การประเมินการก่อสร้างอุโมงค์ลอดใต้อุโมงค์รถไฟฟ้าใต้ดิน
ที่มีอยู่เดิมในชั้นดินกรุงเทพฯ

นายกุลภัทร์ พิสิษฐ์กุล

วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิศวกรรมศาสตรดุษฎีบัณฑิต
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ASSESSMENT OF TUNNELLING UNDER EXISTING MRT TUNNEL IN BANGKOK SUBSOIL

Mr. Kullapat Phisitkul

A Dissertation Submitted in Partial Fulfillment of the Requirements
for the Degree of Doctor of Philosophy Program in Civil Engineering
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By	Mr. Kullapat Phisitkul
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Thesis Advisor	Associate Professor Wanchai Teparaksa, D.Eng.

Accepted by the Faculty of Engineering, Chulalongkorn University in Partial Fulfillment of the Requirements for the Doctoral Degree

....Dean of the Faculty of Engineering
(Associate Professor Boonsom Lerdhirunwong, Dr.Ing.)

THESIS COMMITTEE

.... Chairman
(Associate Professor Boonsom Lerdhirunwong, Dr.Ing.)

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.... Thesis Advisor
(Associate Professor Wanchai Teparaksa, D.Eng.)

.... Examiner
(Associate Professor Tirawat Boonyatee, D.Eng.)

.... External Examiner
(Associate Professor Korchoke Chantawarangul, Ph.D.)

Pornpot Tanseng
.... External Examiner
(Assistant Professor Pornpot Tanseng, Ph.D.)

กุลภัทร์ พิสิษฐ์กุล : การประเมินการก่อสร้างอุโมงค์ลอดใต้อุโมงค์รถไฟฟ้าใต้ดินที่มีอยู่เดิมในชั้นดินกรุงเทพฯ. (ASSESSMENT OF TUNNELLING UNDER EXISTING MRT TUNNEL IN BANGKOK SUBSOIL) อ. ที่ปรึกษาวิทยานิพนธ์หลัก : รศ. ดร.วันชัย เทพรักษ์, 106หน้า.

การก่อสร้างอุโมงค์แห่งใหม่ลอดใต้อุโมงค์ที่มีอยู่เดิมในเขตกรุงเทพฯ มีแนวโน้มเพิ่มมากขึ้นในปัจจุบัน การตรวจวัดพฤติกรรมการเคลื่อนตัวของอุโมงค์ที่มีอยู่เดิมในอดีตนั้นไม่สามารถกระทำได้ เนื่องจากอุโมงค์เหล่านั้นเป็นอุโมงค์ส่งน้ำของการประปานครหลวง งานวิจัยนี้ศึกษาการประเมินการก่อสร้างอุโมงค์ระบายน้ำของกรุงเทพมหานครจากบึงมักกะสันไปยังสถานีสูบน้ำที่วัดช่องลม ซึ่งส่วนหนึ่งของโครงการจำเป็นต้องทำการขุดเจาะอุโมงค์เข้าใกล้เสาเข็มทางด่วนขั้นที่ 1 และลอดใต้อุโมงค์รถไฟฟ้าใต้ดินที่มีอยู่เดิม การตรวจวัดในสนามพบว่าขอบเขตการทรุดตัวของผิวดินมีระยะประมาณสองเท่าของความลึกอุโมงค์โดยวัดจากบริเวณหน้าเครื่องขุดเจาะไปทั้งด้านหน้า และด้านหลังของเครื่องขุดเจาะ ปริมาณการทรุดตัวส่วนใหญ่จะเกิดขึ้นในระยะสั้น และการทรุดตัวจะเริ่มคงที่ในระยะเวลาสองเดือน การศึกษายังพบว่าการเปลี่ยนแปลงของแรงดันน้ำส่วนเกินระหว่างเม็ดดินจะมีการเปลี่ยนแปลงในแนวราบเป็นหลัก โดยมีขอบเขตจำกัดอยู่เพียงหนึ่งเท่าของเส้นผ่าศูนย์กลางเครื่องขุดเจาะอุโมงค์ในแนวดิ่ง และสองเท่าของเส้นผ่าศูนย์กลางเครื่องขุดเจาะอุโมงค์ในแนวราบ นอกจากนี้แรงดันน้ำส่วนเกินดังกล่าวยังลดลงจนกลับสู่ภาวะปกติในระยะเวลาอันสั้นเช่นเดียวกับการเคลื่อนตัวของดินโดยรอบ การศึกษายังแสดงให้เห็นว่าการตั้งแนวตรวจวัดในโครงการสามารถช่วยให้การวิเคราะห์ทางวิศวกรรมมีความถูกต้องมากขึ้น และยังช่วยลดความเสี่ยงในการก่อสร้างลงได้ ยิ่งไปกว่านั้นผู้ควบคุมเครื่องขุดเจาะอุโมงค์ยังสามารถใช้เกณฑ์การควบคุมเครื่องจักรที่ตั้งไว้เพื่อรักษาระดับการทรุดตัวของดินให้อยู่ในเงื่อนไขการก่อสร้างได้อย่างเหมาะสม บันทึกการขุดเจาะอุโมงค์ยังแสดงให้เห็นว่าการควบคุมแรงดันบริเวณหน้าเครื่องขุดเจาะ และอัตราส่วนการอัดฉีดน้ำปูนมีความสำคัญในการลดการเคลื่อนตัวของดินรอบ ๆ และทำให้การขุดเจาะอุโมงค์ใกล้สิ่งก่อสร้างใต้ดินที่มีอยู่เดิมสามารถกระทำได้โดยมีผลกระทบต่อโครงสร้างนั้น ๆ น้อยมาก

ภาควิชา..วิศวกรรมโยธา..........................ลายมือชื่อนิสิต..

สาขาวิชา..วิศวกรรมโยธา.........................ลายมือชื่อ อ.ที่ปรึกษาวิทยานิพนธ์หลัก.......................

ปีการศึกษา...2552..............................

4871835221 : MAJOR CIVIL ENGINEERING
KEYWORDS : INSTRUMENT / TBM / TUNNELLING / PREDICTION / BANGKOK CLAY / ASSESSMENT

KULLAPAT PHISITKUL : ASSESSMENT OF TUNNELLING UNDER EXISTING MRT TUNNEL IN BANGKOK SUBSOIL. THESIS ADVISOR : ASSOC. PROF. WANCHAI TEPARAKSA, D.Eng., 106 pp.

Tunnelling under existing tunnels in Bangkok has been undertaken in recent years. Unfortunately, monitoring in the vacant tunnel is inapplicable as most tunnels are constructed underneath the Metropolitan Water Tunnel Network. Hence, close investigation of any deformation has not been carried out and the deformation of existing tunnels in Bangkok geology is unknown. This research studies assessment of the Bangkok Metropolitan Admission (BMA) water diversion from Makasan Reservoir to Wat Chonglom pumping station. The tunnel is close to pile foundation of the expressway and also tunnelling under existing MRT subway tunnel in Bangkok area. Field monitoring shows that boundary of surface settlement is about two times of tunnel depth both in front and behind the TBM face. Most of the tunnel deformation and ground settlement, about 90%, has occurred over a short term period and is likely to achieve full displacement within 2 months. Fluctuation of excess pore-water pressure surprisingly changes only in the horizontal plane. The influence zone is limited to about one time of TBM diameter in vertical axis and limited to about two times of TBM diameter in horizontal axis. Moreover, similar to ground movement, most excess pressure is also fully dissipated over a short term period. The study also shows that setting up the test sections can provide validation of numerical analysis and prevent possible risk during construction. The TBM operator could use suggested TBM parameters as a guideline for ease of operation, while maintaining settlement in the construction criteria. Tunnel records show that additional face pressure and grout filling ratios could minimize ground movement and displacement of the surrounding underground tunnel. Tunnelling close to existing underground tunnels is applicable and has less effect on the structure.

Department : Civil Engineering	Student's Signature ....
Field of Study : Civil Engineering	Advisor's Signature Wanchai Tepaal
Academic Year : 2009	

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LIST OF SYMBOLS

C_{inc}	Increment cohesion
D	Outside diameter of tunnel
E_u	Undrained Young's Modulus
E'	Effective Young's Modulus
E_{inc}	Increment Young's Modulus
E_{ref}	Young's Modulus at reference point
i	Distance from the center line to the inflection point (m)
K	Constant value of surrounding soil
K_0'	Effective coefficient of earth pressure at rest
P_0	Total stress in the front of TBM
P_i	Face Pressure
P_w	Water pressure at springline level
R, r_0	Outside radius of tunnel
S	Settlement at distance x from the center line of tunnel (m)
S_{max}	Maximum settlement at center line of tunnel (mm)
S_u	Undrained Shear Strength
SPT-N	SPT-N value
V_{ex}	Volume of excavated earth per linear length of tunnel axis
V_L	Volume of ground loss in to tunnel
V_s	Volume of surface settlement trough
ΔV	Volume change in soil
w_s	Width of settlement

x	Lateral distance at point of interest (m)
Z	Depth of interest point
Z_0	Depth of springline level from ground surface
β	Angle of trough
σ_v'	Vertical effective stress
η	Excavation ratio
δ	Settlement value

CHAPTER I
INTRODUCTION

1.1 Statement and Significant of Problem

Tunneling is relatively new challenge for geotechnical engineering in Thailand. Since last decade, only a few tunnels have been constructed in Thailand, mostly in Great Bangkok Metropolitan area. Some examples of important project are water tunnel network of Metropolitan Water Authority (MWA), the first mass rapid transit (MRT) system of MRT Authority and lately, water diversion tunnels of Bangkok Metropolitan Authority (BMA). The construction of water diversion tunnel is a part of solution to prevent flooding in Bangkok area. The BMA has launched a few projects since last decade, to provide short diversion route for excess stormwater that accumulate in the inner part of the city to flow out to Chao Praya River. First two projects were constructed underneath water supply tunnel and bridge foundation, while the third project will be constructed, first time, underneath the MRT tunnel. The MRTA has scheduled to monitor the tunnel regularly however; tunnel construction period in the critical zone underneath the MRT tunnel is considerably short, only about 2 days, which may not sufficient to solve any problem. Inspection period is also limited and therefore, any risk is unacceptable. The main contractor shall then provide strong evidence, in order to prove that construction process is optimal, before getting approval from MRTA and BMA.

Tunnelling under existing tunnels in Bangkok has been undertaken in recent years. However, monitoring in the vacant tunnel is inapplicable as most tunnels are constructed for the metropolitan water network. Hence, close investigation of any deformation has not been carried out and the deformation of existing tunnels in Bangkok geology is unknown. Only ground deformation directly around water supply tunnel was monitored in previous projects. Moreover, the city is developing, so the underground space will become smaller. Therefore, tunnel construction may not be able to avoid such close interaction with existing underground structure in the near future. This thesis presents a study of the deformation of the MRT subway tunnel during the construction of new underneath diversion tunnels. Lining deformation of the existing MRT subway tunnel is discussed in connection with the performance of TBM operation, including face pressure, advance rate, grout pressure and volume loss. In addition, ground movement and changing pore-water pressure around the existing tunnel are also covered.

1.2 Objectives of the Study

1. To examine a prediction process and appropriate tunnelling parameters for tunnelling in Bangkok clay, where the tunnel boring machine (TBM) has to pass underneath existing tunnels.
2. The behaviour of the surrounding ground, porewater pressure and existing structures, such as bored pile and MRTA tunnel, are also investigated in order to provide more comprehensive information for analysis.

1.3 Scope of the Study

The scope of the study is to assess a prediction process of tunnelling under existing MRT tunnel in Bangkok subsoil. Validation of adopted parameters used for prediction process is necessary and hence, monitoring of related information such as tunnelling parameters, displacement of surrounding soil and underground structure as well as changing of porewater pressure around construction proximity cannot be ignored. The information will be collected from Water Diversion Tunnel Project of BMA from Makkasan Reservoir to Wat Chong Lom, which is constructed by Ch.Karnchang Public Company Limited. Figure 1.1 presents the tunnel alignment from Makkasan to Wat Chong Lom.

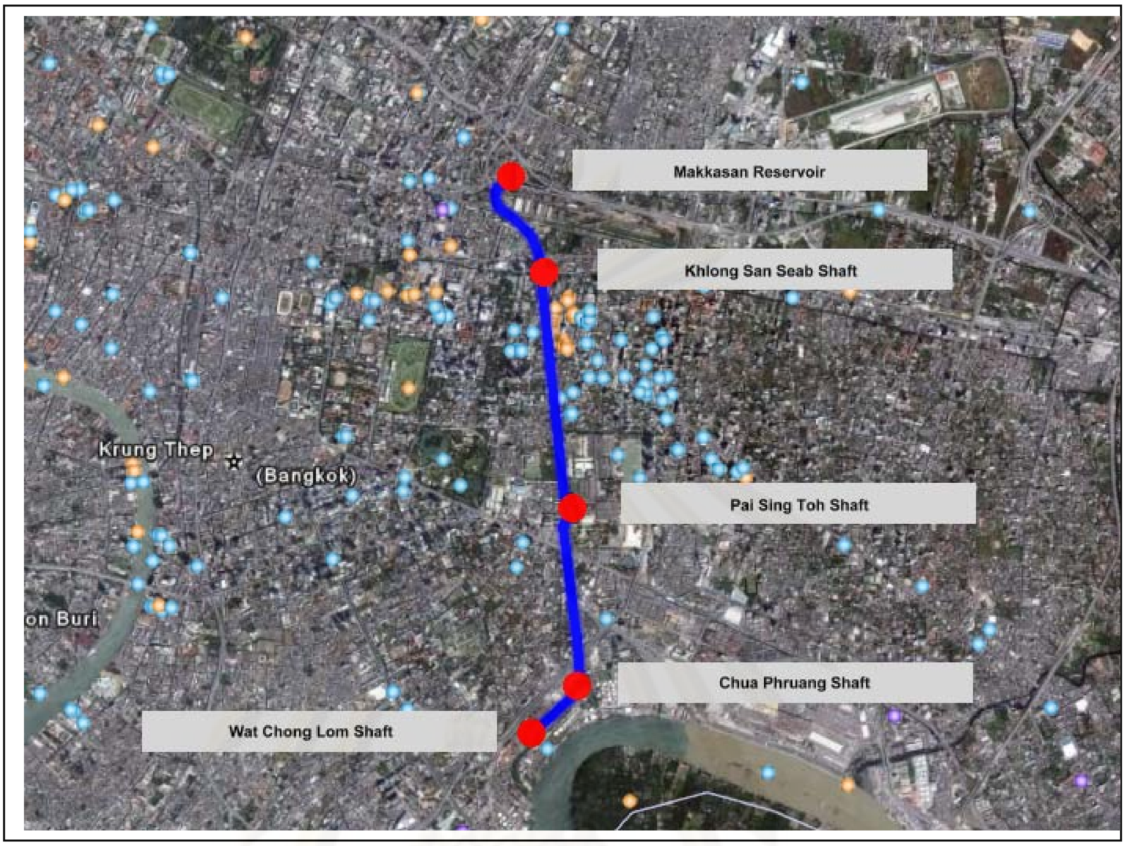


Figure 1.1 Path of diversion tunnel (MW – diversion tunnel)
between Makkasan and Wat Chong Lom Pumping Station

1.4 Benefit of the Study

1. Provide appropriate prediction approach for tunnelling work closed to underground structure.
2. Understand general behaviour of underground tunnel due to ground movement caused by new tunneling.
3. Understand behaviour of ground movement and changing of porewater pressure when tunnelling near underground obstruction.

CHAPTER II
LITERATURE REVIEW

2.1 Estimation of Ground Settlement due to Tunnelling

In present, tunnelling in Bangkok Metropolitan area has been increased and therefore, unlikely to avoid tunnel excavation in dense underground structure area. Unfortunately, surface settlement is normally occurred during the construction in soil strata. Although, the tunnelling technology has improved, the consecutive settlement is still large due to consolidation of disturbed ground around the tunnel (Hashimoto et al., 1999). In general, a trial and error process of tunnel operation was carried out on site to control ground deformation and construction performance (Teparaksa, 2002, 2005a). Hence, tunnelling procedure, information management, damage assessment as well as monitoring pattern are very important in studying and determination of behavior and displacement of underground structure and surrounding ground.

2.2 Determination of Settlement at Ground Surface

Study of Schmidt and Peck in 1969 shows that excavation of tunnel will cause surrounding ground, especially in front and above the tunnel, to move inward. The pattern of settlement can be presented with the normal probability curve or Gaussian distribution as shown in Figure 2. The maximum displacement will be occurred at the centerline of the tunnel. The vertical displacement at distance x from centerline of the tunnel can be estimated from the following equation.

$$S = S_{\max} \exp\left(\frac{-x^2}{2i^2}\right)$$

where S Settlement at distance x from the center line of tunnel (m)

S_{max} Maximum settlement at center line of tunnel (mm)

x Lateral distance at point of interest (m)

i Distance from the center line to the inflection point (m)

According to the above equation, the maximum settlement point will be at the center line or x equals to zero. Distance to the inflection point can be determined from the Figure 2.1.

