Surface Settlement caused by Pipe Jacking Works crossing KL-Seremban Highway

by

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Dissertation submitted in partial fulfilment of
the requirements for the
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CERTIFICATION OF APPROVAL

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A project dissertation submitted to the
Civil Engineering Programme
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_____

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TRONOH, PERAK

September 2022

CERTIFICATION OF ORIGINALITY

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

NUR DIANA NATASYA BINTI RAZALI

ABSTRACT

The rapid growth of rehabilitation systems has resulted in an increasing demand for tunnelling technology that can operate in extreme conditions which involves deep excavations and interruptions of utility lines and traffic flow. The chosen project comprising the installation of DN2500 mm sewerage reinforced jacking pipe (RCJP) using pipe jacking method. The proposed project site in Kuala Lumpur is located at KM D7.4 along KL-Seremban Highway. This paper aims to correlate the TBM parameter with ground condition due to huge excavation induced by tunnelling activity and to produce Gaussian Distribution Curve by applying empirical method and determine the volume loss of tunnelling. Any proposed projects might affect its surrounding environment due ground excavation which leads to surface settlement. GSMs were used to monitor the environmental impact of tunnelling on the surface road along tunnel alignment thus providing a solution based on the form of empirical assessment for settlement measurement and obtaining the best fitted line according to Gaussian Distribution Curve. During the tunnelling stage, detailed surface settlement monitoring was carried out using levelling survey techniques to comprehend the ground condition nearby public access, such as highways and utilities. Results indicate that the ground settlement may persist in granular soil long after the passage of a TBM. The validated trough width parameter, K was 0.5 and the tunnel volume loss observed about CH363 and CH395 was 5.47% and 15.23%, respectively. The volume loss (V_L) can be considerable as compared to the volume loss reported by previous researchers. By assembling data on drives in various ground conditions, there is the possibility for improved knowledge of the pipe-jacking process.

Keywords: Pipe jacking, surface settlement, ground settlement marker, empirical method

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By the grace of Almighty and Merciful Allah upon all His blessings.

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No amount of adversity can break us if we have gratitude, positivity, resolve and community.

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LIST OF ABBREVIATIONS

BH	Bore Hole
DN	Nominal Pipe Diameter
GSM	Ground Settlement Marker
HDPE	High Density Polyethylene
IB	Intervention Block
ID/OD	Inner Diameter/Outer Diameter
IJS	Intermediate Jacking Station
PVC	Polyvinyl Chloride
RCJP	Reinforced Concrete Jacking Pipe
RL	Reduced Level
SI	Soil Investigation
TBM	Tunnel Boring Machine
VCP	Vitrified Clay Pipe

CHAPTER 1

INTRODUCTION

1.1 Background of Study

Trenchless technology can be defined as a group of installation and rehabilitation techniques for underground utilities that involve minimal surface excavation and offer significant new options to the old open-cut utility pipe installation procedures (Gupta et al., 2021). The main countries where contemporary microtunneling machine excavation was developed were Japan, Germany and UK. Patents from the UK and Germany from the early 19th century helped to advance the concept of a slurry pressure balance shield. Although the technique did not become widely employed in the USA until the 1950s, pipe jacking has been reported as experiencing as early as 1896 to install a concrete culvert beneath the Northern Pacific Railroad (Sterling, 2020). Pipe jacking can be applied to install pipes underground that are deeper than 20 m below the earth's surface. The traditional open-cut methods are no longer appropriate due to disturbance to traffic, safety concerns, environmental effects, high costs and other factors, while trenchless construction techniques can solve these concerns. In the past, when damage to the sewer was severe, Malaysia's local authority, Indah Water Konsortium (IWK), used the open-cut trench method to lay a new sewer line next to the damaged one and the old line was bypassed as reported by Gupta et al., (2021). This appalling process, which severely disrupts traffic and people's daily lives and is extremely expensive, involves digging up pipelines beneath main streets, commercial areas and historic structures. This technique has been implemented in Malaysia for mostly all the applicable trenchless construction.

Prior to the tunnelling construction, an extensive soil investigation had been carried out to comprehend the soil properties whereby the detailed calculation considers the impacts of arching due to soil cohesion. According to Davis (2008), particularly in many densely and populated urban areas, trenchless technology has proved that the tunnelling construction tends to prevent pavement failure but sometimes failure might occur and can result in damage to adjacent structures.

FIGURE 1.1 Pipe jacking with mixed ground cutting disc

Subsidence is frequently addressed by tunnelling with a TBM, which can impart face pressure to stabilise the ground (Lee et al., 2021). The surface settlement occurs when the annular area between the pipe surface and the underlying soil collapses. Geologists found that fractures and cavities in the Kuala Lumpur limestone bedrock are frequent characteristics associated with preconstruction in this environment (Ismail et al., 2011). Therefore, the associated ground treatment works subjected to be performed in specific location and not limited to soft ground area (sand, silty and etc.) or highwater table with high permeability. The ground treatment shall be done prior the commencement of pipe-jacking works and soil consolidation are necessary for shaft location at the break in and break out zone.

Any undue settlement or irregularities observed during construction can be detected promptly and appropriate remedial measures, such as use of a different annulus filler to encounter the actual ground conditions and change of the advance speed of TBM can be considered to prevent ground surface failure from happening (Haryono et al., 2021). Niu et al. (2020) defined the technology provides the advantages of rapid construction speed, high economic value and low impact on the urban environment otherwise the mechanical soil contact will cause settling of the upper part of pipe jacking and surrounding soil.

1.2 Problem Statement

Any proposed projects might affect its surrounding environment due to ground excavation which leads to pavement distress and variety of points such as settlement and lowering the groundwater table. As TBM required a capacity to manage huge-excavated soil production to install lining pipe, a road settlement was observed at which a sudden sinking of road surfaces section. The tunnel alignment was crossed the heavy traffic road thus site investigation and crosshole seismic for pipe jacking alignment has been conducted earlier.

In fact, the stoppage of TBM below highway attributes to the risk of TBM and pipeline misalignment, seize, blockage and other conditions that may lead to surface settlement, engulfment, frac-out, heave, pipe or TBM dislocation and other unforeseen cases. The soil contact had caused settling of the top section of the pipe jacking and surrounding soil thus overcoming the soil settlement issue is critical during the tunnelling advancement. Cases like heaving may happen when excessive force is applied to the excavation face, while subsidence occurs when over-excavation is anticipated. For a better pavement quality, empirical method approach on surface settlement during and after the construction has become vital. However, the main causes of subsidence, such as unanticipated geological circumstances and TBM operational concerns, among which cutterhead pressure and thrust pressure parameters are the most important to be accounted by analytical or empirical methods.

1.3 Objectives

The objectives below have been presented in order to offer a clear overview throughout this project by solving the problem statement.

1. To correlate the TBM parameters and ground condition due to huge excavation induced by tunnelling activity crossing the KL-Seremban Highway.
2. To produce Gaussian Distribution Curve by applying Empirical Method and determine the volume loss (V_L) of tunnelling by using information from instrumentation data.

1.4 Scope of Study

This study focused on the environmental impact due to underground construction of treated water pipelines. Data collected which is based on project materials such as Soil Investigation Report, Crosshole Seismic, Jacking Log, Pipe Allocation Plan and etc. During the Tunnelling Advancement Phase where a total of 96 concrete cladded mild steel pipes installed to form a 337 m long. The field monitoring had been conducted at KM D7.4 along KL-Seremban Highway, both inbound and outbound. The trenchless excavation technique employs four hydraulic jacks to push concrete pipes behind a pipe jacking machine from a 28 m depth launching shaft to a receiving shaft. This method has utilised the slurry shields in which capable to provide continuous support to the face by slurry pressure after considering the site and geological condition, tunnel diameter and distance of drive (F-F Length: 336.75 m) with a gradient of +0.2019% to the receiving shaft.

This project has operated crossing the sensitive features in the dense traffic flow, therefore surface settlement along the pipe jacking axis is monitored regularly. A total station is used to precisely monitor these reference points daily until the construction is completed. A condition survey of the existing road surface within 15 m from the pipe jacking alignment was conducted prior to the tunnelling construction to observe the settlement measurement at each chainage. In order to study the transverse and longitudinal settlement on both sides of the chainage in the pipe jacking process, MN4 was set to be the axis of the pipe lining.

Concurrently, it is important to control the amount of materials excavated from the tunnel opening to prevent significant ground loss. This could be achieved by comparing the excavated volume with the theoretical excavation volume to check if significant voids have been formed in the soil above the tunnel and checking the amount of grout used to fill the voids around the pipes. A better method should be devised for the similar operation to measure the amount of materials excavated for each section of the tunnel excavation.

CHAPTER 2

LITERATURE REVIEW

Throughout this chapter, explanation on the difference between the selection of pipe installation methods are emphasized. A comprehensive literature review was conducted and the need for understanding of the behaviour of surface settlement under different influencing parameters using previous namely project and model testing were established.

2.1 Efficient Trenchless Technology

Research conducted in many areas around the world revealed that the expenses incurred as a result of the inconveniences caused by open-cut trench excavations in urban areas considerably exceeded the costs of building construction (Khan & Abdullah, 2016). The rapid growth of rehabilitation systems has resulted in an increase in demand for tunnelling technology that can operate in extreme conditions, which involves deep excavations and interruptions to utility lines and traffic flow. The installation of sewers, water supply systems, gas pipelines and power and telecommunication networks have in fact, turned the underground into untouchable networks of pipes (Lee et al., 2021). According to statistics, Japan proposed to construct a complete pipe gallery of approximately 1100 km by the turn of the century (Liu et al., 2018).

The feasibility of open-cut method as wide consideration needs to be taken to avoid improvidence for the project cost (Patel et al., 2014). By definition, open trench excavation involves several ancillary procedures such as road closures, on-site storage of excavated materials, backfilling and compaction, ground water management and surface restoration after pipe installation. Hence, trenchless technology may be more preferable method in not only reduce the cost but also less disturbance on the surrounding environments.

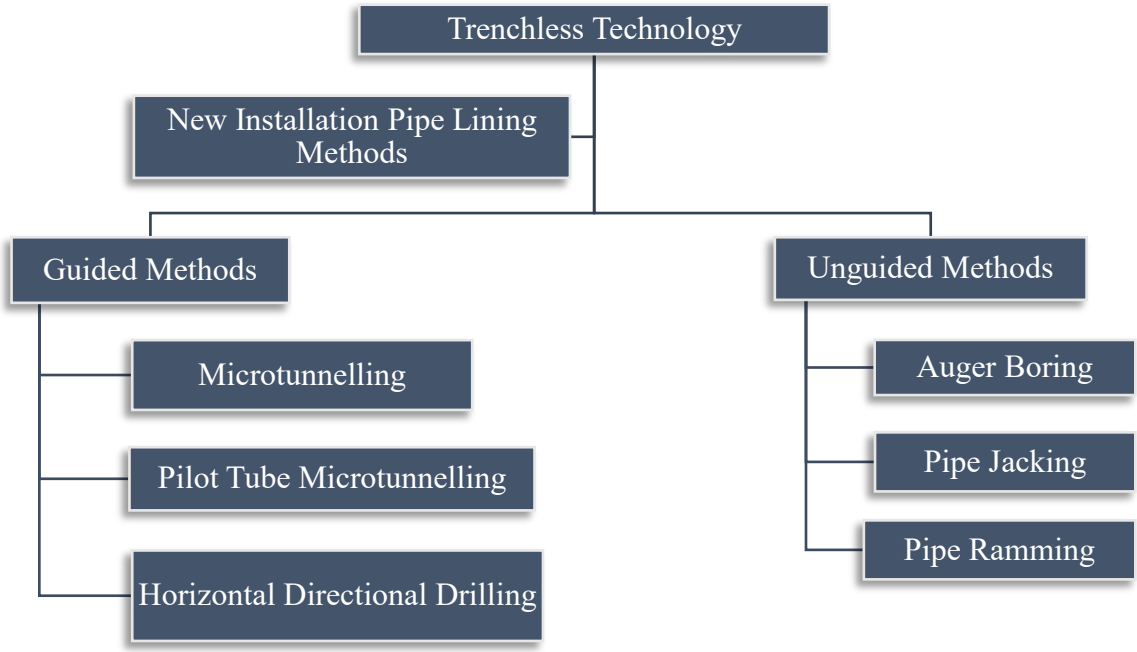

FIGURE 2.1 Methods employed under trenchless construction (Kumar & Patel, 2019).

In any proposed projects, determining the appropriate excavation method for span urban tunnel is a vital part for successful project completion (Jorjam & Hammad, 2021). The installation techniques are widely classified into two types: guided methods and unguided methods. Guided techniques employ modern technology to guide and track pipe installations, resulting in greater precision tolerance and grade for completed pipes or conduits. Each trenchless approach is varied of its intention, application and functionality. As reported by Aiman et al., (2020), the construction sector has been able to build tunnels of increasing size in part to the development of techniques and equipment that were previously inconceivable, such as soil consolidation, admixtures

injections and shielded equipment. The proposed method in this study is based on a single line of pipe jacking and does not consider other factors or settlements caused to nearby construction and buildings.

TABLE 2.1 Applicability of Various Trenchless Construction Methods (American Society of Civil Engineers & Construction Institute), 2022; Conway & Jahren, 2008; Kumar & Patel, 2019)

Parameters	Technology					
	Guided			Unguided		
	Microtunnelling Method	**Pilot-tube Microtunnelling**	**Horizontal Directional Drilling**	**Auger Boring**	**Pipe Jacking**	**Pipe Ramming**
Pipeline Diameters and Pipe Material	25 to 3000 mm diameter (OD); Steel, RCP, VCP	25 to 125 mm diameter (OD); Steel, RCP, Vitrified Clay pipe	200 to 1800 mm diameter (OD); Steel, HDPE, PVC, Ductile	100 to 4500 mm diameter (OD); Steel	800 mm to 3000 mm diameter (OD); Steel, RCP, fiber glass pipes	100 to 4500 mm diameter (OD); Steel
Accuracy of Installation	±10 mm	±25 mm	±100 mm	±1% of the bore length	±25 mm	Dependent on setup
Typical Length	25 – 450 m	< 100 m	12 – 1800 m	12 – 150 m	150 – 305 m	12 – 90 m
Soil Suitability	Loose to very dense sand, soft to hard clay/silt, rock; high groundwater	Loose to medium dense Sand/Silty Sand to medium stiff clay/silt	Clay, cohesionless sand/silt	Medium to very dense sand/silty clayey gravel, firm to hard clay	Uniform soil free from boulders & cobbles	Medium dense sand/gravel, soft to stiff clay, with low groundwater
Soil Unsuitability	Running/flowing groundwater, very loose/loose sand, very soft and soft clay/silt, boulder, hard rock, mixed face condition	Groundwater level above the pipe and very dense sand	Soils containing more than 50% gravel/loose soils and rock sizes more than 3 inch in diameter	Rock with high groundwater	Loose sandy soils with high water inflows	Rock/sand with groundwater

2.2 Definition of Surface Settlement

The existing ground stress and hydro-geological conditions are inevitably affected by the tunnel construction. The changes of normal stress conditions are typically followed by a rapid inward displacement of the face and tunnel wall convergence. Water pressure is a significant part of pressure exerted at the tunnel face for the tunnel alignment below the water table (Chee, K. M., 2020). Referring to Mamaqani et al., (2014) and Cheng et al., (2017), overcut is required to reduce the frictional loads and injected lubricants to enable steering the pipe. Therefore, the soils around the pipe induced trailing settlement**.** Additional long-term deformations may be observed in soft cohesive soils due to pore pressure changes. However, ground movement can be reduced by applying pressure to the surrounding soils during the grouting process (Ren et al., 2018).

There are number of possible types of settlements above a tunnel that may be associated but the most related to normal settlements which should be taken as the reference for impact assessment and to which observed settlements are compared during construction. The jacking force will vary depending on the amount of overcut, lubrication, work stops, geology, and misalignment at each site (Ji et al., 2019). In addition, Bogusz et al., (2021) pointed out the settlements above a tunnel are typically more or less symmetrical across its vertical axis.

