain gauges measures strains by the measurement of changes in natural frequency of the wire. The strain gauge connected with a sensor will produce a magnetic field that will vibrate the wire. The sensor will measure the natural frequency of the strain gauge and by converting the constant of the instrumentation, the stress and strain of the wire can be derived.

Inclinometer is the device used for the measurement of lateral deformation of retaining structure or soils. The inclinometer tracks can be installed inside or outside of the diaphragm wall, preferably on the section with the largest lateral displacement. The pair of tracks perpendicular to retaining structure is called A axis and the other pair paralleling is called B axis. Tilt reading is obtained at 0.5m intervals as the probe is drown from the top of the casing. The value taken from the inclinometer is the relative horizontal displacement between two points.

Observation well is instrumentation for monitoring the changes groundwater level before excavation and during excavation. A vertical pipe with many holes enveloped with two layers of nylon net is placed after boring to a designed depth. The bottom of the borehole and space will be filled with sand and the water observation pipe will be inserted into the hole. After groundwater flows into pipe and reaches the stable state, the water level in the pipe is then the groundwater level.

Precise Levelling is a method for settlement survey. Levelling staff will be placed at all settlement points and reading will be taken by the precise level with reference to a datum or benchmark. The measurement principle is similar to inclinometer and the tilt of a building can be estimated by the relative settlement between two references points.

2.2.6 Parametric Studies for PLAXIS

The following discussed some of the important considerations to model the construction sequence with reference to **BS8002:1994 Code of Practice for Earth Retaining Structures.**

The soil parameters that are sensitive in PLAXIS are (Y.C. Tan, 2008) :

- Tangent/Shear strength parameters (c' and φ')
- Stiffness parameter (E)
- Coefficient of earth pressure at rest (Ko)
- Wall-Ground Interface factor (δ)
- Permeability of soil (k)

The obligatory surcharges of 10kN/m² on retain soil can be applied throughout the X-axis of the modelled site although normally assumed as 5m beyond the excavation pit. Designing with C'= 0 is not appropriate in Hardening Soil Model especially when there is presence of sand. Zero cohesion will reduce calculation performance. Soil body collapses in a few trials. However, C'=0 can be used in Mohr coulomb model.

As stated in the code, minimum depth of additional unplanned excavation in front of the wall, should be not less than 0.5m and not less than 10% of the total height retained for below the lowest support level for propped or anchored wall. It's a general requirement to have allocation of unplanned excavation in design of retaining structures.

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CHAPTER 3

METHODOLOGY

3.1 Methodology

The flow chart of research is as shown in Figure 3.1.

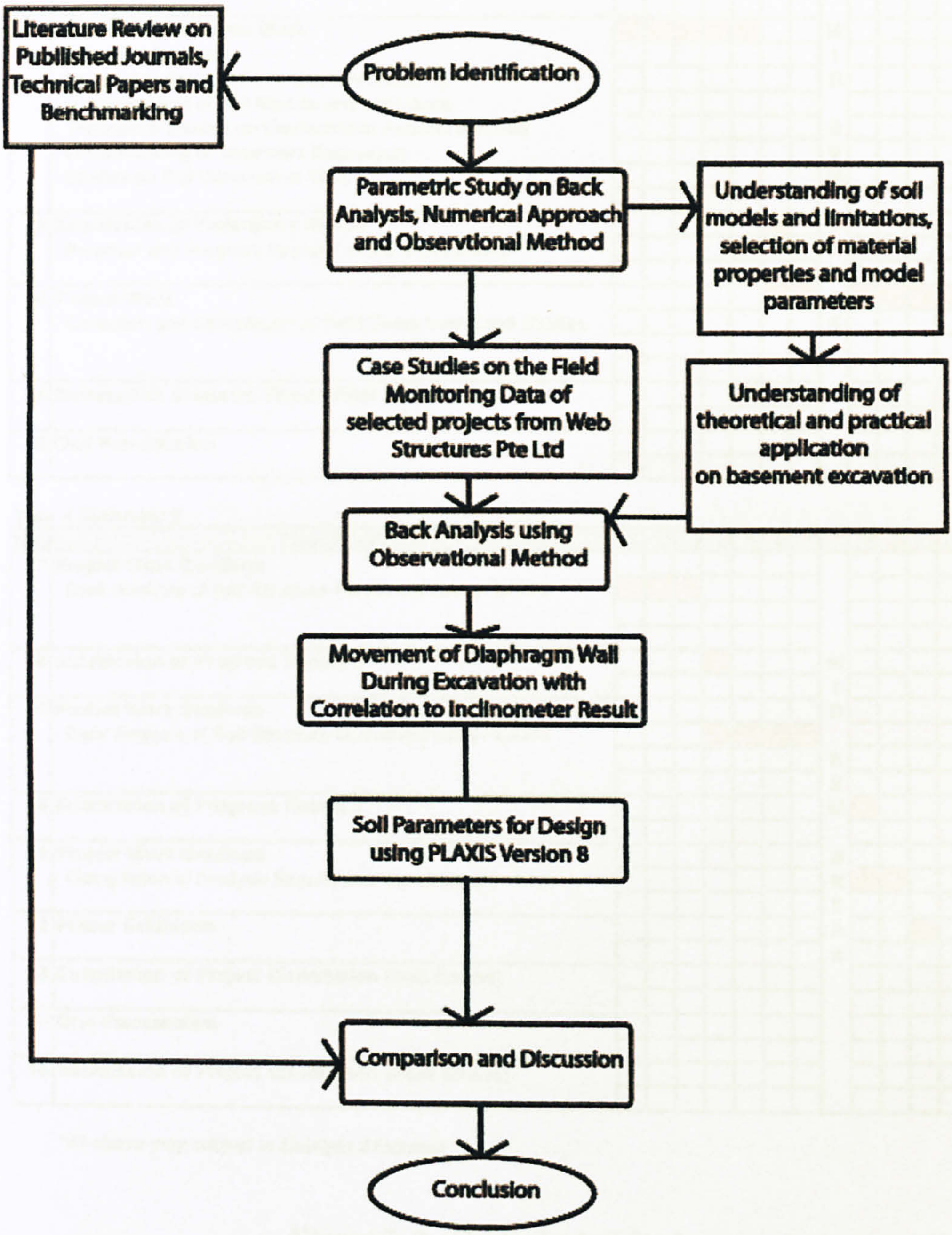


Figure 3. 1 Flow Chart of Research

3.2 Gantt chart

The completion of the research is scheduled within the duration of one year. Figure 3.2 illustrates the Gantt Chart of she research.