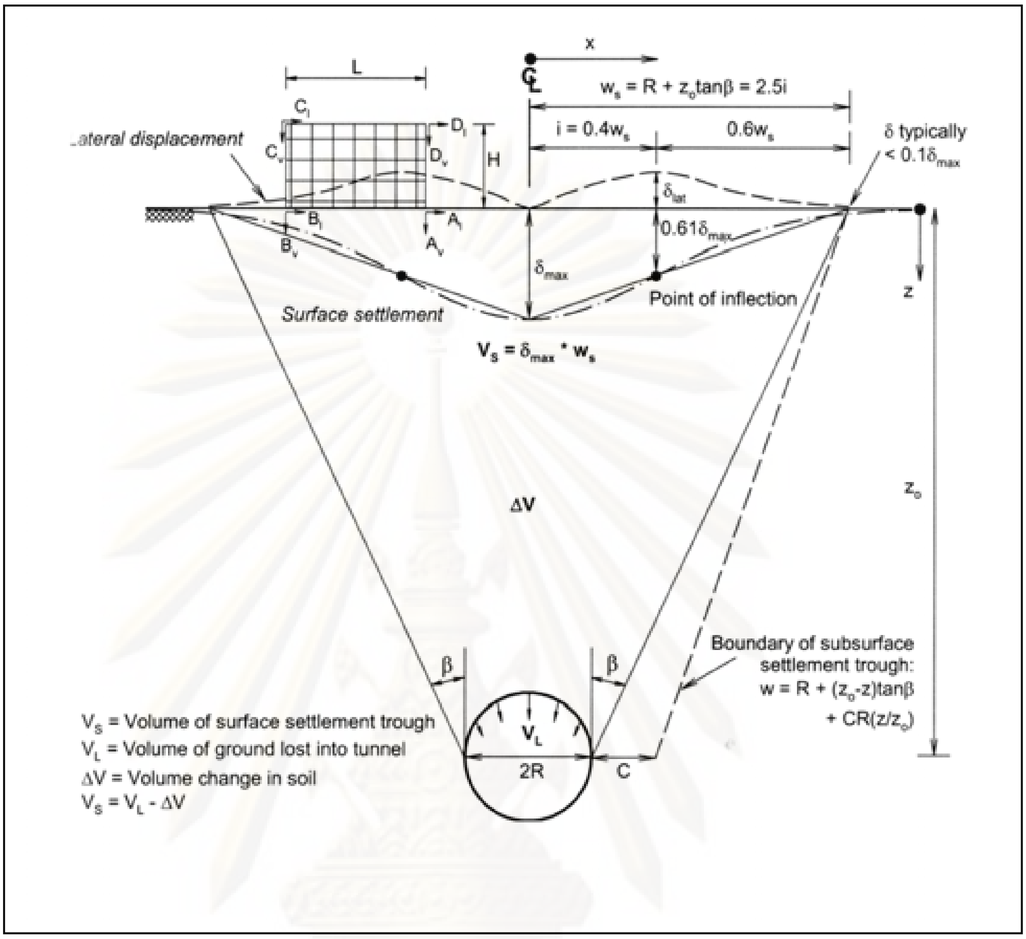


Figure 2.1 Movement of ground surface due to tunnelling

2.3 Determination of i value

Peck (1969) and O'Reilly & New (1982) have proposed a method to estimate i value and to calculate ground settlement in tunnelling work. The method is widely used in engineering sector as the initial value of calculation.

2.3.1 Determination of i-value by Peck method (1969)

Peck (1969) has proposed relationship between reflection distance/outside diameter ratio (i/R) and springline depth/tunnel diameter ratio (Z_0/D) to estimate i-value, based on several settlement information of tunnelling in various ground condition.

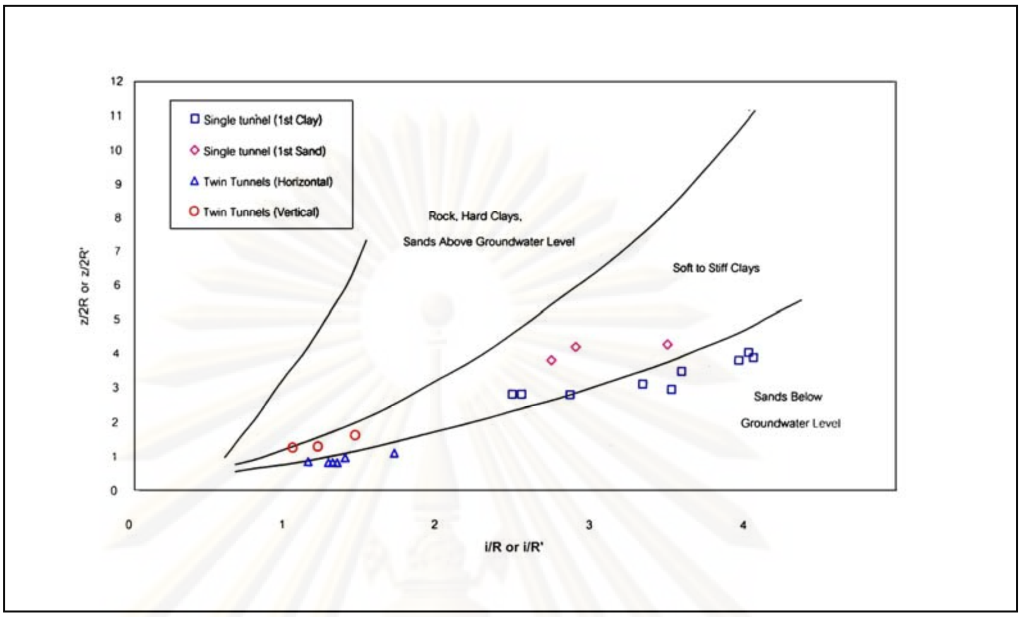


Figure 2.2 The relationship between depth per tunnel diameter and trough width per tunnel outer radius (adapted from Chanchaya, 2000)

As shown in Figure 2.2, the distance to inflection point (i) is also varied, based on surrounding ground condition, i.e. the i-value is increased when tunnelling in soft ground and decreased when tunnelling in stiff ground. Chanchaya (2000) shows that relationship, between depth per tunnel diameter and trough width per tunnel outer radius, of Bangkok clay tends to follow the boundary between stiff clay and sand.

2.3.2 Determination of i-value by O'Reilly & New method (1982)

O'Reilly & New (1982) has found linear relationship between distance of inflection point (i) and depth of springline level from site monitoring in the field. The relationship can be categorized into two types e.g. (1) tunnelling in cohesive soil and (2) tunnelling in cohesiveless soil, as presented in Figure 2.3.

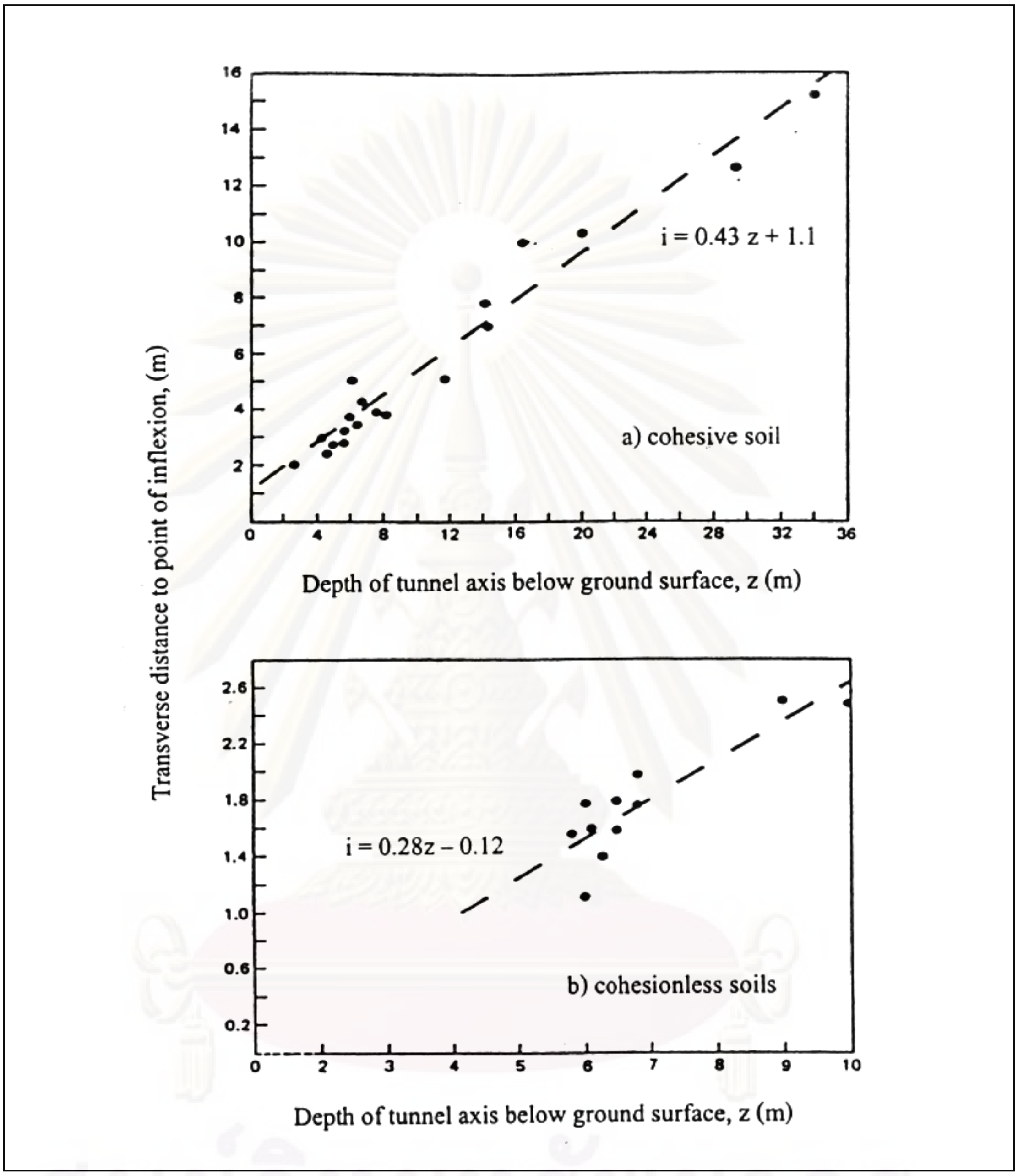


Figure 2.3 Relationship between distance of inflection point (i) and depth of tunnel axis (O'Reilly and New, 1982)

Moreover, O'Reilly & New (1982) has also proposed the method of i-value estimation from the following equation, where K is constant of surrounding soil.

$$i = KZ_0$$

where K = 0.50 for tunnelling in cohesive soil

K = 0.25 for tunnelling in cohesiveless soil

Z_0 depth of tunnel axis from ground surface (m)

2.3.3 Determination of i-value and K-value in Bangkok ground condition

In recent years, various studies are carried out in Bangkok tunnelling projects, in order to estimate i-value and K-value based on Peck (1969) and O'Reilly & New (1982) method, respectively.

Charnchaya (2000) studies the movement of ground during MRT construction in Bangkok according to Peck (1969) method and has found that i-values are about 8-13m and 9-19m for single and double tunnel, respectively. Moreover, the study shows that K-values in cohesive soil, according to O'Reilly & New (1982) method, are about 0.45-0.55 and 0.70-0.80 for single and double tunnel, respectively. Furthermore, the study also indicates that K-values in cohesiveless soil are about 0.35-0.40 and 0.42-0.48 for single and double tunnel, respectively.

In 2001, Pitaksaithong also studies the movement of ground surface during construction of MRT tunnel and Premprachakorn water diversion tunnel projects, based on Peck and O'Reilly methods. The study shows that i-value is about 8-12m and K-value is about 0.34-0.50. Moreover, the study also mentions that Mair, et al. (1993) method has underestimated the movement of ground in Bangkok area.

2.4 Determination of ground loss and maximum ground movement at the surface

Settlement of ground in tunnelling work may definite in term of ground loss or volume loss (V_L) which is the portion between volume of settlement at the ground surface and volume of excavation in the tunnel.

$$V_L = \frac{V_S}{Vex}$$

where V_s volume of settlement at ground surface per linear length of tunnel axis

V_{ex} volume of excavated earth per linear length of tunnel axis

The value of settlement volume (V_s) can be determined by integration of Gaussian distribution curve or estimated from triangle area, based on Cording and Hansmire (1975).

In later method, the height of triangle is assumed to be equal to maximum settlement value (δ_{max}) at the center line of tunnel, while the base of triangle is equals to boundary of settlement ($2w_s$), as shown in Figure 2.1. Therefore, a half of settlement boundary is equals to 2.5 times of i-value and can be written as follow

$$w_S = 2.5i = \frac{V_S}{\delta_{\max}}$$

When D is diameter of tunnel, volume of excavated material is then equals to volume of TBM per linear length of tunnel axis. Therefore, volume loss (V_L) is

$$V_L = \frac{4V_S}{\pi D^2}$$

When substitute V_s into above equation, maximum settlement (S_{max}) is

$$S_{\max} = \frac{0.314 V_L D^2}{i}$$

$$S_{\max} = \frac{0.314 V_L D^2}{K Z_0}$$

Hence, V_L, i and K-value are depend upon various factors such as geological profile, equipment and construction performance. O'Reilly & New (1982) also found that ground loss (V_L) is about 1-4 % for tunnelling in cohesive soil.

Moreover, the equation also presents that magnitude of settlement is increased when diameter of tunnel (D) increases but magnitude of settlement is decreased when tunnelling depth (Z_0) increases. In the other word, large tunnel construction will create larger deformation than small tunnel construction at the same depth. However, tunnel construction in deep layer will generate less ground settlement than construction in shallow layer.

2.5 Ground loss and quality assessment of construction

Volume of ground loss due to tunnel construction can also be used to assess the quality of the construction process. O'Reilly & New (1982) has proposed relationship between volume loss and tunnelling quality as shown in Table 2.1.

Table 2.1 Relationship between ground loss and construction quality

Case	V_L(%)
Good practice in firm ground	0.5
Good practice in medium ground	1.5
Fair practice in soft ground	2.5
Poor practice in soft clay	4.0 or more

In general, most construction sites have setup monitoring system of ground movement to investigate volume loss regularly and also used to control the quality of tunnelling work. For example, ground loss and ground settlement tend to increase when performance of construction is decreases.

Moreover, ground loss also relates to both advancing rate of excavation and rate of muck extrusion which is varied on ground condition. Hence, TBM operator is normally uses surveying information to adjust advancing rate and face pressure of the machine in the beginning of construction and/or different ground condition. Furthermore, monitoring result can also be used for prediction purpose where the TBM has to pass near critical area. The monitoring results add value to the prediction (Bakker et al., 1999).

2.6 Determination of ground movement at various depths

In order to obtain information of ground movement at various depths, engineer requires installing at least 3 series of extensometer along the cross section of tunnel axis. The installation and monitoring expense are quite expensive and therefore, less information is available. Moreover, most information is collected from tunnelling project in clay layer.

In 1993, Mair studied information of ground movement in British and had found that ground movement at any particular depth has similar pattern as found at ground surface.

However, width of settlement trough is decreases with the depth. Moreover, vertical movement at the center line is higher than the movement at the ground surface. Mair, et al. (1993) has proposed the following equation to calculate the maximum settlement at depth z.

$$S_{\max,z} = \frac{0.314 V_L D^2}{K(Z_0 - Z)}$$

where V_L Ground loss at the surface (m)

D Diameter of tunnel (m)

Z Depth from ground surface to point of interest (m)

Z_0 Depth from ground surface to center of tunnel (m)

K Constant value derived from Figure 2.4

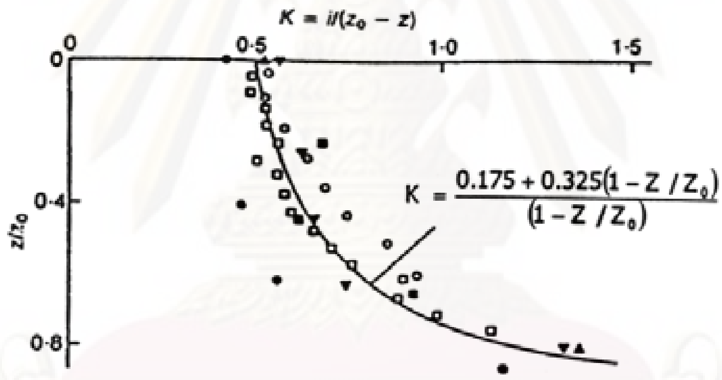


Figure 2.4 Relationships between K-value and ratio of depth of interest per depth of tunnel (Mair, et al., 1993)

However, Pitaksaithong (2001) says that actual settlement, which was monitored from several tunnel projects in Bangkok, is actually 10-40% higher than prediction based on study of Mair. Therefore, Pitaksaithong has proposed new relationship between maximum settlement at depth (z), radius of tunnel (r_0) and springline level (z_0) as shown in Figure 2.5.