2.2.1 Tunnelling underneath Road Pavement

Pavement structure maintenance may be costly, thus it is important to choose the appropriate technology, engineering solutions and materials for soil excavation construction. Most recently, Zhen et al. (2021) reported that Zhengzhou Zhongzhou Avenue project utilised 10.12 m × 7.52 m of rectangular pipe jacking, overburden thickness of 3.0 – 4.2 m, had caused the road surface to settle whereas the settlement volume is within 3 cm. Prior to the tunnelling construction, the research recommended a complete construction strategy to mitigate the influence factors of pipe jacking construction on highway deformation. The shallow overburden rectangular pipe

jacking project employed thixotropic mud synchronous grouting and secondary grouting techniques, with typical grouting pressures ranging from 0.027 to 0.049 MPa.

2.3 Empirical Estimation of Surface Settlement

Empirical methods rely on measured values from past projects and regression analysis to link the project elements such as soil properties and project specifications to ground movement. By definition, Peck (1969)'s most widely known empirical solution is based on settlement data from over 20 case histories projects and were adapted to TBM excavations in geologically mixed-face conditions for urban projects in congested urban areas. These simplified approaches include predicting surface settlements based on a particular number of characteristics that allow for the consideration such as the excavation size and depth, ground conditions and the volume of ground loss or convergence induced by tunnelling. Figure 2.2 shown diagrammatically a schematic settlement associated with microtunneling. A transverse trough and a longitudinal trough along the tunnel center-plane described the three-dimensional trough (Athar et al., 2022).

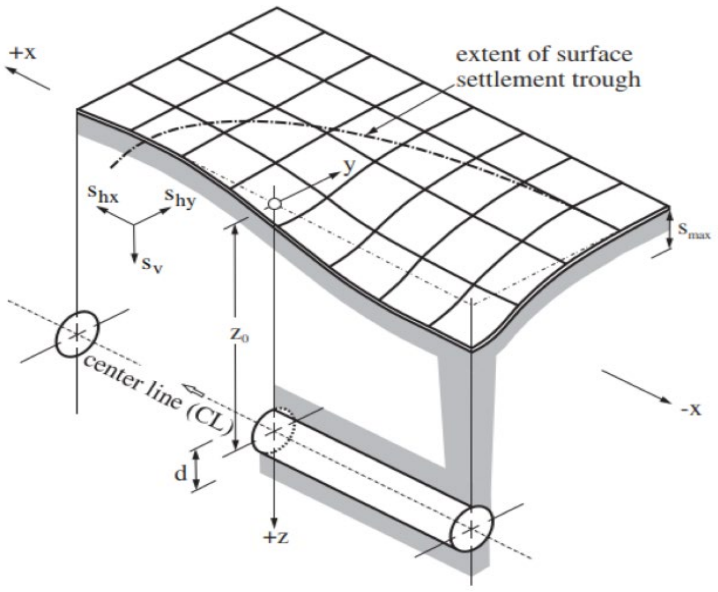

FIGURE 2.2 3D settlement trough caused by tunnel advance (Attewell et al., 1986)

2.3.1. Transverse Settlement

Transverse settlement is characterised as ground movement caused by tunnel construction, with a settlement trough developing above and ahead of the tunnel. Analysis of a considerable number of case records, Peck (1969) and the recent researchers in the studies, Athar et al., (2022) have demonstrated that the resulting of transverse settlement trough immediately after a tunnel has been constructed is well described by a Gaussian Distribution Curve.

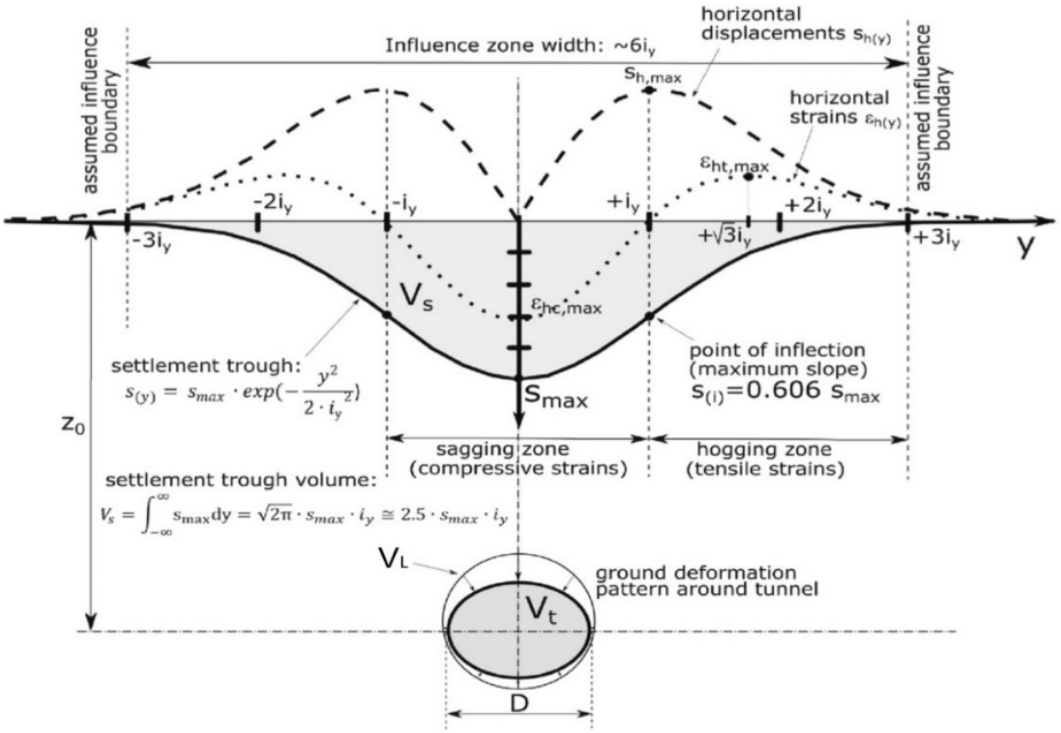

FIGURE 2.3 Transverse settlement trough approximated by Gaussian distribution curve (Bogusz et al., 2021)

In order to specify each point of the settlement profile, Peck (1969) suggested the Equation 2.1.

$$S_v(y) = S_{max} exp\left(-\frac{y^2}{2i^2}\right) \qquad 2.1$$

where y = The horizontal distance from the tunnel axis

i = The horizontal distance from the tunnel axis to the inflection point of settlement trough

According to Peck (1969), the maximum settlement above a tunnel (S_{max}) varies between tunnel cross-sections. V_L and i were parameters needed to obtain maximum surface settlement, S_{max} in Equation 2.2.

$$S_{max} = \frac{0.313 V_l D^2}{i} \tag{2.2}$$

where D = Tunnel diameter

S_{max} = Maximum settlement occurring above the tunnel axis

As identified in the past studies, Rankin (1988) assumed that i can be related to the vertical distance between the tunnel axis depth and trough width parameter, K. By definition, i in Equation 2.3 is the point that separates the settlement curve in a sagging and hogging zone. The value of K (as shown in Figure 2.4), on the other hand, may need reasonable judgement because it is significantly dependent on the ground itself, whether it is mostly granular or cohesive, and to determine whether it is below or above the water table (Wang, 2021). Therefore, a simple equation is presented to estimate optimal K values for empirical calculation. In all empirical methods subsequently, z_0; tunnel depth and R: tunnel radius.

$$i = K z_0 \tag{2.3}$$

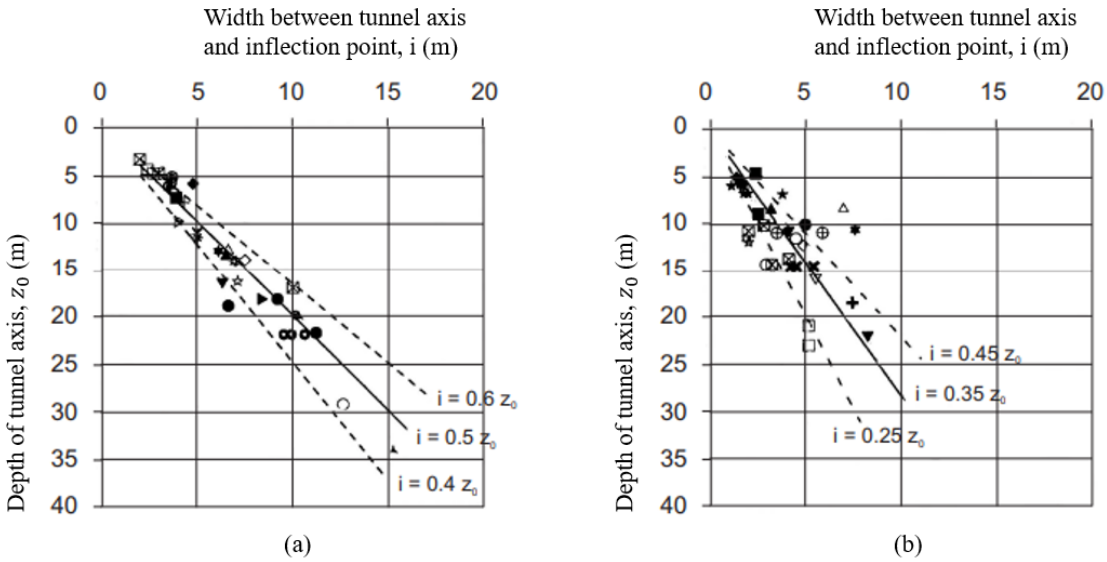

FIGURE 2.4 Variation of trough width parameter K with tunnel depth in (a) clays, (b) sands and gravels (Mair and Taylor, 1993)

2.3.2. Volume Loss

Volume loss, V_L is the ratio of the excavated volume of soil to the volume of the tunnel (measured by the tunnel's outer diameter) over the volume of the tunnel. According to Bennett (1998), a process known as ground loss when the soils surrounding the overcut are expected to collapse completely into the annular gap. However, the ground losses are depending on the ground condition and the construction method.

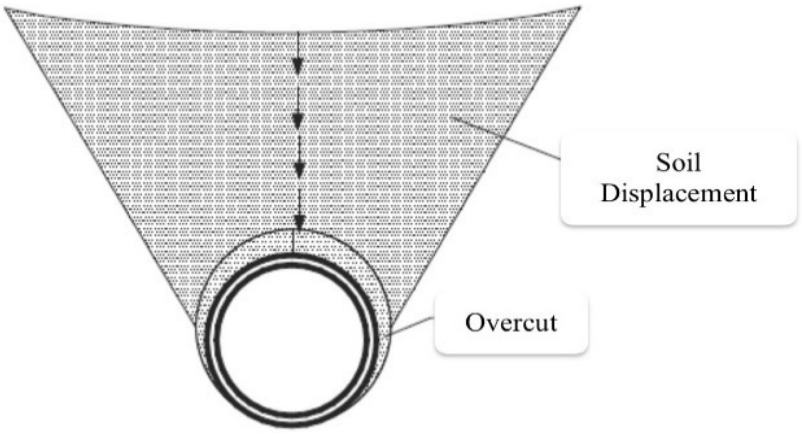

FIGURE 2.5 Surface settlement caused by volume loss (Mamaqani et al., 2014)

The volume of settlement trough (per meter length of the tunnel), V_s is expressed in Equation 2.4.

$$V_s = \sqrt{2\pi} x_i S_{max} \tag{2.4}$$

The volume of the surface settlement trough as a percentage of the tunnel's excavated area per unit length of tunnel constructed, V_L is defined as follows,

$$V_t = \frac{4}{\pi D^2} \tag{2.5}$$

$$V_L = \frac{V_s}{V_t}.100\% \tag{2.6}$$

where V_s = Settlement trough volume

V_t = Tunnel opening volume

R = Radius of the tunnel

According to various studies, the greater value of volume loss (> 1%) is unfavourable as it strongly related to the tunnelling techniques (Hui, 2020; Paul Ha Tiing Tai BEng, 2018). The conservative choice of a too high of volume loss will result in overestimation of damage to surrounding structures. As a result, unnecessary work connected with reinforcing or underpinning may require and significantly raising the entire cost of tunnel construction (Bogusz et al., 2021).

2.3.3. Longitudinal Settlement

Besides the transverse settlement profile, tunnelling process that causes longitudinal displacements in a form of a settlement trough which also needs to be analysed. The incremental longitudinal settlement trough is followed Gaussian curve and have the shape of a cumulative probability curve. According to field studies of settlements above slurry shield tunnelling machines, most of the settlement is associated with the tail void and the surface settlement directly above the tunnel face is often significantly less than $0.5S_{max}$.

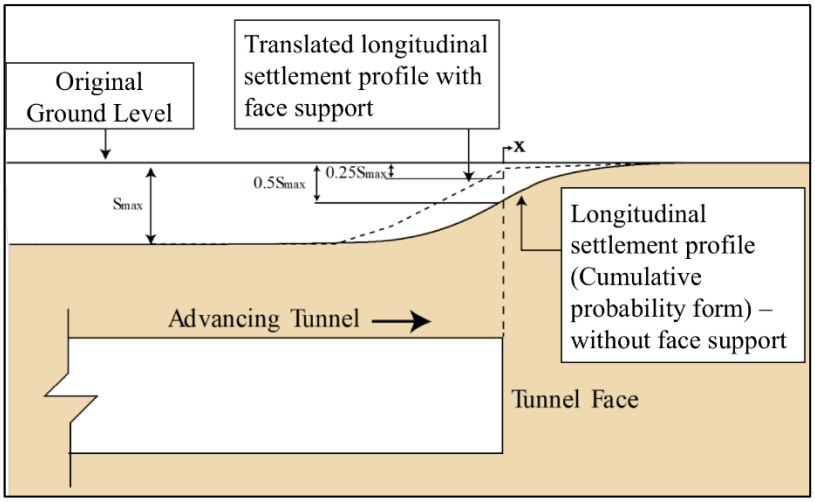

FIGURE 2.6 Illustration of surface settlement caused by shield tunnelling (Attewell, 1986)

In addition, different field studies observed five components of settlement phase by the shield tunnelling to illustrate the location of the settlement phase which has been suggested. Closed-face tunnel excavation might reduce the settlement to less than $0.5S_{v,max}$ by 25% to 30% compared to open-face tunnel excavation (Mair & Taylor, 1993). Equation 2.7 can be used to calculate the settlement above the tunnel center line at position x.

$$S_v(x) = S_{v,max} \cdot \frac{1}{i\sqrt{2\pi}} \cdot \int_{-\infty}^{x} e^{-\frac{x^2}{2i^2}} \quad (2.7)$$

Where, x is the distance from the tunnel face in the settlement trough's longitudinal direction.

2.4 TBM Parameters

Subsidence during tunnel construction is widely known to be closely connected to the driving parameters of the TBM. In general, Ni et al., (2016) explained the total jacking load imposed by the jacking hydraulic cylinder combined with the cutter head pressure or face resistance is determined by the type of ground encountered by the TBM, as well as the friction force acting on the outer pipe surface area. Frictional forces on a slurry TBM cutterhead are typically low, however, it can rise if clogging occurs in the space between the main cutterhead structure and the excavated face (Haryono et al., 2021). As reported by Zhen et al., (2021) and Ye, Peng, Yang, et al., (2020), they observed the importance to employ efficient pipe jacking parameters control such as frontal jacking pressure, friction resistance of the slurry sleeve and slurry pressure. Figure 2.7 illustrates the horizontal pressures imposed on a TBM during its advancement.