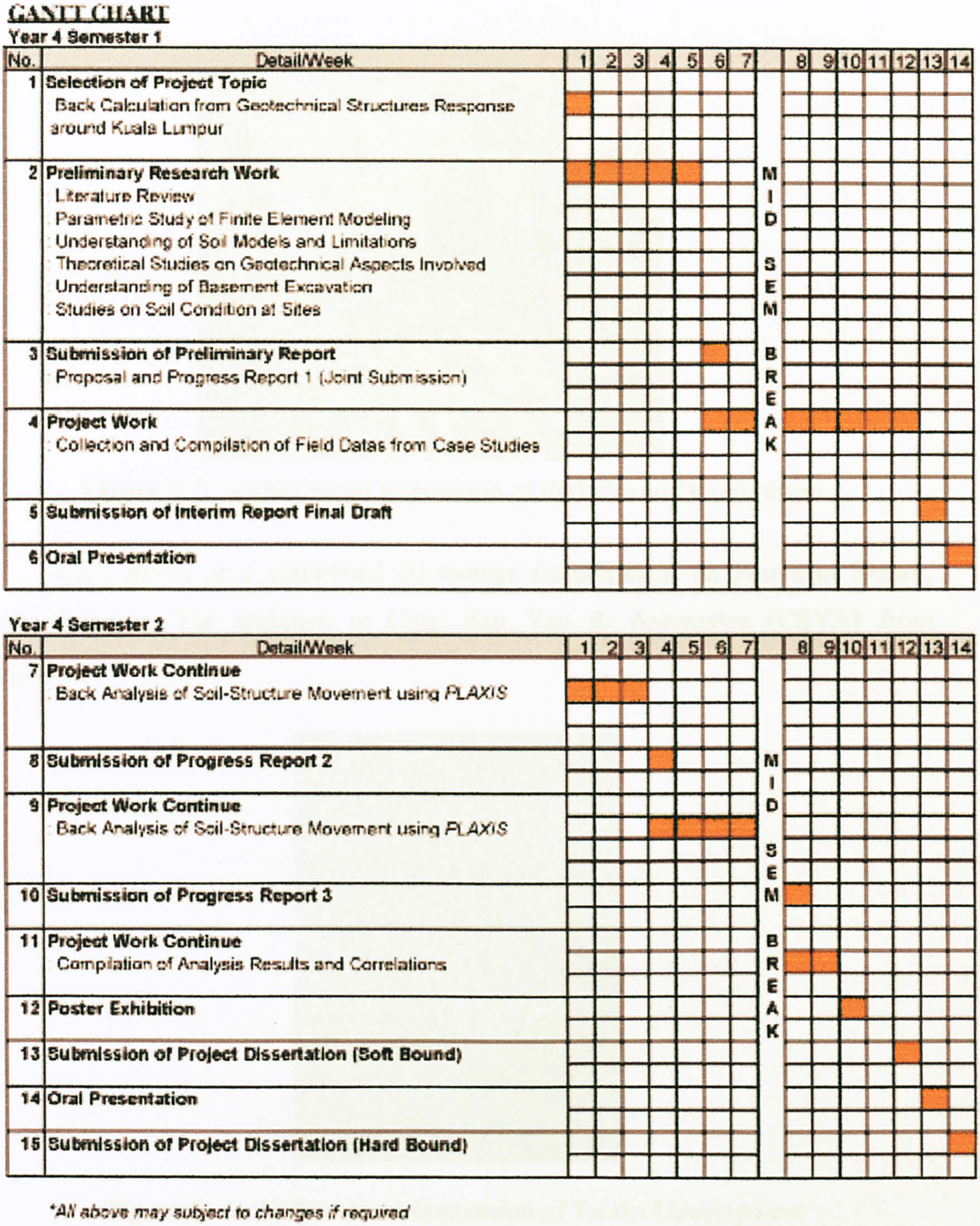


Figure 3. 2 Gantt chart of Study

3.3 Case Studies

The three (3) case studies involved are ***Park Seven***, ***Troika*** and ***IMC Parkville***. All three (3) are high rise projects are located in the vicinity of Kuala Lumpur City Centre.

Figure 3. 3 Architectural Impression of Park Seven Development

Park Seven is a completed 20 storeys development on Persiaran Stonor, Kuala Lumpur. The architect is Chan Sau Yan & Associates (CSYA) from Singapore.

Figure 3. 4 Architectural Impression of Troika Development

Troika is on-going three towers development of 38, 44 and 50 storeys on Jalan Binjai, Kuala Lumpur. One of the three towers will be the tallest residential development in Kuala Lumpur. The architect is Foster & Partners from United Kingdom.

Figure 3. 5 Architectural Impression of IMC Parkville Development

IMC Parkville is a current on-going development comprising of two towers of 51 storey and 48 storey luxury apartment on Jalan Tun Razak, Kuala Lumpur. The architect is Tsao & Mckown from New York.

3.4 Selection Field Data

Inclinometer reading is field data required for correlation purposes.

CHAPTER 4
RESULTS AND DISCUSSIONS

4.1 Location and General Geology of Case Studies

All three case studies are located in the vicinity of Kuala Lumpur City Centre.

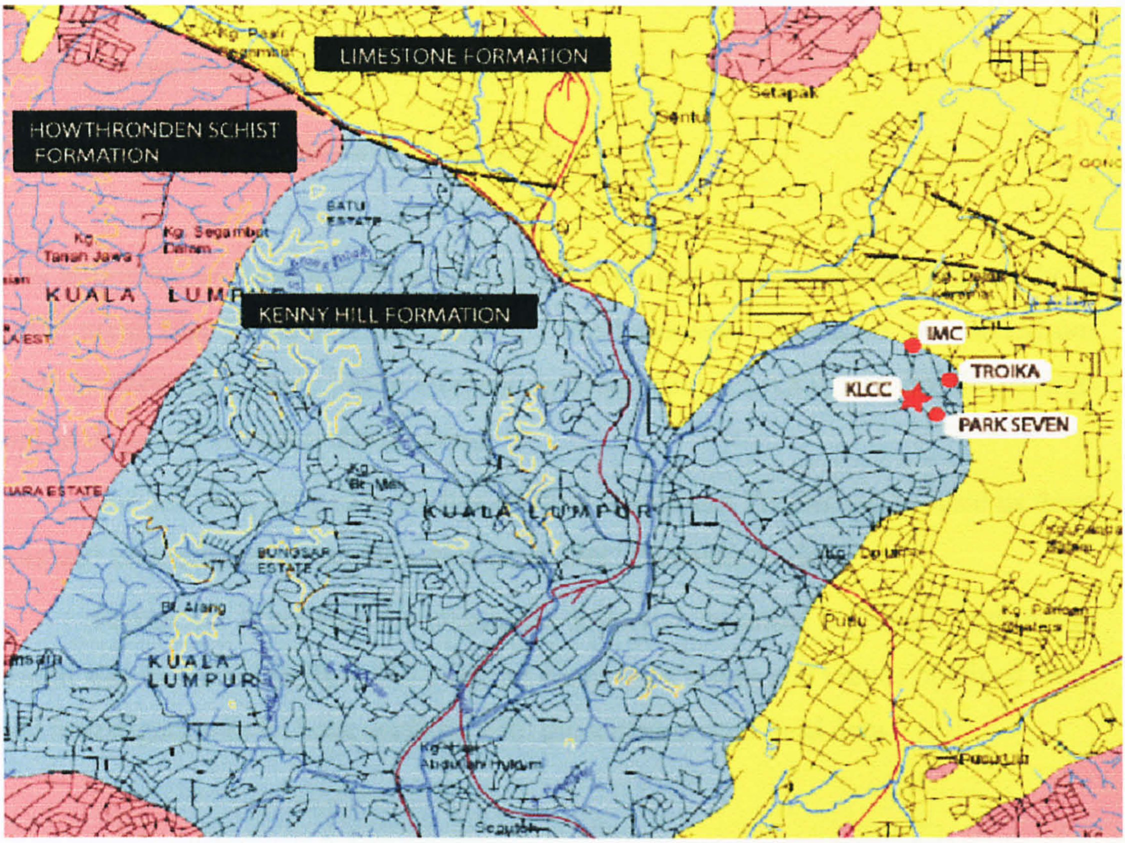


Figure 4. 1 Locations of Case Studies

The sites are located above the Kuala Lumpur Limestone and Kenny Hill formation. Kenny Hill rock mass is a meta sedimentary rock formation, consists of interbedded sandstones and shale of Upper Silurian-Devonian age (Mohamed *et al*,2004).

Kuala Lumpur Limestone is highly erratic karstic features from pure calcitic limestone. Kuala Lumpur Limestone is Upper Silurian marble, finely crystalline grey to cream, thickly bedded, variably dolomite rock.

Experiences had shown that limestone bedrock can change drastically within short distances whereby cavities which are partially filled or without in-fill. The in-fills are usually slimy, having low N-values. The highly erratic limestone profile is overlain by weak sandy, silty soil with low SPT values. About one-third of the area in Kuala Lumpur is on limestone formation (Gue S.S, 1997).

The information on the soil borehole data are extracted and represented in Figure 4.4.

4.2 Basement Excavation and Construction

4.2.1 Simulation of Sequences in PLAXIS

It is important to control deformation of the wall and retained ground especially in urban areas like Kuala Lumpur where by the basement excavation are carried out close existing structures. Finite element analysis using PLAXIS can predict, design and analyse soil-structure deformation.

For this part, only case studies of ***Park Seven*** and ***Troika*** can be used as there was still no inclinometer reading data from ***IMC Parkville*** to date.

Figure 4.2 described the sequences involved in actual basement excavation which was modelled in PLAXIS with the exact time frame and values obtained from soil investigation to be back analysis with the instrumented field data.