Subsurface settlement above tunnel center line in stiff Bangkok Clay

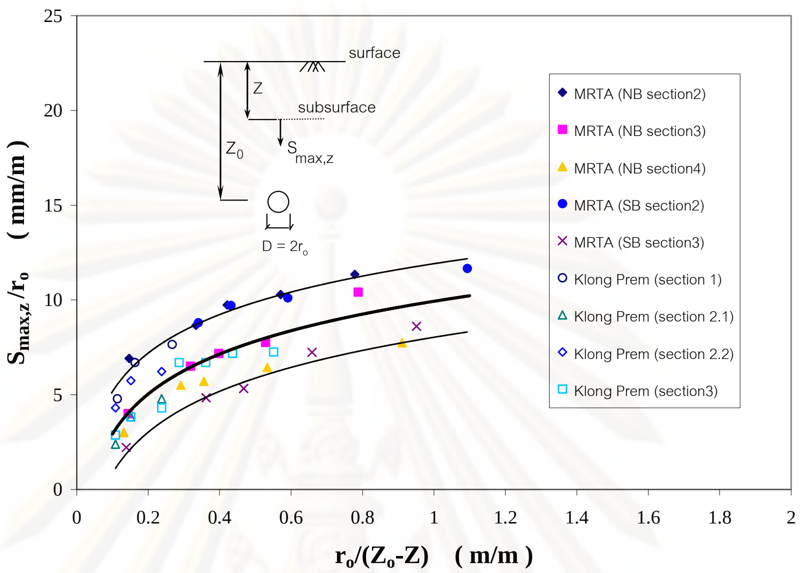


Figure 2.5 Relationship between maximum settlement, depth of springline and radius of tunnel (Pitaksaithong, 2001)

According to Figure 2.5, Pitaksaithong has also proposed the following equation to determine the maximum vertical movement at various depths.

$$\frac{S_{max,z}}{r_o} = 3.04\ln\left(\frac{r_o}{Z_o - Z}\right) + 9.93$$

where $S_{max,z}$ Maximum settlement at depth z

r_0 Radius of tunnel (m)

Z Depth from ground surface to point of interest (m)

Z_0 Depth from ground surface to center of tunnel (m)

However, the above equation is limited to 4-6.5m dia. tunnel which is constructed in stiff Bangkok clay only. Furthermore, the tunnel shall be constructed by Earth Pressure Balance Type TBM with ground loss of 1-3%.

2.7 Ground loss and causes of settlement due to tunnelling

Ground movement around excavation area is mainly caused by ground loss during the construction. Volume of ground loss is varied depend upon several factors such as type and selection of machine, work standard, performance of equipment, face pressure, ground improvement around the excavation area, rate of excavation, grouting pressure, etc. In general, cause of ground loss can be categorized into four groups which are as follow.

2.7.1 Ground Loss into Face

Ground loss into face is normally occurred due to usage of inappropriate face pressure in front of TBM. Face pressure also relates to rate of excavation and muck extraction of the machine. Hence, machine operator has to finely adjust the rate of extraction, in order to control face pressure and must check monitoring gauge regularly. Assessment of in-situ stress or surrounding earth pressure could be performed by using series of pressure gauge in the front of TBM or equation proposed by Lee, et al. (1992), which also shown below.

$$P_0 = (K_0^{'}\sigma_v^{'} + P_w) - P_i$$

where	P_0	Total stress in the front of TBM
	K_0'	Effective coefficient of earth pressure at rest
	σ_v'	Vertical effective stress
	P_w	Water pressure at springline level
	P_i	Face Pressure

2.7.2 Ground Loss over Shield

Ground loss, in this case, is mainly occurred by controlling the TBM out of alignment, either over pitching or yawing. When operator controls the machine back into assigned route, the action then causes wider gap between shield and surrounding soil. Therefore, operator shall check the coordination of the machine frequently during the operation.

Another reason that creates ground loss over shield is the construction technique and direction planning. TBM operator normally controls the machine to pitch up a little bit, in order to compensate the settlement of the heavy machine. The pitch angle then generates over excavation zone above the machine. However, the operator can minimize over excavation by drilling from the deepest section to shallower section (Lee et al., 1992).

2.7.3 Ground Loss due to Tail Void

Ground loss due to tail void is actually created by structure of the machine. TBM tends to have larger cutting face than the machine body to minimize frictional force around the machine. Moreover, concrete segment used in tunnelling is also smaller than TBM body. Therefore, total space between cutting face and concrete segment is substantial. Furthermore, process of drilling, segment assembly and grouting requires considerably time, which also relates to TBM length, and highly affects to ground movement. Hence, ground loss due to tail void could create substantial magnitude of settlement.

In order to decrease construction cycle, project planner may select articulate shield machine to decrease over excavation for the project. Automatic grouting system which can inject cement milk through injection port on the TBM shield directly is also available in the market.

2.7.4 Ground Loss due to Deformation of Tunnel

Changing of surrounding earth pressure may cause constructed tunnel to deform and hence, create ground loss around the segment. However, ground movement due to deformation of tunnel is very small when compared to other categories.

2.8 Ground settlement along tunnelling axis

Characteristic of ground settlement along tunnel route can be divided into 4 periods as stated below (Sramoon and Sugimoto, 1999).

Initial settlement is normally happens before TBM approaches to monitoring section. Movement of ground can be either settlement or heave, depend upon soil characteristic, site condition and face pressure. Moreover, in case that TBM operator uses higher face pressure than earth pressure, surrounding cohesive soil may have volume change, due to consolidation process.

Shield passing settlement is the movement that occurs during the passing of TBM through monitoring section. The displacement is mostly created by friction and shear forces around the machine as well as the collapsing ground, prior to grouting process.

Tail settlement is usually happens behind the TBM, due to collapsing ground around tail void before substitution of grout material. Therefore, duration and procedure of construction are significantly effect tail settlement. Other factors that affect tail settlement are ground condition and groundwater.

Long term settlement is the movement, due to consolidation and creep processes which caused by grouting work, changing of water pressure and tunnel deformation. However, many studies in Bangkok area show that long term settlement is not significant in the stiff Bangkok clay layer (Phienwej et al., 2006; Suwansawat, 2006; Soktay and Teparaksa, 2007). Figure 2.7 presents vertical movement along tunnel path during TBM driving in Bangkok ground

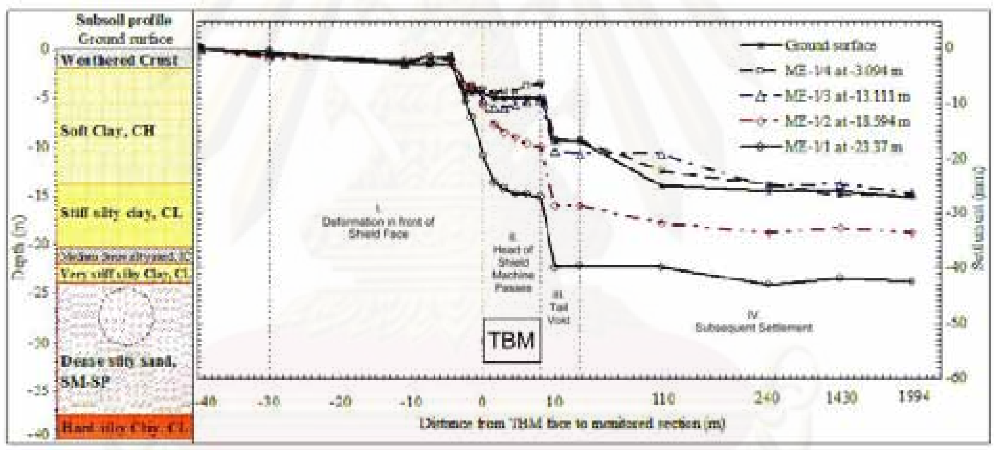


Figure 2.6 Various types of ground movement during tunnel excavation
(Soktay and Teparaksa, 2007)

2.9 Changing of earth pressure during tunnel construction

Method of tunnel construction is the major source that changes surrounding earth pressure. If TBM operator uses high excavation rate, surrounding ground then tends to move into the tunnel, and cause active earth pressure. In the other hand, if TBM operator uses low excavation rate with high face pressure and high grouting pressure, surrounding ground then tends to be pushed out from tunnel and cause passive earth pressure.

Prior to tunnelling, Myrainthis (1981) found that undisturbed ground has lateral earth pressure equal to K_0. When TBM operator has removed muck from excavation chamber, horizontal stress and K_0 in the front of TBM decrease. Surrounding ground is then subjected to active earth pressure. When driving cylinder extends and grouting work starts, additional pressure then pushes soil particle outward from the tunnel. Surrounding ground is then subjected to passive earth pressure. Behavior of earth pressure during tunnel construction is also shown in Figure 2.7.

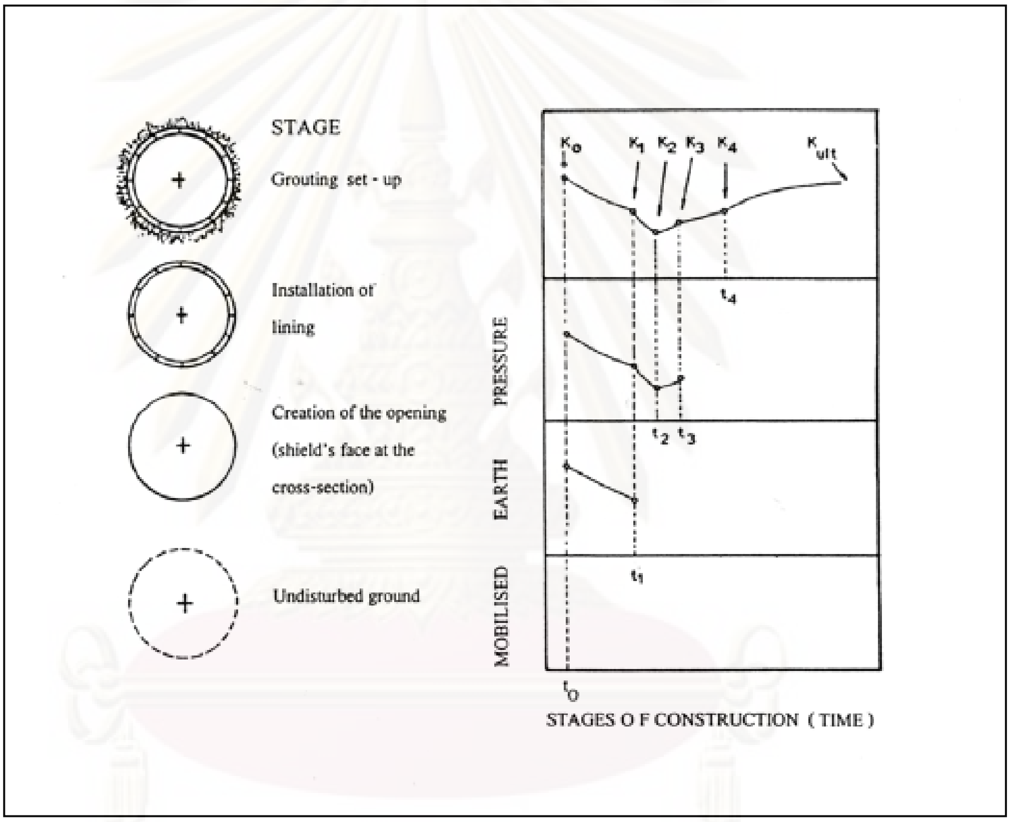


Figure 2.7 Changing of earth pressure during tunnel construction
(Myrianthis, 1981)

2.10 Finite Element Modelling (FEM) and Parameter Validation

Time limitations and MRT criteria have forced the main contractor to the edge. Therefore, numerical modelling is necessary to estimate the possible effects of construction and displacement of existing tunnels. However, available numerical software also has a limited capability to model tunnels in three dimensions. Moreover, three-dimensional analysis also requires large computer resources and calculations over a long time, while time is critical. Hence, two-dimensional analysis was originally selected for the project.

Although two-dimensional FEM has been accepted for the prediction of tunnel behaviour, simplification of the problem is concerned for this particular case. The nature of two-dimensions is absolutely limited to plane strain analysis, while the actual tunnel axis is not perpendicular to the MRT axis. The setting up of the model is completely on a single plane. The modelling of structure and activity in the third dimension has to be simplified based on input from the designer (Teparaksa, 2005b). Furthermore, the construction sequence is also simulated directly in all slices, which may provide exaggerated results in some cases. Actual construction normally varies and contains uncertainty. Therefore, simulation may be completely incorrect.

Calibrated 2D FEM with actual field information can provide good estimations of prediction (Negro and Queiroz, 1999). Hence, typical numerical parameters will have to be calibrated with the first two monitoring sections, in order to compensate for the above reasons. According to previous studies of the tunnelling in the Bangkok area, it is not necessary to concern all tunnelling parameters in the numerical process (Teparaksa et al., 2004 and Phienwej et al., 2006).

The model was based on the 2-D FEM and the Mohr-Coulomb failure criterion. Two dimensional FEM with Mohr Coulomb failure criterion has been confirmed to be appropriate for the prediction of tunnel behaviour for the MRT project in Bangkok (Prinzl and Davies, 2006 and Teparaksa, 2005c). Teparaksa (2002) has analysed the results of the self-boring pressure meter test and developed the E_u/S_u ratio, based on the relationship between soil stiffness and the degree of shear strain. The non-linear relationship between soil stiffness and strain, which was considered by assuming the relationship between Young's modulus and shear strain, varies from 0.1-1% (Fig. 2.8).

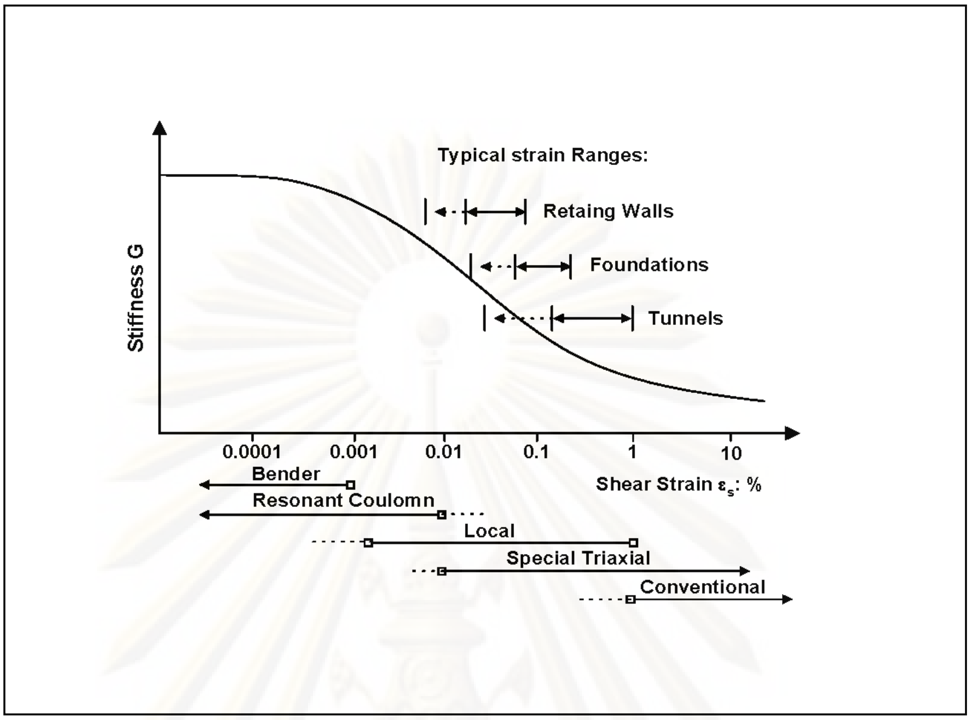


Figure 2.8. Typical shear modulus and shear strains for foundation works (Mair, et al., 1993).

Based on field monitoring, the adopted parameters are validated before applying them to numerical prediction. A critical criterion of the MRT tunnel is the displacement limitation. Therefore, the finite element model shall be verified by ground displacement (Teparaksa, 2004 and 2005d). Validation will concentrate on vertical movement as the settlement value is predominant displacement on site. Lateral displacement is then automatically validated at the same time (Teparaksa, 2005c). Hence, numerical results will be compared with surface settlement and the deep settlement point only.

CHAPTER III
RESEARCH METHODOLOGY

3.1 Preliminary Study

Preliminary study involves literature review and also studies any damage assessment reports of underground structure that relates to tunneling work in BMA area. In the previous two diversion tunnel projects, in which diversion tunnels were bored underneath the main water supply tunnel (ID=3.50m), a trial and error process of tunnel operation was carried out on site to control ground deformation and construction performance (Teparaksa, 2002 and 2005c). A series of instrumented sections were used to monitor the relationship between ground movement, operation performance and tunnel parameters. Hence, in this project, five test sections have been set up to identify the ground interaction with TBM parameters. Information collected from geotechnical instruments and records of tunnelling parameters will then be assessed so as to improve construction performance and to be used for back analysis of the Finite Element Method (FEM). Simulations of validated analysis shall provide appropriate guidelines for tunnelling parameters for the critical part of the project.

Apart from the numerical analysis, various kinds of geotechnical instruments have been installed before approaching the MRT area. The monitoring results add value to the prediction (Bakker et al., 1999). Response levels and contingency plans have also been set up, based on numerical modelling and monitoring, to cope with emergency situations. Moreover, the deformation of the existing tunnel was also monitored by the installation of reference points and surveying inside the tunnel. The above predictions and monitoring results provide sufficient confidence for the BMA and MRTA to grant approval for the construction.

3.2 Site Description

The third diversion tunnel was constructed to solve the flooding problem in the inner zone of the Bangkok Metropolitan Area. The reinforced concrete segment tunnel collects the water from between the Makkasan Reservoir and diverts it to the Chao Praya River at Wat Chong Lom. The Diversion tunnel also collects water from two main canals, which intersect the route. These two canals are Khlong San Saab and Khlong Phai Sing Toh. The tunnel also collects water from the pumping station at Rama 4 road. The length of the tunnel is about 6,200m with an internal diameter of 4.6m. The length of each ring segment is 1200mm. The lining of the tunnel consists of 5+key reinforced concrete segments of 275mm thick. The weight of each ring is about 130kN. The minimum curve radius of the tunnel is 80m. Figure 1 shows the path of the tunnel from the Makkasan Reservoir to Wat Chong Lom Pumping Station. Figure 3.1 presents the tunneling path from Makkasan Reservoir to Wat Chong Lom Pumping Station.

