NUMERICAL MODELING OF A DEEP EXCAVATION AND COMPARISON WITH MONITORING DATA

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DECLARATION

I declare that this is my own work and this thesis/dissertation 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. I retain the right to use this content in whole or part in future works (such as articles or books).

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The above candidate has carried out research for the PhD/MPhil/Masters thesis/dissertation under my supervision. I confirm that the declaration made above by the student is true and correct.

Name of Supervisor: Prof. S.A.S. Kulathilaka

Signature of the Supervisor: Date: 08.05.2024

DEDICATION

This thesis is dedicated to my loving parents Mr. D.A. Padmasiri and Mrs. H.H.V.R. de Silva and my loving husband Mr. K.K. Karunaratne

For their endless love, support and encouragement

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Finally, yet importantly, I appreciate my husband for lending his devoted time to me during the last few years to read for the MSc and encouraging me to complete this final hurdle.

ABSTRACT

With urbanization, space available for construction is limited and available space has to be used very effectively. As a result, people tend to use underground spaces for parking and other service requirements. Accordingly, deep excavations with several levels of basements have become a standard practice in the construction of high rise buildings in urban areas. Any excavation which extends beyond the ground water level is required to be supported by an appropriate earth retaining system to retain the soil and ensure the water tightness of the retaining system while proceeding with the excavation. The main purpose of the retaining systems is to maintain both horizontal and vertical ground movements within allowable limits to avoid any damage to the adjacent built environment and the excavation itself. To achieve the main purpose of retaining walls for deep excavations, the estimation of lateral deformation is required to be done very precisely during the design stage. Further, the estimated deformations are required to be compared with the actual monitoring data during the execution stage. In this research, monitoring data of an excavation supported by a diaphragm wall laterally supported at several levels were compared with the results of the numerical simulation obtained by the finite element method, and for the first analysis Mohr-Coulomb model was used as a constitute model. Then the finite element modelling method was streamlined to obtain more accurate results. Two modifications were adopted for the basic model to improve the performance. First, the elastic modulus of soil was increased considering the unloading effect of soil and next the small strain theory was used. The results of the analysis showed that the model with the combination of both modifications provides more sensible results. There were some anomalies in the actual monitoring data.

Keywords: Deep excavation, Earth retaining system, Diaphragm wall, Lateral deformation**,** Monitoring data, Elastic modulus, Small strain theory

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LIST OF ABBREVIATIONS

Abbreviation	Description
AAA	Alert, Action, Alarm
BH	Borehole
CIRIA	Construction Industry Research and Information Association
ELS	Earth Lateral Supports
FEM	Finite Element Method
HSS	Hardening Soil Model
PSZ	Primary Strain Zone
SPT	Standard Penetration Test
SS	Small Strain
SSZ	Small Strain Zone
UCS	Uniaxial Compressive Strength

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CHAPTER 1

INTRODUCTION

1.1 Background

With urbanization, there is a scarcity of space available for construction, and available space has to be used very effectively. As a result, in the construction of high rise buildings developers are compelled to use the underground space for parking and other service requirements. Accordingly, deep excavations with several levels of basements have become a standard practice in the construction of high rise buildings. Construction of these causing minimum disturbance to the existing structures is a great challenge faced by the construction industry.

Most of the said deep excavations extend below the groundwater level and thus the retaining structure supporting the excavation needs to be watertight. Unexpected water leakages through the retaining system would cause damage to the adjacent structures due to subsidence which will occur with the movement of soil partials with the seeping water. Considering this aspect "diaphragm walls" and "secant pile walls" were considered to be the most appropriate form of excavation support systems under these circumstances. These retaining structures can be designed to be a part of the final structure and a skin wall may have to be constructed in the final stages to avoid any minor water leakages and to enhance the aesthetics.

The most critical concern during the execution of deep excavations is the control of excessive ground movements to protect the adjacent structures. Estimation of deformations can be done by numerical techniques such as the Finite Element Method in the design stage. In this regard, it is very important to idealize the subsoil conditions and to obtain the foundation details of the structures in the close vicinity. An understanding of their structural integrity and ability to withstand additional settlements is also critically important.

A practically plausible construction sequence should be planned and the same should be numerically simulated in the design stage. Critical locations could be identified with this analysis and an appropriate instrumentation plan should be developed to monitor the deformations, stresses, and pore water pressures at appropriate locations and elements. This will enable a comparison between theoretical predictions and the field observations at each construction stage. Observations in the initial construction stages may be used to calibrate stiffness parameters in the subsoil layers. Also, any anomaly in the observation would be able to indicate or provide early warnings of any flaws in the construction process. As there is a possibility that some instruments may malfunction with time it would be necessary to install an additional redundant number of instruments initially.

The scope of this study is to re-analyze the earth retaining system that has been used to retain the 15.75 m deep excavation for the substructure of ODEL Development project at No.10, Ward Place, Colombo 07 using the Finite Element Method. There will be five levels of basements. The basement slabs will act as lateral support for the diaphragm wall finally. In the excavation stage, there are three levels of temporary supports.

The complete stage by stage construction which involves installing temporary supports as excavation progresses and the installation of basement slabs and removal of temporary supports process will be simulated by the finite element technique. As there was no detailed testing to determine the strength and stiffness of subsoil layers the data from initial stages would be used to calibrate the model. The appropriate method of finite element analysis, to model the actual behavior of earth retaining systems is to be selected from the literature.

The probable reasons for any differences between values obtained from analysis results and monitoring data are to be discussed. A series of remedial actions will be proposed for probable unexpected behavior of the structure during the excavation.

1.2 Problem identification

Limiting the ground movements due to deep excavation is very important since it is the main cause of the disturbance and failures of the adjacent structures during the excavation. To minimize those problematic scenarios, deep excavations should be supported by a properly designed earth retaining system. The retaining system may be a temporary or permanent support system which needs to be decided considering the requirements of the project and geological conditions at the site.

A proper monitoring system should be implemented during the excavation to monitor the critical conditions that were identified in the design. However, due to the lack of experience of Sri Lankan contractors regarding the monitoring process, few problems have already occurred.

The monitoring data are required to be collected at pre decided time frequencies and they can be analyzed considering the construction sequence and compared with estimated values. When the data on the strength and stiffness of the subsoil layers are not obtained through an extensive test program, the monitoring data in the early stages can be used to calibrate the model parameters.

Further, the data acquired during monitoring should be promptly analyzed and graphically presented to identify any anomalies.

The lack of attention to the finer aspects of monitoring and necessary corrective actions would lead to catastrophic failures.

The analysis of monitoring data is helpful to assess the acceptability of the construction. If the data collected during monitoring is different from the designed values at the considered time, it indicates that the actual behavior of the structure is different from the anticipated behavior. Accordingly, if the actual behavior of the construction is unsatisfactory, immediate remedial actions should be taken to avoid probable failures.

1.3 Objectives

Objectives of this study are,

- Conduct a Finite Element Method for the earth retaining system that has been used to retain the deep excavation of the ODEL Development project at No.10, Ward Place, Colombo 07
- Compare the analysis results with the actual monitoring data
- Identify the probable reasons for any differences between the two results
- Recommendations on the methods to be followed for a proper monitoring system for deep excavation

1.4 Methodology Applied

The first step was to develop a Finite Element Model for simulating the earth retaining system support for a 15.75 m deep excavation. In this regard, the diaphragm wall used was idealized as a plate element, and permanent and temporary supports were idealized as fixed end anchors. The behavior of soils was modeled with the Mohr Coulomb model and stiffness parameters of soils were assigned using the empirical relationships present in the literature. In the analysis the construction stages used for the actual site conditions were closely simulated Plain strain model was considered for the analysis.

The second step was to idealize the stiffness properties of permanent and temporary supports since they were not perpendicular to the earth retaining system. To satisfy the requirement of plain strain behavior the supports need to be perpendicular to the earth retaining system. Then four sub-soil profiles were selected for analysis by considering the locations of boreholes and inclinometers to cover the four sides of the retaining system.

Predictions made with the Finite Element Modelling were compared with the observed deformations.

Attempts were made to correct the differences between the two sets of values by modifying the stiffness parameters of the soil layers as appropriate.

1.5 Thesis Outline

Chapter 2 of the thesis is a review of the available literature related to the types of retaining structures, construction sequences adopted for deep excavations, the conventional method of designing embedded walls, and the finite element method of designing embedded walls.

Chapter 3 of the thesis presents the details of the project including site location, details of the borehole investigation, details of the earth retaining system, details of both temporary and permanent supports, and details of monitoring.

Chapter 4 of the thesis illustrates the details of idealization and adopted methods to idealize the diaphragm wall, temporary and permeant support systems, and subsurface profile.

Chapter 5 of the thesis presents the results of the finite element analysis and a comparison of the results of the analysis with actual monitoring data.

Chapter 6 of the thesis is summarizing the findings of this research and makes suggestions for further studies in this research area.

CHAPTER 2

LITERATURE REVIEW

A literature review was conducted on past studies of numerical modeling of deep excavations and comparison with monitoring data. Aspects such as types of embedded walls, the conventional approach to designing the embedded walls, and the use of finite element analysis to model the behavior of deep excavations supported by embedded walls and the comparison of monitoring data with numerical modeling were mainly discussed.

2.1. Introduction

The fundamental objective of the support of deep excavation is to keep the surrounding ground environment stable during and after construction by carefully controlling the movements of the sides of the excavation. Endicott (2020) elaborated that, to achieve this, the adjoining ground needs to be fully supported with an Earth Lateral Support (ELS) system during the excavation process. An ELS system normally consists of a robust wall around the perimeter of the excavation and shoring to prevent the lateral wall movements. As illustrated in Simpson and Powrie (2021) the walls that penetrate the ground are considered as embedded retaining walls. Also, they heavily rely on the passive resistance of the ground.

When the ground is strong and space is available, excavation can be done in the form of a sloping ground without using an ELS system. However, when the groundwater table is high and space is limited, ELS systems are essential.

Whittle et. al. (1993) highlighted that when deep excavations are done in an urban environment, it is important to make a reliable prediction of the ground movements and a realistic estimation of the effect of ground movement on the adjacent structures. It further highlighted that, even though numerical methods like finite element analysis can be used to estimate ground deformations, it is difficult to obtain reliable analytical predictions of soil deformations due to the factors highlighted below.

- Deficiencies in the available data from the site investigation and geometric approximations considered in analytical model
- Uncertainties in the selection of engineering properties. There are two major uncertainties involved in the analysis, such that inadequate laboratory and field tests relevant to the engineering properties of all soil layers and the use of approximate representations of constitutive behavior in finite element model
- The construction process can be complex and involve activities which cannot be analyzed easily from a finite element model

- Most of the finite element software relevant to geotechnical applications falls to satisfy the principle of uniqueness for unloading in an elastic medium

2.2. Types of Embedded Walls

Endicott (2020), illustrated that, several methods of ELS systems have developed by considering various factors such as soil condition, level of water table, existing adjacent structures, etc. The five common methods mentioned there were,

- Driven steel sheet piling
- King post and lagging
- Contiguous bored pile walls
- Secant piled walls
- Diaphragm walls

As highlighted in Simpson and Powrie (2021), soldier piles and steel sheet piles were used as embedded retaining walls in the first half of the twentieth century but with the development of the concrete walls formed either in slurry trenches or by contiguous or intersecting (secant) piles became increasingly common in second half of the century.

Endicott (2020), elaborated that in deep excavations ensuring water tightness during the process is very important. Water leaking through the earth lateral support system will carry the soil partials in the adjacent land causing ground subsidence. Secant pile walls and Diaphragm walls are the most effective watertight earth lateral support systems.

<u>Secant Piled Walls</u>

As elaborated in Endicott (2020) secant piles are formed as a row of bored piles and they overlap with adjacent piles by creating a wide contact zone between adjacent piles. They are primarily cast in-situ bored piles with alternate primary unreinforced weak piles and secondary reinforced concrete piles. Normally, primary piles are formed from plain concrete, cement-stabilized soil or jet grout. The secondary piles are conventional bored piles usually filled with strong concrete with steel reinforcement. The main structural element is the secondary pile and the primary piles used as a support for the ground between the secondary piles and to act as a water seal between two secondary piles. Endicott (2020)

Normally, low strength concrete is used to cast the primary piles as they are to be cut by the secondary piles. When primary piles are too strong, the drilling for the secondary piles tends to go off-line resulting in an unacceptable lack of verticality and gaps between the primary piles and the secondary piles. Endicott (2020)

Diaphragm Walls

Endicott (2020) illustrated that, Diaphragm walls are formed with panels of reinforced concrete. Since it has a large structural capacity they are used for very deep excavations. Diaphragm walls are generally made from cast-in-situ reinforced concrete and when it uses precast panels smooth formed inside face can be obtained for permanent uses. Diaphragm walls are built with a wide range of capacities from plain concrete walls about 600mm thick up to reinforced concrete walls more than 1500mm thick. Another important fact is that, Diaphragm walls can be used for shafts, in a circular layout requiring no bracing because of the circular arching action of the completed wall. Endicott (2020)

To construct diaphragm walls, first guide walls are constructed along the alignment for the diaphragm wall with reinforced concrete up to a depth of 1.2m. The purpose of the guide walls is to provide locations for excavating deep trenches and provide support at the edges of the trenches where heavy plants will be operated. After constructing the guide wall short panels are excavated up to the final excavation level. Normally width of the excavation varies from 2.8m to 6m. While being excavated, trenches are kept filled with bentonite slurry. After the completion of excavation, steel reinforcement cages are installed and the trench is filled with concrete using the tremie method. (Endicott,2020)

Endicott (2020) further illustrated that, for deep excavations bracing system is required to limit lateral deformations within acceptable limits. Bracing can be used as a temporary structure or permanent structure as per the design of the structure.

Bracing is commonly provided by struts spanning from one side of the excavation to the other. However, when the walls of excavation are too wide, it is economical to use inclined struts or tieback anchors to the surrounding ground. Endicott (2020)

When inclined strutting is adopted, support of the strut is important. The top of the inclined struts must be restrained to prevent sliding up the walls. Also, the bottom should be supported with a foundation. There are several foundation methods to support struts such that, cast a reaction block of concrete, constructing part of a base slab for the inclined strut to bear against and providing piles at the toes of the struts Endicott, (2020).

Tieback anchors are formed by drilling through the walls into the ground outside the excavation area for a distance far enough to achieve reliable anchorage. High strength steel bars or bundles of high tensile steel cables are threaded through the walls and grouted into the ground. A bearing plate is added to the inside face of the wall Endicott, (2020).

2.3. Sequence of Construction

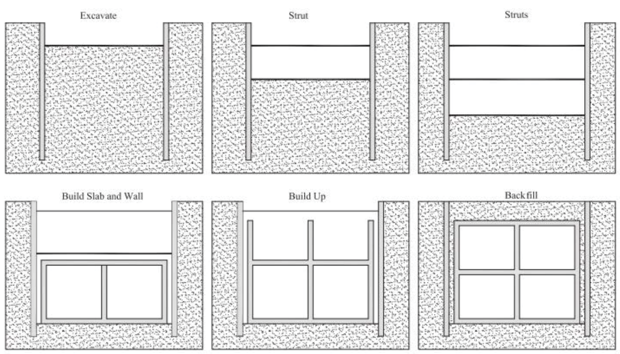
As elaborated in Endicott (2020), there are three major method of sequence of construction.

Bottom – up sequence of construction

In this method, first excavation is completed to the final level with lateral support provided by temporary systems and then the permanent structure is built from the bottom to up. The temporary support system is dismantled with the introduction of permanent supports which are usually in the form of basement slabs. The sequence is illustrated in **Figure 2.1**.

Figure 2.1

Bottom up Construction Sequence (Endicott,2020)

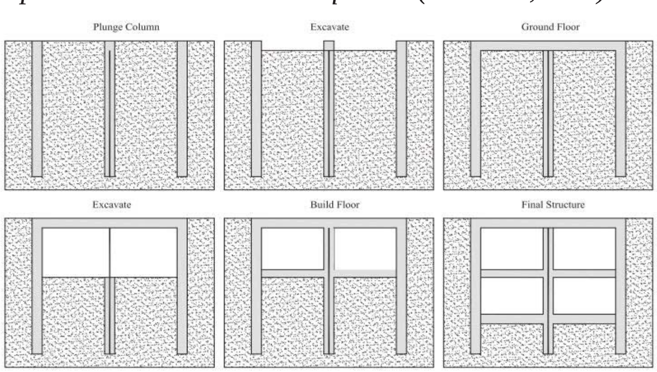


Top-down sequence of construction

In the top-down approach, the permanent underground structures are built simultaneously as the excavation is carried out. First, the outer walls are constructed. While excavation is progressing, permanent structures are constructed and hence temporary bracings are not needed. **Figure 2.2** illustrates the sequence of top-down construction.

Figure 2.2

Top –down Construction Sequence (Endicott,2020)



Hybrid sequence of construction

Some projects are carried out by adopting hybrid schemes using partially top-down and partially bottom up sequences of construction.

2.4. Conventional Design of Embedded Retaining Walls

The stability of the embedded retaining walls in the cantilever form or with a single level of support could be assessed through conventional design approaches through methods such as “Free earth support method” and “Fixed earth support method”. When there are multiple levels of support system becomes statically indeterminate and the conventional methods cannot be used. Struct load envelopes by Peck (1969) were used to estimate the load on lateral supports in such cases. However, all these methods are developed with many assumptions and their applicability to a complex ground condition is questionable. Furthermore, these methods do not provide any information about the deformations associated with the embedded wall or in the surrounding ground. Considering these limitations in the conventional approach methods to numerically simulate the actual process of excavation and lateral support were developed over the last few decades.

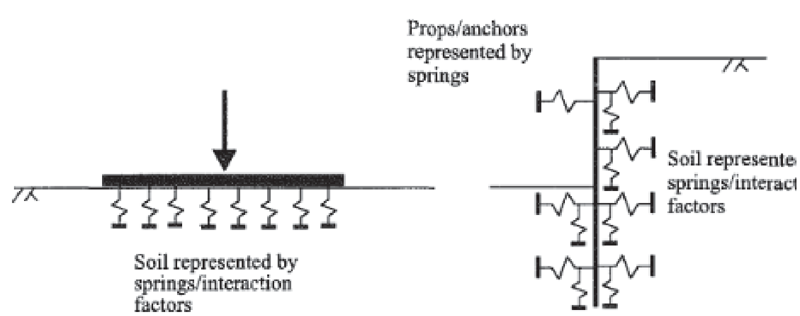
2.5. Analytical Methods to Design of Embedded Retaining Walls

In the initial stages of the development of the analysis of embedded retaining walls the soil mass was represented by a series of springs while modelling the wall and the lateral supports through appropriate structural elements. The values of stiffness in the springs were evaluated through soil characteristics (Liong, 2022). Computer software (WALLAP) was developed under this approach. Although it could estimate the deformations of the wall no information could be obtained on the deformations in the surrounding ground.

As elaborated in Potts and Zdravkovic, 1999 only a single structure can be accommodated in the analysis. Further approximations must be introduced if more than one structure is involved. **Figure 2.3** illustrates the idealization of the soil spring concept.

Figure 2.3

Subgrade Reaction Model (Potts and Zdravkovic, 1999)



As illustrated in Liong, 2022 another limitation of the spring constant model is that the behavior of soil is assumed as linear or elastic. In addition, when the soil is layered and with different elastic parameters, an equivalent model must be developed to derive a representative modulus of subgrade reaction. Another limitation is that the effect of variation of the water table such as dewatering cannot be simulated in this method (Simpson and Powrie (2021).

However, from 1970's numerical procedures were developed idealizing the soil medium also through the finite element approach adopting appropriate soil constitutive models. Nowadays, commercial finite element software specially developed for analyzing geotechnical problems such as PLAXIS, CRISP, and SIGMA are available (Liong, 2022).

As elaborated in Simpson and Powrie (2021), using Finite Element Method (FEM) it is possible to study the behavior of complete problem of a strutted excavation. This method facilitates the analysis using 2D and 3D. Also, the ground movements, and behavior of both the wall and other connected structures in a complete interaction analysis can be done in this method.

Liong, 2022 elaborated that PLAXIS is the widely used software for geotechnical problems and it facilitate both beam type and slip/interface elements which is very helpful for modeling of structural element and the relation of soil-structure interfaces. Furthermore, there are different types of soil models which can be used to simulate the behavior of soil continua and they are illustrated below.

- Linear Elastic Model

Hooke's law of isotropic linear elasticity represents this theory. However, linear elastic behavior is minimal to simulate the soil behavior.

- Morh-Coulomb Model

In general, this well-known model is used as the first approximation of soil behavior. This consists of five parameters namely Young's modulus, Poisson's ratio, cohesion, friction angle and dilatancy angle.

- Hardening Soil Model

This is an elastoplastic type of hyperbolic model. It is formulated in the framework of friction hardening plasticity. To simulate the behavior of sands, gravel, and over consolidated clays, this type of second order models can be effectively used

- Soft Soil Model

This is a Cam-Clay type of model, which can be used to simulate the behavior of soft soils like normally consolidated clays and peat. The model best suited in situations of primary compression.

- Soft Soil Creep Model

This is a second order model formulated in the framework of viscoplasticity. The model can be used to simulate the time-dependent behavior of soft soils.

Lade 2005, illustrated that parameters for a model required to derive from any set of experiments which have data on the compression and shearing behavior of soil. This is due to the fact that, the mathematical expressions in an ideal constitutive model may be specialized and correspond to the particular tests available for parameter determination. Conventional triaxial tests and isotropic compression tests are the simplest and most appropriate experimental results that provide the necessary information.

Liong (2014) elaborated that, to use sophisticated geotechnical software, there should be a sufficient geotechnical background. Otherwise, there is a possibility of making mistakes that will cause a catastrophic failure.

Following are the common mistakes found when applying the PLAXIS 2D as elaborated in Liong (2014).

- Failure to identify the right model of plane strain or axisymmetry

In plane strain it is assumed that strains can only take plain in xy plane and along the out of plane direction strain is assumed to be zero. Also, length of the excavation should be significantly larger compared to the width of the excavation. In the axisymmetric model, it is considered that, radial strains of the model are equal in all directions. Also, the structure is symmetric along the Y axis. And it is rotated at the Y axis in left boundary. The failure to identify the correct model will result in an incorrect output.

- Interface Element

The interaction between the soil and structural element is modeled as an interface. This is used to minimize the friction between the soil and structural element. The interface value is used to minimize the friction between the soil and structural element and the interface value is varying from 0.01 to 1. The lower limit of the value indicates that there is no friction between the soil and structural element. The upper limit of the value indicates that, the soil and structural element cannot slip relative to each other as they are completely rigid. Values in between lower limit and upper limit mean that the friction is reduced by the given number and soil mass and structural element able slip relative to each other.

It further illustrated that, lower interface value results a larger bending moments. Hence it is very important to estimate a reasonable value for the interface. Liong (2014) suggested interface values for different conditions are elaborated in **Table 2.1**.

Table 2.1

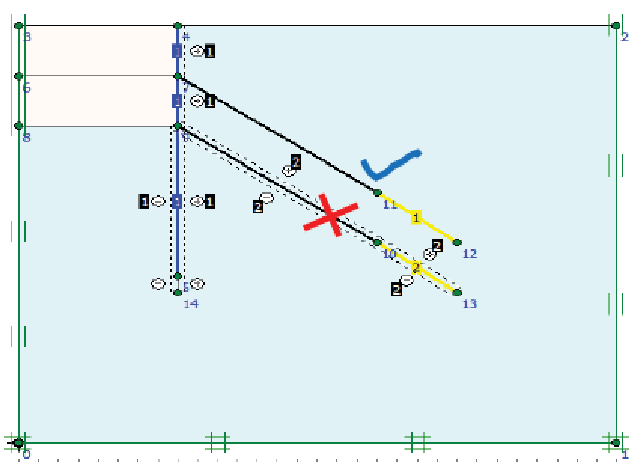
R_{inter} for Different Conditions

Condition	R_{inter}
Interaction sand/steel	0.6 - 0.7
Interaction clay/steel	0.5
Interaction sand/concrete	1.0 - 0.8
Interaction clay/concrete	1.0 – 0.7
Interaction soil/geogrid (grouted body)	1.0 (Interface is not necessary)

The common mistake of the application of interface is when modeling the pressure grouted anchors. As the node to node anchor is connected to structural elements at both ends, there is no contact with the surrounding soil. Therefore, there is no need of application of interface along the body of the node to node anchor. Another thing is that, the bond length of the ground anchor is modeled as a geogrid element. In normal practice, the bond length is pressure grouted and thus the soil around the grouted body is totally in contact with the grouted body. In that case, applying interface element and giving value less than 1 is incorrect. The correct way is to not applying the interface as indicated in **Figure 2.4.**

Figure 2.4

Mistake in modeling ground anchors (Liong (2014)



- Material Models

There are several options available in PLAXIS to simulate the soil behavior as follows.

- Mohr-Coulomb model
- Soft soil model

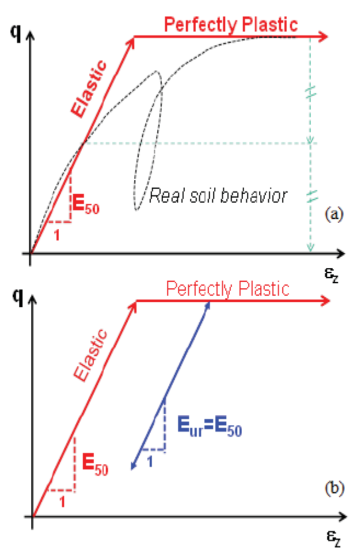
- Hardening soil model
- Soft soil Creep
- Hardening soil with a Small strain
- Modified Cam-clay

Liong, 2014 describes the common mistakes when applying the Mohr Coulomb Model for deep excavation problems.

Mohr-Coulomb model models the non-linear behavior of the soil into two bilinear lines as illustrated in **Figure 2. 5.** In this approach it is considered that soil stiffness E_{50} is constant at elastic region while soil actually has a non-linear behavior. With that, when the stress level is less than 50% the model over predicts the ground movement while when the stress level is higher than 50% model under-predicts the ground movement.

Figure 2.5

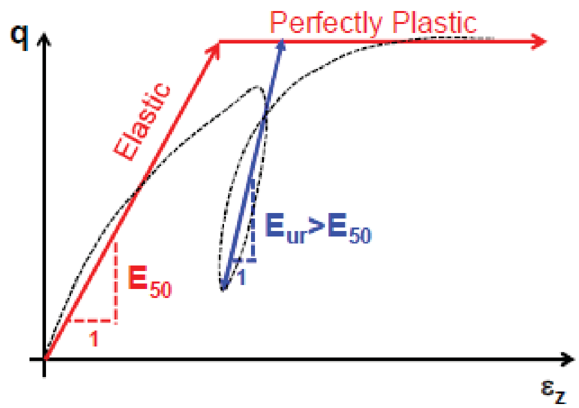
Mohr Coulomb Model (Liong (2014)



One other drawback of this model is that, the assumption of soil unloading –reloading stiffness modulus is equal to the loading stiffness. In actual case, in the unloading reloading condition soils have a much stiffer modulus compared to the loading condition and this is normally of 2 to 5 times the loading stiffness. This is illustrated in **Figure 2. 6.** Due to this effect, Mohr Coulomb model is over predicting the soil heave in excavation. To avoid this, it is suggested to use a greater unloading –reloading soil stiffness rather than loading stiffness of soil.

Figure 2.6

Loading unloading Modulus of soil Liong (2014)

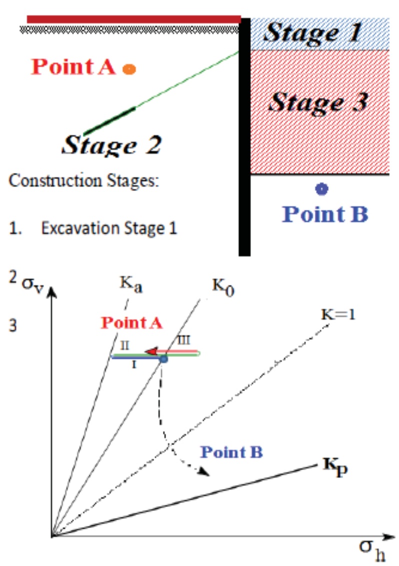


- Material Behavior

Figure 2.7 illustrates the stress paths experienced by soil mass below the excavation level and behind the retaining wall in typical excavation problems.

Figure 2.7

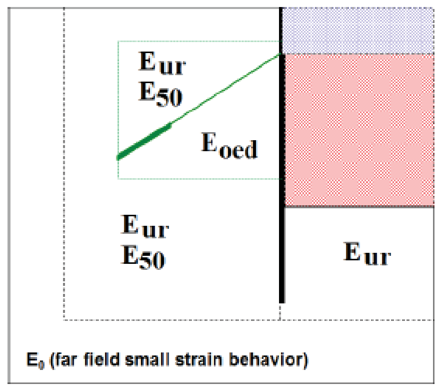
Stress path under a Typical Excavation Problem Liong (2014)



Accordingly, below the excavation level (point B) soil is undergoing unloading in all construction stages. And behind the retaining wall (point A) soil is subjected to several changes. At Stage 1, soil undergoes unloading, Stage 2 it undergoes reloading and Stage 3 again it undergoes unloading. Hence it is very important to use different soil stiffness when dealing with excavation problems. **Figure 2.8** illustrates the predicted behavior of material at various zones in a typical excavation.

Figure 2.8

Expected Material Behavior in Excavation Problem Liong (2014)



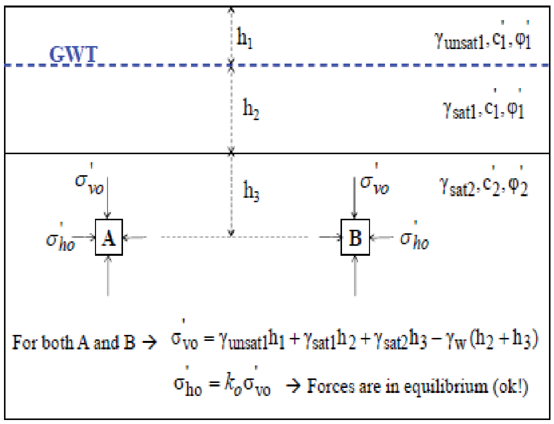
When deep excavations are modeled with Mohr Coulomb model, it uses only one E value for these complex behavior of soil and therefore it provides unrealistic deformations, overestimated bottom heave, and sometimes predicts unrealistic heave of soil behind the wall. However, this behavior can be modeled by using the Hardening Soil Model and can predict more accurate wall deformations, bottom heave, and settlement trough behind the wall. The improved version of this model behavior is Small Strain Hardening soil model which gives far filed small strain behavior with more realistic settlement trough behind the retaining wall.

- Initial Condition

In the initial condition, original stresses within the soil body are calculated. There are two processes to calculate this. The most common method to calculate the initial condition is the use of ko procedure to obtain the initial water pressure and the initial effective stress of the ground. However, this method is valid only when the geometry of the ground surface, ground layers, and the groundwater table are horizontal. Figure 2.9 Illustrated the valid condition for use of ko procedure.

Figure 2.9

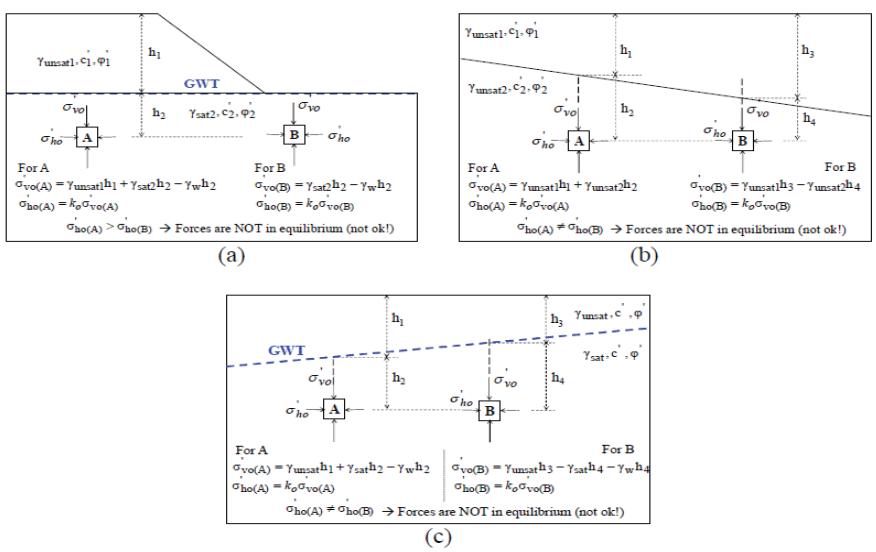
k_o Procedure for Horizontal Geometry, Liong (2014)



Another method that use of gravity loading procedure which have to use when ground surface, the sub soil layers or the ground water level is not horizontal. These conditions are illustrated in **Figure 2.10.**

Figure 2.10

Cases where k_o procedure is inaccurate, Liong (2014)



If the k_o procedure is used for such situations, it will lead to the existence of unbalance forces or non-equilibrium of initial forces within the soil body. This is incorrect and shear stresses should be developed within the soil body to maintain equilibrium. And this is considered when using the gravity loading procedure.

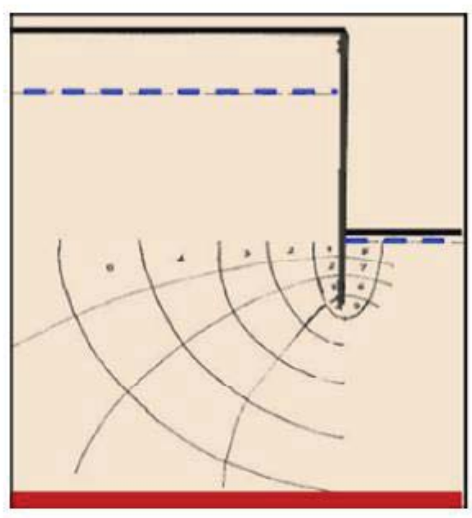
- Ground Water Pressures

The effect of groundwater seepage toward the excavated area should be properly modeled otherwise it will lead to the instability of the earth retaining systems.

First, it is very important to identify whether water can penetrate through the retaining wall or not and whether the retaining wall is installed as a water cut-off system. Even though diaphragm walls or secant piles are considered as impermeable retaining walls, when the toe of the wall is located in a relatively permeable soil layer, during the dewatering and excavation process, water can seep from retained side the walls into the excavation area through the permeable soil layer below the toe of the wall as illustrated in **Figure 2.11**.

Figure 2.11

Impermeable Wall and Bottom Seepage, Liong (2014)



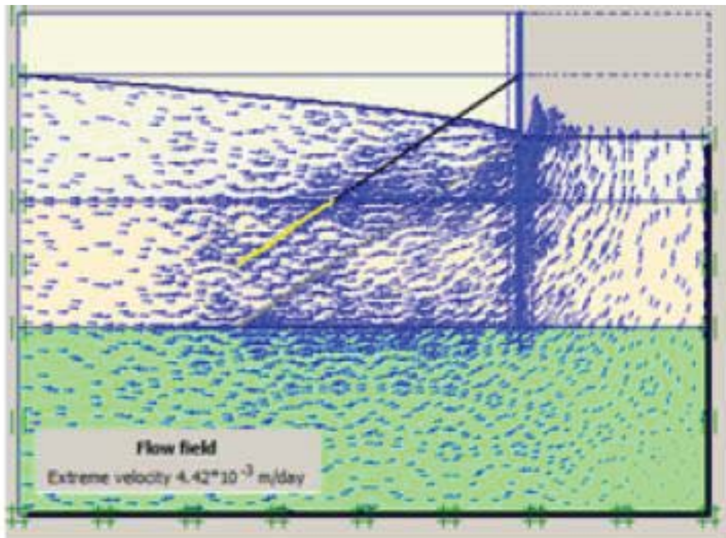
The effect of water seepage, increases the effective overburden pressure in the active side of the walls and reduces the effective stress in the passive side. A heavy seepage can reduce the effective overburden pressure and subsequently may lead to piping and boiling.

To analyze the behavior of groundwater table in PLAXIS 2D the initial step is to activate the interface along the wall. Activation of the interface in soil mode does not mean that the interface of water mode also activated. The interface in soil mode indicates the reduction of the contact friction and allows slippage within the soil and wall. However, in the water mode active interface means the wall is impermeable, and inactive interface means water can pass through the wall.

In the case of permeable walls such as soldier piles, an inactive interface should be used in water mode. In this case, the leaked wall causes deeper water drawdown in unexcavated area as illustrated in **Figure 2.12**. However, PLAXIS do not predict whether there is any erosion of soil particles behind the wall or not.

Figure 2.12

Groundwater Flow through Permeable Wall, Liong (2014)



Another aspect to be considered when analyzing the effect of groundwater table is that it restricts the groundwater flow along the boundary of the system considered. The flow boundary should be closed along the symmetric line and the opposite line the flow boundary should not be closed as it is the side where water comes from. As the water will not flow out to the bottom, the bottom boundary has to be closed.

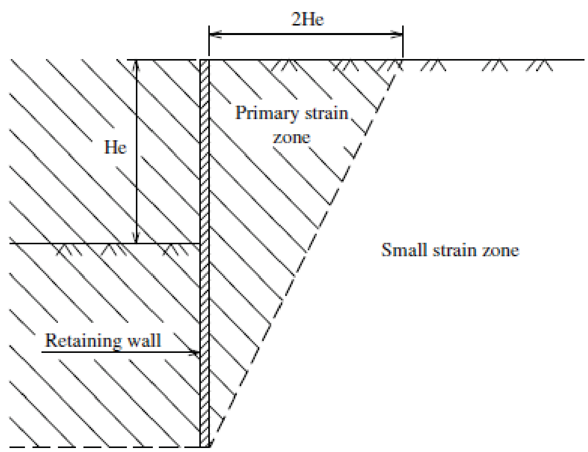
There are some limitations in using 2D models for the analysis of deep-embedded retaining walls. Amarasinghe. et al, (2017) illustrated that determining the maximum wall deflection from 2D plain strain analysis gives an overestimated value. Another point highlighted is that, neglecting the corner effect of excavation. The corner effect is the effect of wall deformation and ground movements being smaller close to the wall corner than around the wall center.

Hsiung and Dao (2015) illustrated that, it is difficult to do a proper prediction of ground surface settlement by using a basic constitutive model of soil. It further highlighted that, to achieve the proper prediction of wall deformation and ground settlement, an advanced constitutive model of soil, which considers the small strain characteristic of soil should be used during analysis. However, input soil parameters of such type of advanced models are not available and they can only be derived from the complex test procedures in comparison to the conventional test method.

To avoid these difficulties, Hsiung and Dao (2015) suggested a simple method to estimate the ground surface settlements during deep excavation using a basic constitutive model of soil. In this method, excavation was divided in to two zones primary strain zone (PSZ) and the small strain zone (SSZ) as shown in **Figure 2.13**. As there is a small strain in SSZ the soil stiffness can be increased compared to the PSZ.

Figure 2.13

Two strain zones of an excavation, Hsiung and Dao (2015)



Ene et.al. (2015) also illustrated that, for the effective design of earth retaining systems, it is required to use advanced soil models. Because, some of the soil parameters defining non-linear behavior, plasticity criterion and their variations are highly sensitive for the results of the analysis. Another point is that soil structure interaction phenomena are also very complex and it is very difficult to control by theoretical modeling. It further elaborated that, using the advanced constitute soil model with uncertain geotechnical parameters may not necessarily give realistic results.

2.6. Back Analysis and Monitoring of Deep Excavation

Kuleaza, et al., (2018) illustrated that, using finite element analysis the displacements are over estimated and following are the reasons highlighted by them.

- Difficulties in determining the strength and compressibility parameters for hard desiccated soils
- Lack of information on the stiffness anisotropy of soil
- Conservatism in selecting the soil parameters to predict excavation performance for design purposes that would not be exceeded in construction

Ene et.al, (2015) further illustrated that, it is difficult to accurately assess the behavior of earth retaining systems during excavation even when design has been done using complex constitutive models. Because, there are uncertainties in soil parameters for the advanced calculations, limitations of the numerical model, and unexpected events that will occur during project execution phase.

Due to this phenomena, it is important to conduct a back analysis while the excavation is going on. Bhatkar, (2017) illustrated that monitoring deformation in the initial stage of excavation provides important information for back analysis. It further illustrated that, the method of back analysis helps to re-assess the already determined soil parameters from laboratory and field investigations. Also selection of soil parameters is an iterative approach to minimize the uncertainties involved by comparing predicted and measured soil behavior during excavation. Particularly those parameters, that are case-sensitive and could be affected by excavation method, excavation size, and any other project specific parameters. Thus on most occasions, this approach could result in considerable cost effectiveness by optimizing the design.

Ene et.al, (2015) illustrated that, monitoring work should include the proper arrangement of a number of monitoring instruments and the location of monitoring instruments. Also, monitoring data must be correlated with the execution stages and they must be clearly stated in the monitoring reports. It further illustrated that, the monitoring results should be compared with the results of the design and also with similar works to validate and calibrate the design model.

Endicott, (2020) illustrated that performance evaluation during the construction process is the best practice to validate assumptions that were made during the design and make necessary adjustments for ground conditions that are encountered.

One concern during the monitoring process is that the deciding when to stop the monitoring works. For that purpose, three limits have been defined as follows.

- Check the design assumptions and the trend of the variation of actual readings obtained from monitoring and decide whether the work can be continued safely. Otherwise, continue to work on keeping the monitoring results under review
- Check the design assumptions with the designer and decide what action needs to be taken
- Stop work and revise the design or method of working, and do any necessary remedial work until it is safe to proceed

The same is defined as Alert, Action, and Alarm Limits (AAA)

As per the Endicott, (2020). The following parameters need to be monitored during deep excavations.

- Lateral deformation of the wall (several points at around 5m horizontal intervals should be established along the wall and coordinates (x,y and z) should be established by surveying techniques to an accuracy of 1mm or better)

- Lateral deformation profile along the depth of wall (several inclinometers shall be installed through the retaining wall or just outside the wall and monitor the lateral deformation profile of the wall regularly)
- Ground settlement (several survey points or settlement plates shall be installed near important structures and the ground settlement shall be monitored regularly)
- Ground water level (several observation wells shall be installed to monitor the groundwater table variation with dewatering. The same can be used for groundwater recharge if necessary)
- Condition of adjacent structures (pre-excavation condition of the adjacent structures shall be established before commencing the excavation and propagation of existing cracks, any structural distress, etc. shall be monitored during and after the excavation)

Instrumentation and monitoring are not only means of control; they can be used positively when the Observational method is adopted.

The Observational Method

The observational method suggested by Endicott, (2020) is a very pragmatic means of risk reduction inherent in ground engineering on account of uncertainty about the type of ground, its properties and how it will behave during engineering works. The observational method is particularly useful for deep excavations where the consequence of unanticipated performance could be of great consequence.

The principle of this method is, to design according to the behavior of the ground and associated structures after estimating the ground condition. Variations from the design assumption of the performance of ground and related structures will be monitored in a systematic manner by instrumentation during the construction period. Then monitoring results are compared with design assumptions and if there is a difference, the designer evaluates the reason. The next step is to decide whether the design or the method of construction needs to be modified to comply with the required standards of safety and protection of property or whether the construction work can be continued with further monitoring. When using the observational method, it is prudent to plan on what performance might be different during construction and what remedial works would then be needed in advance of construction. An additional consideration is whether to have equipment and materials on hand in case they are needed for a short period of time. (Endicott,2020)

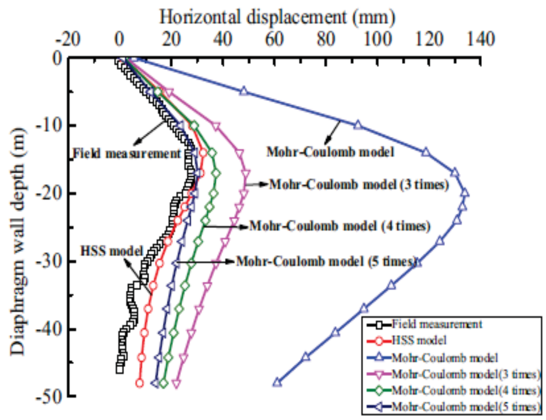
Wu and Chern (2016) described the details of two cases of back analysis conducted for gravel layers on Taiwan. The first analysis was conducted for 17.3m deep excavation supported by a 27 m depth diaphragm wall located at the southern side of Minquan Road in the Xindian District of New Taipei City in Taiwan. As per Wu and Chern (2016) five bracings were used with maximum vertical spacing of 4.4 m. The second analysis was conducted for a 13m deep excavation supported by a 20m depth diaphragm wall located at the eastern side of Zhongzheng Road in the same district. Three bracings were used with a maximum 3.8m vertical spacing within the bracings. PLAXIS 2D software was used for numerical analysis. As the diaphragm wall was socked into the rock, in the model horizontal displacement of the wall bottom was restrained. For case 1, lateral deformation was estimated approximately as 12 mm and in case 2, 23 mm lateral deformations were obtained. Wu and Chern (2016) concluded that, the elastic modulus of the gravel layer in the Xindian district ranged between 7.84MPa and 9.8 MPa, which is in the acceptable range as per the literature.

Another study was carried out by Hsiung, (2009) in Taiwan for a 19.6 m deep excavation supported by a 36m deep diaphragm wall. Five numbers of bracings have been used with a 3.8m maximum vertical spacing within the bracings. It illustrated that predictions based on constant soil elasticity overestimated the lateral deformations below the excavation level. Moreover, Hsiung, (2009) elaborated that surface settlement are also not estimated correctly in both closer or far away from the excavation. It further illustrated that, analysis using small strain parameters provided more consistent predictions. Moreover, Hsiung, (2009) indicated that excavation induced seepages have only a limited effect on vertical displacement.

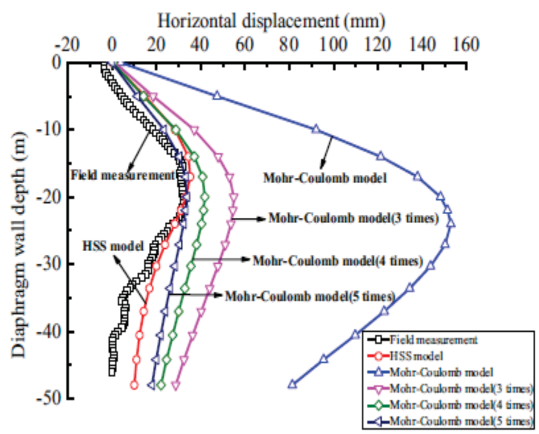
Wang et.al., (2023) has carried out another study. In this research study, 24 m deep excavation supported by a 1m thick 48 m deep diaphragm wall supported by six numbers of lateral supports were considered. Several analyses have been carried out by varying constitute model of soil as Hardening soil model with small strain stiffness (HSS), Mohr-Coulomb parameters and using increased soil stiffness with Mohr-Coulomb soil model. **Figure 2.14** (a), (b) and (c) illustrated the results of the analysis for end of the third, fourth and sixth supports respectively.

Figure 2.14

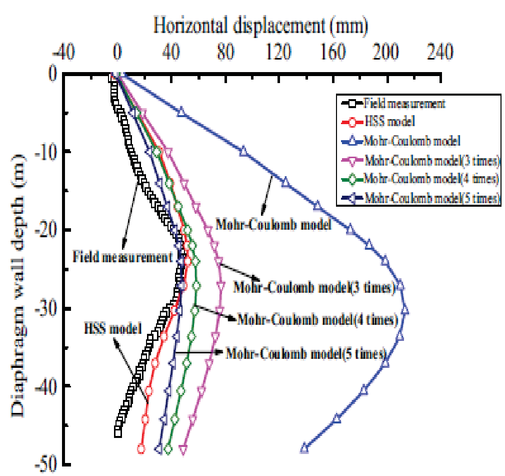
Lateral deformation curves Wang et.al., (2023)



(a)



(b)



(c)

According to the above graphs, it is clear that numerical simulation based on the Hardening soil model has a better agreement with the measured actual deformation. This indicates that, consideration of small strain behavior of soil will give reliable numerical simulation. Also, when only the Mohr-Coulomb model was adopted, the horizontal displacement of the wall was much larger than the monitoring data. These results indicate that Mohr Coulomb model is unsuitable if an experimental modulus is directly adopted (Wang et.al., 2023).

Wang et.al., (2023) further illustrated that, to obtain model parameters for HSS model complicated experiments such as Triaxial consolidated drained shearing test, Loading/unloading shearing test, and Triaxial test at small strain state has been carried out. It further illustrated that, even though Mohr Coulomb model is much simpler and practicable for engineers, it underestimates the stiffness of the soil. Therefore, it is essential to adopt a method to upgrade the modulus.

CHAPTER 03

INITIAL DESIGN AND DETAILS OF THE PROJECT

In this development, the proposal is to construct a structure with eight above ground floors and five basement floors for parking and other service needs. Thus, there is a need to make a 15.75m deep excavation in an urban residential environment. Hence the deformations in the adjacent structures and the disturbance during the construction are to be strictly controlled. The excavation support system needs to be perfectly watertight and of sufficient stiffness. As such, the developers have decided to use a diaphragm wall of width 600mm and support it at three levels during the construction phase. The basement slabs will provide lateral support during the service period. Another requirement is that adequate parking spaces should be provided during the period of construction. As such, a floor slab will be constructed on the ground level temporary support system. This will be supported by king posts founded on piles taken to weathered rock.

3.1. Site Location and General Environment

The site is located in Colombo city in Western Province, Sri Lanka. The main entrance to the site premises is from CWW Kannangara Mawatha. (Refer **Figure 3.1**)

Figure 3.1

Site Location

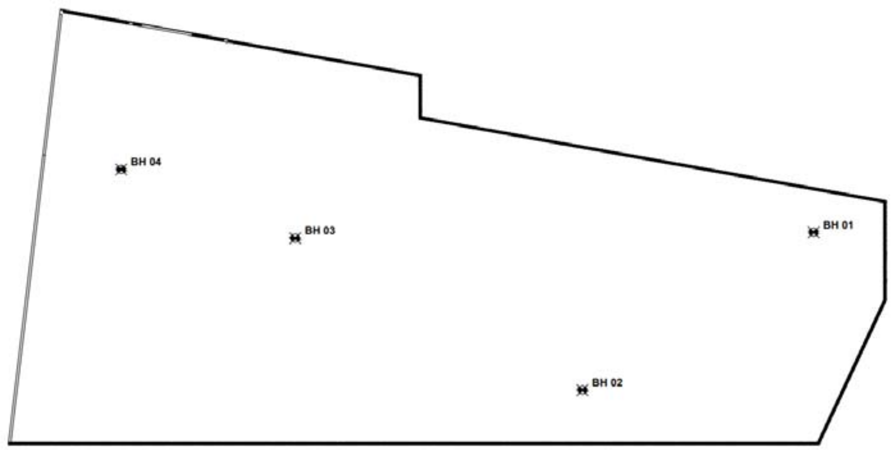
The site is of a flat terrain located in a highly urbanized area. The site is bounded by CWW Kannangara Mawatha in the West and by Ward Place in the North. There are a few multi-storied buildings located in the surrounding area.

3.2. Details of Soil Investigation

Four number of boreholes have been drilled within the site area using the core drilling technique. The borehole locations are illustrated in **Figure 3.2.**

Figure 3.2

Borehole locations



Details of depths of drilling including the depth of Ground Water Level (GWL) are tabulated in **Table 3.1**.

Table 3.1

Details of the depths of drilling

	BH 01	BH 02	BH 03	BH 04
Depth to GWL (m)	1.6	1.4	1.7	1.2
Depth to rock (m)	25.5	25.5	29.0	27.5
Depth of borehole (m)	27.0	28.8	32.0	29.5

Subsurface conditions

By considering the site condition and borehole logs following subsurface layers can be identified as illustrated in **Table 3.2.** The deduced vertical subsurface profile through all boreholes is illustrated in **Appendix A.**

Table 3.2

Summary of subsurface profile

Layer	Description	Depth (m)			
		BH 01	BH 02	BH 03	BH 04
Layer 1	Mainly consists of SAND/ clayey SAND to a depth ranging 2.0 m to 3.0 m.	0.00 – 2.00	0.00 – 3.00	0.00 – 1.00	0.00 – 3.00
Layer 2	Generally, SAND/ silty SAND. SPT N values range between 11 to >50, indicating medium dense to very dense.	2.00 – 9.00	3.00 – 15.00	1.00 – 9.00	3.00 – 10.50
Layer 3a	Organic CLAY/ Organic SILT layer with SPT N value varied between 1 to 21, indicating a very soft to very stiff layer. However, this layer was not encountered at BH 04.	9.00 – 11.10	15.00 – 19.50	9.00 – 10.50	-
Layer 3b	Sandy CLAY encountered at only BH 04 with SPT N equal to 4, which indicated a soft layer.	-	-	-	10.50 – 12.00
Layer 3c	Sandy CLAY layer was encountered at BH 03 and BH 04 with SPT N varies from 7 to 23 indicating a firm to very stiff layer.	-	-	10.50 – 12.0	12.0 - 17.0
Layer 4	Silty SAND/ clayey SAND/ SAND layer was encountered with SPT N value varied between 30 to >50 indicating a dense to very dense layer. However, this layer was not encountered in BH 02.	11.10 – 17.00	-	12.0 – 13.5	17.0 – 21.0
Layer 5a	Organic CLAY layer was only encountered in BH 01 and BH 03 with SPT N varied between 9	17.0 – 19.0	-	13.5 – 15.0	-

	to 15 indicating a firm to stiff layer.				
Layer 5b	Sandy CLAY layer was encountered only in BH 03 with SPT N equal to 31 indicating a hard layer.	-	-	15.0 – 19.0	-
Layer 6a	SAND layer was encountered only in BH 02, with SPT N equal to 4 indicating a loose layer.	-	19.5 – 23.0	-	-
Layer 6b	Clayey SAND/ SAND/ sandy SILT layer was encountered with SPT N varied from 15 to >50 indicating medium dense to very dense layer. However, this layer was not encountered in BH 04.	19.0 – 25.0	23.0 – 25.0	19.0 – 25.0	-
Layer 7	The completely weathered rock layer exhibits soil properties and could be described as a silty SAND/ SAND. SPT N values range generally >50, indicating very dense condition.	25.0 – 25.20	25.0 – 25.0	25.0 – 29.0	21.0 – 27.50
Layer 8	Highly weathered to Moderately weathered BIOTITE GNEISS.	25.5 – 27.0	25.5 – 28.8	29.0 – 32.0	27.5 – 29.5

Details of Rock

Table 3.3 illustrates the details of rock coring in the boreholes.

Table 3.3

Rock Coring Parameters

Borehole No.	Depth (m)	Rock Coring Parameters	
		CR (%)	RQD (%)
BH 01	25.5 – 27.0	96	49
BH 02	25.5 – 26.8	97	00
	26.8 – 27.8	90	35
	27.8 – 28.8	100	85
BH 03	29.0 – 30.0	63	00
	30.0 – 31.0	70	44
	31.0 – 32.0	100	57
BH 04	27.5 – 28.5	40	00
	28.5 – 29.5	100	65

Three numbers of rock samples have been tested for their Uniaxial Compressive Strength (UCS) and results are given in **Table 3.4**.

Table 3.4

UCS values of rock samples

Borehole No.	Depth (m)	UCS (N/mm^2)
BH 02	27.80 – 27.95	24.34
BH 03	31.45 – 31.60	22.24
BH 04	28.75 – 28.90	97.08

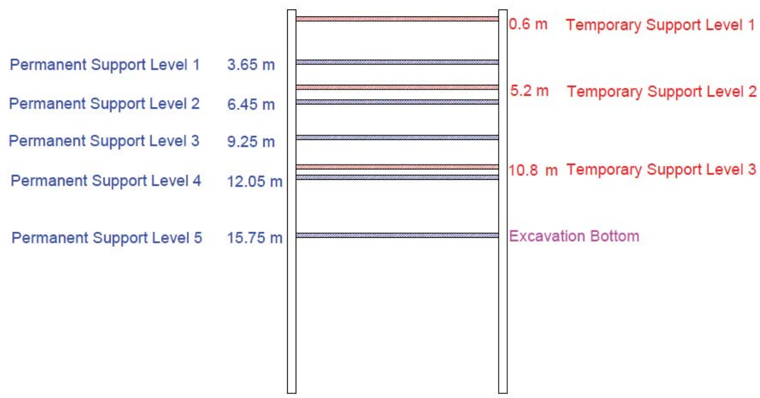
3.3. Details of the Earth Retaining System

Initial Design

A 0.6 m thick diaphragm wall was used as the earth retaining system to retain the 15.75 m deep excavation. The embedded depth was nearly 11m. While excavation continued, three concrete temporary supports were used at three levels to provide lateral support to the wall. Five floors act as permanent supports during the operational stage of the building. **Figure 3.3** illustrates the levels of temporary and permanent supports.

Figure 3.3

Levels of temporary and permanent supports



Details of Temporary Support System

G30 Concrete slab has been used as the first level of temporary support. A slab was provided to ensure adequate spaces for parking for the existing shopping mall during the construction period. G30 Concrete beams have been used as second and third levels of temporary supports as well. **Table 3.5** elaborates the section details of the temporary supports and **Figure 3.4**, **Figure 3.5** and **Figure 3.6** illustrate the arrangements of temporary supports of three levels respectively. Openings were left in the top slab for the transportation of excavated soils (**Figure 3.4**).

Table 3.5

Details of Temporary Supports

Temporary Support Level	Type of Temporary support	Dimensions (mm)
Level 1	Concrete Slab	250 thick
Level 2	Concrete Beam ZC 3	1000 x 800 (w x d)
	Concrete Beam ZC 4	800 x 800
Level 3	Concrete Beam ZC 5	1200 x 1000
	Concrete Beam ZC 6	800 x 800

Figure 3.4

Temporary Support Arrangement of Level 1

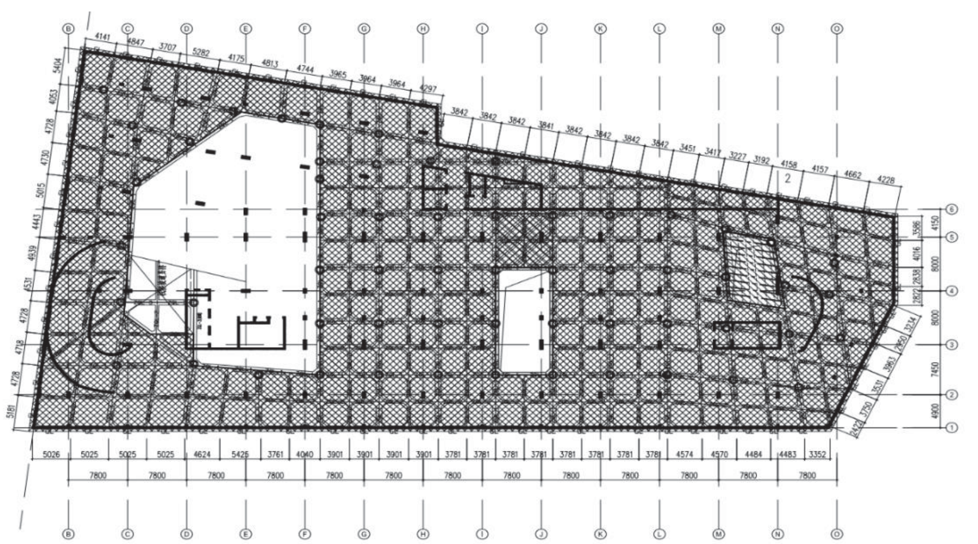


Figure 3.5

Temporary Support Arrangement of Level 2

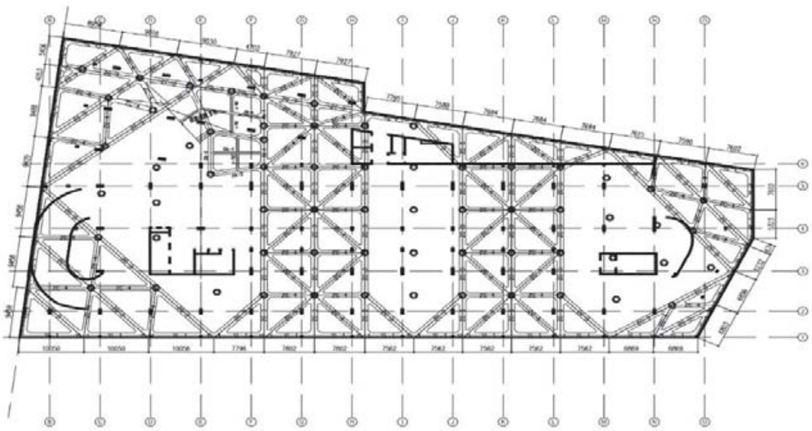


Figure 3.6

Temporary Support Arrangement of Level 3

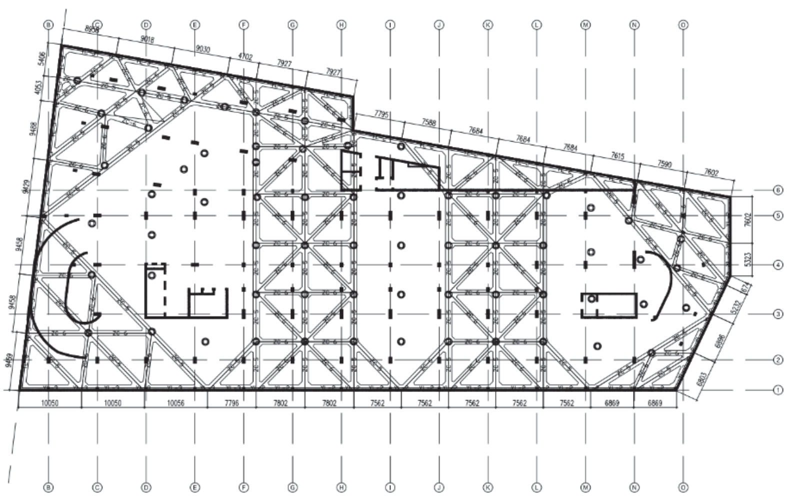


Figure 3.7

Temporary Supports at Site

Temporary Supports

Details of Permanent Supports

G30 concrete slabs have used as permanent supports of the diaphragm wall. **Table 3.6** elaborates the details of the permanent supports.

Table 3.6

Details of Permanent Supports

Permanent Support Level	Type of Permanent Support	Dimensions (mm)
Level 1	Concrete Slab	150, 175 & 200 thick
Level 2	Concrete Slab	150, 175 & 200 thick
Level 3	Concrete Slab	150, 175 & 200 thick
Level 4	Concrete Slab	150, 175 & 200 thick
Level 5	Concrete Slab	1000 thick
	Concrete Beam	2000 x 2000

Details of Construction Sequence

At the initial stage, the total area was divided into two sections Area A and Area B. After that both sections were further divided into two sections and final areas are Area A1, Area A2, Area B1, and Area B2. **Figure 3.8** elaborates on the shapes of zones during excavation. Excavation was done as zone-wise as per the requirements of the client. The construction stages adopted are presented in **Table 3.7**.

Figure 3.8

Zones during excavation

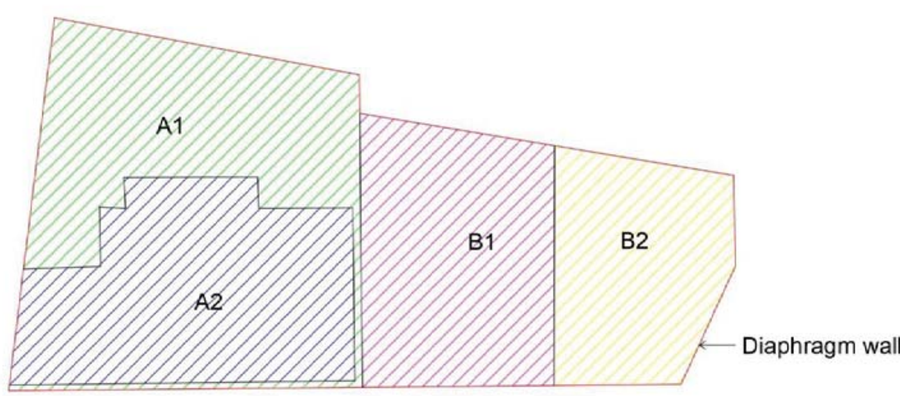


Table 3.7

Construction sequence

Stage	**Construction Activity**
Stage 1	Construction of Diaphragm wall
Stage 2	Excavate up to 1.1 m and installing the supporting roof slab at a depth of 0.6m -Temporary support 1
Stage 3	Excavation to 5.6m after installing the temporary support 1
Stage 4	Excavation to 11.3m after installing temporary support 2 at a depth of 5.2m
Stage 5	Excavate up to 15.75 m after installation of temporary support 3 at a depth of 10.8m
Stage 6	Construction of Basements 4 and 5 and removal of temporary support 3
Stage 7	Construction of Basements 3 and 2 and removal of temporary support 2
Stage 8	Construction of Basement 1and removal of temporary support 1

3.4. Monitoring

To identify the behavior of the earth retaining system during the excavation, it was planned to monitor the followings.

- Lateral deformation of diaphragm wall using inclinometers
- Fluctuation of groundwater table by observation wells
- Vertical settlement of adjacent lands using settlement plates
- Stress in the elements of the lateral support system using strain gauges

Lateral deformation was to be monitored using 6 numbers of inclinometers around the excavation while 8 numbers of observation holes were used to monitor the groundwater level fluctuations. The locations of the instruments are presented in the proceeding chapters.

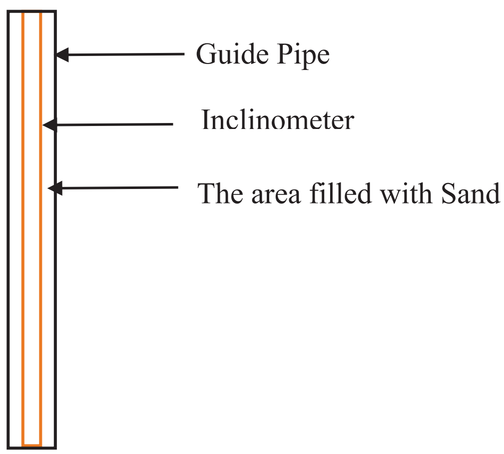
However, only the inclinometer readings were available at the time of this research study.

Inclinometers

Inclinometers were installed in holes drilled in the diaphragm wall. This had been done at a later stage when the excavation proceeded to a depth of 11.3 m. The inclinometer had been terminated at the level of the excavation without extending to the full depth of embedment of the diaphragm wall. Also, during the process of installation, the space between the inclinometer tube and the drilled hole had been filled with sand instead of grouting.