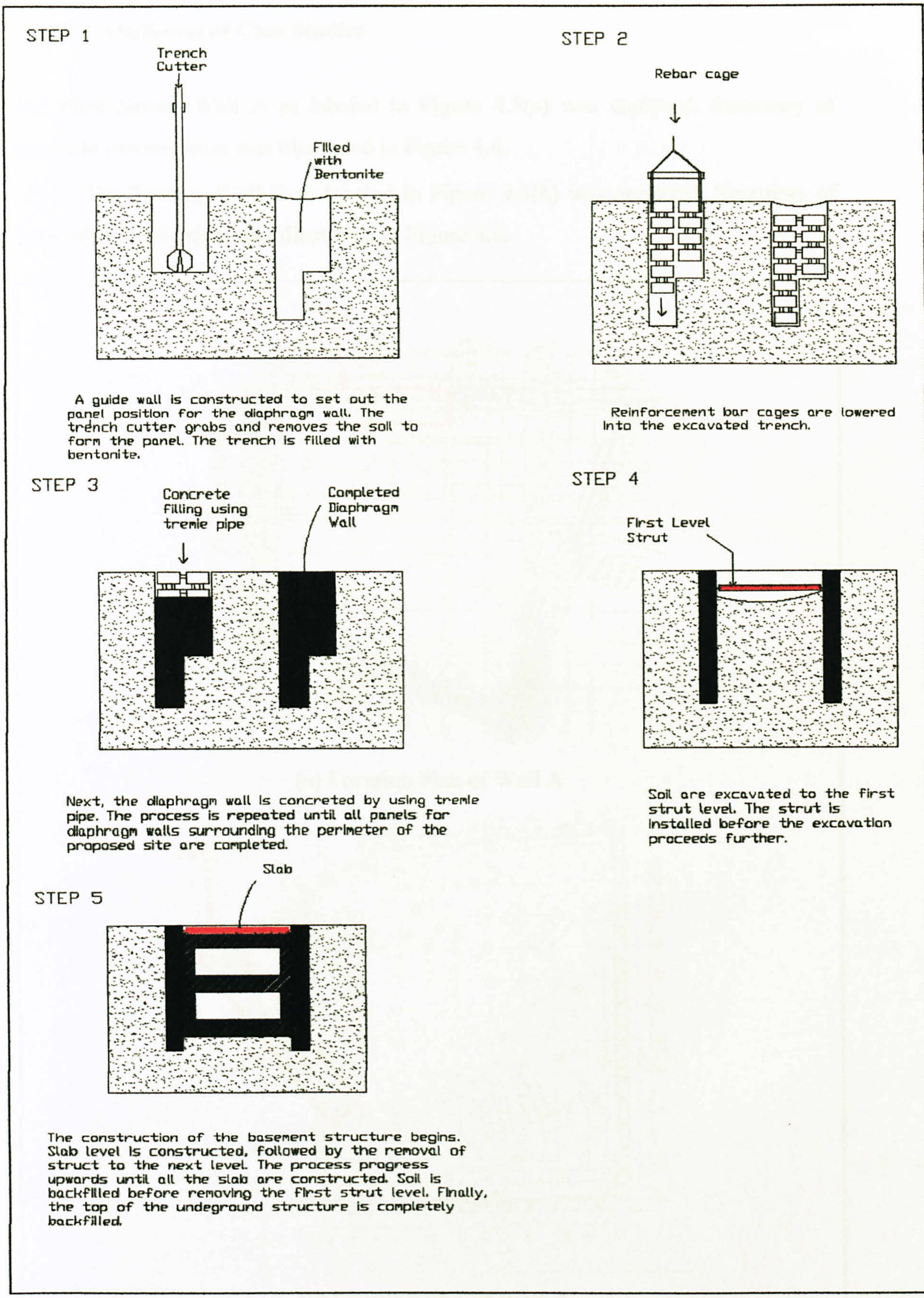
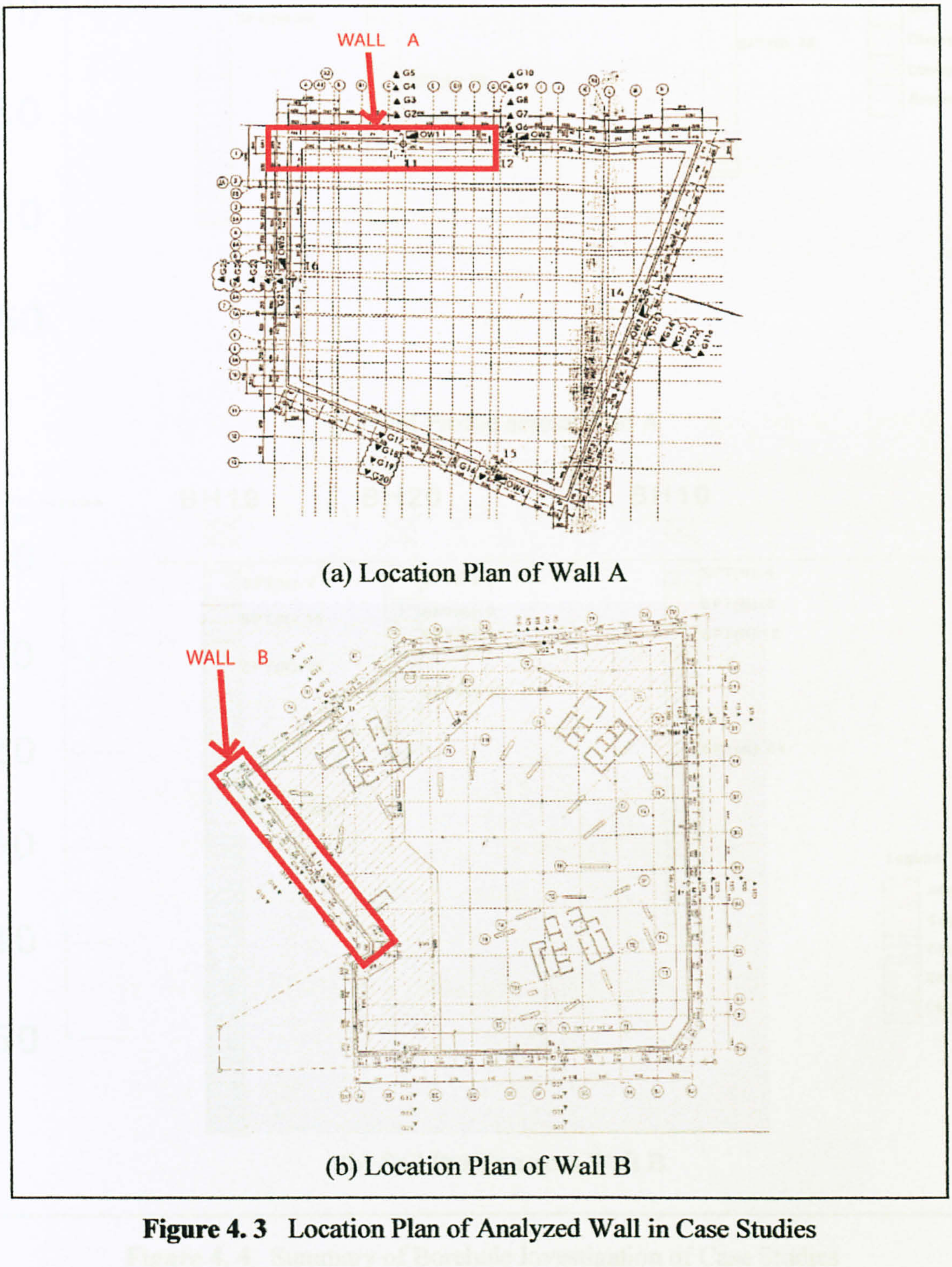


Figure 4. 2 Schematic Representation of Basement Excavation and Construction of Diaphragm Wall

4.2.2 Attributes of Case Studies

For ***Park Seven***, Wall A as labeled in Figure 4.3(a) was analyzed. Summary of borehole investigation was illustrated in Figure 4.4.

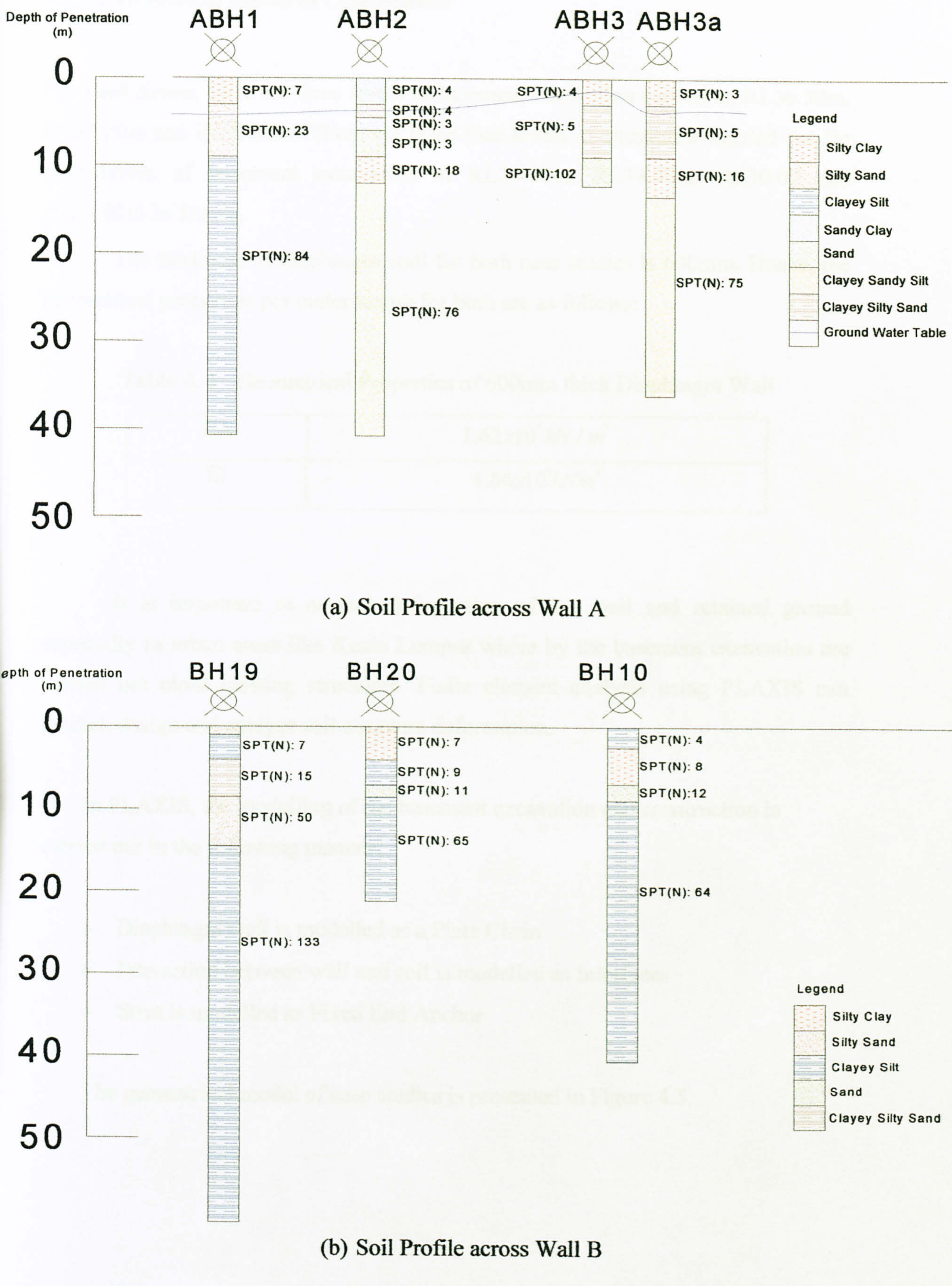
For **Troika**, Wall B as labeled in Figure 4.3(b) was analyzed. Summary of borehole investigation was illustrated in Figure 4.6.