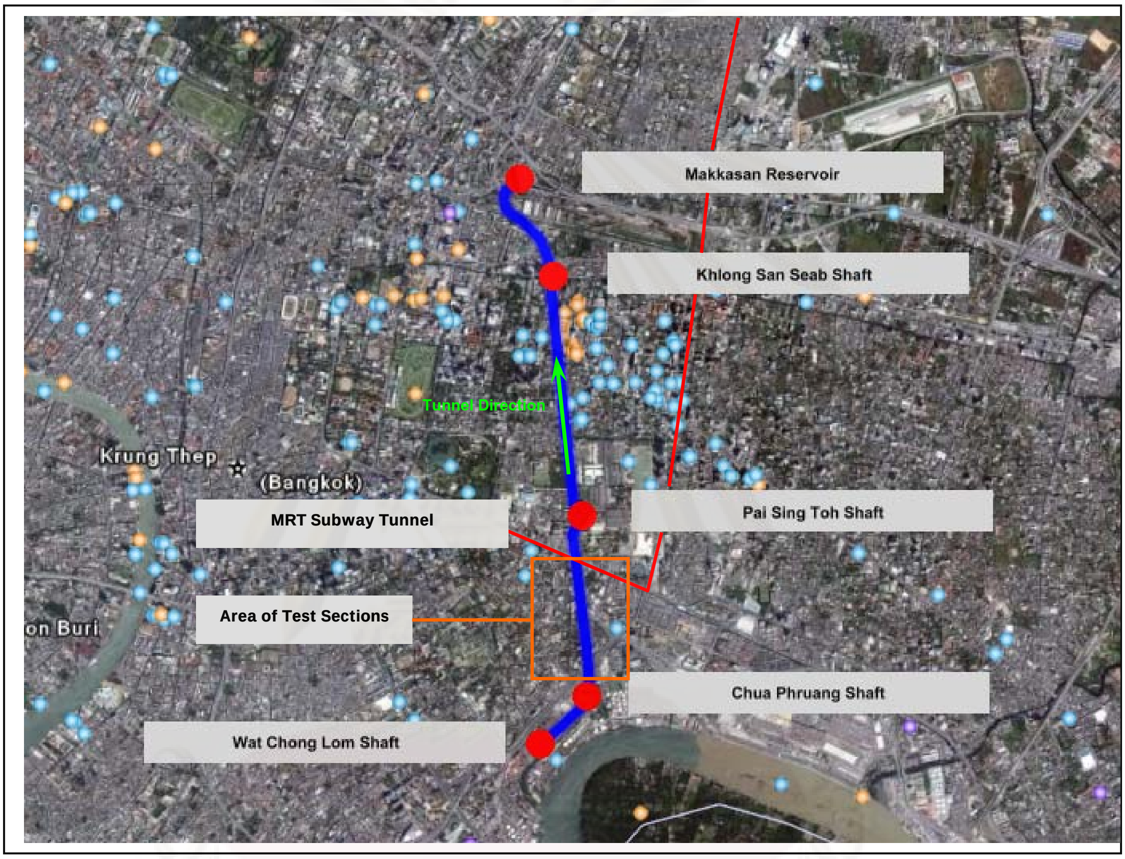


Figure 3.1 Path of diversion tunnel between Makkasan Reservoir and Wat Chong Lom Pumping Station

The spring-line level of the diversion tunnel is about -18.0m below the ground surface at the Makkasan Reservoir, while the deepest part of the tunnel is at -27.5m below the ground surface at Chong Lom Temple Pumping Station. The tunnel was planned to be constructed from the deepest area at the outlet shaft of Wat Chong Lom Station to the shallower area at the Makkasan Shaft, mainly due to limitations of the construction areas, and also to minimize ground loss over the shield during the construction of the tunnel (Lee et al., 1992). Several caissons, each of 15m in diameter, were constructed as service shafts which are also used as part of the pumping stations. Several caissons, 15m in diameter, were constructed as service shafts and have also been used as part of pumping stations, also presents in Figure 3.2.

Figure 3.2 Photo of sinking shaft at Wat Chong Lom

Figure 3.3 The assembly of TBM machine

3.3 Geological Condition

The geological condition of Bangkok normally consists of a silty sand layer overlaid by soft to very stiff clay, up to 30m in some areas. Longitudinal section of geological profile between Makkasan Reservoir and Wat Chong Lom Pumping Station is also shown in Figure 3.4. Piezometric pressure starts at the depth of about 23m from ground surface (Teparaksa, 1999). Owing to the nature of formation, non-uniform geological conditions and sand pockets may be found across the area. Hence, site investigation is necessary. Soil investigation was carried out along the tunnelling path. Samples were collected from 27 bore holes for geotechnical laboratory testing.

According to Figure 3.4, soft clay is about 9 m thick at Makkasan Reservoir. Second layer is medium stiff clay, about 4m thick. The third layer of very stiff silty clay, about 7m thick and finally, hard clay which starts from 20m below ground surface. The tunnel springline is about 21m below ground surface i.e. at 15m BMA. Excavated tunnel is gradually descents to Khlong San Seb Shaft.

At Khlong San Seb Shaft, new pumping station will control the volume of intake water from the canal. Geological profile at this section is quite similar to Makkasan Reservoir however; seam of clayey sand, about 2m, was found between very stiff silty clay and hard clay. The springline level is 25m below ground surface. Therefore, crown of new tunnel is just below clayey sand layer. Moreover, three bore holes between Khlong San Seb and Khlong Pai Sing Toh shows that clayey sand and sand pockets are in the tunnel route.

At Pai Sing Toh Shaft, soft clay layer is about 9m thick while the medium stiff clay is 4m. Stiff silty clay layer is thicker at the third shaft, about 10m. Again, sand pocket (about 4m thick) is found in the upper part portion of TBM route. The lower portion of tunnel route is preferred hard clay. Springline level is about 28m below ground surface. From Pai Sing Toh shaft, TBM will further descent to RL7.00m in order to excavate under MRT tunnel and approach Chua Phruang Shaft. According to Figure 3.4, clayey sand pockets are located under MRT section.

At Chua Phloeng Shaft, geological profile is similar except that sand layer is found at RL-3.00 or 33m below ground level. The top of sand layer is increases to RL+9.00 around Wat Chong Lom Shaft and hence, TBM has to excavate in sand layer at this section. Ground

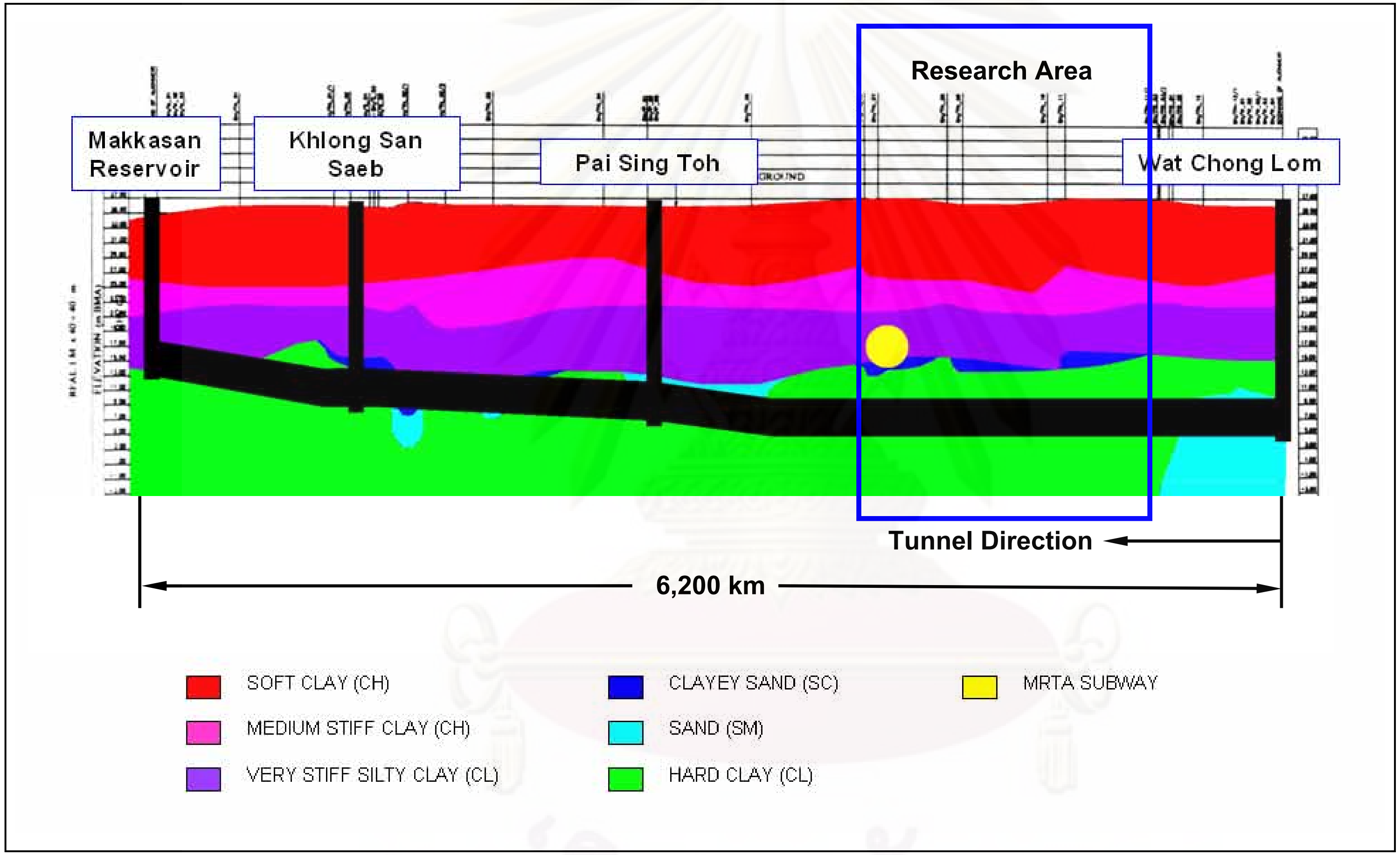


Figure 3.4 Longitudinal section of geological profile along tunneling construction path of the project (adapted from Teparaksa, 2005)

water is also quite high, due to the location which closed to Chao Praya River. Sand layer also creates problem for shaft sinking operation as ground water tends to leak from high permeable layer.

3.4 Obstruction

Although construction path is mainly under property of Royal Railway Authority of Thailand, TBM operator still has to control the machine passing two critical areas which are (1) bored pile of Expressway Phase 1 at Chua Phloeng Road and (2) Mass Rapid Tunnel (MRT) System under Rama IV Road before entering to Makkasan Railway Service Center and Receiving Shaft at Makkasan Reservoir. Details of structure are mentioned in the next section.

3.4.1 Expressway Intersection

Tunnel path is locates in the most crowded area of Bangkok and therefore, obstruction during construction cannot be avoided. Major obstruction in this project is foundation of Bangkok Expressway. The mega concrete structure links between Din Daeng and Dao Ka Nong District and known as the fast track for Bangkokian people to across the city. The tunnel path has to pass near pile foundation of expressway. The pile tip is only about 2m away from the edge of TBM. Unfortunately, previous drawing of pile foundation cannot be obtained from the authority as the structure has been constructed since 20 years ago. Therefore, the main contractor has to excavate around the pile cap, in order to measure the pile diameter. Length of the pile has been examined by down hole integrity method. Section of reinforcement has been assumed, based on typical arrangement of reinforcement during construction period. Down hole technique has been used in the past to determine the depth of pile in Bangkok area (Teparaksa et al, 2005). As shown in Figure 3.5, original tunnel route is designed to pass between two columns of concrete structure. Hence, TBM operator has to turn the machine with short radius.

Geological condition at this particular area consists of silty sand (backfill) layer, about 3.5m thick, at the surface. Layer of soft clay then locates from 3.5m to 12.0 m while a layer of medium clay starts from 12.0m to 15.0m. Stiff clay layer could be found between 15.0m and 19.5m and very stiff clay could be found between 19.5m and 25.5m. A 3m thick layer of very dense sand locates at about 25.5m. Finally, hard clay layer extends from 28.5m to 40.0m. Piezometric pressure starts at the depth of about 20m from ground surface (Teparaksa, 2005b;

Seah et al., 2006). Due to nature of formation, non-uniform geological condition and additional sand pocket may be found across the area. TBM was driven at 30.4m from ground surface, i.e. in very dense sand and hard clay layer. The pile tip is locates at 29.2m and about 1.9m from the edge of the diversion tunnel.

Figure 3.5 Tunnel route under Expressway

3.4.2 MRT Subway Tunnel

The MRT tunnels are located directly under one of the busiest roads in Bangkok, Rama IV, Figure 3.6. The eight-lane asphaltic road was constructed on a 10m layer of soft clay. Above the crown of 5.6m ID MRT tunnel, there is a 4m layer of medium stiff clay. The MRT tunnel itself was constructed in very stiff silty clay, as shown in Figure 3.7. One meter below the 0.3m thick concrete segment of the MRT tunnel is a pocket of clayey sand and hard Bangkok clay. The new diversion tunnel will be constructed 6m below the existing tunnel in a layer of very stiff clay. As mentioned above, the TBM has to pass under dual existing parallel

MRT tunnels at 61 degrees from the MRT axis. Rail track and high pressure fuel pipelines (1m diameter) are also located above the MRT tunnel. The distance between the Northbound tunnel and the Southbound tunnel is about 7.8m. The gap between the crown of the new tunnel and the existing tunnel is quite thin and hence, poses a critical issue to the MRTA, Figure 3.8.

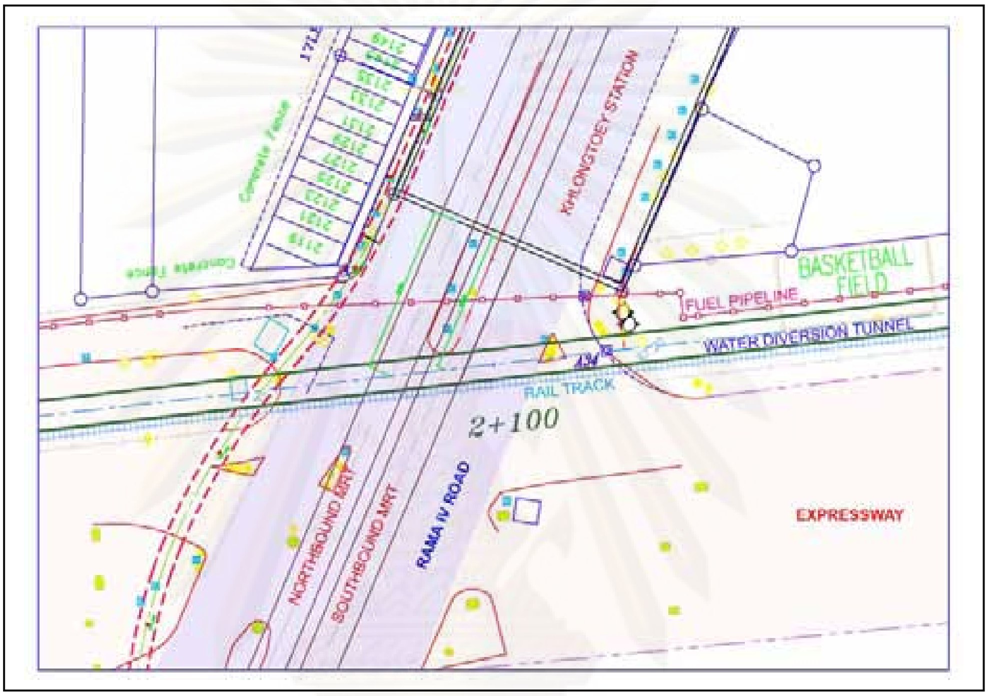


Figure 3.6 Intersection plan at Khlong Toey Station.