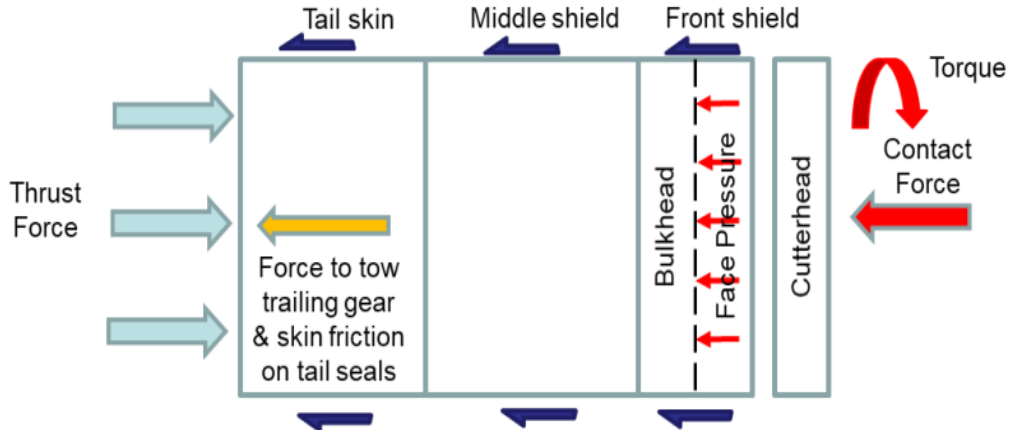

FIGURE 2.7 Horizontal forces on a closed shield tunnel (Haryono et al., 2021)

In the concept, TBM employs equal and inverse forces on the ground. The cutterhead is the first point of contact between the ground and the TBM. As part of the excavation process, further force is delivered to the tools on the cutterhead to push them into the face, which is known as contact force. The primary pressure bulkhead is subjected to face pressure. Ye et al., (2020) explained the friction occurs along the TBM shield and at the tail seals where they contact the rings. As the TBM tows the trailing gear, there is a drag force at the tail. The hydraulic jacks used at the back of the TBM overcome the total forces delivered to the TBM (in the direction of tunnelling). Therefore, the total jacking load increases with distance as friction resistance increases.

$$F_{total} = F_{fp} + F_c + F_{fr} + F_d \tag{2.8}$$

where F_{total} = Total jacking force

F_{fp} = Force due to face pressure

F_c = Contact force

F_{fr} = Force due to friction along the shield and the friction between the tail seals and lining rings

F_d = Tail drag

The numbers and locations of the intermediate jacking stations (IJS) were determined by the pipeline design and were subject to change if the actual jacking force varied considerably from the expected values (Zhan & Chen, 2020). These install pipes, with a strength of 50 MPa, were tested to verify that they could withstand the overburden pressure and surcharge loads imposed from the ground, as well as the needed jacking force for both straight and curved pipelines built as well as the required jacking force for both straight and curved pipelines designed. Zhen et al. (2021) thoroughly discussed the analysis of the impact of the eastern line pipe jacking on the overlying highway and the employment of pipe jacking parameters control.

CHAPTER 3

METHODOLOGY

3.1 Overview of Research

Methodology is part of the required process to predominantly achieve the project objective's goals. Throughout this project, data related to the research problems is collected nearby the construction site and thoroughly discussed. Most of the data was focused on obtaining quantitative data in which the data collected through measuring the variables based on surveying works and further field investigation on secondary data such as books, journal and internet sources.

A condition survey of the existing road and structures within 15 m of the pipe jacking alignment was conducted by following the Environmental Impact Assessment (EIA), then prevent any possible environmental impacts before, during and after tunnelling construction. It is practically difficult to conduct a full geological assessment of soil parameters at each tunnel instrumentation segment and to record the existing condition of the adjacent structures. Therefore, research is involved simulating using Microsoft Excel.

The overview of project flow is illustrated as shown on the flowchart below:

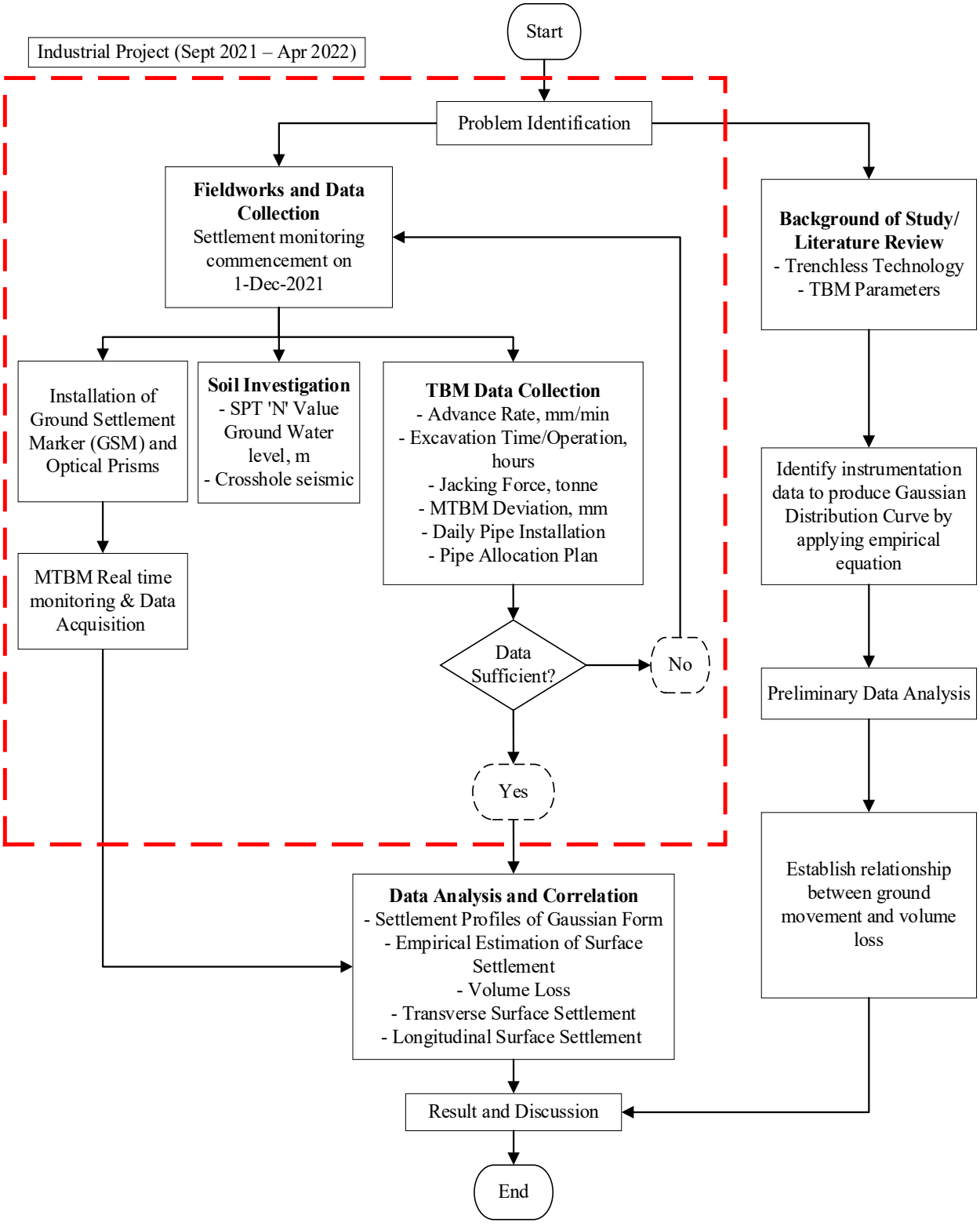

FIGURE 3.1 Research Flow Chart

3.2 Geotechnical Information

An interpretative report is required to determine the characteristics of the soils likely to be encountered. The aim of a geotechnical study for trenchless construction is to provide necessary data for the Contractor to propose construction approach and to eliminate the risks during advancing. In addition to general information such as existing underground utilities and potential for obstructions, appropriate knowledge of geological soil and groundwater conditions along the pipeline alignment and comprehend of the pipe-soil interaction are the keys for maintaining the pipelines that meet the performance requirements.

The selection of the trenchless technology method based on the information below is useful such as (American Society of Civil Engineers & Construction Institute, 2022):

- The presence and depth of gravels, cobbles, boulders, loose sand, soft clay, manmade structures and utilities;
- Soil and rock stratigraphy;
- Depth to bedrock;
- Rock type, core recovery, Rock Quality Designation (RQD), weathering, hardness, unconfined compressive strength, orientation and spacing of bedding;
- Groundwater elevation, groundwater conditions at geologic contacts;
- Difficulties encountered during geotechnical investigation, such as loss of drilling fluid or caving of a drilled hole;
- Soil and rock engineering properties, including corrosivity evaluation, particle size distribution, hydraulic conductivity, unit weight and shear strength;
- The presence of expansive or collapsible materials

The tunnelling project commenced in November 2021 and was completed in March 2022, featuring a crossing underneath the KL-Seremban Highway near Jalan Teknologi 3, Taman Teknologi Malaysia (TPM) Parking. Figure 3.2 illustrates a plan of the tunnel alignment and section under consideration.

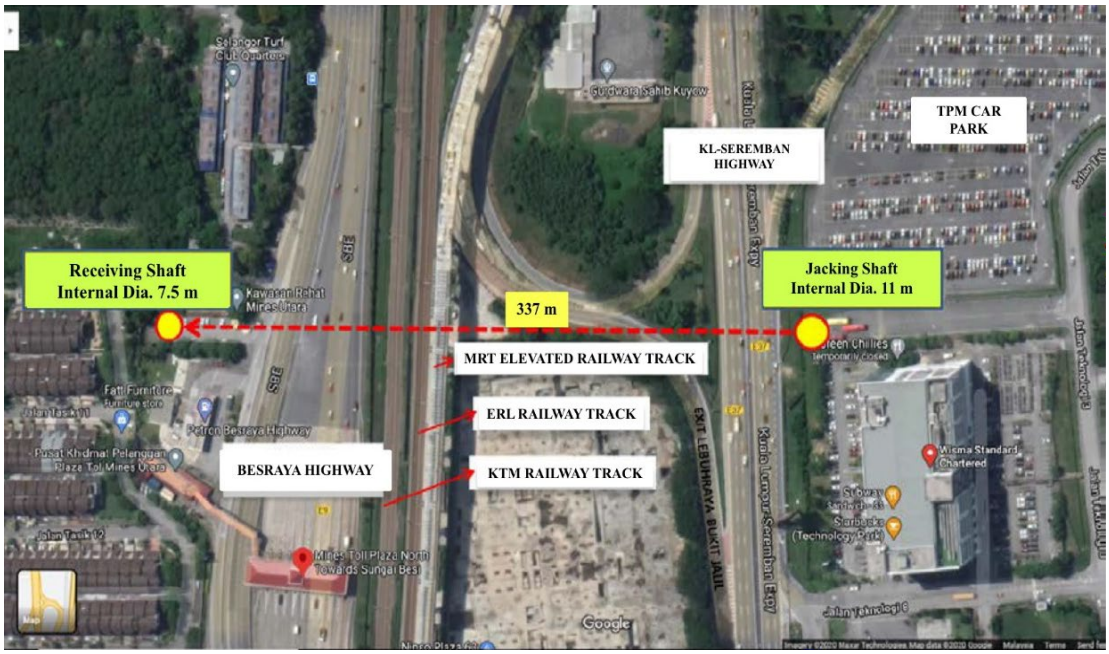

FIGURE 3.2 Site Plan of Study area

Borehole drilling and geophysical studies were used to gather information on the geotechnical characteristics of the ground during the design stage. A total of six boreholes were advanced by conventional rotary drilling with water as the flushing medium along the pipe jacking alignment starting from jacking shaft to Intervention Block (IB 1).

The aim of the drilling activity is to get overview on the soil condition in terms of engineering properties, based on visual observations, in-situ testing such as the Standard Penetration Test (SPT) and laboratory testing of undisturbed and disturbed samples. One Toho D2-K boring machine was equipped for the rotary wash boring. The boreholes were advanced by the rotary action of the drill bits and the cut was washed up to the surface by the flushing action of water. The boreholes were cased

with NW size casings of 76 mm inner diameter to prevent the wall of the hole from collapsing.

3.3 Monitoring Instrumentation

Exact geotechnical observations are monitored using instruments during tunnel advances, allowing the construction team and operators to make a judgement and modify the construction methodology or design parameters as needed. Therefore, instrumentation monitoring plays an important role in construction control, design verification and structural safety. The list of some major instruments used in the tunnelling project at E37 KL-Seremban Highway and its function is summarised.

3.3.1 Ground Settlement Marker (GSM)

For tunnelling related works, the GSMs were installed in arrays as shown in Figure 3.3 in order to record the cross-sectional settlement that will be used to determine the volume loss and to adjust the TBM parameters for more effective tunnelling ahead. A PVC pipe attached with nail represented the ground settlement marker.

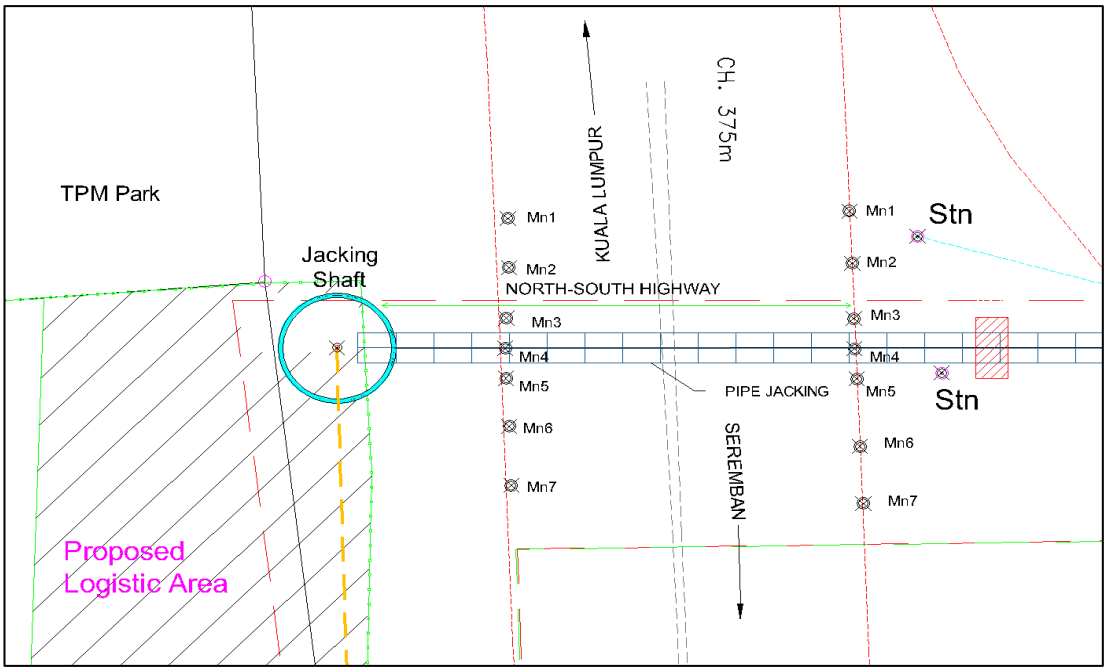

FIGURE 3.3 Pipe jacking alignment

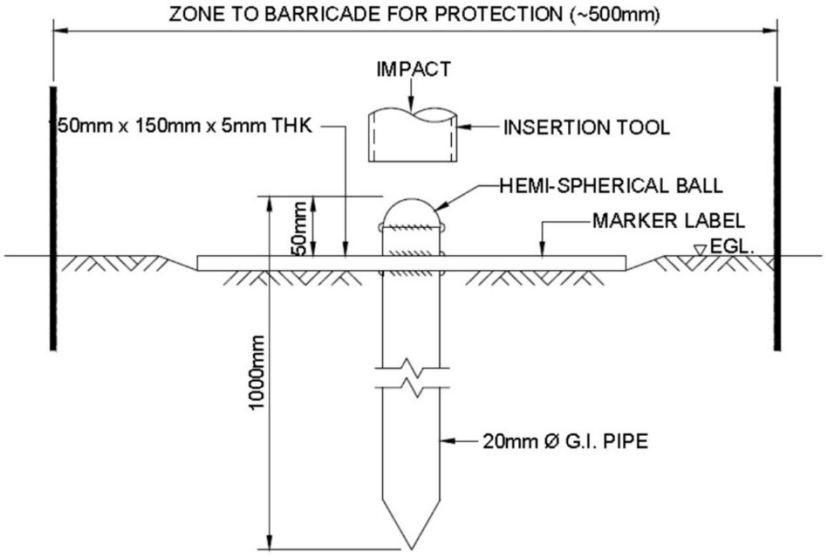

FIGURE 3.4 Schematic diagram of installed settlement marker on ground

The settlement markers were installed to determine any localised surface ground settlement or heave of roads and buildings as shown in Figure 3.4. A total of seven settlement markers were installed provisional in KL-Seremban Highway. Galvanized Iron (GI) pipe was installed using the hammering method and the settlement markers, which were 1000 mm long and 20 mm in diameter, were installed on the site based on the ground surface. It will be monitored by a total station with reference to the temporary datum, which was an existing STN 1, a nail on top of walkway.