(a) Location Plan of Wall A

(b) Location Plan of Wall B

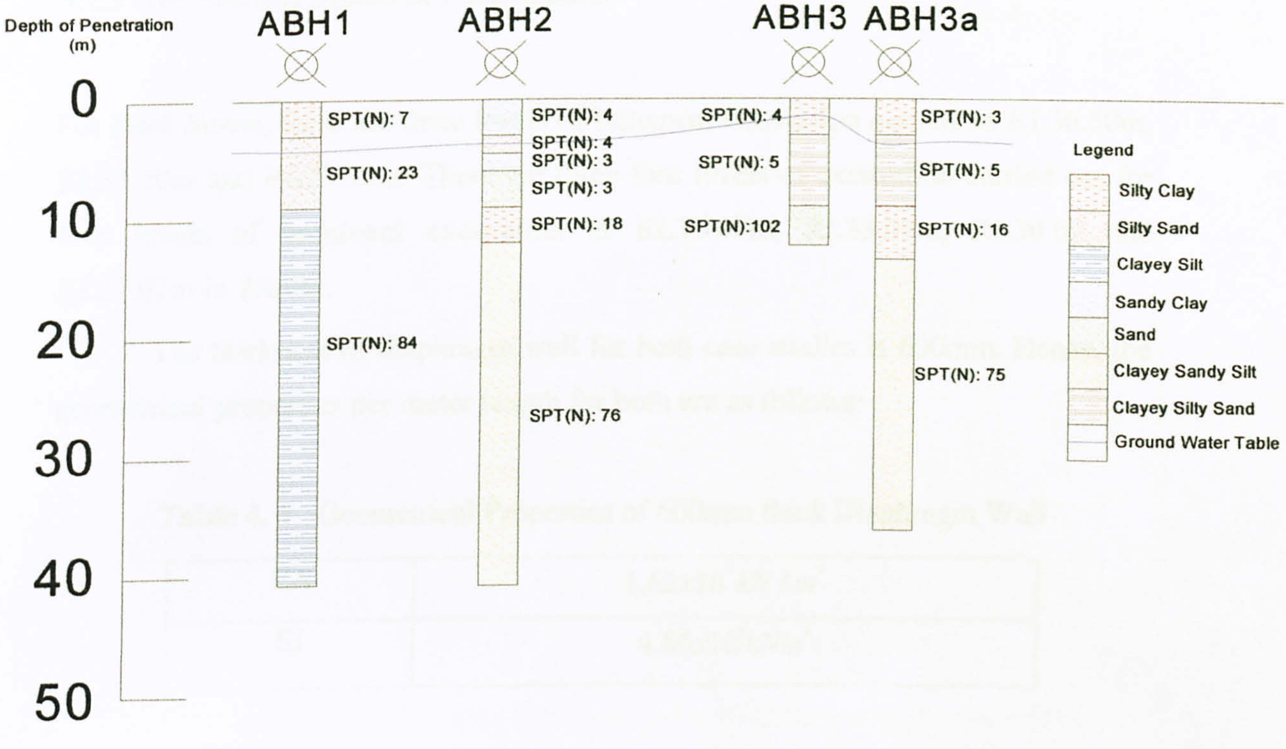
Figure 4. 3 Location Plan of Analyzed Wall in Case Studies



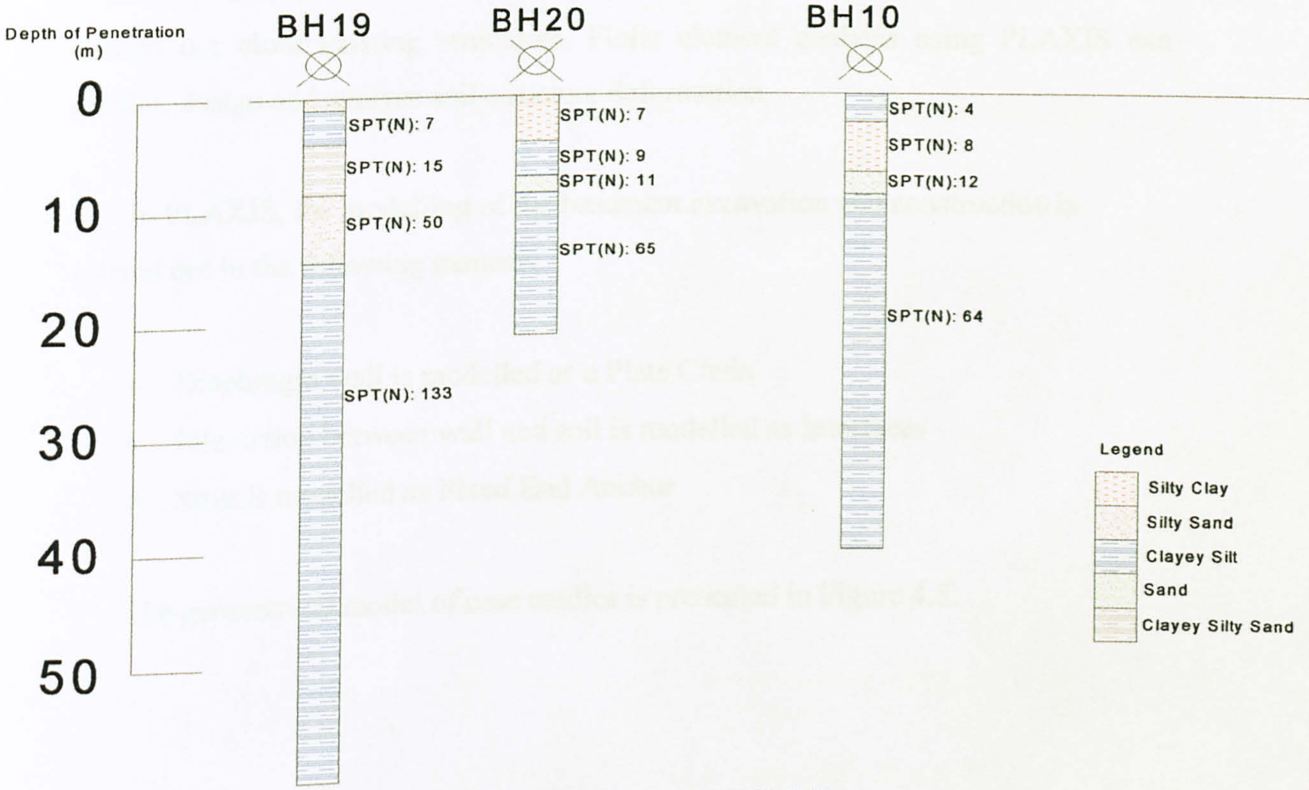
(a) Soil Profile across Wall A

(b) Soil Profile across Wall B

Figure 4. 4 Summary of Borehole Investigation of Case Studies



(a) Soil Profile across Wall A



(b) Soil Profile across Wall B

Figure 4. 4 Summary of Borehole Investigation of Case Studies

4.2.3 Geometrical Model of Case Studies

For ***Park Seven***, there are three levels of basement excavation carried on RL36.50m, RL33.20m and RL29.90m. There are three four levels of excavation carried out for four levels of basement excavation at RL36.07m, RL33.07m, RL30.07 and RL27.07m in ***Troika***.