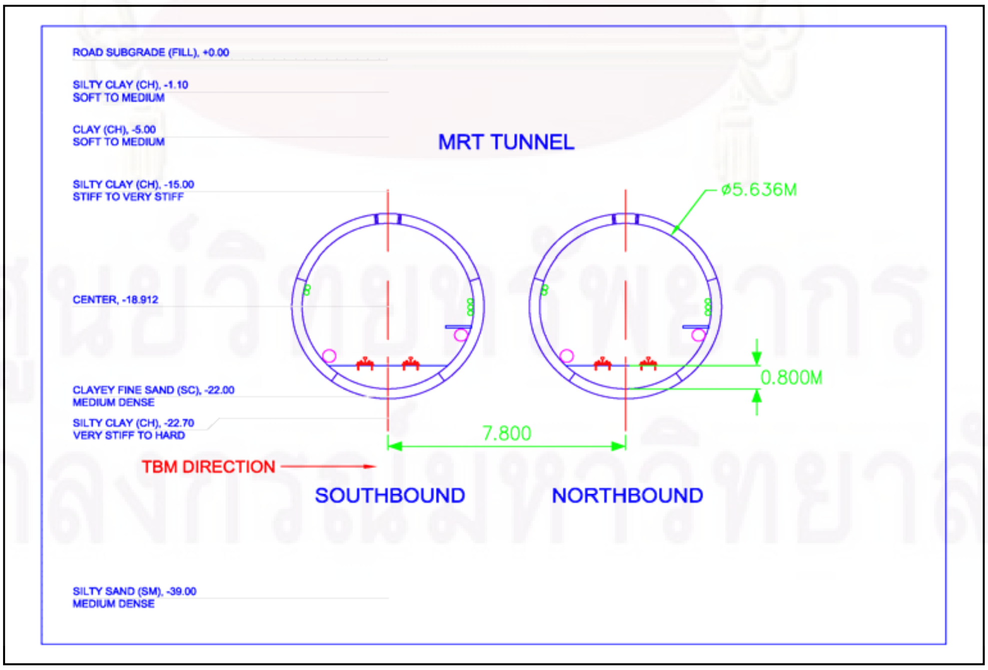


Figure 3.7 Cross section of MRT Tunnel

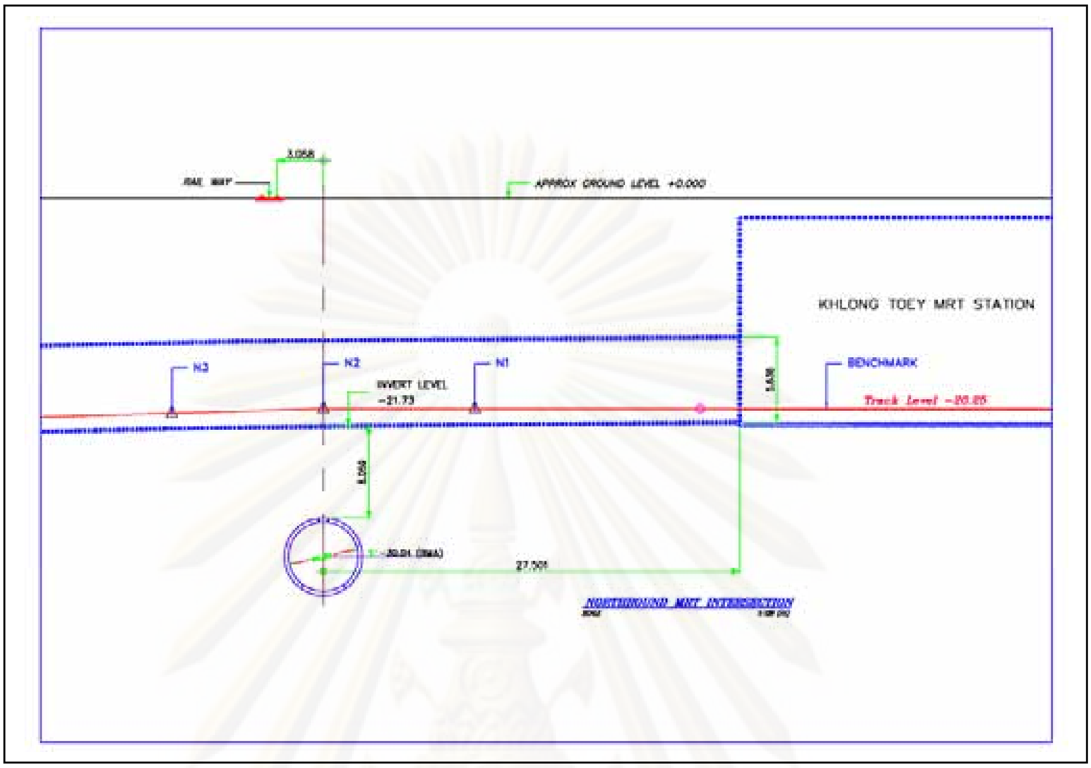


Figure 3.8 Longitudinal section at northbound tunnel

The BMA and MRTA have asked the main contractor to provide strong evidence to prove that the construction process will not damage the MRT tunnel. Based on the MRT criteria, the allowable tunnel deformation is only 6 mm in any direction. The operating hours of the MRT run from 5 am to 12 pm and are not allowed to be stopped for the construction. Any monitoring, servicing or remedial work has to be done between 1 am and 4 am or only 3 hours daily.

3.5 Research Area

Generally, the monitoring programme is adapted from Bakker et al. (1999) and consists of three technical parts, i.e. TBM parameters, geotechnical deformations and structural deformations. The monitoring programme also concentrates on ground deformation towards the TBM face caused by any stress relief (Mair and Taylor, 1997). Hashimoto et al. (1999) also stated that the consolidation of disturbed ground around the TBM is the main reason for settlement. Therefore, various patterns of instrument installation are described in this section, which are designed to cope with monitoring objectives, site conditions, and the period of installation. Three sections of monitoring are installed along the tunnel path, mostly on Chua Phraung Road, also shown in Figure 3.9.

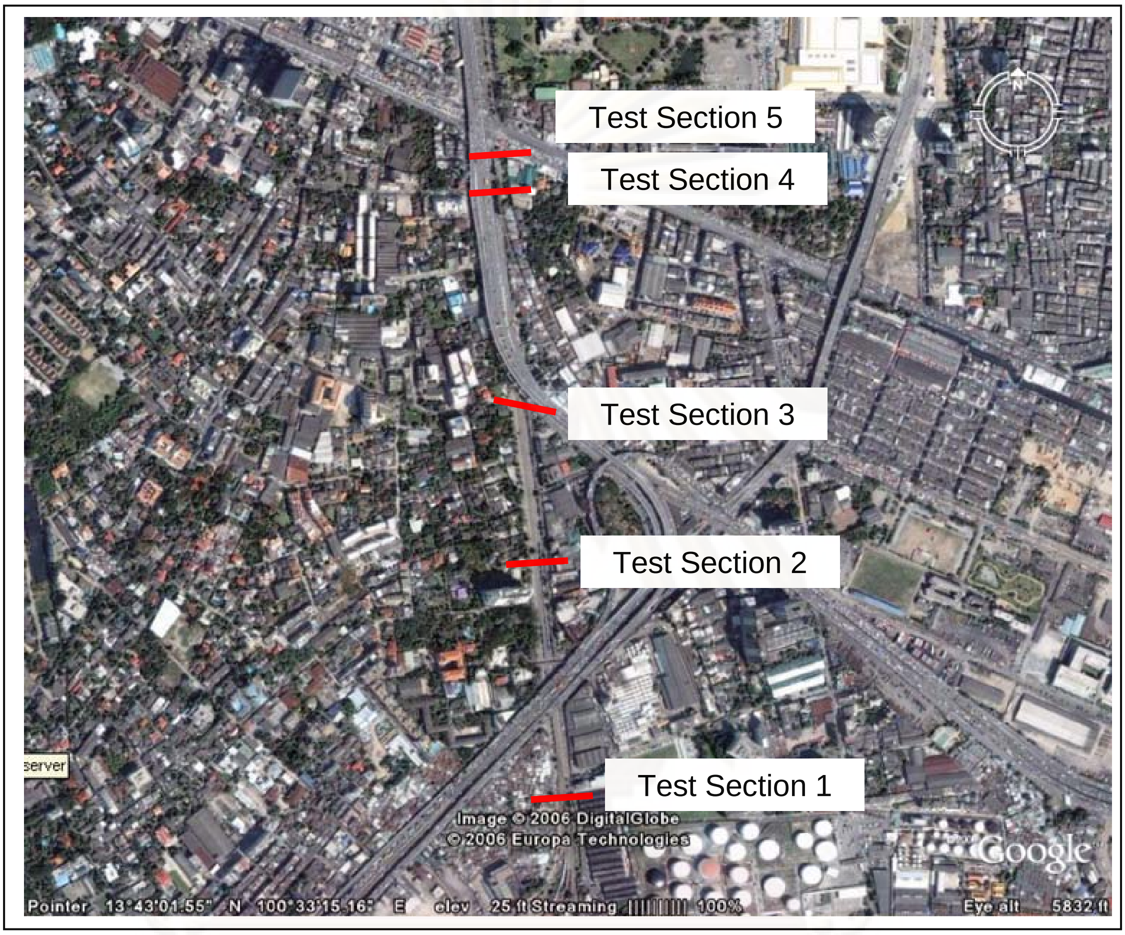


Figure 3.9 Test sections of instrument along tunnel path

3.5.1 Test Section 1

Test Section 1 is located at station 0+853 or about 300m from the Chua Phraung Shaft. The section was mainly designed to check and improve the performance of the TBM operation. As mentioned in the previous section, ground deformation can reflect the quality of construction. Hence, the surveyor has to monitor ground surface settlement intensively. Moreover, changing pore water pressure was also measured after installation and during the construction process. The purpose of monitoring the results after installation is to allow one to determine the dissipation time of excess pore pressure which is caused by the installation process. The section contains 11 surface settlement points, 3 combined inclinometers with a series of extensometers, 2 deep settlement points and 1 pneumatic piezometer, as shown in Figure 3.10.

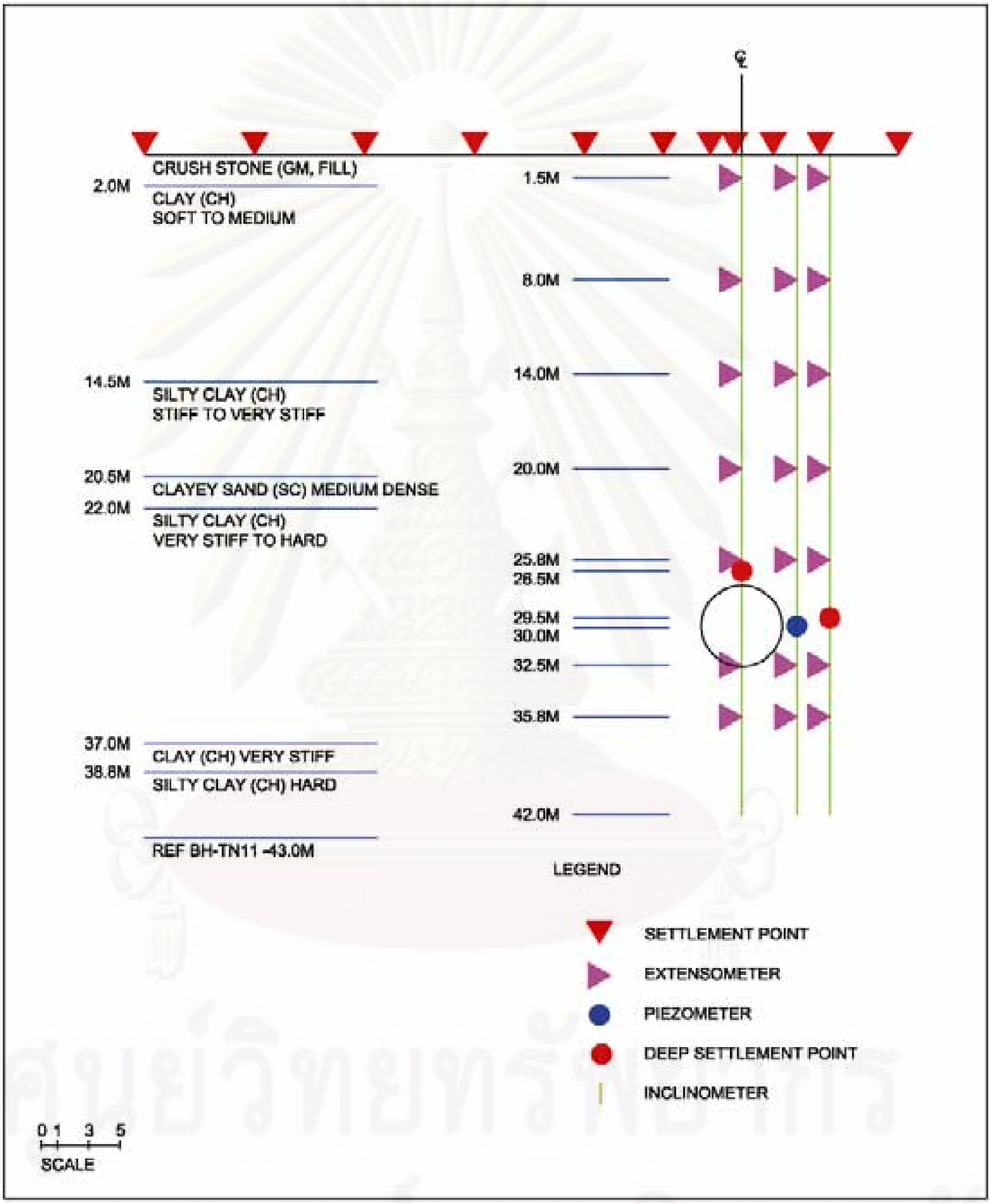


Figure 3.10 Typical section of instrument at test section 1

According to Figure 3.10, the locations of the surface settlement points on both sides of the centre-line are not the same due to site constraints. Settlement points on the left hand side were installed on a small road,. However, the points on the right hand side are limited by private property. Assuming that the ground movement is symmetrical on both sides, three combined inclinometers (inclinometers with a series of extensometers) were installed only on the right hand side of the tunnel path. The instrument at the centre-line was installed into hard clay at a depth deeper than the tunnel spring line, at 42m below ground level. Monitoring was planned to be carried out until the machine cuts the inclinometer pipe. Series of extensometers, also called spiders, were installed along the inclinometer pipe to monitor the profile of vertical ground movement during tunnelling.

Kishio et al. (1994) pointed out that successive settlement is caused by compressive deformation, within 1m from the crown of the tunnel. Hence, a deep settlement point on the centre-line of the tunnel, was installed above the tunnel path, while another one was installed at the spring line level. Only one piezometer was installed at this section, due to time limitations.

3.5.2 Test Section 2

Test Section 2 is located at station 1+446 which is close to the Mae Nam Railway Station. The section was mainly designed to confirm the performance of operation which was adjusted after passing Test Section 1. Moreover, the result of the pneumatic piezometer of Test Section One shows that the instrument is effective at observing the changing pore pressure around the tunnel. Hence, in this section, series of piezometers were also installed to closely measure the profile of water pressure during construction.

According to the previous section, an inclinometer at the tunnel axis had caused serious problems to the TBM. Therefore, in Test Section Two, an inclinometer is terminated above the crown of the diversion tunnel. Test Section Two contains nine surface settlement points, three combined inclinometers with a series of extensometers, three deep settlement points and three pneumatic piezometers, also shown in Figure 3.11.

As shown in Figure 3.11, the locations of the surface settlement points on both sides of the centre-line are also not the same due to site constraints, such as rail tracks and carrier bogies. Settlement points on the left hand side were installed on the road and into private property. However, points on the right hand side are limited by a concrete wall and very small road. Again, assuming that the ground movement is symmetrical on both sides, three combined inclinometers (inclinometers with a series of spiders) were installed only on the right hand side of the tunnel path. The instrument at the centre-line was installed at depth into a hard clay layer, except for the one at the centre-line which was installed just above the TBM. Cutting the inclinometer pipe had created an interruption of the TBM operation, due to the very high elasticity of the pipe. Spiders were installed equally along inclinometer pipes. A deep settlement point, on the centre-line of tunnel, was installed above the tunnel path while another one was installed at the spring line level.

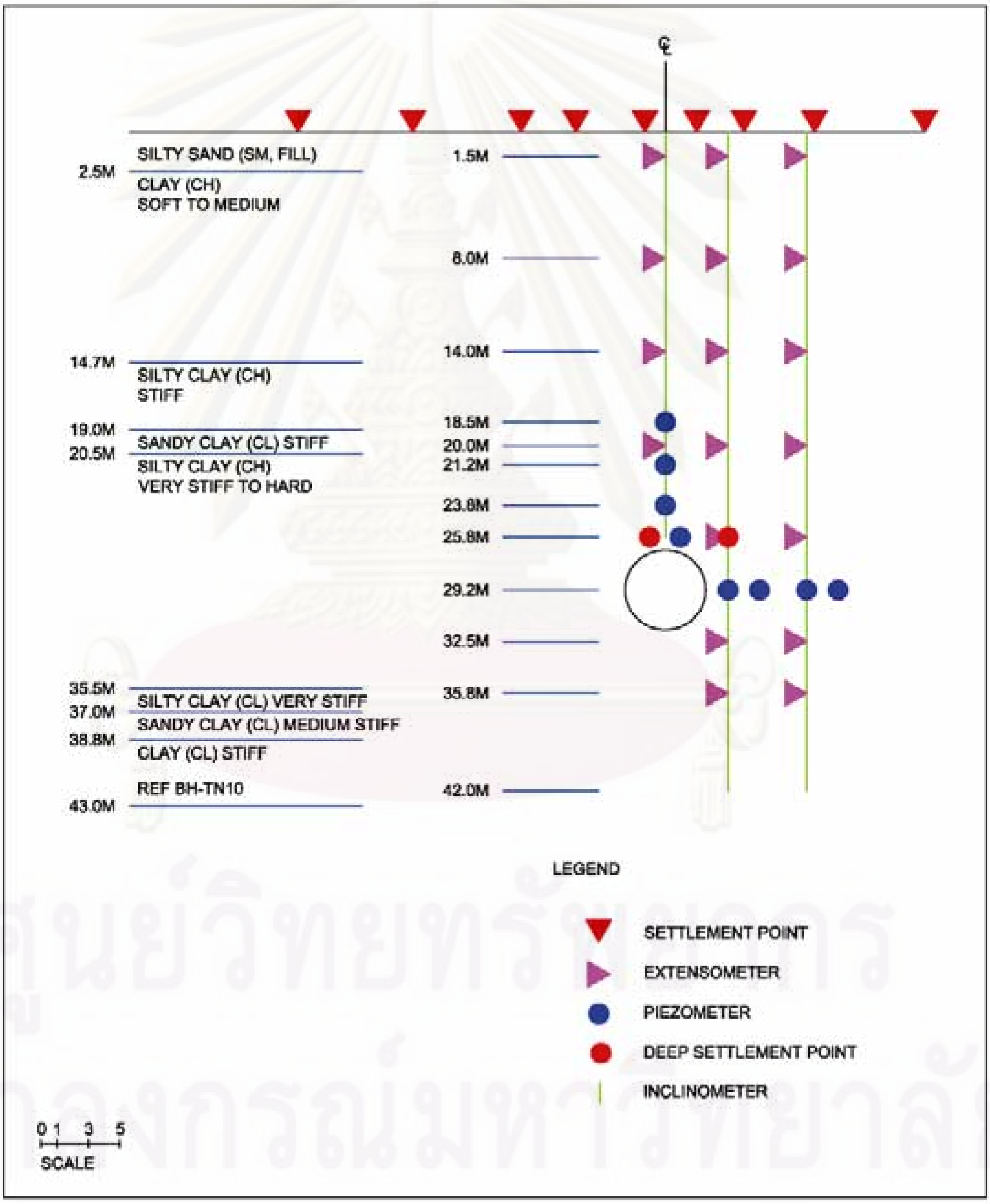


Figure 3.11 Typical section of instrument at test section 2

Although, the tunnelling technology has improved, the consecutive settlement is still large due to consolidation of disturbed ground around the tunnel (Hashimoto et al., 1999). Based on information of pore pressure dissipation from the first test section, several piezometers were planned and installed to monitor the profile of pore pressure changes.