Figure 3.9

Details of Inclinometer



Accounting for the variability of the subsoil conditions the numerical simulation of the excavation was conducted for four different idealized sections. Inclinometers related to the selected four sections for the analysis can be tabulated as follows.

Table 3.8

Inclinometer Related to Analyzed Section

Section	Inclinometer
Section 1	TT 6
Section 2	TT 4
Section 3	TT 2
Section 4	TT 3

The inclinometer readings were available up to the final excavation level. Available monitoring data are plotted for each section.

Section 1

The construction sequence for the excavation process of Section 1 can be tabulated as follows.

Table 3.9

Construction Sequence for Section 1

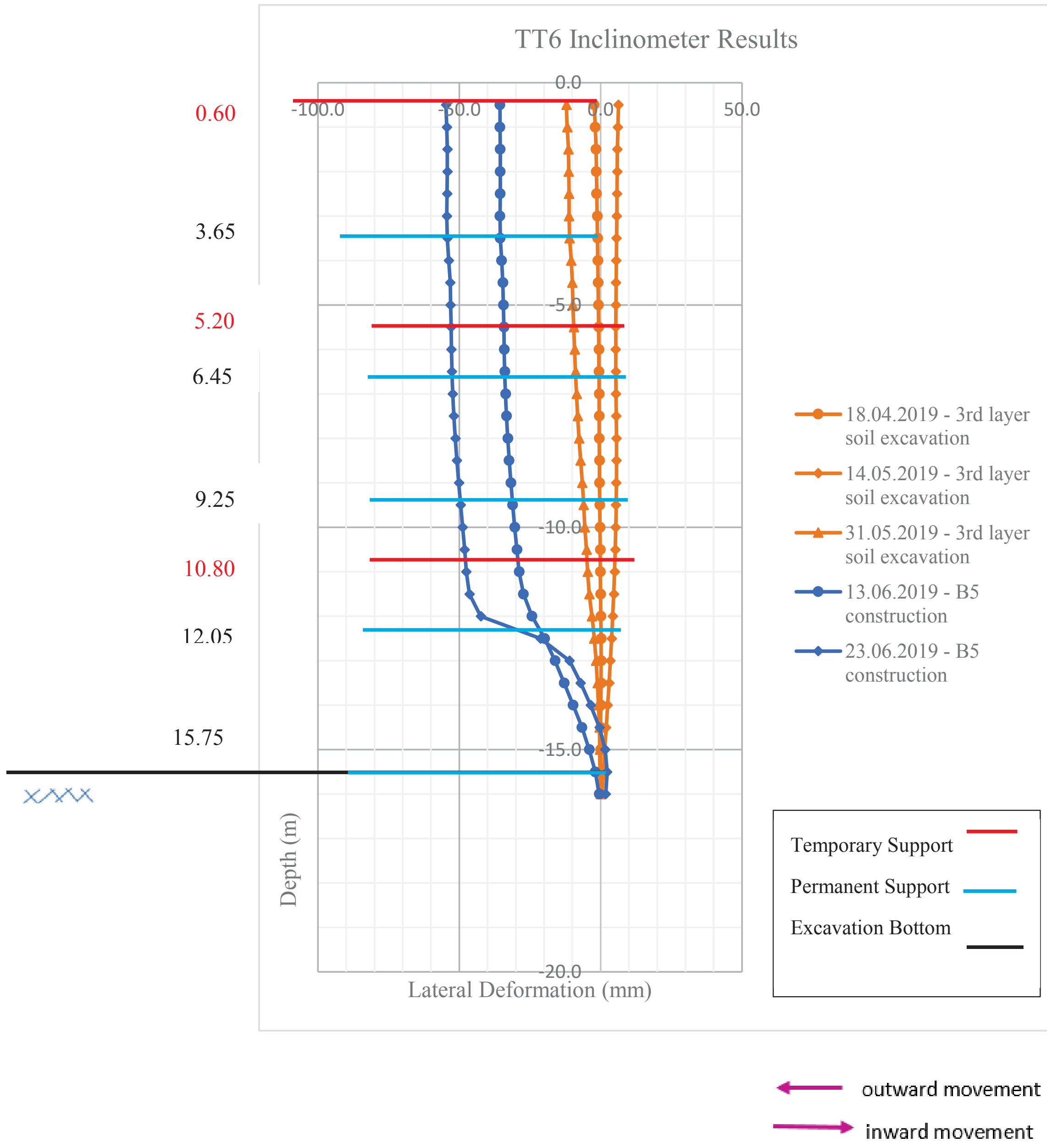
Construction Activity	**Period**
1st layer temporary support	15.08.2018-10.12.2018
1st layer soil excavation	27.11.2018-20.12.2018
2nd layer temporary support	15.12.2018-12.01.2019
2nd layer soil excavation	20.01.2019-16.02.2019
3rd layer temporary support	10.02.2019-11.03.2019
3rd layer soil excavation	28.03.2019-05.06.2019
B5 Construction	07.05.2019-15.07.2019
B4 Construction	14.08.2019-30.08.2019
3rd layer temporary support removes	13.10.2019-26.10.2019
B3 Construction	30.11.2019-27.12.2019
B2 Construction	13.12.2019-09.01.2020
2nd layer of temporary support removes	03.02.2020-13.02.2020
B1 Construction	17.02.2020-11.06.2020
1st layer of temporary support removes	12.05.2021-01.09.2021

The behavior of the diaphragm wall for Section 1 was monitored using TT6 inclinometer. It was informed that, this inclinometer was broken and hence only a few data are available.

Inclinometer graphs not follow an exact pattern. An outward movement and inward movement of graphs with the time is indicated. Also, the shape of the graph is not complying with that reported in the literature. The monitoring graphs in Wang (2023), elaborated that maximum deformation occurs below the level of the bottommost lateral support. As described previously in this chapter, the reason for the outward and inward movement of the inclinometer may be not filling the space between the inclinometer pipe and guide pipe properly by grout. Wang (2023), further illustrated that, the inclinometer has to be installed beyond the excavation depth until fixed toe condition is achieved, but that was not done in this project.

Figure 3.10

Inclinometer Readinas



Section 2

The construction sequence for the excavation process of Section 2 is summarized in **Table 3.10**.

Table 3.10

Construction Sequence for Section 2

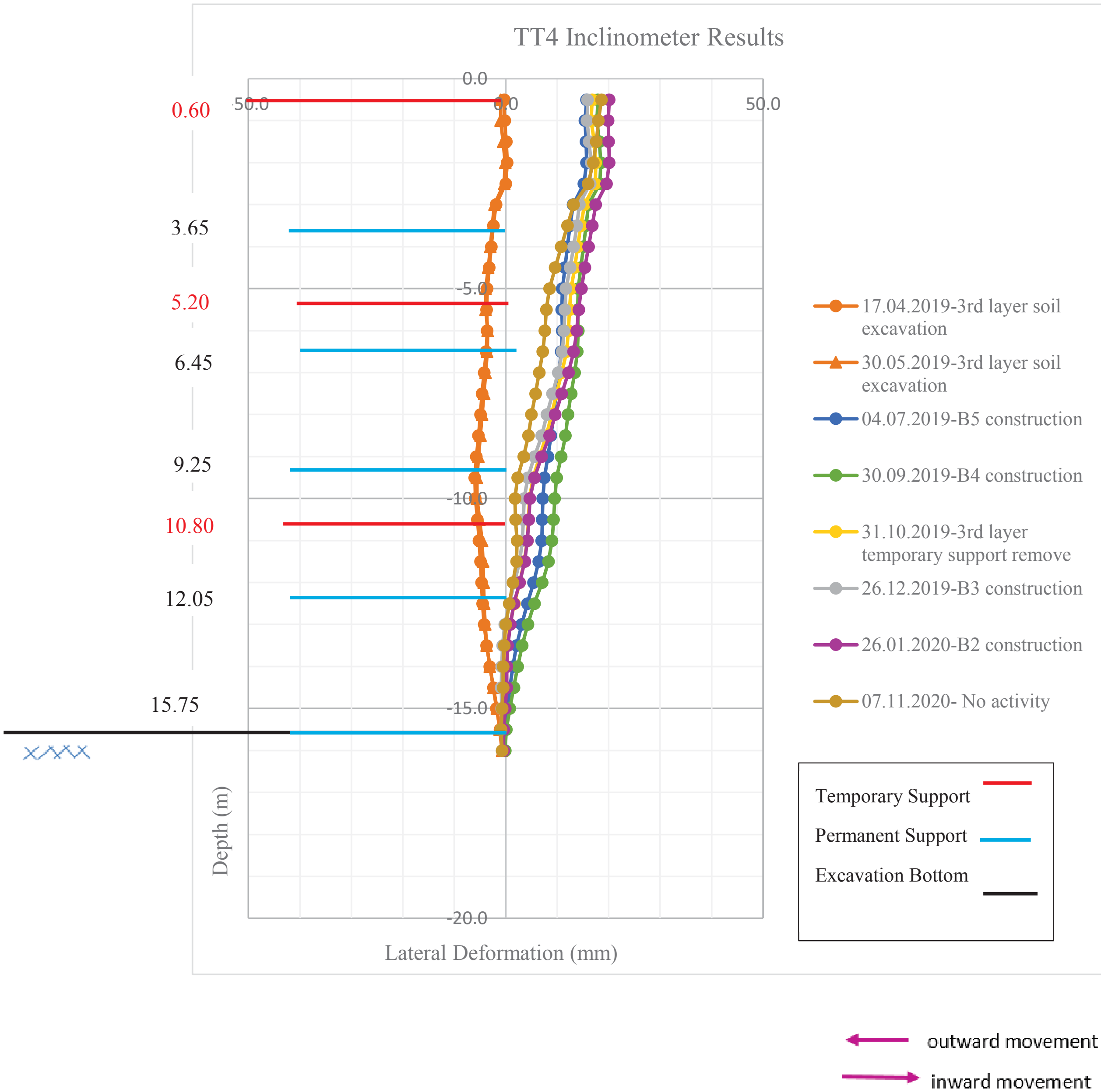
Construction Activity	**Period**
1^{st} layer inner support	26.04.2018-30.06.2018
1^{st} layer soil excavation	27.11.2018-20.12.2018
2^{nd} layer inner support	15.12.2018-12.01.2019
2^{nd} layer soil excavation	20.01.2019-16.02.2019
3^{rd} layer inner support	10.02.2019-11.03.2019
3^{rd} layer soil excavation	28.03.2019-30.05.2019
B5 Construction	10.06.2019-29.07.2019
B4 Construction	09.09.2019-30.09.2019
3^{rd} layer temporary support removes	16.10.2019-11.11.2019
B3 Construction	27.112019-09.01.2020
B2 Construction	06.01.2019-31.01.2019
2^{nd} layer temporary support removes	06.02.2020-16.02.2020
B1 Construction	Completed on 11.07.2020
1^{st} layer temporary support removes	20.04.2021-06.06.2021

The relevant inclinometer for Section 2 is TT 4. The available monitoring data for the TT4 inclinometer was plotted in one graph in **Figure 3.11**.

Although this inclinometer shows both inward and outward movement like previous inclinometer, inward movement is larger when compared with outward movement in here. Also the pattern is not complying with that reported in the literature. Here also, the reason for these differences may be the inappropriate method adopted when installing inclinometer.

Figure 3.11

Inclinometer Readings – Section 2



Section 3

The construction sequence for the excavation process of Section 3 is summarized in **Table 3.11**.

Table 3.11

Construction Sequence for Section 3

	Construction Activity	**Period**
1	1st layer inner support	26.04.2018-30.06.2018
2	1st layer soil excavation	10.12.2018-15.01.2019
3	2nd layer inner support	10.01.2019-28.01.2019
4	2nd layer soil excavation	16.02.2019-24.03.2019
5	3rd layer inner support	14.03.2019-06.04.2019
6	3rd layer soil excavation	06.05.2019-29.07.2019
7	B5 Construction	10.07.2019-12.09.2019
8	B4 Construction	30.09.2019-22.10.2019
9	3rd layer temporary support removes	24.10.2019-06.11.2019
10	B3 Construction	05.11.2019-26.11.2019
11	B2 Construction	27.11.2019-18.12.2019
12	2nd layer temporary support removes	02.01.2020-02.02.2020
13	B1 Construction	25.01.2020-19.02.2020
14	1st layer temporary support removes	05.03.2020-29.05.2020

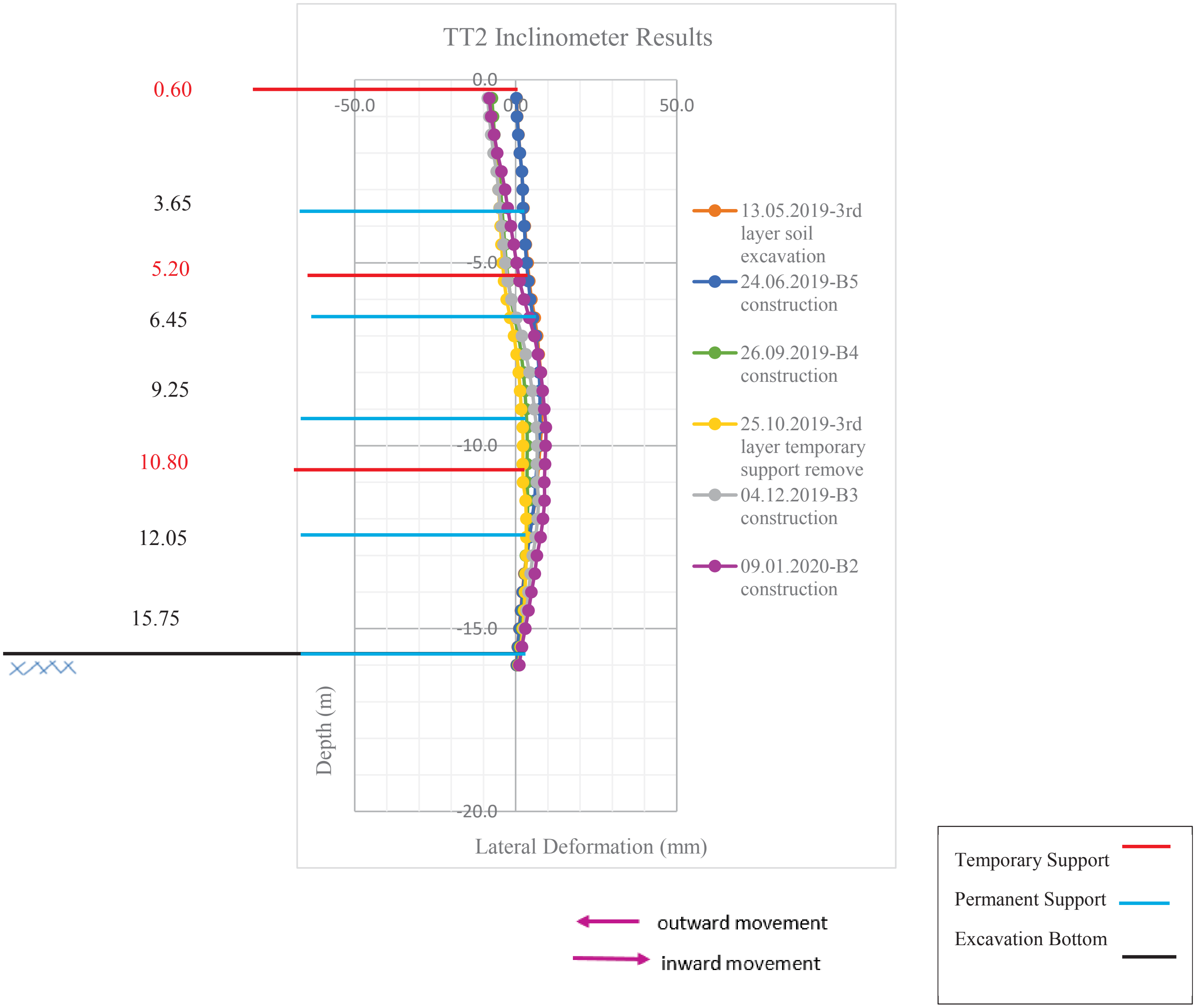
The relevant inclinometer for Section 3 is TT 2. The available monitoring data for the TT2 inclinometer has plotted in one graph as elaborated in **Figure 3.12**.

In this inclinometer also, the inward and outward movement of inclinometers are indicating. However, there is considerable inward movement of graphs at the mid depth of the graph, which was not shown in other two sections.

As previously mentioned, the most probable reason for this is inappropriate method adopted to install inclinometers.

Figure 3.12

Inclinometer Readings – Section 3



Section 4

The construction sequence for the excavation process of Section 4 are summarized in **Table 3.12**.

Table 3.12

Construction Sequence for Section 4

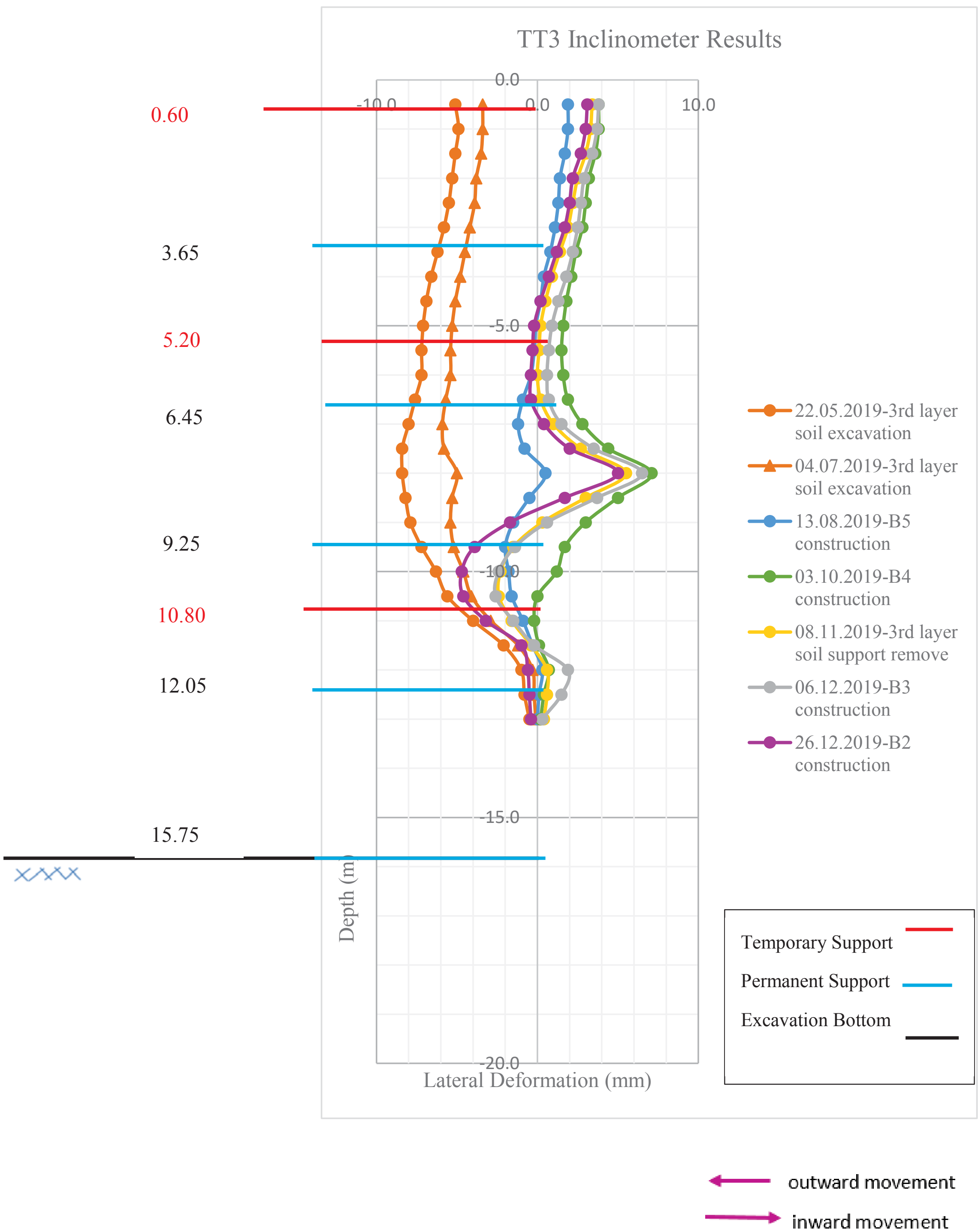
	Construction Activity	**Period**
1	1st layer inner support	15.08.2018-10.12.2018
2	1st layer soil excavation	10.12.2018-15.01.2019
3	2nd layer inner support	10.01.2019-28.01.2019
4	2nd layer soil excavation	16.02.2019-24.03.2019
5	3rd layer inner support	14.03.2019-06.04.2019
6	3rd layer soil excavation	20.05.2019-29.07.2019
7	B5 Construction	05.07.2019-20.08.2019
8	B4 Construction	17.09.2019-14.10.2019
9	3rd layer temporary support removes	30.10.2019-15.11.2019
10	B3 Construction	22.11.2019-10.12.2019
11	B2 Construction	07.12.2019-30.12.2019
12	2nd layer temporary support removes	02.01.2020-18.01.2020
13	B1 Construction	01.02.2020-01.03.2020
14	1st layer temporary support removes	04.11.2020-31.12.2020

The relevant inclinometer for Section 4 is TT 3. The available monitoring data for the TT3 inclinometer has plotted in one graph as elaborated in **Figure 3.12.**

In this section unusual behavior of inclinometer is observed after the 04.07.2019. It seems like some obstacle has stuck in the inclinometer pipe during monitoring period. Also, this inclinometer has not installed up to excavation bottom.

Figure 3.13

Inclinometer Readings – Section 4



CHAPTER 04

IDEALIZATION PROCESS

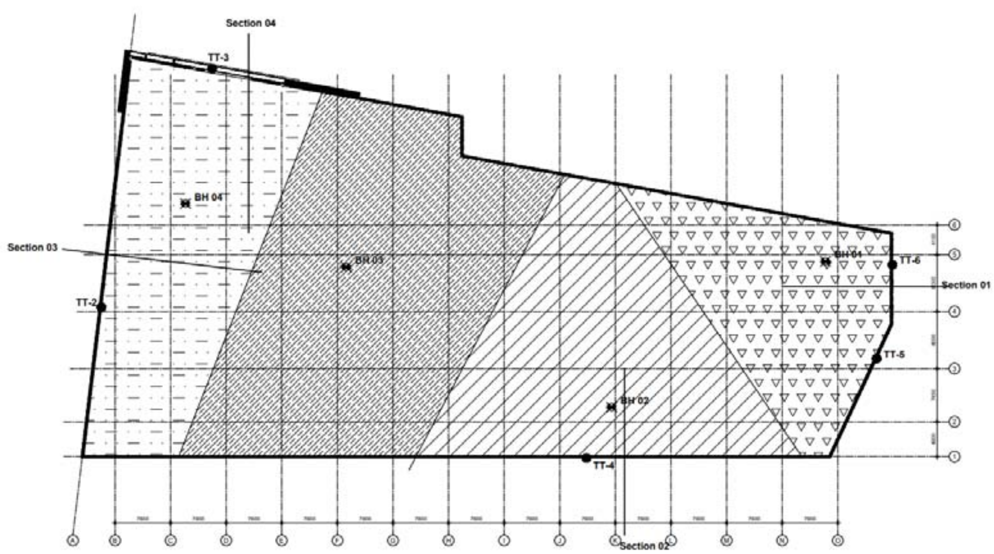
The idealization process which was used to select the sections to be analyzed and, idealize the subsoil profile, diaphragm wall, permanent supports, and temporary supports is discussed in this chapter.

4.1. Selection of Sections

Total excavated area was divided into four zones considering the borehole locations and the variations in the subsoil profiles. Four sections were selected for the analysis covering from each of the four sides of the excavation. Further, the sections were selected closer to inclinometers to compare with monitoring data. **Figure 4.1** illustrates the selected sections.

Figure 4.1

Selection of sections



The selected sections for the analysis, the relevant borehole and the relevant inclinometer are presented in **Table 4.1.**

Table 4.1

Selected Sections to Analysis

Section	Borehole	Inclinometer
Section 1	BH 01	TT 6
Section 2	BH 02	TT 4
Section 3	BH 04	TT 2
Section 4	BH 04	TT 3

4.2. Idealization of Soil Properties

Mohr Coulomb model was used to model the sub soil as it is appropriate for the nature of the soils encountered. As there was no detailed laboratory testing of soils, the necessary model parameters were deduced from establishes correlations and local experiences on similar soils. The following properties have to be decided for the soil layers.

1. General Properties (γ_{sat} , γ_{unsat})
2. Strength Properties (c', φ', ψ)
3. Stiffness Properties (E' , v'(nu))
4. Flow Parameters (k_x, k_y)
5. R_{inter}

The subsoil profile across the four boreholes in presented in **Appendix A**.

<u>Soil Strength Parameters of Soil Layers</u>

The energy method of SPT correction (Bowles, 1996) was used in the process of estimating the strength parameters of soil layers. The energy method of SPT correction uses the following relationship to determine the N'_{70} from the field SPT blow counts (N_{Field}):

$$N'_{70} = N_{Field} C_N \eta_1 \eta_2 \eta_3 \eta_4$$

Where

$$C_N = \sqrt{\frac{95.76}{p'_o}} \qquad \eta_1 = \frac{E_r}{70}$$

$P_o^{/}$ = Effective overburden pressure at the test level

E_r = Efficiency of the hammer used (taken as 55%)

η_i = Modification factors (Bowles, 1996)

The estimated $N^{/}_{70}$ together with the particle size can be used to estimate the soil strength parameters at respective depths. **Table 4.2**, **Table 4.3** and **Table 4.4** present the estimated soil strength parameters from SPT for BH 01, BH 02, and BH 04 respectively. During the estimation of soil strength parameters for each layer, the corrected SPT values at each depth were estimated and then the relevant average soil strength parameters were deduced.

Table 4.2

Shear strength properties of BH 01

Layers	**Description**	**γ_{wet} kN/m^3**	**γ_{sat} kN/m^3**	**Ø°**	**C kN/m^2**
1	V.loose to M.dense SAND/Clayey SAND	16	17	31	0.01
2	M.dense to V.dense SAND/Silty SAND	17	18	28	7
3a	Organic CLAY/Organic SILT	15	16	24	3
5a	Dense to V.dense Silty SAND/ Clayey SAND/SAND	17	18	29	8
6b	Organic CLAY	15	16	25	5
7	M.dense to V.dense Clayey SAND/SAND/Sandy SILT	17	18	28	7
8	CWR	20	22	28	8

Table 4.3

Shear strength properties of BH 02

Layers	Description	γ_{wet} kN/m³	γ_{sat} kN/m³	Ø°	C kN/m²
1	V.loose to M.dense SAND/Clayey SAND	16	17	26	0.01
2	M.dense to V.dense SAND/Silty SAND	17	18	29	8
3a	Organic CLAY/Organic SILT	15	16	24	2
6a	Loose SAND	16	17	26	0.01
6b	M.dense to V.dense Clayey SAND/SAND/Sandy SILT	17	18	25	5
7	CWR	20	22	28	8

Table 4.4

Shear strength properties of BH 04

Layers	Description	γ_{wet} kN/m³	γ_{sat} kN/m³	Ø°	C kN/m²
1	V.loose to M.dense SAND/Clayey SAND	16	17	26	0.01
2	M.dense to V.dense SAND/Silty SAND	17	18	29	8
3b	Soft Sandy CLAY	15	16	24	2
3c	Firm to V.Stiff Sandy CLAY	16	17	26	0.01
4a	Dense to V.dense Silty SAND/ Clayey SAND/SAND	17	18	25	5
7	CWR	20	22	28	8

Soil Stiffness Parameters of Soil Layers

The drained Stiffness parameters of soil layers were calculated using the following empirical equations. (Bowls, 1997)

Sand Es = 320 (N55 + 15)

Clay Es = 300 (N55 + 6)

Stiffness parameters deduced for the soil layers in respective boreholes BH 01, BH 02 and BH 04 are given in **Table 4.5**, **Table 4.6,** and **Table 4.7**.

Table 4.5
Stiffness parameters of BH 01

Layers	Description	E' (kPa)	v'(nu)
1	V.loose to M.dense SAND/Clayey SAND	7000	0.3
2	M.dense to V.dense SAND/Silty SAND	13100	0.3
3a	Organic CLAY/Organic SILT	2400	0.35
5a	Dense to V.dense Silty SAND/ Clayey SAND/SAND	17900	0.3
6b	Organic CLAY	6300	0.35
7	M.dense to V.dense Clayey SAND/SAND/Sandy SILT	18500	0.3
8	CWR	20800	0.3

Table 4.6
Stiffness parameters of BH 02

Layers	Description	E' (kPa)	v'(nu)
1	V.loose to M.dense SAND/Clayey SAND	5100	0.3
2	M.dense to V.dense SAND/Silty SAND	16300	0.3
3a	Organic CLAY/Organic SILT	3600	0.35
6a	Loose SAND	6000	0.3
6b	M.dense to V.dense Clayey SAND/SAND/Sandy SILT	11800	0.3
7	CWR	20800	0.3

Table 4.7
Stiffness parameters of BH 04

Layers	Description	E' (kPa)	v'(nu)
1	V.loose to M.dense SAND/Clayey SAND	10200	0.3
2	M.dense to V.dense SAND/Silty SAND	18500	0.3
3b	Soft Sandy CLAY	3000	0.35
3c	Firm to V.Stiff Sandy CLAY	7500	0.35
4a	Dense to V.dense Silty SAND/ Clayey SAND/SAND	20800	0.3
7	CWR	20800	0.3

Flow Parameters of Soil Layers

As per the geotechnical investigation report vertical permeability of some soil layers have been obtained using filed permeability tests. Based on the availability test results, vertical permeability of other soil layers was also deduced. In this regard, it the absence of detained data the horizontal and vertical permeability values were assumed to be the same. **Table 4.8**, **Table 4.9** and **Table 4.10** presents the permeability values.

Table 4.8

Flow parameters of BH 01

Layers	Description	$k_x = k_y$ (ms^{-1})
1	V.loose to M.dense SAND/Clayey SAND	1.00E-05
2	M.dense to V.dense SAND/Silty SAND	1.10E-05
3a	Organic CLAY/Organic SILT	2.26E-06
5a	Dense to V.dense Silty SAND/ Clayey SAND/SAND	1.00E-06
6b	Organic CLAY	1.00E-08
7	M.dense to V.dense Clayey SAND/SAND/Sandy SILT	1.00E-06
8	CWR	1.00E-08

Table 4.9

Flow parameters of BH 02

Layers	Description	$k_x = k_y$ (ms^{-1})
1	V.loose to M.dense SAND/Clayey SAND	1.00E-05
2	M.dense to V.dense SAND/Silty SAND	1.10E-05
3a	Organic CLAY/Organic SILT	2.26E-06
6a	Loose SAND	1.00E-05
6b	M.dense to V.dense Clayey SAND/SAND/Sandy SILT	1.00E-06
7	CWR	1.00E-08

Table 4.10

Flow parameters of BH 04

Layers	Description	$k_x = k_y$ (ms^{-1})
1	V.loose to M.dense SAND/Clayey SAND	1.00E-05
2	M.dense to V.dense SAND/Silty SAND	3.46E-06
3b	Soft Sandy CLAY	1.00E-07
3c	Firm to V.Stiff Sandy CLAY	2.90E-06
4a	Dense to V.dense Silty SAND/ Clayey SAND/SAND	1.00E-06
7	CWR	1.00E-08

R_{inter}

Shear strength parameters along the soil-wall interface were reduced by introducing an interface shear strength reduction factor. 0.67 was taken as the R_{inter} value for all soil layers.

4.3. Idealization of Rock Properties

Linear elastic behavior was assumed for rock. Accordingly, for the finite element analysis following rock properties have to be decided.

1. General Properties (γ_{sat} , γ_{unsat})
2. Stiffness Properties (E' , v'(nu))
3. Flow Parameters (k_x, k_y)

Stiffness Parameters of Rock Layers

Elastic modulus of rock can be estimated using the following equation as illustrated in Zhang and Einstein (2004).

$$\frac{E_m}{E_r} = 10^{0.0186RQD-1.91}$$

Where,

E_m is elastic modulus of rock mass and

E_r is elastic modulus of intact rock.

RQD is the Rock Quality Designation of rock

Value of E_r is considered as 61.1 GPa according to the Zhang and Einstein (2004). Calculated E values for rock is shown in following tables. When calculating the E value of rock, separate E values of each rock sample were calculated and the average value was used as an E value of the rock layer for the analysis.

Table 4.11

Rock Properties of BH

	E (kPa)	**v'(nu)**	**k_x/ k_y (ms^{-1})**	**γ_{wet}/ γ_{sat}**
BH 1	6.13E06	0.15	1.00E-08	25
BH 2	10.92E06	0.15	1.00E-08	25
BH 4	6.46E06	0.15	1.00E-08	25

4.4. Idealization of Temporary Supports and Permeant Supports of Earth Retaining System

The temporary supports and permeant supports were idealized as fixed-end supports. The section is required to be perpendicular to the diaphragm wall to satisfy the basic requirements of plain strain conditions in finite element modelling.

As both permanent supports and temporary supports at the site are not perpendicular to the diaphragm wall, the following modifications were done during the idealization of the actual condition to the finite element model.

Elasticity and poison ratio of G30 concrete was taken from EN 1992-1-1 Euro code 2. Accordingly, E value of G30 concrete is 31 Gpa and poison ratio is 0.2.

Method 1

First the angle between the selected section and actual support was calculated and then the perpendicular effect of the stiffness properties of support sections were calculated and used for the analysis.

Figure 4.2 and **Table 4.12** illustrate the details of the idealization of section 1 using this method.

Figure 4.2
Idealization Method 1

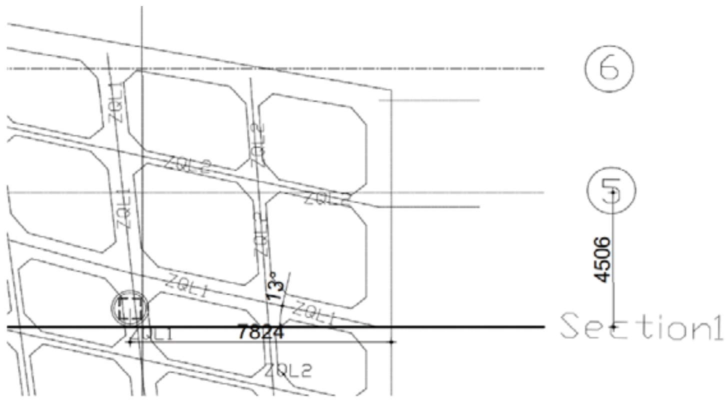


Table 4.12
Idealized Properties of Temporary Support

Section Detail	Dimensions (m)		Angel to Grid 5 (Θ)	Concrete Grade (Design)	E	A	EA	EA cos (Θ)
	depth	width						
Concrete Beam (ZQL 1)	1.2	0.8	13	G 30	3.10E+07	0.96	2.98E+07	2.90E+07

Method 2

When there are two supports at different angles at the same point, the stiffness of anchors was calculated separately as previously, and net effect of the stiffness was taken for the analysis.

Figure 4.3 and **Table 4.13** illustrate the details of the idealization of section 1 using this method.

Figure 4.3

Idealization Method 2

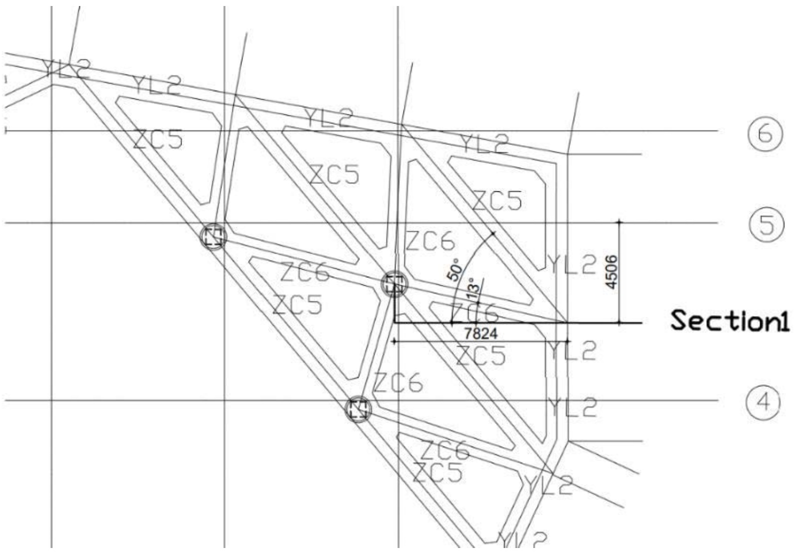


Table 4.13

Idealized Properties of Temporary Support

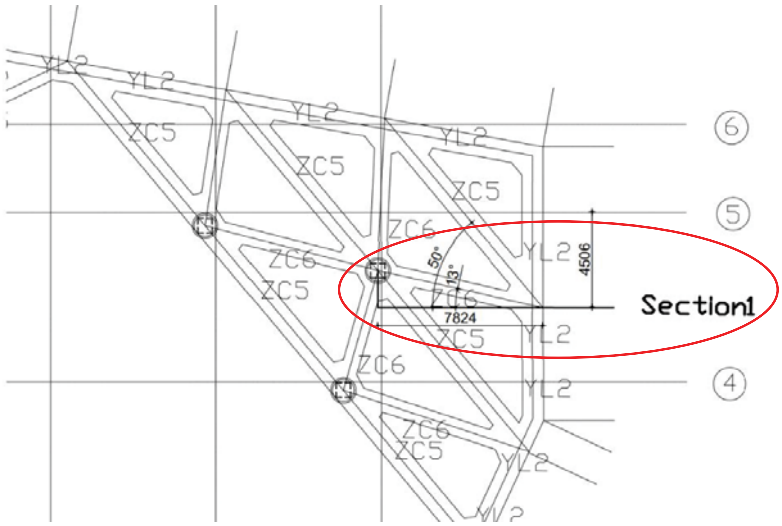
Section Detail	Dimension (m)		Angel to Grid 5 (Ɵ)	Concrete Grade (Design)	E	A	EA	EA cos (Ɵ)	$(EA)_{Net}$
	depth	width							
Concrete Beam (ZC 6)	0.8	0.8	13	G 30	3.10E+07	0.64	1.98E+07	1.93E+07	4.33E+07
Concrete Beam (ZC 5)	1	1.2	50	G 30	3.10E+07	1.20	3.72E+07	2.39E+07	

Method 3

The perpendicular distance to the kingpost from the diaphragm wall was considered as the distance to the king post as illustrated in **Figure 4.4**.

Figure 4.4

Idealization Method 3



Idealized properties of temporary supports at Section 1, section 2, section 3 and section 4 are elaborated in **Table 4.14, Table 4.16, Table 4.18** and **Table 4.20** respectively.

Idealized properties of permeant support at section 1, section 2, section 3 and section 4 are elaborated in **Table 4.15, Table 4.17, Table 4.19** and **Table 4.21** respectively.

Table 4.14

Properties of temporary support at section 1

Temporary Supports	**Depth (m)**	**EA**	**EI**	**ν**	**w**	**Horizontal Spacing (m)**	**Distance to King Post (m)**
First Inner Support	0.6	2.90E+07	3.57E+06	0.2	30.00	3.427	7.824
Second Inner Support	5.2	3.53E+07	1.88E+06	0.2	20.00	6.460	7.824
Third Inner Support	10.8	4.33E+07	3.02E+06	0.2	20.00	6.460	7.824

Table 4.15

Properties of permeant support at section 1

Permanent Support	**Depth (m)**	**EA**	**EI**	**ν**	**w**	**Horizontal Spacing (m)**	**Distance to King Post (m)**
Basement 1	3.65	3.49E+06	7.13E+04	0.2	6.25	4.000	5.932
Basement 2	6.45	3.49E+06	7.13E+04	0.2	6.25	4.000	5.932
Basement 3	9.25	3.49E+06	7.13E+04	0.2	6.25	4.000	5.932
Basement 4	12.05	3.49E+06	7.13E+04	0.2	6.25	4.000	5.932
Basement 5	15.75	1.24E+08	4.13E+07	0.2	6.25	8.000	5.85

Table 4.16

Properties of temporary support at section 2

Temporary Supports	**Depth (m)**	**EA**	**EI**	**ν**	**w**	**Horizontal Spacing (m)**	**Distance to King Post (m)**
First Inner Support	0.6	2.98E+07	3.57E+06	0.2	30.00	3.781	7.852
Second Inner Support	5.2	2.48E+07	1.32E+06	0.2	20.00	7.562	7.852
Third Inner Support	10.8	3.72E+07	3.10E+06	0.2	25.00	7.562	7.852

Table 4.17

Properties of permeant support at section 2

Permanent Support	**Depth (m)**	**EA**	**EI**	**ν**	**w**	**Horizontal Spacing (m)**	**Distance to King Post (m)**
Basement 1	3.65	6.20E+06	2.17E+04	0.2	6.25	1.000	4.9
Basement 2	6.45	6.20E+06	2.17E+04	0.2	6.25	1.000	4.9
Basement 3	9.25	6.20E+06	2.17E+04	0.2	6.25	1.000	4.9
Basement 4	12.05	6.20E+06	2.17E+04	0.2	6.25	1.000	4.9
Basement 5	15.75	1.24E+08	4.13E+07	0.2	6.25	7.800	4.9

Table 4.18

Properties of temporary support at section 3

Temporary Supports	Depth (m)	EA	EI	ν	w	Horizontal Spacing (m)	Distance to King Post (m)
First Inner Support	0.6	2.97E+07	3.57E+06	0.2	30.00	4.691	8.868
Second Inner Support	5.2	3.65E+07	1.95E+06	0.2	20.00	9.444	9.389
Third Inner Support	10.8	5.48E+07	4.57E+06	0.2	25.00	9.444	9.389

Table 4.19

Properties of permeant support at section 3

Permanent Support	Depth (m)	EA	EI	ν	w	Horizontal Spacing (m)	Distance to King Post (m)
Basement 1	3.65	4.62E+06	9.54E+04	0.2	6.25	8.230	4.046
Basement 2	6.45	4.62E+06	9.54E+04	0.2	6.25	8.230	4.046
Basement 3	9.25	4.62E+06	9.54E+04	0.2	6.25	8.230	4.046
Basement 4	12.05	4.62E+06	9.54E+04	0.2	6.25	8.230	4.046
Basement 5	15.75	1.23E+08	4.10E+07	0.2	6.25	6.075	4.046

Table 4.20

Properties of temporary support at section 4

Temporary Supports	Depth (m)	EA	EI	ν	w	Horizontal Spacing (m)	Distance to King Post (m)
First Inner Support	0.6	2.98E+07	3.57E+06	0.2	30.00	4.277	10.391
Second Inner Support	5.2	3.41E+07	1.82E+06	0.2	20.00	8.988	5.394
Third Inner Support	10.8	4.33E+07	3.14E+06	0.2	20.00	8.988	5.394

Table 4.21

Properties of permeant support at section 4

Permanent Support	Depth (m)	EA	EI	ν	w	Horizontal Spacing (m)	Distance to King Post (m)
Basement 1	3.65	4.96E+06	6.51E+04	0.2	6.25	3.525	3.958
Basement 2	6.45	4.96E+06	6.51E+04	0.2	6.25	3.525	3.958
Basement 3	9.25	4.96E+06	6.51E+04	0.2	6.25	3.525	3.958
Basement 4	12.05	4.96E+06	6.51E+04	0.2	6.25	3.525	3.958
Basement 5	15.75	1.24E+08	4.13E+07	0.2	6.25	10.920	3.958

4.5. Idealization of Earth Retaining System

The diaphragm wall was idealized as a plate element. Idealized stiffness parameters of the diaphragm wall area are elaborated in **Table 4.22**.

Table 4.22
Idealized stiffness parameters of diaphragm wall

Earth Retaining System	**Thickness (m)**	**Width (m)**	**A**	**I**	**E**	**EA**	**EI**	**ν**	**w**
Diaphragm Wall	0.6	1	0.6	0.018	3.10E+07	1.86E+07	5.58E+05	0.2	15

4.6. Idealization of Loads from Adjacent Environment

Surcharge loads acting near the excavation area were modelled as a uniformly distributed load. The applied loads from the adjacent environment in each section are elaborated in **Table 4.23**.

Table 4.23

Details of applied loads as surcharge

Section	Applied load
Section 1	3 storey buildings 30kN/m2 up to 5m, 1 m away the D wall
Section 2	Road, 25 kN/m2 up to 6m, 1 m away from the D wall
Section 3	Road, 25 kN/m2 up to 6m, 1 m away from the D wall
Section 4	Two storey building, 20 kN/m2 up to 6m, 1 m away from the D wall

4.7. Details of the Finite Element Model

Finite Element Modelling (FEM) was carried out using PLAXIS V20 software. The plain strain model was used to idealize the actual scenario into to 2D model by considering a uniform cross-section of the excavation. During the analysis, it was further assumed that there is no deformation in the z direction. Also, 15 noded triangular element, which is the default element, was selected for the modelling of soil layers, and during the analysis, drain parameters of soil were used. The dewatering effect was idealized by using steady-state ground water flow conditions. The groundwater level inside the excavation area was set at 0.5 m below the bottom of the excavation for all stages. Boundary conditions that were used during the analysis are as follows.

- Boundary xMin : Normally Fixed
- Boundary xMax : Normally Fixed
- Boundary yMin : Fully Fixed
- Boundary yMax : Free

After the above idealization, an initial analysis was carried out lateral deformation at final excavation depth were compared with the monitoring data. As there were considerable differences, the following modifications were made.

Modification 1

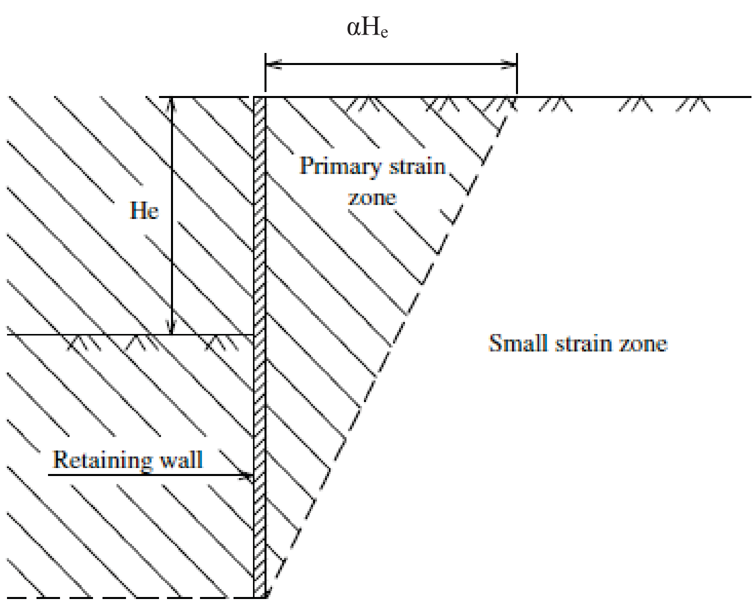
As elaborated in PLAXIS manual elastic of modulus can be increased five times of the initial value in unloading. This modification was done to the elastic modulus of soil as they are experiencing unloading.

Modification 2

As elaborated by Hsiung and Dao (2015) ground of the excavation was divided into two zones as the primary strain zone and the small strain zone as shown in **Figure 4.4**. This is due to the fact that, in general, settlements in the primary influence zone are underestimated while the settlements in the secondary influence zone are overestimated. Therefore, to achieve more realistic predictions of wall deflection during the analysis elastic modulus of each soil layer in the small strain zone was increased compared to the respective value in the primary strain zone. Hsiung and Dao (2015) conducted trail analyses varying the α value and increment of elastic modulus of soil to identify the most appropriate combination to obtain results that are comparable with the monitoring data.

Figure 4.5

Two strain zones of an excavation (Hsiung and Dao, 2015)



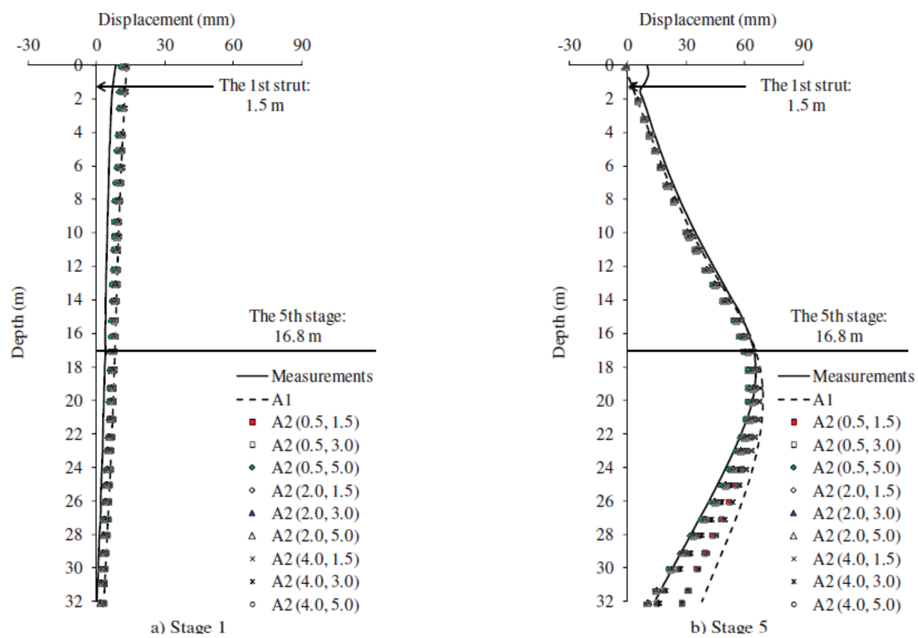
For the analysis of Hsiung and Dao (2015), 16.8m deep excavation which was supported by a 0.9m thick 32m deep diaphragm wall was considered. The retaining wall was propped by a steel struts at four levels. The two analysis have been conducted and for the first analysis Mohr Coulomb model was used without any modifications. For the second analysis, Mohr coulomb model with a modification by considering the small strain behavior of the soil during the excavation process has considered. And results obtained from both analytical method, compared with monitoring data.

During the analysis, the ratio between the width of primary strain zone behind the retaining wall to the excavation depth is defined as "α". Then the values of α is varied as 0.5,1.0,1.5,2.0,2.5,3.0,3.5 and 4.0. Ratio between the Young's modulus of each soil layer in the small strain zone to that in the primary strain zone, is defined as "β" and the value of is β varied as 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, and 5.0.

Figure 4.5 illustrated the results of the analysis of Hsiung and Dao (2015). From A1, it is indicated that first analysis and A2 (x,y) indicate the second analysis with (α=x, β=y). Lateral deformation for the Stage 1 and Stage 5 are shown in Figure 4.5. First graph indicates the behavior of wall at the first excavation while second graph indicating the behavior of wall at the final excavation level. Accordingly, shape of the first graph indicating the cantilever behavior while shape of second graph indicating the outward movement at the excavation bottom. Moreover, it is clear that, the results of the second analysis give closer graph to the field measurements.

Figure 4.6

Results of the analysis (Hsiung and Dao., 2015)



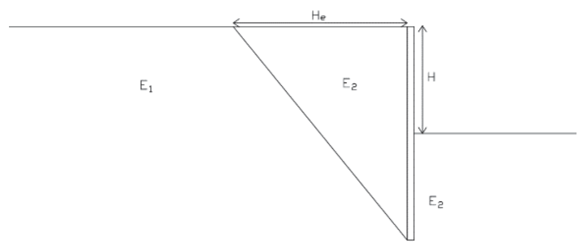
It further summarized that, wall deflection of second analysis are much affected by the parameter of β but less affected by the parameter α. Hsiung and Dao., 2015 further illustrated that for the first analysis, single young modulus has used and then it does not distinguish between loading and unloading stiffness of soil. This has causes an over prediction of the excavation bottom heave. Also, this over prediction bottom heave is a reason to larger wall deflection at the lower part of the wall.

By considering above facts, during this study the combination of both conditions (**Modification 1** and **Modification 2**) was considered. First the elastic modulus of soils was increased in primary strain zone and small strain zone by different factors and found the most suitable combination for the analysis for section 1. **Table 4.24** elaborates the various elastic modulus used in the analysis.

Table 4.24

Various E values used in the analysis

Trail No.	H_e	E_1	E_2
1	2H	3E	3E
2	2H	5E	2E
3	**2H**	**6E**	**3E**
4	2H	6E	4E



Then the analysis was carried out again by varying the distance of the primary strain zone with respect to the excavation depth to identify the most suitable zone for primary strain zone as elaborated in **Table 4.25**.

Table 4.25

Various H_e used in the analysis

Trail No.	H_e	E_1	E_2
1	H	6E	3E
2	2H	6E	3E
3	**3H**	**6E**	**3E**
4	4H	6E	3E
5	4.5H	6E	3E

Finally, the most suitable combination was selected as $H_e = 3H$, $E_1 = 6E$, $E_2 = 3E$ and this combination was used to analyze the other three sections.

CHAPTER 05

RESULTS OF THE ANALYSIS

This chapter presents the results of the numerical simulation of the staged excavation process adopted in this project. In the latter part of the Chapter, the results are compared with the monitoring data at the site and observations in a similar project published in the literature. As described in **Chapter 4** the process was modelled under three scenarios changing the stiffness of the soils. The three scenarios are;

1. Initial Analysis with standard soil modulus values
2. **Modification 1** (Increase the elastic modulus of soil (E) from E to 5E)
3. **Modification 2** (Considering Small Strain Theory described in **Section 4.6** of **Chapter 4**)

The construction involves the installation of the diaphragm wall and excavation to the bottom level with temporary supports at three levels. The support system at the top level is a slab that has a greater stiffness. The other two levels of temporary supports consist of props (concrete beams) connected to king posts as illustrated in **Figure 3.5** and **Figure 3.6**. Appropriate stiffness values were assigned to the supports at different levels in the process of modelling the excavation. Once the excavation reached the bottom level the installation of pile caps and the construction of the structure commenced. As the basement floor slabs at different stages were constructed the temporary support systems were dismantled. The complete construction sequence was modelled with the construction stages listed here. The analyses were done separately for the three scenarios.

The stages modelled are;

- Stage 1 - Construction of the Diaphragm wall
- Stage 2 - Excavate up to 1.1 m and install the supporting roof slab at a depth of 0.6m -Temporary support 1.
- Stage 3 – Excavation to 5.6m after install the temporary support 1
- Stage 4 – Excavation to 11.3m after install temporary support 2 at a depth of 5.2m.
- Stage 5 - Excavate up to 15.75 m after installation of temporary support 3 at a depth of 10.8m.
- Stage 6 – Construction of Basements 4 and 5 and removal of temporary support 3
- Stage 7 - Construction of Basements 3 and 2 and removal of temporary support 2
- Stage 8 - Construction of Basement 1and removal of temporary support 1

Lateral deformations of the diaphragm wall were estimated through the modelling for all stages of construction.

5.1. Lateral Deformation of Diaphragm Wall for Initial Stiffness Values Condition

Considering the differences in the subsoil profiles as presented in **Appendix A** separate analysis were done for Sections 1, 2, 3 and 4 (**Figure 4.1).**

Figure 5.1 shows the variation of lateral deformations of **Section 1** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and during the installation of permanent slabs while removing the temporary supports (Stage 6 to Stage 8).

In Stage 2 soil has been excavated without any lateral support. In that case, the wall deforms as a cantilever. When the excavation advances to deeper levels (Stage 3 to Stage 5), wall movement is restrained by the installation of new support as indicated by the deformed shape. The temporary support 1, which is of a greater stiffness due to the presence of the floor slab has restrained the deformation at that level very significantly. Also, a movement was observed at the toe as the diaphragm was not socketed into bedrock.

The excavation to 10.8m depth commences after the installation of Temporary support 2. As such, the outward deformation at Temporary Support 2 is restrained, but the wall moves outward towards the excavation below the level of Support 2. This behavior in deformation is demonstrated when the curves corresponding to stages 3 and 4 are compared. A similar pattern can be observed when curves corresponding to stages 4 and 5 are compared after the installation of temporary support 3. When further excavation was carried out to the depth of 15.75m after the installation of temporary support 3 there was a significant outward movement of the wall below the support level, but the movement at the support level 3 was restrained. Further lateral deformations after the installation of the support system at any level is minimal.

Any significant variation of lateral deformations was not observed during the installation of permanent slabs while removing temporary supports. (Stages 6, 7 and 8) Permanent supports are basement slabs that possess significant structural stiffness.

Figure 5.1

Lateral Deformation– Section 1

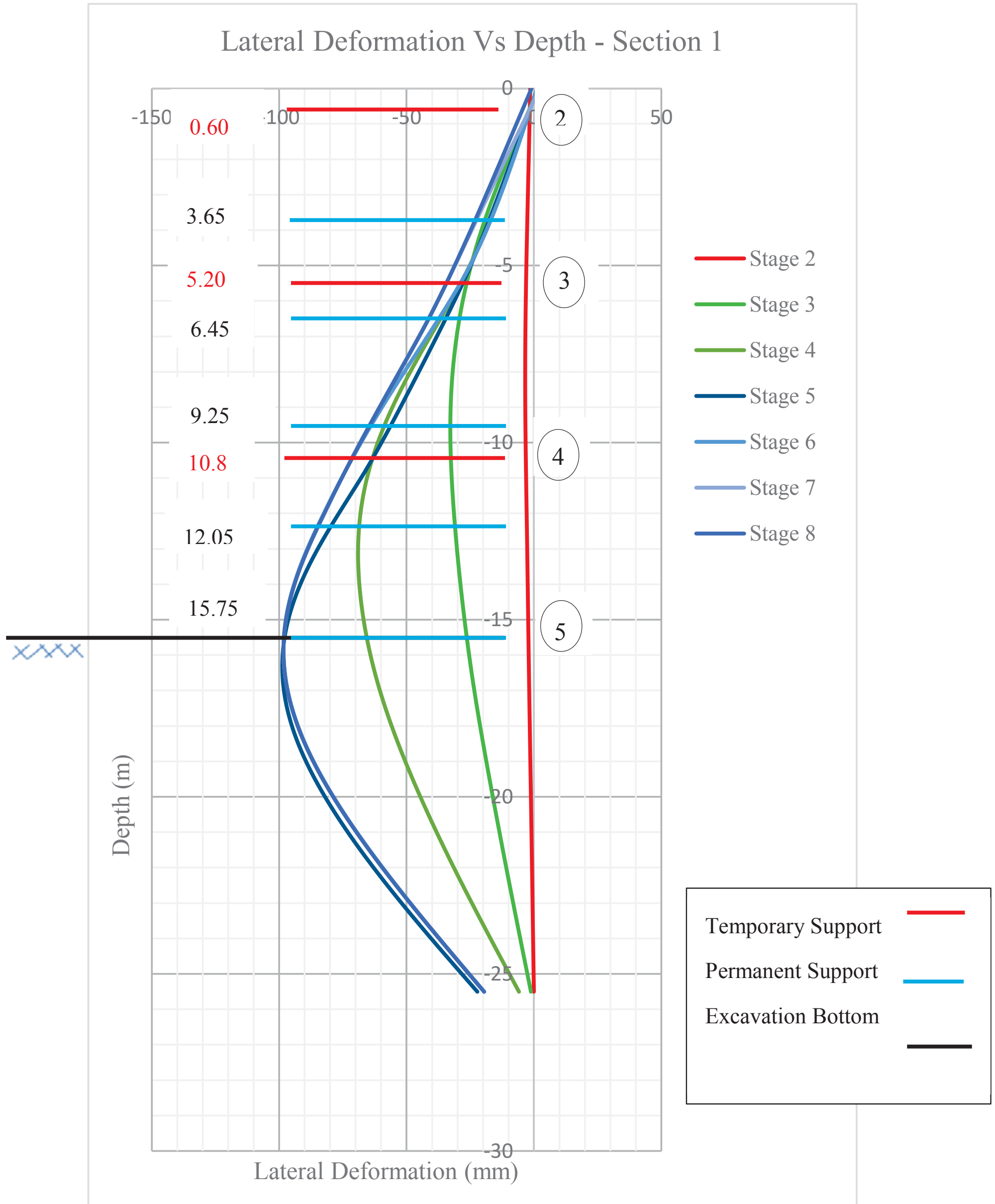


Figure 5.2 shows the variation of lateral deformations of **Section 2** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8). The trends in the deformation shape at different construction stages are similar to those of Section 1, but the numerical values of outward deformations are greater due to the lower stiffness values in the subsoil layers encountered.

Figure 5.2

Lateral Deformation– Section 2

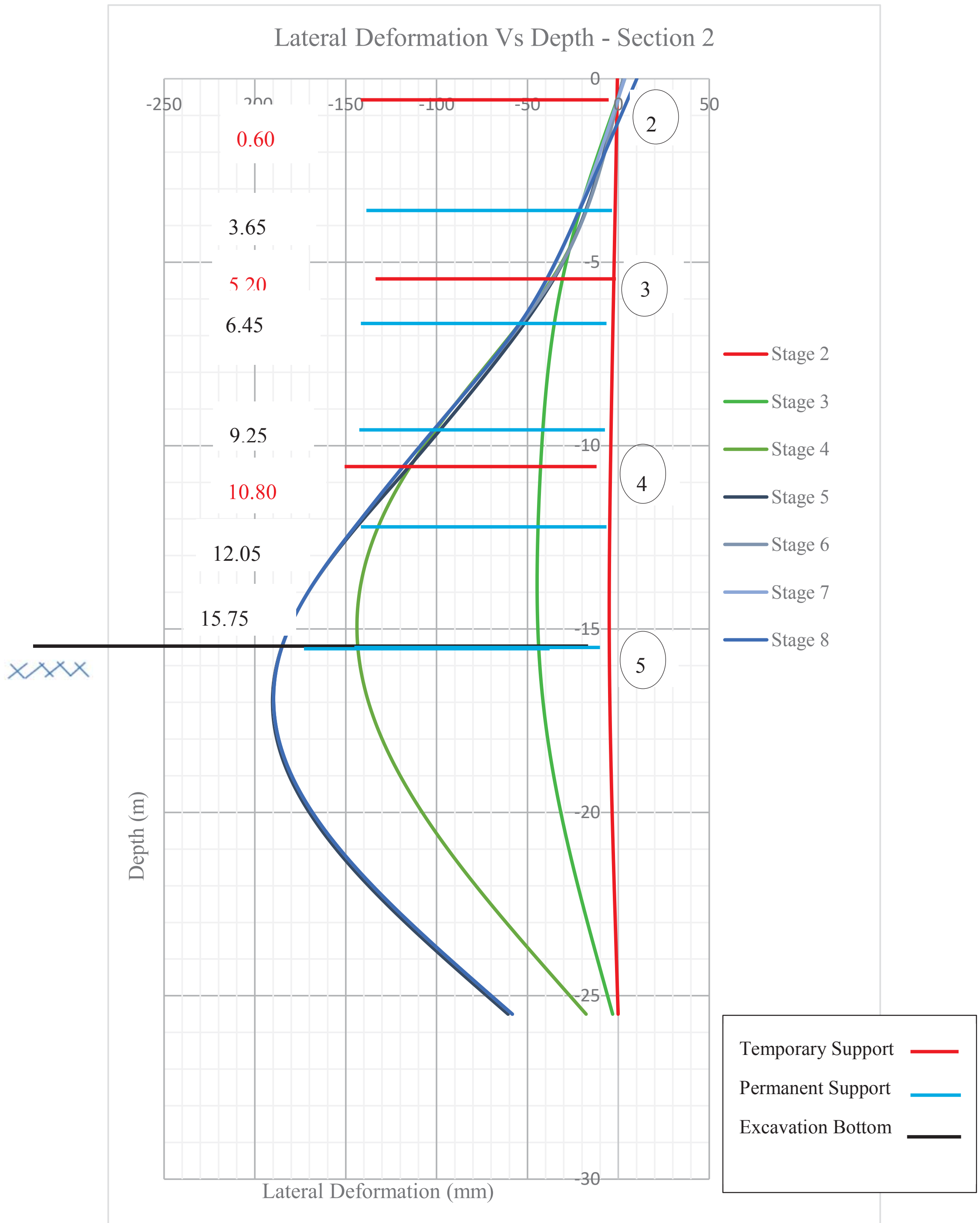


Figure 5.3 shows the variation of lateral deformations of **Section 3** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8). As observed in **Section 1** and **Section 2** restraining of the movements due to the lateral supports and outward movements below the support level can be observed.