The thickness of diaphragm wall for both case studies is 600mm. Hence, the geometrical properties per meter length for both are as follows:

Table 4. 1 Geometrical Properties of 600mm thick Diaphragm Wall

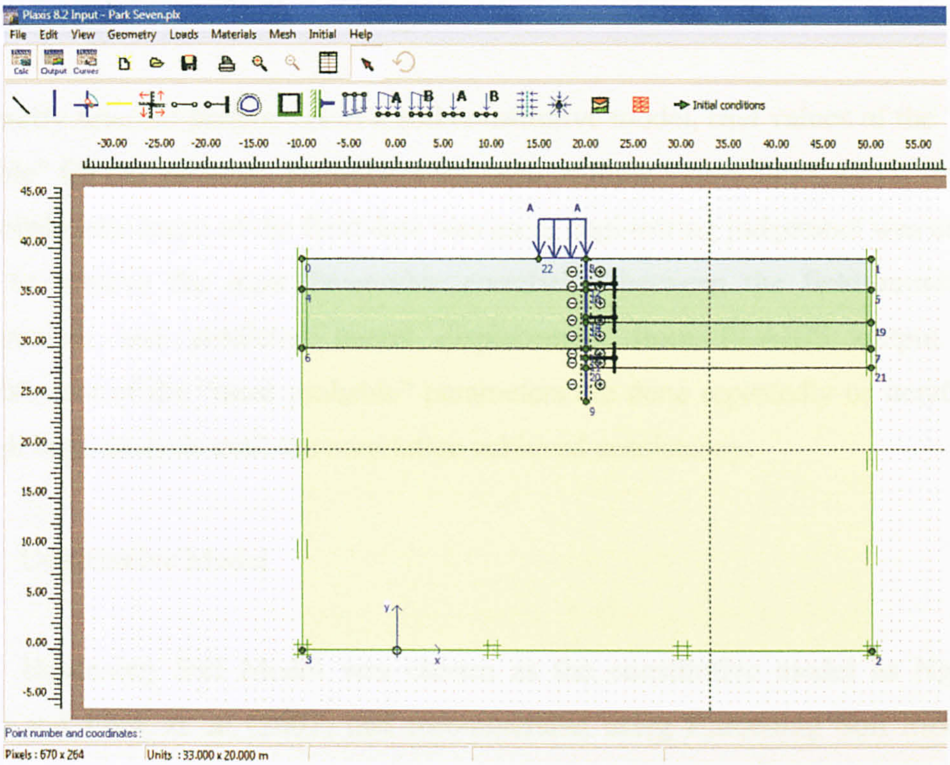
EA	$1.62x10^7 kN/m^3$
EI	$4.86x10^5 kNm^2$

It is important to control deformation of the wall and retained ground especially in urban areas like Kuala Lumpur where by the basement excavation are carried out close existing structures. Finite element analysis using PLAXIS can predict, design and analyse soil-structure deformation.

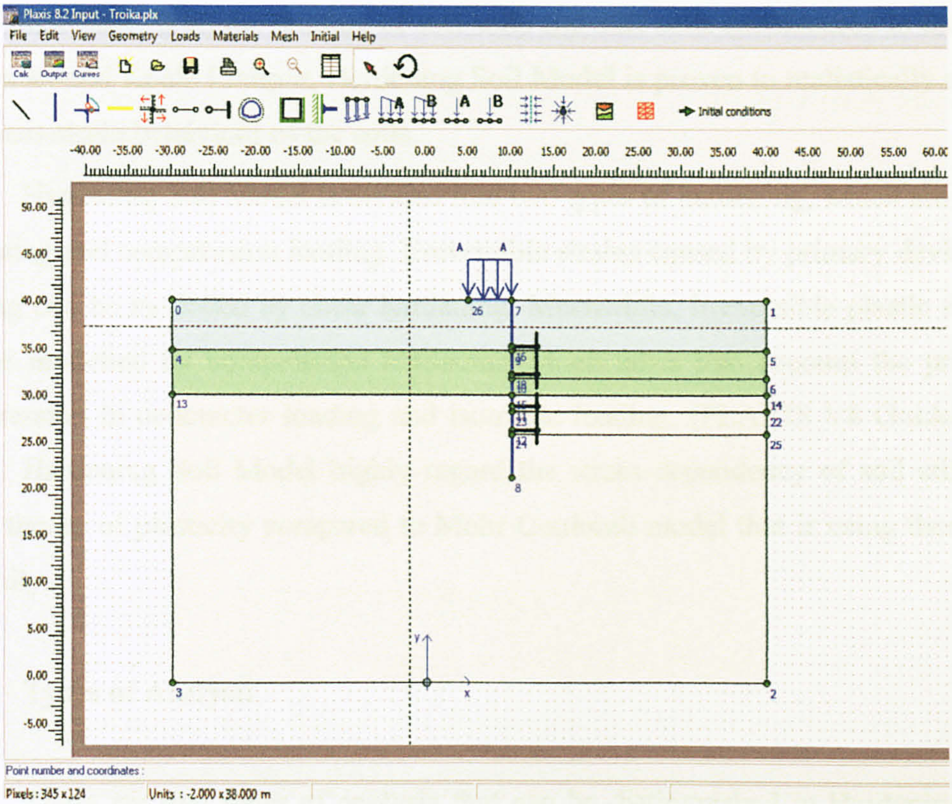
In PLAXIS, the modelling of the basement excavation and construction is carried out in the following manner:

- Diaphragm wall is modelled as a Plate Chain
- Interaction between wall and soil is modelled as Interfaces
- Strut is modelled as Fixed End Anchor

The geometrical model of case studies is presented in Figure 4.5.



(a) PLAXIS Interface of Excavation Model for Wall A



(b)PLAXIS Interface of Excavation Model for Wall B

Figure 4. 5 Geometrical Model of Case Studies

4.2.4 Soil Parameter Input

To identify selected parameters of a soil constitutive model, trial values of the "most probable" for the unknown parameters are used as input values in PLAXIS. Besides the probabilistic mean of the field data was used, engineering judgement was used in order to achieve the most favourable correlation between the field monitoring measurement and predicted lateral displacement from PLAXIS output. The establishment of the "most probable" parameters are done repeatedly or iteratively, through back analysis until the correlation achieved consistency.

- Constitutive Model

Hardening Soil Model was chosen as the constitutive model as Ng H.H (2007) and Liew *et. al* (2007) had recommended using Hardening Soil Model in PLAXIS as constitutive modelling for prediction of soil deformation compared to Mohr-Coulomb. With the mixture of soft soils and stiff soils encountered in the case studies around Kuala Lumpur, Hardening Soil Model is proven to realistically model the stress-strain behaviour of the soils.

Hardening Soil Model is divided into two types of hardening, which are shear hardening and compression loading. Irreversible strains caused by primary deviatoric loading can be modelled by shear hardening. Meanwhile, irreversible plastic strains can be modelled by compression hardening which takes into account the primary compression in oedometer loading and isotropic loading. (PLAXIS V8 Guidebook, 2007). Hardening Soil Model highly regard the stress dependency of soil stiffness using theory of plasticity compared to Mohr Coulomb model that is using theory of elasticity.

- Types of Analysis

There are two types of analysis that can be distinguished in Hardening Soil Model, namely Drained Analysis and Undrained Analysis. For this study, Undrained Analysis was used instead of Drained Analysis. Undrained Analysis, a total stress analysis approach is able to compute excess pore pressure when the soil layers are subjected to loads and able to conclude the stability of geotechnical structures.

Drained Analysis, an effective stress analysis is only suitable for conditions that are relatively constant which is not true in the case studies evaluated as high variability was encountered in Kuala Lumpur's Limestone Formation. Furthermore, it does not take into account pore water pressure analysis.

However, both Drained Analysis and Undrained Analysis must be done in an event of a design work to evaluate long term and short term analysis.

- Stiffness Parameter

E_{50}^{ref} is the parameter that yield plastic straining due to primary deviatory loading while E_{ur}^{ref} yield plastic straining due to compression and E_{oed}^{ref} is the reference parameter for elastic loading-unloading. These stiffness parameters are known to be the parameters that influenced the behaviour of the soil-structure movement the most.

For the effective Young Modulus parameter for residual soils in Malaysia, (Tan *et. al,* 2002) had suggested the following correlation:

$$E_{50}^{ref} = 2000 \text{ x SPT'N' (kN/m}^2\text{)} \tag{1}$$

$$E_{oed}^{ref} = 3 \text{ x } E_{50}^{ref} \text{ (kN/m}^2\text{)} \tag{2}$$

An attempt was made to establish a correlation of:

$$E_{50}^{ref} = 3000 \text{ x SPT'N' (kN/m}^2\text{)} \tag{3}$$

Output also showed a relatively good agreement with the monitored data. For subsoil condition, the mean value of test result is only applicable for back analysis purposes. For initial design stage, it's appropriate to select the worst SPT'N value for stiffness parameters.

- Hydraulic Conductivity of Soil

The property of the ability for water to flow through pores of soil mass is known as the Hydraulic Conductivity. In all the case studies, water table was discovered between 4-5m below ground level. Cohesion in soils can be reduced by water in soil. There may be a marked reduction in the shear strength component. The parameters in PLAXIS for this property were chosen based on the soil types which are mostly clayey silt and silty sandy clay. Assuming the major component is of the clayey type, a lower hydraulic conductivity values was selected for the parameter input. Also, an assumption of same values for both k_x and k_y was made although $k_y$values can be of different magnitude than k_x values.