3.5.3 Test Section 3

Test section 3 is locates near Express way, only about 20m before approaching concrete structure. The pattern of test section 3 was proposed to reconfirm the performance of operation however; settlement points were installed outside the area of Expressway due to limitation of area. The section contains 8 surface settlement points which is shown as red line in Figure 3.12. Other instruments which consist of two combined inclinometer and one pneumatic piezometer, were installed next to column of expressway.

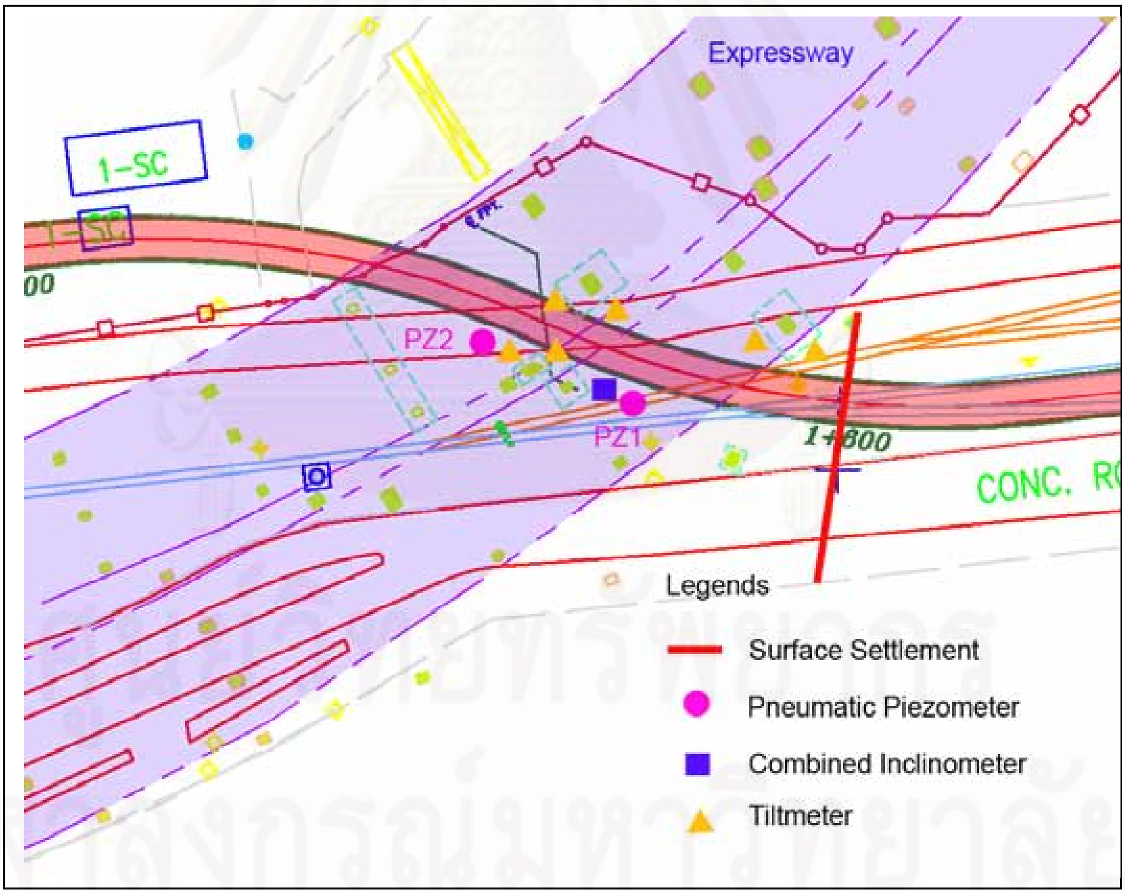


Figure 3.12 Typical layout of instrument at test section 3

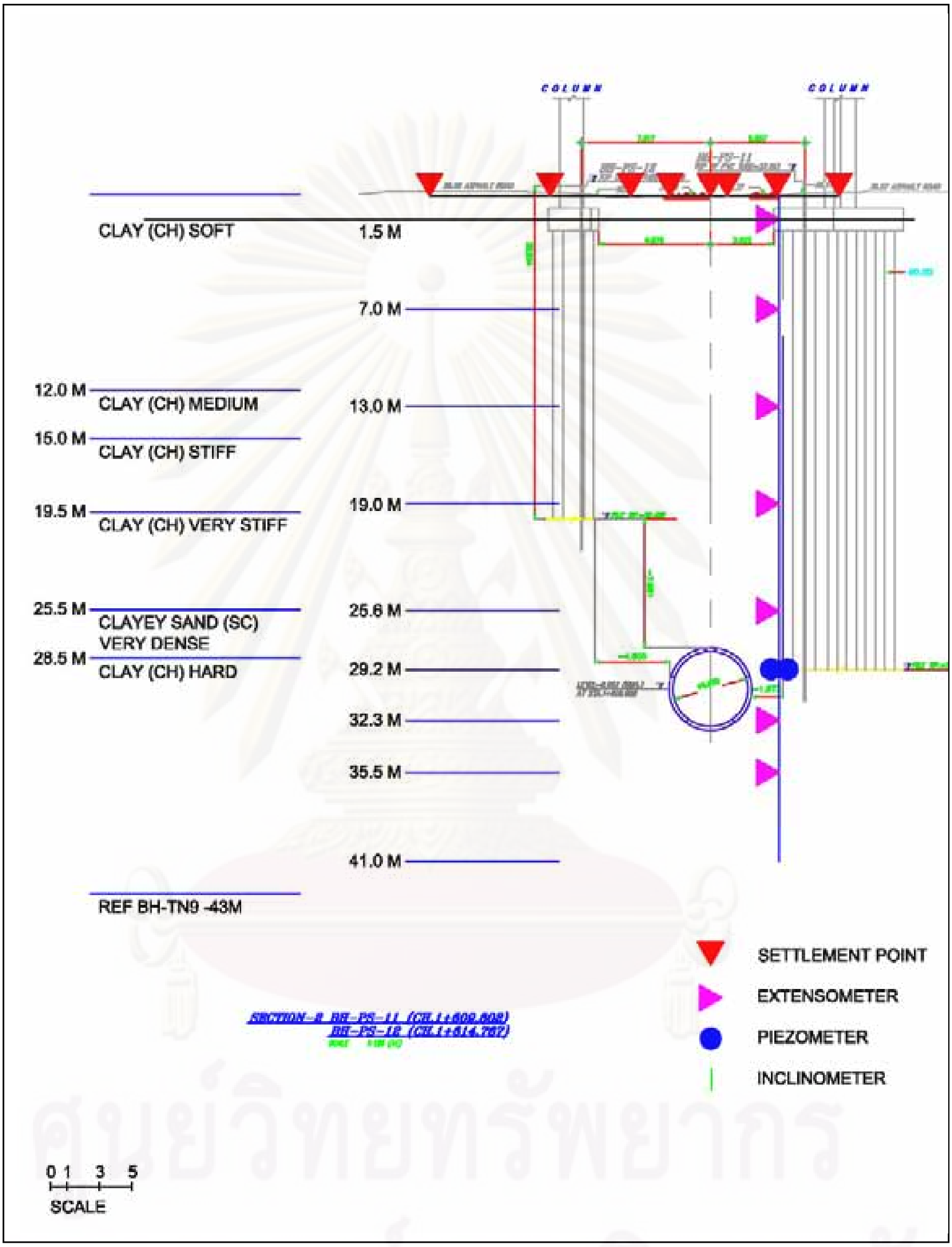


Figure 3.13 Typical section of surface settlement at test section 3

Six tiltmeters were also installed on the column in perpendicular axis to monitor movement of the structure. Typical layout and section of instrument is also shown in Figure 3.12 and 3.13. Due to limitation of installation area; only few instruments are installed at critical point e.g. closest column. As shown in Figure 3.12, one combined inclinometer was set up in front of structure foundation. The toe of inclinometer was installed at 41m deep from ground level, i.e. in hard strata. Only two piezometers were installed in this area. Unfortunately, the opposite side is recycling warehouse and hence, surveyor is not allowed to enter the property.

As shown on Figure 3.13, location of surface settlement point on both side of centerline is not the same due to rail tracks and junction. Settlement points on the left hand side were installed until reaching private property but instruments on the right hand side are limited by thick concrete slab, high pressure fuel pipeline and narrow road.

3.5.4 Test Section 4

Test section 4 is locates about 100m before approaching Mass Rapid Transport (MRT) line. The section was setup to confirm the quality of construction before approaching MRT tunnel. The installation area is originally used as community playground with single rail track and expressway toll on the left hand side hence; installation process can only be performed on the right hand side, as presented in Figure 3.14. Apart from rail track and expressway, the area also contains 1m diameter high pressure fuel pipeline which makes installation process become more difficult. Although official warning sign is already setup on site, drilling process shows that the pipeline is shifted by 1 meter.

Six surface settlement points were installed perpendicular to tunnel axis. Based on previous result of inclinometer, only series of extensometer were installed at the centerline of tunnel while two combined inclinometers were installed near tunnel boundary, as shown in blue symbol. Inclinometers were installed to 42m below ground surface in order to prevent any movement of the toe. Deep settlement points and pneumatic piezometers were installed at both centerline and perpendicular axis, in order to monitor the profile in both directions

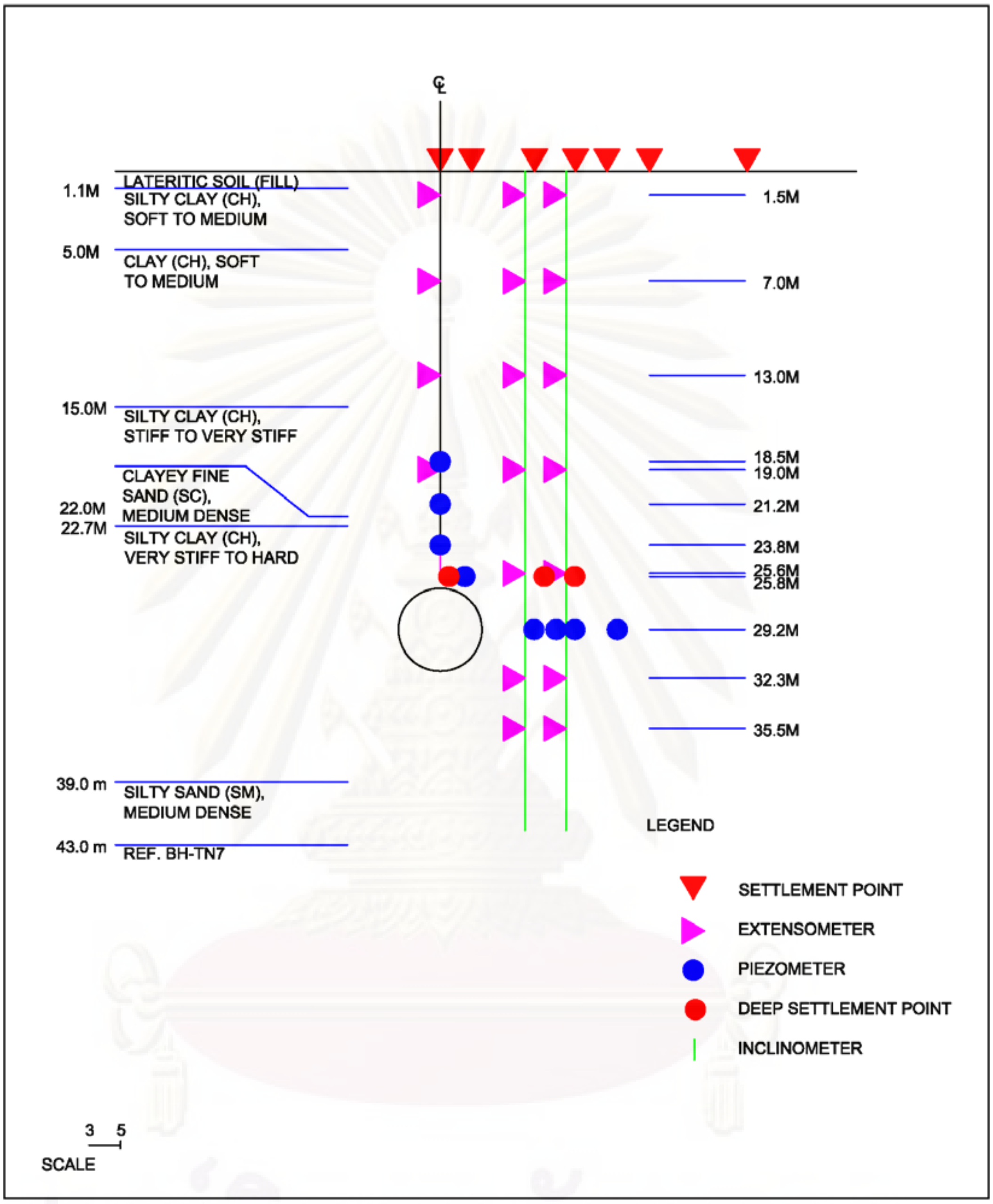


Figure 3.14 Typical section of instrument at test section 4

3.5.5 Test Section 5

Test Section Three is located at station 2+060 or about 15m before approaching the MRT tunnel. Unfortunately, the section cannot be sited directly above the MRT tunnel, due to heavy traffic on Rama IV Road. Therefore, in order to obtain the most information, Test Section Three is then divided into two parts: an outside tunnel monitoring section and an inside tunnel monitoring section. The former has to be set up on the side of the road, about 47m away from the centre-line of the Southbound tunnel, Figure 3.15.

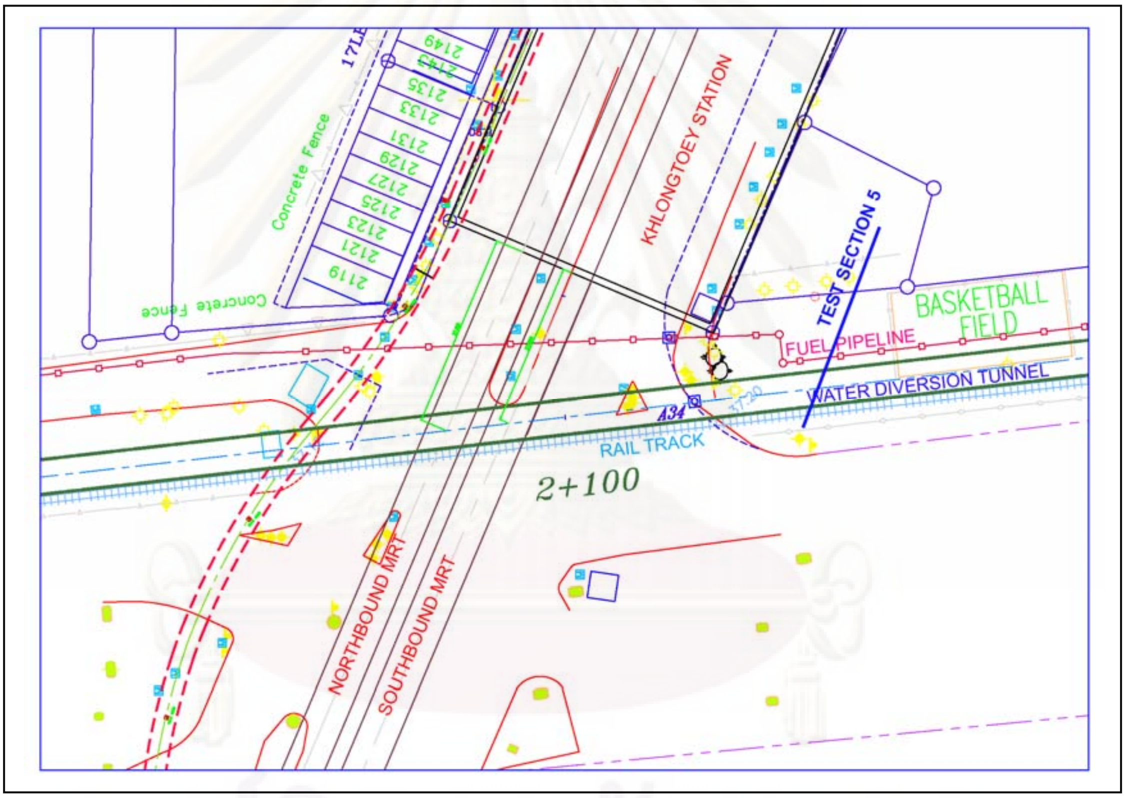


Figure 3.15 Location of test section 5 on Rama 4 Road.