Figure 5.3

Lateral Deformation – Section 3

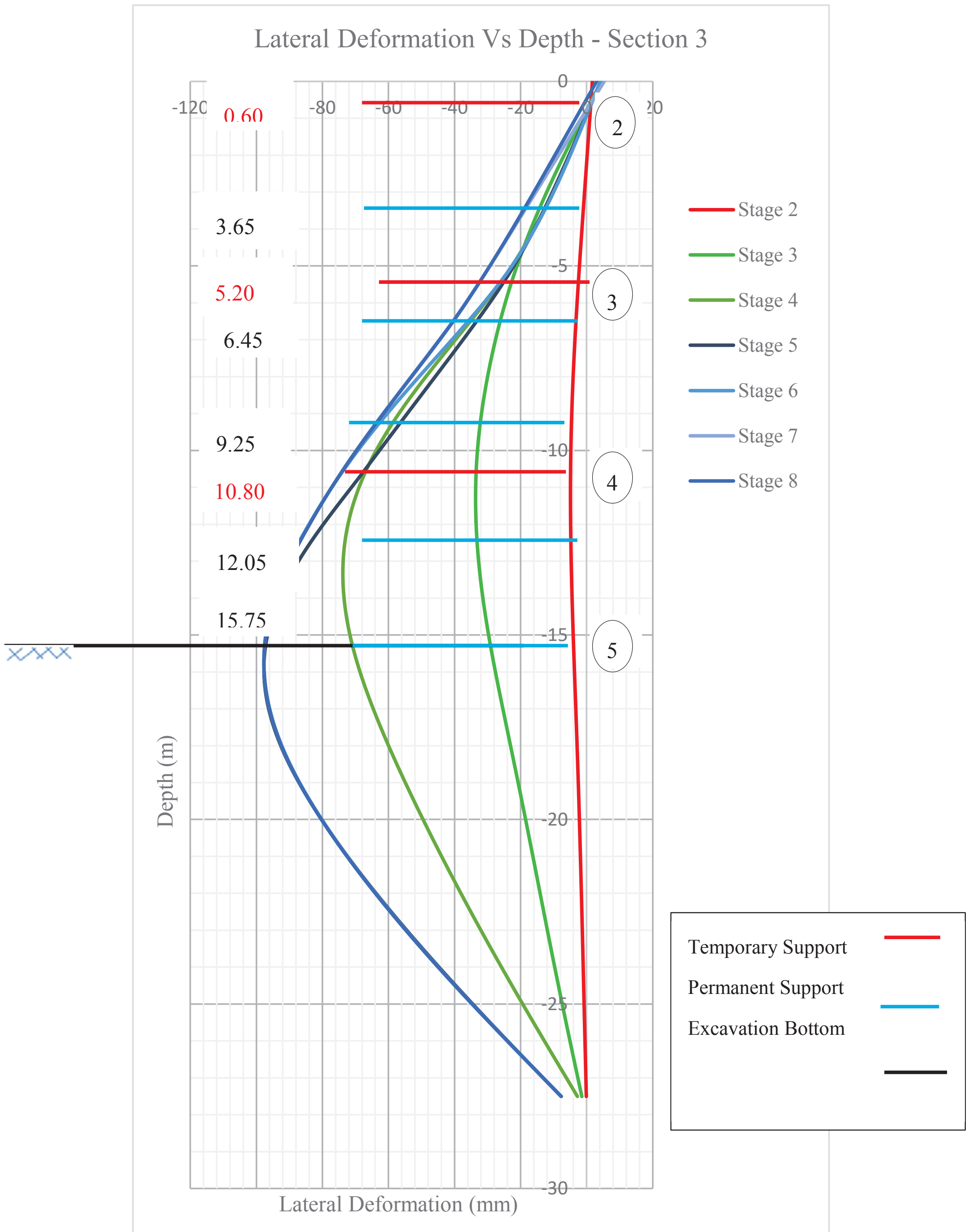
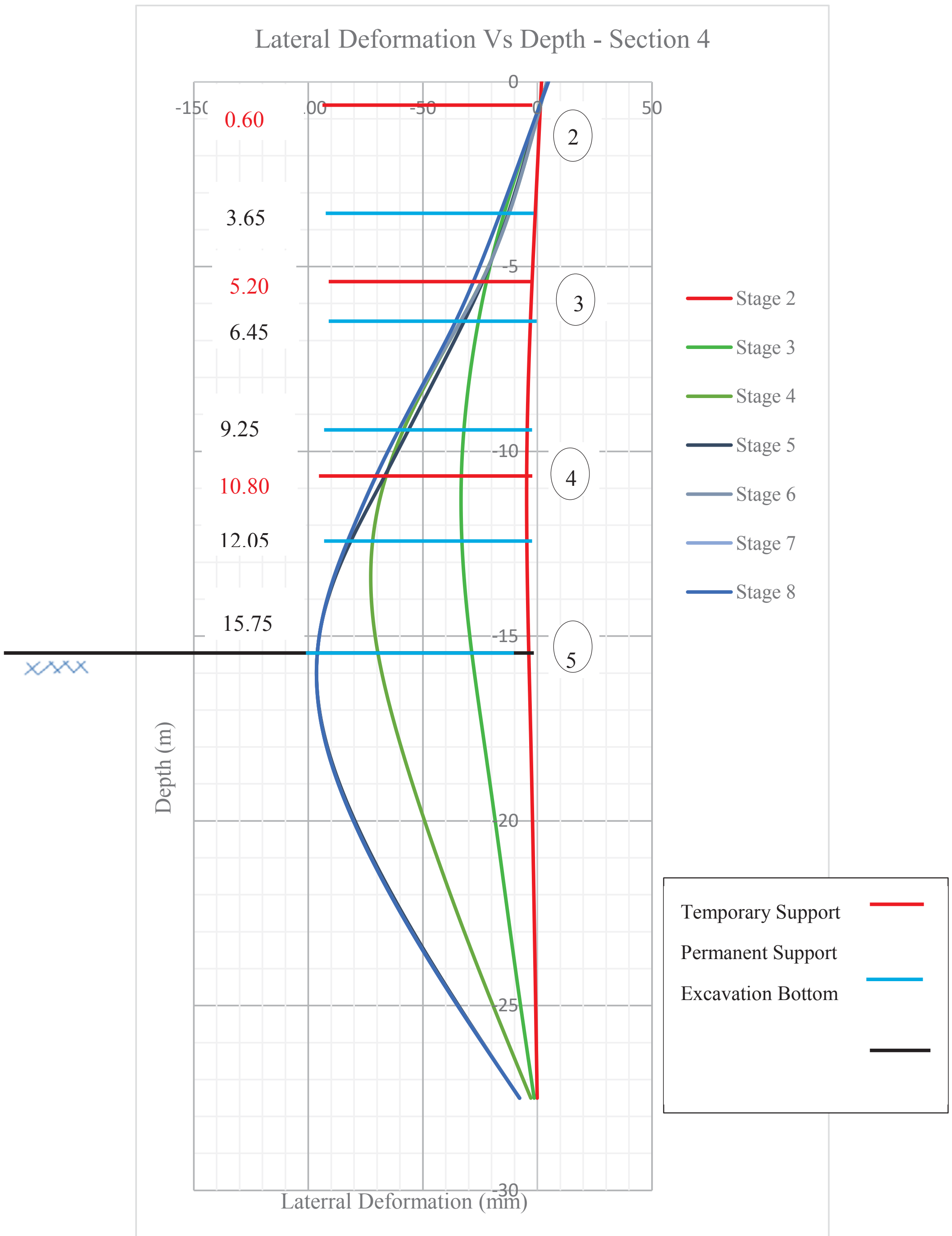


Figure **5.4** shows the variation of lateral deformations of **Section 4** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8). The observations are similar to those in the other three sections.

Figure 5.4

Lateral Deformation– Section 4



5.2. Lateral Deformation of Diaphragm Wall for Modification 1

Although there is no direct measured evidence from the performance of the excavation support system at the site it could be deduced that the outward wall movements are much smaller than the values estimated in the analyses presented in **section 5.1**. As such, further analyses were performed making logical modifications to obtain more sensible results.

As the excavation process primarily involves unloading, the relevant stiffness values would be greater (Liong,2014). Hence the series of analyses were repeated multiplying the soil stiffness values obtained by the conventional correlations by a modification factor of 5. This is a value suggested in the PLAXIS (2020) manual. **Figure 5.5** shows the variation of lateral deformations of **Section 1** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and during the installation of permanent slabs while removing the temporary supports (Stage 6 to Stage 8) after multiplying the soil stiffness in all layers by a factor of 5. (**Modification 1**).

The behavior in the initial cantilever stage and the subsequent laterally supported stages is similar to previous analyses. However, the maximum outward deformation has decreased from 99 mm to 83.02 mm.

With the modification of the initial stiffness of the soil E to 5E, lateral deformation of all stages has been reduced, but it has not got closer to the recommended allowable lateral deformations in CIRIA guidelines. The recommended allowable lateral deformation in CIRIA guideline for a 15.75 m deep excavation is 32 mm.

Figure 5.5 - *Lateral Deformation Section 1*

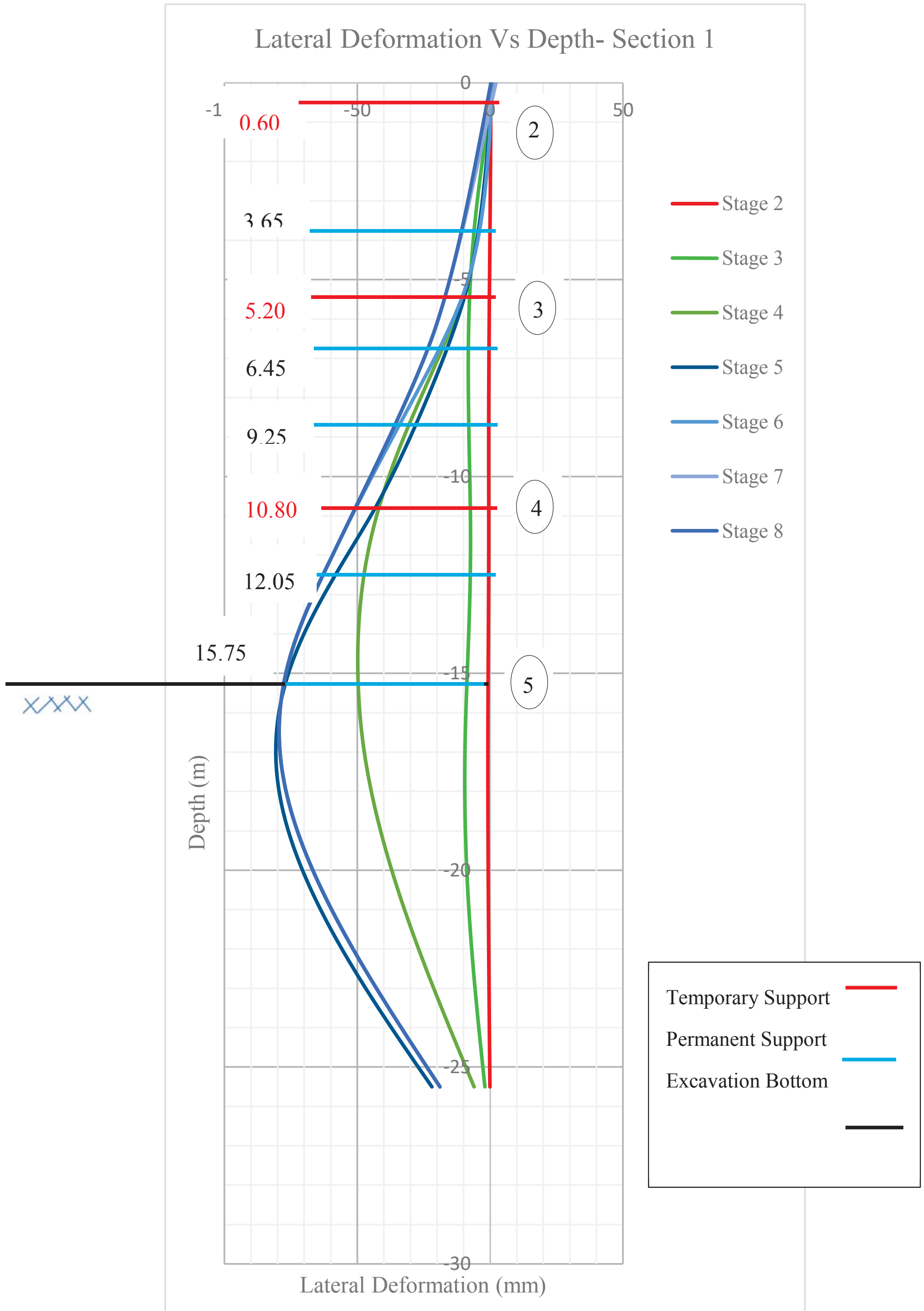


Figure 5.6 shows the variation of lateral deformations of **Section 2** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8) after modifying the soil stiffness E to 5E (**Modification 1**). The maximum lateral movement has reduced from 192.1mm to 128mm. Although the, lateral deformation of all stages has reduced, it has not been closer to the recommended allowable lateral deformations in the CIRIA guideline of 32mm.

Figure 5.6

Lateral Deformation– Section 2

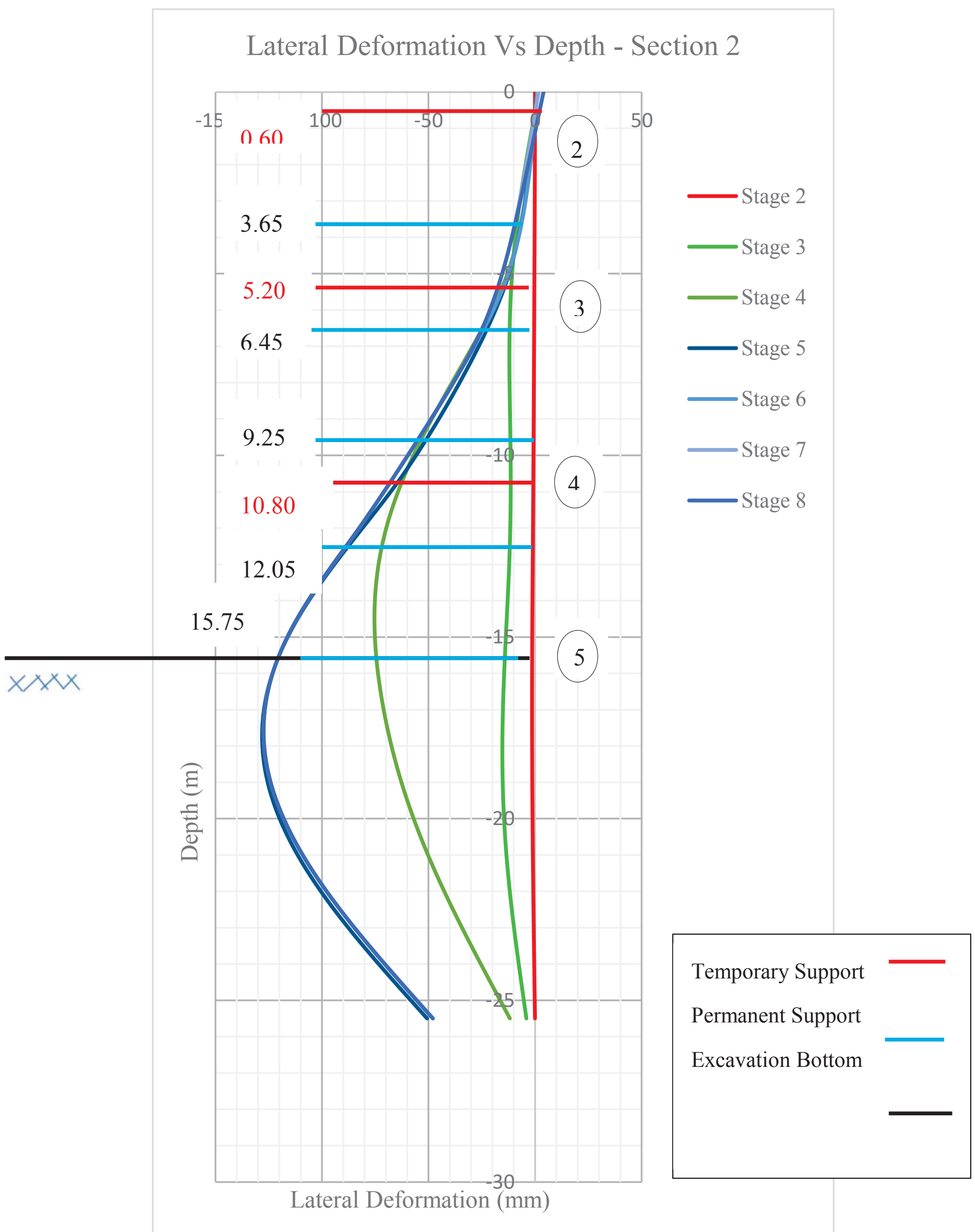


Figure 5.7 shows the variation of lateral deformations of **Section 3** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8) after modifying the soil stiffness E to 5E (**Modification 1**). The maximum outward movement has reduced from 100.1 mm to 75.75mm. As with the other two sections, Section 1 and Section 2, the obtained lateral deformation after the modification is higher than the recommended value in CIRIA guidelines.

Figure 5.7

Lateral Deformation– Section 3

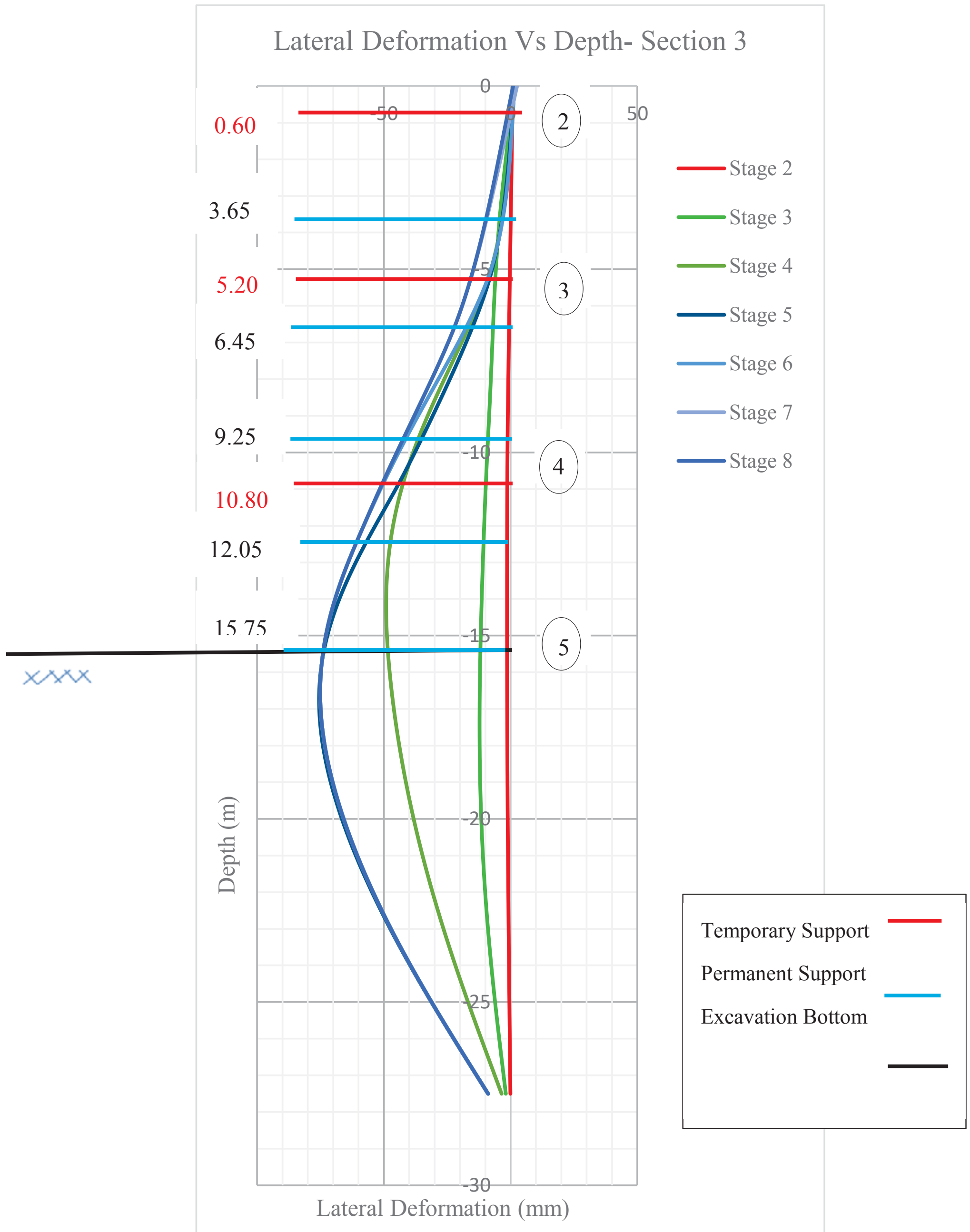
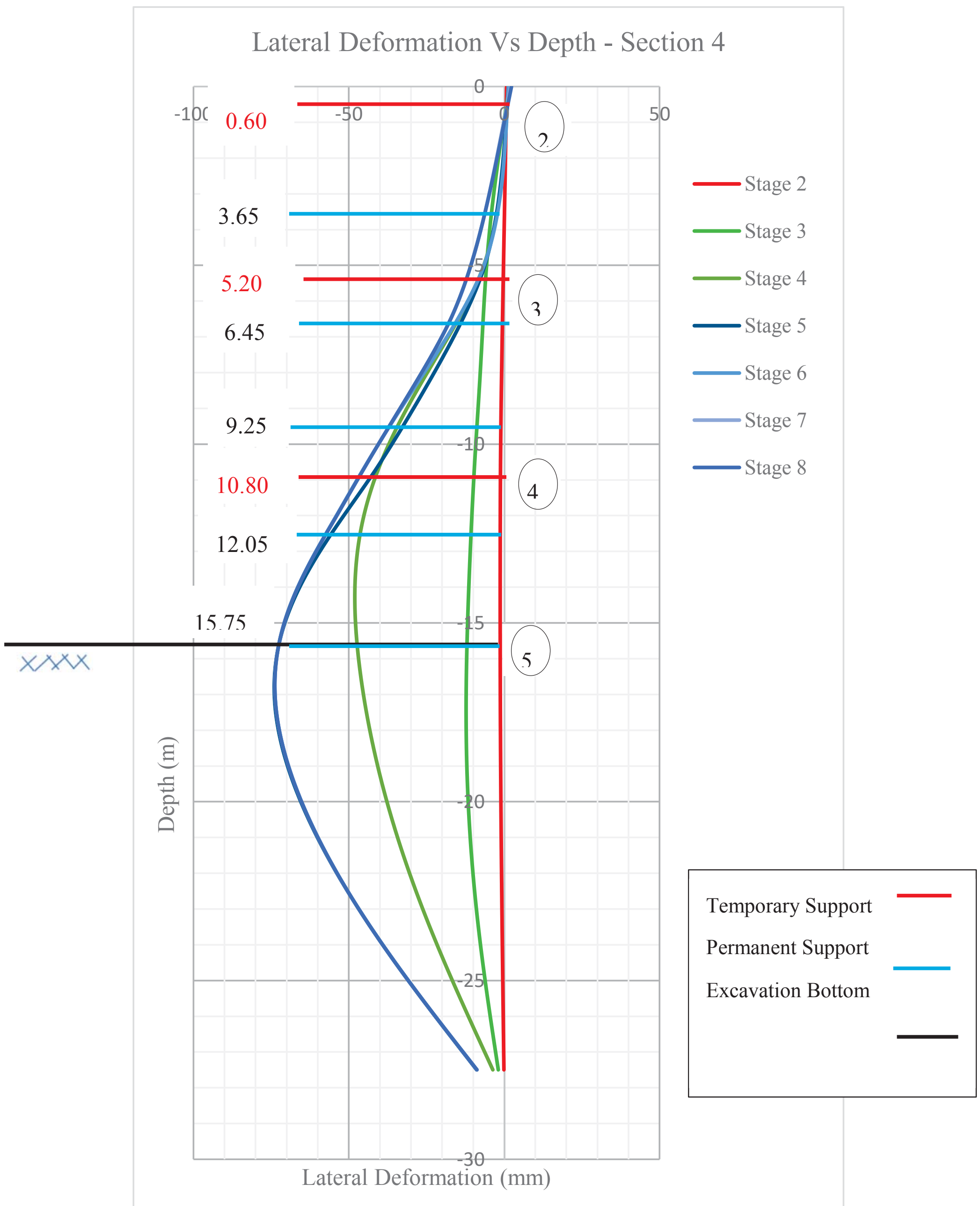


Figure 5.8 shows the variation of lateral deformations of **Section 4** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8) after modifying the soil stiffness E to 5E (**Modification 1**). The maximum outward movement has been reduced from 98.71 mm to 74.13 mm.

In this section also, even after modification of soil stiffness, the lateral deformation is not closer to the recommended value in CIRIA guidelines.

Figure 5.8

Lateral Deformation– Section 4



5.3. Analysis Methodology for Modification 2

Even after **Modification 1**, it was unable to obtain satisfactory values of deformation as specified by the CIRIA guideline. Therefore, another series of analyses was done using the small strain stiffness values for all these soil elements which are undergoing an unloading process following the suggestion by Hsiung and Dao (2015).

As discussed in **Section 4.6** under **Modification 2** several trail analyses were carried out by varying the distance to the primary strain zone and the value of elastic modulus of soil. (**Figure 4.4**) Initially, this analysis was carried out for **Section 1**(Ref **Figure 4.1**) by taking the H_e values constant and varying only the E values in the primary and the small strain zone as indicated in **Table 4.24**. Thereafter further analyses were carried out adopting the various combinations summarized in **Table 4.25**.

The deformation patterns observed in Section 1 for the final excavation stage for several different combinations are presented in **Figure 5.9**. This plot illustrates that with the increase of stiffness in both primary and small strain zones the deformations are significantly reduced. The increase of modulus only in the small strain zone will still lead to large deformations. It is logical to use an increased E value for both zones as the soil is undergoing unloading. However, a greater E value can be used for the small strain zone.

Similar observations were made by Ou and Hsieh (2011) and they illustrated that, different soil stiffness in the primary zone and small strain zone provide more accurate analysis. It further illustrated that, a strain of less than or equal to 10^{-3} % is assumed as a small strain zone, and the behavior of soil is assumed as linear elastic and unloading /reloading elastic modulus can be used. When a strain of greater than 10^{-3} % is considered as the primary strain zone and elastic modulus (E_{ur}) can be calculated as the following equation.

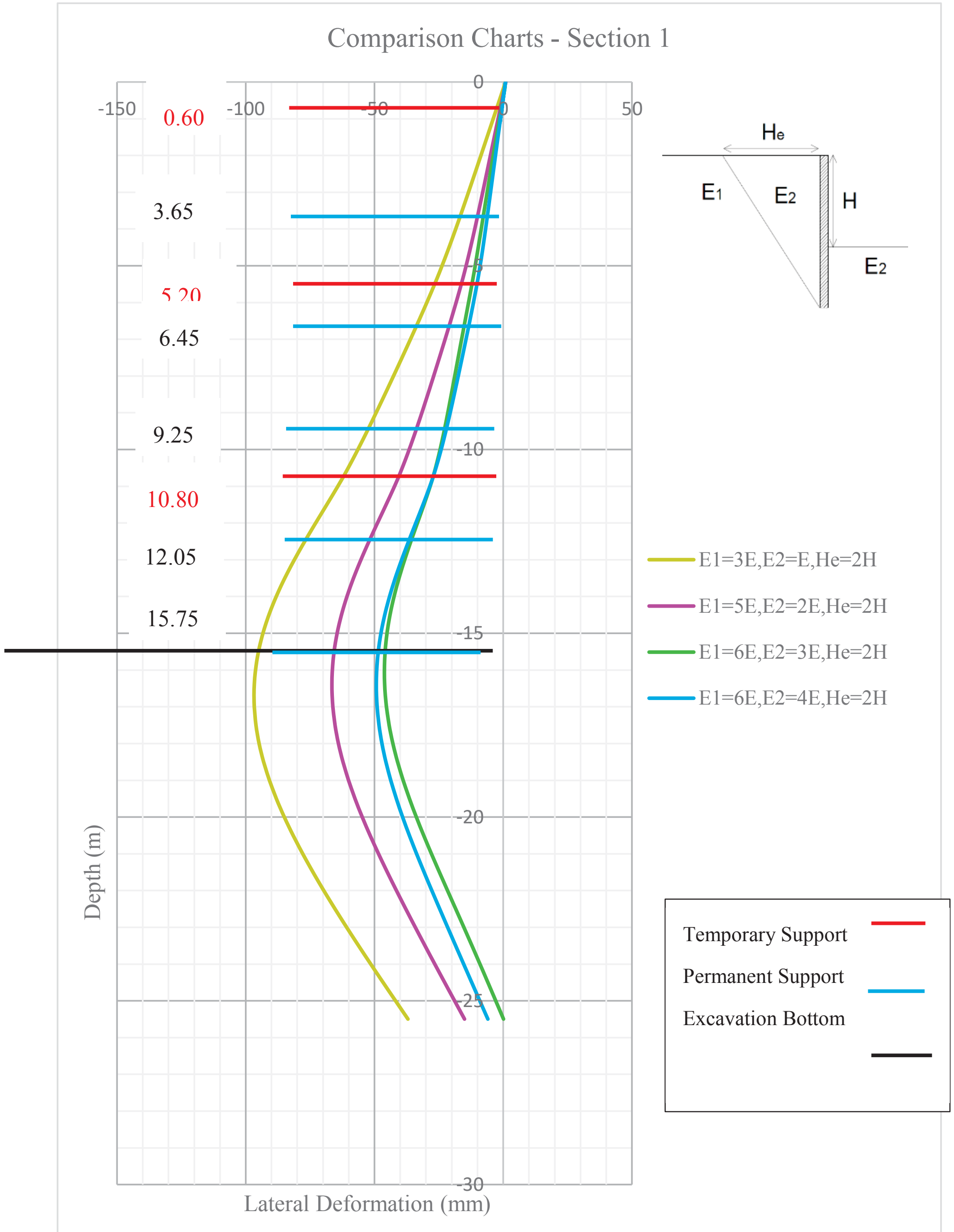
$$\frac{E_{ur}}{E_i} = 1 - \frac{SL - SL_i}{m + n(SL - SL_i)}$$ Ou and Hsieh (2011)

Where E_{ur} degradation of the unloading/reloading elastic modulus, E_i initial elastic modulus, SL is the stress level, SL_i is the stress level corresponding to the threshold value of the small strain, m and n are the degradation parameters and can be obtained from the unloading/reloading tests. E_i initial elastic modulus which can be obtained through bender element tests or in situ seismic tests.

As the elastic modulus of the primary zone cannot be calculated using the above equation due to the unavailability of relevant data, the elastic modulus of both zones was assumed by considering the unloading/reloading effect. Lateral deformation at the final excavation level with the assumed combinations is illustrated in **Figure 5.9**. It shows that 3E for the primary zone and 6E for the small strain zone is the most appropriate combination as it gives closer lateral deformation recommended in CIRIA guidelines.

Figure 5.9

Comparison of Results with different stiffness modifications



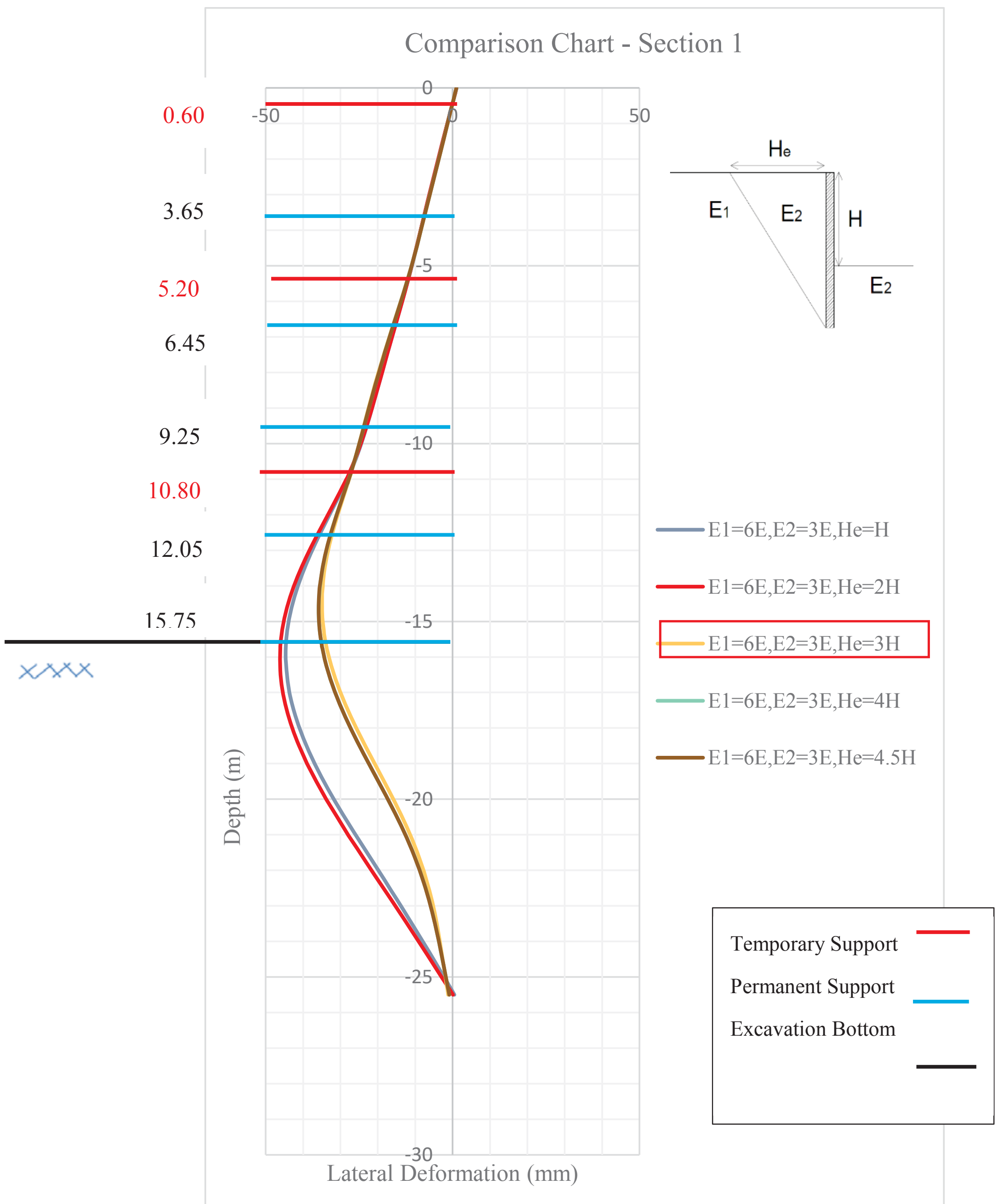
After that, analysis was carried out by varying the distance to the primary strain zone, and not varying the elastic modulus of soils (For Trial No. 3 3E 6E 2H combination). A comparison of the results obtained from different analyses presented section **Table 4.25** in **Section 4.6** is shown in **Figure 5.10**. The results show that 3E 6E 3H combination is the most appropriate combination.

Ou and Hsieh (2011) conducted a study to obtain an appropriate distance to the primary strain zone. It shows that when $\sqrt{B^2} + {H_e}^2 \leq 2H_e$ where B is the excavation width and the H_e is the excavation depth the distance of the primary zone is approximately equal to $\sqrt{B^2} + {H_e}^2$. In this study, the value of $\sqrt{B^2} + {H_e}^2$ is approximately equal to the $3H_e$.

Analysis for other sections was done using the same combination used for **Section 1** (For Trial No. 3 3E 6E 3H combination)**.**

Figure 5.10

Comparison of Results with different distance to primary strain zone



5.4. Lateral Deformation of Diaphragm Wall for the other sections under the selected combination of E in Modification 2

The excavation process of all four sections was modelled for the selected combination of E values and the region of small strain, for all the construction stages. The results are presented from **Figure 5.11** to **Figure 5.14**.

With **Modification 2** the maximum lateral deformation has reduced from 99 mm to 35 mm. With the modification lateral deformation has reduced up to the recommended value in CIRIA guidelines.

Figure 5.11
Lateral Deformation– Section 1

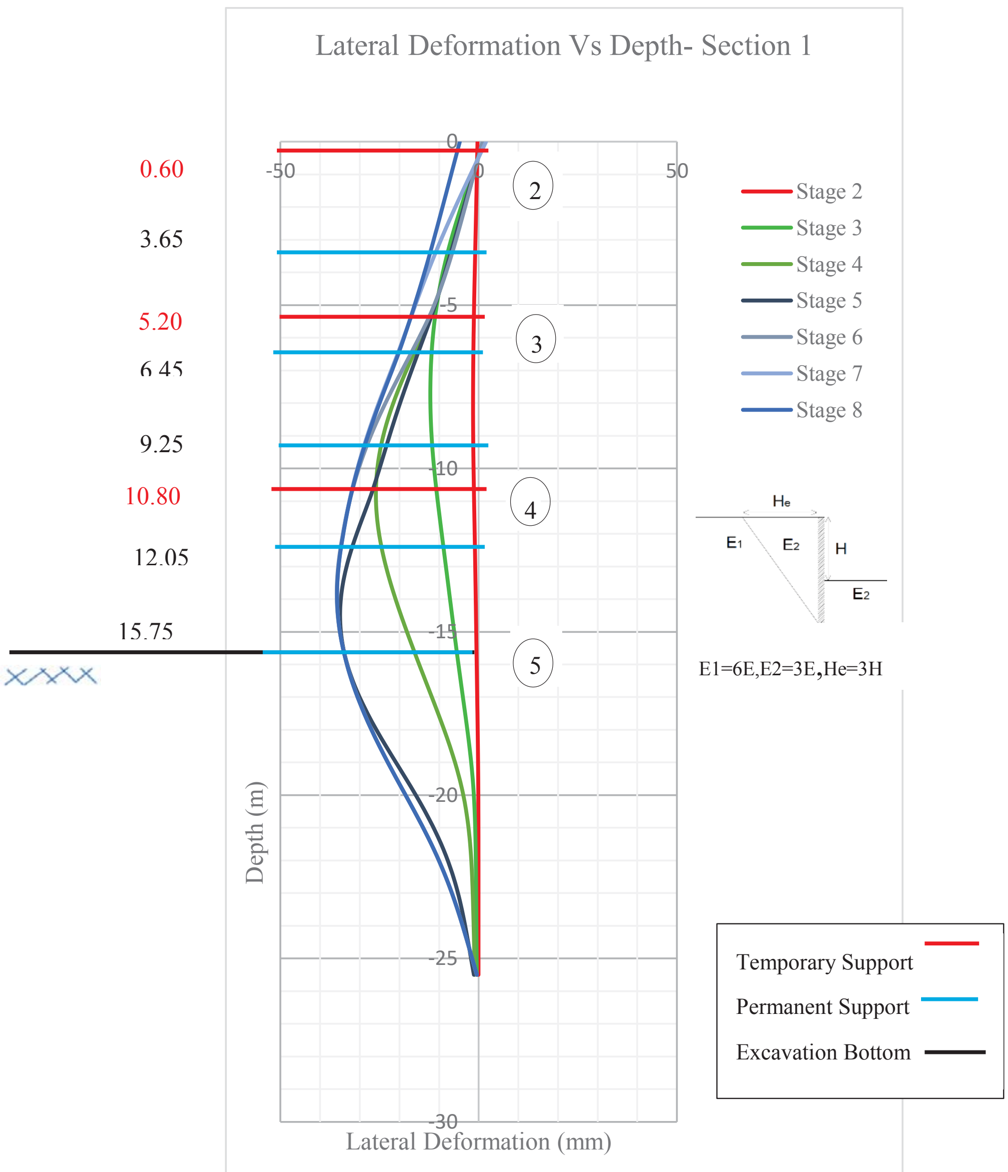


Figure 5.12 shows the variation of lateral deformations of **Section 2** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8) after modifying the soil stiffness according to the **Modification 2** (Ref **Section 4.6**). The maximum deformation has reduced from 192.1 mm to 115.6 mm.

With the **Modification 2**, lateral deformation of all stages has reduced. However maximum lateral deformation has not reached to the recommended lateral deformation for 15.75 m deep excavation in CIRIA guideline which is 32mm. The reason may be presence of low stiffen soil layers at the bottom of the excavation

Figure 5.12 –
Lateral Deformation– Section 2

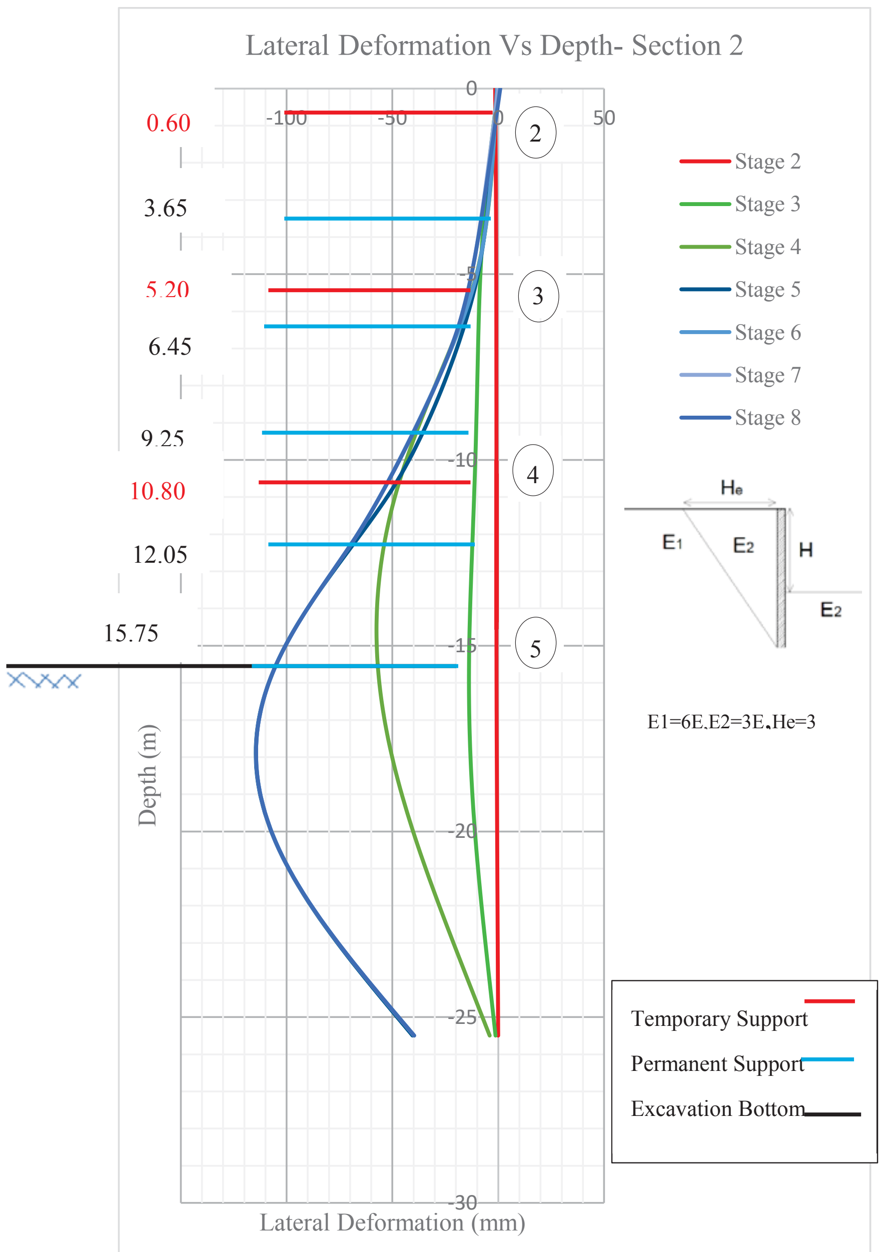


Figure 5.13 shows the variation of lateral deformations of **Section 3** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8) after modifying the soil stiffness according to the **Modification 2** (Ref **Section 4.6)**. The maximum deformation has reduced from 100.1 mm to 50.47 mm.

With **Modification 2**, lateral deformation of all stages has reduced and maximum lateral deformations have got closer to the recommended value in CIRIA guidelines.

Figure 5.13

Lateral Deformation– Section 3

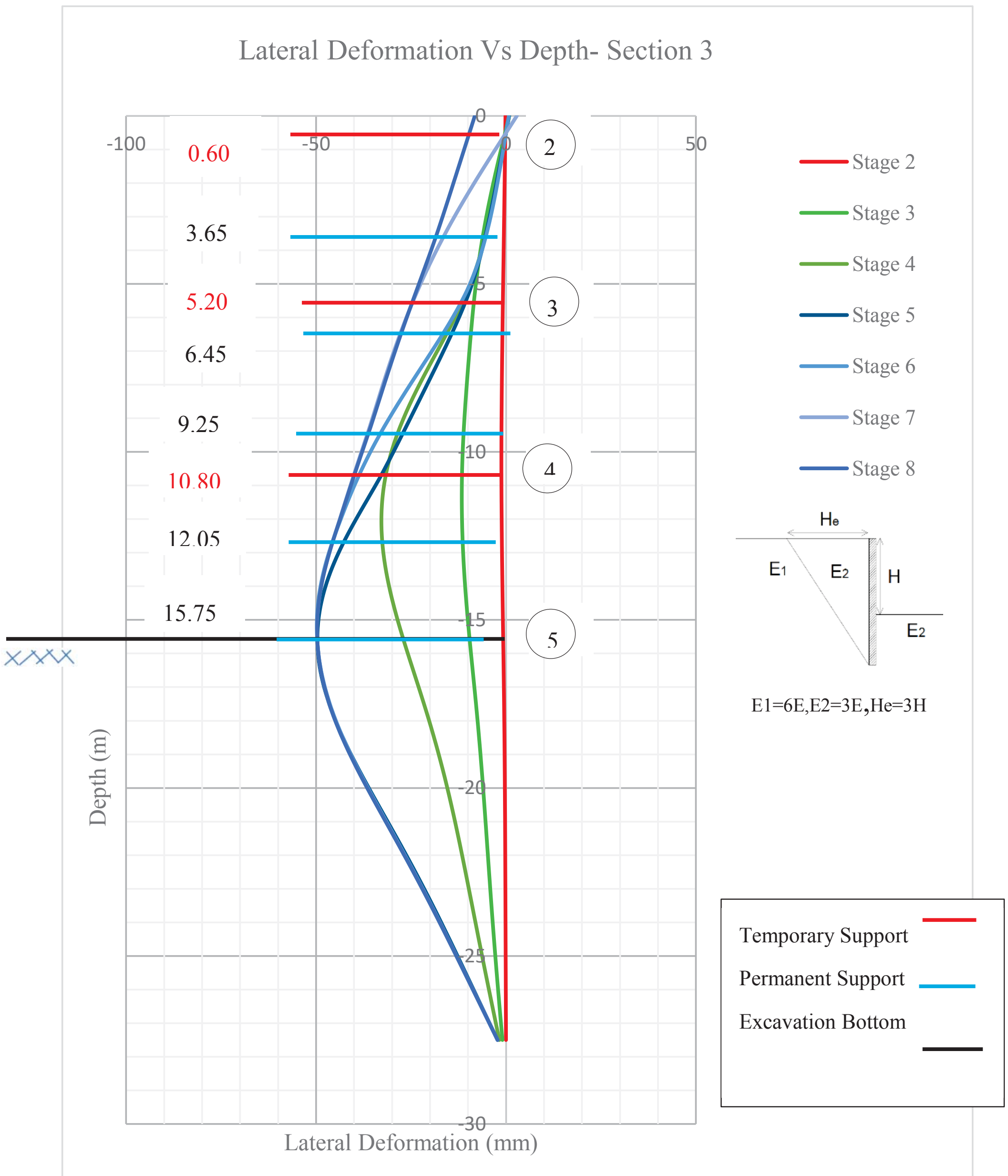
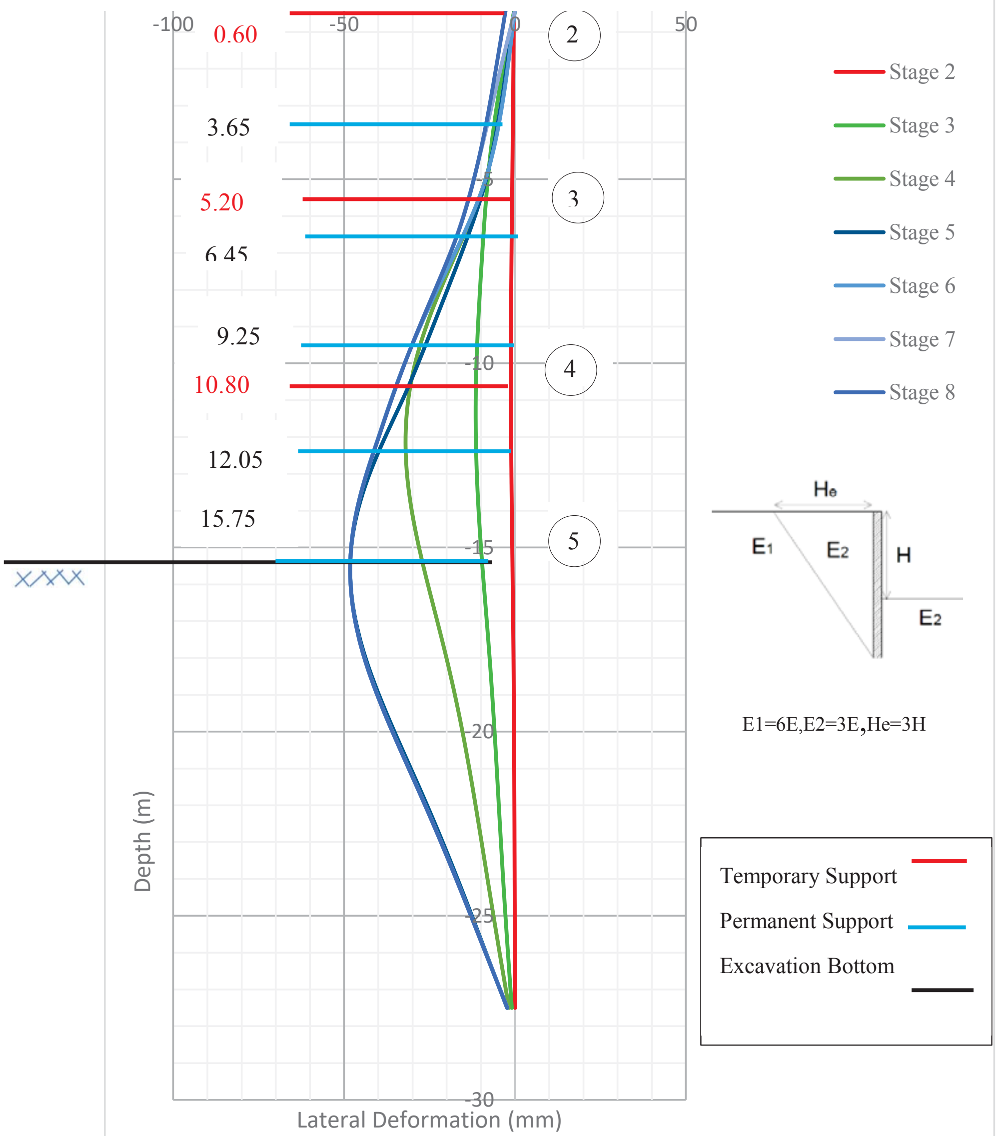


Figure 5.14 shows the variation of lateral deformations of **Section 4** (Ref **Figure 4.1**) during the excavation process (Stage 2 to Stage 5) and installation of permeant slabs while removing temporary supports (Stage 6 to Stage 8) after modifying the soil stiffness according to the **Modification 2** (Ref **Section 4.6**). The maximum deformation has reduced from 98.71 mm to 49.06 mm.

This requirement of CIRIA guidelines was satisfactory in Section 1, Section 3 and Section 4. But in section 2 values were greater than the recommendation in CIRIA guidelines.

Figure 5.14

Lateral Deformation – Section 4



5.5. Comparison of Analytical Results with Monitoring Data

Lateral deformations of the diaphragm wall obtained from analysis conducted under the above three conditions were compared with the available monitoring data. As per the information received, the monitoring has been commenced at the stage where excavation of 10.8 m to 15.75 m. As shown by the results of the analysis, the shape of the lateral deformation graphs follows the same pattern through the final excavation up to the completion of permanent supports. Therefore, for comparison with monitoring data the lateral deformations of the wall that occurred between the construction stages 4 and 5 were extracted from the respective analytical results.

Figure 5.15 shows the variation of lateral deformations of **Section 1** (Ref **Figure 4.1**) relevant to excavation between 10.8 m to 15.75 m with the depth separately with respect to followings.

- Based on monitoring data
- Based on the results obtained from the initial analysis which described in **Section 4.6** of this thesis
- Based on the results obtained from analysis with **Modification 1** which described in **Section 4.6** of this thesis
- Based on the results obtained from analysis with **Modification 2** which described in **Section 4.6** of this thesis

Wang et.al., 2023 showed that when numerically simulating the behavior of excavation using Mohr Coulomb model, it will result in much larger lateral deformation even with the increment of modulus of elasticity. It further illustrated that, when increasing the elastic modulus, lateral deformation is reduced and the same was observed in this analysis also. Also, there is larger deformation observed at the bottom of the excavation and the reason for this may be the outward movement of the wall with the removal of soil and not having any lateral support at lower levels to restrain this movement.

When it considering the monitoring data of this site, maximum deformation is at the top of the wall, and there is less lateral deformation at the bottom of the excavation. According to the monitoring data shown in Wang et.al., 2023(**Figure 2.14**), the monitoring graphs also follow the same shape as the analytical graphs. Most probable reason for the difference between the shape of graphs at ODEL site may be the inappropriate method adopted during the installation of inclinometers as described in **Chapter 3**.

And Wang et.al., 2023 inclinometers are installed up to the bottom of the wall and in the ODEL site the inclinometers are installed only up to the bottom of the excavation level. Because of this formation, the lateral deformation at that level may have been considered zero by default.

Figure 5.15

Comparison of Monitoring Data with Analysis Result – Section 1

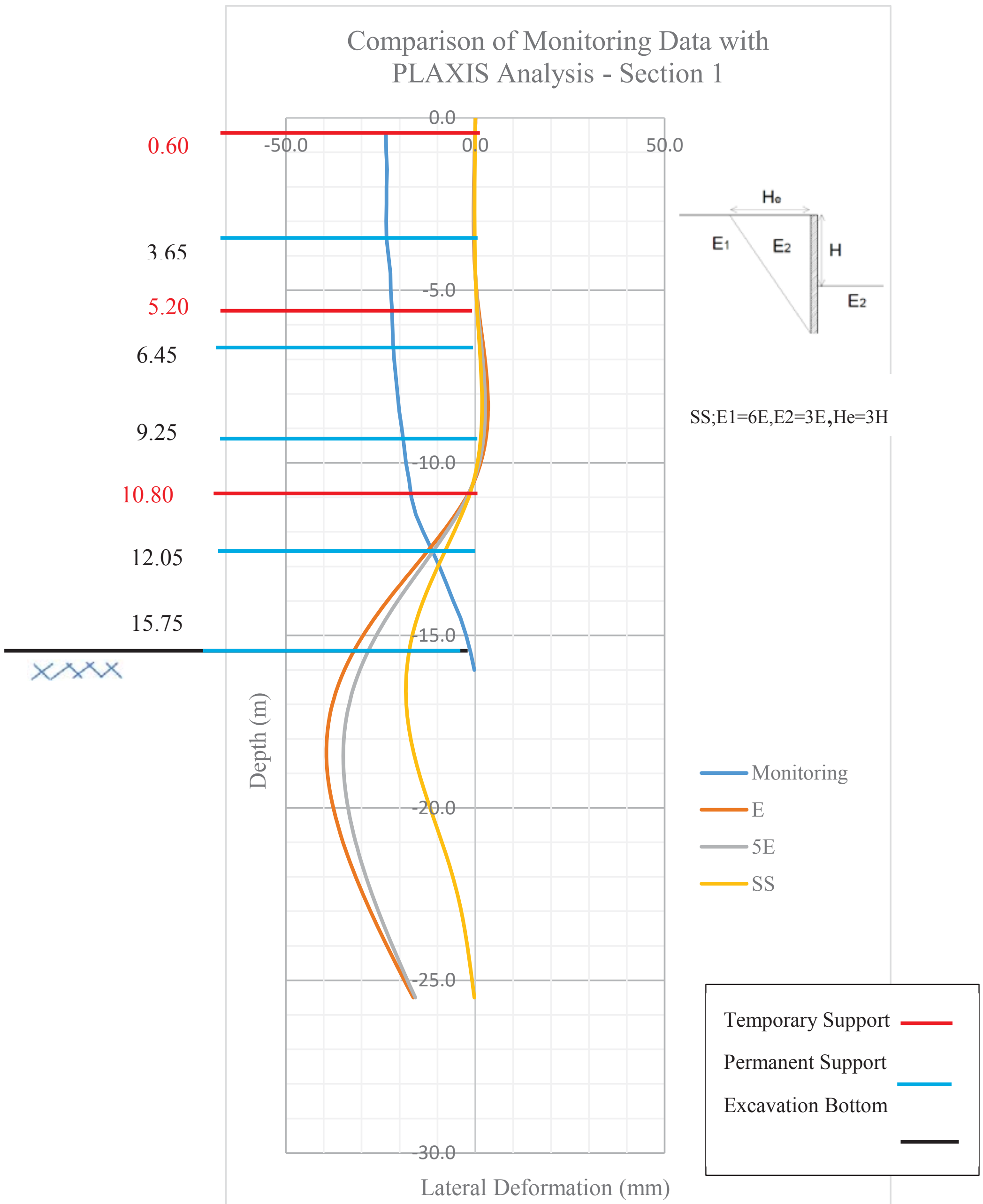


Figure 5.16 shows the variation of lateral deformations of **Section 2.** At section 2 the, maximum deformation is at the middle of the as compared to the maximum at the top in Section 1. However, this comparison has the same issues as discussed with Section 1.

Figure 5.16

Comparison of Monitoring Data with Analysis Result – Section 2

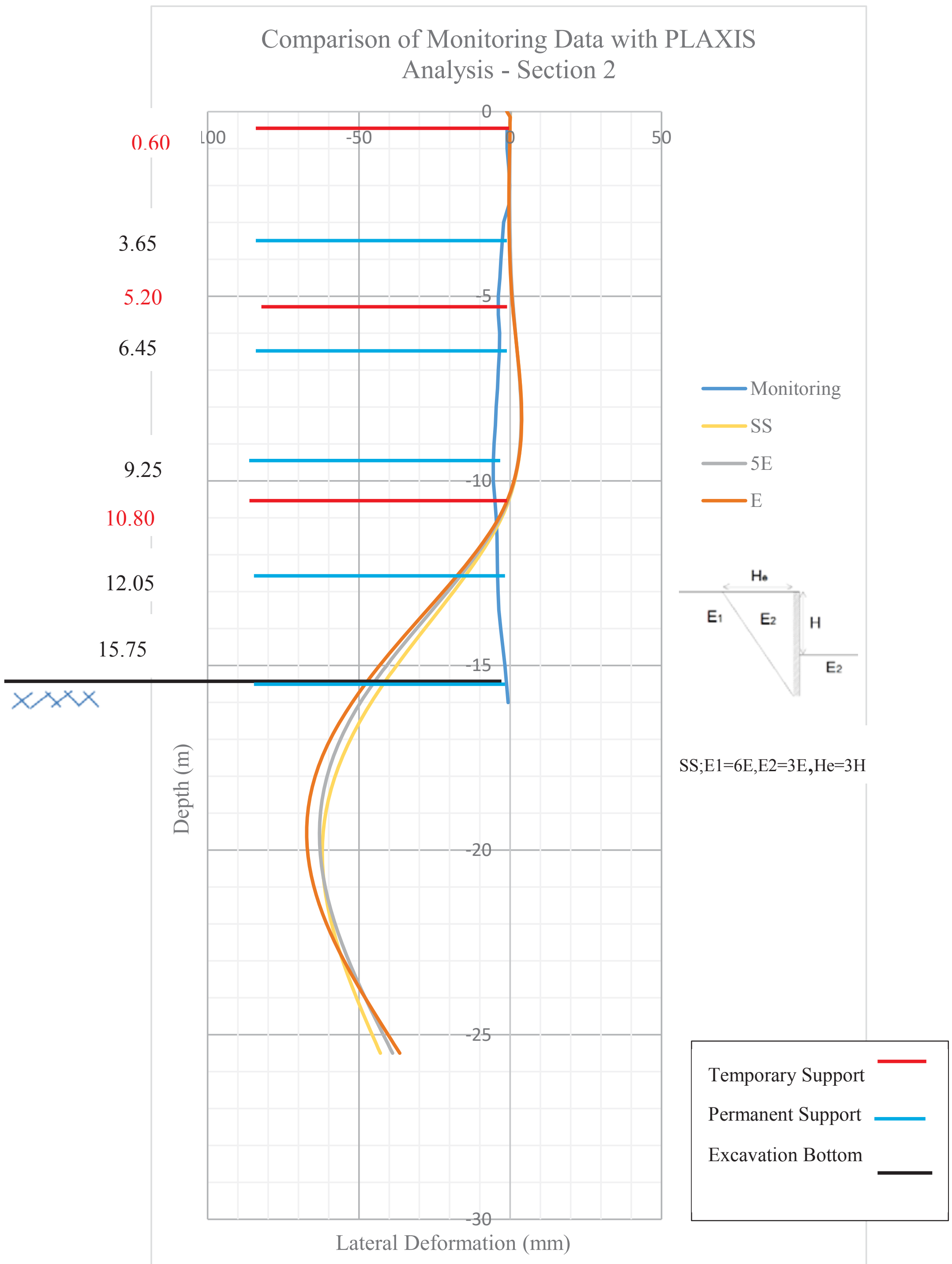


Figure 5.17 shows the variation of lateral deformations of **Section 3.** In the monitoring graphs, there is inward deformation at the top of the retaining wall while in analysis result deformation, there is no deformation at the top due to the issues discussed in previously.

Figure 5.17

Comparison of Monitoring Data with Analysis Result – Section 3

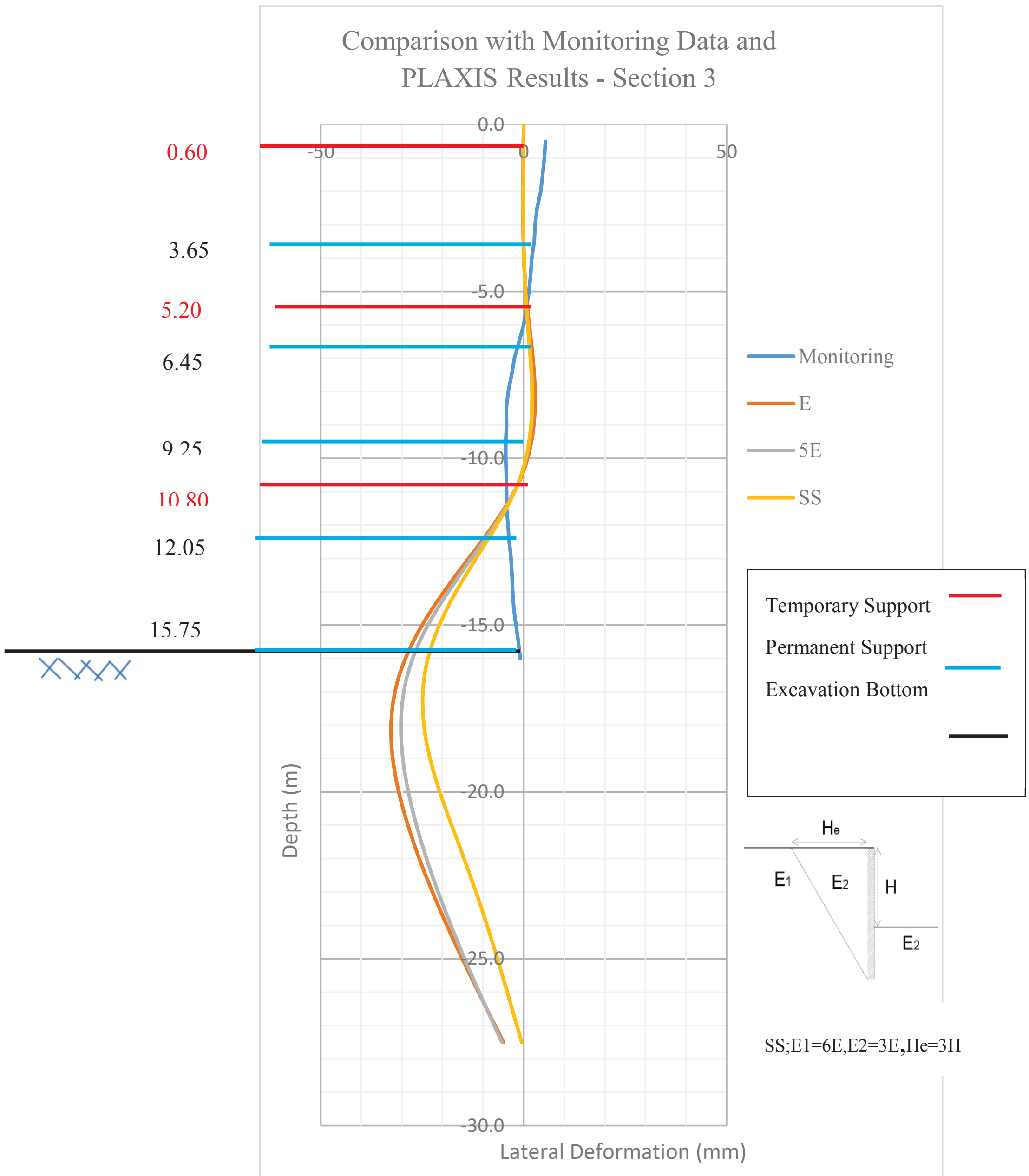
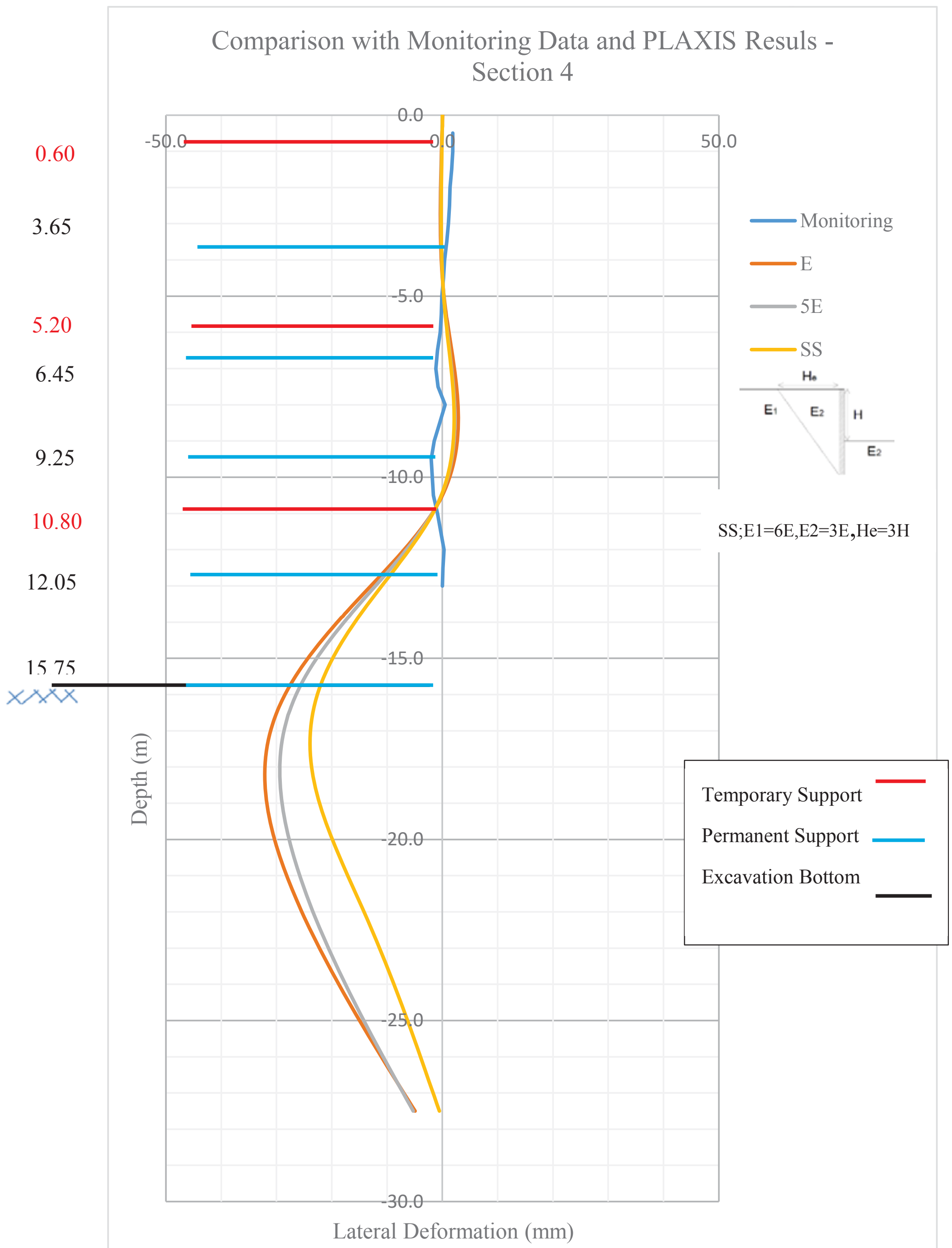


Figure 5.18 shows the variation of lateral deformations of **Section 4.** At the **Section 4** there is an inward deformation at the top of the retaining wall as **Section 3**.