3.3.2 Optical Prism

The surface settlement is monitored using a total station with an optical prism placed on top of the attached nails. Surveyor uses the optical prism to determine the coordinates of the proposed area. Any obstructions in the measurement path must be avoided and the current refraction condition must be considered.

3.4 Principle AVN Shielded

AVN Slurry technology is fundamentally based on the use of the excavated material in the MTBM face mixed with the liquid medium that circulating the system. The AVN machines so called the Slurry Pressurised Balance Microtunnelling and Pipe Jacking machines are tunnel boring machines with liquid conveyance systems that are applicable to the ground condition.

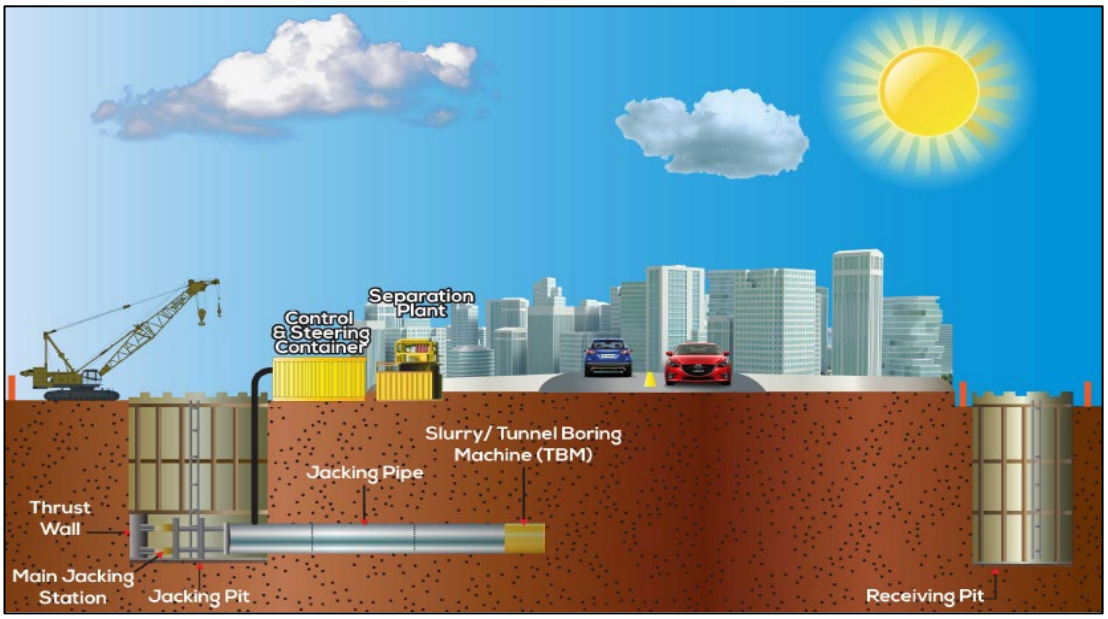

FIGURE 3.5 Trenchless pipe laying

Figure 3.5 above illustrates when TBM remotely operated by the operator using a control board that allows the operator to monitor and change various settings for advancement according to soil conditions. The personnel entrance in the tunnel is limited to secondary works such as topographic survey, regular checks on equipment and regular maintenance. The TBM hosts a rotating cutter head that crushes the excavated materials while advancing. The optimum equipment and cutting tools must be specified for the rock and ground conditions to be encountered in order to successfully employ TBM technology in mining. The excavated material is then transported to the surface by a closed-loop slurry system. The rate of soil excavation and extraction are controlled to minimise soil settlement on the ground surface.

3.3.3 Extension Kit (MTBM Upsizing Method)

The techniques of microtunnelling for the treated water pipeline installation has been fitted with an extension kit as shown in Figure 3.6. By applying an extension kit, the range of application can be emphasized in terms of the diameter. Occasionally, it called skinning or upsizing in which the process involves bolting a custom-manufactured, fitted steel cylinder onto the exterior of the MTBM. Extension kits are used to drive different diameters of pipes with one machine with consequences on the slurry treatment management, lubrication effectiveness and jacking pressure.

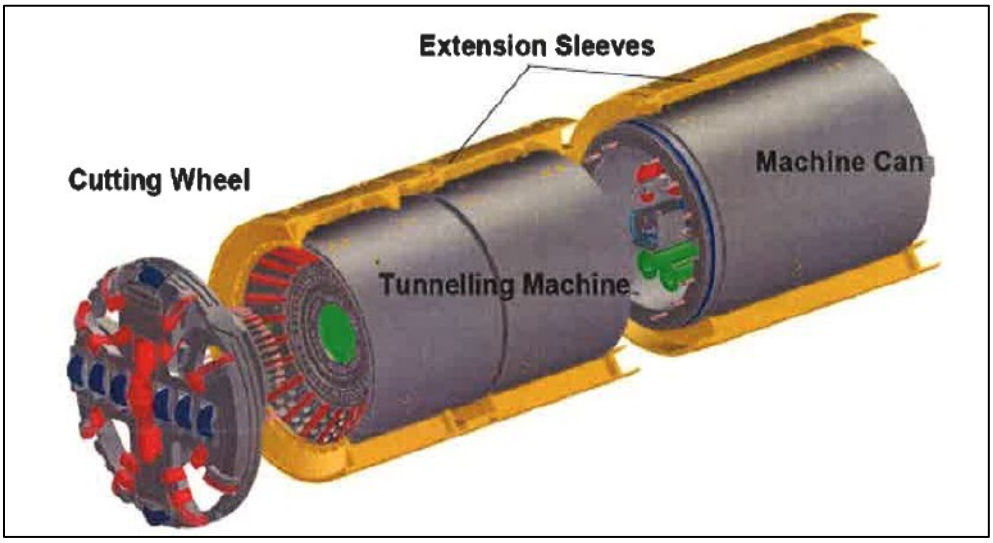

FIGURE 3.6 Schematic of an AVN MTBM

TABLE 3.1 Specification of the Herrenknecht AVN1800TB w/EXT 3025

Type/Model	AVN1800TB w/EXT 3025	
Length without cutterhead	3670	mm
Outer Diameter	2241	mm
Nominal Diameter	1800	mm
Permanent Torque	424	kNm
Steering force per cylinders at 450 bar push/pull	1145/636	kN
Steering cylinder maximum pressure	500	bar
Total Length of machine	7520	mm

The AVN1800TB w/EXT 3025 is designed for mixed ground especially hard rock conditions by using different cutting wheels (full face excavation) and central access to cutting wheel for tool replacement.

3.3.4 Excavation Progress

The TBM information is transmitted to the computer which established the position of the TBM and any necessary adjustments. A remote-controlled guidance system keeps track of the shield's position. As the jacking forces increase during pipe installation due to skin friction and the length of the installed pipeline, the intermediate jacking station (IJS) will be installed and used to reduce and distribute the jacking forces, thereby reducing the forces applied by the hydraulic jacks at the jacking shaft.

FIGURE 3.7 TBM operating room

FIGURE 3.8 Jacking shaft

3.4.3 Concrete Cladded Mild Steel Jacking Pipes

In accordance with the project Scope of Provision, the product pipes with built-in Mild Steel collar and Spigot joint inclusive of Rubber Ring and Hydrophilic Seal) was fully supplied. The pipes are considered an important factor to the works and prior controls and manufacturing including supply efficiencies shall be considered. A proper design base on the latest manufacturing is mandatory to ensure a fail proof installation and also against any uplifting scenarios. In relation to the standard technical requirement, but not limited to the followings:

- Pipe Joints design
- Pressure Transfer Ring/Packing (Chipboard V100 or equivalent)
- Steel Reinforcement at the pipe ends
- Sealing material and installation method
- Pipe End Squareness
- IS Construction and connection tolerances
- Eccentricity of the Steel Pipe and Concrete Pipes

The scope of pipe supplies carried out under the provision includes several types of DN2500 pipe as follows:

- Lead Pipes (LP) = 3.5 m (Special Standard pipes without Steel Pipe and inner lining)
- Lubrication Pipes (LD) – L = 3.50 m (with 3 sets of non-return valve injection nozzles)
- Intermediate Jacking Pipe (IS) – L = 3.00 m (Lead and Trailing Pipe)

FIGURE 3.9 Timber packing

All pipe joints are supplied with a compression distribution packing in accordance with the technical requirement of the project. The joint packing shall be resilient and shall distribute pipe stresses arising from jacking loads. The packing material shall extend around the full circumference of the pipe joint, outer edge approximately 15 mm from the outer face of the pipe MS Collar at the joint and inner edge approximately 10 mm from the outer OD of the Steel Pipe Lining.

CHAPTER 4

RESULTS AND DISCUSSION

This chapter represents the field data collection from TBM control room, ground settlement from GSM, soil investigation and cross-hole seismic tomography. The correlation between the TBM performances such as TBM deviation, advancement rate and jacking force to the encountered ground condition which is SPT 'N' value and soil type, can be determined by the most influencing factors that maximise the TBM productivity. On the other hand, the value of the maximum surface settlement can be used to back-calculated the volume loss of soil and the friction coefficient also can be determined form the back-calculation of the jacking force prediction equation.

4.1 Background of Case Study

Pengurusan Aset Air Berhad in the development of Langat 2 Water Treatment Plant awarded Pembinaan IKHASAS-MERAK Sdn. Bhd. in an effort to install the treated water gravity main from Sungai Besi Highway to existing Dengkil reservoir in Selangor Darul Ehsan/Wilayah Persekutuan Kuala Lumpur. The proposed Langat 2 Treatement Phase 1 Works involves a major works component such as Western Corridor Treated Water Bulk Transfer Works including the construction of the proposed Bukit Enggang Balancing Reservoir, proposed service reservoirs at Sg. Besi, Bukit Jalil and Petaling as shown in Figure 4.1, and 31 km length of pipe connections works of various diameters pipe ranging from 2.5 m diameter to 1.95 m diameter pipe.

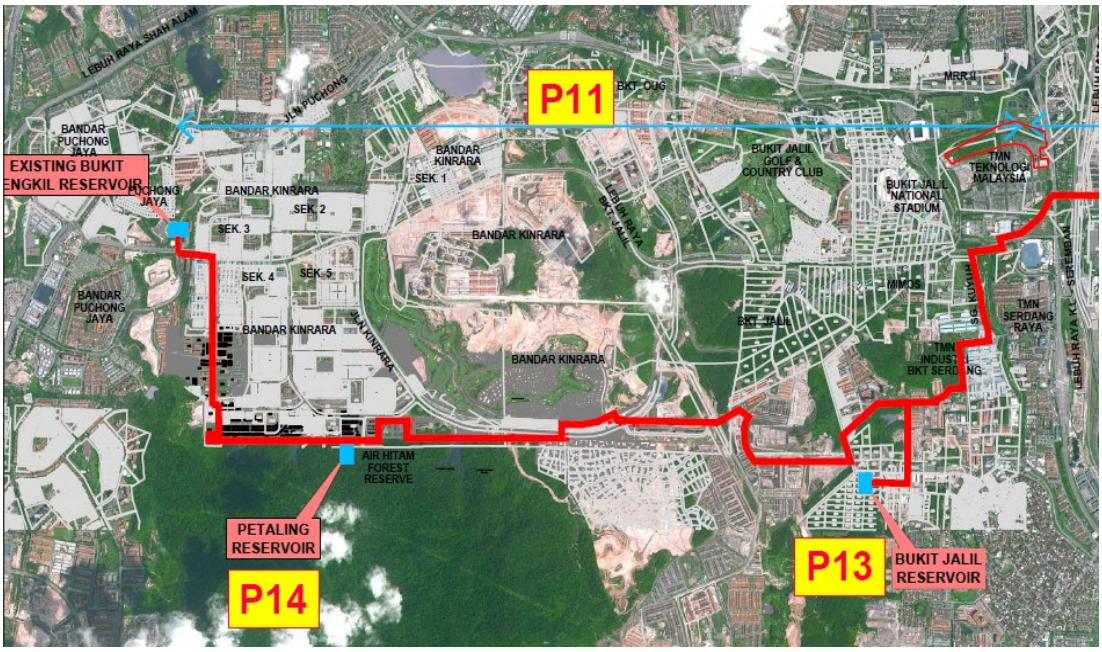

FIGURE 4.1 Treated water bulk transfer work location plan connected to P11

The project involves the construction of water treatment and distribution facilities for water transported from the Pahang-Selangor Raw Water Transfer (PSRWT), with an ultimate nominal treated water output capacity of 2260 Mld. One section of the installation works required the use of trenchless technology to install the DN2500 mm (ID2455/OD3000) Concrete Cladded Mild Steel Jacking pipe.

4.1.1 Site Selection

The initial phase of the project for pipe jacking alignment involving crossing under the KL-Seremban Highway located at near the TPM Parking as shown in Figure 4.1. The TBM drives crossed underneath the following launch out of ERL Track, KTMB Track and BESRAYA Highway, Kuala Lumpur. In terms of environmental and physical facilities, the KL-Seremban Highway is extremely congested which had faced various limitations while completing the project. A survey of the existing ground level within 20 m of the pipe jacking alignment was conducted where it is most suitable location for ground monitoring instrument installation.