A series of settlement points were installed to monitor surface deformation. In this case, the alignment is parallel to the MRT tunnel (Figure 3.16 and 3.17). Moreover, the operator has to avoid a pressurized fuel pipeline during the installation process. Similar alignment is also applied to the layout of the piezometer and combined inclinometer. A series of deep settlement points and piezometers were also installed along the centre-line of the tunnel at various depths to collect a profile of behaviour.

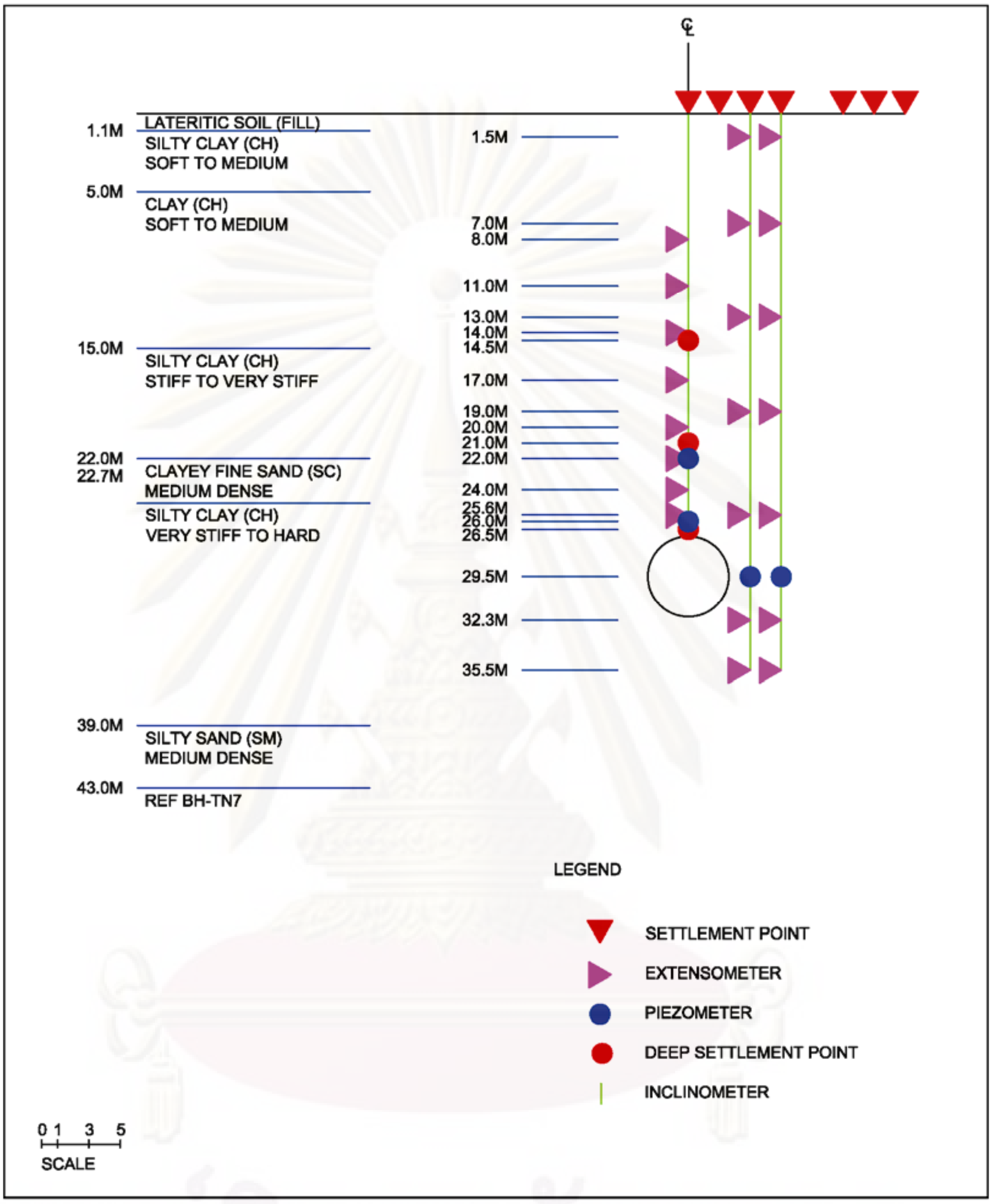


Figure 3.16 Typical section of instrument at test section 5.

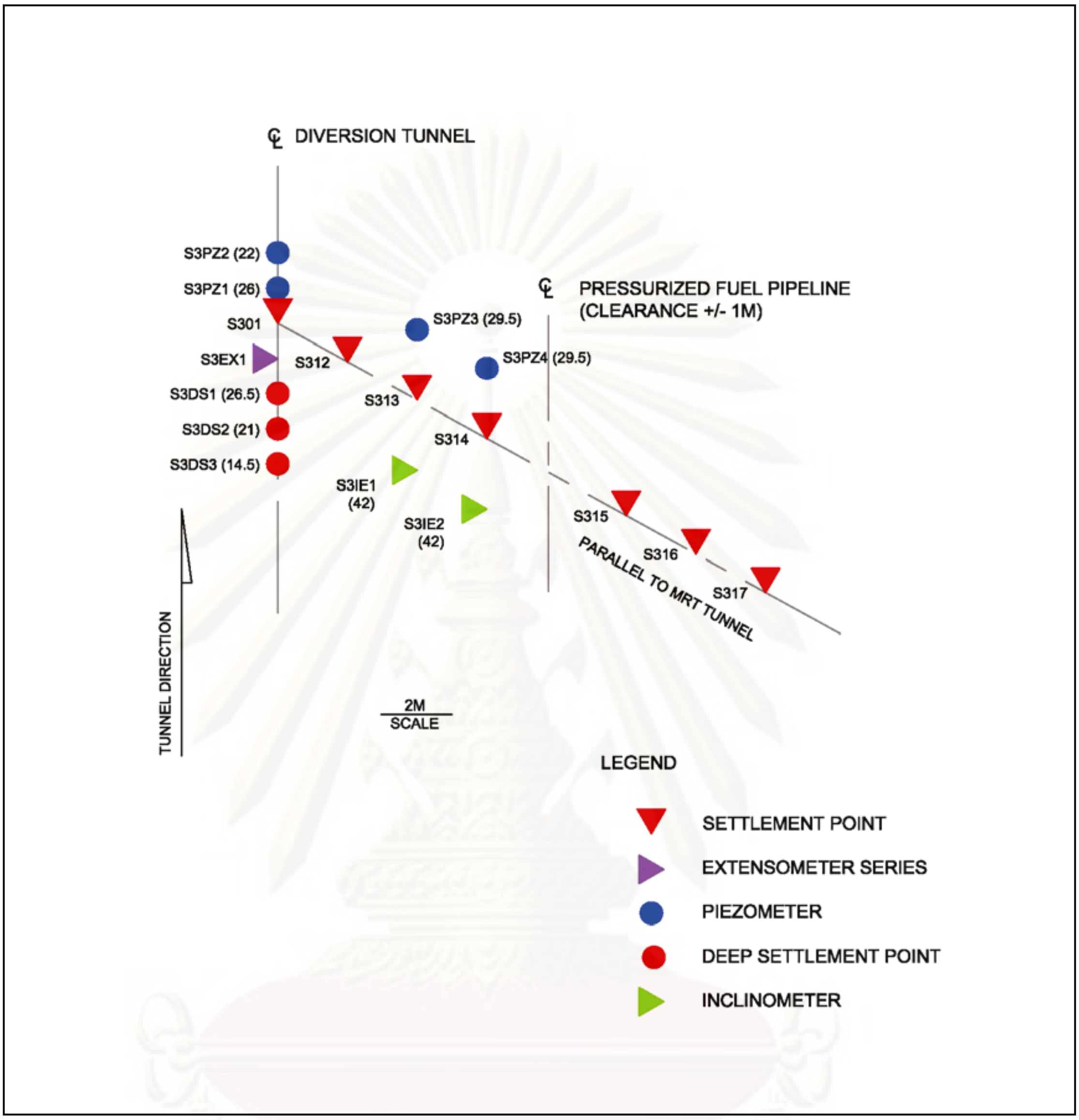


Figure 3.17 Instrument layout at test section 5.

In the MRT tunnel, monitoring was carried out to determine the movement of the tunnel in different axes, including longitudinal deformation and cross sectional deformation in both Northbound and Southbound Tunnels. Three settlement monitoring points, namely N(S)1, N(S)2 and N(S)3 were marked 10m apart inside both Northbound and Southbound tunnels, as shown in Figure 3.18. The benchmark is located inside Khlong Toey Station where movement is assumed to be zero. Vertical monitoring points (N2 and S2) are located directly at the new water diversion tunnel axis, about 27m and 25m from the station, respectively. The TBM of the diversion tunnel will approach from the Southbound Tunnel direction. The result will also be used to define the exact deformation of the tunnel with cross sectional movement.

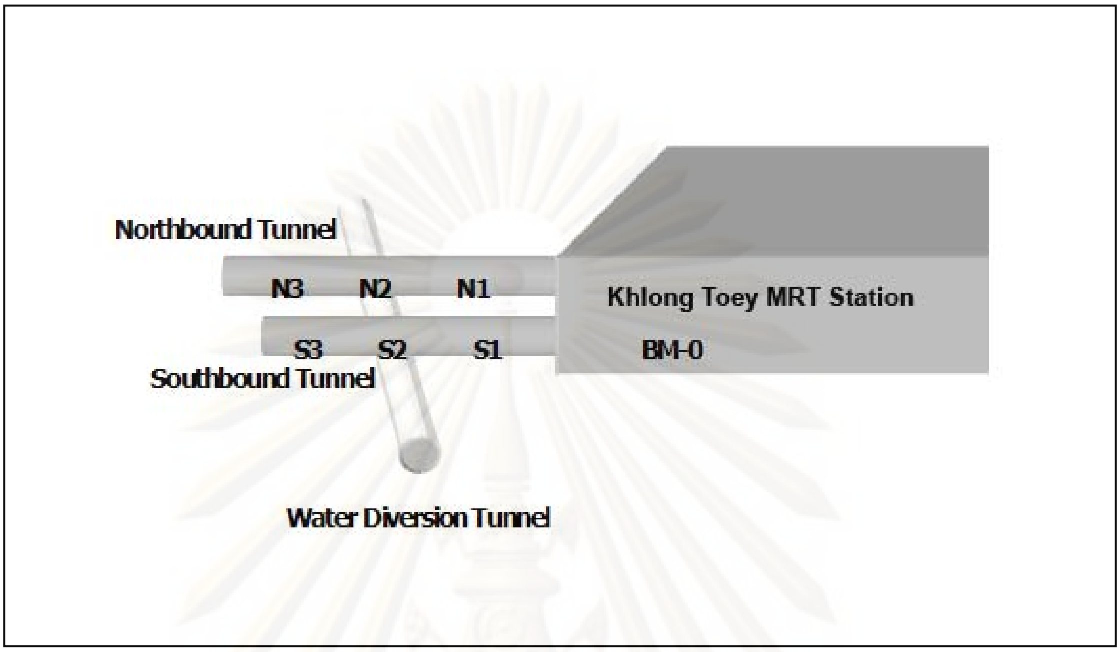


Figure 3.18 Approximate location of tunnel intersection points and location of settlement monitoring points.

Cross sectional deformation of the MRT tunnel is then monitored by a distance measuring device. Prior to the construction, the surveyor marked points of measurement on the segments of the MRT tunnel and obtained initial readings in both tunnels, Figure 3.19. Four target series, called TN(S)1, TN(S)2, TN(S)3 and TN(S)4, were set up to monitor deformation in the four axes of each tunnel, as shown in Figure 3.20. The laser device can considerably reduce monitoring time when compared to the tape extensometer method. The surveyor can then measure the deformation of the tunnel more frequently within the available time.

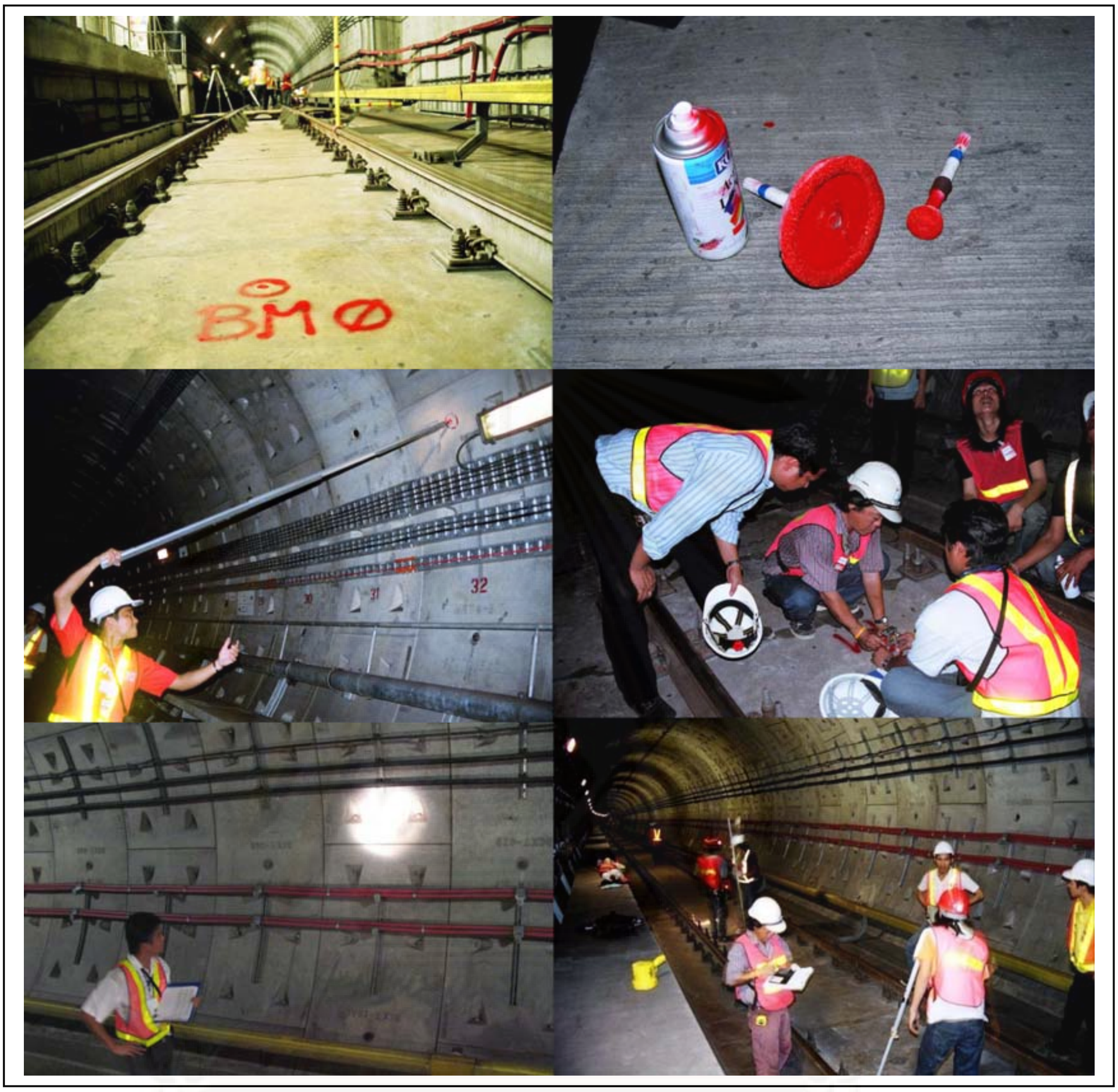


Figure 3.19 Setting up of survey target and initial reading process.