Figure 5.18 - *Comparison of Monitoring Data with Analysis Result – Section 4*



5.6. Lateral Deformations Further away from the wall at the Retained Side

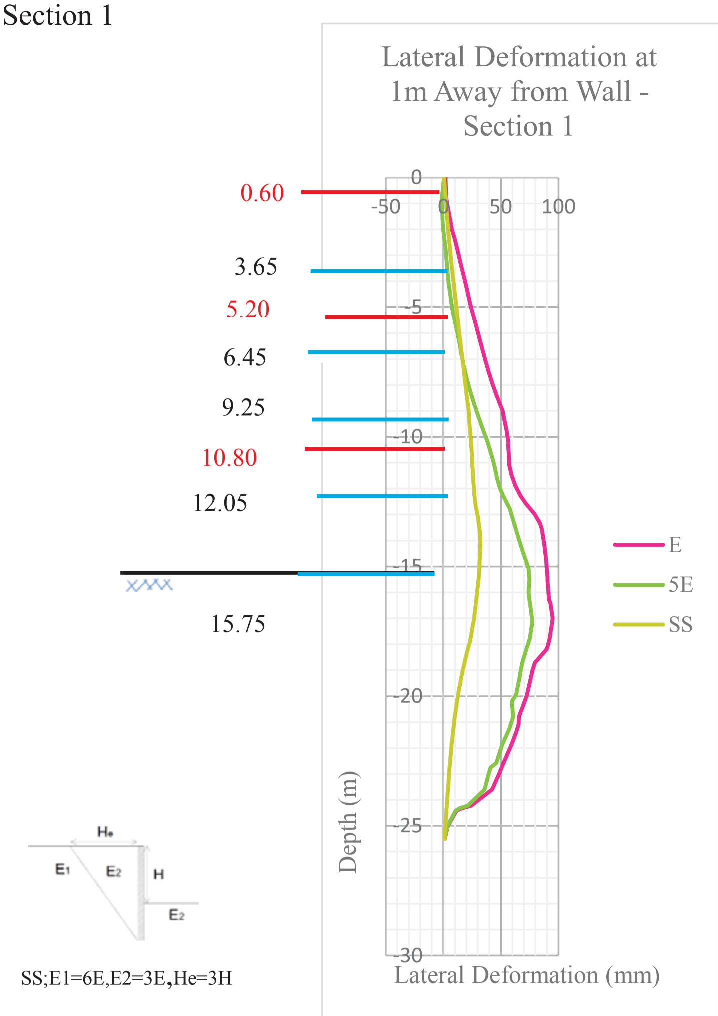
Lateral deformations of the soil at the retained side away from the retaining wall was plotted with the excavation depth. As structures are located 1m and 5m away from the wall, lateral deformations at that distances were considered. It was observed that, there is slight chance of lateral deformation of retaining wall and the soil 1m away from the wall. However, there is a considerable reduction of lateral deformation 5m away from the wall in all sections. **Figure 5.19**, **Figure 5.20**, **Figure 5.21** and **Figure 5.22** illustrated the lateral deformation curves for 1m and 5m away from wall in the retained side at the four sections. Comparison was done for initial stiffness E, 5E and the selected final SS combination.

Figure 5.19

Lateral Deformations away from the Retaining Wall – Section 1

Lateral Deformation at Retained Side

Section 1



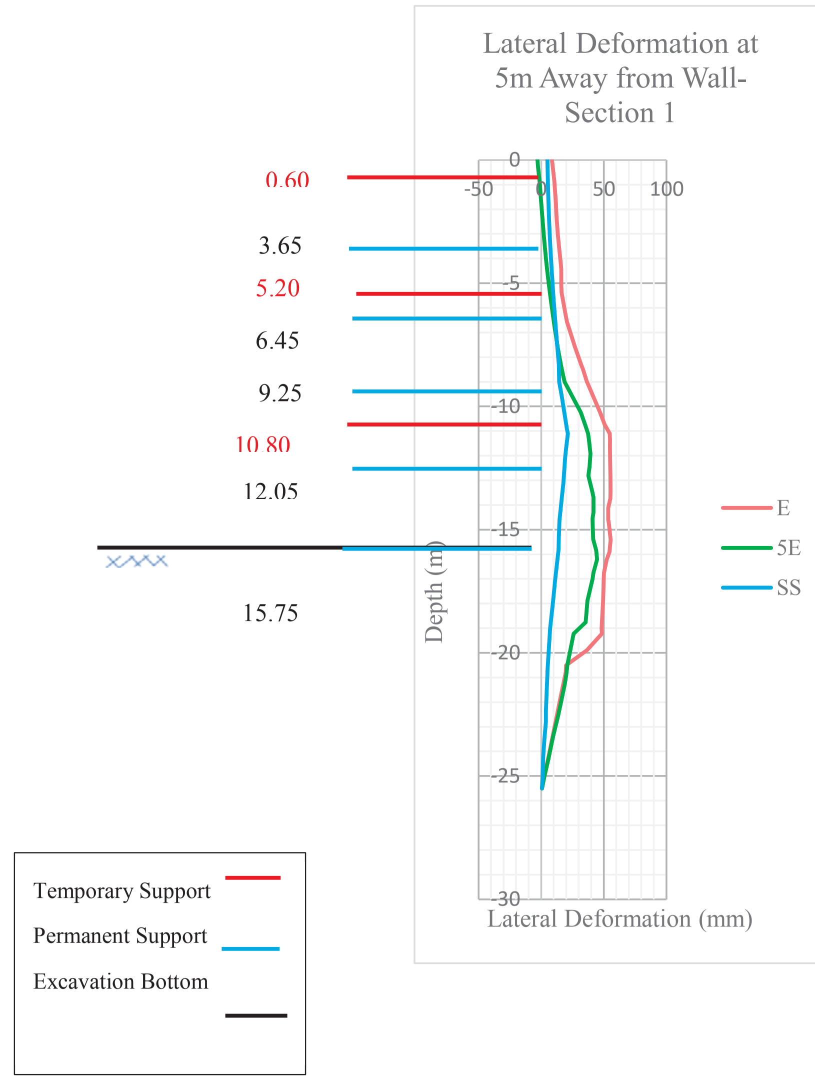


Figure 5.20

Lateral Deformations away from the Retaining Wall-Section 2

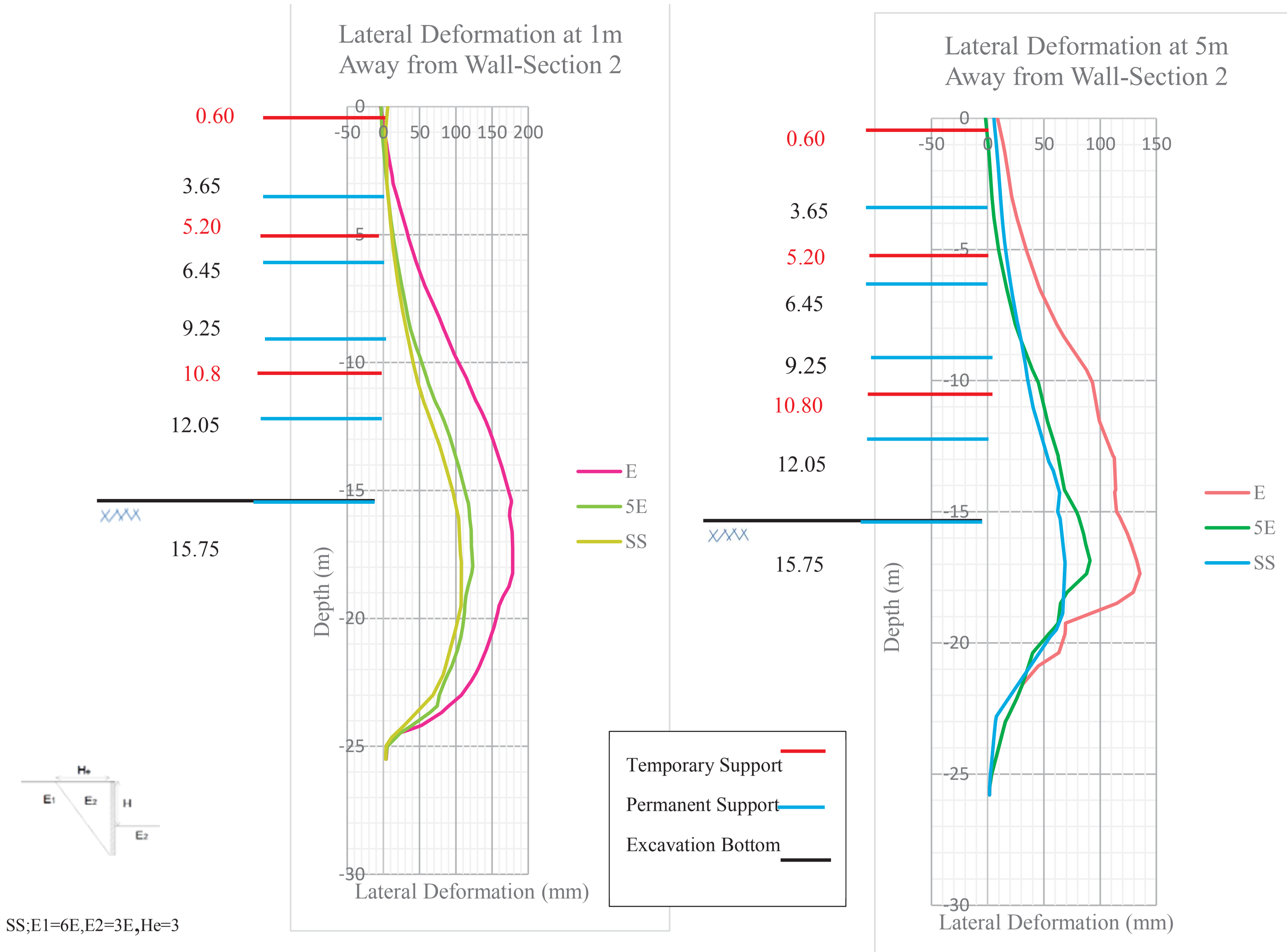


Figure 5.21

Lateral Deformations away from the Retaining Wall-Section 3

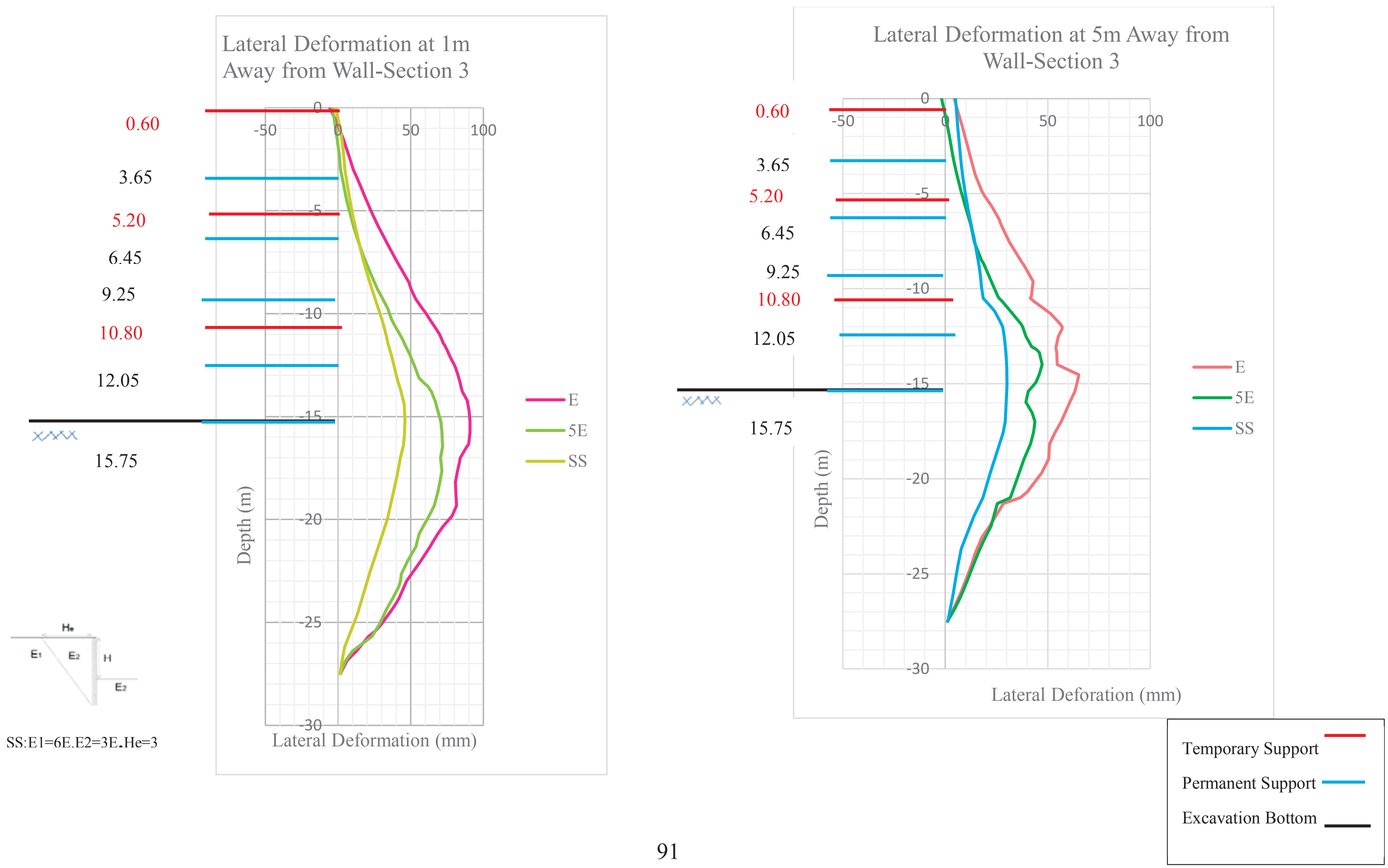
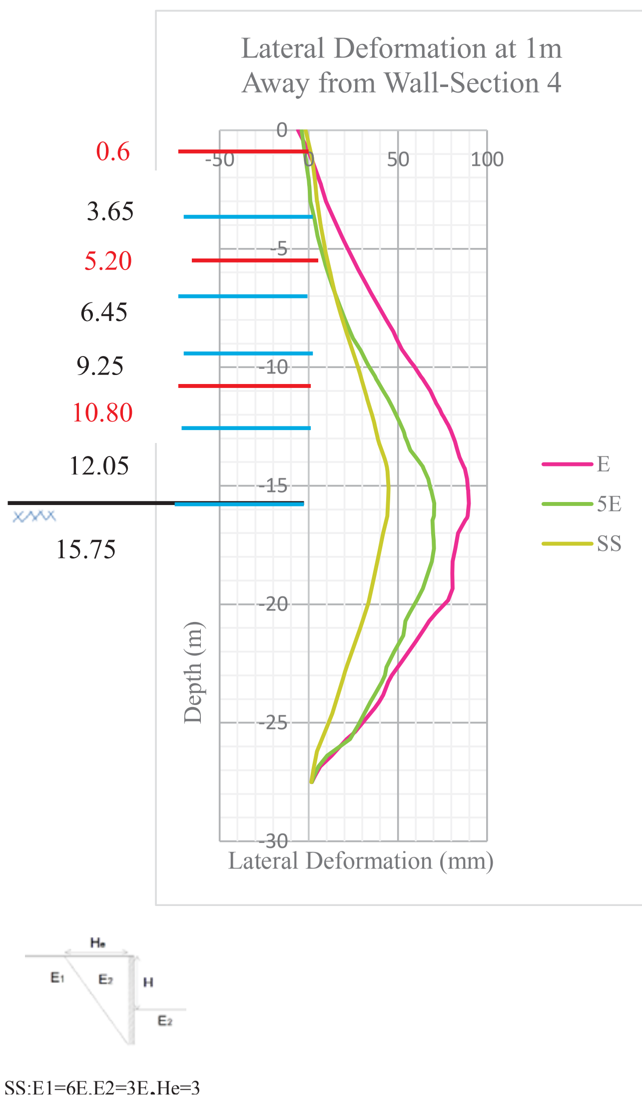
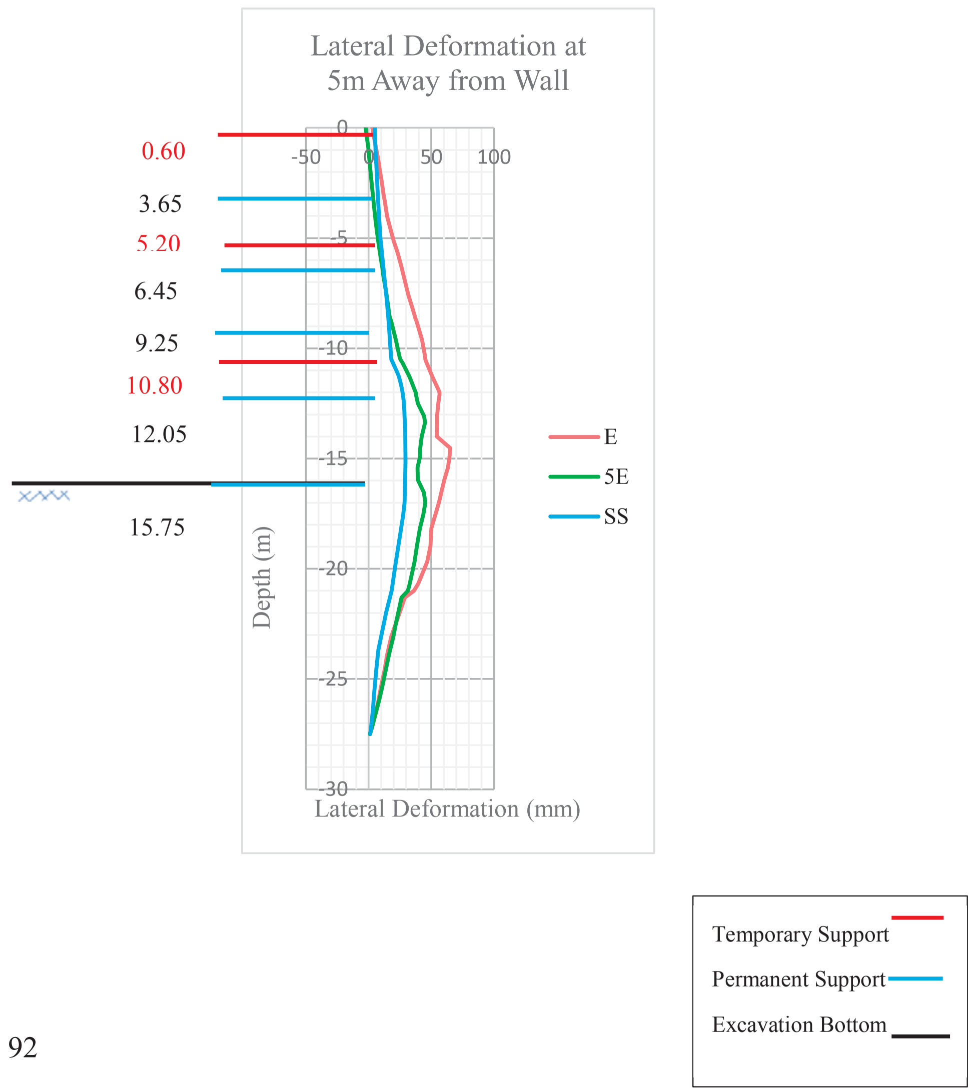


Figure 5.22 –
Lateral Deformations away from the Retaining Wall-Section 4





5.7. Comparison of Vertical Settlement

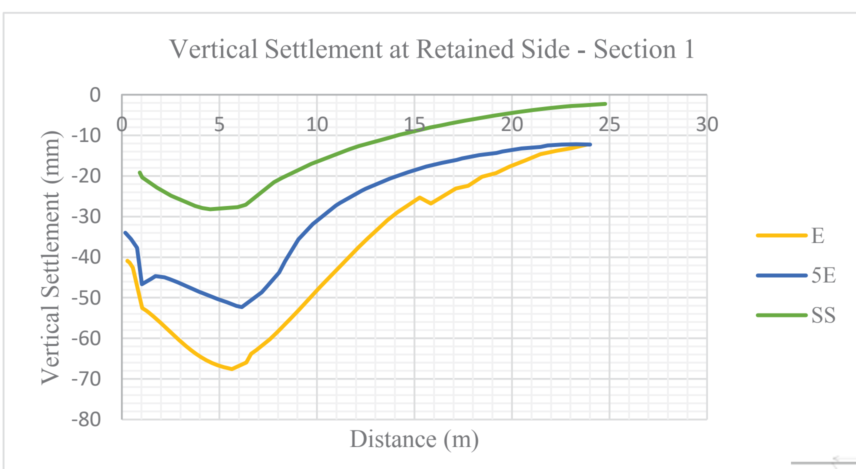
Vertical settlement at retained side was compared in final excavation level for all sections as the maximum lateral deformation is obtained at the final excavation. Settlement graphs are illustrated in **Figure 5.23**, **Figure 5.24**, **Figure 5.25** and **Figure 5.26**. In all graphs, vertical settlement is reducing with the modifications and minimum vertical settlement is obtained in **Modification 2**. Another thing is that, in all cases maximum vertical settlement has occurred nearly away 6m from the wall. Wang, 2021 illustrated that, when only the Mohr-Coulomb model is used, it will result high ground settlement as it gives much larger horizontal displacements. Also, it further highlighted that, HSS model is have a good agreement with field data and this indicate that rationality of considering small strain dependent behavior and the reliability of the adopted numerical simulations. Hsiung and Dao, 2015 elaborated that, use of small strain theory to analysis will result the better prediction if ground surface settlements.

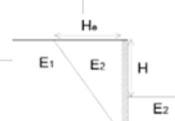
Nimr et at, 2022 illustrated that, at the initial stage where lateral supports are not installed the settlements can be represented in a triangular shaped distribution as wall behave like a cantilever. However, with the excavation proceed with the lateral supports, vertical settlements have a trapezoidal distribution due to effect of outward movement of retaining wall. As mentioned above the shape of the all curves have trapezoidal distribution.

Figure 5.23 illustrated the vertical settlement of section 1 in final excavation level. With the modifications the maximum vertical settlement has reduced. As most of adjacent structures are with a shallow footing, the reduced settlement after the modification is at an acceptable limit.

Figure 5.23

Vertical Settlement – Section 1



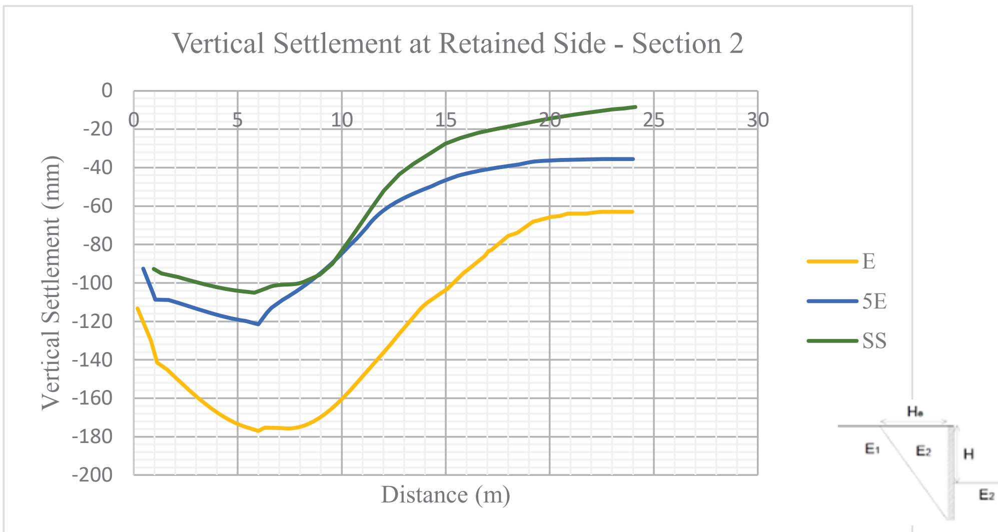


SS;E1=6E,E2=3E,He=3H

Figure 5.24 illustrated the vertical settlement of section 2 in final excavation level. With the modifications the maximum vertical settlement has reduced. However, there is no considerable reduction of settlement between **Modification 1** and **Modification 2** like Section 1. Moreover, the reduced settlements are higher to the acceptable limits.

Figure 5.24

Vertical Settlement – Section 2

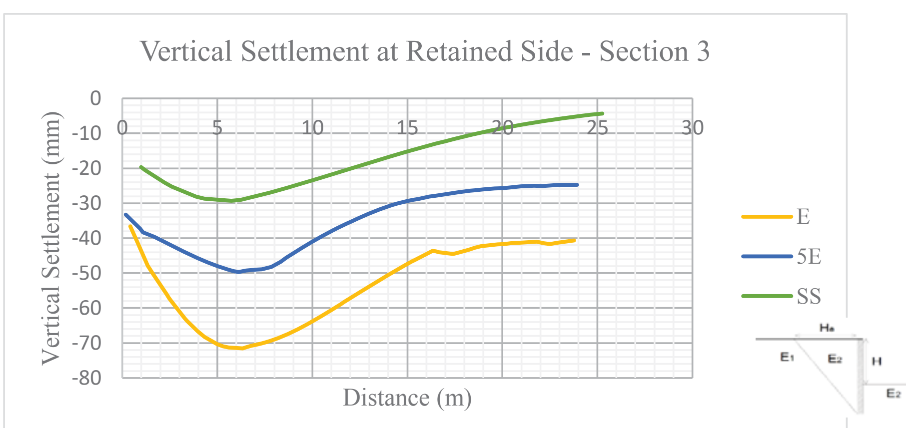


SS;E1=6E,E2=3E,He=3H

Figure 5.25 illustrated the vertical settlement of section 3 in final excavation level. With the modifications the maximum vertical settlement has reduced and within the acceptable limits as Section 1.

Figure 5.25

Vertical Settlement – Section 3

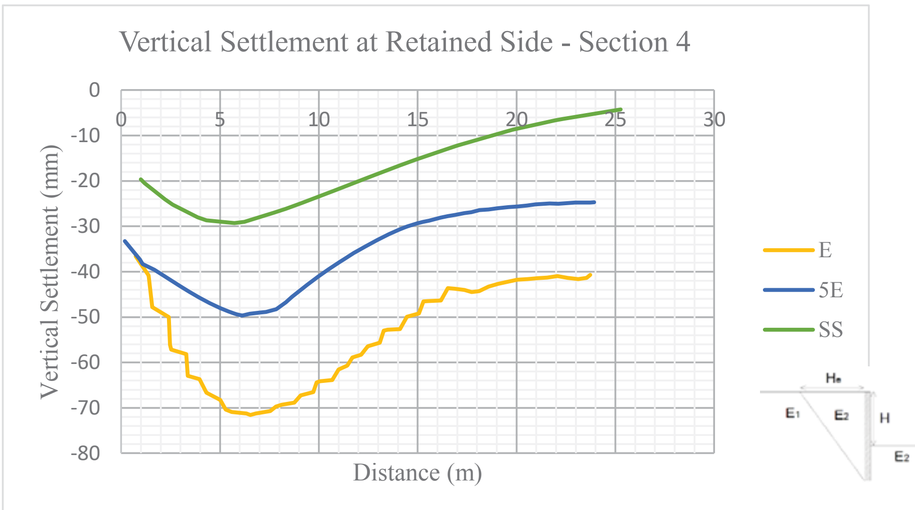


SS;E1=6E,E2=3E,He=3

Figure 5.26 illustrated the vertical settlement of section 4 in final excavation level. As Section 1 and Section 3 maximum vertical settlement has reduced with the modifications and within the acceptable limits.

Figure 5.26

Vertical Settlement – Section 4



SS:E1=6E.E2=3E.He=3

5.7 Horizontal Pressure Distribution Under Three Different Scenarios

Pressure distribution at retained side and pressure distribution at excavated side of the Section 1 at final excavation was obtained for all considered scenarios and presented in **Figure 5.28**.

According to the graph, the soil with **Modification 1** where soil stiffness has increased five times from initial value (5E) the wall deformations were reduced. As such, the reduction of earth pressures from the at rest state is much smaller and the earth pressure values in the retained side are much greater. Similarly, in front of the wall increase of lateral pressure from the at rest values is much smaller. As such, earth pressure values for the 5E case in much smaller than the case of E

Figure 5.27

Relative Earth Pressure Coefficient for Three Scenarios

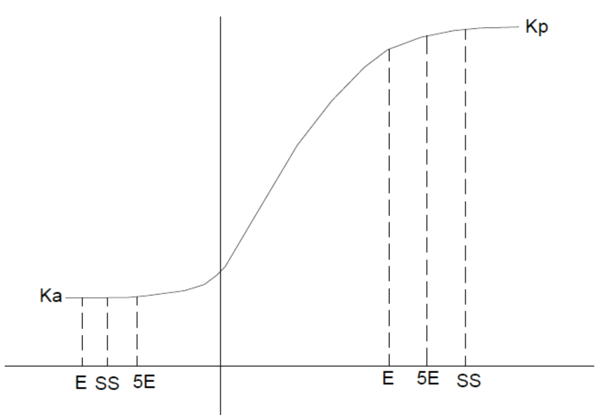
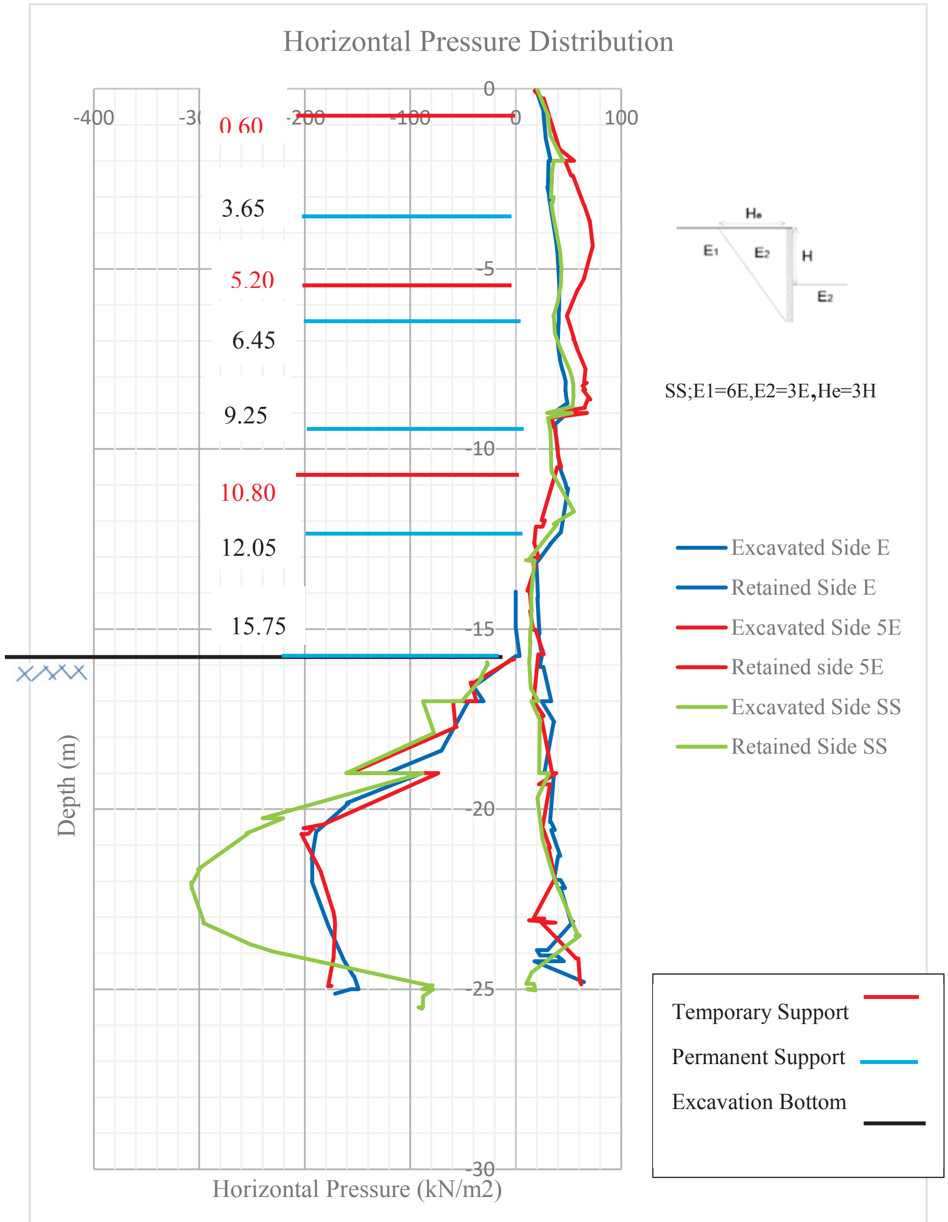


Figure 5.28

Pressure Distribution – Section 1

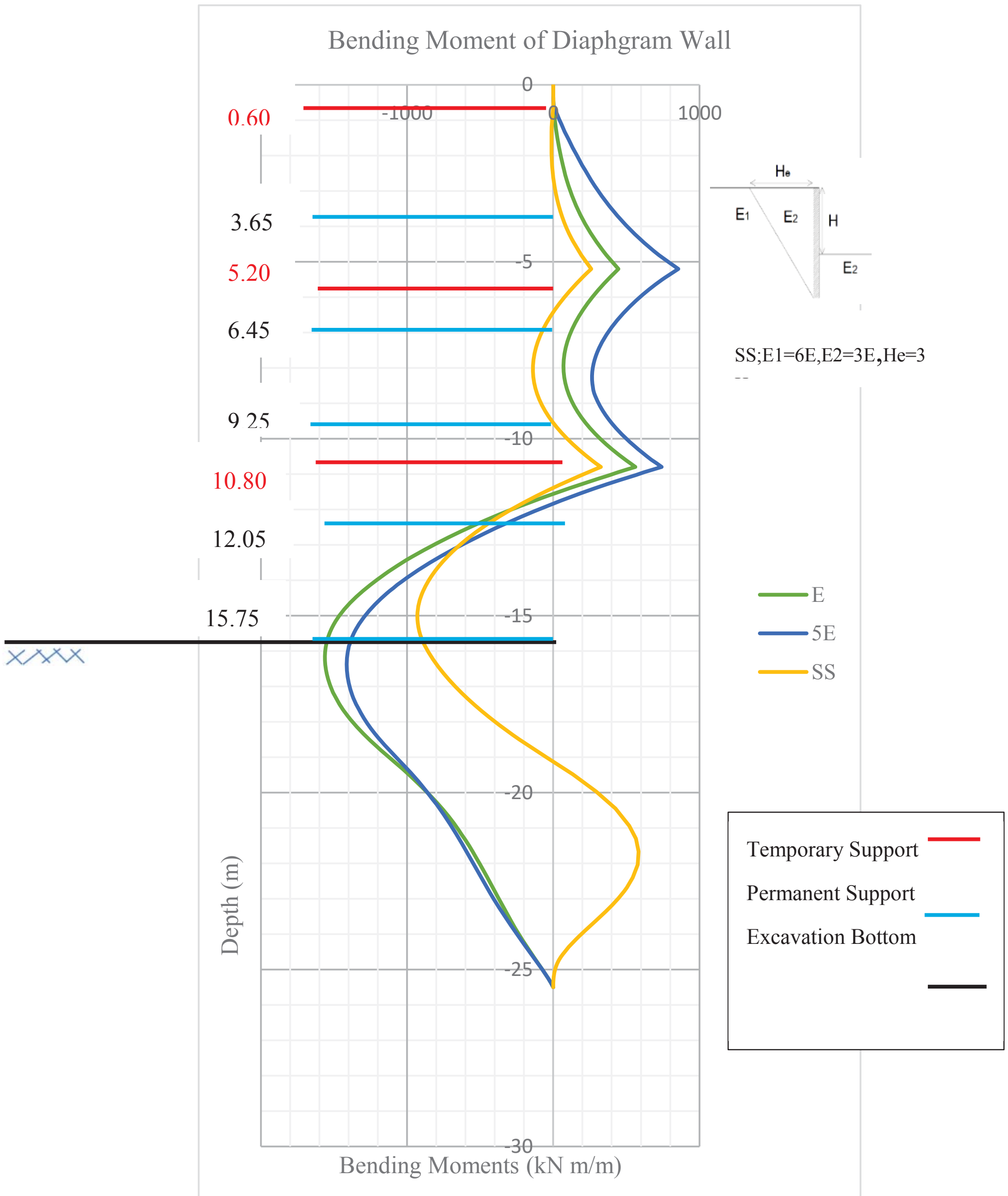


5.8. Comparison of Bending Moments of Diaphragm Wall

Bending moments of the diaphragm wall were compared for considered three scenarios of stiffness and graphs for Section 1 for the final excavation is presented in **Figure 5.29**. As there is no lateral support at the excavation bottom a larger bending moment can be observed there. Also, the bending moments have reduced with the increase of the assigned soil stiffness under SS condition.

Figure 5.29

Bending Moment of Diaphragm Wall – Section 1



CHAPTER 06

SUMMARY AND CONCLUSIONS

In this research the stresses and deformations during a 15.75 m deep excavation supported by a 0.6 m thick diaphragm wall with temporary supports at three levels during the excavation and permanent supports by concrete slabs at five levels were studied. The construction process was numerically simulated by 8 stages with the commercial Finite Element software PLAXIS. Mohr Coulomb model was used as the constitute model for the soil. As the stiffness parameters of soil, were not obtained through direct experimental procedures correlations with SPT were used.

The lateral deformations obtained from the initial model of the numerical analysis were found to be excessive based on the actual performance of the structure. Unfortunately, there were many shortcomings in the instrument process adopted and reliable information on deformations could not be obtained. Therefore, the model was improved and analysis was carried out again based on the guidance published in the literature regarding the stiffness parameters of the soil. Two modifications were adopted to the stiffness parameters (Modification 1 and 2) considering that soil elements are experiencing an unloading situation and the strains of the elements further away from the structure are small. With that considerable reductions were obtained in the predicted deformations except at one section. The most probable reason for this behavior in that section may be the presence of a clay layer at the bottom level of the excavation.

There were many shortcomings in the installation of the inclinometers for the monitoring of lateral deformations in the diaphragm wall. Inclinometers have not been installed up to the bottom level of the diaphragm wall but terminated at the bottom level of the excavation. While installing the inclinometers, the gap between the inclinometer tube and the borehole was filled with sand instead of grouting. This allows for relative displacements of the tube leading to many errors in the measurements. Also, the installation was not done at the start of the excavation but after excavation to a depth of 11.3m (After stage 4) in the modelling. As such the deformations recorded by the system were not reliable. Although there had been some strain gauges installed in the prop supports no data were available. As such, this research was mainly focused on a parametric study of the important parameters. The findings of this study were compared with published results from similar projects (Wang et.al., (2023)) where accurate instrumentation was done.

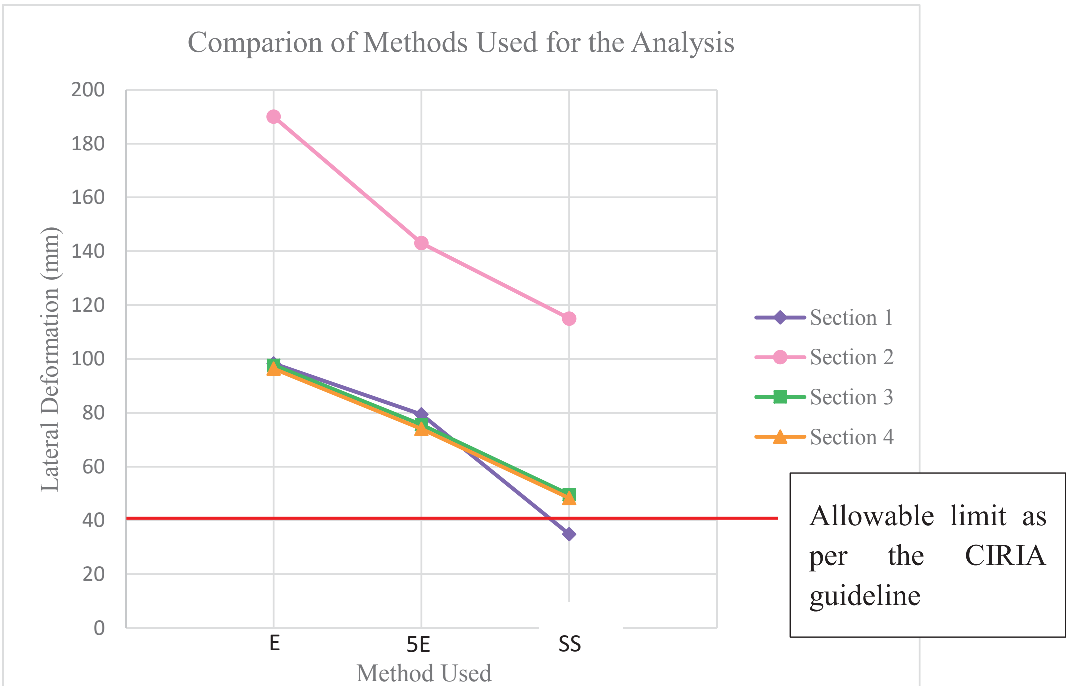
As such, one major lesson learned from this research study is the need to pay close attention to the process of installation of instrumentation.

The **Figure 6.1** illustrates the maximum lateral deformations of all four sections at the final excavation stage obtained from the numerical analysis under various scenarios. The lateral deformation at final excavation depth is closer to the limit acceptable under

CIRIA guidelines when the small strain concept (Modification 2) was used for the analysis (except at Section 2). The Small Strain concept appears to be more acceptable in modelling these types of excavations.

Figure 6.1

Comparison of lateral deformations of the wall at different sections



Lateral deformations in the soil on the retained side decreased with the distance from the wall. The vertical settlements of the ground were also decreasing with the distance from the wall. These values are lower under the Small Strain Concept. These findings made it clear that the designed and constructed earth retaining system has ensured that the deformations of the buildings in the surrounding site are kept within acceptable limits.

Lateral stress distribution on the wall on both the retained side and in front of the wall varied with the assumed stiffness condition. With the assumed Small Strain condition lateral stresses have reduced less from the at rest values on the retained side due to the smaller lateral movements.

Also, under the Small Strain concept, the bending moments in the wall were smaller.

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APPENDIX A

Vertical Sub-surface Profile

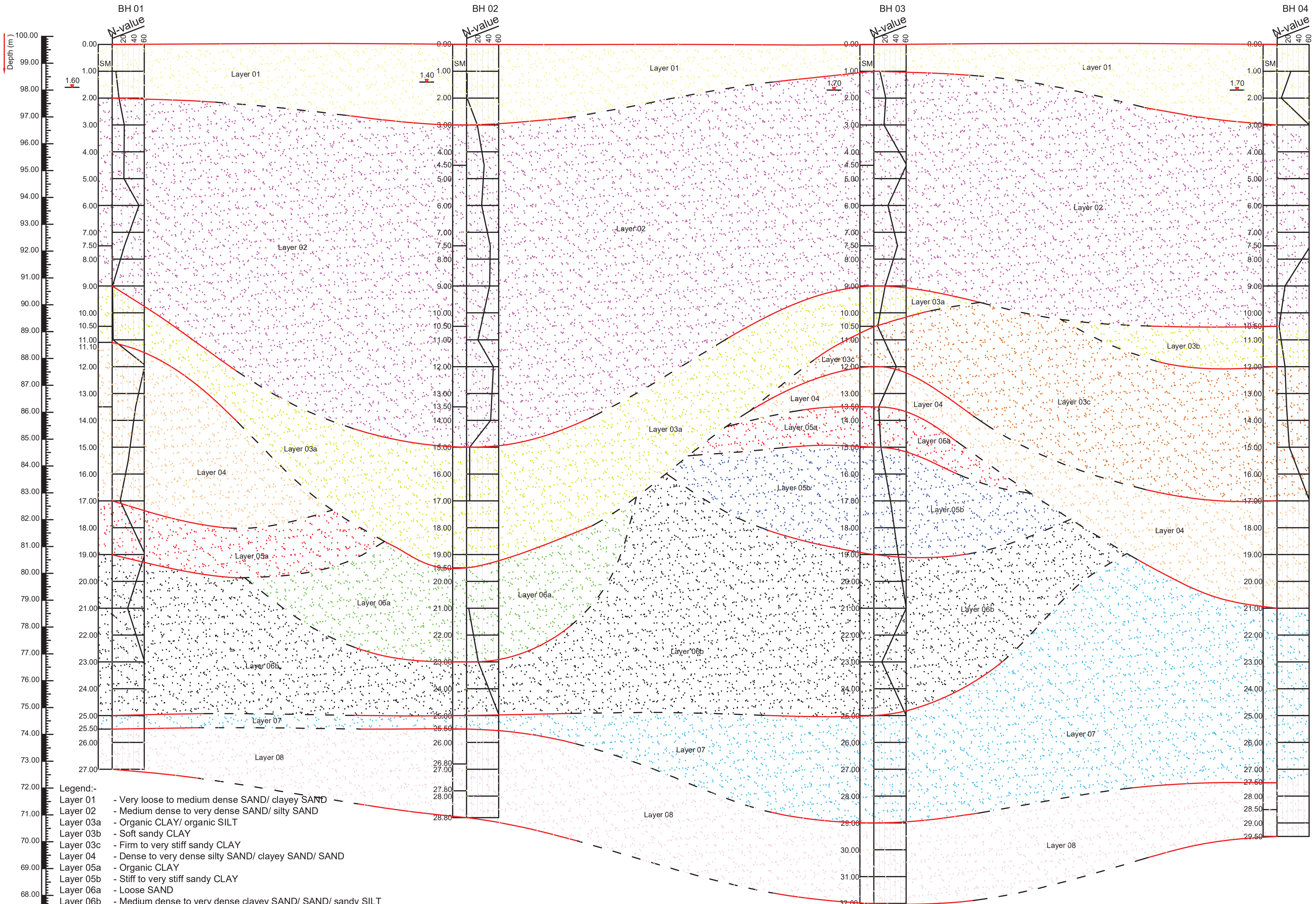

Figure 3.3 : - Assumed vertical subsurface profile through BH 01,BH 02, BH 03 & BH 04

APPENDIX B

Borehole Investigation Report

Geotechnical Engineering Division

Quality is Our Trademark

GEOTECHNICAL INVESTIGATION REPORT

ELS SI 3746

PROJECT

Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.

CLIENT

M/s. Odel PLC

November 25, 2015.

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			Document No		ELS/3746
0	25/11/2015	Soil Investigation Report at Colombo 07	M.A.D.N.Kumari	Prof.B.L.Tennekoon	R. M .Wasantha Rathnayake
Rev	**Date**	**Description**	**Prepared By**	**Foundation Recommendation given By**	**Approved By**

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FIGURES

ANNEXURE

GEOTECHNICAL INVESTIGATION FOR THE PROPOSED SIX STORIED BUILDING AT ALEXANDRA PLACE, COLOMBO 07

1.0 Introduction

The project involves the construction of twelve storied building which consists with three basement floors + Ground floor + 8 – Upper floors at Odel PLC premises at Alexandra place, Colombo 07.

M/s. Engineering and Laboratory Services (Pvt) Ltd. was authorized by **M/s. Odel PLC,** to carry out the soil investigation at the above site and prepare the soil investigation report with recommendations for foundation design.

2.0 Site Description

The proposed lad for the construction is situated at the premises of Odel PLC and adjacent 4 storied building at Alexandra place, Colombo 07.

The land is bounded by C.W.W.Kannangara Mawatha in West and ward place in North. There are few multi storied buildings surrounding the site and the land is bounded by a wall

Figure 1(a): Investigated area

The drilling of boreholes was carried out under sunny and rainy weather conditions.

3.0 Field Investigation

3.1 Borehole Investigation

The field investigation was consisted of advancing six boreholes at the location marked as BH-01 to BH-06 in Figure 1(b).

The boreholes were advanced by means of a rotary - drilling machine. The drilling was carried out with overburden cutting tools, and the wash boring process was adopted to remove the cuttings from the bottom of the borehole.

Standard Penetration Test (SPT) was carried out in regular intervals in the overburden. This test was carried out as specified in BS 1377.

Disturbed samples of soil were collected both from the SPT tube and the cuttings were collected from the washings.

Groundwater Level (GWL) was determined as the depth at which the water level stabilized inside the borehole.

3.2 Field Permeability Test

Field permeability tests were carried out in investigated area to determine hydraulic conductivity, a measure of rate of flow. The determination in – situ permeability was done Variable Head Method as specified in Section 4: 25.4 of BS: 5930:1999The result of the field permeability test is given in the Annexure I.

Table 1: Summary of field permeability test

Borehole No	Depth (m)	Permeability (m/sec)
BH - 01	3.0 – 3.5	1.1 E-5
BH - 03	9.0 – 9.5	2.26 E-6
	10.0 – 10.5	2.7 E-8
BH - 04	8.0 – 8.5	6.8 E-7
	12.0 – 12.5	2.9 E-6
BH - 05	9.0 – 9.5	7.7 E-7
	12.5 – 13.0	8.8 E-7

4.0 Laboratory Investigation

Laboratory investigations were taken place in order to the sub surface assessment in geotechnical investigation. In connection with the entire laboratory testing the performance has been made as per BS 1377 unless otherwise stated. Laboratory investigations were taken place in order to assess the sub surface conditions within the site area. The detailed results of the laboratory investigation will be submitted as Report II

4.1 Laboratory Tests on Soil Samples

- Particle size distribution
- Natural Moisture Content
- Specific Gravity
- Consolidation
- Unit weight
- Triaxia (UU) Test
- Atterberg Limit

4.2 Laboratory Tests on Water Samples

- Chemical Analysis on water

4.3. Laboratory Tests on Rock Samples

- Unconfined Compressive Strength of Rock

5.0 Subsurface Conditions

The results of the borehole investigation are given in Annexure II.

Using this, profiles of subsurface conditions across the boreholes have been constructed and these are shown in Figure 2.

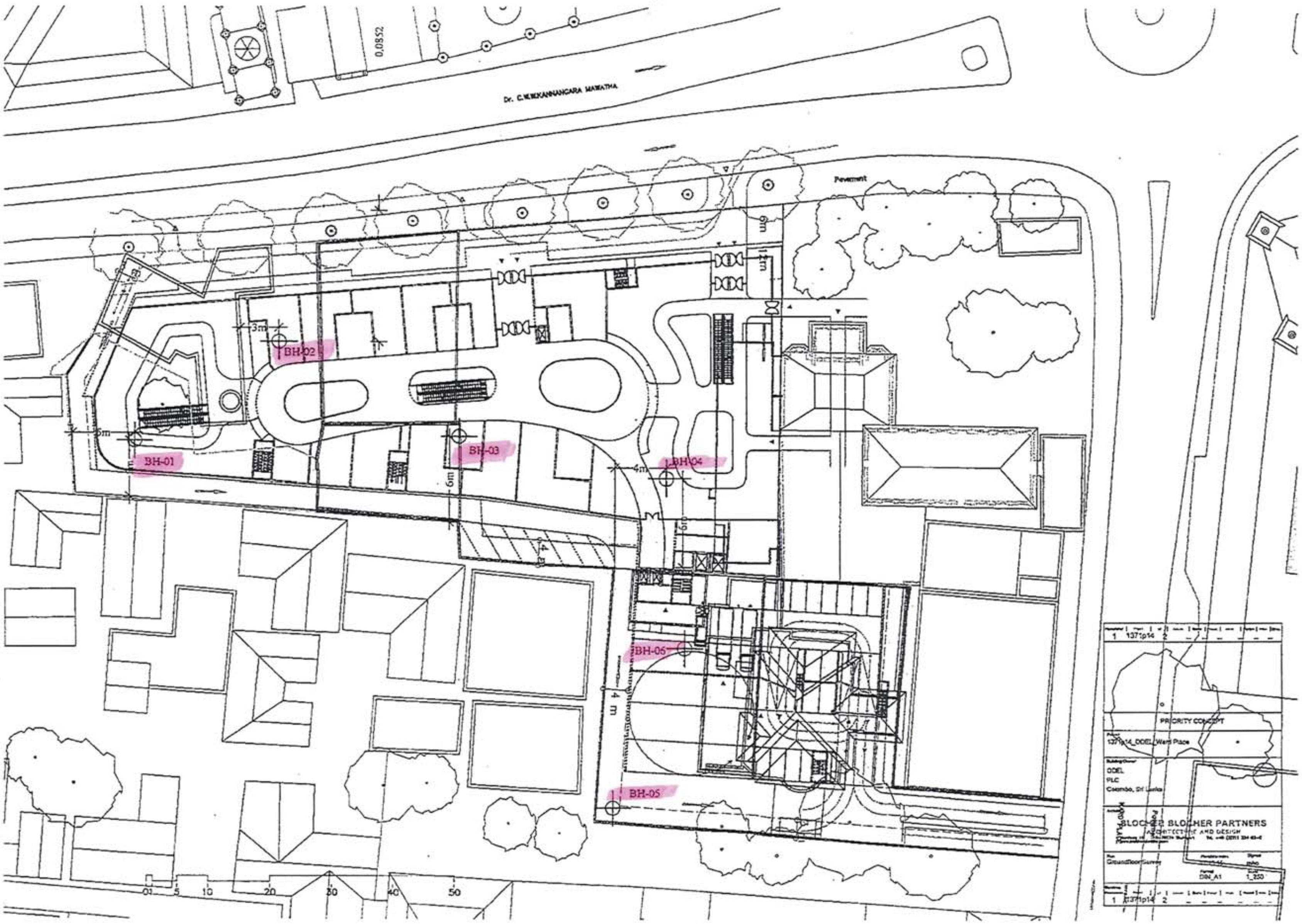


Figure 1(b): Location of the Boreholes

NOTATION:-
BH-BORE HOLE

BH-01

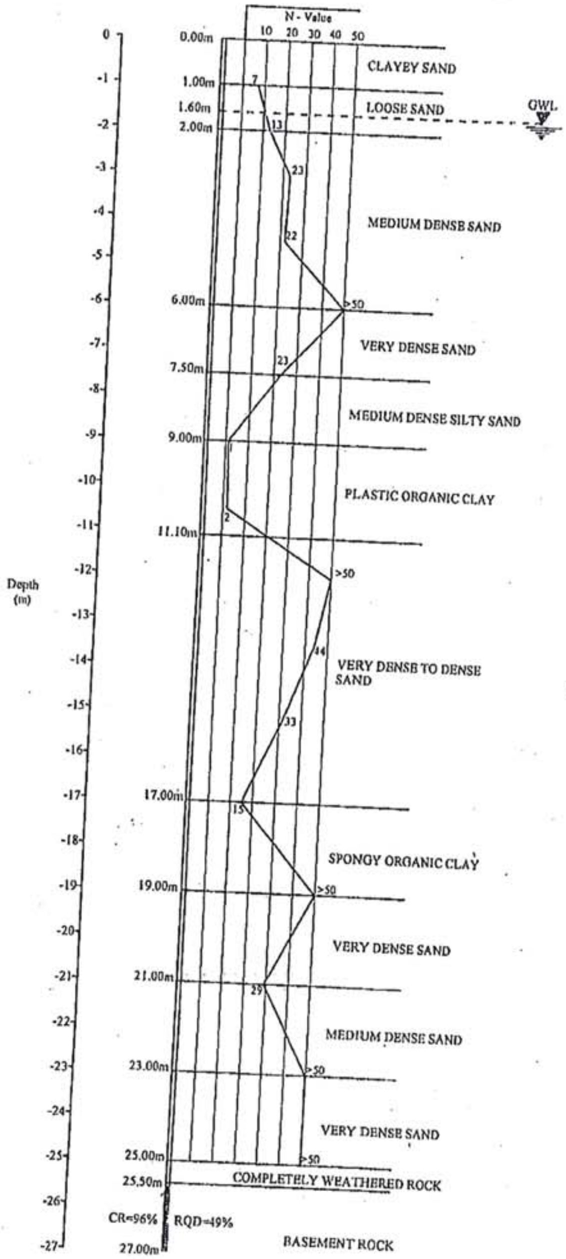


Figure 2(a): Possible Soil Profile at the borehole BH-01	Project: Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Vertical scale 1:150

Ac: Profile is plotted only according to the data obtained from the borehole locations and actual soil profile may vary from this profile

BH-02

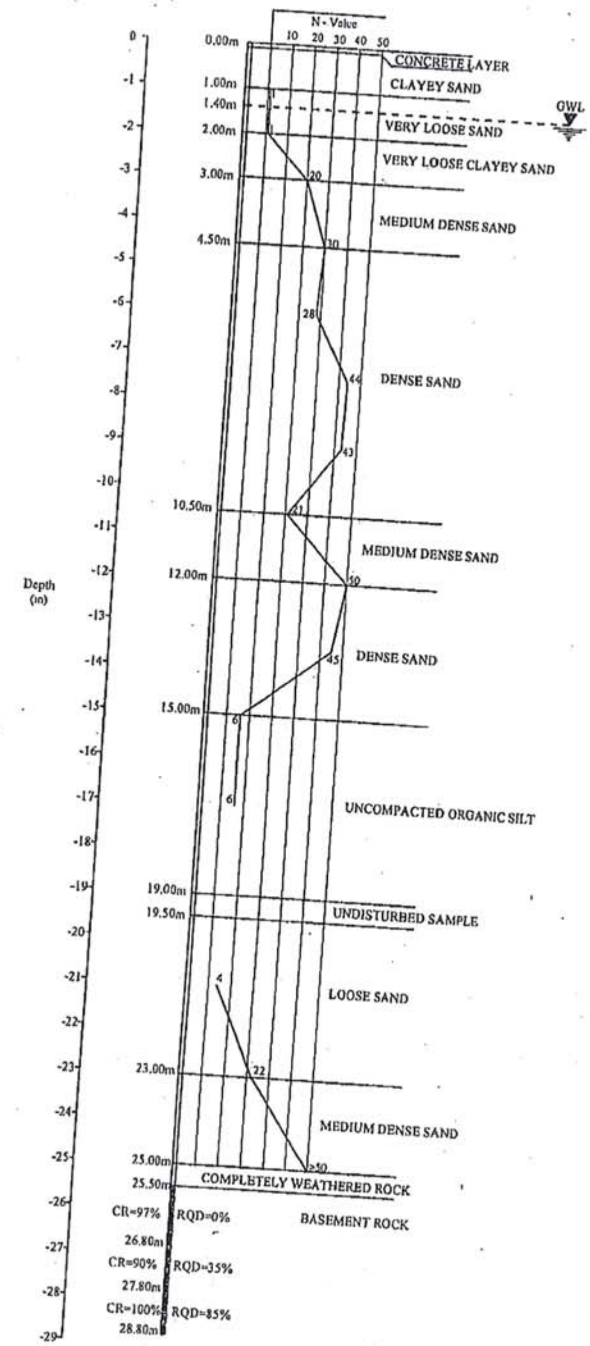


Figure 2(b): Possible Soil Profile at the borehole, BH-03	Project: Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07	Vertical scale 1:150

Profile is plotted only according to the data obtained from the borehole location and actual soil profile may vary from this profile

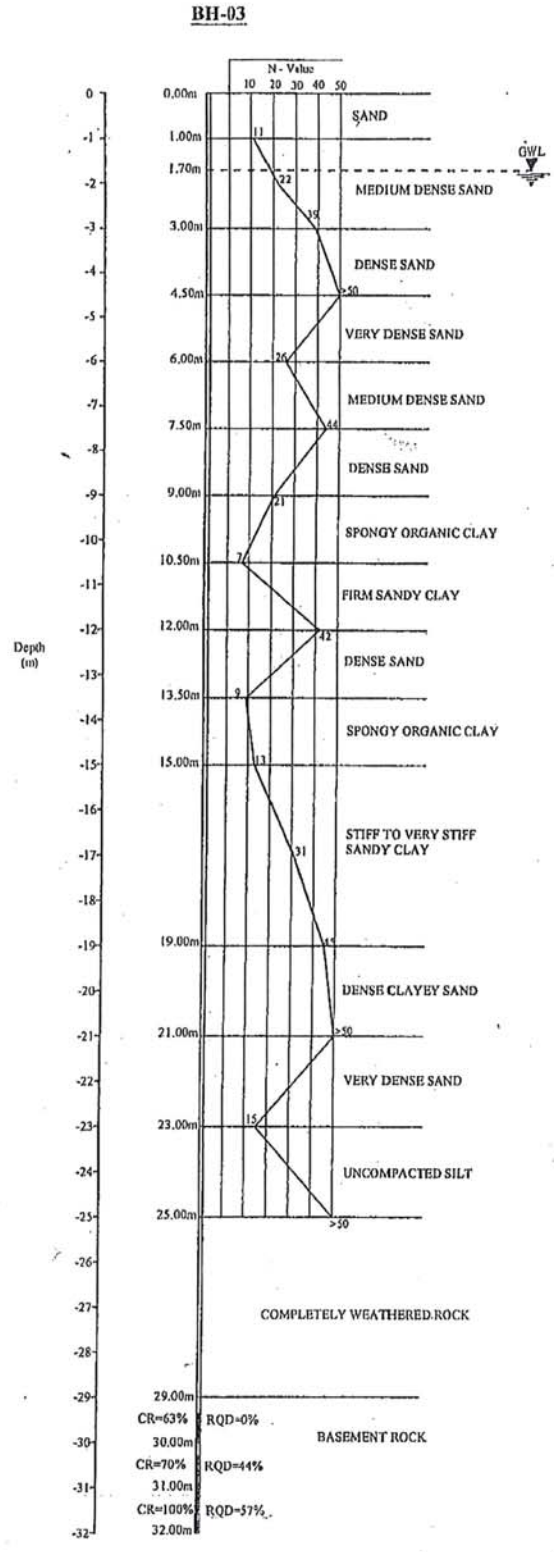


Figure 2(c): Possible Soil Profile at the borehole, BH-03	Project: Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Vertical scale 1:150

Note: Profile is plotted only according to the data obtained from the borehole locations and actual soil profile may vary from this profile

BH-04

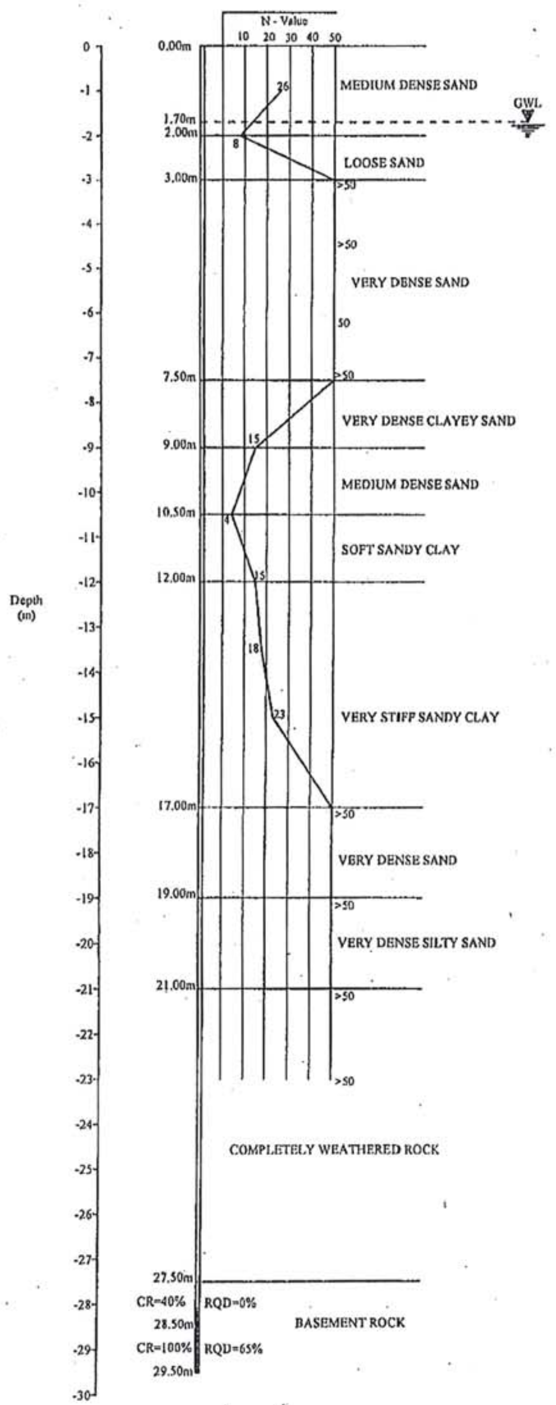


Figure 2(d): Possible Soil Profile at the borehole, BH-04	Project: Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Vertical scale 1:150

Note: Profile is plotted only according to the data obtained from the borehole locations and actual soil profile may vary from this profile

BH-05

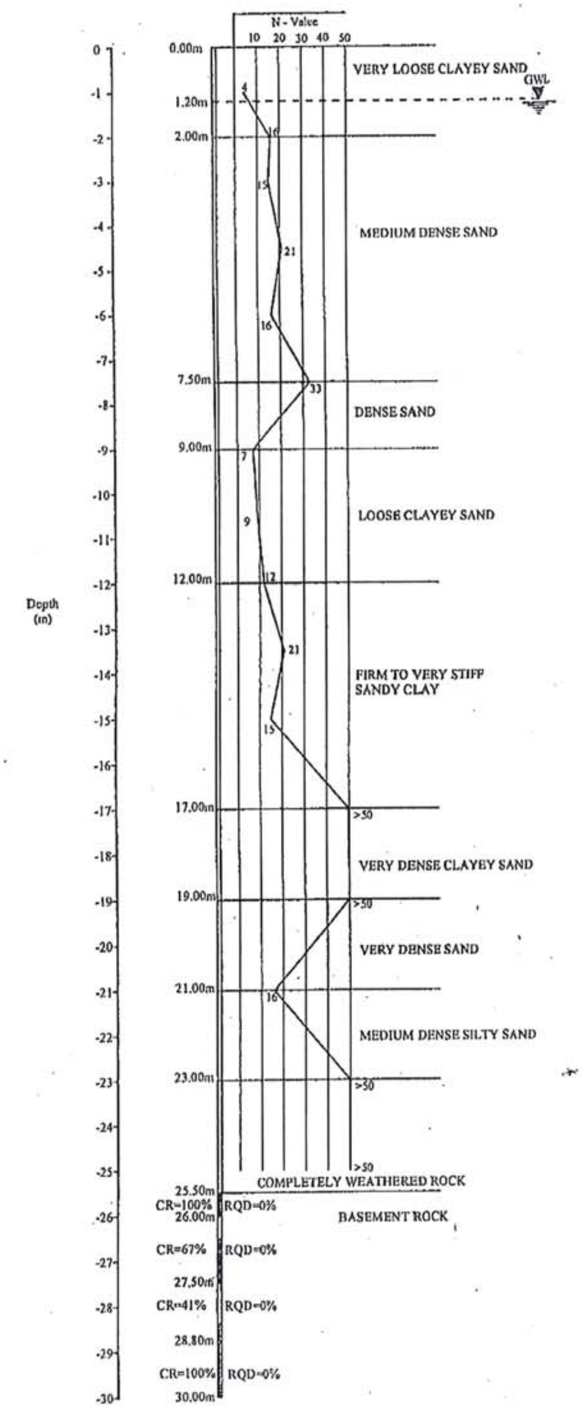


Figure 2(e): Possible Soil Profile at the borehole, BH-05	Project: Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Vertical scale 1:150

Note: Profile is plotted only according to the data obtained from the borehole locations and actual soil profile may vary from this profile

BH-06

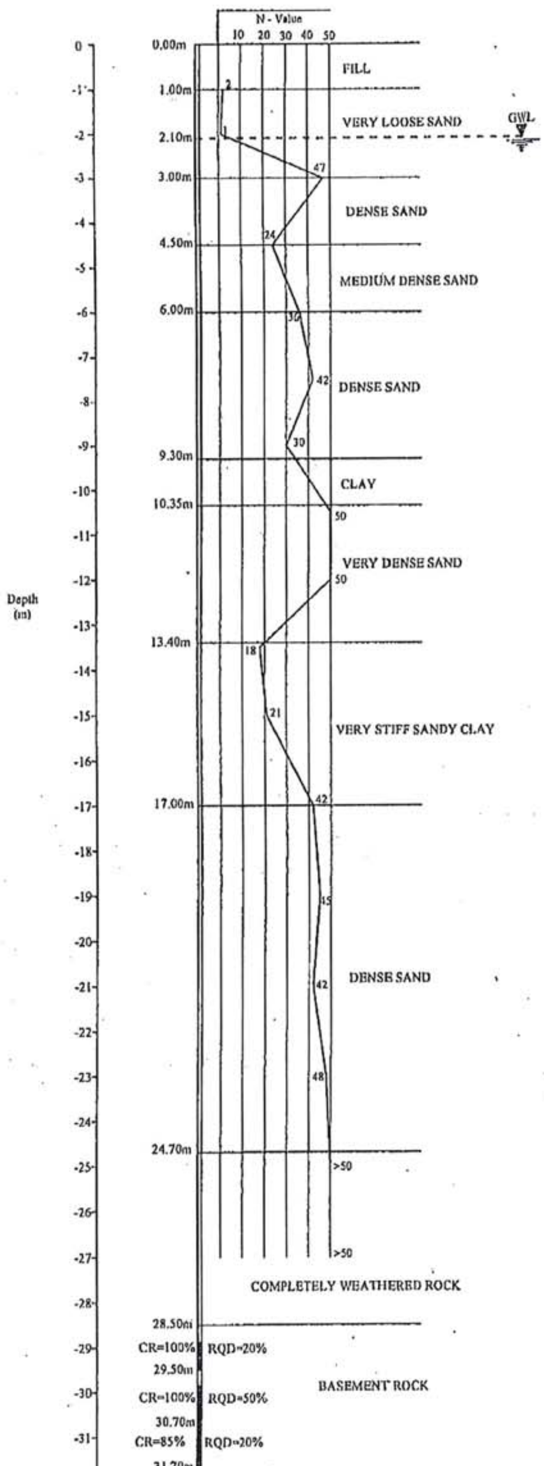


Figure 2(f): Possible Soil Profile at the borehole, BH-06	Project: Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Vertical scale 1:150

Note: Profile is plotted only according to the data obtained from the borehole location and actual soil profile may vary from this profile

6.0 Recommendations for foundations of proposed 12-Level building at Alexandra Place, Colombo 07 for Odel

This report should be read in conjunction with the Factual Report of the Soil Investigation carried out at the site and reported in November 2015.