FIGURE 4.2 Settlement at KL-Seremban outbound towards KL at KM D7.4 (11m from tunnel axis) – 10th December 2021

FIGURE 4.3 Settlement at KL-Seremban outbound towards Seremban at KM D7.4 (15m from tunnel axis) – 10th December 2021

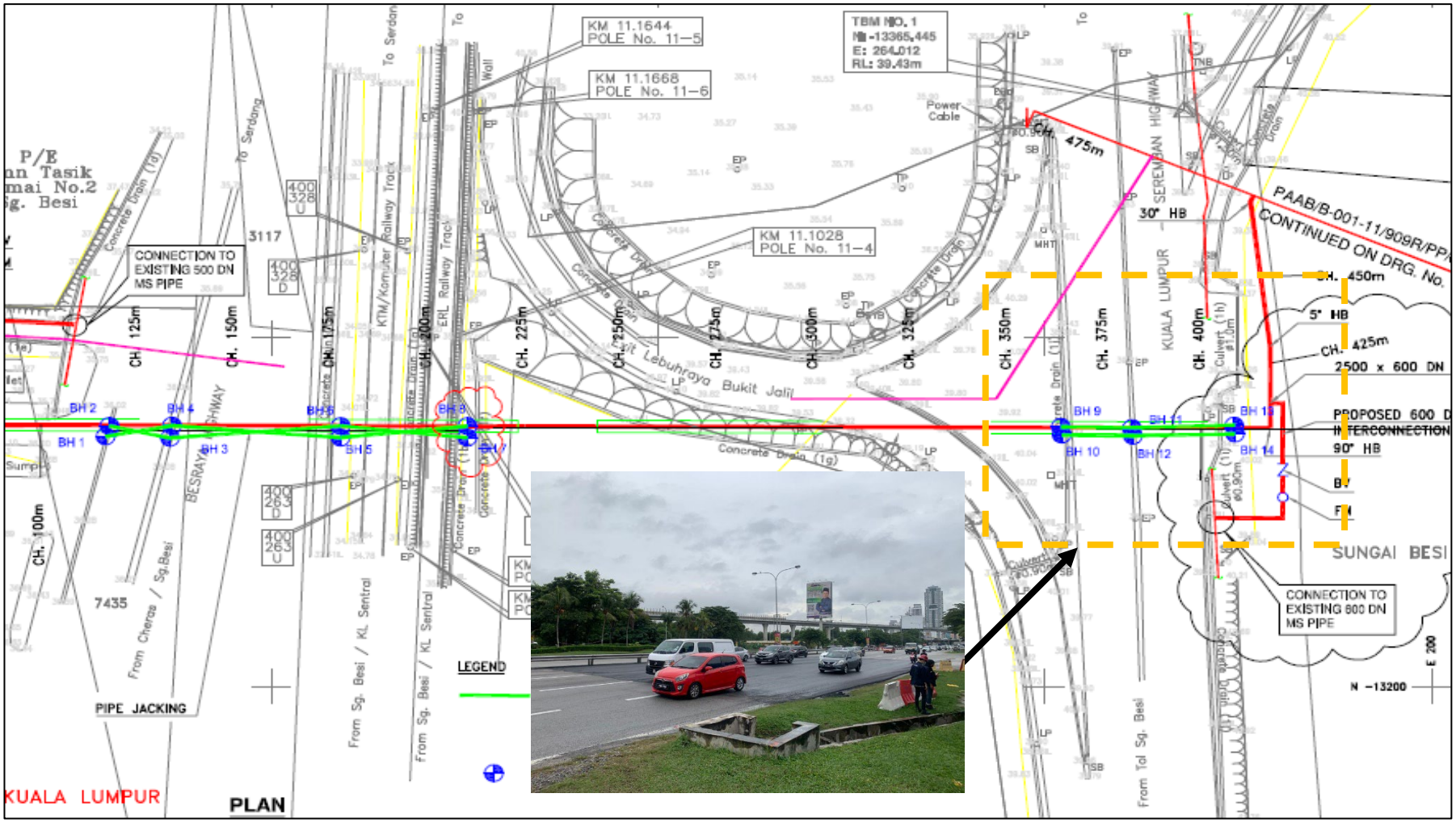

FIGURE 4.4 Site location at KL-Seremban Highway

4.1.2 Geological Condition

A complete site investigation and interpretative report are required for designing and estimating pipe jacking projects to determine the characteristics of the soils likely to be encountered. In addition to general information such as existing underground utilities and potential for obstructions, environmental conditions, required pipeline depth, grade, tolerances, appropriate knowledge of geological soil and groundwater conditions along the designed pipeline alignment and understanding of the pipe-soil interaction are the keys for maintaining the pipelines that meet the performance requirements. Seismic Final Report and Soil Investigation Report has been prepared and submitted by Subsurface Engineering Sdn. Bhd.

6 30-m deep boreholes namely as BH 9, BH 10, BH 11, BH 12, BH 13 and BH 14 as shown in Figure 4.6, were constructed for identifying the geological conditions along the drive as described in Table 4.1 . As much as possible, descriptions of soil and rock samples were in accordance with BS 5930: 1981 Code of Practice for Site Investigations formerly known as CP 2001. According to the soil investigation, it is confirmed by drilling that the stratum layer crossed by the jacking pipe is mainly fractured limestone.

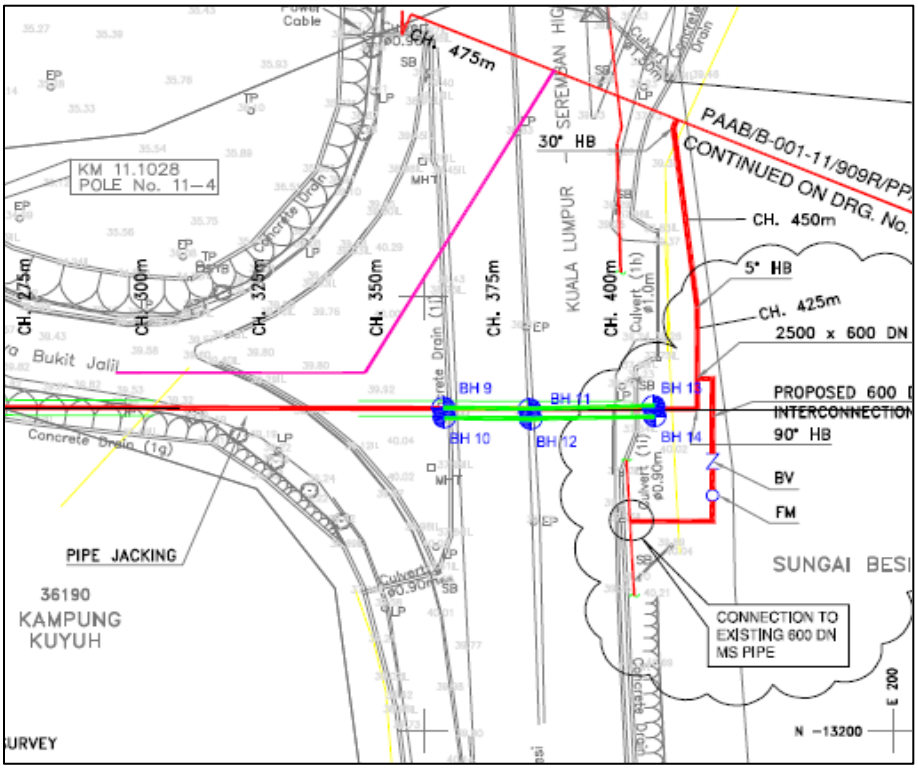

FIGURE 4.5 Boreholes location crossing KL-Seremban Highway

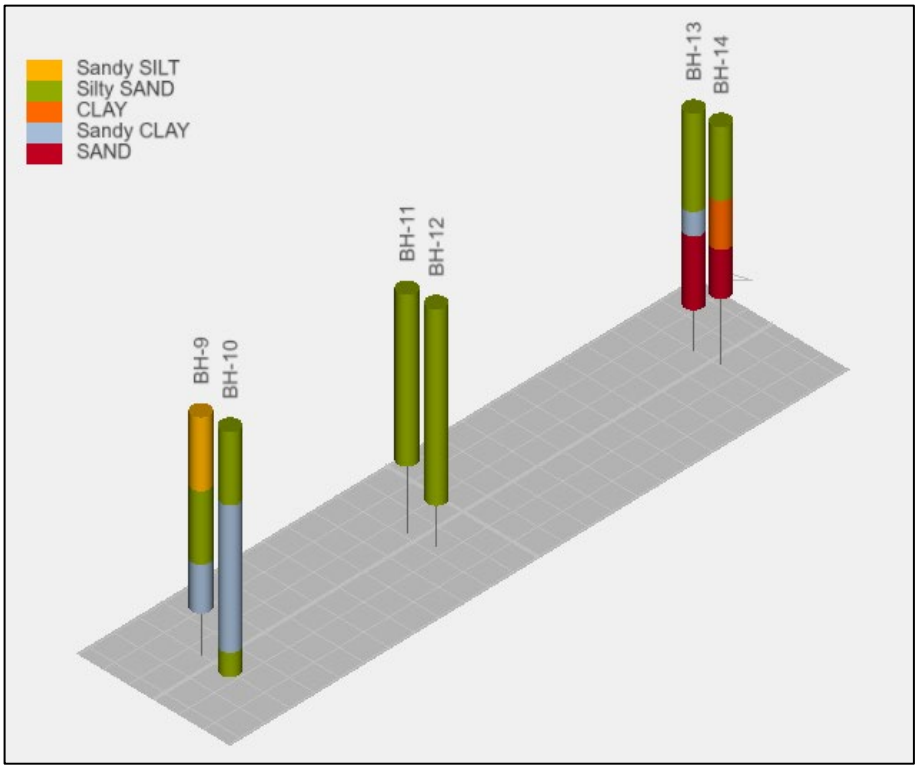

FIGURE 4.6 Soil Profile

From the soil investigation, the soils are mainly consisting of silty sand at every borehole. There is formation of rock (limestone) layer along the pipe jacking alignment. The borehole point that encountered the highest rock layer were found in BH11 and BH14 at 10.5 m to 10.95 m depth from the ground surface level which is above the pipe jacking alignment.

TABLE 4.1 SPT "N" value for each borehole

Soil Investigation for Pipe Jacking Alignment Crossing North-South Highway, Kuala Lumpur.						
BH No.	BH 9	BH 10	BH 11	BH 12	BH 13	BH 14
GL (m)	39.44	39.44	39.76	39.76	37.54	37.54
Depth (m)	SPT-N VALUE					
0 – 0.1						
1.5 –1.95	8	21	9	9	9	11
3.0 – 3.45	40	31	7	11	11	10
4.5 – 4.95	9	5	0	13	8	5
6.0 – 6.45	8	8	1	19	4	UD
7.5 – 7.95	11	9	7	10	13	14
9.0 – 9.45	5	7	2	6	11	10
10.5 – 10.95	0	6	Hit Rock	5	4	Hit Rock
12.0 – 12.45	Hit Rock	1		Hit Rock	Hit Rock	
13.5 – 13.95		13				
15.0 – 15.45		Hit Rock				
16.5 – 16.95						
18.0 – 18.45						
19.5 – 19.95						
21.0 – 21.45						
22.5 – 22.95						
24.0 – 24.45						
25.5 – 25.95						
27.0 – 27.45						
28.5 – 28.95						
30.0 – 30.45						
UD: Undisturbed Sample						

Table above shows the SPT "N" value for each borehole along the pipe jacking alignment. The highest SPT "N" value number was 40 found in BH9 at 3.0 m to 3.45 m depth meanwhile the lowest value was 0 recorded in borehole namely BH9 and BH 11 below 10.95 m and 4.95 m, respectively. This type of soil for each borehole dominantly consists of sand, clay with a trace of silts was also observed as shown is Table 4.2. Figure 4.7 shown the pattern of SPT "N" value where the values vary the depth inconsistently.

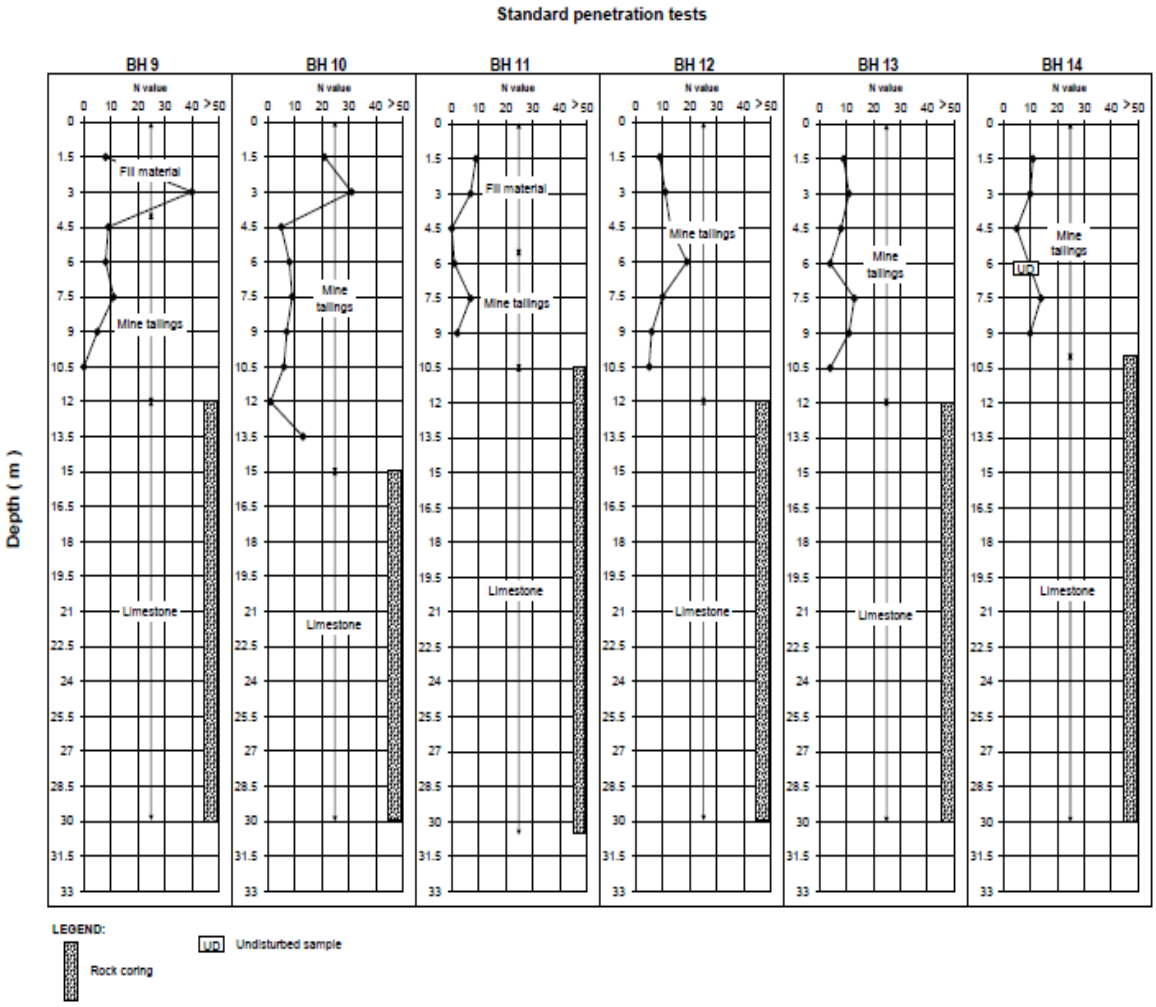

FIGURE 4.7 Pattern of SPT "N" value represents every borehole

The geology of this site is made up of mine tailing deposits, which are composed of sand, silt and clay underlain by limestone bedrock. The construction site is underlain by a limestone formation that is suspected of having multiple cavities, making it vulnerable to sinkhole occurrences. The study area is underlain with fractured limestone of low slope which varies from 10° to 80°. The rock uniaxial compressive strength of limestone ranges from 60 to 140 MPa. Table 4.2 shows the properties of soil layers comprising the upper soil stratum.

TABLE 4.2 Soil Description encountered for each borehole

Soil Investigation for Pipe Jacking Alignment Crossing North-South Highway, Kuala Lumpur.						
BH No.	BH 9	BH 10	BH 11	BH 12	BH 13	BH 14
Depth (m)	SOIL DESCRIPTION					
0 – 0.1	Topsoil – Sandy SILT	Topsoil – Silty SAND	Topsoil – Silty SAND	Topsoil – Silty SAND	Topsoil – Silty SAND	Topsoil – Silty SAND
1.5 –1.95	H. Sandy SILT	MD. Silty SAND	L. Silty SAND	L. Silty SAND	L. Silty SAND	MD. Silty SAND
3.0 – 3.45	H. Sandy SILT	D. Silty SAND	L. Silty SAND	L. Silty SAND	MD. Silty SAND	MD. Silty SAND
4.5 – 4.95	L. Silty SAND	F. Sandy CLAY	L. Silty SAND	MD. Silty SAND	L. Silty SAND	F. CLAY
6.0 – 6.45	L. Silty SAND	F. Sandy CLAY	VL. Silty SAND	MD. Silty SAND	S. Sandy CLAY	F. CLAY
7.5 – 7.95	L. Silty SAND	F. Sandy CLAY	L. Silty SAND	MD. Silty SAND	MD. SAND	MD. SAND
9.0 – 9.45	F. Sandy CLAY	F. Sandy CLAY	L. Silty SAND	L. Silty SAND	MD. SAND	MD. SAND
10.5 – 10.95	F. Sandy CLAY	F. Sandy CLAY	Hit Rock: LIMESTONE	L. Silty SAND	MD. SAND	Hit Rock: LIMESTONE
12.0 – 12.45	Hit Rock: LIMESTONE	F. Sandy CLAY		Hit Rock: LIMESTONE	Hit Rock: LIMESTONE	
13.5 – 13.95		MD. Silty SAND				
15.0 – 15.45		Hit Rock: LIMESTONE				
16.5 – 16.95						
18.0 – 18.45						
19.5 – 19.95						
21.0 – 21.45						
22.5 – 22.95						
24.0 – 24.45						
25.5 – 25.95						
27.0 – 27.45						
28.5 – 28.95						
30.0 – 30.45						
*MD = Medium Dense *D = Dense *St = Stiff *VSt = Very Stiff S = Soft *L = Loose *VL = Very Loose *H = Hard *F = Firm						

4.1.3 Cross-hole Seismic Tomography

The geophysical method was utilised to correlate the type of soil underneath the highway with the data from borehole. BH9/BH10 and BH11/BH12 were tested using cross-hole seismic topography method. The borehole positioned in pair along the tunnel alignment as shown in Figure 4.6. The results cross-hole seismic tomography is presented in Figure 4.8. The properties of the soil are shown by the wave velocities that travel from one borehole to the one opposite it. Higher velocity denotes layers of rock or other hard materials, while low velocity denotes sand and clay. High velocity observations may also be caused by the presence of groundwater.