Soil permeability is able to ensure that the soil model can model the drained and undrained behaviour correctly. Assumptions of k_x and k_y had been made according to the following **Figure 4.6** adapted from BS8004 Code of Practice:

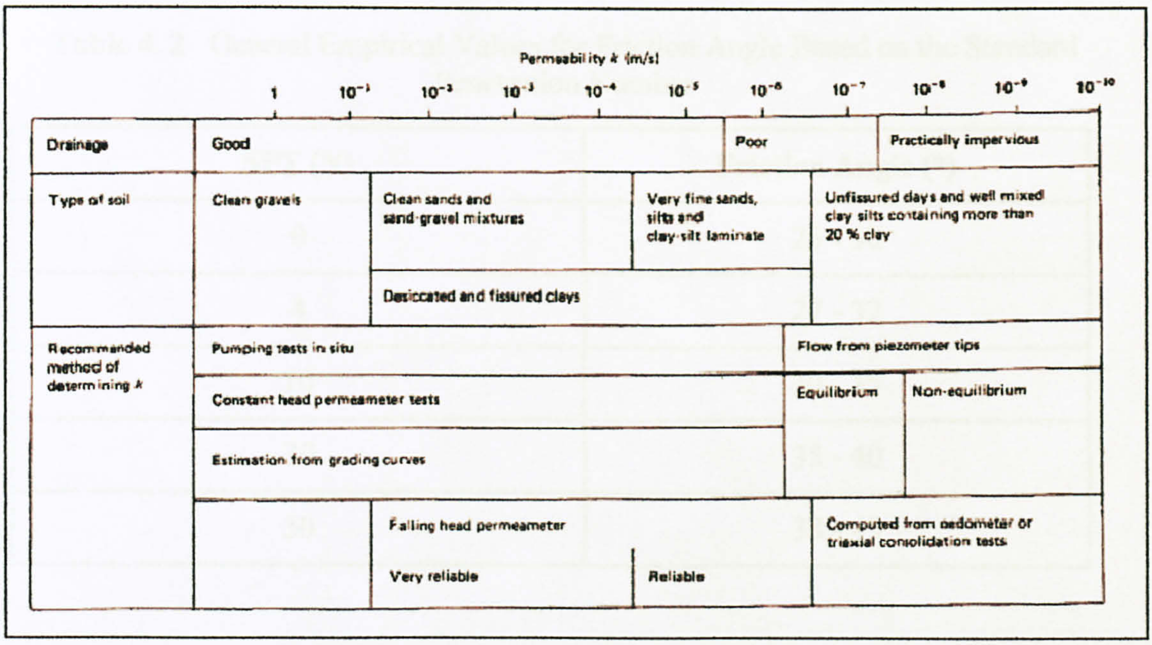


Figure 4. 6 Permeability and drainage characteristics of soils

- Cohesion

Cohesion is another a major factor which affects the final output in PLAXIS. Presence of water may destroy cohesion, and thus, conservative values have to be properly selected. Cohesion is the component of shear strength of the soil which consists of a force that holds the soil particles together within a soil. Total stress analysis conducted for Undrained Analysis is typically used for cohesive soil.

- Friction Angle

Angle of internal friction for a given soil is a component of shear strength besides cohesion. This property represents the angle on the Mohr Circle's graph where shear failure occurs as depicts by the shear stress and normal effective stress. The following table is the general empirical values for ϕ based on the standard penetration number (Bowles, 1982)

Table 4. 2 General Empirical Values for Friction Angle Based on the Standard Penetration Number

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

However, an assumption for more conservative values was made for the back analysis. The values of the back analysis are tabulated in Table 4.3 and Table 4.4.

Table 4. 3 Soil Parameters using Hardening Soil Model for Park Seven

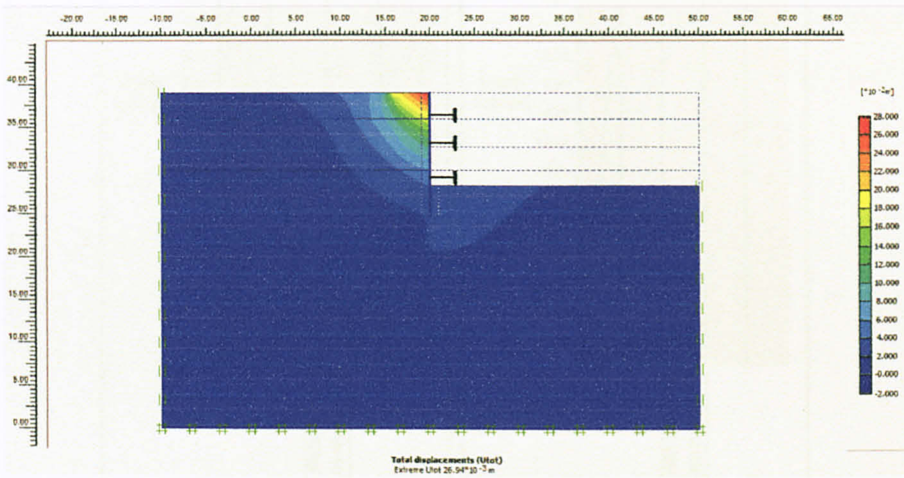
Hardening Soil Parameter	Layer 1	Layer 2	Layer 3
Type	Undrained	Undrained	Undrained
k_x and k_y (m/day)	0.0864	0.0864	0.864
γ_{unsat} (kN/m³)	18	18	19
γ_{sat} (kN/m³)	19	20	20
Average SPT (N)	6	13	107
E_{50}^{ref} (kPa)	12000	26000	214000
E_{ur}^{ref} (kPa)	12000	26000	214000
E_{oed}^{ref} (kPa)	36000	78000	642000
Power Coefficient (m)	0.5	0.5	0.5
Cohesion c (kPa)	1	3	10
Friction Angle ϕ (°)	25	30	35
Dilatancy Angle ψ (°)	0	0	0

Table 4. 4 Soil Parameters using Hardening Soil Model for Troika

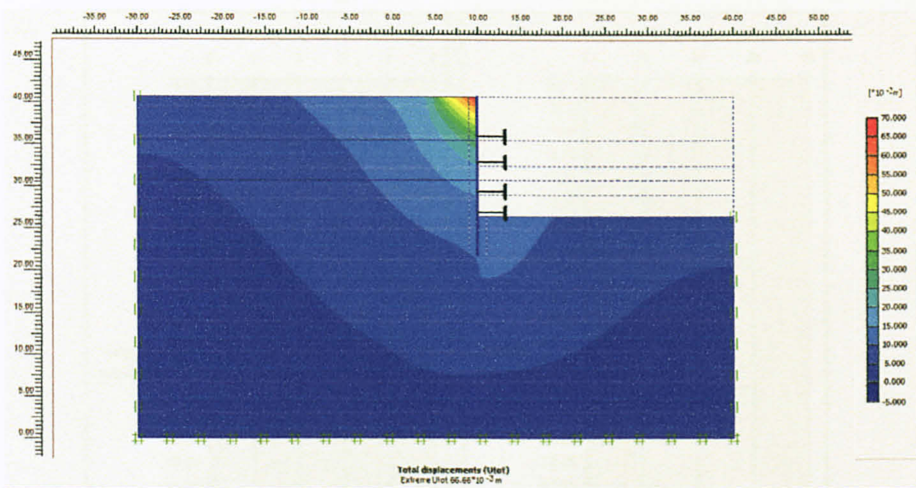
Hardening Soil Parameter	Layer 1	Layer 2	Layer 3
Type	Undrained	Undrained	Undrained
k_x and k_y (m/day)	0.0864	0.0864	0.864
γ_{unsat} (kN/m³)	18	18	19
γ_{sat} (kN/m³)	19	20	20
E_{50}^{ref} (kPa)	14000	22000	56000
E_{ur}^{ref} (kPa)	14000	22000	56000
E_{oed}^{ref} (kPa)	42000	66000	168000
Power Coefficient (m)	0.5	0.5	0.5
Cohesion c (kPa)	2	3	5
Friction Angle ϕ (°)	28	30	35
Dilatancy Angle ψ (°)	0	0	0

4.2.5 Output of Soil-Structure Analysis

Figure 4.7 is the output on soil-structure deformation generated from PLAXIS modelling.



(a) Output for Park Seven



(b) Output for Troika

Figure 4. 7 Outputs for Final Stage of Excavation

Figure 4.8 summarized the result of correlation fit between monitored and predicted lateral displacement fit for both Park Seven and Troika.