6.1 Project & Site details

Odel PLC proposes to construct a 12-Level building at Alexandra Place, Colombo 07. The building is to comprise of 3 basement floors, ground floor and eight upper floors.

The site has two road frontages; one to C.W.W. Kannangara Road in Western direction, and the other to Ward Place in Northern direction. The Layout Plan for the proposed development together with the positions of the borehole locations are shown in Fig. 1a.

The building is proposed to be constructed in two phases. The section of building covered by BH-01 to BH-04 will be constructed in the first stage; and the section of building covered by BH-05 and BH-06 will be constructed in the second stage.

The area is a relatively flat land which is situated in a highly urbanized area. The few temporary buildings and existing four storied building in southern area of the site will be demolished before construction.

6.2 Borehole Investigations

Six nos. boreholes were advanced at locations BH-01 to BH-06, as indicated previously. All boreholes were initially advanced up to hard rock. Thereafter, they were further advanced by coring the rock using a double tube core barrel. Details of the depths of drilling, including the depth to Ground Water Level (GWL) are indicated below.

Table 2: Details of the depths of drilling

Location	BH-01	BH-02	BH-03	BH-04	BH-05	BH-06
Depth to GWL (m)	1.6	1.4	1.7	1.2	1.2	2.1
Depth to rock (m)	25.5	25.5	29.0	27.5	25.5	28.5
Depth of borehole (m)	27.0	28.8	32.0	29.5	30.0	31.7

Rock coring in the boreholes had given the following results for the Core Recovery (CR) and the Rock Quality Designation (RQD):

Table 3: Rock coring parameters

Borehole No	Depth (m)	Rock Coring Parameters	
		Core Recovery (%)	RQD (%)
BH-01	25.5 – 27.0	96	49
BH-02	25.5 – 26.8	97	00
	26.8 – 27.8	90	35
	27.8 – 28.8	100	85
BH-03	29.0 – 30.0	63	00
	30.0 – 31.0	70	44
	31.0 – 32.0	100	57
BH-04	27.5 – 28.5	40	00
	28.5 – 29.5	100	65
BH-05	25.5 – 26.0	100	00
	26.0 – 27.5	67	00
	27.5 – 28.8	41	00
	28.8 – 30.0	100	00
BH-06	28.5 – 29.5	100	20
	29.5 – 30.7	100	50
	30:7 – 31.7	85	20

Three Nos. of rock cores were tested for their uniaxial compressive strength (UCS). These results are given below.

Table 4: Uniaxial compressive strength of rock cores

Borehole No.	Depth (m)	UCS (N/mm^2)
BH-02	27.80 – 27.95	24.34
BH-03	31.45 – 31.60	22.24
BH-04	28.75-28.90	97.08

6.3 Sub-surface conditions

From a study of the borehole logs, it is concluded that the overburden at BH-01 can be modeled by successive layers as indicated below.

Table 5: Successive layers of BH-01

Layer No.	Position (m)	Layer description	Average SPT
1a	0.0 – 2.0	Loose sand	7
1b	2.0 – 6.0	Dense sand	20
2c	6.0 – 7.5	Very dense sand	>50
3a	7.5 – 23.0	Completely weathered rock - I (*)	25-40
3b	23.0 – 25.5	Completely weathered rock - II	>50
4	>25.5	Basement rock	

(*) Layer No. 3a had intrusions of peaty clay; first between depths of 9.0m and 11.1m, and a second intrusion between depths of 17.0m and 19.0m.

Ground water level (GWL) was at a depth of 1.6m from the surface.

Similar layering was observed at the other borehole locations.

6.4 Recommendations for geotechnical design parameters

Recommendations for the geotechnical design parameters for sub-surface layers are given in the table below based on the visual observation of the soil samples collected, the measured SPT values, and experience with similar sub-surface conditions.

Table 6: Geotechnical design parameters

Layer description	Average SPT	Shear strength parameters	Ultimate bearing capacity (kN/m^2)	Elastic Modulus E (kN/m^2)
Loose sand	7	c' = 0 kPa, φ' = 26	150	7000
Dense sand	20	c' = 0 kPa, φ' = 33	420	20000
Very dense sand	> 50	c' = 0 kPa, φ' = 43	2000	25000
Completely weathered rock - I	25-40	c' = 10 kPa, φ' = 30	530	20000
Completely weathered rock - II	> 50	c' = 10 kPa, φ' = 38	1500	25000
Organic peaty clay	-	$\Phi_u = 0$, c_u = 15 kPa	110	1000

(kg/m3)
BD (Intern
1600

6.5 Recommendations for sub-surface construction

6.5.1 Factors affecting sub-surface construction

The significant factors affecting the sub-surface construction are:

a) The building is to be provided with three basements;

b) The overburden consisting of successive layers of 'dense sand' and 'very dense sand' between depths of 2.0m and 7.5m;

c) The presence of intrusions of peaty clay within the 'completely weathered formation' present below 7.5m depth;

d) The GWL was at a depth of around 1.6 m; and

e) The loads to be transferred are from a 12-level structure.

6.5.2 Recommendations for basement construction

Construction of the proposed structure with three basements will pose large problems to the contractor for the reasons indicated previously in Section 6.5.1. Therefore, it is recommended that an initial proposal with costing be obtained for the basement construction.

6.5.3 Recommendations for foundation design

It is recommended that bored and cast in-situ Reinforced Concrete pile foundations, which are socketed in to the fresh rock be used.

It has been established that the ultimate end bearing capacity for piles in rock is related to the Rock Quality (shown as RQD during rock coring), and the Uniaxial Compressive Strength (UCS) of rock cores.

Three measurements of UCS values have indicated widely varying values. Therefore, adopting a conservative approach, it is recommended that initial designs be carried out assuming a net allowable end bearing capacity of 2 N/mm^2.

It is also recommended that 4 N/mm^2

(i) A minimum of one pile diameter of rock socketing in to **hard fresh rock** is ensured;

(ii) The carrying capacity in skin friction be considered within the basement rock, for which the ultimate skin friction coefficient (f_u) can be taken as 100 kN/m^2. This is less than that recommended in the ICTAD Guidelines (*).

(*) ICTAD publication ICTAD/DEV/15 on "Guidelines for interpretation of Site Investigation data for estimating the carrying capacity of piles for the design of bored and cast in-situ RC piles".

B. L. Tennekoon

Prof. B.L. Tennekoon
Emeritus Professor,
University of Moratuwa

25th November 2015

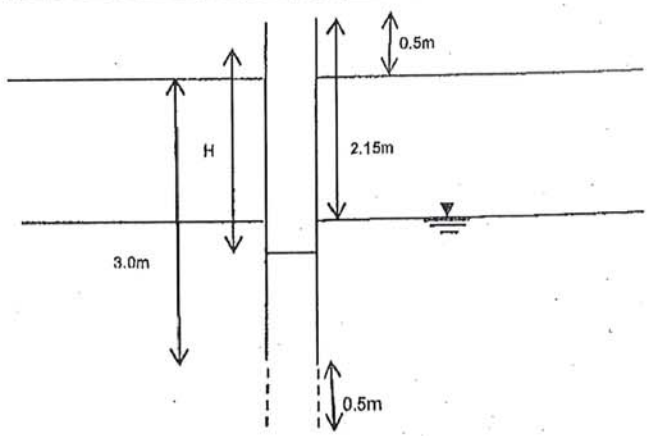
Annexure I: Field permeability test results

FIELD PERMEABILITY TEST

Variable Head

Client:	M/s. Odel PLC	Job ref.	ELS / 3746
		Client ref.	
Project::	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Hole No.	BH-01
Depth(m)	3.0-3.5		
Location::	Alexandra Place, Colombo 07.	Date of test	12/11/2015
Soil description:	Clayey sand	Date of report	12/11/2015
Diameter of Casing:	7.6 cm A = 45.3647 L = 50 cm	F =	121.64

Time	Elapse Time	Mesured level	H
	min	cm	cm
11.00am	0.00	0.0	215.0
	1.00	81.0	134.0
	2.00	122.0	93.0
	3.00	151.0	64.0
	4.00	172.0	43.0
	5.00	185.0	30.0
	10.00	208.0	7.0
	15.00	212.0	3.0
	20.00	213.0	2.0
	25.00	214.0	1.0
	30.00	215.0	0.0
	35.00	215.0	0.0
	40.00	215.0	0.0
	45.00	215.0	0.0
	50.00	215.0	0.0
	60.00	215.0	0.0
	70.00	215.0	0.0



$$F = \frac{2\pi L}{\log_e\left[\left(L/D\right) + \sqrt{1 + \left(L/D\right)^2}\right]}$$

$$k = \frac{A}{F(t_2 - t_1)} \log_e \frac{H_1}{H_2}$$

$H_1 = 134\ cm ; t_1 = 1\ \text{min}$

$H_2 = 2cm ; t_1 = 25\ \text{min}$

$k = 6.5 * E^{-2} cm / \text{min}$

$k = 1.1 * E^{-5} m/s$

Where;

K is the permeability of soil;

H1 is the variable head measured at time t1 after commencement of test;

H2 is the variable head measured at time t2 after commencement of test;

A is the cross sectional area of borehole casing;

Tested by .... Checked by .... Approved by ....

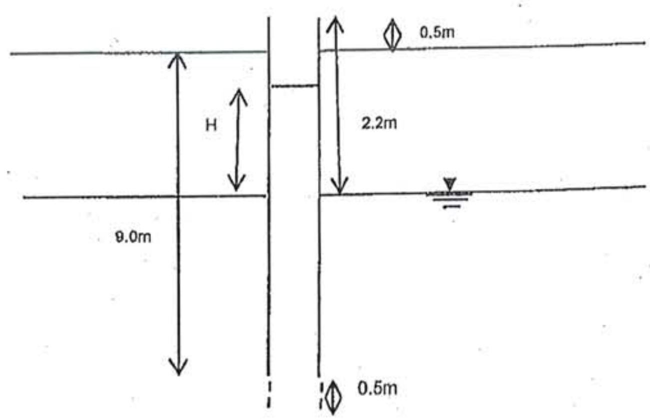
ENGINEERING & LABORATORY SERVICES (PVT) LTD
62/3, Neelammahara Road, Katuwawala, Boralesgamuwa, Sri Lanka.
Telephone : 0094 01 517037 / 517365 / 519727, Fax : 0094 01 509806, E-Mail : els@lanka.ccom.

FIELD PERMEABILITY TEST

Variable Head

Client:	M/s. Odel PLC	Job ref.	ELS / 3746
		Client ref.	
Project.:	Geotechnical Investigation for the Proposed Six Storied Building at Alexandra Place, Colombo 07.	Hole No.	BH-03
Depth(m)	9.0-9.5		
Location:	Alexandra Place, Colombo 07.	Date of test	12.11.2015
Soil description:	Spongy Organic Clay	Date of report	12.11.2015
Diameter of Casing:	7.6 cm, A = 45.3647, L = 50 cm	F =	121.64

Time	Elapse Time	Mesured level	H
	min	cm	cm
8.00am	0.00	0.0	220.0
	1.00	12.0	208.0
	2.00	25.0	195.0
	3.00	36.0	184.0
	4.00	43.0	177.0
	5.00	52.0	168.0
	10.00	82.0	138.0
	15.00	105.0	115.0
	20.00	121.0	99.0
	25.00	133.0	87.0
	30.00	142.0	78.0
	35.00	149.0	71.0
	40.00	153.0	67.0
	45.00	157.0	63.0
	50.00	159.0	61.0
	60.00	163.0	57.0



$$F = \frac{2\pi L}{\log_e\left[\left(\frac{L}{D}\right) + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]}$$

$$k = \frac{A}{F(t_2 - t_1)} \log_e \frac{H_1}{H_2}$$

$H_1 = 208cm; t_1 = 1\min$

$H_2 = 87cm; t_2 = 25\min$

$k = 1.35 * E^{-2} cm / \min$

$k = 2.26 * E^{-6} m / s$

Where;

K is the permeability of soil;

H1 is the variable head measured at time t1 after commencement of test;

H2 is the variable head measured at time t2 after commencement of test;

A is the cross sectional area of borehole casing;

Tested by .... Checked by .... Approved by ....

ENGINEERING & LABORATORY SERVICES (PVT) LTD

62/3, Neelammahara Road, Katuwawala, Boralesgamuwa, Sri Lanka.

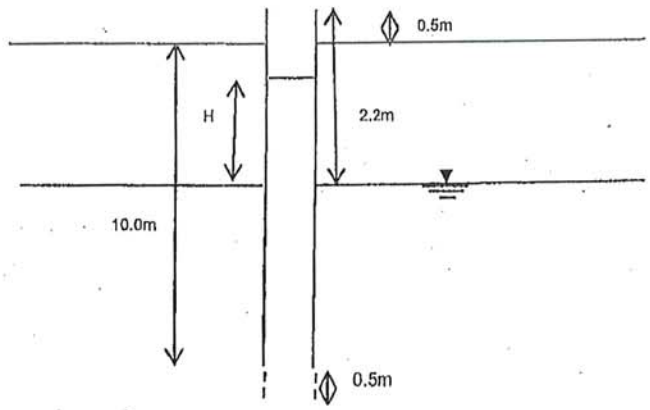
Telephone : 0094 01 517037 / 517365 / 519727, Fax : 0094 01 509806, E-Mail : els@lanka.ccom.

FIELD PERMEABILITY TEST

Variable Head

Client:	M/s. Odel PLC	Job ref.	ELS / 3746
		Client ref.	
Project.:	Geotechnical Investigation for the Proposed Six Storied Building at Alexandra Place, Colombo 07.	Hole No.	BH-03
Depth(m)	10.0-10.5		
Location:.	Alexandra Place, Colombo 07.	Date of test	20.11.2015
Soil description:		Date of report	20.11.2015
Diameter of Casing: 7.6 cm	A = 45.3647 L = 50 cm	F =	121.64

Time	Elapse Time min	Mesured level cm	H cm
8.00am	0.00	0.0	220.0
	1.00	0.0	220.0
	2.00	0.0	220.0
	3.00	0.0	220.0
	4.00	0.0	220.0
	5.00	0.0	220.0
	10.00	0.0	220.0
	15.00	0.0	220.0
	20.00	1.0	219.0
	25.00	1.0	219.0
	30.00	1.0	219.0
	35.00	1.0	219.0
	40.00	1.0	219.0
	45.00	2.0	218.0
	50.00	2.0	218.0
	60.00	2.0	218.0



$$F = \frac{2\pi L}{\log_e\left[\left(L/D\right) + \sqrt{1+\left(L/D\right)^2}\right]}$$

$$k = \frac{A}{F(t_2 - t_1)} \log_e \frac{H_1}{H_2}$$

$H_1 = 220cm; t_1 = 4\min$

$H2 = 218cm, t_1 = 25\min$

$k = 1.62 * E^{-4} cm/\min$

$k = 2.7 * E^{-8} m/s$

Where;

K is the permeability of soil;

H1 is the variable head measured at time t1 after commencement of test;

H2 is the variable head measured at time t2 after commencement of test;

A is the cross sectional area of borehole casing;

Tested by .... Checked by .... Approved by ....

ENGINEERING & LABORATORY SERVICES (PVT) LTD

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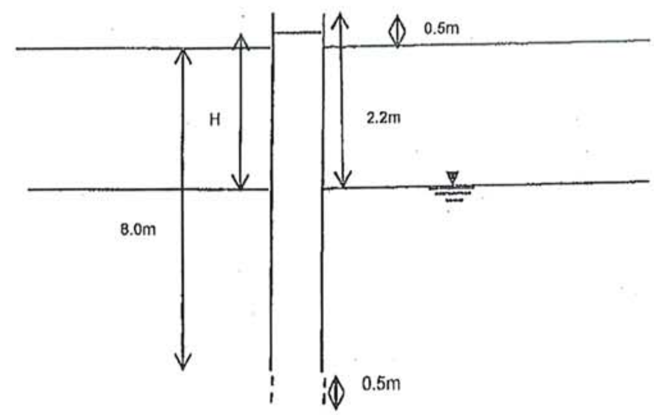
FIELD PERMEABILITY TEST

Variable Head

Client: M/s. Odel PLC		Job ref.	ELS / 3746
		Client ref.	
Project.: Geotechnical Investigation for the Proposed Six Storied Building at Alexandra Place, Colombo 07.		Hole No.	BH-04
Depth(m) 8.0-8.5			
Location:. Alexandra Place, Colombo 07.		Date of test	12.11.2015
Soil description: Clayey sand		Date of report	12.11.2015
Diameter of Casing: 7.6 cm	A = 45.3647 L = 50 cm	F =	121.64

Time	Elapse Time	Mesured level	H
	min	cm	cm
8.00am	0.00	0.0	220.0
	1.00	45.0	175.0
	2.00	67.0	153.0
	3.00	83.0	137.0
	4.00	97.0	123.0
	5.00	109.0	111.0
	10.00	142.0	78.0
	15.00	158.0	62.0
	20.00	169.0	51.0
	25.00	174.0	46.0
	30.00	178.0	42.0
	35.00	180.0	40.0
	40.00	181.0	39.0
	45.00	182.0	38.0
	50.00	183.0	37.0
	60.00	183.0	37.0
	70.00	183.0	37.0



$$F = \frac{2\pi L}{\log_e\left[\left(\frac{L}{D}\right) + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]}$$

$$k = \frac{A}{F(t_2 - t_1)} \log_e \frac{H_1}{H_2}$$

$H_1 = 175\ cm; t_1 = 1\ \text{min}$

$H_2 = 46\ cm; t_1 = 25\ \text{min}$

$k = 2.1 * E^{-2} cm\ /\ \text{min}$

$k = 3.46 * E^{-6} m\ /\ s$

Where;

K is the permeability of soil;

H1 is the variable head measured at time t1 after commencement of test;

H2 is the variable head measured at time t2 after commencement of test;

A is the cross sectional area of borehole casing;

Tested by .... Checked by .... Approved by ....

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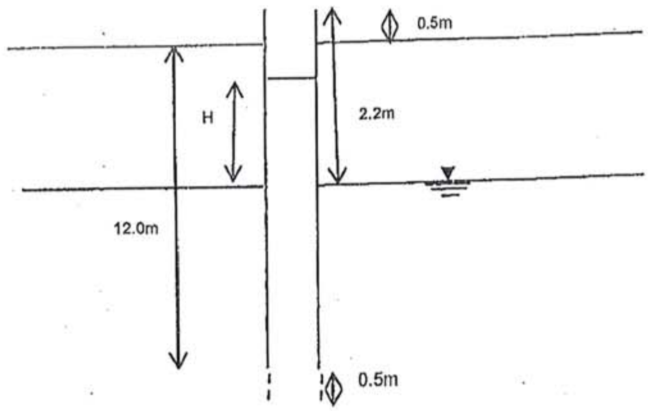
FIELD PERMEABILITY TEST

Variable Head

Client:	M/s. Odel PLC	Job ref.	ELS / 3746
		Client ref.	
Project.:	Geotechnical Investigation for the Proposed Six Storied Building at Alexandra Place, Colombo 07.	Hole No.	BH-04
Depth(m)	12.0-12.5	Date of test	12.11.2015
Location:.	Alexandra Place, Colombo 07.	Date of report	12.11.2015
Soil description:	Sandy Clay	F =	121.64
Diameter of Casing:	7.6 cm	A = 45.3647	L = 50 cm

Time	Elapse Time min	Mesured level cm	H cm
10.00am	0.00	0.0	220.0
	1.00	18.0	202.0
	2.00	30.0	190.0
	3.00	40.0	180.0
	4.00	49.0	171.0
	5.00	57.0	163.0
	10.00	90.0	130.0
	15.00	114.0	106.0
	20.00	136.0	84.0
	25.00	154.0	66.0
	30.00	168.0	52.0
	35.00	180.0	40.0
	40.00	192.0	28.0
	45.00	202.0	18.0
	50.00	209.0	11.0
	60.00	220.0	0.0



$$F = \frac{2\pi L}{\log_e\left[\left(\frac{L}{D}\right) + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]}$$

$$k = \frac{A}{F(t_2 - t_1)}\log_e \frac{H_1}{H_2}$$

$H_1 = 202\ cm; t_1 = 1\ min$

$H_2 = 66\ cm; t_1 = 25\ min$

$k = 1.7 * E^{-2} cm / min$

$k = 2.9 * E^{-6} m / s$

Where;

K is the permeability of soil;

H1 is the variable head measured at time t1 after commencement of test;

H2 is the variable head measured at time t2 after commencement of test;

A is the cross sectional area of borehole casing;

Tested by .... Checked by .... Approved by ....

ENGINEERING & LABORATORY SERVICES (PVT) LTD

62/3, Neelammahara Road, Katuwawala, Boralesgamuwa, Sri Lanka.

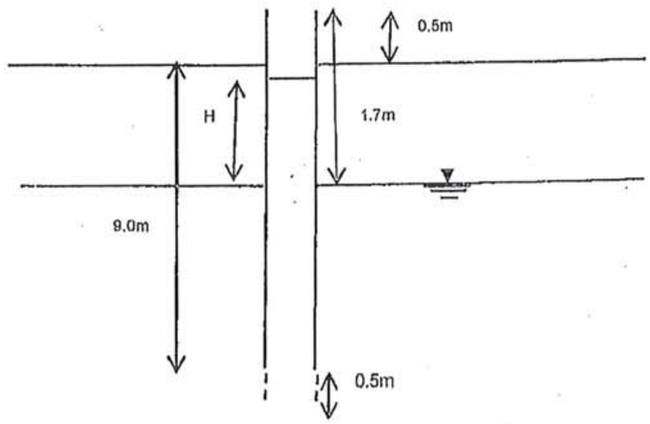
Telephone : 0094 01 517037 / 517365 / 519727, Fax : 0094 01 509806, E-Mail : els@lanka.ccom.

FIELD PERMEABILITY TEST

Variable Head

Client:	M/s. Odel PLC	Job ref.	ELS / 3746
		Client ref.	
Project.:	Geotechnical Investigation for the Proposed Six Storied Building at Alexandra Place, Colombo 07.	Hole No.	BH-05
Depth(m)	9.0-9.5		
Location:.	Alexandra Place, Colombo 07.	Date of test	19.11.2015
Soil description:	Clayey sand	Date of report	19.11.2015
Diameter of Casing:	7.6 cm A = 45.3647 L = 50 cm	F =	121.64

Time	Elapse Time	Mesured level	H
	min	cm	cm
11.00am	0.00	0.0	170.0
	1.00	3.0	167.0
	2.00	4.0	166.0
	3.00	6.0	164.0
	4.00	9.0	161.0
	5.00	10.0	160.0
	10.00	20.0	150.0
	15.00	30.0	140.0
	20.00	39.0	131.0
	25.00	46.0	124.0
	30.00	53.0	117.0
	35.00	59.0	111.0
	40.00	64.0	106.0
	45.00	69.0	101.0
	50.00	74.0	96.0
	60.00	82.0	88.0



$$F = \frac{2\pi L}{\log_e\left[\left(\frac{L}{D}\right) + \sqrt{1+\left(\frac{L}{D}\right)^2}\right]}$$

$$k = \frac{A}{F(t_2 - t_1)}\log_e\frac{H_1}{H_2}$$

$H_1 = 167cm; t_1 = 1\min$

$H_2 = 124cm; t_2 = 25\min$

$k = 4.63 * E^{-3} cm / \min$

$k = 7.7 * E^{-7} m / s$

Where;
K is the permeability of soil;
H1 is the variable head measured at time t1 after commencement of test;
H2 is the variable head measured at time t2 after commencement of test;
A is the cross sectional area of borehole casing;

Tested by ....   Checked by ....   Approved by ....

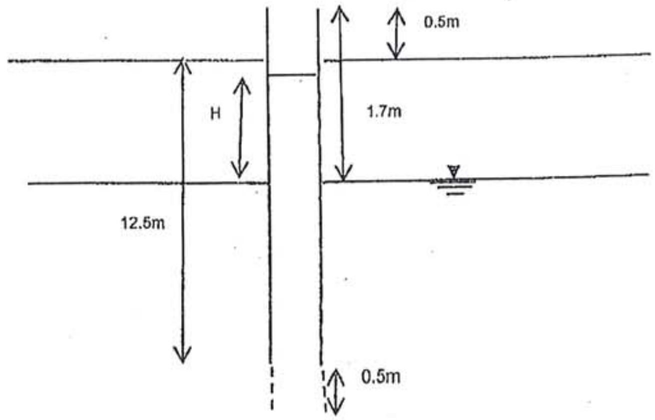
ENGINEERING & LABORATORY SERVICES (PVT) LTD
62/3, Neelammahara Road, Katuwawala, Boralesgamuwa, Sri Lanka.
Telephone : 0094 01 517037 / 517365 / 519727, Fax : 0094 01 509806, E-Mail : els@lanka.ccom.

FIELD PERMEABILITY TEST

Variable Head

Client: M/s. Odel PLC		Job ref.	ELS / 3746
		Client ref.	
Project.: Geotechnical Investigation for the Proposed Six Storied Building at Alexandra Place, Colombo 07.		Hole No.	BH-05
Depth(m) 12.5-13.0			
Location:. Alexandra Place, Colombo 07.		Date of test	19.11.2015
Soil description: Sandy Clay		Date of report	19.11.2015
Diameter of Casing: 7.6 cm	A = 45.3647 L = 50 cm	F =	121.64

Time	Elapse Time min	Mesured level cm	H cm
8.00am	0.00	0.0	170.0
	1.00	7.0	163.0
	2.00	11.0	159.0
	3.00	13.0	157.0
	4.00	16.0	154.0
	5.00	18.0	152.0
	10.00	29.0	141.0
	15.00	39.0	131.0
	20.00	47.0	123.0
	25.00	54.0	116.0
	30.00	59.0	111.0
	35.00	65.0	105.0
	40.00	69.0	101.0
	45.00	73.0	97.0
	50.00	77.0	93.0
	60.00	85.0	85.0



$$F = \frac{2\pi L}{\log_e\left[\left(\frac{L}{D}\right) + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]}$$

$$k = \frac{A}{F\left(t_2 - t_1\right)} \log_e \frac{H_1}{H_2}$$

$H_1 = 163cm; t_1 = 1\min$

$H_2 = 116cm; t_2 = 25\min$

$k = 5.29 * E^{3} cm/mi$

$k = 8.8 * E^{-7} m/s$

Where;

K is the permeability of soil;

H1 is the variable head measured at time t1 after commencement of test;

H2 is the variable head measured at time t2 after commencement of test;

A is the cross sectional area of borehole casing;

Tested by ...Lakshree......

Checked by ......[signature]......

Approved by ......[signature]......

ENGINEERING & LABORATORY SERVICES (PVT) LTD

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Annexure II: Borehole Logs

ENGINEERING & LABORATORY SERVICES (PVT) LTD.

SITE INVESTIGATIONS DIVISION

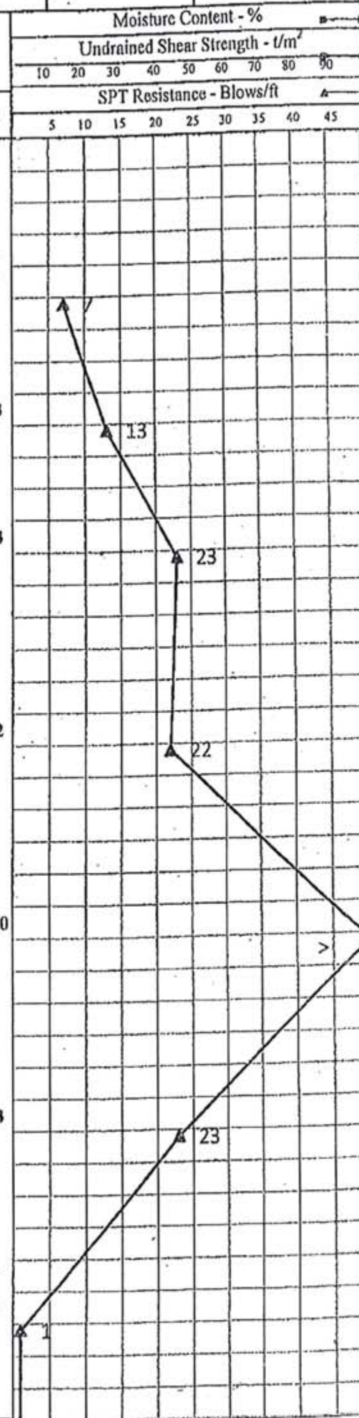
NO 62/5, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Borehole No	BH-01
Client	M/s. Odel PLC	Sheet	1 of 3
Location	Colombo 07	Ground Water level	1.60m
Date of Started	06.11.2015	Coordinates	
Date of Finished	08.11.2015		

Rig	Joy	Core Diameter	54.00mm
Drilling Method	Rotary	Casing depth	25.50m
Casing Diameter	76.00mm	Elevation (m)	

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
0.00							Ground level				
	X	D1	DS		0.00	Sand	Dark brown, fine to medium clayey SAND				
1.00	X	D2	SS		1.00	Sand	Loose, light gray, fine SAND	2	3	4	7
			WS	G.W.L. at 1.60m							
2.00	X	D3	SS		2.00	Sand	Medium dense, light gray fine SAND	4	5	8	13
			WS								
3.00	X	D4	SS					3	6	17	23
4.00			WS								
	X	D5	SS					7	9	13	22
5.00			WS								
6.00	X	D6	SS		6.00	Sand	Very dense, dark brown, fine SAND	25/HB			>50
7.00			WS								
	X	D7	SS		7.50	Silty Sand	Medium dense, gray, fine, silty SAND	8	9	14	23
8.00			WS								
9.00	X	D8	SS		9.00	Organic	Plastic, dark gray, amorphous organic CLAY	1	0	1	1
10.00			WS								



Sample Key / Test Key

SPT: Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL: Ground Water Level observed inside the Borehole, after the saturation
NE: Not Encountered
HB: Hammer Bounce
FD: Free Down

D - Disturbed Sample
SS - SPT Sample
W - Water Sample
WS - Wgrey Sample
UD - Undisturbed Sample
CS - Core Sample
Cr - Core Recovery (%)
RQD - Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG - Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT - Unconfined Compression
CU - Consolidated Undrained
UU - Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl - Clorido Content

Remarks: Existing ground level considered as the zero level.

Logged By: Dimuthu
Supervised By: Chaminda
Drilled By: Sarath

Legend: Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock | Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

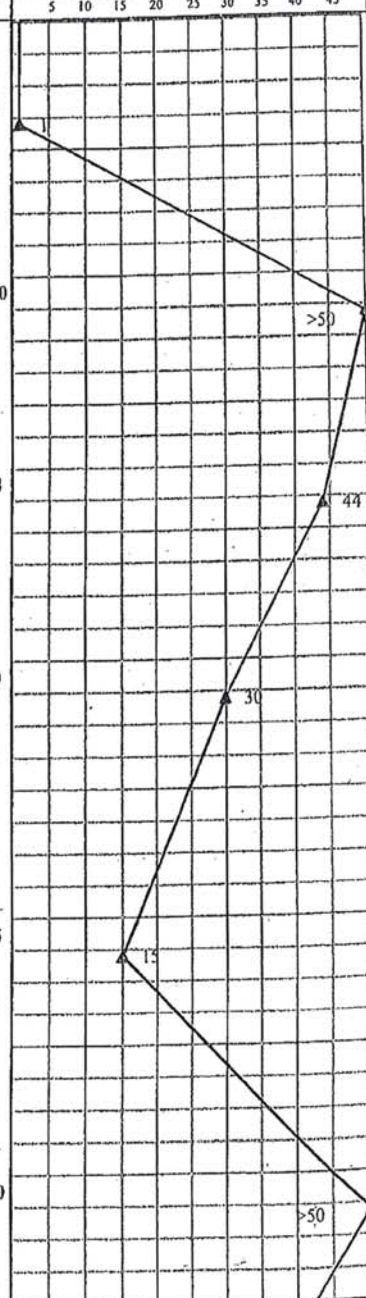
NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.					Borehole No	BH-01
Client	M/s. Odel PLC					Sheet	2 of 3
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm	Ground Water level	1.60m
Date of Started	06.11.2015	Drilling Method	Rotary	Casing depth	25.50m	Coordinates	
Date of Finished	08.11.2015	Casing Diameter	76.00mm	Elevation (m)			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
10.00							Continue from Page 1				
							Same as previous description				
	☒	D9	SS		10.50		Plastic, black, amorphous organic CLAY	1	1	1	2
11.00											
			WS		11.10		Very dense, gray, fine to medium SAND				
12.00											
	☒	D10	SS					11	27	HB	>50
13.00			WS								
	☒	D11	SS		13.50		Dense, gray, fine to medium SAND	14	22	22	44
14.00											
			WS								
15.00											
	☒	D12	SS					11	14	16	30
16.00			WS								
17.00	☒	D13	SS		17.00		Spongy, black, amorphous organic CLAY	8	6	9	15
18.00			WS								
19.00	☒	D14	SS		19.00		Very dense,gray coarse SAND	13	HB		>50
								NO SAMPLE			
20.00											

Moisture Content - %
Undrained Shear Strength - t/m²
SPT Resistance - Blows/ft



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Chaminda
Drilled By: Sarath

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

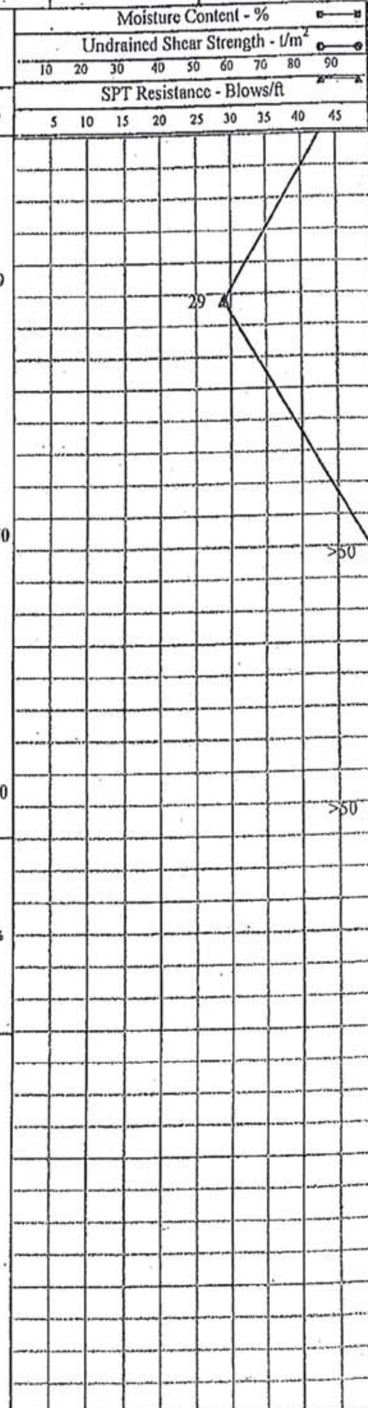
NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Borehole No	BH-01
Client	M/s. Odel PLC	Sheet	3 of 3

Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm	Ground Water level	1.60m
Date of Started	06.11.2015	Drilling Method	Rotary	Casing depth	25.50m	Coordinates	
Date of Finished	08.11.2015	Casing Diameter	76.00mm	Elevation (m)			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
20.00							Continue from Page 1				
			WS				Same as previous description				
21.00	X	D15	SS		21.00		Medium dense, light gray,gray coarse SAND	26	14	15	29
22.00			WS								
23.00	X	D16	SS		23.00		Very dense, offwhite, coarse SAND	6	27	50	>50
								NO SAMPLE			
24.00			WS				Very dense, gray, brown, silty fine sand				
25.00	X	D17	SS		25.00		COMPLETELY WEATHERED ROCK ROCK LEVEL	HB			>50
26.00			CS		25.50		Moderately weathered, fractured, gray, black, brown, BIOTITE GNEISS; fracture surfaces are weathered	CR=96%		RQD=49%	
27.00					27.00		END OF THE BOREHOLE AT 27.00m DEPTH				
28.00											
29.00											
30.00											



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain-Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic Content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Chaminda
Drilled By: Sarath

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock | Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

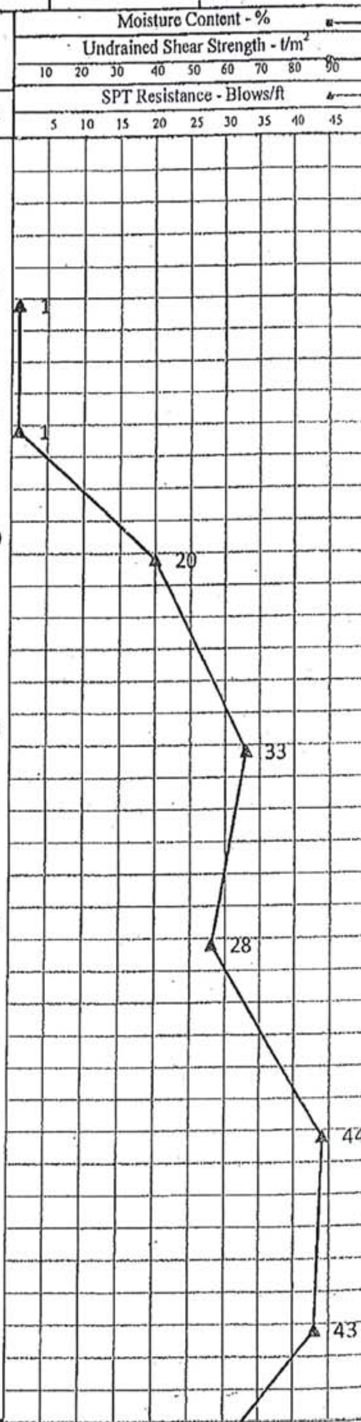
ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

Katuwawala, Sri Lanka.
Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Borehole No	BH-02
Client	M/s. Odel PLC	Sheet	1 of 3
Location	Colombo 07	Rig: TW	Core Diameter: 54.00mm	Ground Water level: 1.40m
Date of Started	07.11.2015	Drilling Method: Rotary	Casing depth: 25.50m	Coordinates
Date of Finished	08.11.2015	Casing Diameter: 76.00mm	Elevation (m)	

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
0.00							Ground level				
	⊠	D1	DS		0.00		Concrete layer				
					0.10		Reddish brown, gray, fine to coarse, clayey SAND				
1.00	⊠	D2	SS		1.00		Very loose, brown, fine to medium SAND	0	0	1	1
			WS	G.W.L. at 1.40m							
2.00	⊠	D3	SS		2.00		Very loose, brown, fine to coarse, clayey SAND	0	0	1	1
			WS								
3.00	⊠	D4	SS		3.00		Medium dense, gray, light gray, fine to medium SAND	4	7	13	20
4.00			WS								
	⊠	D5	SS		4.50		Dense, dark brown, fine to medium SAND	12	15	18	33
5.00			WS								
6.00	⊠	D6	SS					5	12	16	28
7.00			WS								
	⊠	D7	SS		7.50		Dense, gray, fine to coarse SAND	9	19	25	44
8.00			WS								
9.00	⊠	D8	SS		9.00		Dense, brown, fine SAND	10	18	25	43
10.00			WS								



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Samantha
Drilled By: Nishantha

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

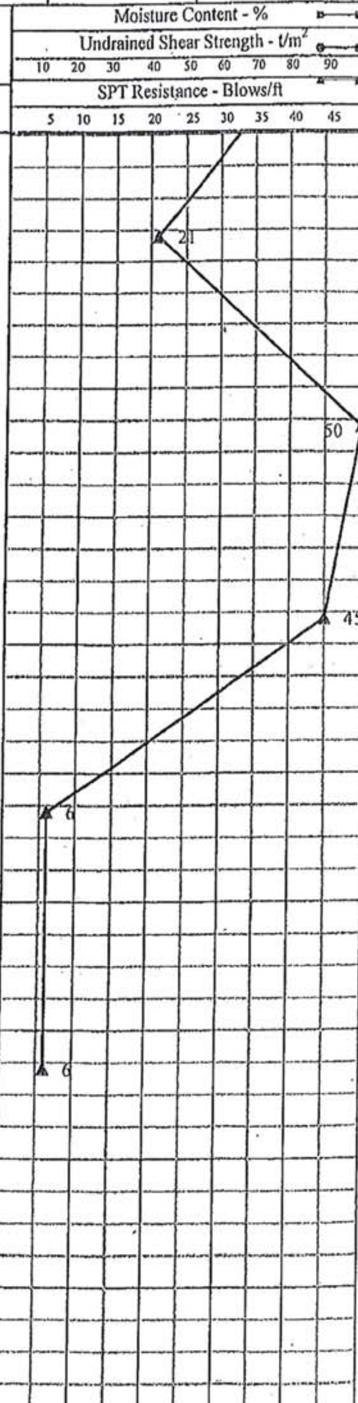
ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.					Borehole No	BH-02
Client	M/s. Odel PLC					Sheet	2 of 3
Location	Colombo 07	Rig	TW	Core Diameter	54.00mm	Ground Water level	1.40m
Date of Started	07.11.2015	Drilling Method	Rotary	Casing depth	25.50m	Coordinates	
Date of Finished	08.11.2015	Casing Diameter	76.00mm	Elevation (m)			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
10.00							Continue from Page 1				
						Sand	Same as previous description				
11.00		D9	SS		10.50	Sand	Medium dense, gray, fine to coarse SAND	7	9	12	21
			WS								
12.00		D10	SS		12.00	Sand	Dense, light brown, offwhite SAND	17	27	23	50
13.00			WS					NO SAMPLE			
14.00		D11	SS		13.50	Sand	Dense, offwhite, fine to coarse SAND	15	21	24	45
			WS								
15.00		D12	SS		15.00	Silt / Organic	Uncompacted, black, organic SILT	2	3	3	6
16.00			WS								
17.00		D13	SS		17.00	Silt / Organic	Uncompacted, black, organic SILT with some gray, fine to medium sandy CLAY	2	3	3	6
18.00			WS								
19.00		D14	UD		19.00		Undisturbed Sample				
20.00					19.50	Sand	Loose, offwhite, fine to coarse SAND				



Sample Key / Test Key

SPT – Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation.
NE – Not Encountered
HB – Hammer Bounce
FD – Free Down

D - Disturbed Sample
SS - SPT Sample
W - Water Sample
WS - Wgrey Sample
UD - Undisturbed Sample
CS - Core Sample
Cr - Core Recovery (%)
RQD - Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG - Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT - Unconfined Compression
CU - Consolidated Undrained
UU - Unconsolidated Undrained
pH - Chemical
O - Organic Content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Samantha
Drilled By: Nishantha

Made Ground · Silt · Gravel · Laterite Nodules · Completely Weathered Rock · Clay · Sand · Organic Matter · Silty Sand · Highly Weathered Rock · Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

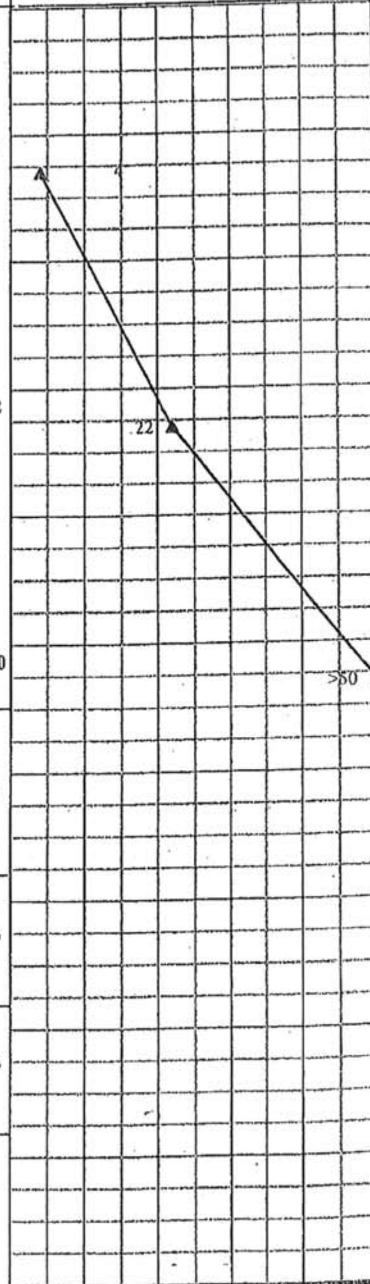
Katuwawala, Sri Lanka.
Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.				Borehole No	BH-02
Client	M/s. Odel PLC				Sheet	3 of 3
Location	Colombo 07	Rig	TW	Core Diameter 54.00mm	Ground Water level	1.40m
Date of Started	07.11.2015	Drilling Method	Rotary	Casing depth 25.50m	Coordinates	
Date of Finished	08.11.2015	Casing Diameter	76.00mm	Elevation (m)		

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
20.00							Continue from Page 1				
			WS				Same as previous description				
21.00	X	D15	SS					1	2	2	4
								NO SAMPLE			
22.00			WS								
23.00	X	D16	SS		23.00		Medium dense, coarse SAND with fine gravels	15	11	11	22
24.00			WS								
25.00	X	D17	SS		25.00		COMPLETELY WEATHERED ROCK ROCK LEVEL	30/ HB			>50
26.00			CS		25.50		Moderately weathered, highly fractured, brown, gray, reddish brown, BIOTITE GNEISS; fracture surfaces are highly fractured	CR=97%		RQD=0%	
27.00			CS		26.80		Moderately weathered, fractured, brown, gray, reddish brown, BIOTITE GNEISS; fracture surfaces are highly fractured	CR=90%		RQD=35%	
28.00			CS		27.80		Moderately weathered, fractured, brown, gray, reddish brown, BIOTITE GNEISS; fracture surfaces are highly fractured	CR=100%		RQD=85%	
29.00					28.80		END OF THE BOREHOLE AT 28.80m DEPTH				
30.00											

Moisture Content - %
Undrained Shear Strength - t/m²
10 20 30 40 50 60 70 80 90
SPT Resistance - Blows/ft
5 10 15 20 25 30 35 40 45



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl⁻ - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Samantha
Drilled By: Nishantha

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock | Fresh Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

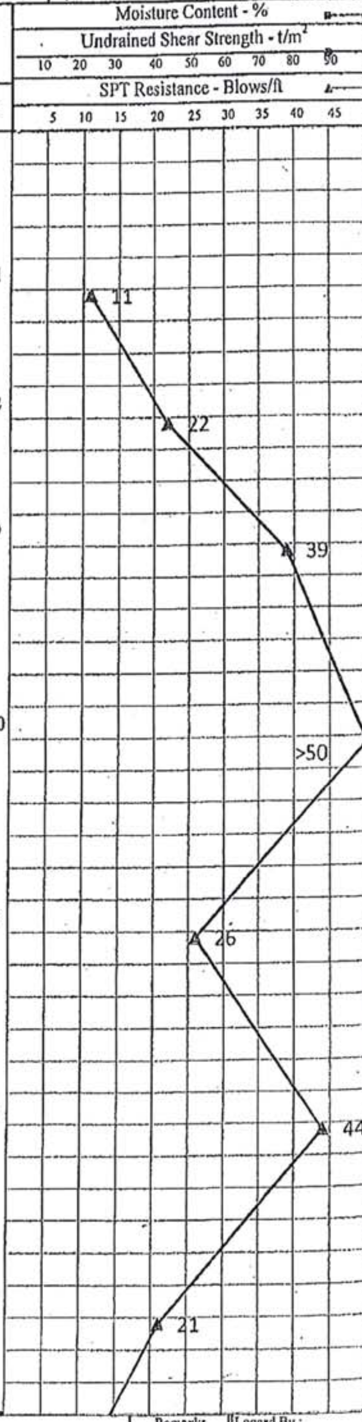
NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.					Borehole No	BH-03
Client	M/s. Odel PLC					Sheet	1 of 4
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm	Ground Water level	1.70m
Date of Started	07.11.2015	Drilling Method	Rotary	Casing depth	33.30m	Coordinates	
Date of Finished	09.11.2015	Casing Diameter	76.00mm	Elevation (m)			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
0.00							Ground level				
	X	D1	DS				Dark brown, offwhite, fine to medium, SAND				
1.00	X	D2	SS		1.00		Medium dense, gray, fine to medium SAND	3	4	7	11
			WS		G.W.L. at 1.70m						
2.00	X	D3	SS					1	6	16	22
			WS								
3.00	X	D4	SS		3.00		Dense, dark brown, fine to medium SAND	9	22	17	39
4.00			WS								
	X	D5	SS		4.50		Very dense, dark brown, fine to coarse SAND	15	30/HB		>50
5.00			WS								
6.00	X	D6	SS		6.00		Medium dense, brown, fine SAND	9	11	15	26
7.00			WS								
	X	D7	SS		7.50		Dense, gray, fine to medium SAND	20	20	24	44
8.00			WS								
9.00	X	D8	SS		9.00		Spongy, black, organic CLAY	4	10	11	21
10.00			WS								

Field Records (SPT)



Sample Key / Test Key

- FPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
- GWL : Ground Water Level observed inside the Borehole, after the saturation.
- NE Not Encountered
- HB -Hammer Bounce
- FD - Free Down

- D - Disturbed Sample
- SS -SPT Sample
- W - Water Sample
- WS-Wgrey Sample
- UD- Undisturbed Sample
- CS- Core Sample
- Cr - Core Recovery (%)
- RQD-Rock Quality Designation (%)

- N - Natural Moisture Content
- L - Atterberg Limit Test
- G - Grain Size Analysis
- SG -Specific Gravity Test
- B - Bulk Density
- V - Vane Shear Test

- C - Consolidation
- UCT-Unconfined Compression
- CU - Consolidated Undrained
- UU-Unconsolidated Undrained
- pH - Chemical
- O - Organic content
- SO_4^{2-} - Sulphate Content
- Cl⁻ - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Lahiru
Drilled By: Rohana

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

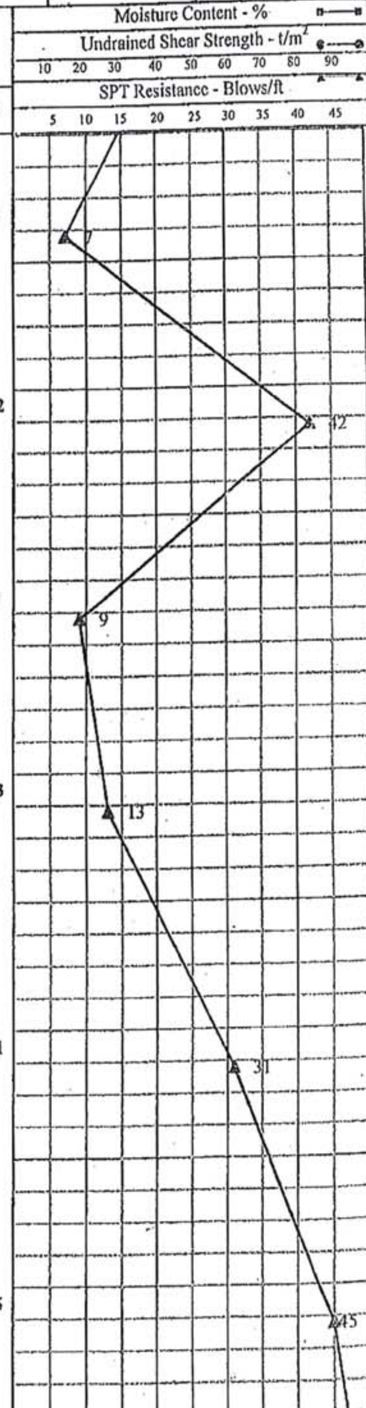
ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.					Borehole No	BH-03
Client	M/s. Odel PLC					Sheet	2 of 4
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm	Ground Water level	1.70m
Date of Started	07.11.2015	Drilling Method	Rotary	Casing depth	33.30m	Coordinates	
Date of Finished	09.11.2015	Casing Diameter	76.00mm	Elevation (m)			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Soil Description	15cm	15cm	15cm	N
10.00						Continue from Page 1				
						Same as previous description				
	X	D9	SS		10.50	Firm, gray, fine to coarse, sandy CLAY	2	3	4	7
11.00			WS							
12.00	X	D10	SS		12.00	Dense, offwhite, gray, fine to coarse SAND	15	20	22	42
13.00			WS							
	X	D11	SS		13.50	Spongy, black, amorphous, organic CLAY	4	4	5	9
14.00			WS							
15.00	X	D12	SS		15.00	Stiff to very stiff, greenish gray, fine to medium, sandy CLAY	3	8	5	13
16.00			WS							
17.00	X	D13	SS				6	10	21	31
18.00			WS							
19.00	X	D14	SS		19.00	Dense, light gray, fine to coarse, clayey SAND	5	15	30	45
20.00										



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL Ground Water Level observed inside the Borehole, after the saturation.
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG - Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Lahiru
Drilled By: Rohana

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

[illegible], Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.			Borehole No	BH-03
Client	M/s. Odel PLC			Sheet	3 of 4
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm
Date of Started	07.11.2015	Drilling Method	Rotary	Casing depth	33.30m
Date of Finished	09.11.2015	Casing Diameter	76.00mm	Elevation (m)	

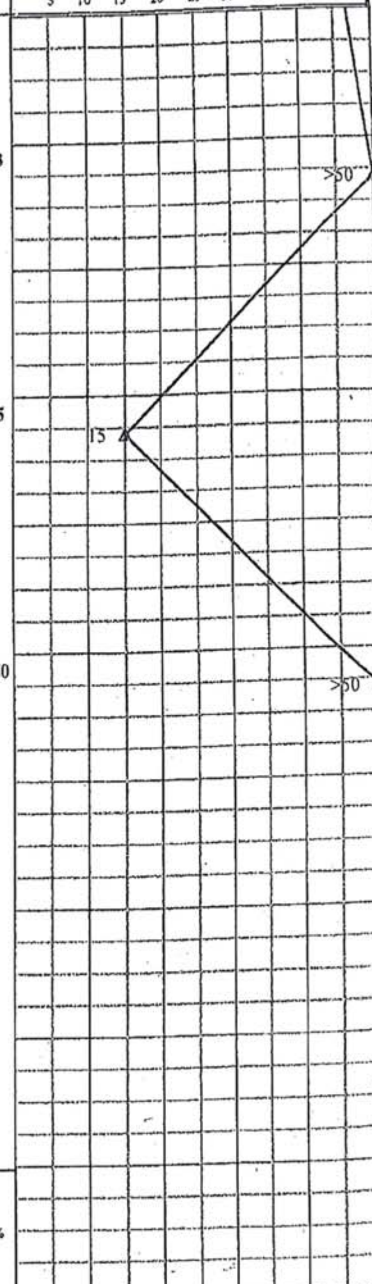
Ground Water level: 1.70m

Coordinates:

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
20.00							Continue from Page 1				
			WS				Same as previous description				
21.00	☒	D15	SS		21.00		Very dense, offwhite, light gray, fine to coarse SAND	25	33	30	63
22.00			WS								
23.00	☒	D16	SS		23.00		Uncompacted, offwhite, light brown, sandy SILT	5	7	8	15
24.00			WS								
25.00	☒	D17	SS		25.00		Brown, blackm fine SAND COMPLETELY WEATHERED ROCK (27.00-29.00)m sample changed to; gray, black, offwhite, fine sand COMPLETELY WEATHERED ROCK ROCK LEVEL	35/ HB			>50
26.00											
27.00											
28.00											
29.00			CS		29.00		Highly weathered, highly fractured, gray, black, brown, BIOTITE GNEISS; Fracture surfaces are weathered	CR=63%		RQD=0%	
30.00					30.00						

Field Records (SPT)

Moisture Content - %
Undrained Shear Strength - t/m²
10 20 30 40 50 60 70 80 90
SPT Resistance - Blows/ft
5 10 15 20 25 30 35 40 45



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)	D - Disturbed Sample	N - Natural Moisture Content	C - Consolidation
GWL : Ground Water Level observed inside the Borehole, after the saturation	SS - SPT Sample	L - Atterberg Limit Test	UCT-Unconfined Compression
NE Not Encountered	W - Water Sample	G - Grain Size Analysis	CU - Consolidated Undrained
HB -Hammer Bounce	WS-Wgrey Sample	SG - Specific Gravity Test	UU-Unconsolidated Undrained
FD - Free Down	UD- Undisturbed Sample	B - Bulk Density	pH - Chemical
	CS- Core Sample	V - Vane Shear Test	O - Organic content
	Cr - Core Recovery (%)		SO_4^{2-} - Sulphate Content
	RQD-Rock Quality Designation (%)		Cl⁻ - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Lahiru
Drilled By: Rohana

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	Fresh Rock

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Borehole No	BH-03
Client	M/s. Odel PLC	Sheet	4 of 4
Location	Colombo 07	Rig: Joy	Core Diameter: 54.00mm
Date of Started	07.11.2015	Drilling Method: Rotary	Casing depth: 33.30m
Date of Finished	09.11.2015	Casing Diameter: 76.00mm	Elevation (m)

Ground Water level: 1.70m
Coordinates:

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	Field Records (SPT) 15cm	15cm	15cm	N
30.00							Continue from Page 1				
			CS		30.00		Moderatelty weathered, fractured, gray, black, brown, BIOTITE GNEISS; Fracture surfaces are weathered	CR=70%		RQD=44%	
31.00			CS		31.00		Moderatelty weathered, fractured, gray, black, brown, BIOTITE GNEISS; Fracture surfaces are weathered	CR=100%		RQD=57%	
32.00					32.00		END OF THE BOREHOLE AT 32.00m DEPTH				
33.00											
34.00											
35.00											
36.00											
37.00											
38.00											
39.00											
40.00											

Moisture Content - %
Undrained Shear Strength - t/m²: 10 20 30 40 50 60 70 80 90
SPT Resistance - Blows/ft: 5 10 15 20 25 30 35 40 45

Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
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C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dinuthu
Supervised By: Lahiru
Drilled By: Rohana

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock | Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

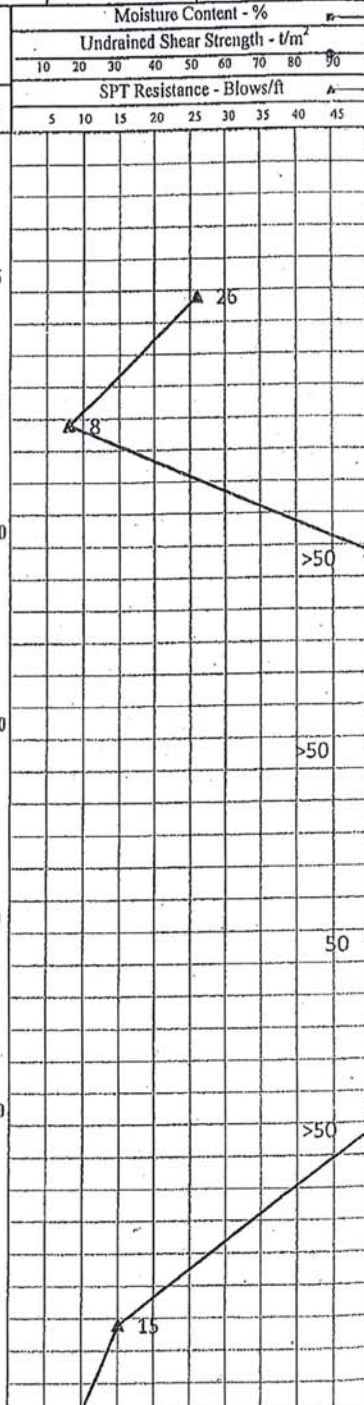
ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.			Borehole No	BH-04
Client	M/s. Odel PLC			Sheet	1 of 3
Location	Colombo 07	Rig: Joy	Core Diameter: 54.00mm	Ground Water level	1.70m
Date of Started	11.11.2015	Drilling Method: Rotary	Casing depth: 25.50m	Coordinates	
Date of Finished	12.11.2015	Casing Diameter: 76.00mm	Elevation (m)		

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
0.00							Ground level				
	X	D1	DS		0.00	Sand	Brown, fine to medium SAND				
1.00	X	D2	SS		1.00	Sand	Medium dense, brown, fine SAND	7	13	13	26
			WS	▽							
2.00	X	D3	SS	G.W.L. at 1.70m	2.00	Sand	Loose, brown, offwhite, fine to medium SAND	2	2	6	8
			WS					NO SAMPLE			
3.00	X	D4	SS		3.00	Sand	Very dense, dark brown, fine to coarse SAND	8	25	HB	>50
4.00			WS								
	X	D5	SS					25	35	HB	>50
5.00			WS								
6.00	X	D6	SS		6.00	Sand	Very dense, brown, fine to medium SAND	10	23	27	50
7.00			WS								
	X	D7	SS		7.50	Sand	Very dense, gray, fine clayey SAND	15	30	HB	>50
8.00			WS								
9.00	X	D8	SS		9.00	Sand	Medium dense, gray, fine to coarse, clayey SAND	7	10	5	15
10.00			WS								



Sample Key / Test Key

SPT: Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL: Ground Water Level observed inside the Borehole, after the saturation
NE: Not Encountered
HB: -Hammer Bounce
FD: - Free Down

D - Disturbed Sample
SS - SPT Sample
W - Water Sample
WS - Wgrey Sample
UD - Undisturbed Sample
CS - Core Sample
Cr - Core Recovery (%)
RQD - Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG - Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT - Unconfined Compression
CU - Consolidated Undrained
UU - Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks: Existing ground level considered as the zero level

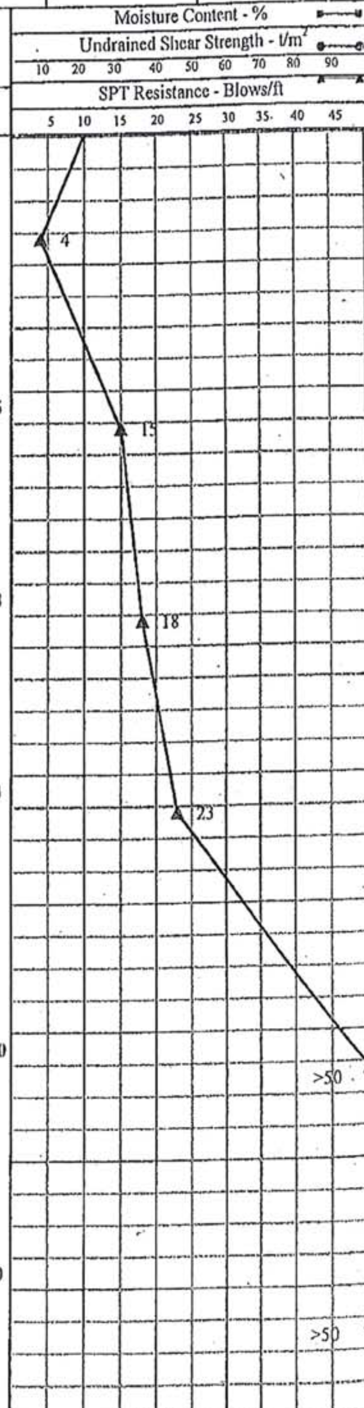
Logged By: Dimuthu
Supervised By: Lahiru
Drilled By: Saruth

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD. SITE INVESTIGATIONS DIVISION	NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494	Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.					Borehole No	BH-04
Client	M/s. Odel PLC					Sheet	2 of 3
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm	Ground Water level	1.70m
Date of Started	11.11.2015	Drilling Method	Rotary	Casing depth	25.50m	Coordinates	
Date of Finished	12.11.2015	Casing Diameter	76.00mm	Elevation (m)			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
10.00							Continue from Page 1				
							Same as previous description				
	X	D9	SS		10.50		Soft, gray, brown, fine to coarse, sandy CLAY	2	1	3	4
11.00			WS								
12.00	X	D10	SS		12.00		Very stiff, gray, fine to coarse, sandy CLAY	4	7	8	15
13.00			WS								
	X	D11	SS					5	8	10	18
14.00			WS								
15.00	X	D12	SS					8	11	12	23
16.00			WS								
17.00	X	D13	SS		17.00		Very dense, offwhite, fine to coarse, slightly clayey SAND	16	HB		>50
18.00			WS								
19.00								18	HB		>50
	X	D14	SS		19.00		Very dense, gray, fine to coarse, silty SAND				
20.00											



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Lahiru
Drilled By: Sarath

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.			Borehole No	BH-04
Client	M/s. Odel PLC			Sheet	3 of 3
Location	Colombo 07	Rig: Joy	Core Diameter: 54.00mm	Ground Water level	1.70m
Date of Started	11.11.2015	Drilling Method: Rotary	Casing depth: 25.50m	Coordinates	
Date of Finished	12.11.2015	Casing Diameter: 76.00mm	Elevation (m)		

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
20.00							Continue from Page 1				
			WS				Same as previous description				
21.00	X	D15	SS		21.00		Very dense, gray, brown, fine to coarse, silty SAND	25	HB		>50
22.00			WS								
23.00	X	D16	SS				COMPLETELY WEATHERED ROCK	HB			>50
24.00			WS								
25.00											
26.00											
27.00							ROCK LEVEL				
28.00			CS		27.50		Modarately weathered, gray, brown, offwhite, highly fractured, BIOTITE	CR=40%		RQD=0%	
29.00			CS		28.50		Modarately weathered, gray, offwhite, black quartzo BIOTITE GNEISS	CR=100%		RQD=65%	
30.00					29.50		END OF THE BOREHOLE AT 29.50m DEPTH				

Chart: Moisture Content - %; Undrained Shear Strength - t/m²; SPT Resistance - Blows/ft. Plotted: >50 at 21.00–21.50 m; >50 at 23.00–23.50 m.