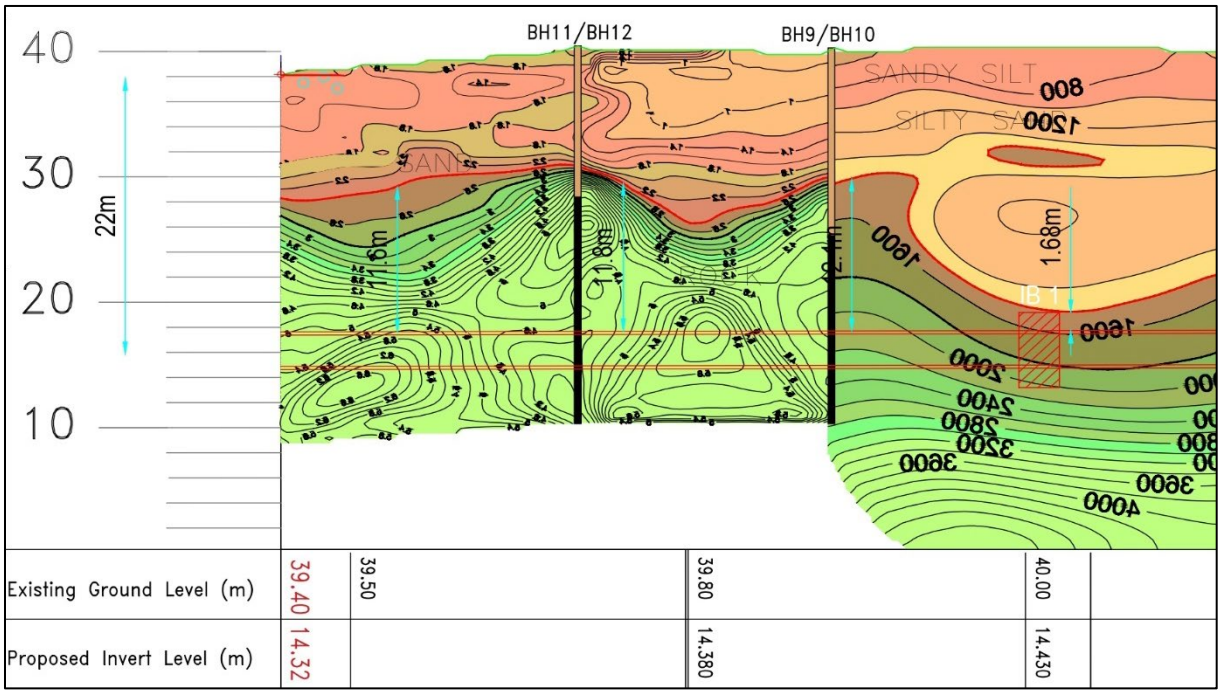

FIGURE 4.8 Result of cross-hole seismic tomography

Low wave travel velocity can be seen over most of the soil, which indicates that the soil is loose and soft. The maximum velocity recorded is 6.6 km/s while the slowest is between 2.6 km/s in the rock formation whereas maximum velocity measured in soil layer range from 0.8 km/s to 2.2 km/s. The method uses the measuring travel times of seismic ray paths between two boreholes to derive an image of seismic velocities. The result obtained from cross hole seismic tomography is quite similar with soil profile

obtained from the wash boring method as discussed previously where the majority of soil is made up of sand and clay material.

Higher velocities signify a harder layer, which is represented by the rock or as a response of groundwater. According to geotechnical exploration, the limestone positioned approximately 10 m between the soil layer and the tunnel crown. The TBM delved in and out of sloping bedrock through a wide zone of limestones between CH325 and CH425. The intervention block (IB) is utilised as a ground treatment assessment on CH350, with a $6\,m \times 3\,m$ in dimension. The intervention block faced both limestone and overlaying soil making it possible to enable timely planning in improved ground conditions for a period of time to inspect the cutter discs. Grouting work was done concurrently during the advancement phase to guarantee safe TBM operations and minor damage to the environment as well as improving the ground by injection by grouting to increase the rock and soil stability.

FIGURE 4.9 Limestone collected from STP

4.2 TBM Driving Data

TBM data and control parameters required for the tunnelling machine's operation are accommodated in the control room and accessible at the surface. The operating parameters collected from the operator's control room can be used to interpret the ground conditions encountered during tunnelling. This study has involved a review of the TBM parameters including rate of advancement, jacking force (kN), deviation of line and grade (mm) and drilling fluid. Graphical figures of the relationships were established in order to interpret the surface settlement induced by tunnelling as it is important to examine the mechanism of tunnel operation.

4.2.1 Advancement Rate

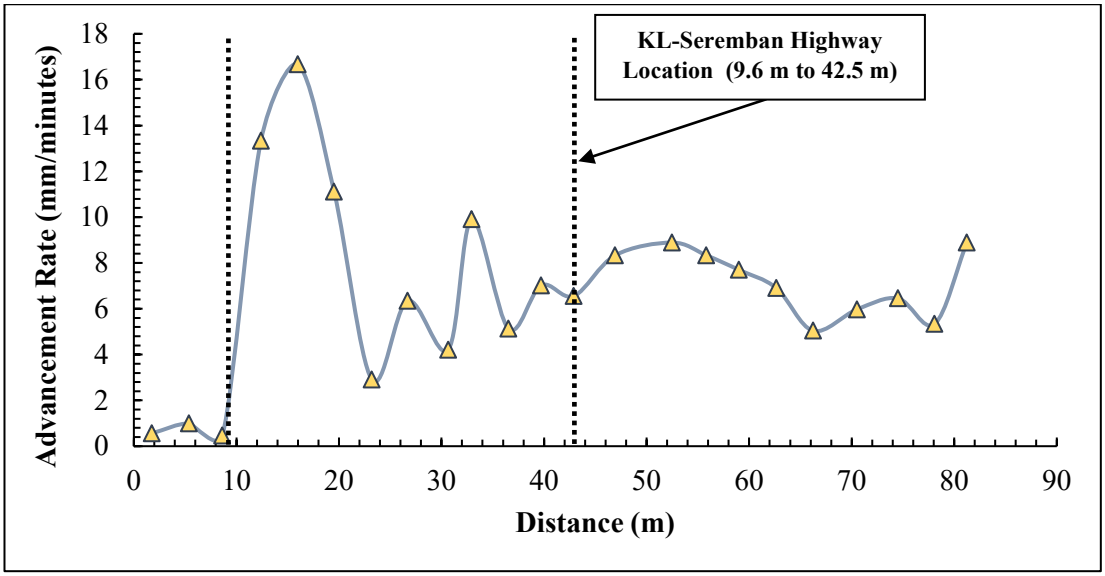

FIGURE 4.10 Advancement Rate of TBM

Earlier mentioned, an accurate estimation of attainable penetration rates is crucial component in TBM applications to any tunnelling project. The advancement rate, which is commonly measured in mm/minutes, indicates how rapidly the shield can advance forward. Based on data diagrammatically in Figure 4.10, 81.23 m of tunnelling distance in limestones formation, a comparison was made between results crossing the KL-Seremban Highway and upon approaching the IB 1.

The TBM performance rising sharply at the beginning of driving with up to 16.67 mm/minutes at 15.98 m distance whereas the rate fell to a low point of 2.91 mm/minutes at 23.3 m distance. The findings showed that the models are applicable in a specific range of geological conditions but require some modifications or correction factors to more accurately represent the rock mass parameters and ground conditions. The rate of advancement on the 81 m drive distance in rock varied greatly, from 0.47 mm/minutes to 17 mm/minutes.

4.2.2 Jacking Force

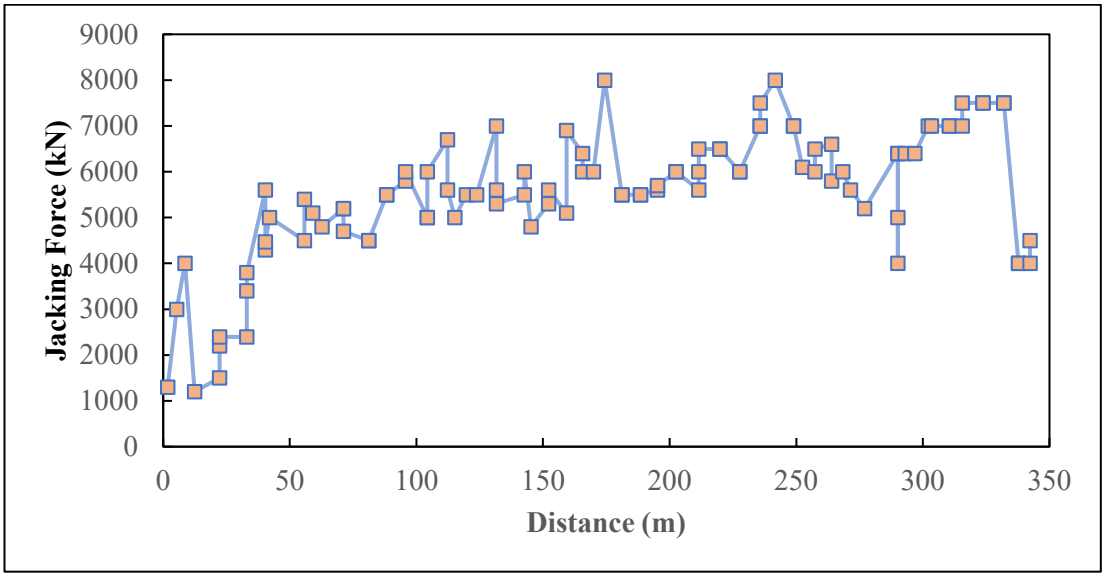

FIGURE 4.11 Jacking force against drive distance

Figure 4.11 presents the relationship between 342.48 m drive distance and jacking force. The jacking force fluctuated and gradually increased over distance and remarkably higher in rock region with the average of 5534 kN. The geological stress of the surrounding rock mass caused the jacking force to increase from 1200 kN to 5400 kN in 33.59 m, stabilising the tunnel itself. The highest jacking force was 8000 kN which can be said that the jacking force increases after a stoppage and largely governed by the stoppage duration.

4.2.3 Deviation of Line and Grade

The shield can be regulated in accordance with system specifications based on the trust cylinder. The proposed tunnel must be able to be excavated following the pre-established line and grade, hence the TBM must be designed and built accordingly. The design and setup of the machine might be significantly impacted by factors like the minimum horizontal or vertical curvature.

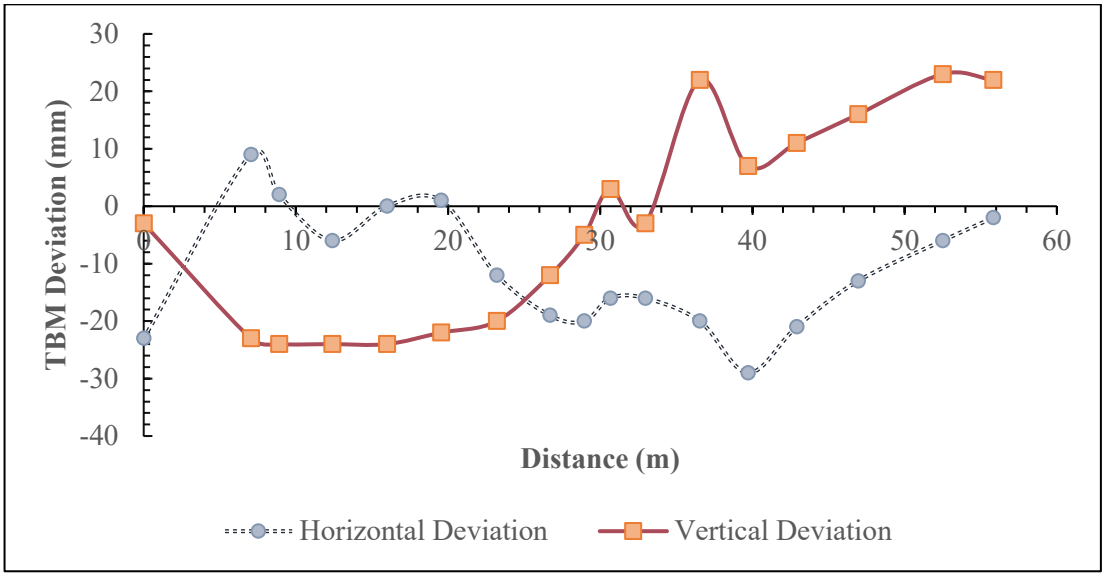

FIGURE 4.12 TBM Deviation

The tunnel alignment is expected to remain steady with +0.2019% of its slope. The proposed jacked pipeline indicated no obstruction to be blocked by the jacking shaft's as-built records, but the construction tolerance may result in an alignment issue. While significant advances have been made, the deviation must be kept to a minimum. In view of the Figure 4.12 above, the TBM deviates around -23 mm in the vertical and 9 mm in the horizontal directions throughout the first 9 m of the TBM drive. The TBM operator achieves a minimum alignment of around 1 mm in the horizontal direction after driving 20 m and 3 mm in the vertical direction in 30.66 m drive distance.

As the machine advances further beyond 40 m, the deviation trend in horizontal direction is slightly to be as high as -2 mm. Overall, the vertical direction trends fluctuation is somehow not exceeding -30 mm. Generally, there is a 50 mm permissible variation from the contractor's procedure up until it reaches the receiving pit. Concurrently, surveyors can fix the existing horizontal and vertical positions of TBM using traverse and levelling techniques.

4.2.4 Back Analyses of Jacking Force and Skin Friction

The output data from the TBM were recorded at 342.48 m drive distance. The skin friction, f_s is jacking resistance per unit surface area or frictional stress expressed in kN/m^2. The face resistance can be significant compared to the friction resistance for big diameter pipe jacking with stabilised overcut and sufficient lubrication.

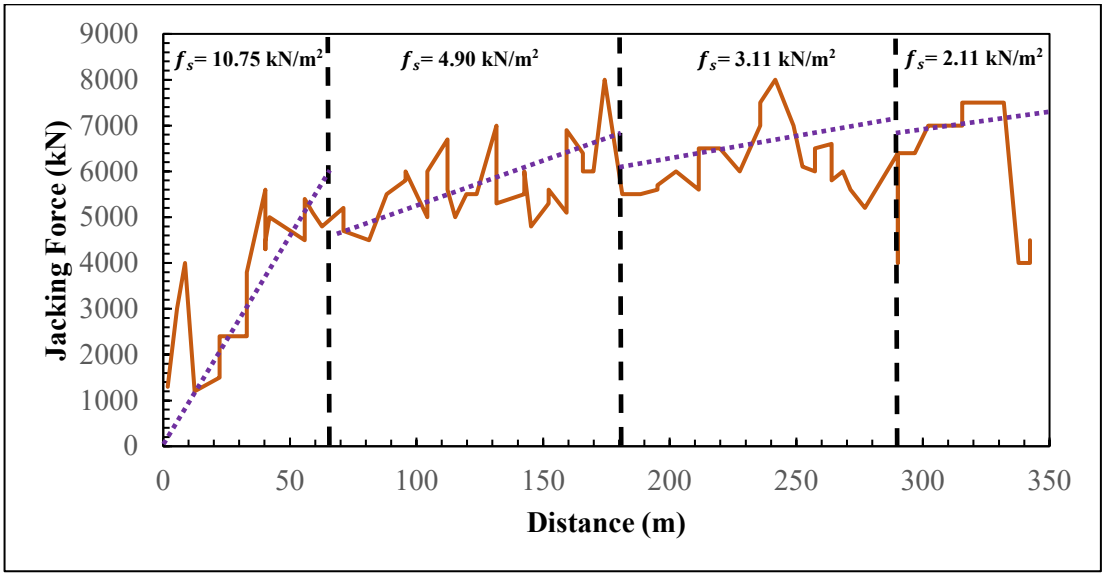

FIGURE 4.13 Jacking force and skin friction

Figure 4.13 further illustrates the TBM attained the highest skin friction value of 10.75 kN/m^2 in the first 60 m from the jacking shaft due to its advancement through a primarily hard limestone. The pattern of the slope was observed when it encountered the limestone located between 60 m to 160 m drive distance, the skin friction decreases to 4.90 kN/m2 and slightly decreased beyond 160 m to 3.11 kN/m^2.