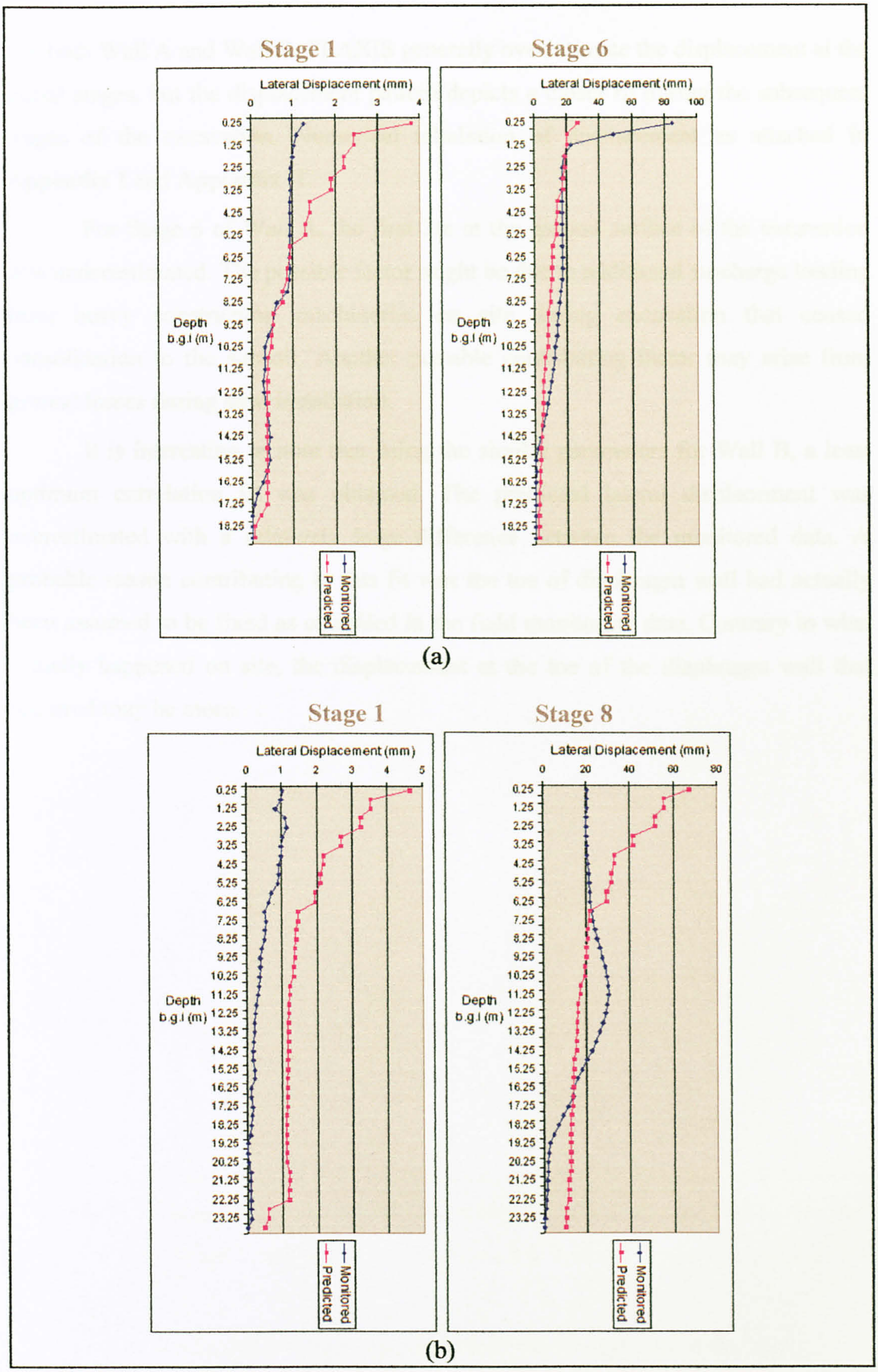


Figure 4. 8 Monitored and Predicted Lateral Displacement Fit of (a) Wall A for Park Seven; (b) Wall B for Troika

For both Wall A and Wall B, PLAXIS generally overestimate the displacement at the initial stages, but the displacement pattern depicts a closer fit during the subsequent stages of the excavation. Numerical tabulation of displacement as attached in **Appendix I** and **Appendix II.**

For Stage 6 of Wall A, the first 2m at the ground surface of the excavation was underestimated. The possible factor might be due to additional surcharge loading from heavy construction machineries on site during excavation that caused consolidation to the subsoil. Another possible contributing factor may arise from ground losses during strut installation.

It is interesting to note that using the similar parameters for Wall B, a least optimum correlation fit was obtained. The predicted lateral displacement was overestimated with a relatively huge difference between the monitored data. A probable reason contributing to this fit was the toe of diaphragm wall had actually been assumed to be fixed as compiled in the field monitoring data. Contrary to what actually happened on site, the displacement at the toe of the diaphragm wall that occurred may be more.

4.2.6 Comparison of Back Analysis with Initial Design

From Figure 4.9, it was proven that the initial design by Contractor X's had underestimated the soil-structure interaction for Stage 6.

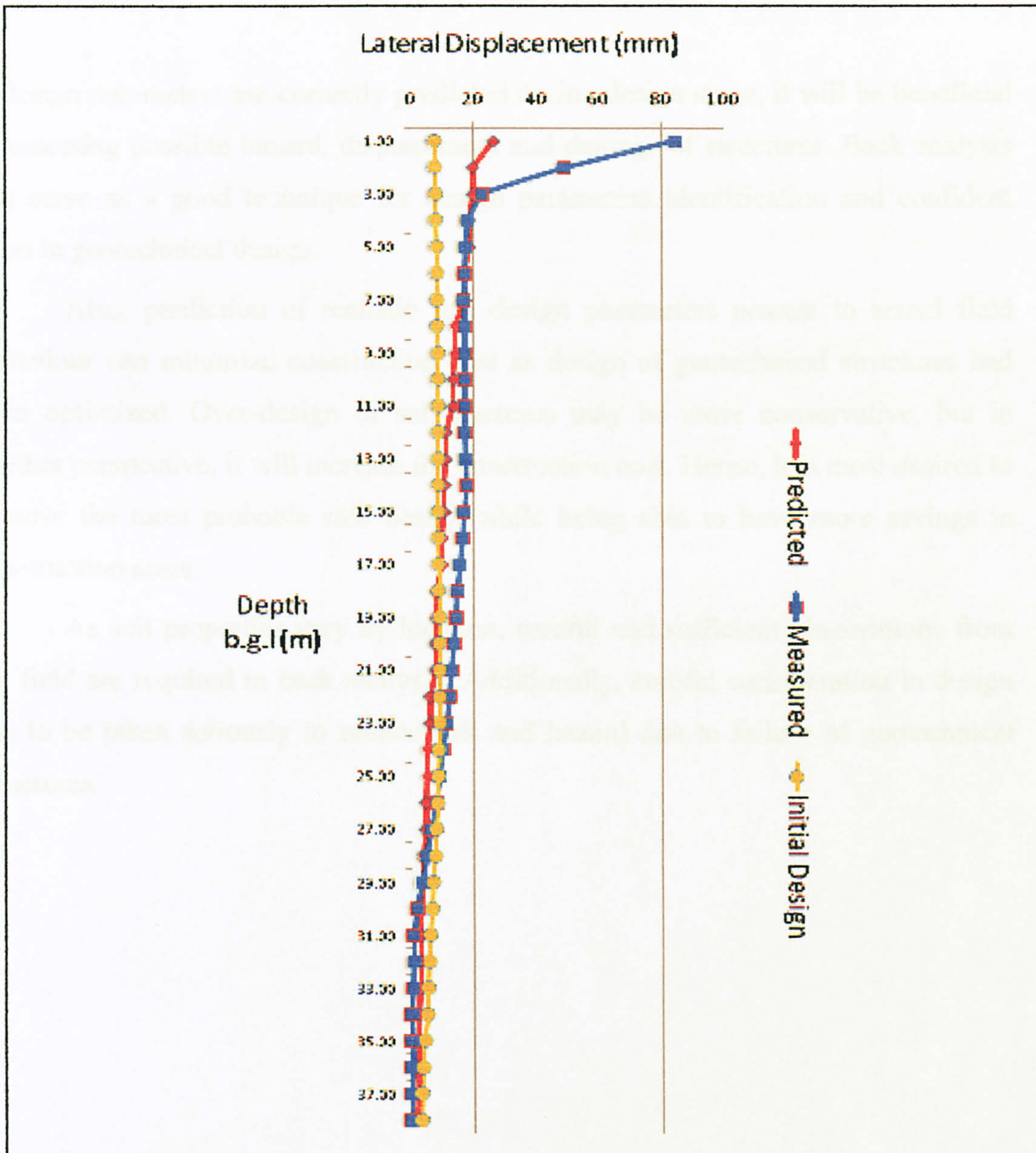


Figure 4. 9 Comparison between Measured and Predicted Lateral Displacement of Stage 6 of Wall A with Contractor X's Initial Design

From a research on past records done, the design was done using Mohr Coulomb as constitutive model. Thus, Mohr Coulomb model is not a suitable constitutive model for basement excavation and construction design as it is unable to model the soil-structure interaction realistically.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Conclusion

If design parameters are correctly predicted during design stage, it will be beneficial in assessing possible hazard, displacement and damage of structures. Back analysis can serve as a good technique for design parameters identification and confident basis in geotechnical design.