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

Sample Key / Test Key
D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
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CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Orgfnic content
SO_4^{2-} - Sulphate Content
Cl^- - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Lahiru
Drilled By: Sarath

Legend: Made Ground; Silt; Gravel; Laterite Nodules; Completely Weathered Rock; Clay; Sand; Organic Matter; Silty Sand; Highly Weathered Rock; Fresh Rock

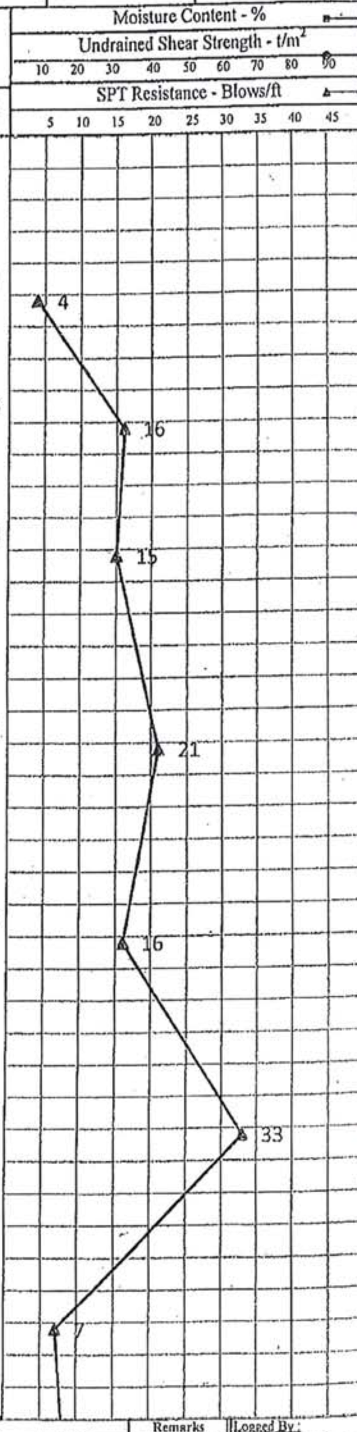
ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka.
Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.			Borehole No	BH-05
Client	M/s. Odel PLC			Sheet	1 of 3
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm
Date of Started	10.11.2015	Drilling Method	Rotary	Casing depth	25.50m
Date of Finished	11.11.2015	Casing Diameter	76.00mm	Elevation (m)	
Ground Water level	1.20m	Coordinates			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
0.00							Ground level				
	X	D1	DS		0.00	Sand	Black, reddish brown, fine to coarse clayey SAND				
1.00	X	D2	SS	G.W.L. at 1.20m	1.00	Sand	Very loose, brown, fine to coarse clayey SAND	1	2	2	4
			WS								
2.00	X	D3	SS		2.00	Sand	Medium dense, light gray, fine to medium SAND	6	6	10	16
			WS								
3.00	X	D4	SS					5	6	9	15
					3.45	Sand	Medium dense, dark brown, fine to coarse SAND				
4.00			WS								
	X	D5	SS					7	9	12	21
5.00			WS								
6.00	X	D6	SS		6.00	Sand	Medium dense, greenish gray, fine SAND	5	7	9	16
7.00			WS								
	X	D7	SS		7.50	Sand	Dense, greensih gray, fine SAND	12	15	18	33
8.00			WS								
9.00	X	D8	SS		9.00	Sand	Loose, gray, fine to coarse clayey SAND	9	5	2	7
10.00			WS								



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl⁻ - Cloride Content

Remarks: Existing ground level considered as the zero level

Logged By: Dimuthu
Supervised By: Chaminda
Drilled By: Sarath

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

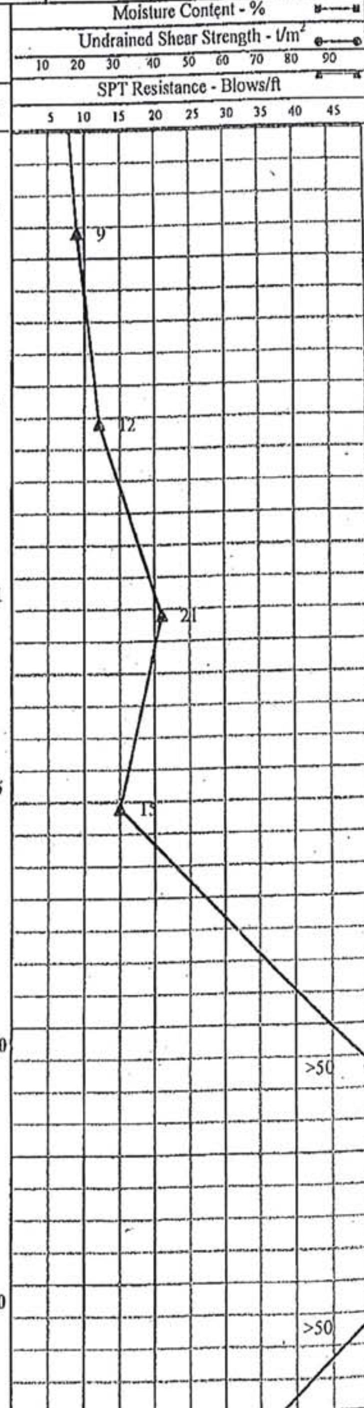
ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.					Borehole No	BH-05
Client	M/s. Odel PLC					Sheet	2 of 3
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm	Ground Water level	1.20m
Date of Started	10.11.2015	Drilling Method	Rotary	Casing depth	25.50m	Coordinates	
Date of Finished	1.11.2015	Casing Diameter	76.00mm	Elevation (m)			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
10.00							Continue from Page 1				
	X	D9	SS			Sand	Same as previous description	9	5	2	7
11.00			WS								
12.00	X	D10	SS		12.00	Clay	Firm, light gray, fine to coarse sandy CLAY	3	4	5	9
13.00			WS								
	X	D11	SS		13.50	Clay	Very stiff, light gray, fine to coarse sandy CLAY	7	9	12	21
14.00			WS								
15.00	X	D12	SS		15.00	Clay	Stiff, light gray, offwhite, fine to coarse sandy CLAY	8	7	8	15
16.00			WS								
17.00	X	D13	SS		17.00	Sand	Very dense, offwhite, fine to coarse, slightly clayey SAND	15	30	HB	>50
18.00			WS								
19.00	X	D14	SS		19.00	Sand	Very dense, gray, offwhite, fine to coarse SAND	29	HB		>50
20.00											



Sample Key / Test Key

SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl⁻ - Cloride Content

Remarks: Existing ground level considered as the zero level

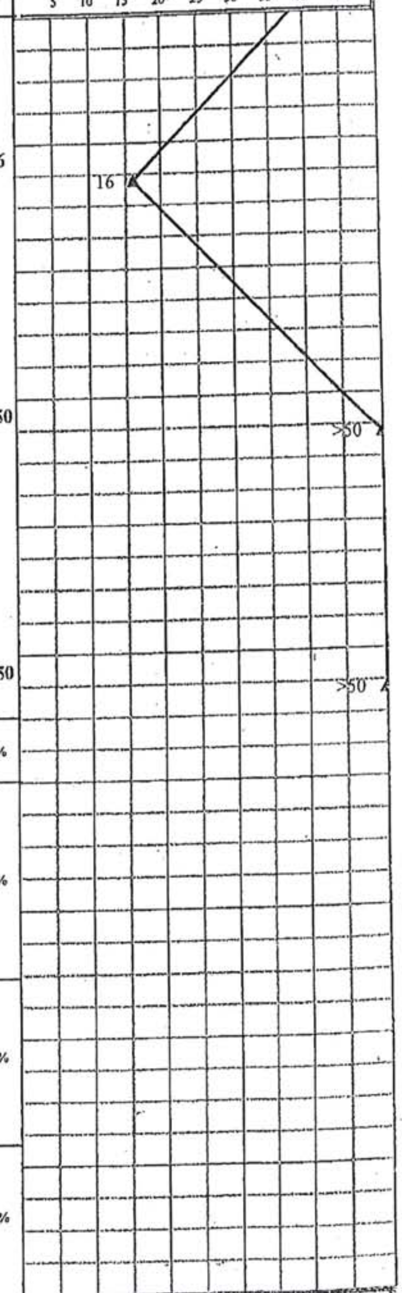
Logged By: Dimuthu
Supervised By: Chaminda
Drilled By: Sarath

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD. SITE INVESTIGATIONS DIVISION	NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494	Format No: ELS-SI-02

Project	**Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.**				Borehole No	BH-05
					Sheet	3 of 3
Client	**M/s. Odel PLC**				Ground Water level	1.20m
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm	
Date of Started	10.11.2015	Drilling Method	Rotary	Casing depth	25.50m	Coordinates
Date of Finished	1.11.2015	Casing Diameter	76.00mm	Elevation (m)		

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Soil Description	15cm	15cm	15cm	N
20.00						Continue from Page 1				
			WS			Same as previous description				
21.00	X	D15	SS		21.00	Medium dense, light gray, greenish gray, fine to medium, silty SAND	4	6	10	16
22.00			WS							
23.00	X	D16	SS		23.00	Very dense, gray, brown, sandy SILT with mica traces COMPLETELY WEATHERED ROCK (25.00-25.45)m sample changed to; gray, black, offwhitem fine to medium SAND COMPLETELY WEATHERED ROCK ROCK LEVEL	34	HB		>50
24.00			WS							
25.00	X	D17	SS				HB			>50
26.00			CS		25.50	Highly weathered, highly fractured, brown, black, BIOTITE GNEISS	CR=100%		RQD=0%	
27.00			CS		26.00	Highly weathered, highly fractured, gray, brown, black, BIOTITE GNEISS	CR=67%		RQD=0%	
28.00			CS		27.50	Highly weathered, highly fractured, gray, brown, black, BIOTITE GNEISS	CR=41%		RQD=0%	
29.00			CS		28.80	Moderately weathered, highly fractured, gray, brown, black, BIOTITE GNEISS	CR=100%		RQD=0%	
30.00					30.00	END OF THE BOREHOLE AT 30.00m DEPTH				



Sample Key / Test Key

				Remarks	Logged By:
SPT Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)	D - Disturbed Sample	N - Natural Moisture Content	C - Consolidation	Existing ground level considered as the zero level	Dimuthu
GWL : Ground Water Level observed inside the Borehole, after the saturation	SS - SPT Sample	L - Atterberg Limit Test	UCT-Unconfined Compression		Supervised By:
NE Not Encountered	W - Water Sample	G - Grain Size Analysis	CU - Consolidated Undrained		Chaminda.
HB - Hammer Bounce	WS-Wgrey Sample	SG - Specific Gravity Test	UU-Unconsolidated Undrained		Drilled By:
FD - Free Down	UD- Undisturbed Sample	B - Bulk Density	pH - Chemical		Sarath
	CS- Core Sample	V - Vane Shear Test	O - Organic content		
	Cr - Core Recovery (%)		SO_4^{2-} - Sulphate Content		
	RQD-Rock Quality Designation (%)		Cl - Cloride Content		

Made Ground	Silt	Gravel	Laterite Nodules	Completely Weathered Rock	Fresh Rock
Clay	Sand	Organic Matter	Silty Sand	Highly Weathered Rock	

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SITE INVESTIGATIONS DIVISION
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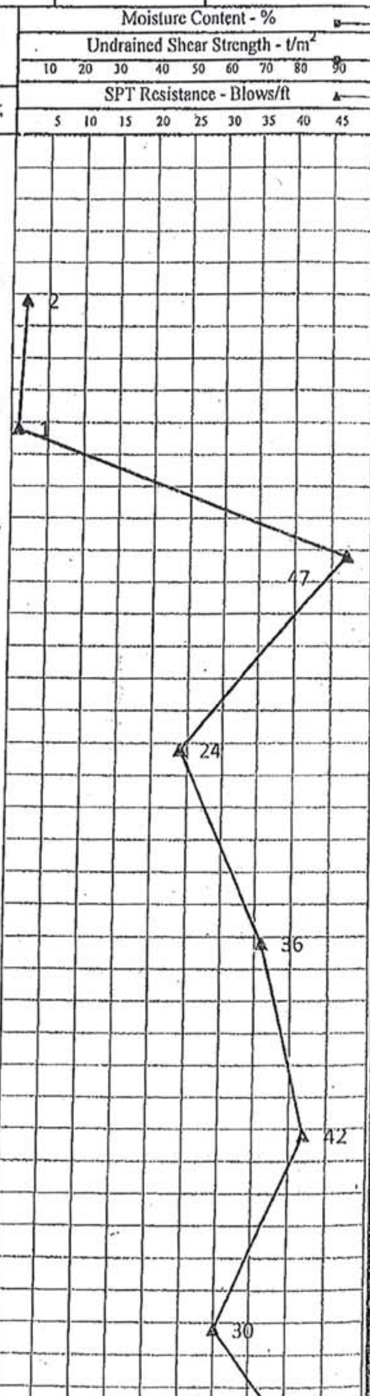
Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.			Borehole No	BH-06
Client	M/s. Odel PLC			Sheet	1 of 4
Location	Colombo 07	Rig	Joy	Core Diameter	54.00mm
Date of Started	10.11.2015	Drilling Method	Rotary	Casing depth	28.50m
Date of Finished	11.11.2015	Casing Diameter	76.00mm	Elevation (m)	
Ground Water level	2.10m	Coordinates			

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
0.00							Ground level				
	X	D1	DS		0.00	Made Ground	Brick fragments FILL				
1.00	X	D2	SS		1.00	Sand	Very loose, gray, brown, fine SAND	1	1	1	2
			WS								
2.00	X	D3	SS		2.00	Sand	Very loose, brown, fine to medium SAND	1	0	1	1
			WS								
3.00	X	D4	SS		3.00	Sand	Dense, brown, fine to medium SAND	8	20	27	47
4.00			WS								
	X	D5	SS		4.50	Sand	Medium dense, brown, fine to medium SAND	12	12	12	24
5.00			WS								
6.00	X	D6	SS		6.00	Sand	Dense, greenish brown, fine to medium SAND	6	12	24	36
7.00			WS								
	X	D7	SS					10	17	25	42
8.00			WS								
9.00	X	D8	SS					15	18	12	30
					9.30	Clay	Gray CLAY				
10.00			WS								



Sample Key / Test Key

SPT — Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl⁻ - Cloride Content

Remarks: Existing ground level considered as the zero level

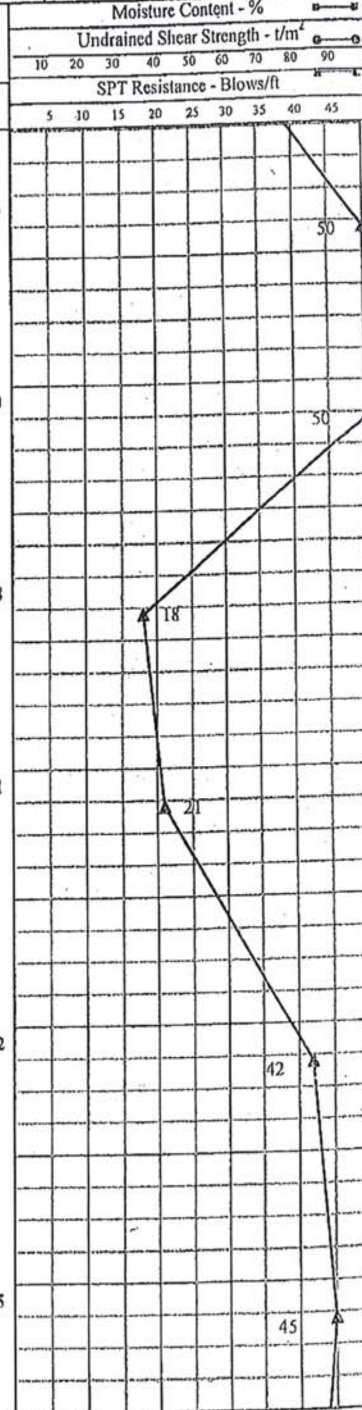
Logged By: Dimuthu
Supervised By: Samantha
Drilled By: Nishantha

Legend: Made Ground; Silt; Gravel; Laterite Nodules; Completely Weathered Rock; Clay; Sand; Organic Matter; Silty Sand; Highly Weathered Rock; Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD. SITE INVESTIGATIONS DIVISION	NO 62/3, Neelammahara Road, Katuwawala, Sri Lanka. Tel: 0114 309 494	Format No: ELS-SI-02	

Project	**Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.**			Borehole No	BH-06
Client	**M/s. Odel PLC**			Sheet	2 of 4
Location	Colombo 07	Rig: Joy	Core Diameter: 54.00mm	Ground Water level	2.10m
Date of Started	10.11.2015	Drilling Method: Rotary	Casing depth: 28.50m	Coordinates	
Date of Finished	11.11.2015	Casing Diameter: 76.00mm	Elevation (m)		

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
10.00							Continue from Page 1				
							Same as previous description				
	X	D9	SS		10.35		Very dense, offwhite, coarse SAND	12	23	27	50
11.00			WS								
12.00	X	D10	SS					14	26	24	50
13.00			WS								
	X	D11	SS		13.40		Very stiff, gray, offwhite, fine to coarse, sandy CLAY	7	8	10	18
14.00			WS								
15.00	X	D12	SS		15.00		Very stiff, gray, fine to coarse, sandy CLAY	8	10	11	21
16.00			WS								
17.00	X	D13	SS		17.00		Dense, offwhite, fine to coarse, slightly clayey SAND	8	20	22	42
18.00			WS								
19.00	X	D14	SS					10	19	26	45
20.00											



Sample Key / Test Key

SPT - Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE - Not Encountered
HB - Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS - SPT Sample
W - Water Sample
WS - Wgrey Sample
UD - Undisturbed Sample
CS - Core Sample
Cr - Core Recovery (%)
RQD - Rock Quality Designation (%)

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L - Atterberg Limit Test
G - Grain Size Analysis
SG - Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT - Unconfined Compression
CU - Consolidated Undrained
UU - Unconsolidated Undrained
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl⁻ - Cloride Content

Remarks: Existing ground level considered as the zero level

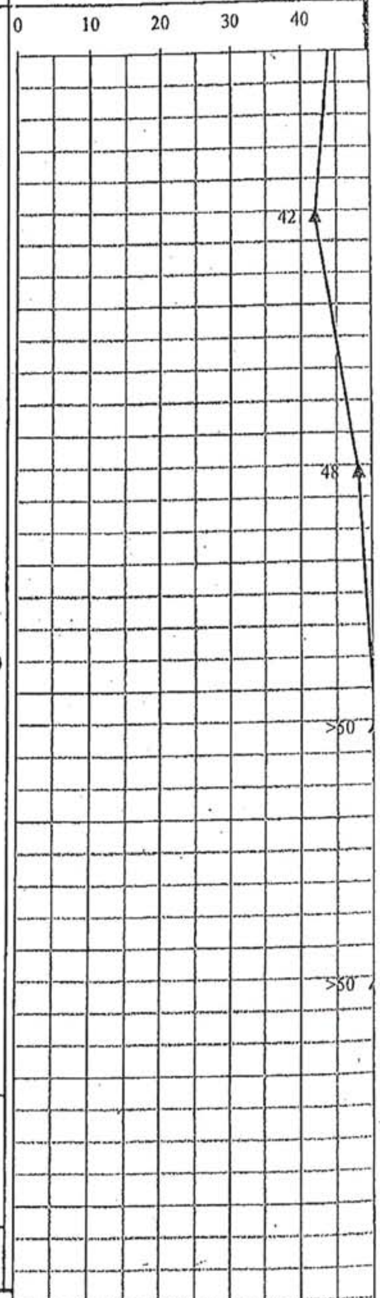
Logged By: Dimuthu
Supervised By: Samantha,
Drilled By: Nishantha

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock | Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.	Borehole No	BH-06
Client	M/s. Odel PLC	Sheet	3 of 4
Location	Colombo 07	Rig: Joy	Core Diameter: 54.00mm
Date of Started	10.11.2015	Drilling Method: Rotary	Casing depth: 28.50m
Date of Finished	11.11.2015	Casing Diameter: 76.00mm	Elevation (m):
Ground Water level	2.10m	Coordinates	

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
20.00							Continue from Page 1				
			WS				Same as previous description				
21.00		D15	SS					14	17	25	42
22.00			WS								
23.00		D16	SS					10	20	28	48
24.00			WS								
25.00		D17	SS		24.70		Offwhite, light gray, lateritic SAND	20	10cm/35	HB	>50
26.00			WS				COMPLETELY WEATHERED ROCK				
27.00		D18	SS				(27.45-28.50)m sample changed to; gray, fine to medium sand	5cm/30			>50
28.00							COMPLETELY WEATHERED ROCK				
							ROCK LEVEL				
29.00			CS		28.50		Moderately weathered, fractured, gray, black, offwhite, BIOTITE GNEISS	CR=100%		RQD=20%	
30.00			CS		29.50		Moderately weathered, fractured, gray, black, offwhite, BIOTITE GNEISS	CR=100%		RQD=50%	

Moisture Content - %
Undrained Shear Strength - t/m²
SPT Resistance - Blows/ft



Sample Key / Test Key

SPT - Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)	D - Disturbed Sample	N - Natural Moisture Content	C - Consolidation
GWL : Ground Water Level observed inside the Borehole, after the saturation	SS - SPT Sample	L - Atterberg Limit Test	UCT-Unconfined Compression
NE Not Encountered	W - Water Sample	G - Grain Size Analysis	CU - Consolidated Undrained
HB - Hammer Bounce	WS-Wgrey Sample	SG - Specific Gravity Test	UU-Unconsolidated Undrained
FD - Free Down	UD- Undisturbed Sample	B - Bulk Density	pH - Chemical
	CS- Core Sample	V - Vane Shear Test	O - Organic content
	Cr - Core Recovery (%)		SO_4^{2-} - Sulphate Content
	RQD-Rock Quality Designation (%)		Cl - Cloride Content

Existing ground level considered as the zero level

Dimuthu — Supervised By:
Samantha — Drilled By:
Nishantha

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock | Fresh Rock

ENGINEERING & LABORATORY SERVICES (PVT) LTD.
SITE INVESTIGATIONS DIVISION

Katuwawala, Sri Lanka.
Tel: 0114 309 494

Format No: ELS-SI-02

Project	Geotechnical Investigation for the Proposed Twelve Storied Building at Alexandra Place, Colombo 07.			Borehole No	BH-06
Client	M/s. Odel PLC			Sheet	4 of 4
Location	Colombo 07	Rig: Joy	Core Diameter: 54.00mm	Ground Water level	2.10m
Date of Started	10.11.2015	Drilling Method: Rotary	Casing depth: 28.50m	Coordinates	
Date of Finished	11.11.2015	Casing Diameter: 76.00mm	Elevation (m)		

Moisture Content - % (10 20 30 40 50 60 70 80 90)
Undrained Shear Strength - t/m²
SPT Resistance - Blows/ft (5 10 15 20 25 30 35 40 45)

Depth (m)	Sa. Cond	Sa.NO.	Sa.Type	Reduced level	Depth (m)	Legend	Soil Description	15cm	15cm	15cm	N
30.00							Continue from Page 1				
			CS				Same as previous description				
31.00			CS		30.70		Moderately weathered, fractured, gray, black, offwhite, BIOTITE GNEISS	CR=85%		RQD=20%	
32.00					31.70		END OF THE BOREHOLE AT 31.70m DEPTH				
33.00											
34.00											
35.00											
36.00											
37.00											
38.00											
39.00											
40.00											

Sample Key / Test Key

SPT - Where full 0.3m penetration has not been achieved the number of blows for the quoted penetration is given (not N-value)
GWL : Ground Water Level observed inside the Borehole, after the saturation
NE Not Encountered
HB -Hammer Bounce
FD - Free Down

D - Disturbed Sample
SS -SPT Sample
W - Water Sample
WS-Wgrey Sample
UD- Undisturbed Sample
CS- Core Sample
Cr - Core Recovery (%)
RQD-Rock Quality Designation (%)

N - Natural Moisture Content
L - Atterberg Limit Test
G - Grain Size Analysis
SG -Specific Gravity Test
B - Bulk Density
V - Vane Shear Test

C - Consolidation
UCT-Unconfined Compression
CU - Consolidated Undrained
UU-Unconsolidated Undrained.
pH - Chemical
O - Organic content
SO_4^{2-} - Sulphate Content
Cl⁻ - Cloride Content

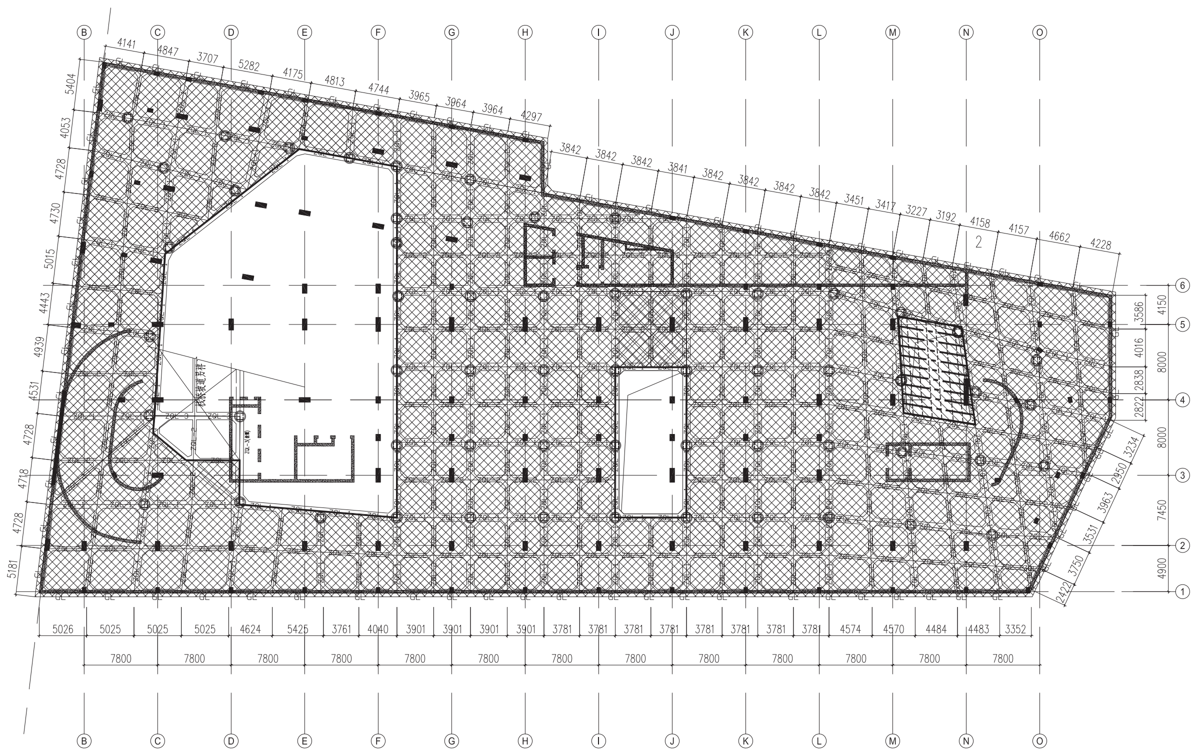
Remarks: Existing ground level considered as the zero level

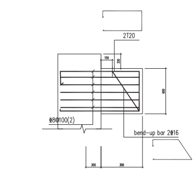
Logged By: Dimuthu
Supervised By: Samantha
Drilled By: Nishantha

Made Ground | Silt | Gravel | Laterite Nodules | Completely Weathered Rock | Fresh Rock
Clay | Sand | Organic Matter | Silty Sand | Highly Weathered Rock

APPENDIX C

Details of Temporary Supports





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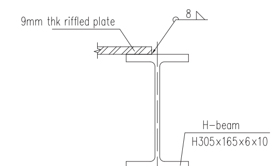
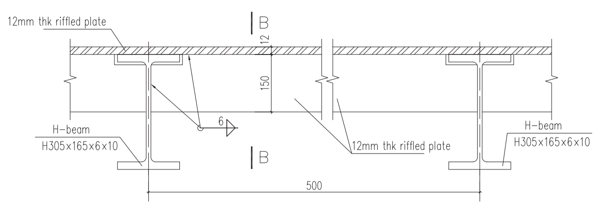


Connection Joint

Legend :

legend	name	location	diameter
⊡	king post LZ	see the plan	1200mm



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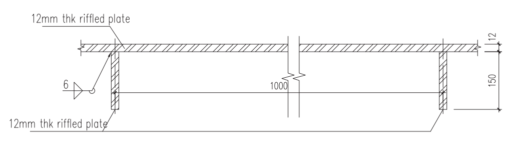


B-B

Plan of First Layer Supporting

Instructions:

1.The unit in the diagram are all in mm except the elevation(in m),which represents relative elevation..

2.The dimensions indicated in the drawing shall prevail,and cannot be measured directly.The plate of ▩ is 250mm thick, concrete grade being C30, and the reinforcement adopts double-layer and two-way T16@150 .

3.The "— - —" in the diagram stands for H-beam,with section of H305×165×6×10,material of Q235.

4.The material of steel slab is Q235,see its thickness in the section diagram.

5.T20 rebar supports are used between the steel beams, and see the detailed drawings of joints for connection between rebars;spacing between supports shall not exceed 2000mm.

6.Before the arrangement of H-beams, the position of the containers must be arranged first to determine the container corner points. When arranging the H-beams, the container corners shall be supported by the H-beams, and the spacing between the steel beams should not exceed 500mm.

7.Please refer to the related specifications and codes for the matters not covered.

序号 NO.	日期 DATE	修改内容 REVISION DESCRIPTION

备注:

出图专用章 STAMP FOR ISSUE

注册章 STAMP FOR REGISTER

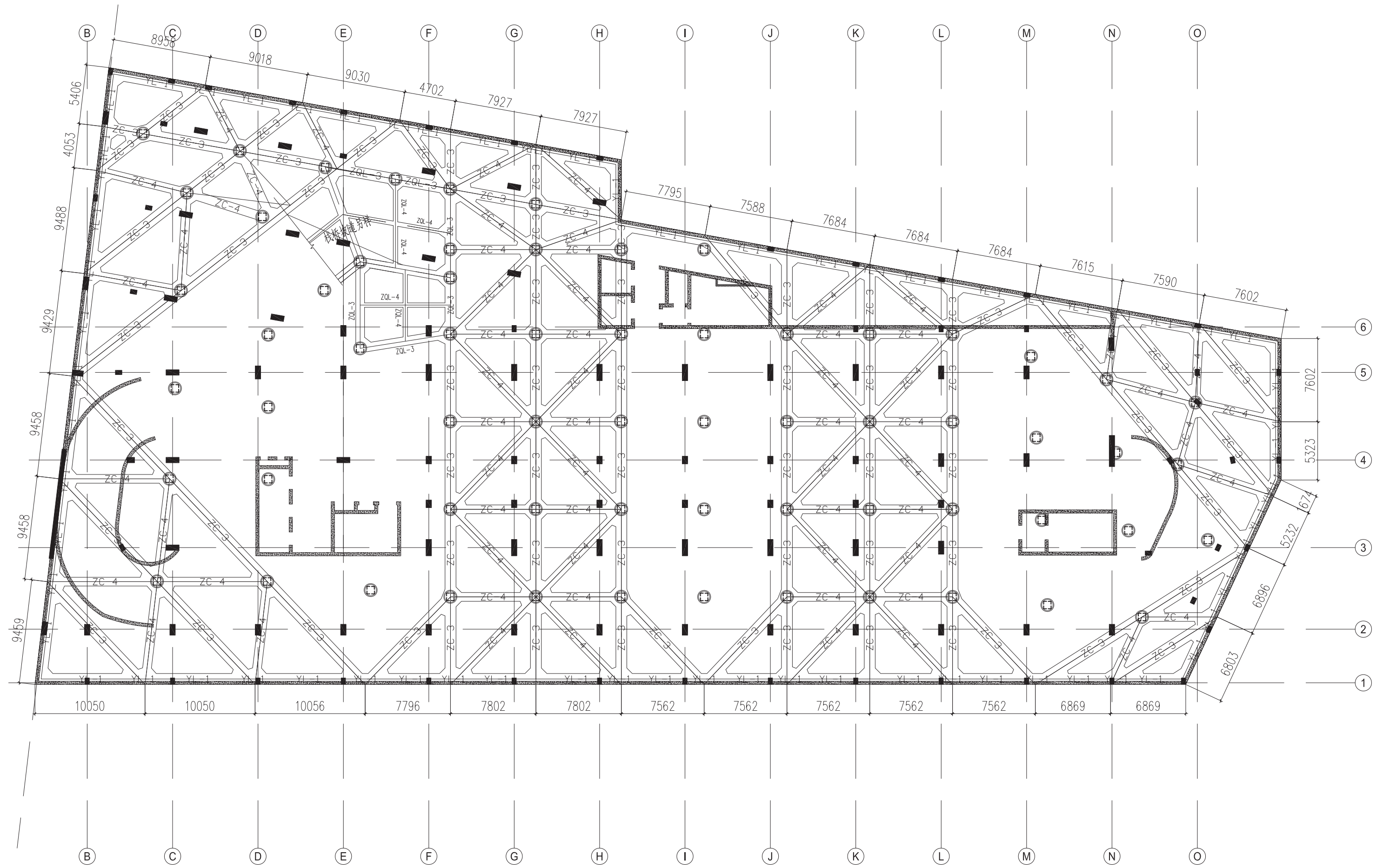
建设单位 CLIENT

项目名称 ITEM

图名 DRAWING TITLE

Plan of the First Layer Inner Support

项目号 PROJECT NO.			
设计总负责人 PROJECT DIRECTOR			
专业负责人 DISCIPLINE CHIEF			
制 图 DRAWN BY			
设 计 DESIGNED BY			
校 对 CHECKED BY			
审 核 APPROVED BY			
审 定 AUTHORIZED BY			
日 期 DATE			
图 别 PHASE		图 号 DRAWING NO.	03-1
比 例 SCALE		版本号 VER.	



Plan of the Second Layer Support

Instructions:

1. The unit in the diagram are all in mm except the elevation(in m),which represents relative elevation..
2. The dimensions indicated in the drawing shall prevail,and cannot be measured directly.
3. Please refer to the related specifications and codes for the matters not covered.

legend	name	location	diameter
(symbol)	king post LZ	see the plan	1200mm

序号 NO.	日期 DATE	修改内容 REVISION DESCRIPTION

备注:

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注册章 STAMP FOR REGISTER

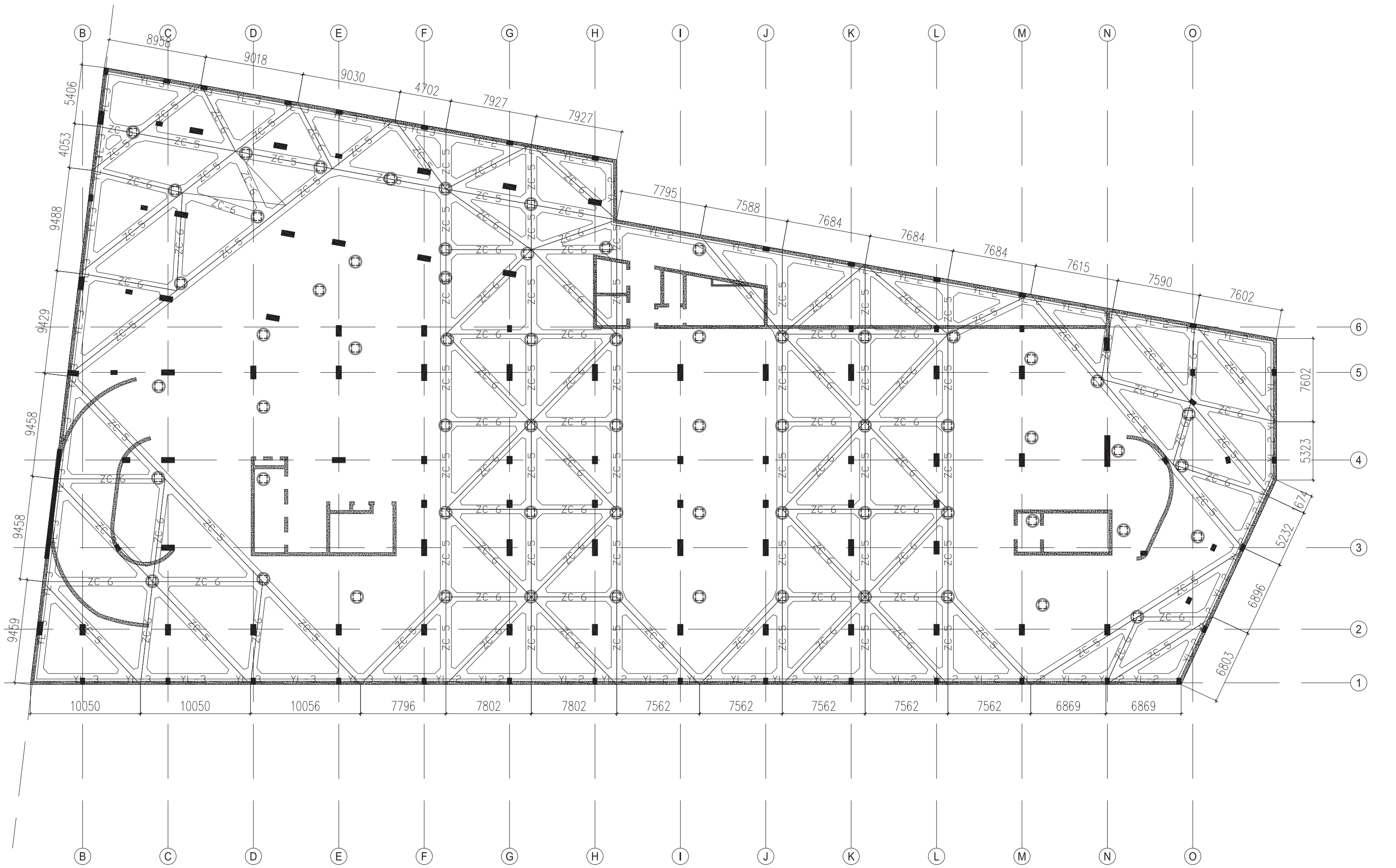
建设单位 CLIENT

项目名称 ITEM

图名 DRAWING TITLE

Plan of the Second Layer Inner Support

项目号 PROJECT NO.			
设计总负责人 PROJECT DIRECTOR			
专业负责人 DISCIPLINE CHIEF			
制图 DRAWN BY			
设计 DESIGNED BY			
校对 CHECKED BY			
审核 APPROVED BY			
审定 AUTHORIZED BY			
日期 DATE			
图别 PHASE		图号 DRAWING NO.	03-3
比例 SCALE		版本号 VER.	



Plan of the Third Layer Support

Instructions:

1. The unit in the diagram are all in mm except the elevation(in m),which represents relative elevation..
2. The dimensions indicated in the drawing shall prevail,and cannot be measured directly.
3. Please refer to the related specifications and codes for the matters not covered.

legend	name	location	diameter
⊡	king post LZ	see the plan	1200mm

序号 NO.	日期 DATE	修改内容 REVISION DESCRIPTION

备注:

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注册章 STAMP FOR REGISTER

建设单位 CLIENT

项目名称 ITEM

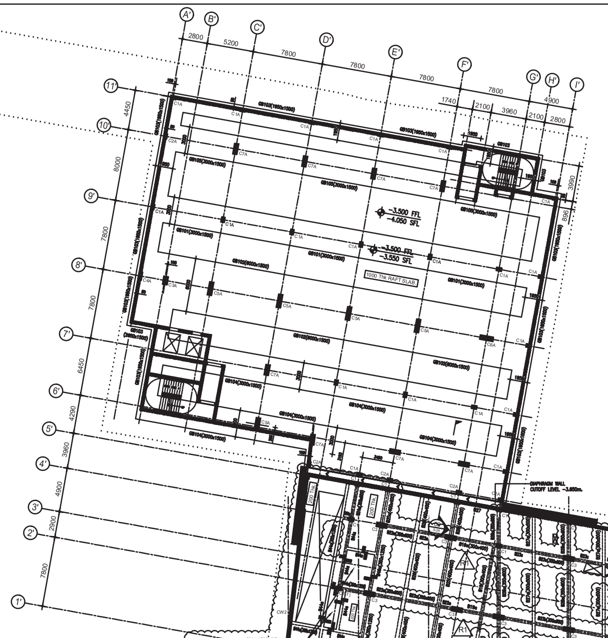
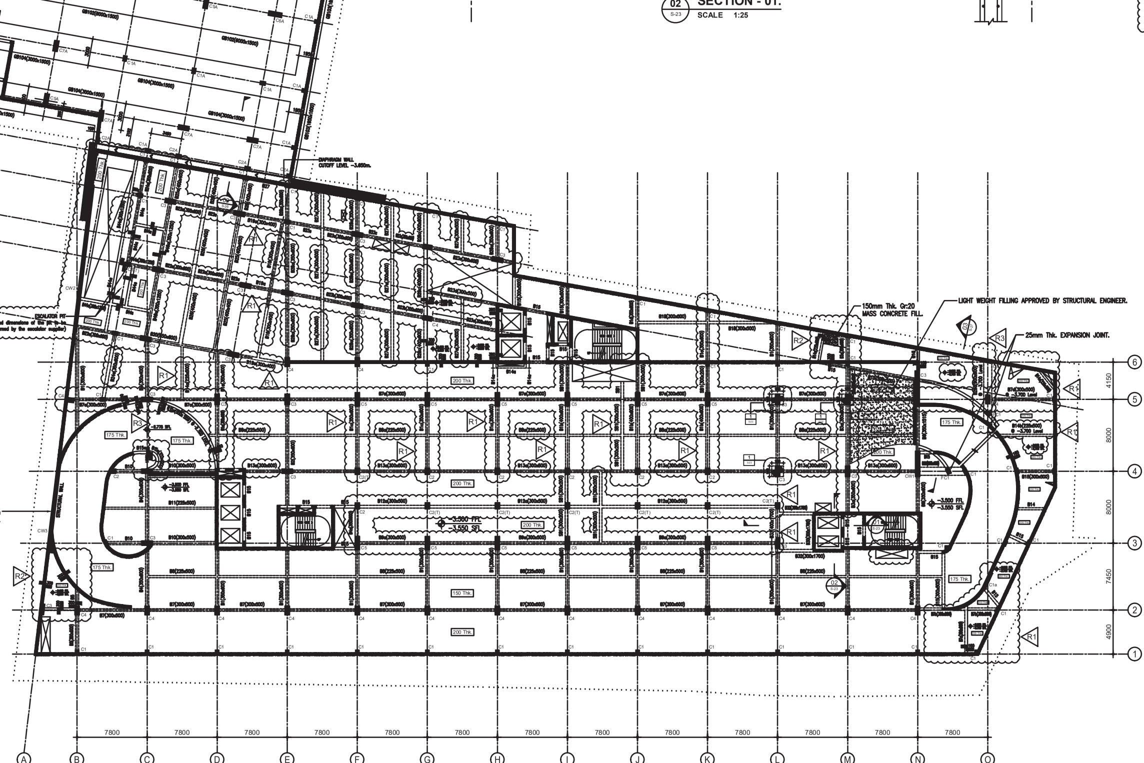
图名 DRAWING TITLE

Plan of the Third Layer Inner Support

项目号 PROJECT NO.			
设计总负责人 PROJECT DIRECTOR			
专业负责人 DISCIPLINE CHIEF			
制图 DRAWN BY			
设计 DESIGNED BY			
校对 CHECKED BY			
审核 APPROVED BY			
审定 AUTHORIZED BY			
日期 DATE			
图别 PHASE		图号 DRAWING NO.	03-4
比例 SCALE		版本号 VER.	

APPENDIX D

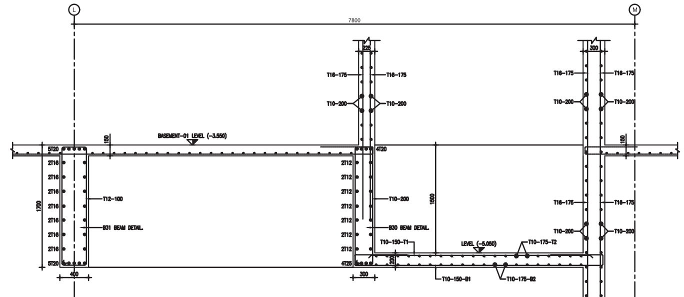
Details of Permanent Supports

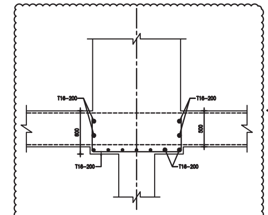
01 BASEMENT - 01 GENERAL ARRANGEMENT.
SCALE 1:200

NOTE :-
1. PTB – POST TENSION BEAM.
2. RCB – REINFORCED CONCRETE BEAM.
3. C – REINFORCED COLUMN.
4. T – REINFORCED COLUMN TERMINATE.
5. FC – FLOATING REINFORCED COLUMN.

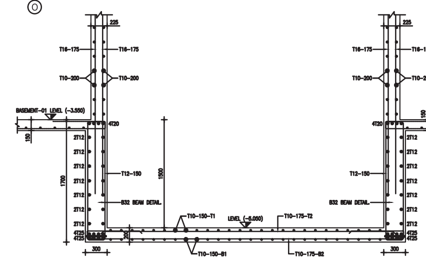
NOTE :-
STIFFENER COLUMNS FOR MASONRY WALLS ARE NOT SHOWN FOR CLARITY.
SLAB THICKNESS IS 150mm UNLESS OTHERWISE SPECIFIED.



02 SECTION - 01.
SCALE 1:25



03 DETAIL - 01.
SCALE 1:25



03 SECTION - 02.
SCALE 1:25

PROJECT
Proposed 5 Basements & G + 7 Storeyed Mixed Development Project for ODEL at Ward Place, Colombo 07.

CLIENT
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ARCHITECT'S SIGNATURE / SEAL

STRUCTURAL ENGINEER'S SIGNATURE / SEAL

CLIENT'S SIGNATURE / SEAL

No	Note	Date	Drawn	Index
R1	REVISIONS ARE CLOUDED	04.10.2019	INDIKA	

REVISIONS

Scale	1:200 & 1:25	Date	2018.05.21
Original Size	A0	Revision No	R1

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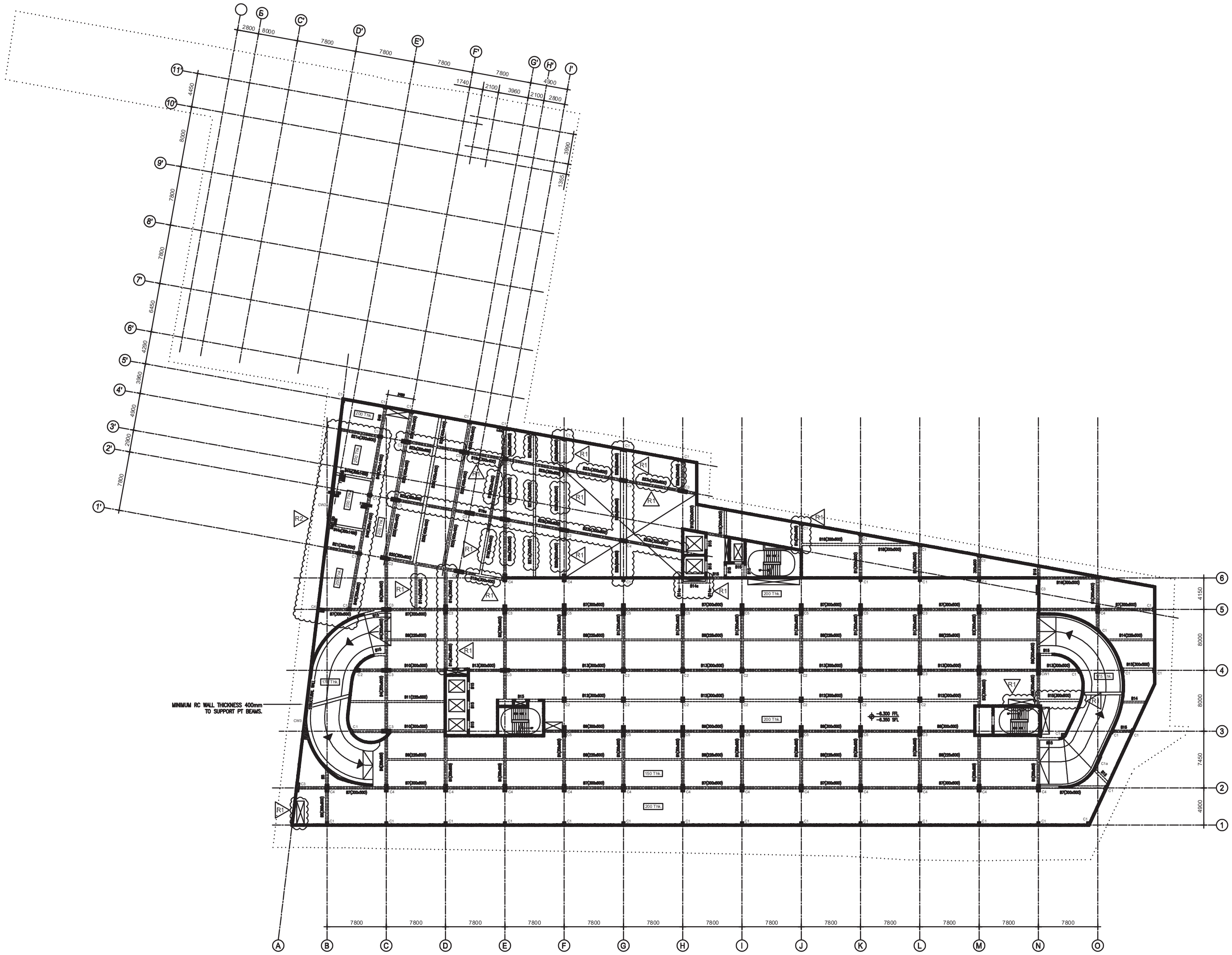
CONSTRUCTION DRAWINGS

Drawing Title: **BASEMENT - 01 GENERAL ARRANGEMENT & SECTIONS.**

Drawing Number: S-23 | Original Size: A0



01 S-20 **BASEMENT - 02 GENERAL ARRANGEMENT.**
SCALE 1:200

NOTE :-
1. PTB – POST TENSION BEAM.
2. RCB – REINFORCED CONCRETE BEAM.
3. C – REINFORCED COLUMN.
4. T – REINFORCED COLUMN TERMINATE.
5. FC – FLOATING REINFORCED COLUMN.

NOTE :-
STIFFENER COLUMNS FOR MASONRY WALLS ARE NOT SHOWN FOR CLARITY.
SLAB THICKNESS IS 150mm UNLESS OTHERWISE SPECIFIED.

PROJECT
Proposed 5 Basements & G + 7 Storeyed Mixed Development Project for ODEL at Ward Place, Colombo 07.

CLIENT
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CLIENT'S SIGNATURE / SEAL

No	Note	Date	Drawn	Initial
R2	REVISIONS ARE CLOUDED	13.12.2019	INDIKA	
R1	REVISIONS ARE CLOUDED	04.10.2019	INDIKA	

REVISIONS

Scale	1:200	Date	2018.05.21
Original Size	A0	Revision No	R2

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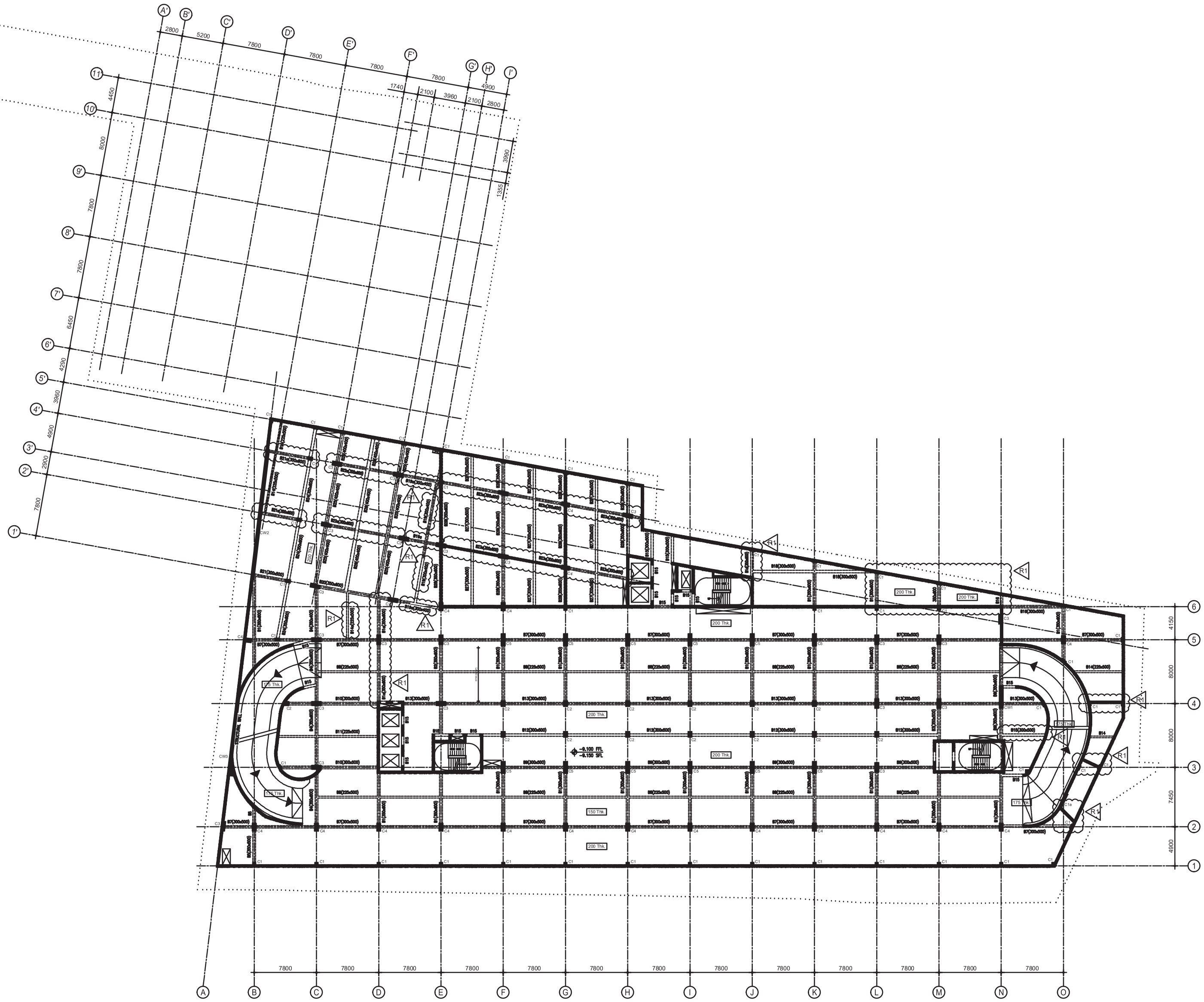
CONSTRUCTION DRAWINGS

Drawing Title: **BASEMENT - 02 GENERAL ARRANGEMENT.**

Drawing Number	S-20	Original Size	A0



01 BASEMENT - 03 GENERAL ARRANGEMENT.

S-17

SCALE 1:200

NOTE :-
1. PTB – POST TENSION BEAM.
2. RCB – REINFORCED CONCRETE BEAM.
3. C – REINFORCED COLUMN.
4. T – REINFORCED COLUMN TERMINATE.
5. FC – FLOATING REINFORCED COLUMN.

NOTE :-
STIFFENER COLUMNS FOR MASONRY WALLS ARE NOT SHOWN FOR CLARITY.
SLAB THICKNESS IS 150mm UNLESS OTHERWISE SPECIFIED.

PROJECT

Proposed 5 Basements & G + 7 Storeyed Mixed Development Project for ODEL at Ward Place, Colombo 07.

CLIENT

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ODEL PLC.
No.475/32, Kotte Road, Rajagiriya, Sri Lanka

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Email: qserveglobal@gmail.com

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STRUCTURAL ENGINEER'S SIGNATURE / SEAL

CLIENT'S SIGNATURE / SEAL

No.	Note	Date	Drawn	Index
R1	REVISIONS ARE CLOUDED	17.06.2018	INDIKA	

REVISIONS

Scale	1:200	Date	2018.05.21
Original Size	A0	Revision No	

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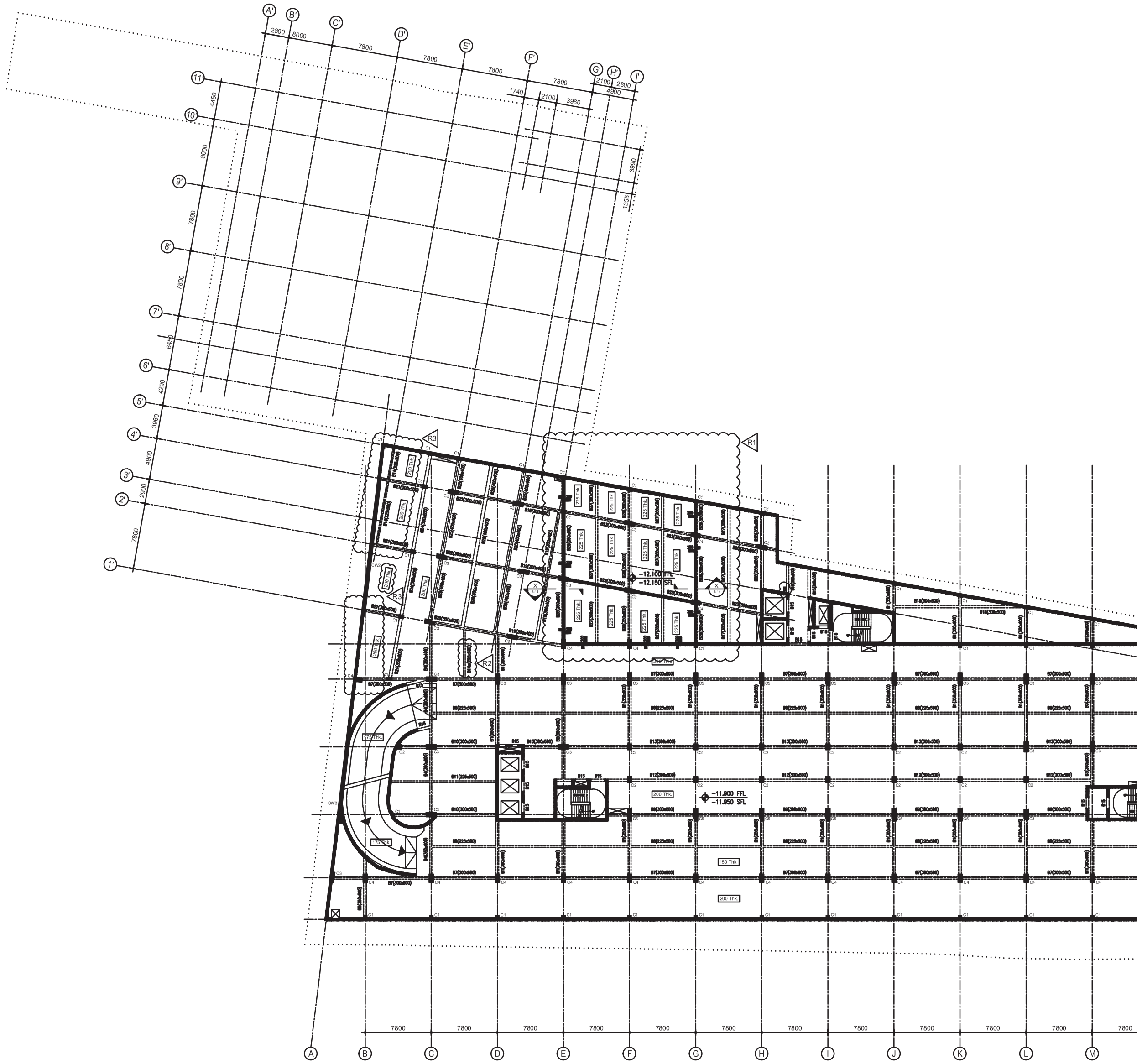
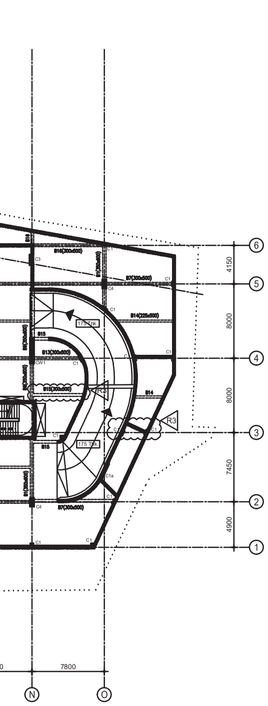
CONSTRUCTION DRAWINGS

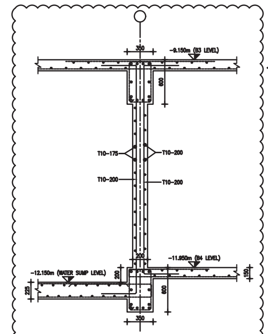
Drawing Title: **BASEMENT - 03 GENERAL ARRANGEMENT.**

Drawing Number: S-17 | Original Size: **A0**

BASEMENT - 04 GENERAL ARRANGEMENT.

01 S-14 SCALE 1:200



TYPICAL SECTION - X.

02 S-14 SCALE 1:25

NOTE :-

1. PTB – POST TENSION BEAM.
2. RCB – REINFORCED CONCRETE BEAM.
3. C – REINFORCED COLUMN.
4. T – REINFORCED COLUMN TERMINATE.
5. FC – FLOATING REINFORCED COLUMN.

NOTE :-

STIFFENER COLUMNS FOR MASONRY WALLS ARE NOT SHOWN FOR CLARITY.
SLAB THICKNESS IS 150mm UNLESS OTHERWISE SPECIFIED.

NOTE :-

- SDL = 0.9KN/m²
- Allowable Live Load = 3.5KN/m²
- Loading on slab, once constructed should not exceed the aforementioned values.
- Any higher loads during construction will require additional measures to be taken.

PROJECT

Proposed 5 Basements & G + 7 Storeyed Mixed Development Project for ODEL at Ward Place, Colombo 07.