As it drives forward into the limestone formation to reach receiving shaft, the skin friction is likely to continue decreasing as low as 2.11 kN/m^2. One factor that complies with the friction to be lowered is the optimal amount of bentonite lubricant injected through pipe lining holes. It is considered likely that much of the friction on the shield skin and tail seals was often fairly minimal. The average unit skin friction in ground conditions was decreased due to the modification to the slurry circulation system drives thus the skin friction appeared to reduce as rock strength increased.

4.3 Impact of lubrication

Bentonite lubrication system introduced in this project ensures the function of lubricant to be maintained throughout driving length to reduce frictional forces and stabilising layer. The rheological (flow) properties, such as viscosity, yield point and gel strength, vary with bentonite content in the combination depending on the specification of bentonites and the quality of the make-up water. Bentonite-polymer slurry injected in the annular space at sufficient amounts is used to compensate the losses into permeable sand and gravel which leads to the stabilization of the bore. It reduces the friction force around the pipes surface. During operation, a lubricant consisting of water, bentonite and mineral oil was injected into the annulus around the pipes to fill any voids around the pipes hence reduce soil movement. To fill up the annular area, a special lubricant of bentonite slurry with polymer (2%) made especially for the gravel formation is used. To provide consistent lubricant distribution, this particular lubricant was injected in a controlled sequence at required pressure through a confined sequence of outlet in the pipes. An adequate mixing time with high-speed mixer (10 to 12 min/batch) and swelling time of 24 hours is necessary to achieve the required properties.

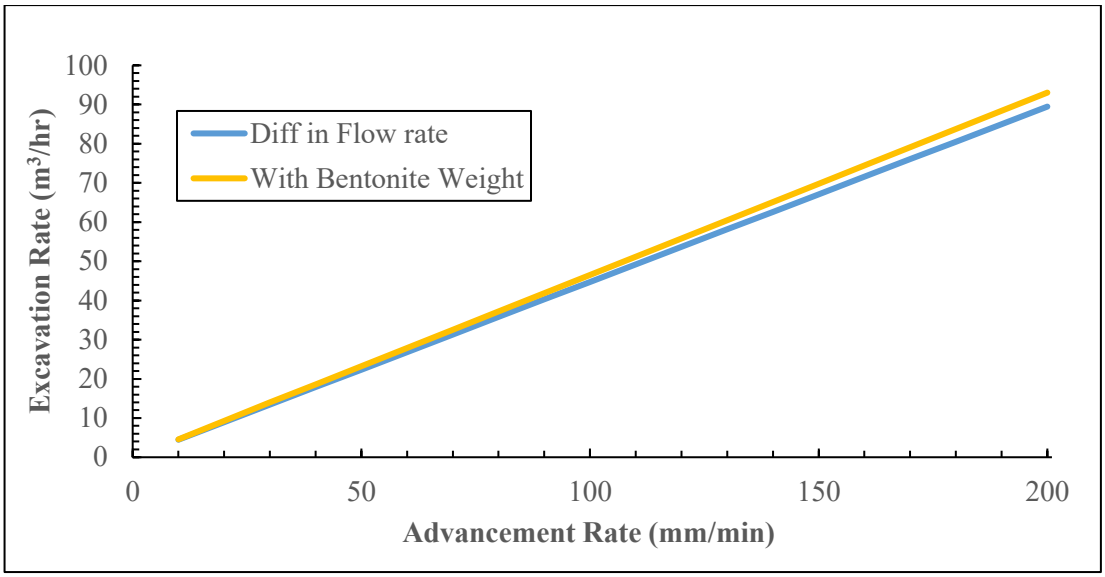

FIGURE 4.14 Estimation of flow rate against advancement rate

During the operation, the impact of the bentonite injected volume in the annular space directly dependent on the friction between the pipe and the soil which directly reducing the friction the pipe and soil. The flow cone testing result is 38 s, indicating that this lubricant's viscosity is sufficient to prevent it from seeping through permeable sand and gravel formation. According to observations, using lubricants can lower frictional stresses by 50% to 90%. The TBM operator determine the required capacity of bentonite density ranging from 1.2 to 1.4 mm/minutes.

4.4 Empirical Estimation of Surface Settlement

Using data gathered during the field monitoring at KM D7.4 along the KL-Seremban Highway, both inbound and outbound, a transversal settlement for CH363 and CH395 were plotted. Seven labelled ground settlement markers are used to examine the ground impact due to tunnelling. To get the best fitted of the measured settlement trough, the value of the maximum settlement, S_{max} and trough width parameter, K are substituted in Equation (2.1).

It was illustrated that the modified Gaussian curve provides a better fit to the data. The theoretical plan included a settlement width in a range of 25 m to 30 m from

the center of the tunnel alignment. To study the transversal settlement, this field data was established as a reference when examining the maximum of surface settlement that were recorded before, during and after passing through two chainages.

4.4.1 Gaussian Distribution of Surface Settlement at CH363

The data in Figures 4.15, 4.16 and 4.17 show that the pattern of surface settlement observed above the tunnel alignment when the TBM was excavating in the limestone formation. Settlement started as soon as the cutterhead passed beneath the monitoring markers. After a few days, there was still some little settlement and the TBM was 60 m away. The tunnel was drawn to illustrate the position of the covered area with tunnel depth of 22 m.

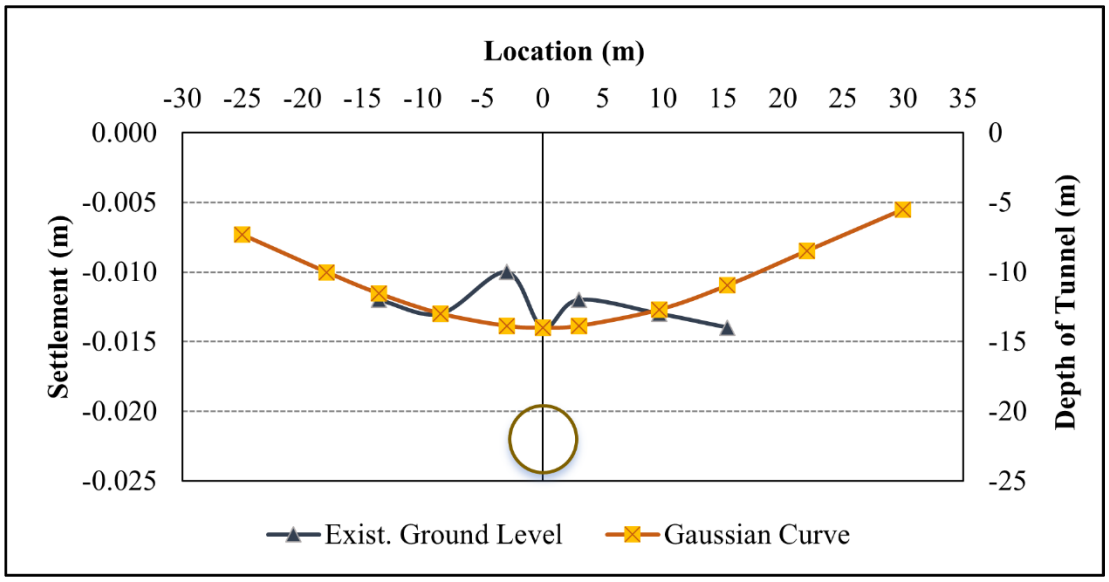

FIGURE 4.15 Gaussian Distribution of ground response at CH363 on 10th Dec 2021

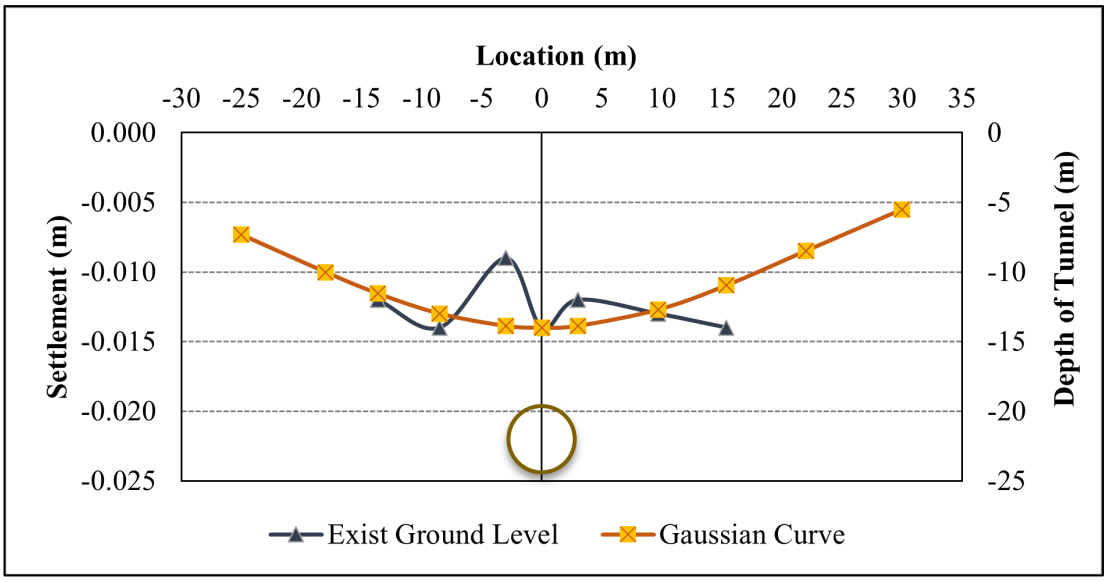

FIGURE 4.16 Gaussian Distribution of ground response at CH363 on 13th Dec 2021

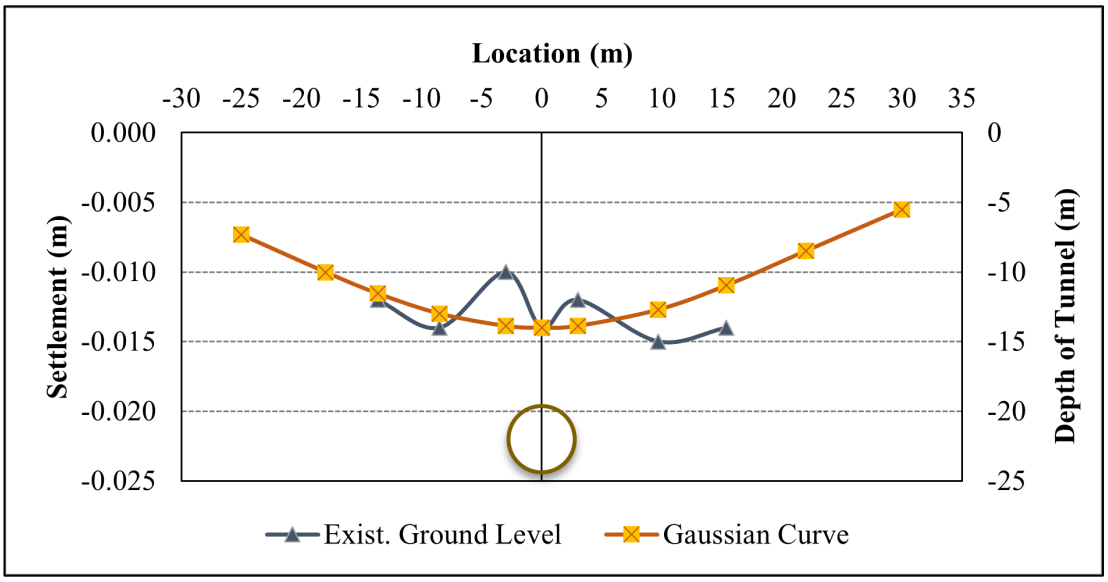

FIGURE 4.17 Gaussian Distribution of ground response at CH363 on 15th Dec 2021

The results of the field measurement data and empirical approaches are illustrated in figure above. The plotted transverse settlement trough for CH363 on 10th, 13th and 15th December 2021, are shown along with a theoretical transverse settlement trough calculated using Equation (2.1). Figure above shows the theoretical Gaussian Distribution accurately estimated the maximum settlement from field monitoring at 14 mm.

Trough width parameter, K is assumed in order to get the best "fit curve" that matches the settlement trough of the field data. It was observed that the settlement trough of the collected field data fits the trough width parameter, $K = 0.5$ and resulted as acceptable fit for the majority of field measurement tunnelling in various ground conditions.

4.4.2 Gaussian Distribution of Surface Settlement at CH395

Figure 4.18, 4.19 and 4.20 presented the pattern of surface settlement observed above the tunnel alignment when the TBM was excavating in the limestone formation. Settlement started as soon as the cutterhead passed beneath the monitoring markers. After a few days, there was still some little settlement and the TBM was 60 m away. The tunnel was drawn to illustrate the position of the covered area with tunnel depth of 22 m.