Also, prediction of realistic soil design parameters nearest to actual field behaviour can minimize construction cost as design of geotechnical structures had been optimized. Over-design of substructures may be more conservative, but in another perspective, it will increase the construction cost. Hence, it is most desired to achieve the most probable safe design while being able to have more savings in construction costs.

As soil properties vary by location, careful and sufficient observations from the field are required in back analysis. Additionally, careful consideration in design has to be taken seriously to reduce risk and hazard due to failure of geotechnical structures.

5.2 Recommendation

Monitored data of pore water pressure from piezometer test can be back-analysed to evaluate the stability of soil structure interaction. A back analysis study can be carried out to identify and reduce inconsistency between in-situ properties with design properties.

A study on economic optimization on geotechnical structures from inverse analysis or back analysis can also be carried out. The significance of this recommendation is that designed geotechnical structures can be safe without over-design in order to keep construction cost at minimum.

As mentioned by Patchay (2007), there are at least fifty upcoming high rise projects in the outlook of Kuala Lumpur's development for 2008. This research was done in response to predict suitable design parameters for geotechnical structures for potential developments in Kuala Lumpur in the near future.

5.1.2 Soil Design Parameters for Basement Excavation and Construction

For future simulation of basement excavation and construction works using PLAXIS, the following parameters may serve as a guide for design purposes:

- Geometrical Model
    - Refer Table 5.1

Table 5. 1 Comparison of Geometry Model Design Value between BS8002 and Back Analysis

	BS 8002	**Back Analysis**
Minimum Surcharge	10kN/m²	15kN/m²
Minimum Depth of Unplanned Excavation	- Not < 0.5m - Not < 10% of retained wall length	1.0m
Groundwater	Info from records of site, geological map or memoirs	Info from Observation Well field data (Worst possible ground water level)
Wall-Ground Interface Factor	Design tan δ = 0.75 x design tan φ'	δ = 0.7
Coefficient of Earth Pressure at Rest	Ko= 1 – sin φ'	Ko = 0.47-0.53

- Constitutive Model
  - Hardening Soil

- Design SPT (N)
  - Lowest SPT (N) value encountered from Borehole Investigation

- Type of Analysis
  - *Drained* for evaluation of long-term stability of constant condition of soil

  - *Undrained* for evaluation short-term stability of loading/unloading condition of soil

- Soil Stiffness Parameter
  - E_{50}^{ref} = 2000 x SPT'N' (kN/m^2)
  - $E_{oed}^{ref} = 3$ x E_{50}^{ref} (kN/m^2)

- γ_{unsat} (kN/m^3)
  - Range of value: 18 to 19 kN/m^3

- γ_{sat} (kN/m^3)
  - Range of value: 19 to 20 kN/m^3

- Friction Angle and Cohesion
  - Refer Table 5.2

Table 5. 2 Suggested Empirical Values for Friction Angle and Cohesion Based on the Standard Penetration Number

SPT(N)	Friction Angle (°)	Cohesion (kPa)
0-10	25 - 29	1
11- 70	30 - 34	3
70 – 150	35 - 40	10

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APPENDIX

APPENDIX 1

Case Study: Park Seven
Day: Stage 1

Depth (m)	Measured Deflection (mm)	Predicted Deflection (mm)
0.25	1.26	3.79
0.75	1.09	2.44
1.25	1.03	2.44
1.75	0.99	2.18
2.25	0.95	2.18
2.75	0.93	1.89
3.25	0.94	1.89
3.75	0.93	1.38
4.25	0.93	1.38
4.75	0.93	1.27
5.25	0.93	1.27
5.75	0.93	0.91
6.25	0.9	0.91
6.75	0.91	0.83
7.25	0.9	0.83
7.75	0.86	0.73
8.25	0.6	0.73
8.75	0.53	0.51
9.25	0.51	0.51
9.75	0.41	0.46
10.25	0.33	0.46
10.75	0.32	0.38
11.25	0.32	0.38
11.75	0.28	0.37
12.25	0.3	0.37
12.75	0.34	0.37
13.25	0.38	0.37
13.75	0.4	0.36
14.25	0.44	0.36
14.75	0.41	0.36
15.25	0.42	0.36
15.75	0.35	0.36
16.25	0.26	0.36
16.75	0.13	0.35
17.25	0.04	0.35
17.75	0	0.2
18.25	0	0.05
18.75	0	0.05

Case Study: Park Seven
Day: Stage 6

Depth (m)	Measured Deflection (mm)	Predicted Deflection (mm)
0.25	84.93	26.84
0.75	48.98	20.17
1.25	22.73	20.17
1.75	18.25	18.81
2.25	17.66	18.81
2.75	17.27	17.46
3.25	17.07	17.46
3.75	17.26	14.38
4.25	17.17	14.38
4.75	17.16	13.72
5.25	17.11	13.72
5.75	17.21	11.32
6.25	17.53	11.32
6.75	17.8	10.83
7.25	17.06	10.83
7.75	16.47	10.10
8.25	15.65	10.10
8.75	14.85	8.39
9.25	14.38	8.39
9.75	13.81	7.89
10.25	12.9	7.89
10.75	12.31	6.09
11.25	11.45	6.09
11.75	10.3	5.18
12.25	9.12	5.18
12.75	7.99	4.76
13.25	6.42	4.76
13.75	4.98	3.94
14.25	3.46	3.94
14.75	1.85	3.09
15.25	0.56	3.09
15.75	0.84	2.96
16.25	0.34	2.96
16.75	0.15	2.43
17.25	0.27	2.43
17.75	0.21	2.00
18.25	0.14	2.19
18.75	0	2.00

APPENDIX II

Case Study: Troika

Day: Stage 1

Depth (m)	Measured Deflection (mm)	Predicted Deflection (mm)
0.25	1.03	4.61
0.75	0.99	3.50
1.25	0.84	3.50
1.75	1.10	3.23
2.25	1.15	3.23
2.75	1.02	2.67
3.25	0.98	2.67
3.75	0.98	2.18
4.25	0.93	2.18
4.75	0.93	2.08
5.25	0.89	2.08
5.75	0.71	1.93
6.25	0.64	1.93
6.75	0.50	1.45
7.25	0.57	1.45
7.75	0.52	1.40
8.25	0.51	1.40
8.75	0.43	1.35
9.25	0.39	1.35
9.75	0.39	1.32
10.25	0.37	1.32
10.75	0.34	1.20
11.25	0.32	1.20
11.75	0.28	1.18
12.25	0.25	1.18
12.75	0.22	1.17
13.25	0.21	1.17
13.75	0.20	1.17
14.25	0.18	1.17
14.75	0.17	1.14
15.25	0.21	1.14
15.75	0.21	1.13
16.25	0.12	1.13
16.75	0.12	1.13
17.25	0.16	1.13
17.75	0.13	1.11
18.25	0.12	1.11
18.75	0.10	1.11
19.25	0.01	1.11
19.75	0.02	1.11
20.25	0.03	1.11
20.75	0.09	1.18
21.25	0.08	1.18
21.75	0.08	1.17

22.25	0.10	1.17
22.75	0.10	0.58
23.25	0.11	0.58
23.75	0	0.46

Case Study: Troika
Day: Stage 8

Depth (m)	Measured Deflection (mm)	Predicted Deflection (mm)
0.25	20.37	67.20
0.75	20.37	55.43
1.25	20.37	55.43
1.75	20.22	51.31
2.25	20.16	51.31
2.75	20.28	41.34
3.25	20.34	41.34
3.75	20.48	32.83
4.25	20.66	32.83
4.75	20.96	31.22
5.25	21.29	31.22
5.75	21.78	28.90
6.25	21.78	28.90
6.75	22.40	21.40
7.25	23.13	21.40
7.75	23.98	20.56
8.25	24.99	20.56
8.75	26.10	19.75
9.25	27.24	19.75
9.75	28.31	19.15
10.25	29.15	19.15
10.75	29.73	16.91
11.25	29.77	16.91
11.75	29.40	15.68
12.25	28.60	15.68
12.75	27.43	15.30
13.25	25.96	15.30
13.75	24.25	14.97
14.25	22.32	14.97
14.75	20.18	13.84
15.25	18.00	13.84
15.75	15.74	13.56
16.25	13.41	13.56
16.75	13.41	13.32
17.25	11.20	13.32
17.75	9.03	12.54
18.25	6.86	12.54
18.75	4.74	12.41
19.25	2.92	12.41
19.75	2.26	12.41

20.25	1.94	12.30
20.75	1.82	12.30
21.25	1.63	11.36
21.75	1.24	11.36
22.25	0.94	11.36
22.75	0.80	9.99
23.25	0.36	9.99
23.75	0.01	9.74