CLIENT

ODEL

ODEL PLC.
No.475/32, Kotte Road, Rajagiriya, Sri Lanka

PROJECT MANAGEMENT

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STRUCTURAL ENGINEER

NCD CONSULTANTS (PVT) LTD.
No. 1A, Ananda Road, Melder Place, Nugegoda, Sri Lanka.
Tel: +94- 11-2809880
Email: ncdconsultantltd@gmail.com
Web: www.ncdconsultants.com

MEP CONSULTANT

G & G Associates.
Chartered Consulting Engineers
No. 19A, Jeswell Place, Mirihana, Nugegoda, Sri Lanka.
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ARCHITECT'S SIGNATURE / SEAL

STRUCTURAL ENGINEER'S SIGNATURE / SEAL

CLIENT'S SIGNATURE / SEAL

No.	Note	Date	Drawn	index
R3	REVISIONS ARE CLOUDED	17.09.2019	INDIKA	
R2	WATER SUMP AREA REVISED	27.08.2019	INDIKA	
R1	WATER SUMP AREA REVISED	08.08.2019	INDIKA	

REVISIONS

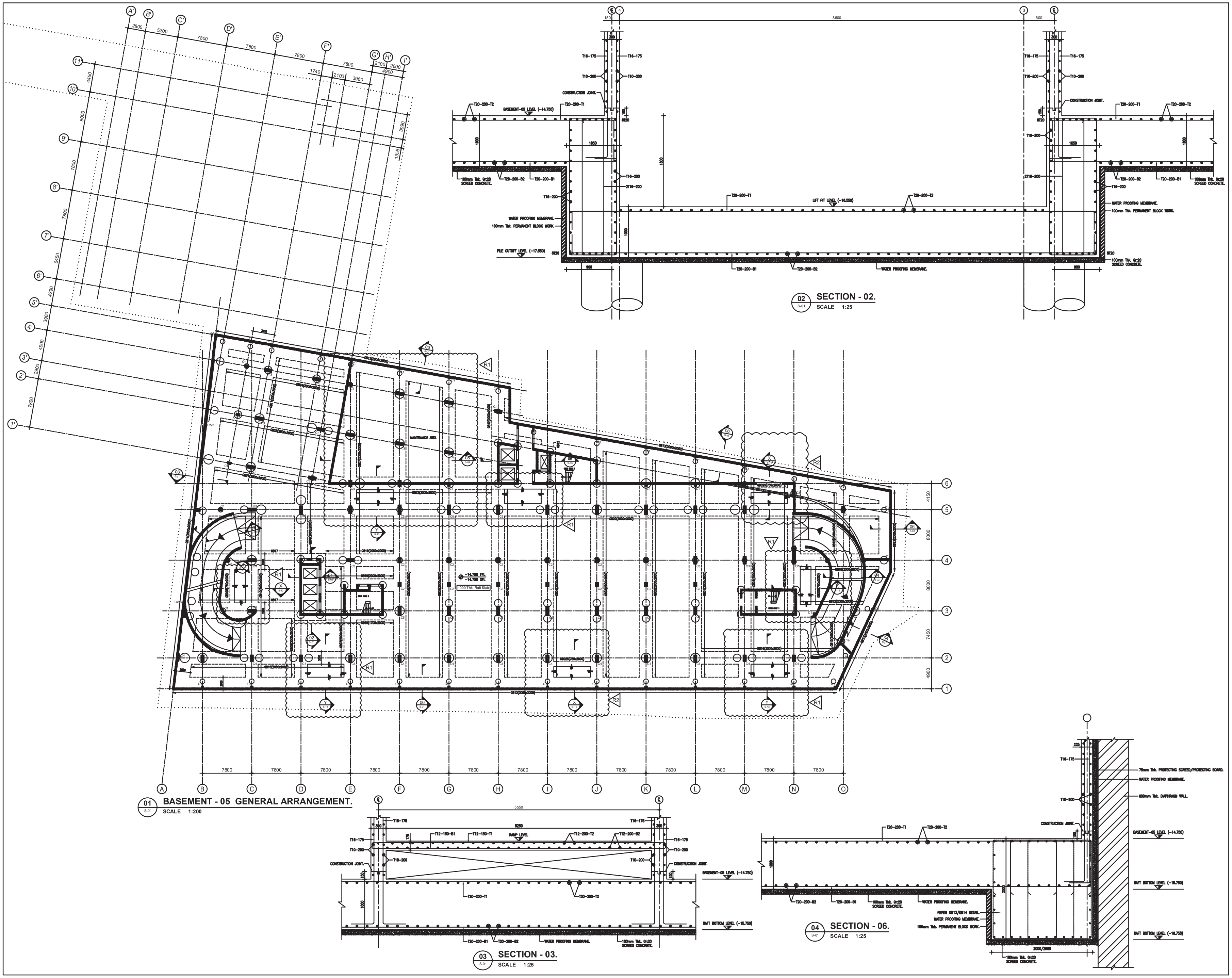
Scale	1:200 & 1:25	Date	2018.05.21
Original Size	A0	Revision No	R2

This Drawing must not be used for Construction unless signed as Approved

CONSTRUCTION DRAWINGS

Drawing Title: **BASEMENT - 04 GENERAL ARRANGEMENT.**

Drawing Number: S-14 | Original Size: A0



PROJECT

Proposed 5 Basements & G + 7 Storeyed Mixed Development Project for ODEL at Ward Place, Colombo 07.

CLIENT

ODEL

ODEL PLC.
No.475/32, Kotte Road, Rajagiriya,
Sri Lanka

PROJECT MANAGEMENT

Project Management Unit
ODEL PLC.
No.475/32, Kotte Road, Rajagiriya, Sri Lanka.
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Fax: +94 (0) 112-871333
Email : info@eodel.com

ARCHITECT

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www.blocherblocher.com

PROJECT ARCHITECT

Surath Wickramasinghe Associates / Archt. Upula Chandrakumara.
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Colombo 3, Sri Lanka
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STRUCTURAL ENGINEER

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MEP CONSULTANT

G & G Associates.
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Sri Lanka.
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Email: qserveglobal@gmail.com

ARCHITECT'S SIGNATURE / SEAL

STRUCTURAL ENGINEER'S SIGNATURE / SEAL

CLIENT'S SIGNATURE / SEAL

No.	Note	Date	Drawn	Index
R1	REVISIONS ARE INDICATE IN CLOUDS	13/05/2018	INDRA	

REVISIONS

Scale	1:200 & 1:25	Date	2018.05.21
Original Size	A0	Revision No	

This Drawing must not be used for Construction unless signed as Approved

CONSTRUCTION DRAWINGS

Drawing Title	BASEMENT - 05 GENERAL ARRANGEMENT LAYOUT & SECTIONS.		
Drawing Number	S-01	Original Size	A0

APPENDIX E

Monitoring Data

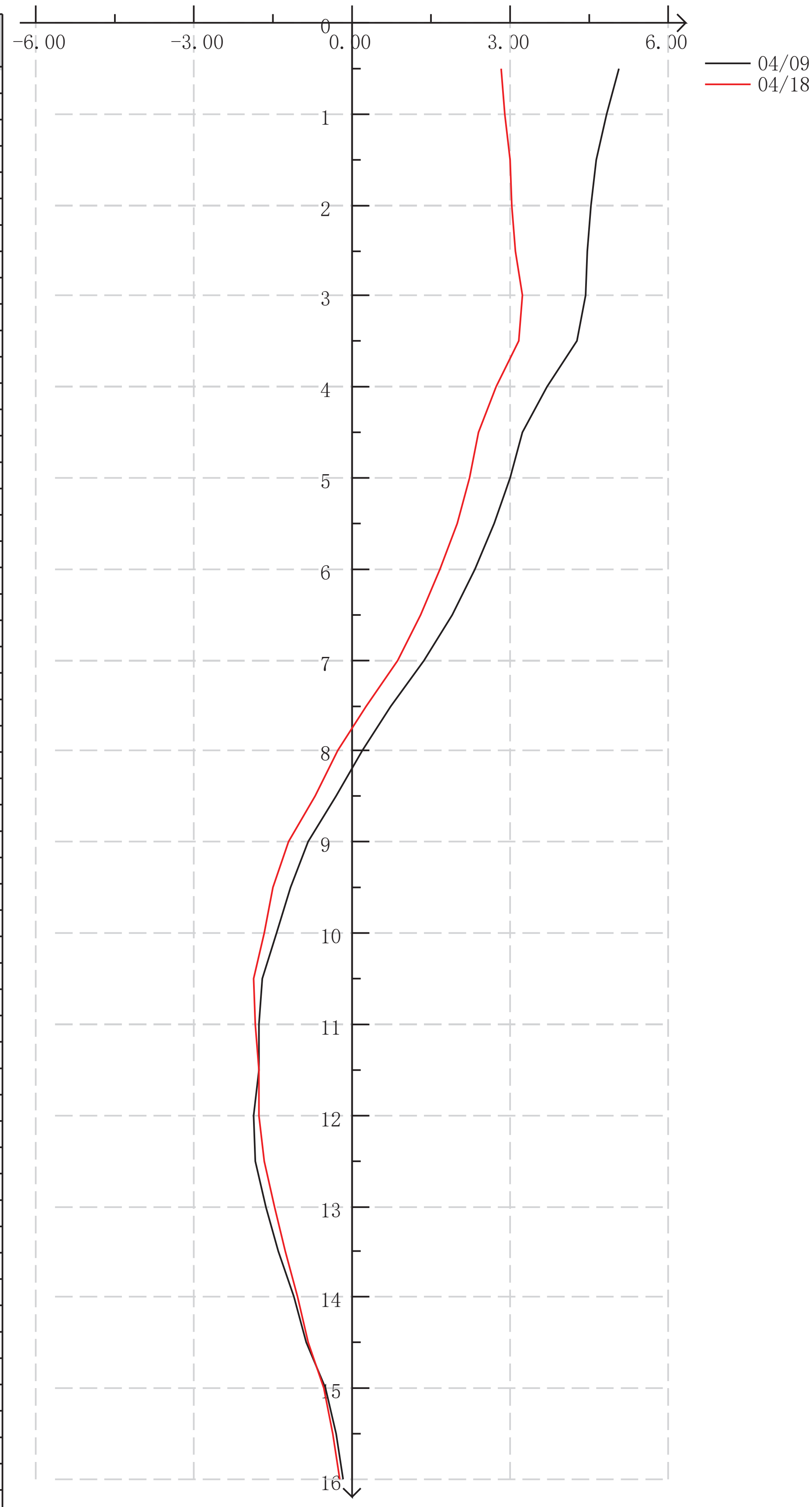
Analyze Report

Name: Deep soil lateral displacement　　No: #1006　　Hole Depth: 16.0m

Direction: S-N　　Point Name: TT-6

Date: his Time: 04/18　　Last Time: 04/09

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0.5	2.8	-2.3	-0.3
1.0	2.9	-1.9	-0.2
1.5	3.0	-1.6	-0.2
2.0	3.0	-1.5	-0.2
2.5	3.1	-1.4	-0.2
3.0	3.2	-1.2	-0.2
3.5	3.2	-1.1	-0.1
4.0	2.7	-1.0	-0.1
4.5	2.4	-0.9	-0.1
5.0	2.2	-0.8	-0.1
5.5	2.0	-0.7	-0.1
6.0	1.7	-0.6	-0.1
6.5	1.3	-0.6	-0.1
7.0	0.8	-0.5	-0.1
7.5	0.2	-0.5	-0.1
8.0	-0.3	-0.5	-0.1
8.5	-0.7	-0.4	-0.1
9.0	-1.2	-0.4	0.0
9.5	-1.5	-0.3	0.0
10.0	-1.7	-0.2	0.0
10.5	-1.9	-0.1	0.0
11.0	-1.9	-0.1	0.0
11.5	-1.8	0.0	0.0
12.0	-1.8	0.1	0.0
12.5	-1.7	0.2	0.0
13.0	-1.5	0.2	0.0
13.5	-1.3	0.2	0.0
14.0	-1.0	0.1	0.0
14.5	-0.8	0.0	0.0
15.0	-0.5	0.0	0.0
15.5	-0.4	-0.1	0.0
16.0	-0.3	-0.1	0.0



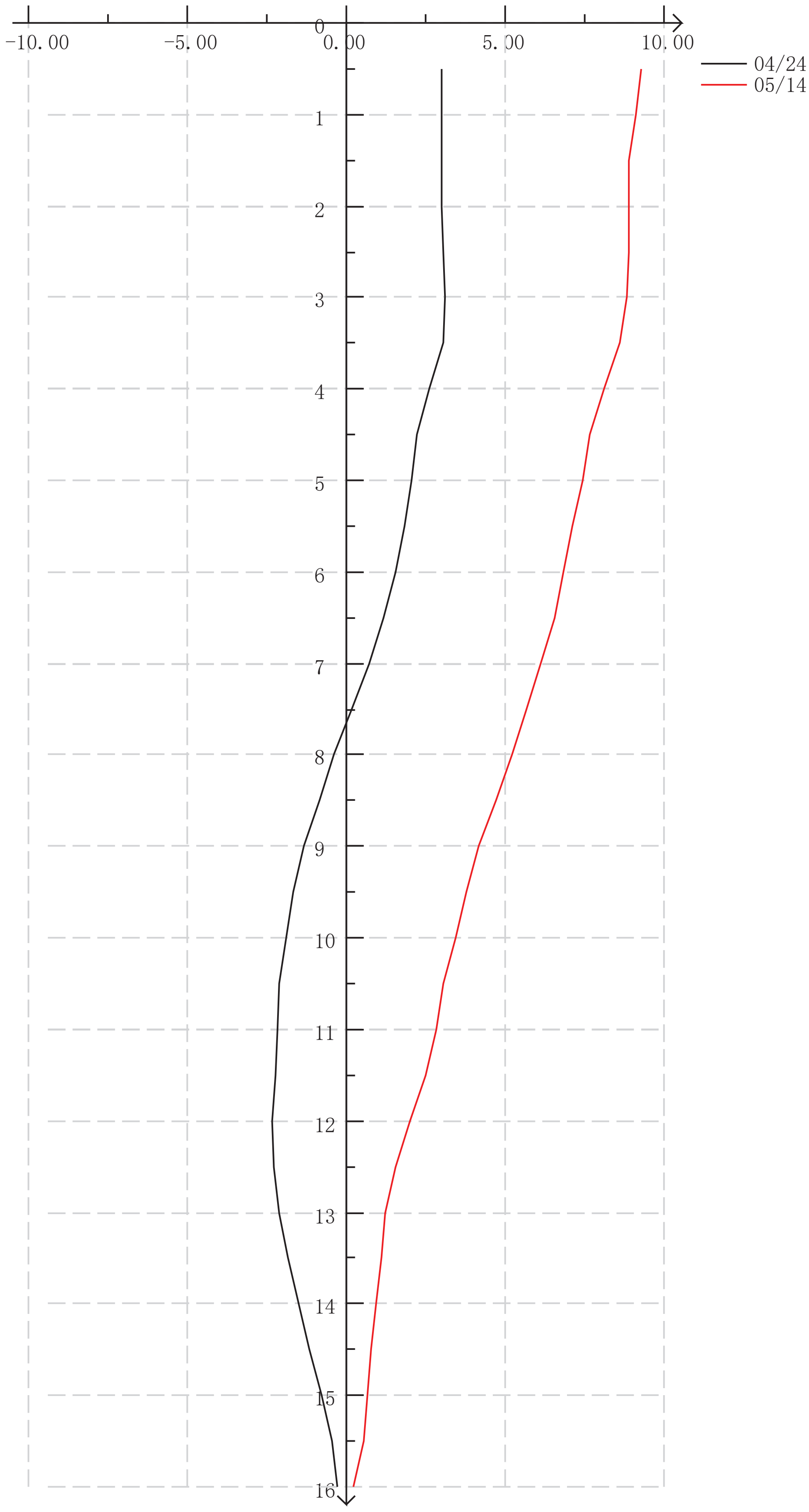
Analyze Report

Name: Deep soil lateral displacement　　No: #1006　　Hole Depth: 16.0mm

Direction: S-N　　Point Name: TT-6

Date: This Time: 05/14　　Last Time: 04/24

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0.5	9.3	6.3	0.3
1.0	9.1	6.1	0.3
1.5	8.9	5.9	0.3
2.0	8.9	5.9	0.3
2.5	8.9	5.8	0.3
3.0	8.8	5.7	0.3
3.5	8.6	5.6	0.3
4.0	8.1	5.5	0.3
4.5	7.7	5.4	0.3
5.0	7.5	5.4	0.3
5.5	7.1	5.3	0.3
6.0	6.8	5.3	0.3
6.5	6.5	5.4	0.3
7.0	6.1	5.4	0.3
7.5	5.7	5.5	0.3
8.0	5.2	5.6	0.3
8.5	4.7	5.6	0.3
9.0	4.1	5.5	0.3
9.5	3.7	5.4	0.3
10.0	3.4	5.3	0.3
10.5	3.0	5.2	0.3
11.0	2.8	5.0	0.3
11.5	2.5	4.7	0.2
12.0	2.0	4.3	0.2
12.5	1.5	3.9	0.2
13.0	1.2	3.4	0.2
13.5	1.1	3.0	0.1
14.0	0.9	2.5	0.1
14.5	0.7	1.9	0.1
15.0	0.6	1.4	0.1
15.5	0.5	1.0	0.0
16.0	0.2	0.5	0.0



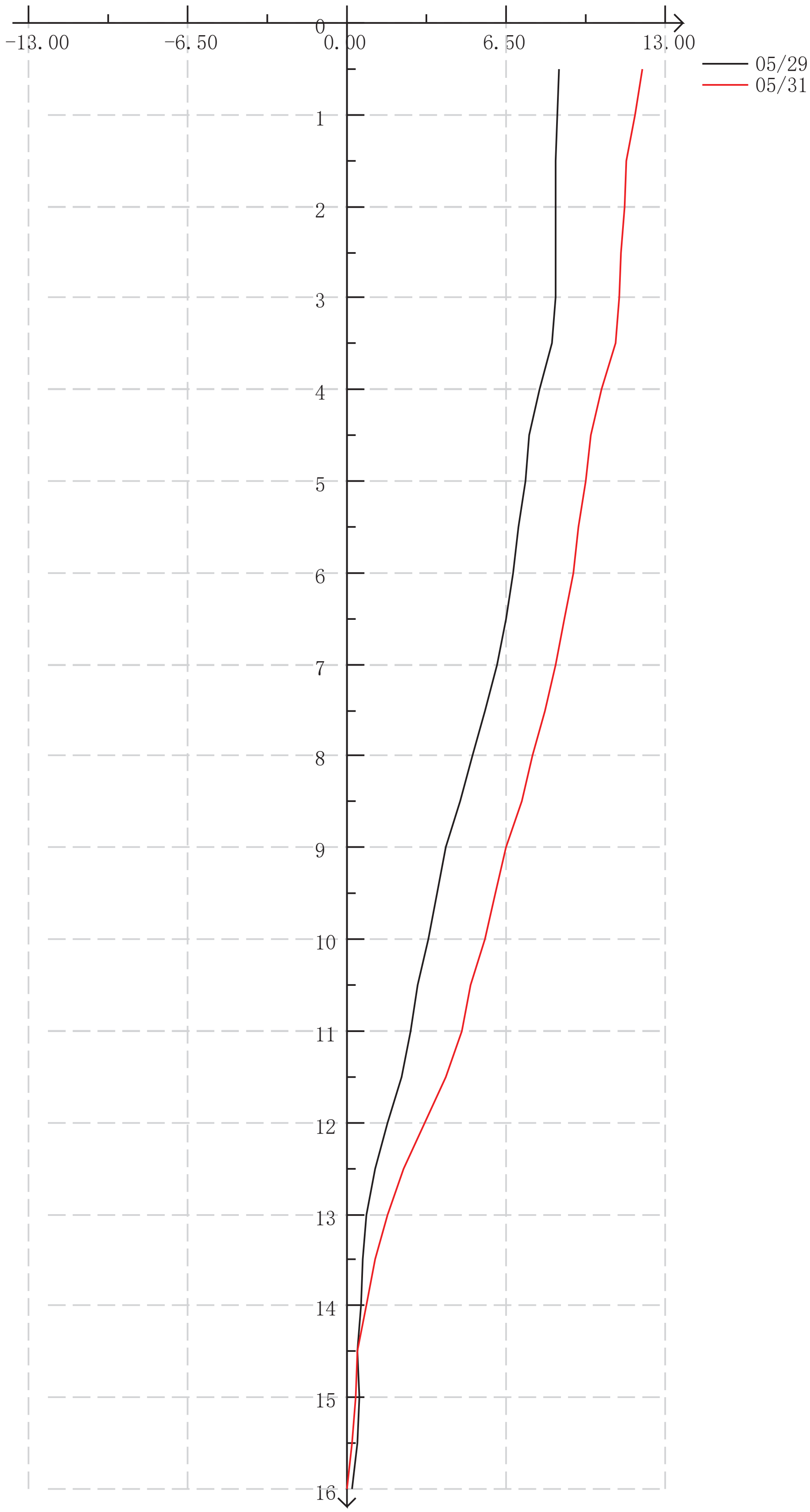
Analyze Report

Name: Deep soil lateral displacement No: #1006 Hole Depth: 16.0m

Direction: S-N Point Name: TT-6

Date: This Time: 05/31 Last Time: 05/29

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	12.1	3.4	1.7
1.0	11.8	3.2	1.6
1.5	11.4	2.9	1.5
2.0	11.3	2.8	1.4
2.5	11.2	2.7	1.3
3.0	11.2	2.6	1.3
3.5	11.0	2.6	1.3
4.0	10.4	2.6	1.3
4.5	10.0	2.5	1.3
5.0	9.8	2.5	1.3
5.5	9.5	2.5	1.3
6.0	9.2	2.4	1.2
6.5	8.9	2.4	1.2
7.0	8.5	2.4	1.2
7.5	8.1	2.5	1.2
8.0	7.6	2.5	1.2
8.5	7.1	2.5	1.2
9.0	6.5	2.4	1.2
9.5	6.0	2.4	1.2
10.0	5.6	2.3	1.2
10.5	5.0	2.2	1.1
11.0	4.6	2.0	1.0
11.5	4.0	1.8	0.9
12.0	3.1	1.5	0.7
12.5	2.3	1.2	0.6
13.0	1.6	0.8	0.4
13.5	1.1	0.5	0.3
14.0	0.8	0.3	0.1
14.5	0.4	0.0	0.0
15.0	0.3	-0.2	-0.1
15.5	0.2	-0.2	-0.1
16.0	0.0	-0.2	-0.1



Analyze Report

Name : Deep soil lateral displacement

No : #1006

Hole Depth : 16.0m

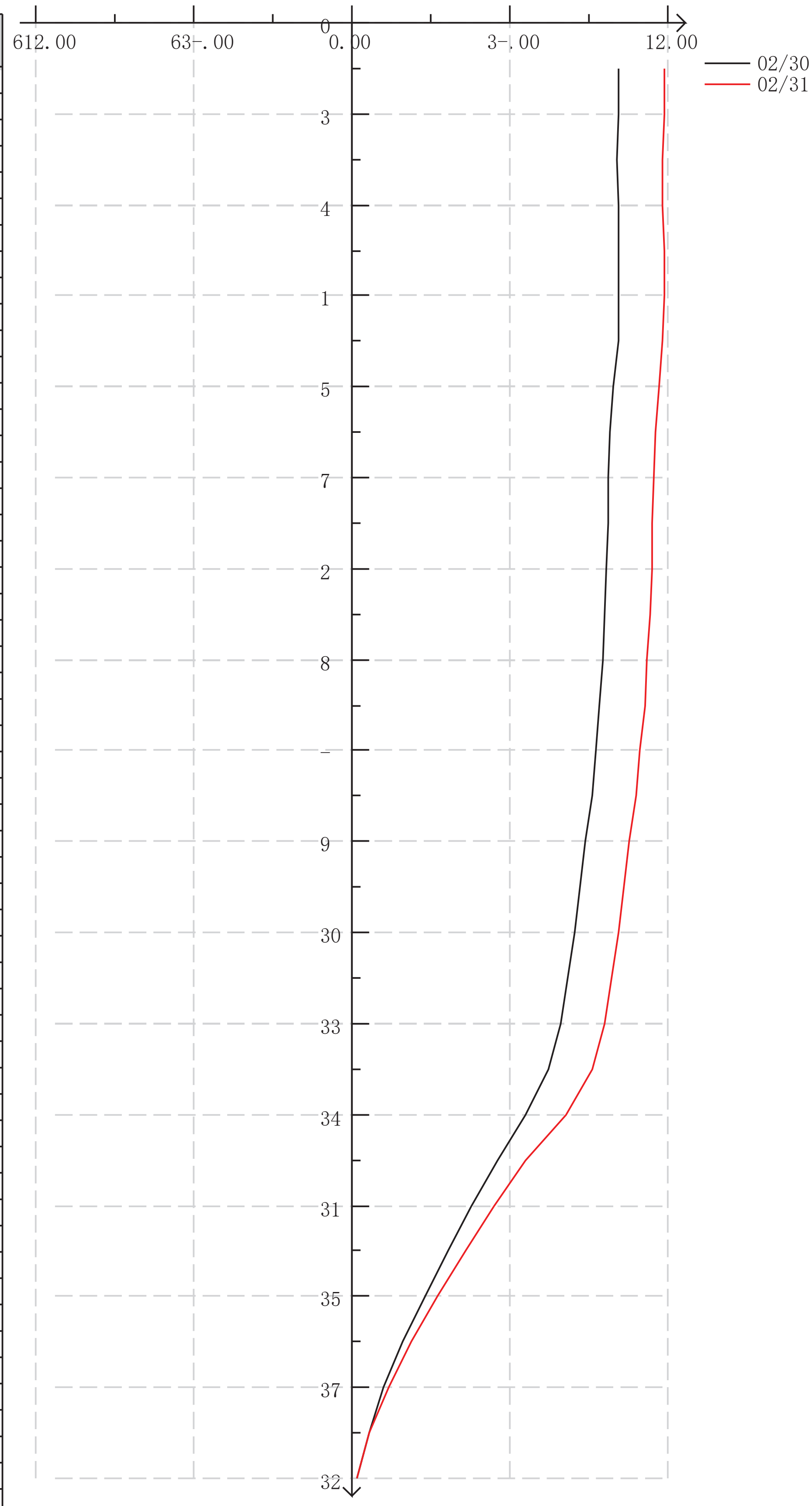
Direction : S-N

Point Name : TT-6

Date : This Time : 06/13

Last Time : 06/10

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0. 7	17. 2	7. 4	3. 8
3. 0	17. 2	7. 4	3. 8
3. 7	17. 7	7. 4	3. 8
4. 0	17. 7	7. 4	3. 8
4. 7	17. 7	7. 4	3. 8
1. 0	17. 2	7. 4	3. 8
1. 7	17. 7	7. 4	3. 8
5. 0	17. 0	7. 4	3. 8
5. 7	15. 2	7. 4	3. 8
7. 0	15. 5	7. 3	3. 8
7. 7	15. 4	7. 3	3. 8
2. 0	15. 3	7. 4	3. 8
2. 7	11. 9	7. 3	3. 8
8. 0	11. 2	7. 3	3. 8
8. 7	11. 1	7. 3	3. 8
−. 0	14. −	7. 3	3. 8
−. 7	14. 5	7. 3	3. 8
9. 0	13. 8	7. 3	3. 8
9. 7	13. 3	7. 3	3. 8
30. 0	10. 5	7. 0	3. 8
30. 7	49. 2	7. 0	3. 8
33. 0	4−. −	5. 9	3. 2
33. 7	48. 1	7. 0	3. 8
34. 0	45. 1	5. 2	3. 7
34. 7	39. −	1. 4	3. 3
31. 0	32. 3	4. 7	0. −
31. 7	34. 9	4. 0	0. 8
35. 0	9. 8	3. 5	0. 7
35. 7	2. 2	0. 9	0. 1
37. 0	5. 0	0. 5	0. 3
37. 7	3. 9	0. 3	0. 0
32. 0	0. 7	0. 0	0. 0



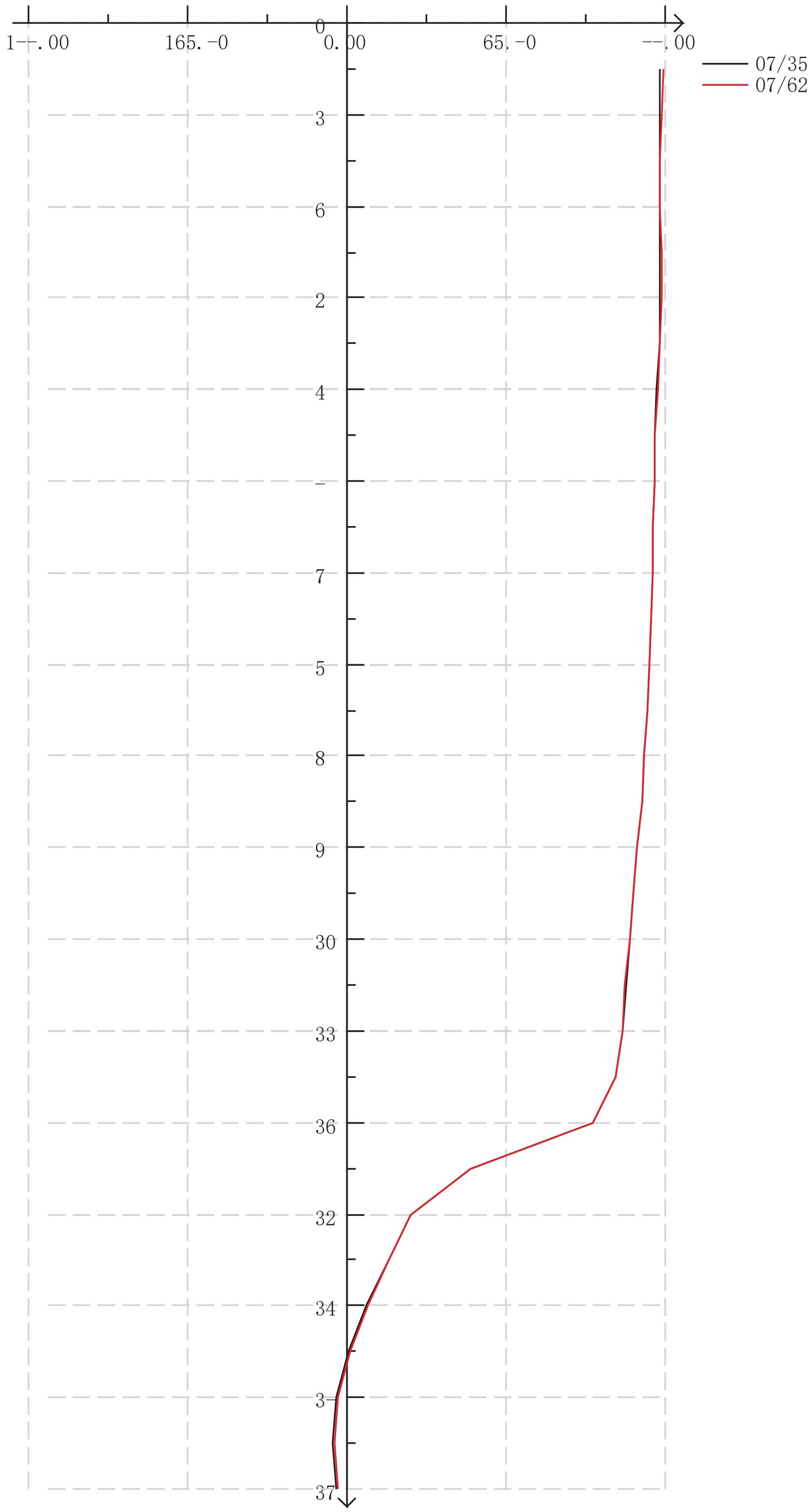
Analyze Report

Name: Deep soil lateral displacement　　No: #1006　　Hole Depth: 16.0m

Direction: S-N　　Point Name: TT-6

Date: This Time: 06/23　　Last Time: 06/17

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0. –	-4. 5	0. 7	0. 3
3. 0	-4. 4	0. 2	0. 3
3. –	-4. 6	0. 6	0. 0
6. 0	-4. 6	0. 2	0. 0
6. –	-4. 2	0. 2	0. 0
2. 0	-4. 4	0. 6	0. 0
2. –	-4. 6	0. 3	0. 0
4. 0	-2. 5	0. 3	0. 0
4. –	-2. 6	0. 0	0. 0
–. 0	-2. 3	0. 0	0. 0
–. –	-6. 9	0. 0	0. 0
7. 0	-6. 8	0. 0	0. 0
7. –	-6. 7	0. 0	0. 0
5. 0	-6. 2	0. 0	0. 0
5. –	-3. 9	0. 0	0. 0
8. 0	-3. 4	0. 0	0. 0
8. –	-0. 9	0. 0	0. 0
9. 0	-0. 3	0. 0	0. 0
9. –	49. –	10. 3	0. 0
30. 0	48. 8	10. 3	0. 0
30. –	48. 3	10. 3	0. 0
33. 0	45. –	10. 6	0. 0
33. –	47. 4	10. 3	0. 0
36. 0	46. 4	10. 3	0. 0
36. –	63. 2	0. 0	0. 0
32. 0	33. 0	0. 3	0. 0
32. –	5. 3	0. 3	0. 0
34. 0	2. –	0. 3	0. 0
34. –	0. 4	0. 2	0. 0
3–. 0	13. 7	0. 2	0. 0
3–. –	16. 2	0. 2	0. 3
37. 0	13. 8	0. 2	0. 0



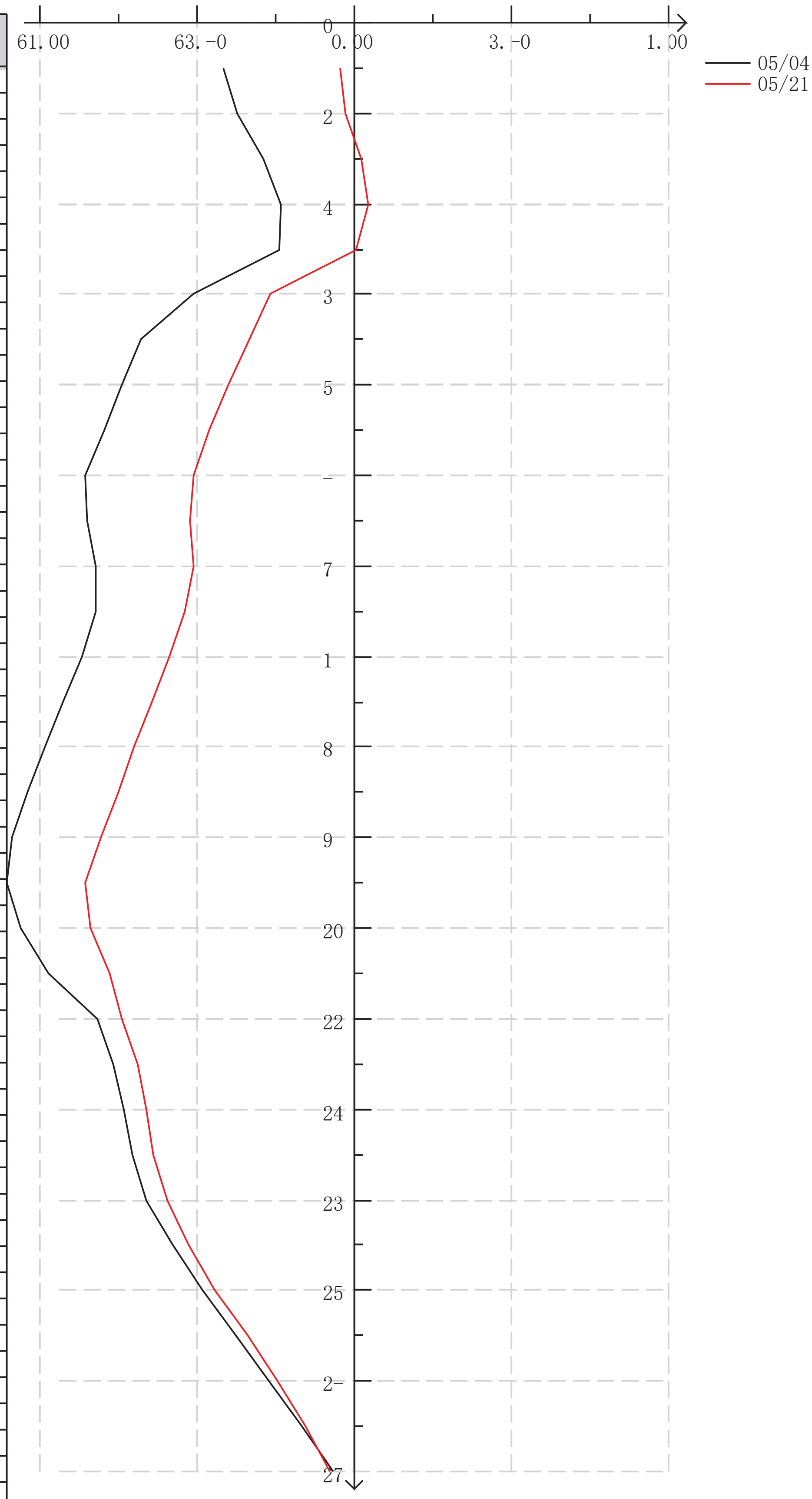
Analyze Report

Name: Deep soil lateral displacement   No: #1004   Hole Depth: 16.0m

Direction: W-E   Point Name: TT-4

Date: This Time: 04/17   Last Time: 04/02

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0. –	60. 3	4. 7	0. 4
2. 0	60. 4	4. –	0. 4
2. –	0. 2	4. 4	0. 4
4. 0	0. 3	2. 9	0. 2
4. –	0. 0	2. 1	0. 2
3. 0	62. 9	2. 1	0. 2
3. –	64. 5	4. 5	0. 4
5. 0	64. 8	4. 5	0. 4
5. –	63. 4	4. 5	0. 4
–. 0	63. 7	4. 5	0. 4
–. –	63. 1	4. 3	0. 4
7. 0	63. 7	4. 4	0. 4
7. –	63. 8	4. 0	0. 2
1. 0	65. 4	2. 9	0. 2
1. –	65. 7	4. 0	0. 2
8. 0	65. 9	4. 0	0. 2
8. –	6–. 3	4. 0	0. 2
9. 0	6–. 1	4. 0	0. 2
9. –	67. 0	2. 8	0. 2
20. 0	6–. 9	2. –	0. 2
20. –	6–. –	2. 5	0. 2
22. 0	6–. 4	0. –	0. 0
22. –	65. 9	0. –	0. 0
24. 0	65. 1	0. –	0. 0
24. –	65. –	0. –	0. 0
23. 0	65. 4	0. 5	0. 0
23. –	63. 1	0. 5	0. 0
25. 0	63. 2	0. 3	0. 0
25. –	64. 5	0. 3	0. 0
2–. 0	62. 8	0. 4	0. 0
2–. –	62. 2	0. 2	0. 0
27. 0	60. 7	60. 2	0. 0



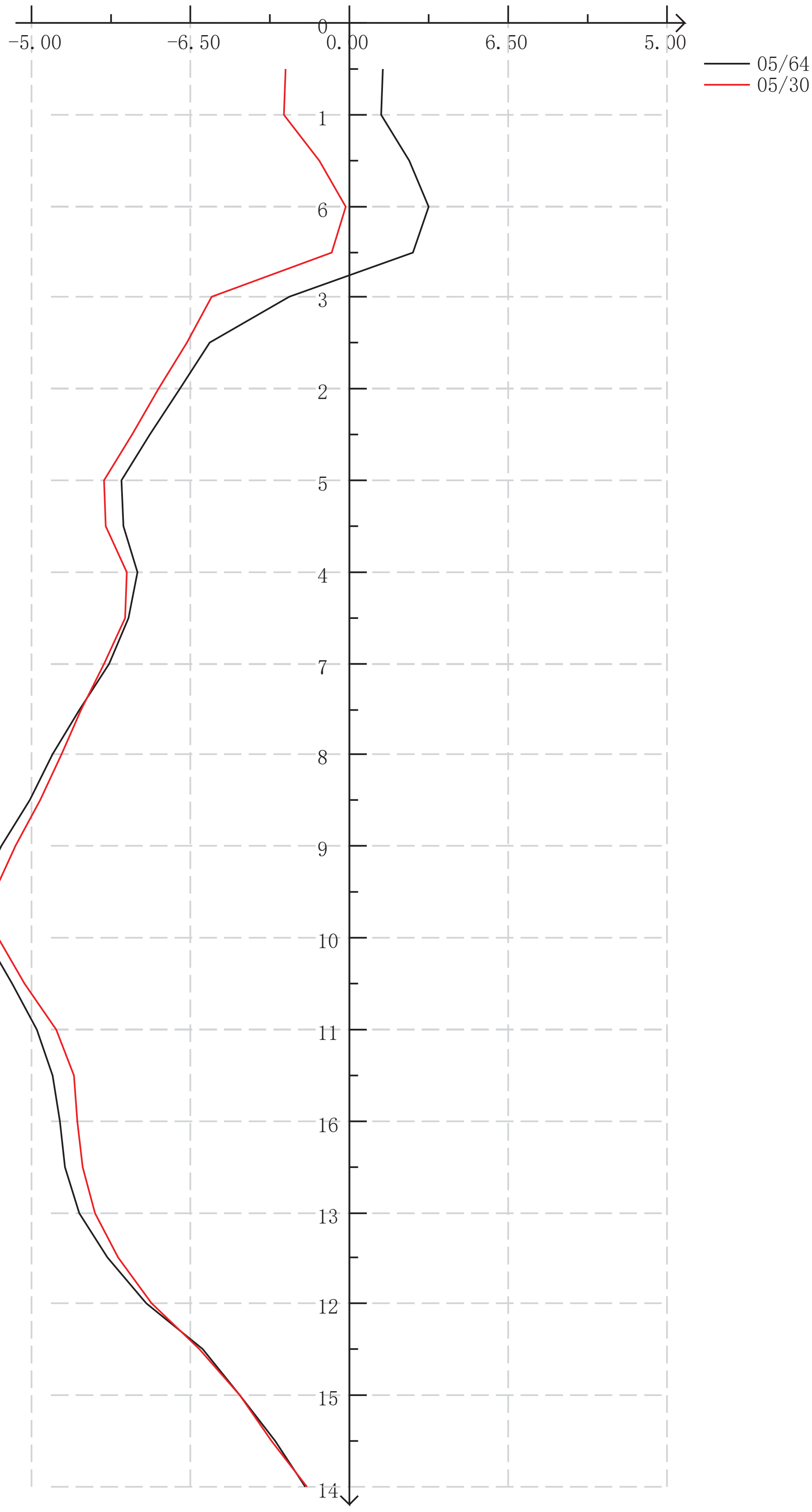
Analyze Report

Name: Deep soil lateral displacement　　No: #1004　　Hole Depth: 16.0m

Direction: W-E　　Point Name: TT-4

Date: This Time: 05/30　　Last Time: 05/26

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0.5	-1.0	-1.5	-0.5
1.0	-1.0	-1.5	-0.5
1.5	-0.5	-1.2	-0.5
6.0	-0.1	-1.3	-0.2
6.5	-0.3	-1.3	-0.2
3.0	-6.6	-1.6	-0.2
3.5	-6.4	-0.2	-0.1
2.0	-3.0	-0.3	-0.1
2.5	-3.2	-0.3	-0.1
5.0	-3.9	-0.3	-0.1
5.5	-3.9	-0.3	-0.1
4.0	-3.5	-0.6	-0.1
4.5	-3.4	-0.1	0.0
7.0	-3.9	-0.1	0.0
7.5	-2.6	0.0	0.0
8.0	-2.4	0.1	0.0
8.5	-2.9	0.6	0.1
9.0	-5.3	0.6	0.1
9.5	-5.4	0.3	0.1
10.0	-5.4	0.6	0.1
10.5	-5.1	0.6	0.1
11.0	-2.4	0.3	0.1
11.5	-2.2	0.3	0.1
16.0	-2.3	0.3	0.1
16.5	-2.6	0.3	0.1
13.0	-2.0	0.6	0.1
13.5	-3.7	0.6	0.1
12.0	-3.1	0.1	0.0
12.5	-6.2	-0.1	0.0
15.0	-1.7	0.0	0.0
15.5	-1.6	-0.1	0.0
14.0	-0.7	0.0	0.0



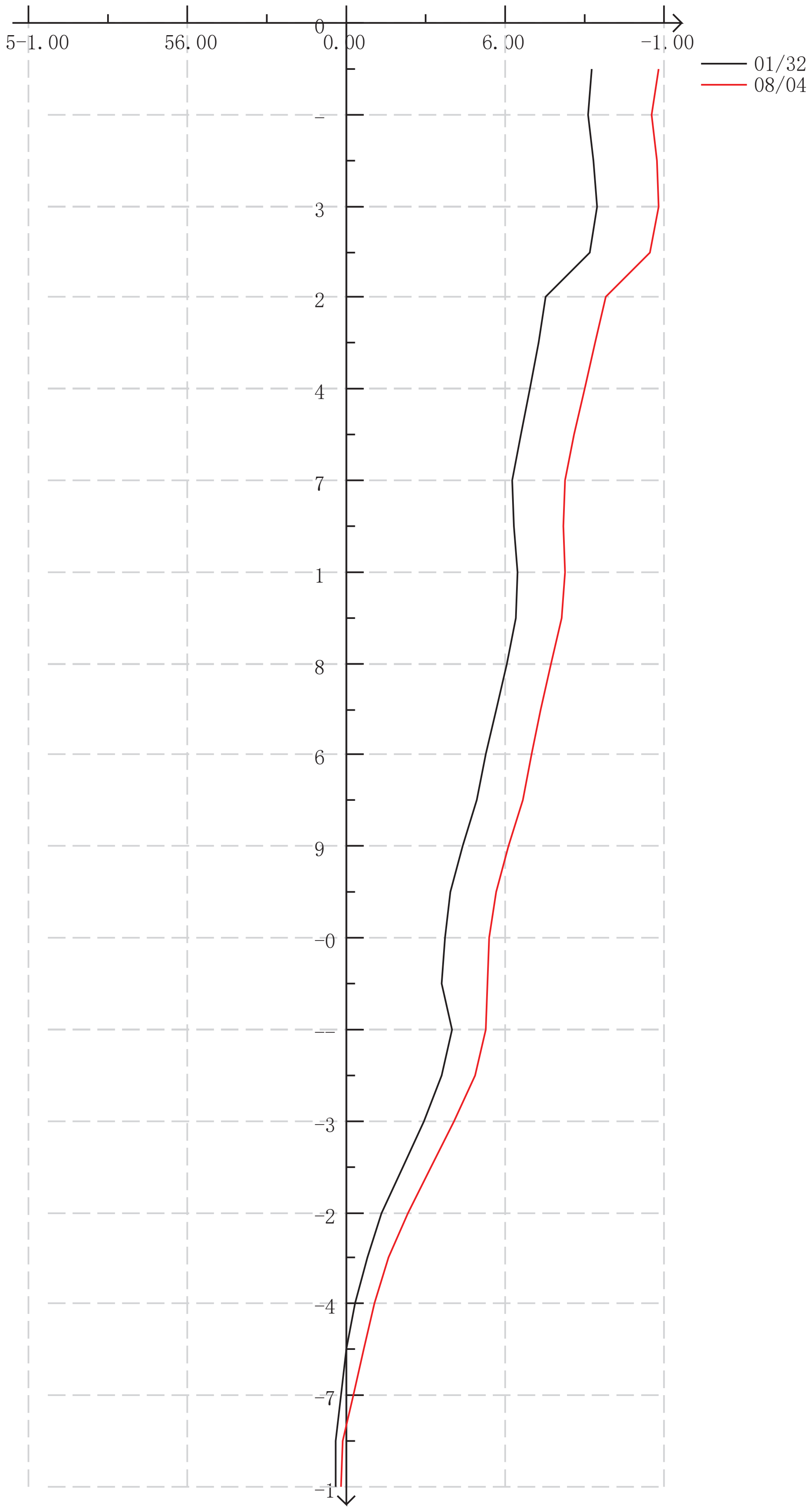
Analyze Report

Name: Deep soil lateral displacement　　No: #1004　　Hole Depth: 16.0m

Direction: W-E　　Point Name: TT-4

Date: This Time: 07/04　　Last Time: 06/23

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.7	−7.8	2.4	0.2
−.0	−7.4	2.3	0.2
−.7	−7.1	2.−	0.2
3.0	−7.8	2.−	0.2
3.7	−7.3	3.9	0.2
2.0	−2.−	2.0	0.2
2.7	−3.7	3.6	0.2
4.0	−3.0	3.6	0.2
4.7	−−.7	3.8	0.3
7.0	−−.0	3.1	0.3
7.7	−0.9	3.7	0.3
1.0	−−.0	3.4	0.3
1.7	−0.6	3.2	0.3
8.0	−0.2	3.3	0.3
8.7	9.6	3.2	0.3
6.0	9.2	3.2	0.3
6.7	6.6	3.2	0.3
9.0	6.3	3.2	0.3
9.7	8.7	3.2	0.3
−0.0	8.3	3.2	0.3
−0.7	8.−	3.2	0.3
−−.0	8.0	−.8	0.3
−−.7	1.4	−.1	0.−
−3.0	7.4	−.7	0.−
−3.7	4.3	−.4	0.−
−2.0	2.−	−.2	0.−
−2.7	3.−	−.−	0.−
−4.0	−.4	−.0	0.−
−4.7	0.6	0.6	0.−
−7.0	0.2	0.1	0.−
−7.7	50.3	0.4	0.0
−1.0	50.2	0.2	0.0



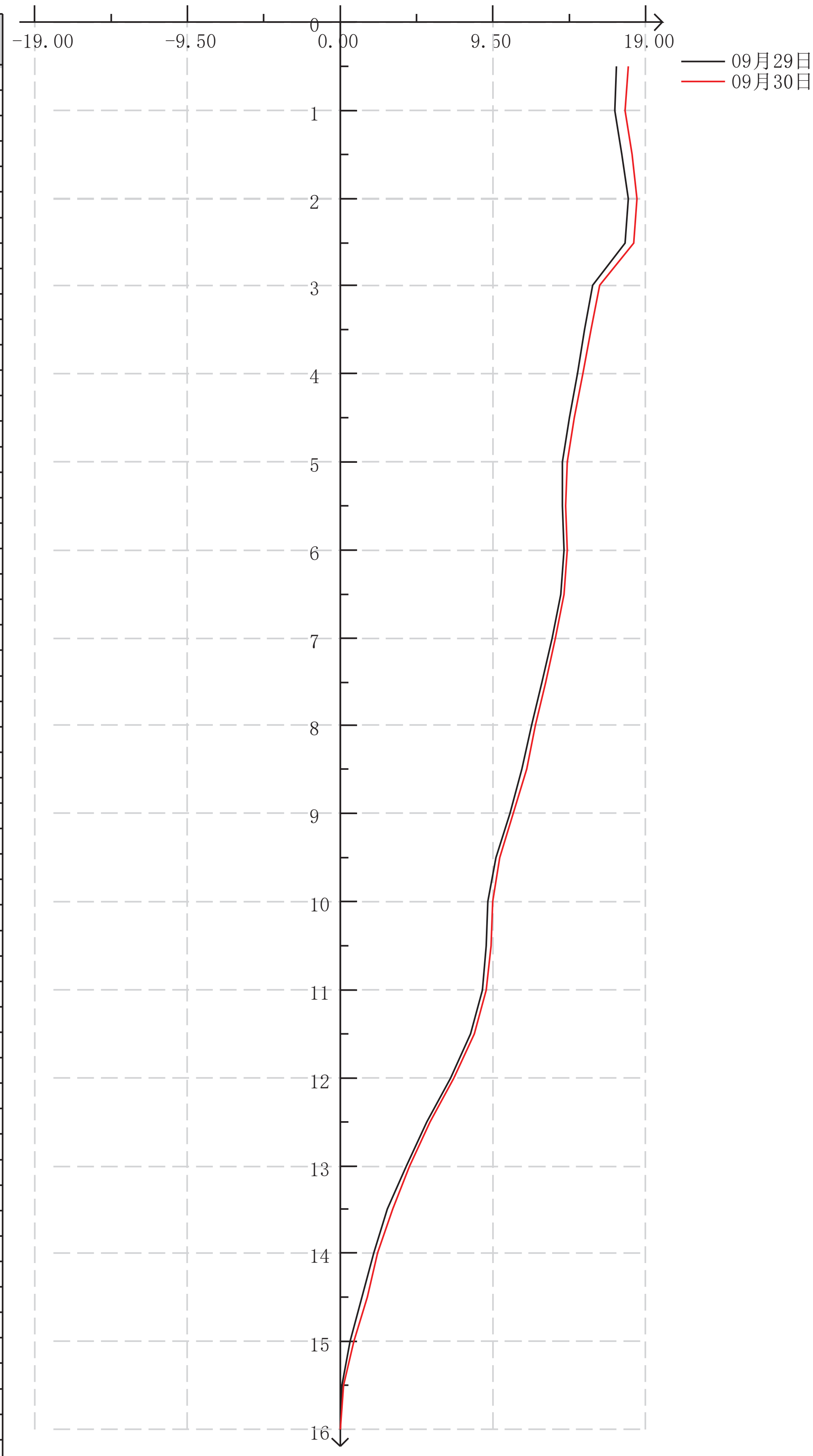
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1004　　观测孔深:16.0m

观测方向:W-E　　测孔位置:TT-4　　当前初值:19年01月03日10时14分57秒

观测日期:当次:09月30日　　前次:09月29日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	17.9	0.7	0.7
1.0	17.7	0.6	0.6
1.5	18.2	0.7	0.7
2.0	18.5	0.6	0.6
2.5	18.3	0.5	0.5
3.0	16.1	0.4	0.4
3.5	15.6	0.4	0.4
4.0	15.1	0.3	0.3
4.5	14.6	0.3	0.3
5.0	14.1	0.3	0.3
5.5	14.0	0.2	0.2
6.0	14.1	0.2	0.2
6.5	13.9	0.2	0.2
7.0	13.4	0.2	0.2
7.5	12.7	0.2	0.2
8.0	12.1	0.2	0.2
8.5	11.6	0.3	0.3
9.0	10.8	0.3	0.3
9.5	9.9	0.3	0.3
10.0	9.5	0.3	0.3
10.5	9.3	0.3	0.3
11.0	9.0	0.2	0.2
11.5	8.3	0.2	0.2
12.0	7.1	0.2	0.2
12.5	5.6	0.2	0.2
13.0	4.3	0.2	0.2
13.5	3.2	0.2	0.2
14.0	2.3	0.3	0.3
14.5	1.6	0.3	0.3
15.0	0.8	0.3	0.3
15.5	0.1	0.1	0.1
16.0	-0.1	0.0	0.0



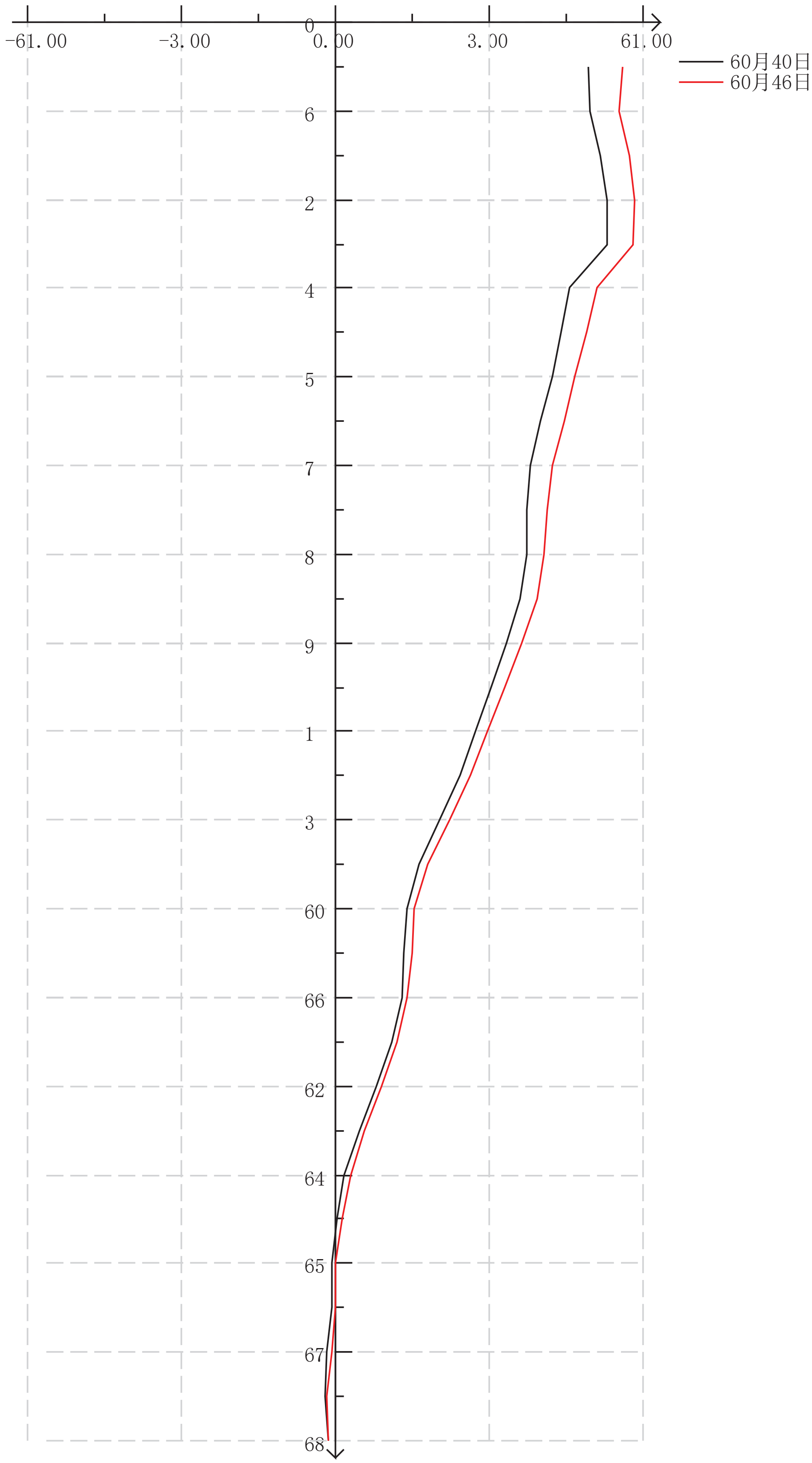
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1004　　观测孔深:16.0m

观测方向:W-E　　测孔位置:TT-4　　当前初值:19年01月03日10时14分57秒

观测日期:当次:10月31日　　前次:10月30日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.7	68.1	2.0	2.0
6.0	68.8	6.1	6.1
6.7	69.2	6.9	6.9
2.0	69.7	6.8	6.8
2.7	69.5	6.7	6.7
4.0	67.4	6.8	6.8
4.7	65.8	6.5	6.5
5.0	65.0	6.5	6.5
5.7	64.5	6.4	6.4
7.0	62.9	6.4	6.4
7.7	62.5	6.4	6.4
8.0	62.2	6.6	6.6
8.7	66.1	6.0	6.0
9.0	60.3	0.3	0.3
9.7	3.3	0.1	0.1
1.0	1.3	0.9	0.9
1.7	9.3	0.8	0.8
3.0	8.8	0.8	0.8
3.7	7.4	0.7	0.7
60.0	5.8	0.7	0.7
60.7	5.5	0.5	0.5
66.0	5.2	0.4	0.4
66.7	4.8	0.4	0.4
62.0	2.9	0.4	0.4
62.7	6.9	0.4	0.4
64.0	0.1	0.4	0.4
64.7	0.4	0.4	0.4
65.0	0.0	0.2	0.2
65.7	-0.6	0.2	0.2
67.0	-0.4	0.2	0.2
67.7	-0.8	0.6	0.6
68.0	-0.7	-0.6	-0.6



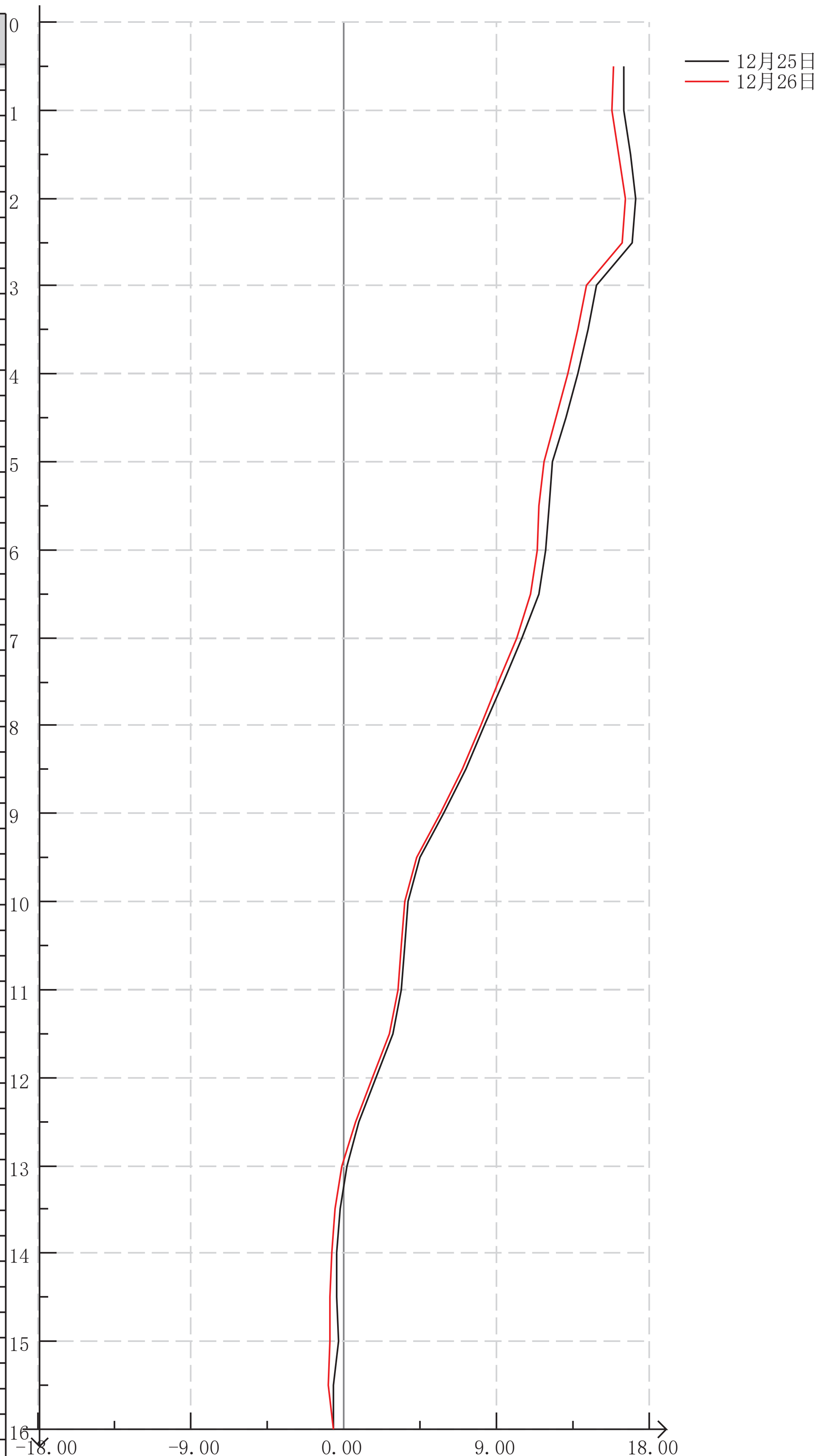
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1004　　观测孔深:16.0 m

观测方向:W-E　　测孔位置:TT-4　　当前初值:19年01月03日10时14分57秒

观测日期:当次:12月26日　　前次:12月25日

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0.5	15.9	-0.6	-0.6
1.0	15.8	-0.7	-0.7
1.5	16.2	-0.6	-0.6
2.0	16.6	-0.5	-0.5
2.5	16.4	-0.6	-0.6
3.0	14.3	-0.6	-0.6
3.5	13.8	-0.6	-0.6
4.0	13.2	-0.6	-0.6
4.5	12.5	-0.6	-0.6
5.0	11.7	-0.6	-0.6
5.5	11.5	-0.6	-0.6
6.0	11.3	-0.5	-0.5
6.5	11.0	-0.4	-0.4
7.0	10.2	-0.3	-0.3
7.5	9.1	-0.2	-0.2
8.0	8.0	-0.2	-0.2
8.5	7.0	-0.2	-0.2
9.0	5.6	-0.2	-0.2
9.5	4.3	-0.2	-0.2
10.0	3.6	-0.2	-0.2
10.5	3.4	-0.2	-0.2
11.0	3.2	-0.2	-0.2
11.5	2.7	-0.2	-0.2
12.0	1.7	-0.2	-0.2
12.5	0.7	-0.2	-0.2
13.0	-0.2	-0.3	-0.3
13.5	-0.6	-0.3	-0.3
14.0	-0.8	-0.3	-0.3
14.5	-0.9	-0.4	-0.4
15.0	-0.8	-0.4	-0.4
15.5	-0.9	-0.3	-0.3
16.0	-0.7	0.0	0.0