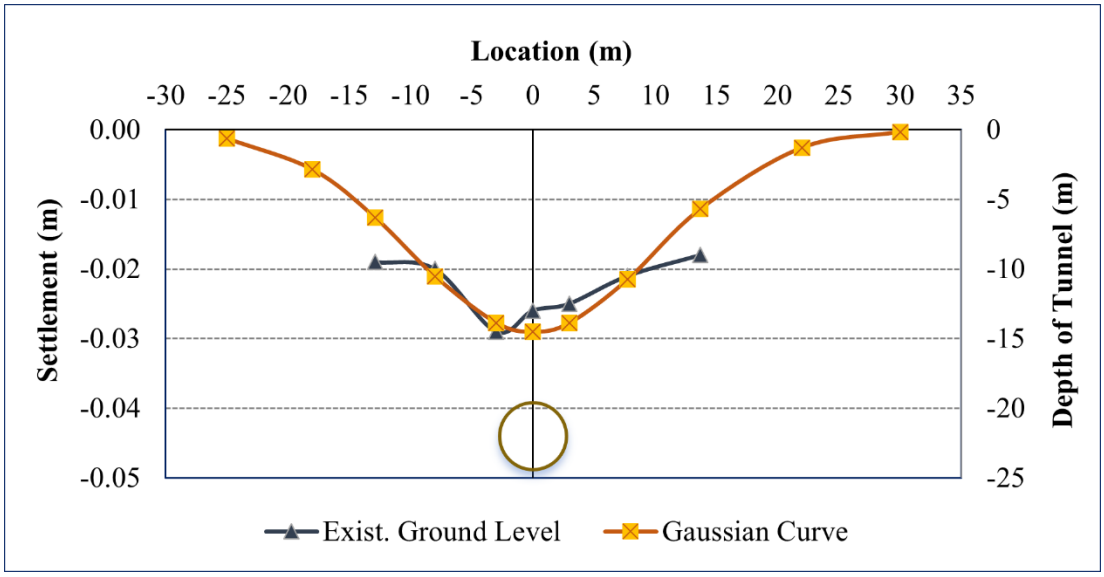

FIGURE 4.18 Gaussian Distribution of ground response at CH395 on 10th Dec 2021

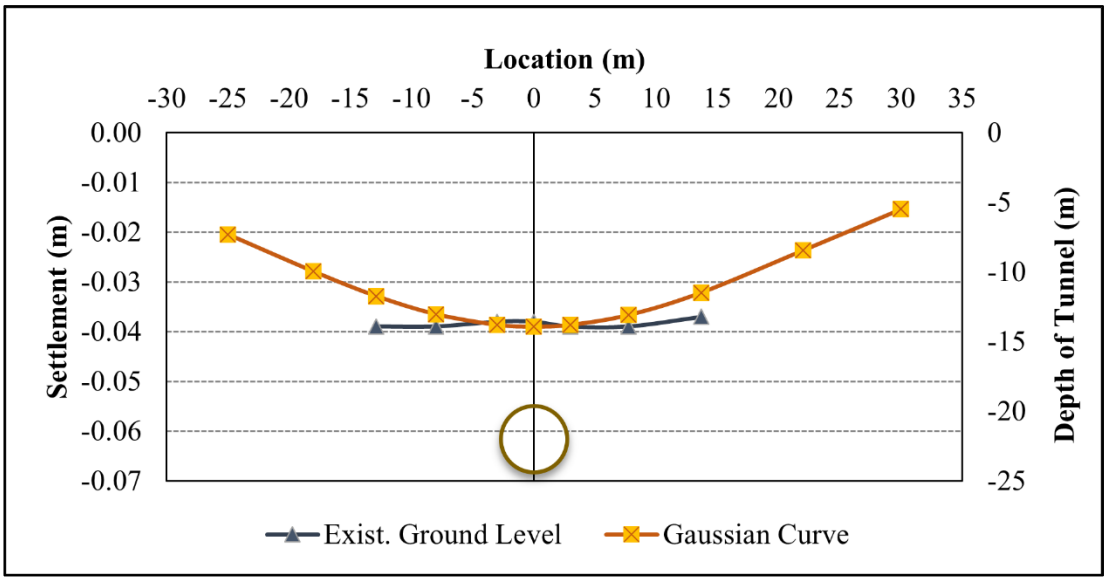

FIGURE 4.19 Gaussian Distribution of ground response at CH395 on 13th Dec 2021

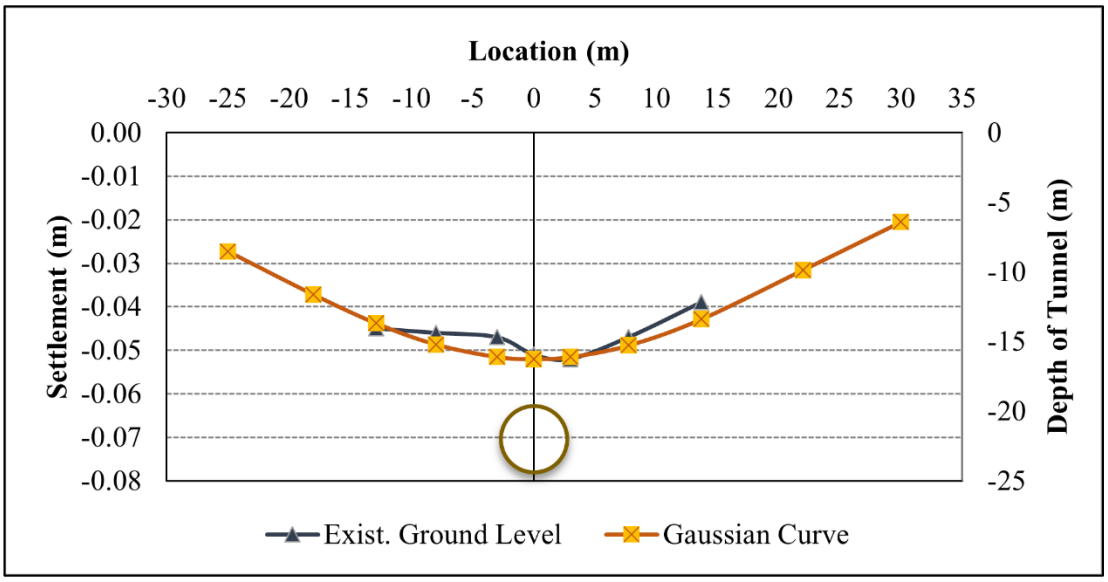

FIGURE 4.20 Gaussian Distribution of ground response at CH395 on 15th Dec 2021

The plotted transverse settlement trough for CH395 on 10th, 13th and 15th December 2021 are well distributed shown in Figure 4.18 to 4.20 along with a theoretical transverse settlement trough calculated using Equation (2.1). Due to its location at the center of the tunnel alignment and higher settlement than the other GSM, MN 4 showed a substantial declination in comparison to the other GSM. Field

observation data indicates that the highest ground settlement was ranging from -18 mm to -56 mm within 39 m to 50 m distance from the jacking shaft, see Figure above.

It should be emphasised that the soil profiles above tunnels often consist of many layers, making it challenging to classify them as either clay or sand. Figure 4.19 depicts a comparison of the magnitude of surface settlement from field monitoring compared to the other two timelines. This is explained by the variation in soil loss volume with depth. As a result of the computation, the broader settlement trough and higher maximum settlement must be considered.

4.4.3 Analysis of Transversal Settlement and Volume Loss

By assuming that the transverse ground settling trough may be represented by a Gaussian Distribution Curve, Peck (1969) suggested an idealisation that has significant mathematical advantages by assuming that the transverse ground settling trough may represent in the form of Gaussian Distribution Curve. The indicative existing ground level gives an idea of the ground surface profile where the settlement is being monitored. The maximum settlement and trough width parameter values from the theoretical transversal settlement plot were utilised to back compute the volume loss at each chainage.

Volume loss (V_L) is calculated as the ratio of the volume of observed settlement trough to the volume of theoretical tunnel lining. The objective of this research is to understand what the actual volume loss for pipe jacking project is by using slurry system in limestone formation. There are few case studies that have been published on the actual ground response associated with pipe-jacking through limestone, however there may be more occurrences in Malaysia without such research being recorded.

TABLE 4.3 Parameters to calculate Volume Loss

Tunnel Type	Reinforced Concrete Jacking Pipe (RCJP)
Tunnel External Diameter	3000 mm
Depth to Tunnel Axis, z_0 (m)	22 m
Trough width parameter, K	0.5
Horizontal distance from tunnel center to the inflection point, *i*	11 m

Table 4.3 shows parameters used to calculate the volume. The best fit curve obtains from Gaussian Distribution Curve (Equation (2.1)) was based on the settlement through a parameter, $K = 0.5$. Consequently, it was employed to determine the volume loss in this study. After calculation of the inflection point, according to the empirical formulas, the maximum settlement obtained from the field data was used in Equation (2.2) to obtain the volume loss express as a percentage fraction. The burial depth of the tunnel was 22 m and the soil loss ratio (i.e., the ratio of the volume of soil loss per unit length of the tunnel to the cross-sectional area of the tunnel) was tabulated in as below.

TABLE 4.4 Volume Loss

Ground Settlement Marker, Chainage	**S_{max} (mm)**		**Volume Loss, %**
	Field data	**Graph Matching**	
CH363	14	14	5.47
CH395	38	39	15.23

Table 4.4 shows the estimated volume loss based on maximum settlement. It is an evident from results of back analysis that the volume loss can be locally much higher when the TBM crossed CH395 with the magnitude of volume loss measured is 15.23% compared to CH363 with 5.47% of volume loss. The cross-sectional area of the tunnel was 7.07 m^2. The volume of the settlement trough per unit length of the tunnel was 0.39 m^3/m at the tunnel depth and 0.14 m^3/m at the ground surface.

Hence, the ground loss was caused by a difference of 14 mm in diameters between the tunnel shield and the concrete pipes and the ground loss ratio was 5.47%. Notably, an empirical approach has taken to determining the subsidence that might occur over the tunnel openings, and calculated values have been compared with the observed values. It is generally accepted that the surface settlement profile above a tunnel in limestone can be presented by a Gaussian distribution curve as well.

4.4.4 Longitudinal Settlement

Throughout the entire influence zone of the tunnelling works, settlement began to occur upon the passage of the cutterhead below the monitoring markers. The magnitude of ground movement was obtained from monitoring the GSM labelled MN4 installed at each chainage of tunnel alignment. Surface settlement began approximately 4D away from the reference point as shown in Figure 4.21 and 4.22 with the development of the longitudinal settlement trough along a 40 m-long section of the tunnel.

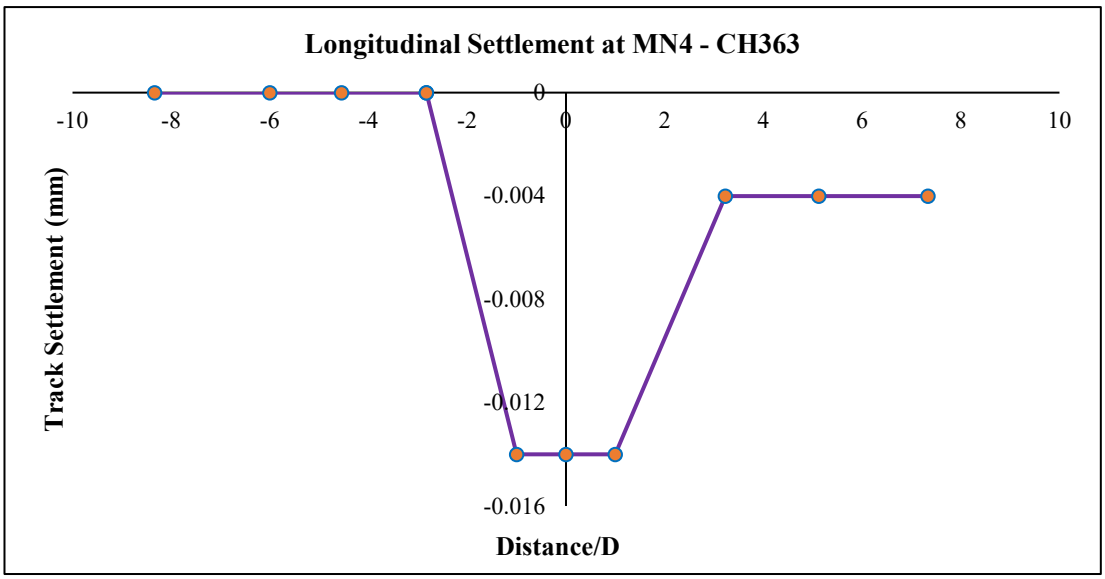

FIGURE 4.21 Longitudinal settlement at CH363

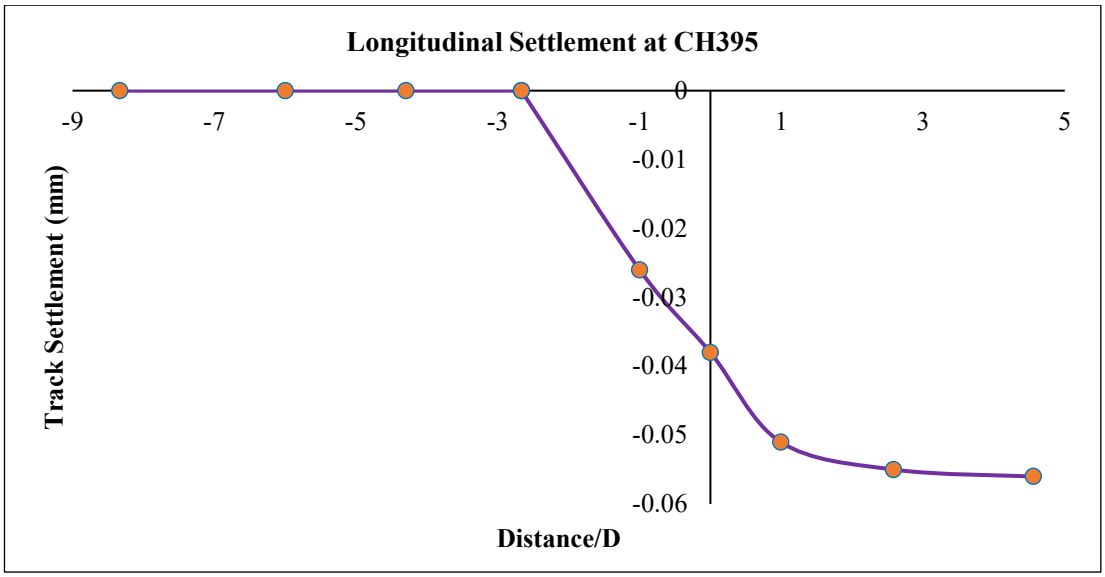

FIGURE 4.22 Longitudinal Settlement at CH395

Figure above shows the longitudinal settlement trough plotted against drive distance. Negative drive distance over *D* indicates that the location of tunnel before reaching the marker whilst positive drive distance over *D* indicates the TBM passing through the marker. Prior to the TBM passing through the monitoring markers, a sudden settlement occurred at CH363 once the excavation face about 2D away before the reference point. However, the heaving can be observed on Day-20 when the TBM passed through under CH363 thus indicates that pipe jacking caused an upward soil movement. The surface settlements started to stabilize when the excavation face was located approximately 3D away from the reference point. On the other hand, the passing of the tail shield and the construction of linings were followed by more than 60% of the overall settlement occurred at CH395. A maximum settlement recorded in front advancing tunnel is 50% from total settlement.

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

A thorough study of the various aspects associated with the tunnelling technique is presented for this project. The overall purpose of this research study was to analyse the surface settlement caused by pipe jacking works crossing KL-Seremban Highway. This research had successfully achieved the first objective which to correlate the TBM parameter and ground condition due to huge excavation induced by tunnelling activity where the relative importance of the various parameters depends on the most likely ground conditions for TBM advance. The tunnelling in the limestone was characterized by low values for contact force particularly when tunnelling in the limestone. The result of encountering soil removal rate should be coordinated in order to maintain the required face pressure at the front of the shield machine which can be affected by geological conditions.

Accordingly, this research also successfully achieved the second objective which is to produce the best fitted Gaussian curve to the ground surface settlement recorded above the center-line of the alignment by applying empirical method. Based on the findings, the results from this project are similar in terms of the validation of the empirical method that may influence the surface settlement and volume loss associated with the tunnelling method. This study has shown that the volume loss caused by tunnelling through limestone bearing layers would probably exceed the under full-face soft ground conditions. The validated trough width parameter, K was 0.5 and the tunnel

volume loss observed about CH363 and CH395 was 5.47% and 15.23% , respectively. Concurrently, the volume loss (V_L) can be considerable as compared to the volume loss reported by previous researcher using comparable tunnelling techniques.

5.2 Recommendation

While it is widely accepted to adopt the Gaussian settlement curve for describing tunnel induced settlements, there are few recommendations that can be implemented in future study to improve this research findings. Field monitoring data from numerous contractors will be made available as Malaysian tunnelling projects increase. To validate and improve these findings, particularly in the Kuala Lumpur area, real-time surface settlement monitoring during the crucial tunnelling operation should be done in a sparsely populated area in order to increase the collection of TBM advancement database that are focus on the effects of TBM driving parameters on volume loss. The tunnelling impact on surrounding ground is predominantly concerned by increasing the number of markers to measure wider transverse settlement trough as well as improving the TBM parameters based on instrumentation results. In addition, close instrumentation monitoring is necessary when excavating underneath sensitive structures along the tunnel alignment.

For the purpose of impact assessment on sensitive facilities are situated above soil-rock interfaces along the alignment, a considerable high-volume loss than what is often attainable should be employed to the estimate ground settlement. Active monitoring should be maintained until the ground settlement has stabilised. In order to verify the assumptions established on the settlement trough profile, it is preferable to measure wider transverse settlement trough of a tunnel drive. Monitoring only along a tunnel's center line would not provide sufficient information to confidently verify the magnitude of the tunnelling influence zone. Therefore, lengthier ground surface monitoring should be carried out, well after tunnel construction has been completed.

As an increasing number of reviews of the history and development of pipe jacking and microtunnelling, the wider the research in pipe jacking projects at other soil conditions and different type of tunnel boring machine. For instance, ground settlements should be anticipated despite of tunnel excavation method and safety precautions against damage to existing facilities prior to tunnelling construction. Otherwise, due to damage to structures caused by surface settlement that exceeds allowable limits, project schedule and tunnelling costs will dramatically increase. Therefore, other methods of analytical and numerical approaches have been developed to evaluate potential settlements and impact on structures. By assembling data on drives in various ground conditions, there is the possibility for improved knowledge of the pipe-jacking process.

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