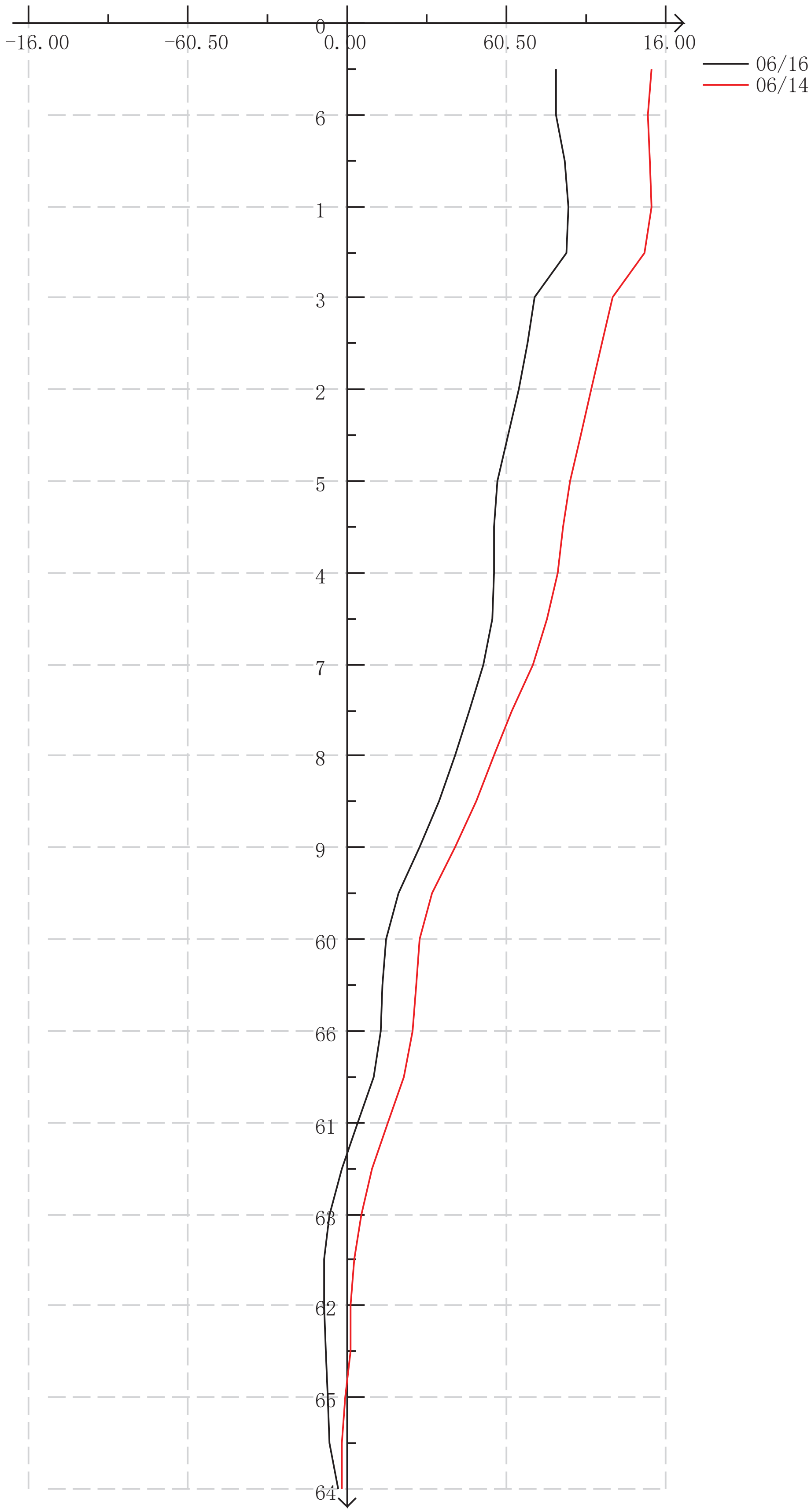
Analyze Report

Name: Deep soil lateral displacement　　No: #1004　　Hole Depth: 16.0m

Direction: W-E　　Point Name: TT-4

Date: This Time: 01/26　　Last Time: 01/21

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0.5	10.6	4.2	6.3
6.0	69.9	4.6	6.1
6.5	10.0	5.7	6.6
1.0	10.6	5.5	6.6
1.5	69.4	5.1	6.0
3.0	67.5	5.6	6.0
3.5	64.8	2.9	6.0
2.0	64.6	2.9	6.0
2.5	65.2	2.8	6.0
5.0	62.7	2.9	6.0
5.5	62.1	2.4	0.9
4.0	63.8	2.1	0.8
4.5	63.1	3.4	0.7
7.0	61.1	3.1	0.4
7.5	60.9	1.8	0.4
8.0	9.4	1.4	0.5
8.5	8.5	1.2	0.5
9.0	7.0	1.3	0.5
9.5	5.4	1.1	0.2
60.0	2.7	1.1	0.2
60.5	2.5	1.1	0.2
66.0	2.1	1.0	0.2
66.5	3.7	1.0	0.2
61.0	1.7	1.0	0.2
61.5	6.4	1.0	0.2
63.0	0.9	1.0	0.2
63.5	0.2	1.0	0.2
62.0	0.1	6.8	0.2
62.5	0.1	6.7	0.3
65.0	-0.6	6.1	0.1
65.5	-0.2	0.9	0.1
64.0	-0.2	0.3	0.6



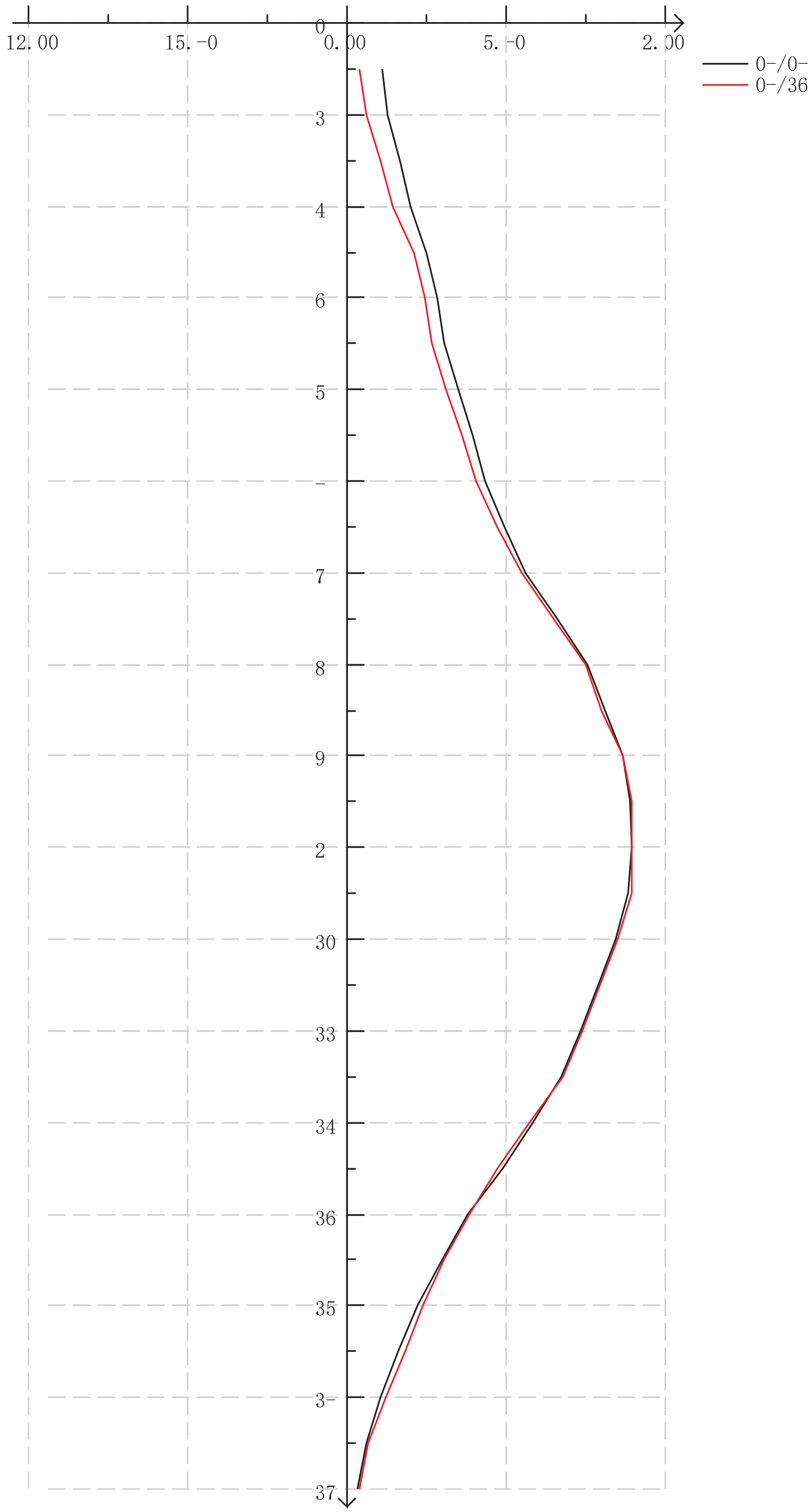
Analyze Report

Name: Deep soil lateral displacement   No: #1002   Hole Depth: 16.0m

Direction: N-S   Point Name: TT-2

Date: This Time: 05/13   Last Time: 05/05

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0. –	0. 6	10. 8	10. 3
3. 0	0. –	10. 7	10. 3
3. –	0. 2	10. 7	10. 3
4. 0	3. 6	10. –	10. 3
4. –	3. 2	10. 5	0. 0
6. 0	4. 4	10. 5	0. 0
6. –	4. 5	10. 5	0. 0
5. 0	4. 9	10. 6	0. 0
5. –	6. 4	10. 6	0. 0
–. 0	6. 8	10. 4	0. 0
–. –	5. 4	10. 4	0. 0
7. 0	5. 2	10. 3	0. 0
7. –	–. 2	10. 3	0. 0
8. 0	7. 8	10. 3	0. 0
8. –	8. 4	10. 3	0. 0
9. 0	8. 9	0. 0	0. 0
9. –	9. 0	0. 0	0. 0
2. 0	9. 3	0. 0	0. 0
2. –	9. 0	0. 3	0. 0
30. 0	8. 7	0. 0	0. 0
30. –	8. 3	0. 0	0. 0
33. 0	7. 7	0. 0	0. 0
33. –	7. 3	0. 0	0. 0
34. 0	–. 4	10. 3	0. 0
34. –	5. 4	10. 4	0. 0
36. 0	6. 5	0. 3	0. 0
36. –	4. 8	0. 3	0. 0
35. 0	4. 3	0. 3	0. 0
35. –	3. 7	0. 4	0. 0
3–. 0	3. 3	0. 3	0. 0
3–. –	0. 7	0. 3	0. 0
37. 0	0. 6	0. 0	0. 0



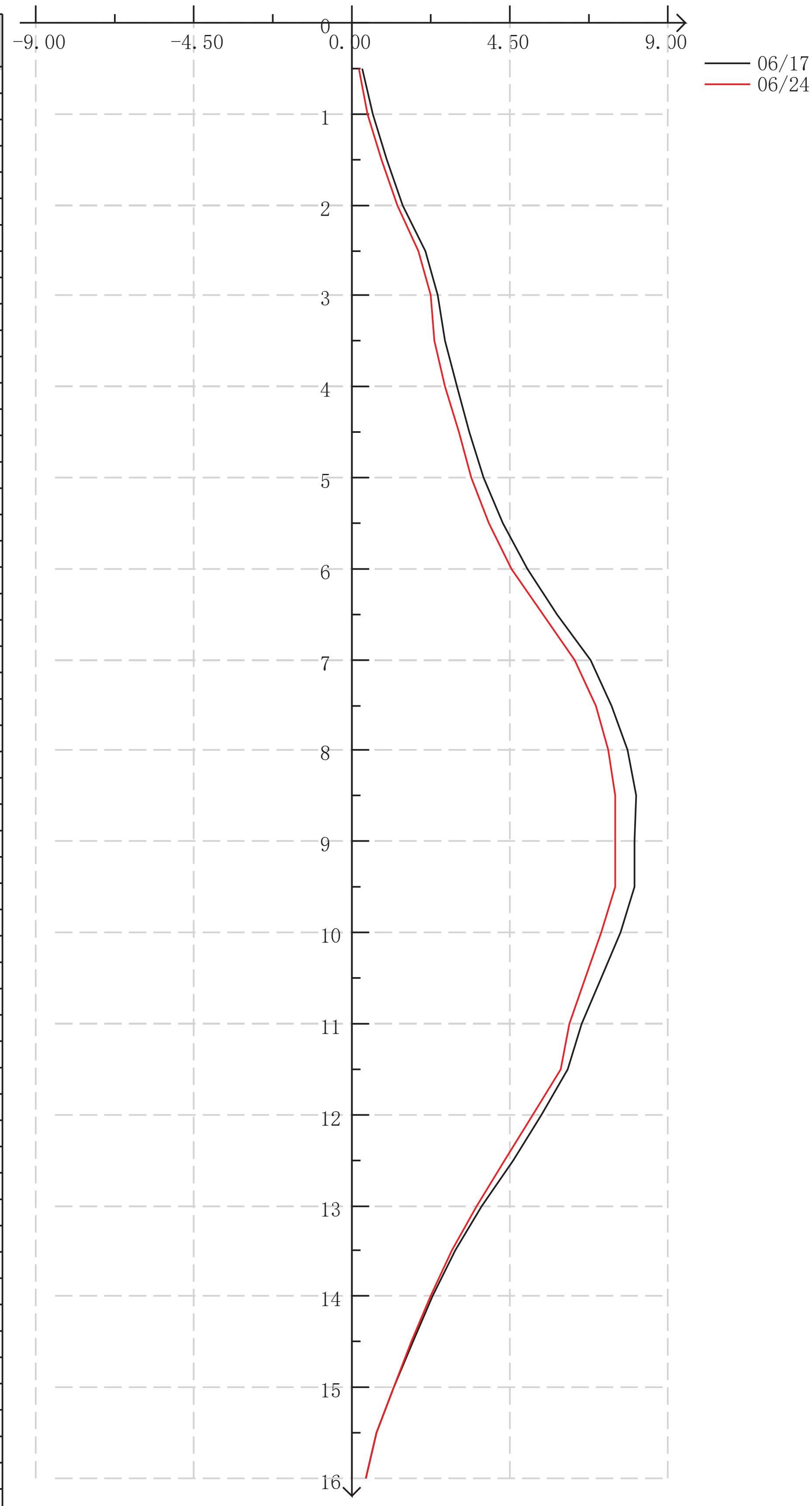
Analyze Report

Name: Deep soil lateral displacement　　No: #1002　　Hole Depth: 16.0m

Direction: N-S　　Point Name: TT-2

Date: This Time: 06/24　　Last Time: 06/17

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0.5	0.2	-0.1	0.0
1.0	0.4	-0.2	0.0
1.5	0.8	-0.2	0.0
2.0	1.3	-0.2	0.0
2.5	1.9	-0.2	0.0
3.0	2.2	-0.2	0.0
3.5	2.3	-0.3	0.0
4.0	2.7	-0.3	0.0
4.5	3.0	-0.3	0.0
5.0	3.4	-0.4	-0.1
5.5	3.9	-0.4	-0.1
6.0	4.5	-0.4	-0.1
6.5	5.4	-0.4	-0.1
7.0	6.3	-0.5	-0.1
7.5	6.9	-0.5	-0.1
8.0	7.3	-0.5	-0.1
8.5	7.5	-0.6	-0.1
9.0	7.5	-0.6	-0.1
9.5	7.5	-0.6	-0.1
10.0	7.1	-0.6	-0.1
10.5	6.6	-0.5	-0.1
11.0	6.2	-0.4	-0.1
11.5	5.9	-0.2	0.0
12.0	5.1	-0.3	0.0
12.5	4.3	-0.3	0.0
13.0	3.5	-0.1	0.0
13.5	2.8	-0.1	0.0
14.0	2.2	0.0	0.0
14.5	1.7	0.0	0.0
15.0	1.2	0.0	0.0
15.5	0.7	0.0	0.0
16.0	0.4	0.0	0.0



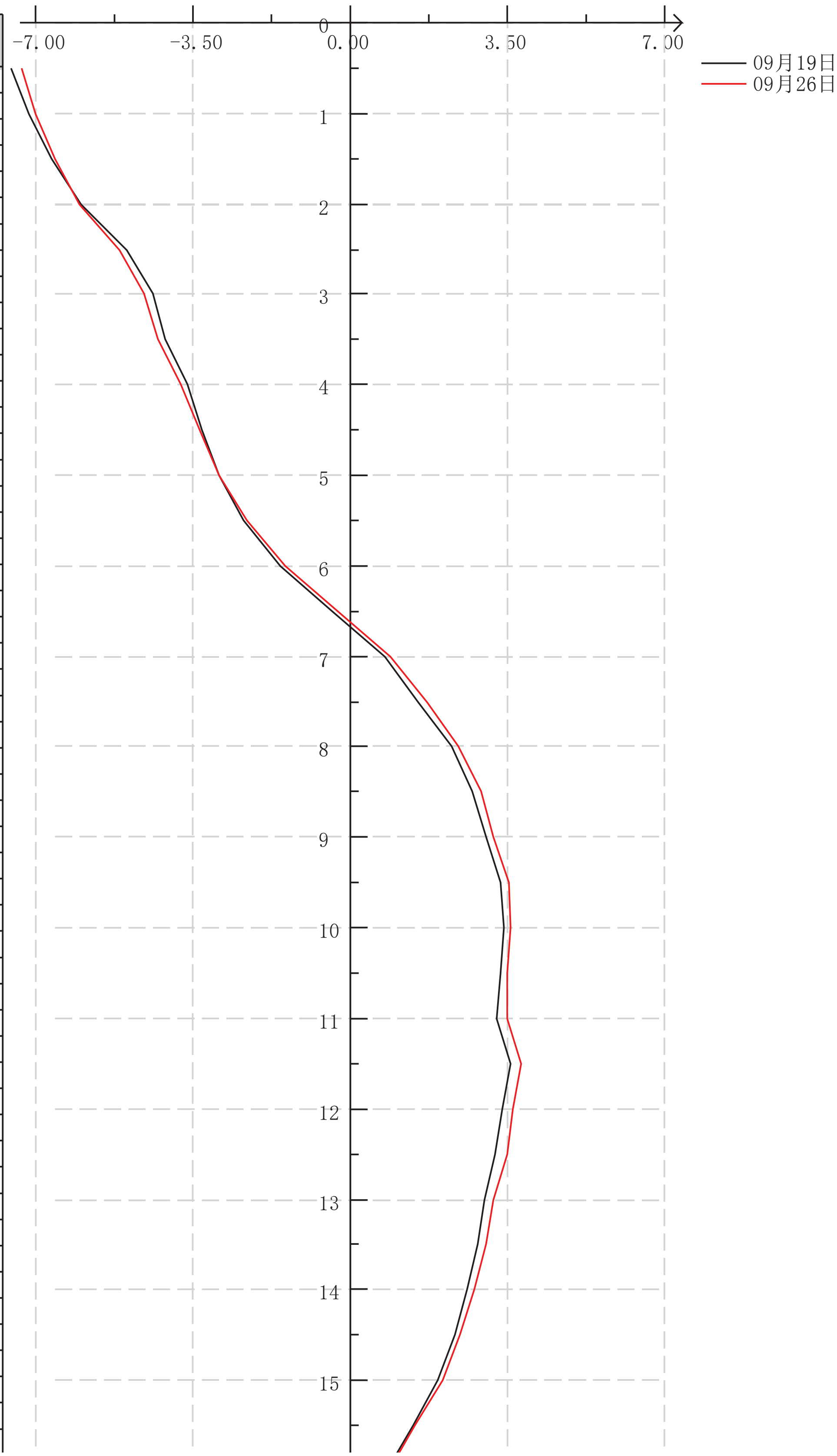
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1002　　观测孔深:16.0m

观测方向:N-S　　测孔位置:TT-2　　当前初值:19年02月09日09时52分32秒

观测日期:当次:09月26日　　前次:09月19日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variatrion
0.5	-7.4	0.2	0.0
1.0	-7.0	0.2	0.0
1.5	-6.6	0.1	0.0
2.0	-6.1	0.0	0.0
2.5	-5.2	-0.2	0.0
3.0	-4.6	-0.2	0.0
3.5	-4.3	-0.2	0.0
4.0	-3.8	-0.1	0.0
4.5	-3.4	0.0	0.0
5.0	-2.9	0.0	0.0
5.5	-2.3	0.1	0.0
6.0	-1.5	0.1	0.0
6.5	-0.3	0.1	0.0
7.0	0.9	0.2	0.0
7.5	1.7	0.2	0.0
8.0	2.4	0.2	0.0
8.5	2.9	0.2	0.0
9.0	3.2	0.2	0.0
9.5	3.5	0.2	0.0
10.0	3.6	0.2	0.0
10.5	3.5	0.2	0.0
11.0	3.5	0.2	0.0
11.5	3.8	0.2	0.0
12.0	3.6	0.2	0.0
12.5	3.5	0.3	0.0
13.0	3.2	0.2	0.0
13.5	3.0	0.2	0.0
14.0	2.8	0.2	0.0
14.5	2.4	0.1	0.0
15.0	2.0	0.1	0.0
15.5	1.4	0.0	0.0
16.0	0.8	0.0	0.0



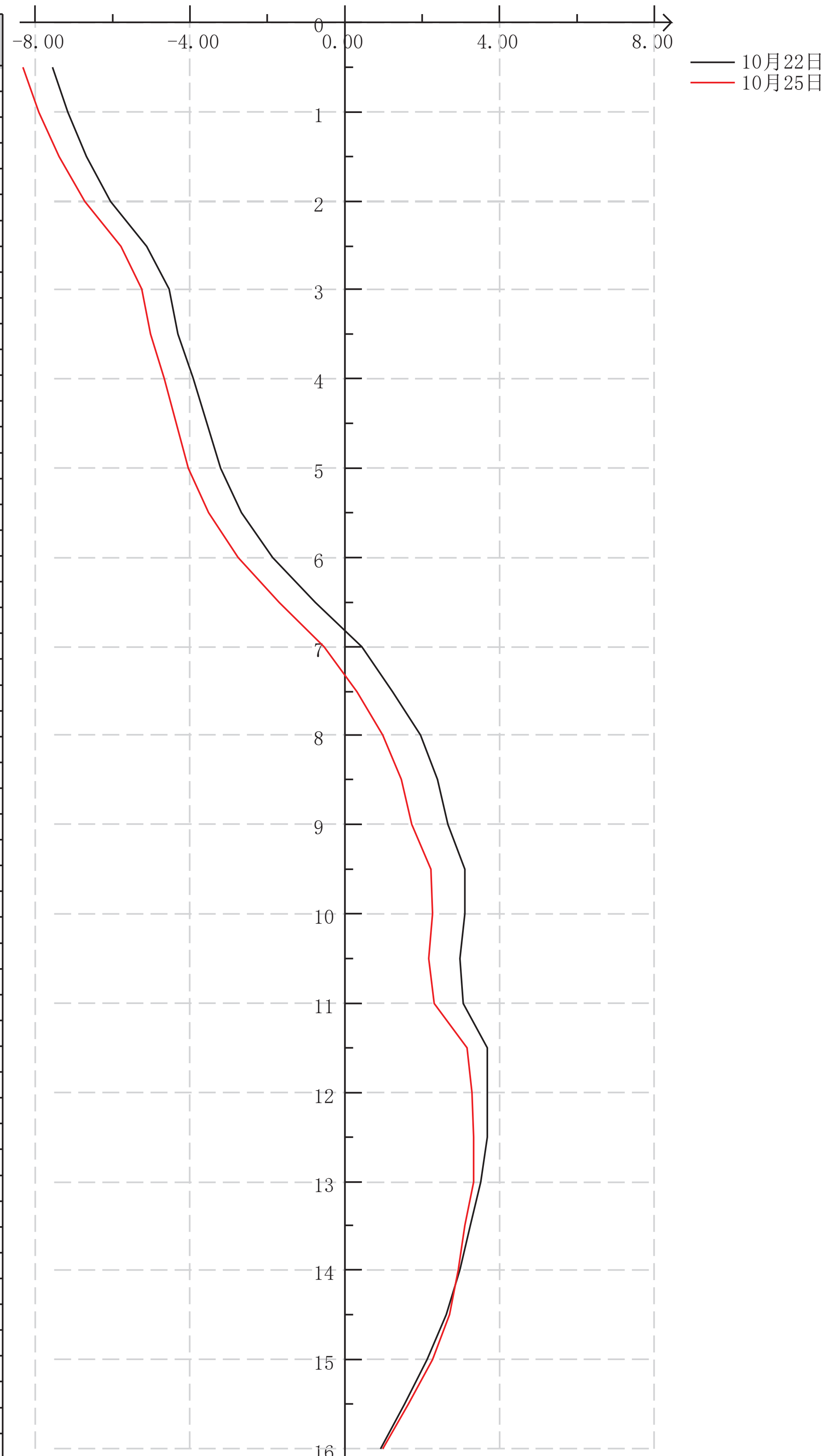
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1002　　观测孔深:16.0m

观测方向:N-S　　测孔位置:TT-2　　当前初值:19年02月09日09时52分32秒

观测日期:当次:10月25日　　前次:10月22日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	-8.4	-0.8	-0.4
1.0	-8.0	-0.8	-0.4
1.5	-7.4	-0.7	-0.4
2.0	-6.8	-0.7	-0.3
2.5	-5.8	-0.7	-0.3
3.0	-5.3	-0.7	-0.3
3.5	-5.0	-0.7	-0.4
4.0	-4.7	-0.8	-0.4
4.5	-4.4	-0.8	-0.4
5.0	-4.1	-0.8	-0.4
5.5	-3.6	-0.8	-0.4
6.0	-2.8	-0.9	-0.4
6.5	-1.7	-0.9	-0.5
7.0	-0.5	-0.9	-0.5
7.5	0.3	-0.9	-0.5
8.0	1.0	-1.0	-0.5
8.5	1.4	-0.9	-0.5
9.0	1.7	-0.9	-0.5
9.5	2.2	-0.9	-0.4
10.0	2.3	-0.8	-0.4
10.5	2.2	-0.8	-0.4
11.0	2.3	-0.7	-0.4
11.5	3.1	-0.5	-0.3
12.0	3.3	-0.4	-0.2
12.5	3.3	-0.4	-0.2
13.0	3.3	-0.2	-0.1
13.5	3.1	-0.1	-0.1
14.0	2.9	0.0	0.0
14.5	2.7	0.1	0.0
15.0	2.3	0.1	0.1
15.5	1.6	0.1	0.1
16.0	1.0	0.1	0.0



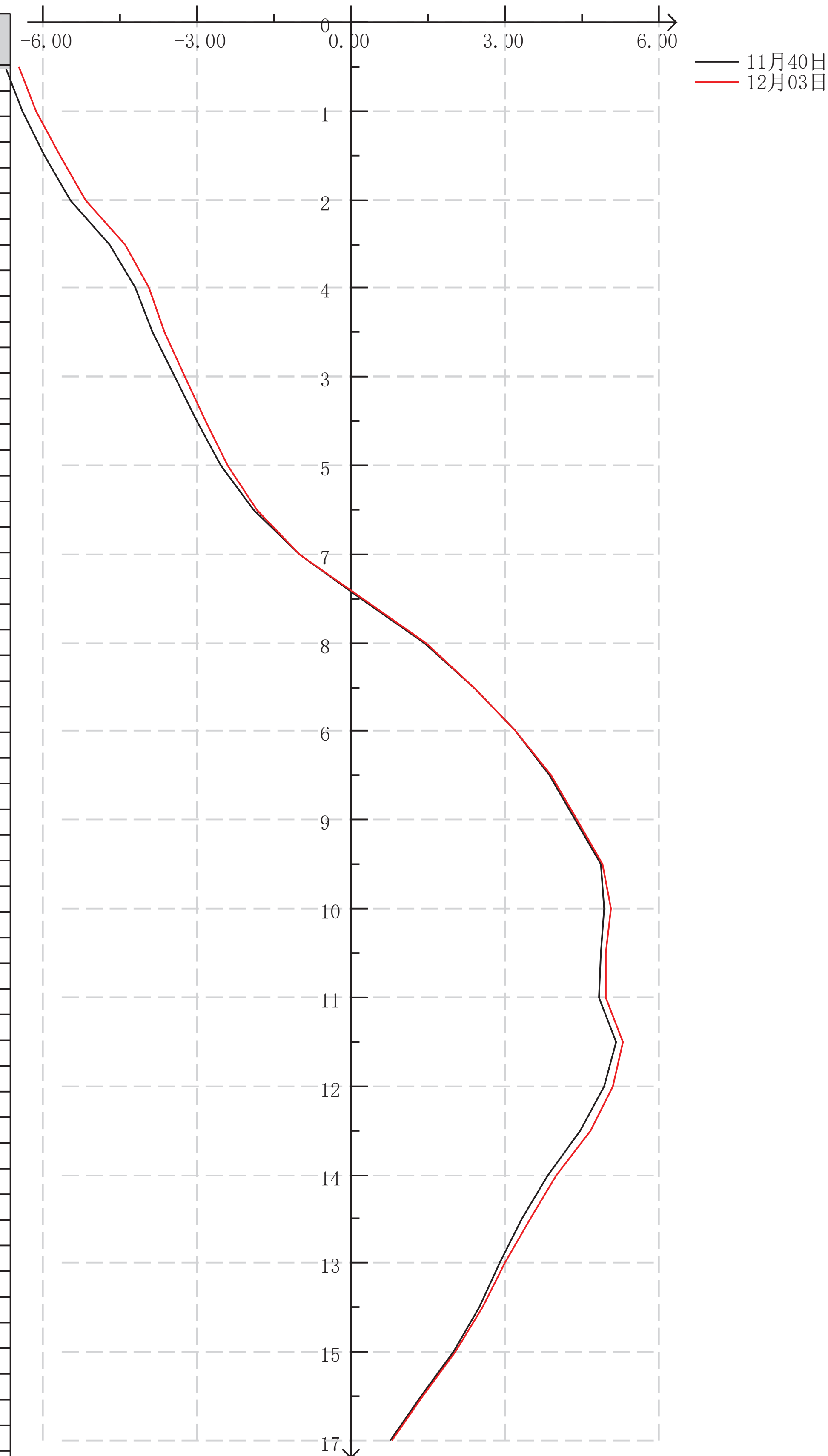
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1002　　观测孔深:16.0m

观测方向:N-S　　测孔位置:TT-2　　当前初值:19年02月09日09时52分32秒

观测日期:当次:12月04日　　前次:11月30日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	-6.8	0.3	0.1
1.0	-6.2	0.3	0.1
1.5	-8.7	0.3	0.1
2.0	-7.9	0.3	0.1
2.5	-5.9	0.3	0.1
4.0	-5.4	0.3	0.1
4.5	-3.9	0.4	0.1
3.0	-3.4	0.4	0.1
3.5	-4.6	0.2	0.1
5.0	-4.2	0.2	0.1
5.5	-2.5	0.1	0.0
7.0	-1.4	0.0	0.0
7.5	0.4	0.0	0.0
8.0	1.9	0.0	0.0
8.5	4.2	0.0	0.0
6.0	3.4	0.0	0.0
6.5	5.2	0.0	0.0
9.0	5.6	0.0	0.0
9.5	7.5	0.1	0.0
10.0	7.8	0.1	0.0
10.5	7.7	0.2	0.1
11.0	7.7	0.2	0.1
11.5	8.1	0.2	0.1
12.0	7.6	0.2	0.1
12.5	7.2	0.4	0.1
14.0	5.4	0.2	0.1
14.5	3.7	0.2	0.1
13.0	3.0	0.1	0.0
13.5	4.3	0.1	0.0
15.0	2.8	0.0	0.0
15.5	1.9	0.1	0.0
17.0	1.1	0.0	0.0



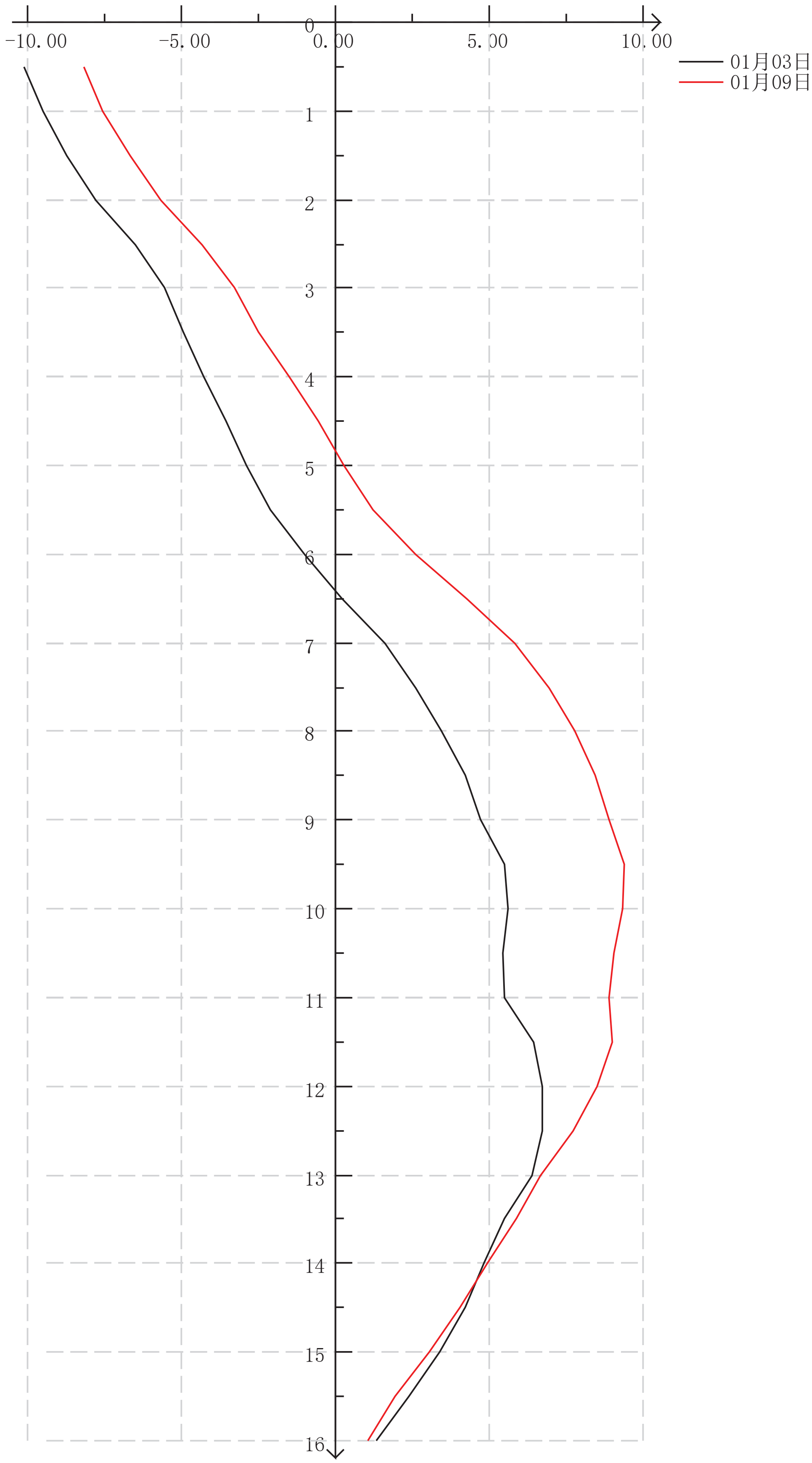
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1002　　观测孔深:16.0mm

观测方向:N-S　　测孔位置:TT-2　　当前初值:19年02月09日09时52分32秒

观测日期:当次:01月09日　　前次:01月03日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	-8.2	1.9	0.4
1.0	-7.6	2.0	0.4
1.5	-6.7	2.1	0.4
2.0	-5.7	2.1	0.4
2.5	-4.4	2.2	0.4
3.0	-3.3	2.3	0.5
3.5	-2.5	2.5	0.5
4.0	-1.5	2.8	0.6
4.5	-0.6	3.0	0.6
5.0	0.3	3.2	0.6
5.5	1.2	3.4	0.7
6.0	2.6	3.6	0.7
6.5	4.2	4.0	0.8
7.0	5.8	4.3	0.9
7.5	6.9	4.3	0.9
8.0	7.8	4.4	0.9
8.5	8.4	4.3	0.9
9.0	8.9	4.2	0.8
9.5	9.4	3.9	0.8
10.0	9.3	3.7	0.7
10.5	9.1	3.6	0.7
11.0	8.9	3.4	0.7
11.5	9.0	2.6	0.5
12.0	8.5	1.8	0.4
12.5	7.7	1.0	0.2
13.0	6.6	0.3	0.1
13.5	5.9	0.4	0.1
14.0	4.9	0.1	0.0
14.5	4.0	-0.2	0.0
15.0	3.0	-0.4	-0.1
15.5	1.9	-0.4	-0.1
16.0	1.1	-0.3	-0.1



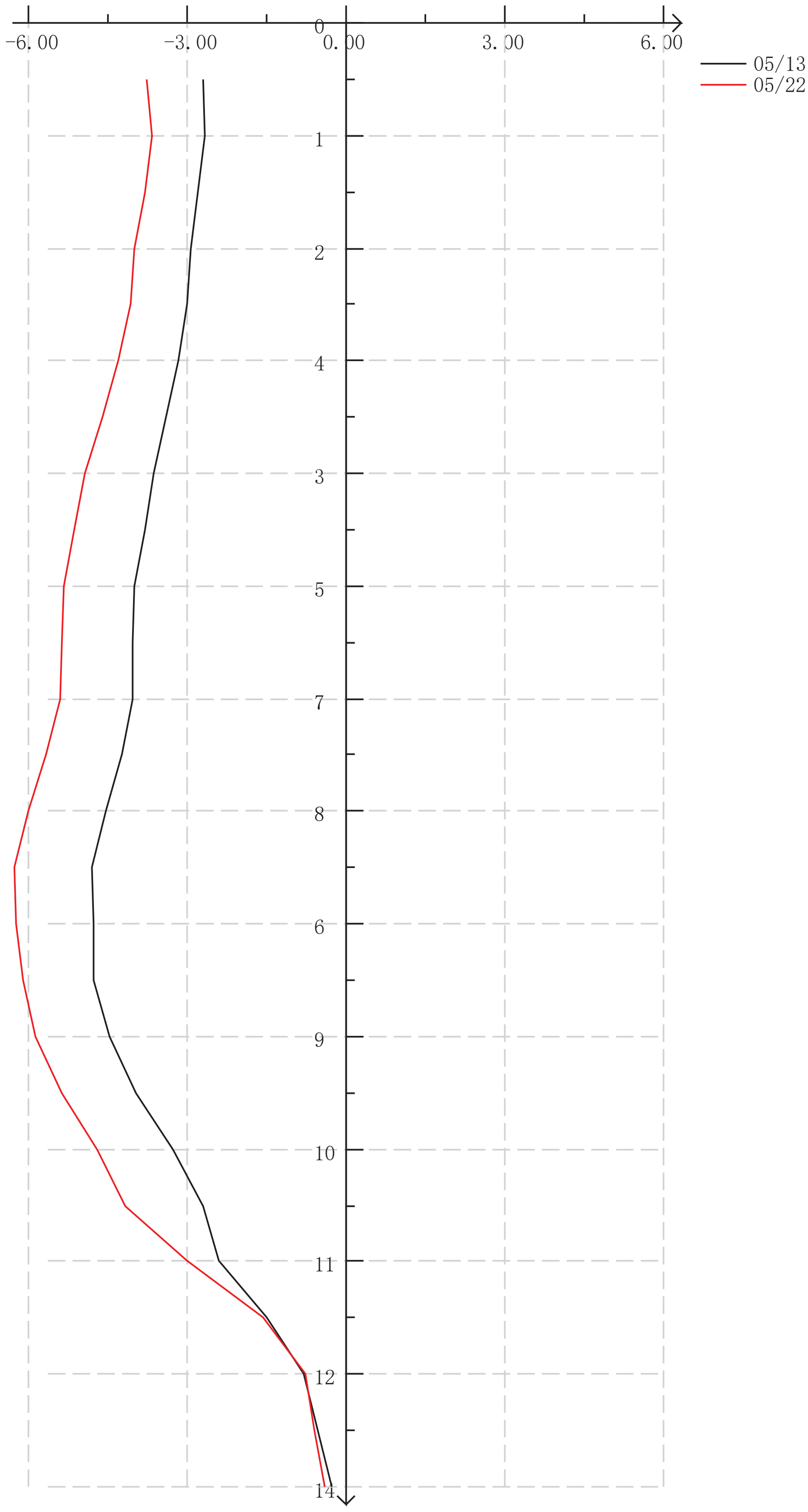
Analyze Report

Name : Deep soil lateral displacement　　No : #1003　　Hole Depth : 13.0m

Direction : E-W　　Point Name : TT-3

Date : This Time : 05/22　　Last Time : 05/14

Depth /(m)	Total (mm) Change	This (mm) Change	Daily (mm/d) Variation
0.5	-5.1	-1.3	-0.2
1.0	-3.9	-1.3	-0.2
1.5	-5.1	-1.4	-0.2
2.0	-5.4	-1.3	-0.2
2.5	-5.5	-1.3	-0.2
4.0	-5.6	-1.5	-0.2
4.5	-7.2	-1.7	-0.2
3.0	-7.7	-1.8	-0.2
3.5	-7.9	-1.6	-0.2
5.0	-8.1	-1.6	-0.2
5.5	-8.2	-1.6	-0.2
7.0	-8.2	-1.6	-0.2
7.5	-8.7	-1.9	-0.2
8.0	-6.0	-1.9	-0.2
8.5	-6.3	-1.9	-0.2
6.0	-6.3	-2.0	-0.2
6.5	-6.2	-1.6	-0.2
9.0	-8.9	-1.9	-0.2
9.5	-8.2	-1.9	-0.2
10.0	-7.4	-1.9	-0.2
10.5	-5.7	-2.0	-0.2
11.0	-3.0	-0.6	-0.1
11.5	-2.1	-0.1	0.0
12.0	-1.0	0.0	0.0
12.5	-0.6	-0.1	0.0
14.0	-0.5	-0.2	0.0



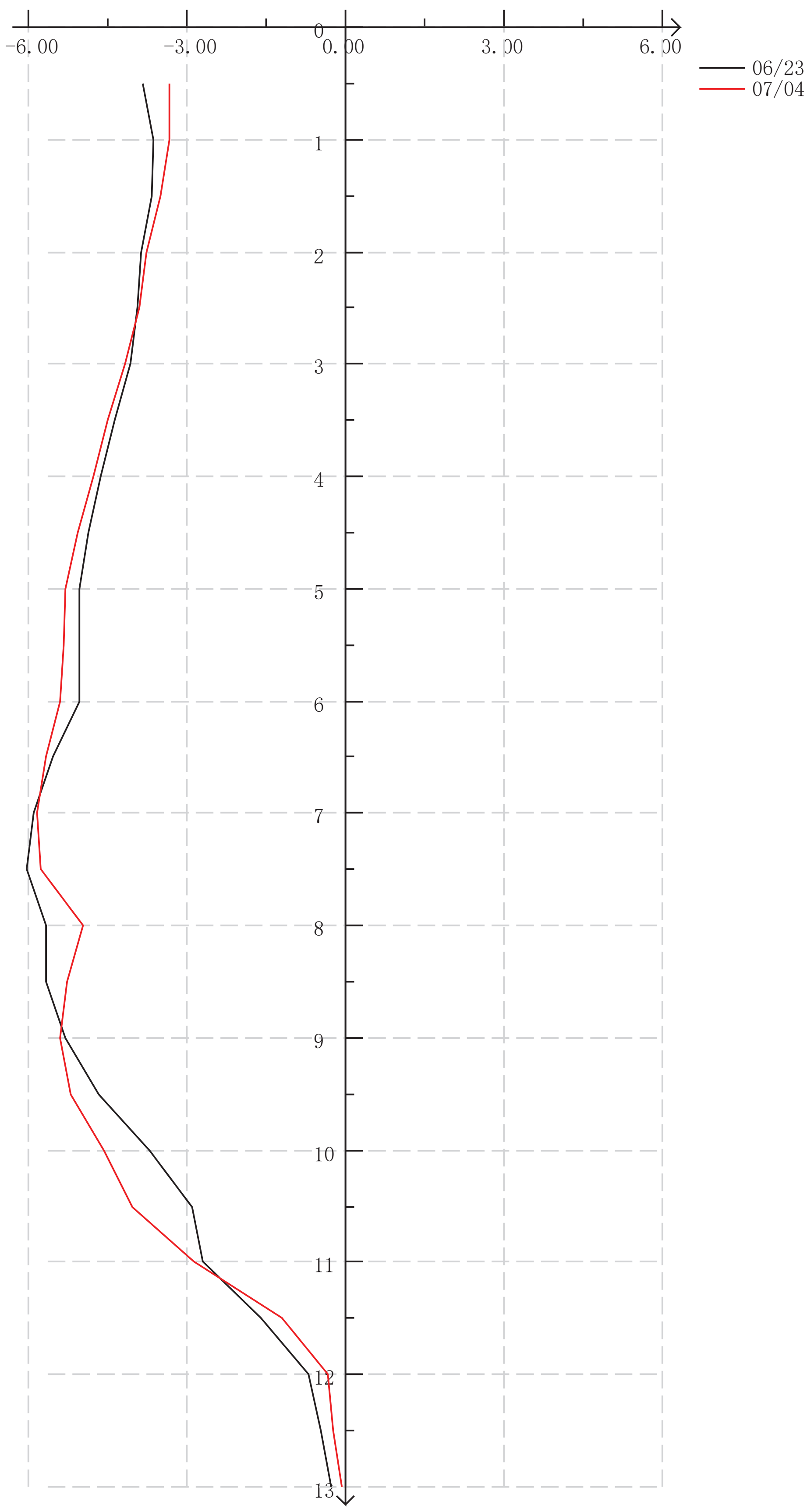
Analyze Report

Name：Deep soil lateral displacement　　No：#1004　　Hole Depth：1460m值：

Direction：. W　　Point Name：EEW4

Date：ETis Eime：0h70/　　Last Eime：02734

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	-3.4	0.5	0.0
1.0	-3.4	0.3	0.0
1.5	-3.5	0.2	0.0
2.0	-3.8	0.1	0.0
2.5	-3.9	0.0	0.0
3.0	-4.2	-0.1	0.0
3.5	-4.5	-0.1	0.0
4.0	-4.8	-0.1	0.0
4.5	-5.1	-0.2	0.0
5.0	-5.3	-0.2	0.0
5.5	-5.4	-0.3	0.0
6.0	-5.4	-0.4	0.0
6.5	-5.7	-0.1	0.0
7.0	-5.9	0.1	0.0
7.5	-5.8	0.3	0.0
8.0	-5.0	0.7	0.1
8.5	-5.3	0.4	0.0
9.0	-5.4	-0.1	0.0
9.5	-5.2	-0.5	0.0
10.0	-4.6	-0.8	-0.1
10.5	-4.1	-1.1	-0.1
11.0	-2.9	-0.2	0.0
11.5	-1.2	0.4	0.0
12.0	-0.3	0.4	0.0
12.5	-0.2	0.2	0.0
13.0	-0.1	0.2	0.0



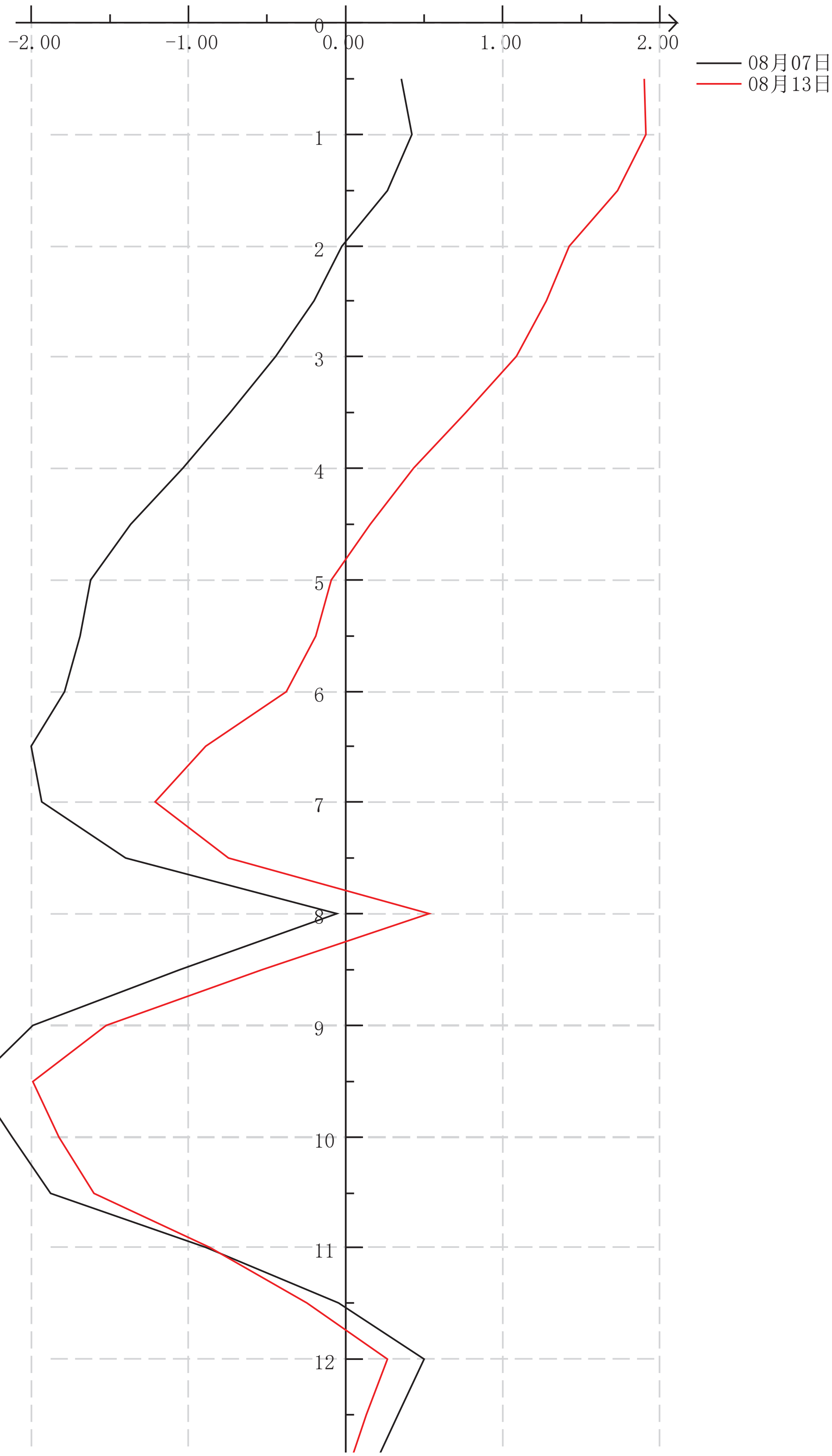
图表分析报告

观测工程名：Deep soil lateral displacement　　测孔编号：#1003　　观测孔深：13.0 m

观测方向：E-W　　测孔位置：TT-3　　当前初值：19年02月18日09时08分38秒

观测日期：当次：08月13日　　前次：08月07日

深度/(m)	累计位移(mm)	本次位移(mm)	变化速率(mm/d)
0.5	1.9	1.5	0.3
1.0	1.9	1.5	0.3
1.5	1.7	1.5	0.3
2.0	1.4	1.5	0.3
2.5	1.3	1.5	0.3
3.0	1.1	1.5	0.3
3.5	0.8	1.5	0.3
4.0	0.4	1.5	0.3
4.5	0.2	1.5	0.3
5.0	-0.1	1.5	0.3
5.5	-0.2	1.5	0.3
6.0	-0.4	1.4	0.3
6.5	-0.9	1.1	0.2
7.0	-1.2	0.7	0.1
7.5	-0.8	0.7	0.1
8.0	0.5	0.6	0.1
8.5	-0.5	0.5	0.1
9.0	-1.5	0.5	0.1
9.5	-2.0	0.4	0.1
10.0	-1.8	0.3	0.1
10.5	-1.6	0.3	0.1
11.0	-0.9	0.0	0.0
11.5	-0.3	-0.2	0.0
12.0	0.3	-0.2	0.0
12.5	0.1	-0.2	0.0
13.0	0.0	-0.2	0.0



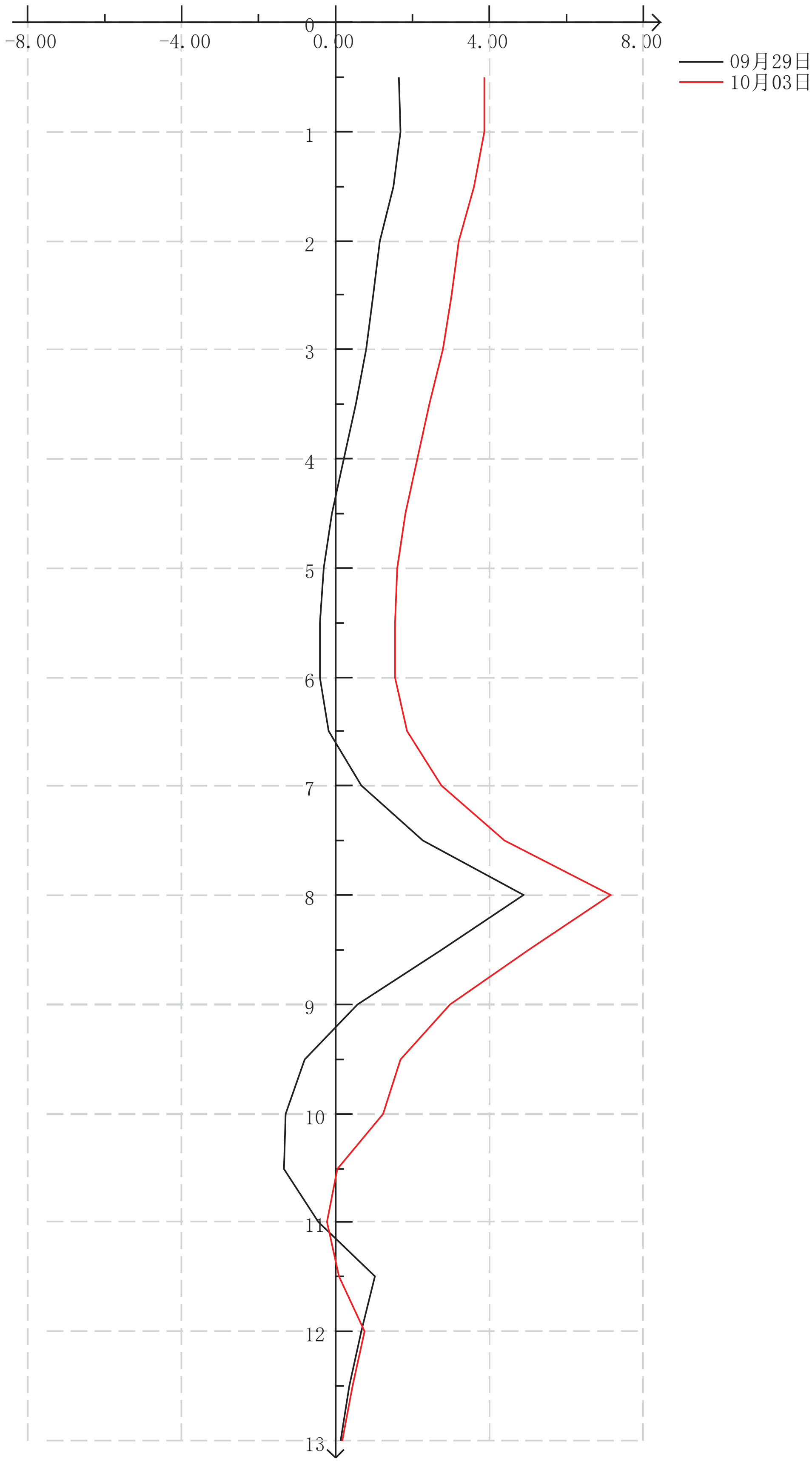
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1003　　观测孔深:13.0m

观测方向:E-W　　测孔位置:TT-3　　当前初值:19年02月18日09时08分38秒

观测日期:当次:10月03日　　前次:09月29日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	3.8	2.2	0.7
1.0	3.8	2.2	0.7
1.5	3.6	2.1	0.7
2.0	3.2	2.1	0.7
2.5	3.0	2.0	0.7
3.0	2.8	2.0	0.7
3.5	2.4	1.9	0.6
4.0	2.1	1.9	0.6
4.5	1.8	1.9	0.6
5.0	1.6	1.9	0.6
5.5	1.5	2.0	0.7
6.0	1.6	2.0	0.7
6.5	1.9	2.1	0.7
7.0	2.8	2.1	0.7
7.5	4.4	2.1	0.7
8.0	7.1	2.3	0.8
8.5	5.0	2.3	0.8
9.0	3.0	2.4	0.8
9.5	1.7	2.5	0.8
10.0	1.2	2.5	0.8
10.5	0.0	1.4	0.5
11.0	-0.2	0.2	0.1
11.5	0.1	-0.9	-0.3
12.0	0.7	0.1	0.0
12.5	0.4	0.1	0.0
13.0	0.2	0.1	0.0



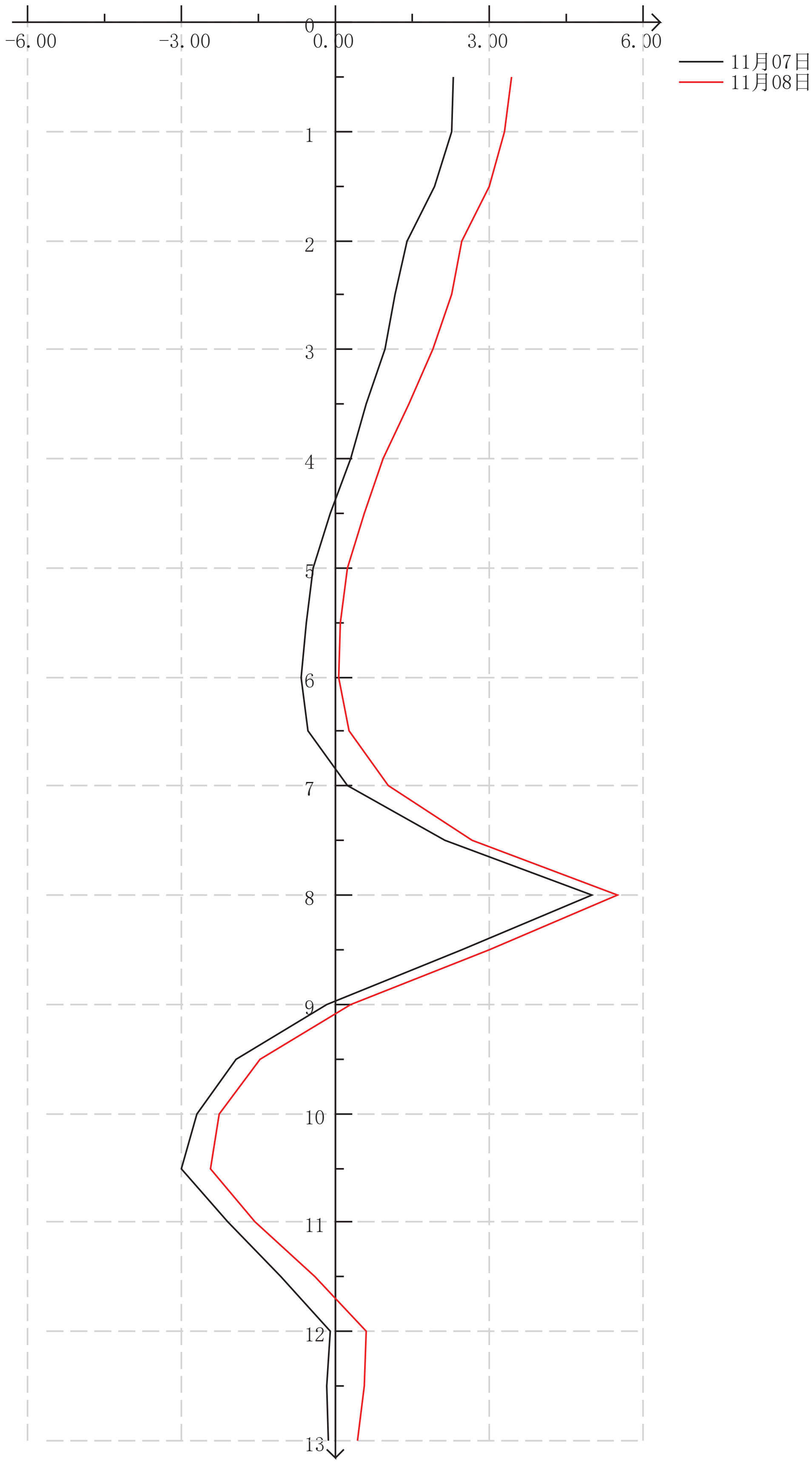
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1003　　观测孔深:13.0m

观测方向:E-W　　测孔位置:TT-3　　当前初值:19年02月18日09时08分38秒

观测日期:当次:11月08日　　前次:11月07日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	3.4	1.1	1.1
1.0	3.3	1.1	1.1
1.5	3.0	1.1	1.1
2.0	2.5	1.1	1.1
2.5	2.2	1.1	1.1
3.0	1.9	0.9	0.9
3.5	1.4	0.8	0.8
4.0	0.9	0.7	0.7
4.5	0.5	0.6	0.6
5.0	0.2	0.7	0.7
5.5	0.1	0.7	0.7
6.0	0.0	0.7	0.7
6.5	0.2	0.8	0.8
7.0	1.0	0.8	0.8
7.5	2.7	0.6	0.6
8.0	5.5	0.5	0.5
8.5	3.0	0.5	0.5
9.0	0.3	0.5	0.5
9.5	-1.5	0.5	0.5
10.0	-2.3	0.5	0.5
10.5	-2.4	0.6	0.6
11.0	-1.6	0.6	0.6
11.5	-0.4	0.7	0.7
12.0	0.6	0.7	0.7
12.5	0.6	0.7	0.7
13.0	0.4	0.6	0.6



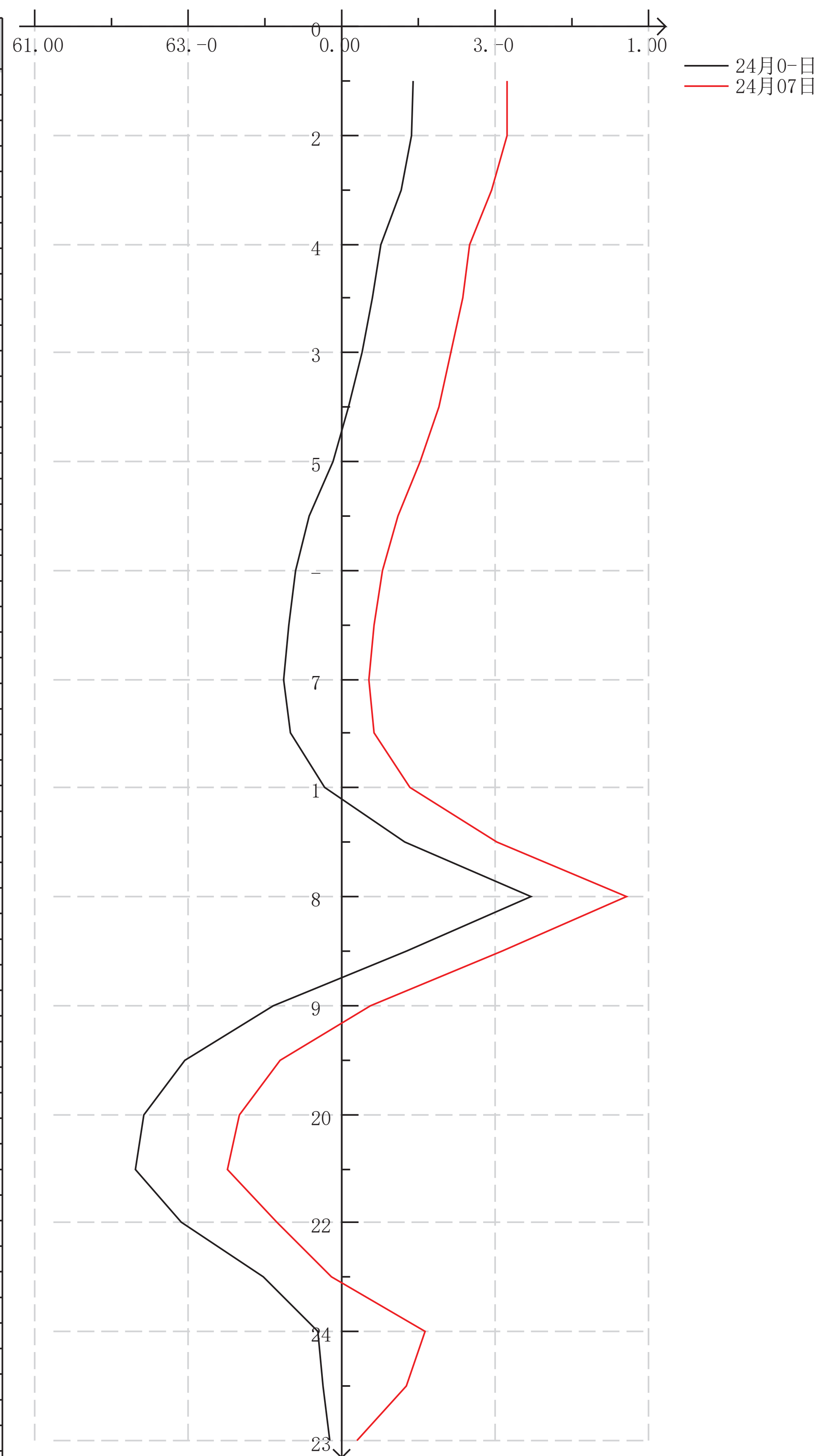
Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1003　　观测孔深:13.0m

观测方向:E-W　　测孔位置:TT-3　　当前初值:19年02月18日09时08分38秒

观测日期:当次:12月06日　　前次:12月05日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.–	3.8	4.2	4.2
2.0	3.1	4.4	4.4
2.–	3.5	4.2	4.2
4.0	4.9	4.0	4.0
4.–	4.1	4.0	4.0
3.0	4.–	4.0	4.0
3.–	4.4	4.0	4.0
5.0	2.8	4.0	4.0
5.–	2.3	4.0	4.0
–.0	0.9	4.0	4.0
–.–	0.1	4.0	4.0
7.0	0.7	4.0	4.0
7.–	0.1	2.9	2.9
1.0	2.–	2.9	2.9
1.–	3.–	4.2	4.2
8.0	7.–	4.4	4.4
8.–	3.1	4.4	4.4
9.0	0.7	4.4	4.4
9.–	62.5	4.4	4.4
20.0	64.5	4.4	4.4
20.–	64.7	4.2	4.2
22.0	62.–	4.4	4.4
22.–	60.4	2.7	2.7
24.0	2.9	4.–	4.–
24.–	2.–	2.9	2.9
23.0	0.3	0.7	0.7



Analyze Report

观测工程名:Deep soil lateral displacement　　测孔编号:#1003　　观测孔深:13.0 m

观测方向:E-W　　测孔位置:TT-3　　当前初值:19年02月18日09时08分38秒

观测日期:当次:12月26日　　前次:12月25日

Depth /(m)	Total(mm) Change	This(mm) Change	Daily(mm/d) Variation
0.5	4.1	-1.4	-1.4
1.0	4.0	-1.4	-1.4
1.5	2.8	-1.4	-1.4
2.0	2.2	-1.4	-1.4
2.5	2.0	-1.4	-1.4
4.0	1.8	-1.4	-1.4
4.5	1.2	-1.3	-1.3
3.0	0.8	-1.5	-1.5
3.5	0.2	-1.5	-1.5
5.0	-0.2	-1.5	-1.5
5.5	-0.4	-1.5	-1.5
7.0	-0.3	-1.5	-1.5
7.5	-0.3	-1.7	-1.7
8.0	0.3	-1.7	-1.7
8.5	2.0	-2.0	-2.0
6.0	5.0	-2.2	-2.2
6.5	1.8	-2.2	-2.2
9.0	-1.8	-2.0	-2.0
9.5	-4.9	-1.9	-1.9
10.0	-3.8	-1.9	-1.9
10.5	-3.7	-1.8	-1.8
11.0	-4.2	-1.7	-1.7
11.5	-1.0	-1.3	-1.3
12.0	-0.7	-2.3	-2.3
12.5	-0.5	-2.5	-2.5
14.0	-0.3	-1.4	-1.4