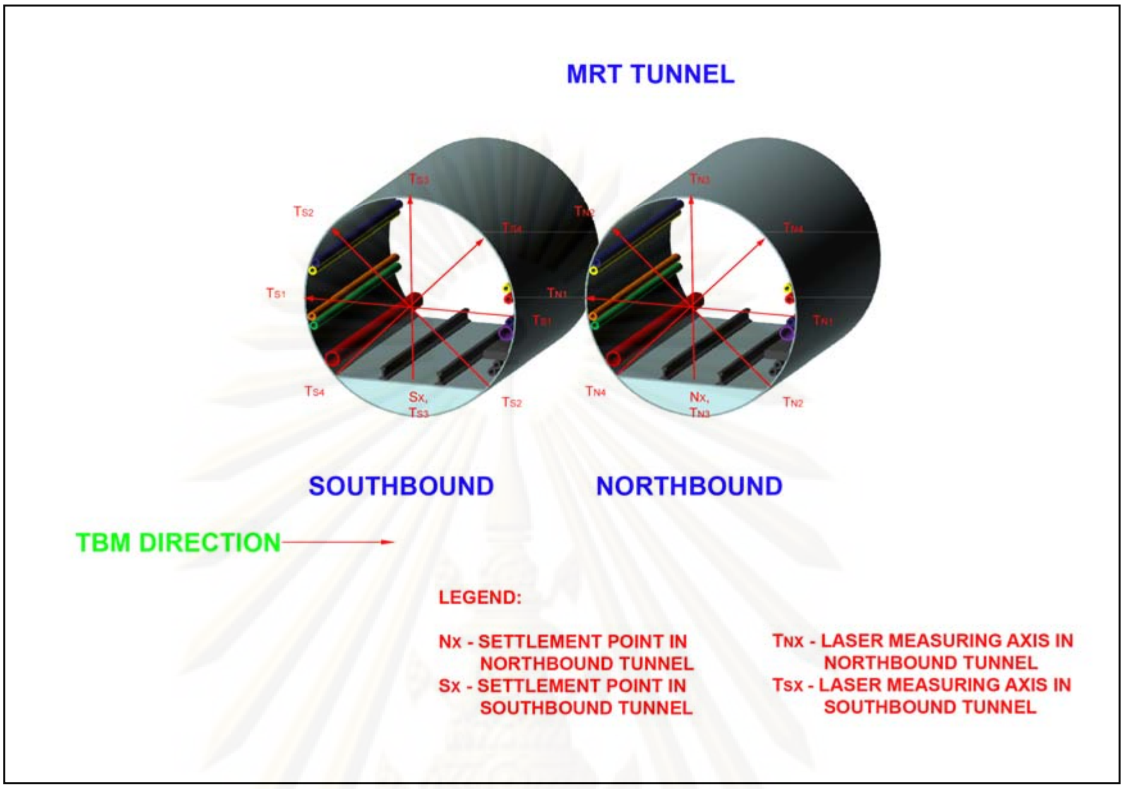


Figure 3.20 Pattern of deformation monitoring inside MRT Tunnel.

3.6 Installation of Geotechnical Instrument

After layout of instrumentation has been approved, the installation of geotechnical instrument shall be carried out on site, based on previous experience and recommendation of manufacturer. However, the installation pattern may also change, due to site constraint. Monitoring information then is collected to assess the quality of measurement and compared to numerical modelling in the next stage. The following section presents typical method of installation and read out of instruments used on site.

3.6.1 Settlement Point

Most of construction activity generally creates ground movement around construction site. Any significant movement around the construction site can affect surrounding structures and therefore, needs to be predicted and measured prior to, during and after the construction process. Vertical displacement of the ground can be measured by using settlement pin, also called settlement point. Series of settlement point are required to create profile of ground movement or displacement contour at the specific section.

3.6.1.1 Method of Installation

Major component of settlement point is basically uniform steel rod. The rod can be made from 20 or 25mm diameter round/deformed bar, if necessary. Survey pin can also be used as the settlement point. The length of steel rod should be at least 30 cm. At least 25 cm of steel bar shall be embedded in the ground. The upper end shall be curved or marked to specify the point of measurement, as shown in Figure 3.21. The top of steel rod shall be covered by mechanical mean and clearly marked on site to prevent any disturbance or damage from people and equipment. The instrument shall also be installed densely where expected ground displacement is high.

Figure 3.21 Upper end of steel rod

Settlement point may be installed deep in the ground where top layer is highly stiffer than the lower layer. Examples of, man made, stiff layer are asphalt and concrete road. The stiff layer can significantly decrease the magnitude of displacement and therefore, interfering the survey value. Figure 3.22 shows the general components of settlement point used to measure vertical displacement below road surface.

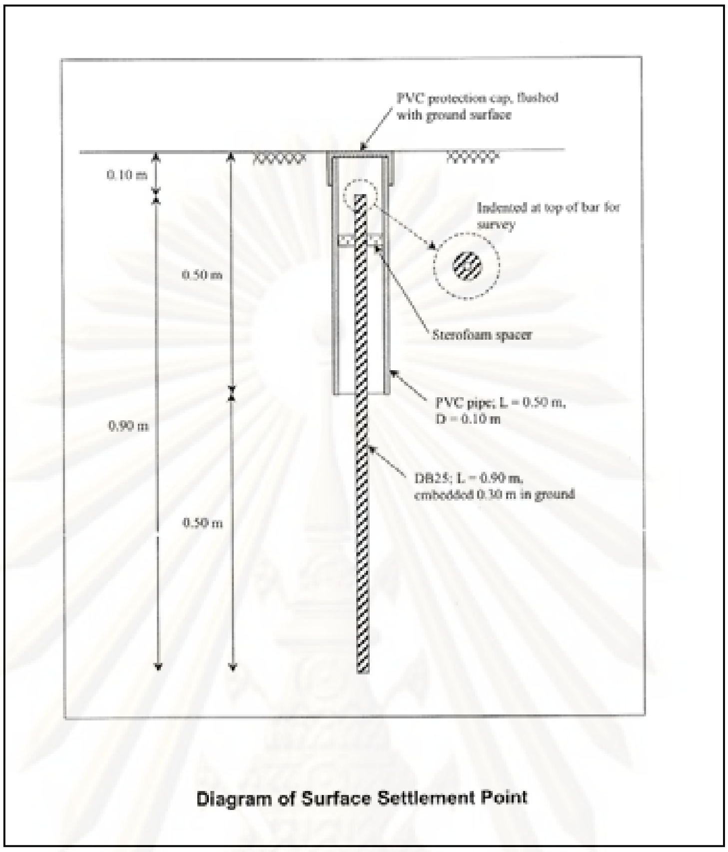


Figure 3.22 General components of settlement point

As shown in Figure 3.22, the steel rod shall be embedded in soft layer to directly reflect actual movement of the ground. In loosened stiff layer such as ballast layer of railway, casing is necessary in order to prevent the hole from collapsing. Cap of casing also provides protection for the instrument. The size of casing should be large enough, in order to provide free space for the movement of steel rod. Figure 3.23 presents the drilling machine used to install settlement point on the road surface. Figure 3.24 shows the settlement point that has already been installed in the ground.

Figure 3.23 Installation of settlement point

3.6.1.2 Measuring Method

Vertical displacement of the ground can normally be measured by surveying. Various surveying equipment are available in the market. Prior to the construction process, initial reading of ground level is highly necessary in order to detect and compare ground movement during the construction period.

Similar to general leveling survey method, reliable benchmark is necessary for the precise reading. The benchmark shall be constructed in undisturbed area i.e. far away from the construction site. Moreover, the survey equipment shall be set up at approximately the same location for each survey and use the same monitoring points for change points. Distances to both intermediate and change points shall not over than 20 m in order to maintain accurate reading. Furthermore, the survey should be closed either by checking to the initial datum or to the new benchmark. The closing error should be within 0.3 mm.

Figure 3.24 Settlement point after installation

Although the consistent monitoring process is performed with the same equipment, the accurate reading is still affected by other factors which are as follow.

Monitoring point and staff error. The particular error occurs due to loosened steel rod or leveling plug on the side wall and non-consistent location of staff. The length of embedded steel can minimize the error of monitoring point. Keeping staff on the measuring point throughout the period of fore- and back-sight can also minimize the staff error.

Weather and environment conditions also influences on the accuracy of reading. Digital leveling equipment is susceptible to strong sun light. Moreover, hot weather can create heat hazes and shimmer which also affects infrared rays between instrument and the staff. Strong or gusting wind also creates error. However, the error due to weather condition can minimize by the overall closing error and quality of the data.

Traffic vibrations are a common source of error in the urban area. As a result, the survey should be carried out during quiet periods if possible.

External influences and instability of the benchmark. The particular error is not significantly affects the differential settlement analysis however, the error is directly affects the absolute settlement value. Some external influences are listed below:

- Seasonal changes in moisture content of the near surface soils
- Damage of surface monitoring points by traffic
- Changes in water level due to tidal or seasonal effects
- Construction procedures, such as groundwater control method
- Temperature changes, such as freezing of the ground, softening of asphalt
- Sinking ground during surveying, caused by rapid tunneling process

Collimation errors. The error could occur when the axis of the instrument telescope is not in exactly horizontal plane. Although modern digital equipment has self leveling devices, the equipment does not guarantee that the axis is in a totally level viewing plane. The error which relates to distance between instrument and staff however, can minimize by using two pegs test.

3.6.2 Deep Settlement Point

In order to measure vertical displacement in the deep section, Heave/Deep settlement point is one option that is available in the market. However, due to the characteristic of installation, the instrument is recommended for soft ground only.

3.6.2.1 Method of Installation

The instrument consists of a three-pronged anchor, a ¼” inner steel rod and a 1” outer steel casing. The inner steel rod is attached to the anchor and allowed to move within steel casing, as shown in Figure 3.25.

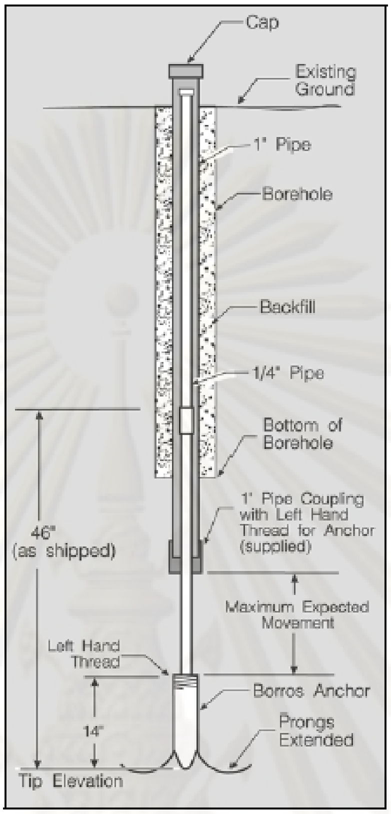


Figure 3.25 Typical section of settlement point

Typical method of installation is described below:

1. Drill a borehole to a depth of about 40cm above specified installation level. The borehole shall be cleaned with flushing water.
2. Thread the first section of ¼” steel rod with the ¼” steel rod already attached with the anchor. Tighten with wrench.
3. Thread the special 1” dia. coupling, included in the package, with general 1” PVC pipe and tighten with wrench.
4. Drill a borehole to a depth of about 40cm above specified installation level. The borehole shall be cleaned with flushing water.
5. Thread the first section of ¼” steel rod with the ¼” steel rod already attached with the anchor. Tighten with wrench.
6. Thread the special 1” dia. coupling, included in the package, with general 1” PVC pipe and tighten with wrench, Figure 3.26.

7. Drill a borehole to a depth of about 40cm above specified installation level. The borehole shall be cleaned with flushing water.

Figure 3.26 Coupling and anchor of deep settlement point

8. Thread the first section of ¼” steel rod with the ¼” steel rod already attached with the anchor. Tighten with wrench.
9. Thread the special 1” dia. coupling, included in the package, with general 1” PVC pipe and tighten with wrench, Figure 3.26.
10. Apply grease on the thread of anchor and slip the 1” pipe with coupling over the ¼” rod. Hand-tighten to the anchor as the pipe shall be detached later.
11. Lower the anchor and pipe to the borehole. Connect additional steel rod and PVC casing if necessary.
12. When the anchor has reached the bottom of borehole, push the 1” dia PVC pipe into the ground until reaching the specified level.
13. Secure the top of 1” PVC pipe and push the inner steel rod for about 20 cm to activate the prongs, also shown in Figure 3.27.
14. Detach the PVC pipe by turning in a clockwise direction, at least 15 complete revolutions.

Figure 3.27 Prongs of deep settlement point

15. Lift the PVC pipe upwards, at least 20 cm or slightly higher than the expected heave value.
16. Withdraw the casing, if used, and backfill around the PVC pipe with bentonite grout.
17. Protect the exposed part with visible protective measure.

3.6.2.2 Readout Method

An optical survey is used to measure the elevation of inner rod. Changes in elevation present an equivalent magnitude of displacement at the anchor. Care shall be taken in reading process as similar errors of surveying as mentioned above can also be found. The level of inner rod, later, has to be compared with reliable benchmark to produce absolute vertical displacement.

3.6.3 Magnet Extensometer (Spider)

Magnet extensometer is normally installed in the borehole and used to measure the heave or settlement of the ground due to operation of construction such as deep excavation, tunneling and mining. Series of magnet extensometer are usually installed together with inclinometer in order to determine both vertical and horizontal ground movement, Figure 3.28 Data obtained from the instrument can present the displacement zone and absolute displacement.

Figure 3.28 Spider used in the project

3.6.3.1 Method of Installation

General components of magnet extensometer are access pipe, datum magnet and spider magnet. Access pipe is made from PVC pipe and normally available in three sizes: 1, 2.75 and 3.34 inch. Spider magnet, which named for spring-steel legs as shown in Figure 3.28, is installed in borehole at specified depth to measure the vertical movement of the ground. The spider magnet, consists of Superlene retaining ring, magnet, duct tape and spring legs, is usually attached to access pipe and installed with the pipe. Datum magnet is used as a reference and is fixed directly to the bottom section of the access pipe. Therefore, the bottom of the pipe is required to be in stable ground.

The datum magnet is normally positioned at least 0.5 m above the bottom of the pipe. The equipment, which contains two retaining rings, can be glued to the access pipe in order to provide maximum security otherwise; the instrument can be locked by tightening the set screws, as shown in Figure 3.29.

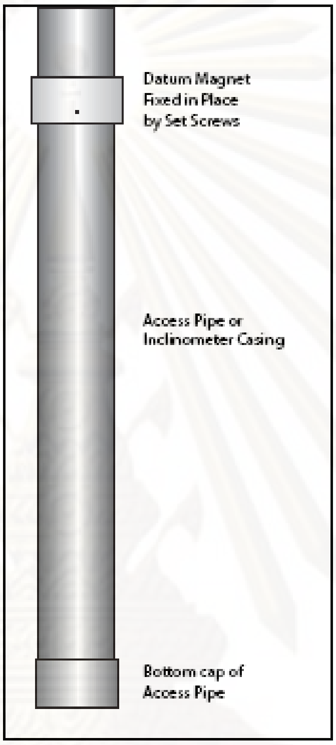


Figure 3.29 Installation of datum magnet

Spider magnets can be installed in various ways. Originally, the magnets must be securely attached o the pipe, prior to the installation. After the pipe is in place, the spring steel legs of the spider magnet are released to fit it to the surrounding ground. Typical method of installation is as follow:

1. Mark order of installation and the location of each magnet on the pipe sections. Avoid locations close to the coupling.
2. Slide the spider magnet to the location.
3. Compress legs and bind them temporarily with safety ties. The ties will be removed later prior to the installation.
4. Prepare sufficient release cord and label the top end with magnet number or depth.

5. Wrap compression wire, as shown in Figure 3.30, around upper legs and lower legs of the spider. Push release pin through the upper loop, guide hole and lower loop.

Figure 3.30 Detail of compression wire

6. Wrap one revolution of vinyl tape at the top and bottom of release pin to prevent premature release during installation.
7. Connect release cord to the pin. Repeat for other magnets.
8. Install pipe with datum magnet in borehole.
9. Install next section of pipe and couple. Make sure that the safety ties are removed carefully before installation.
10. Before removing the release pin, check the location of each magnet with magnetic extensometer probe.
11. Pull drill casing to an elevation above the upper legs of the magnet. Then, pull release cord upward to release legs of the magnet.
12. Repeat above step until all instruments are installed.
13. Backfill the hole with soft bentonite-cement grout.

If the spiders are used with inclinometer, the inclinometer shall be installed first, as mentioned previously. Then, the magnet instrument shall be positioned prior to the backfill process. General installation method of the instrument is as follow:

1. Slip the magnet extensometer to the inclinometer.
2. Wrap compression wire around the upper and lower steel legs. Insert release pin through the loops of compression wire. Apply one revolution of vinyl tape at the top and bottom of release pin.
3. Connect release cord to the pin.
4. Slip the guiding pipe on the inclinometer. The guiding pipe shall be slightly larger than the inclinometer casing.
5. Push the guiding pipe downward until the magnet is in position.
6. Pull the release cord upward to fit the legs to the surrounding ground.
7. Uninstall the guiding pipe and repeat the installation process for the remaining magnets.
8. Backfill the hole with soft bentonite-cement grout.

3.6.3.2 Readout Method

The set of instrument contains probe, a steel measuring tape, a tape reel with built-in light and buzzer, and a number of magnets positioned along the length of an access pipe. The magnets are coupled to the surrounding soil and move up and down as heave or settlement occurs. Typical procedures are as follow:

1. Switch probe power on.
2. Lower probe to bottom of the pipe or inclinometer casing. Raise probe slowly until the buzzer sounds, Figure 3.31.
3. Then carefully raise the probe until buzzer sounds a second time. Determine the exact location from the reading tape and record depth of each magnet on data sheet.
4. The depth of each magnet is actually based on the top of access pipe. Therefore, the obtained data is referenced to the top of the pipe, rather than to the datum. The field data then, has to be inverted before calculating the ground displacement.

Figure 3.31 The process of measurement

3.6.4 Inclinometer

Inclinometer comprises of grooved PVC pipe in x-y axis. The grooved is used as a guide for tilt meter to move along the pipe in correct direction, Figure 3.32. Tilt meter will measure angle at every 0.5 m and send information to data logger. The data is then calculated to define relative and actual displacement.

3.6.4.1 Method of Installation

Check the depth of drill hole to insure the fix length of inclinometer pipe. The depth of installation shall also include the length of grout valve, weight, etc. The bottom of inclinometer pipe shall not move during measurement, in order to calculate correct actual movement of the ground.

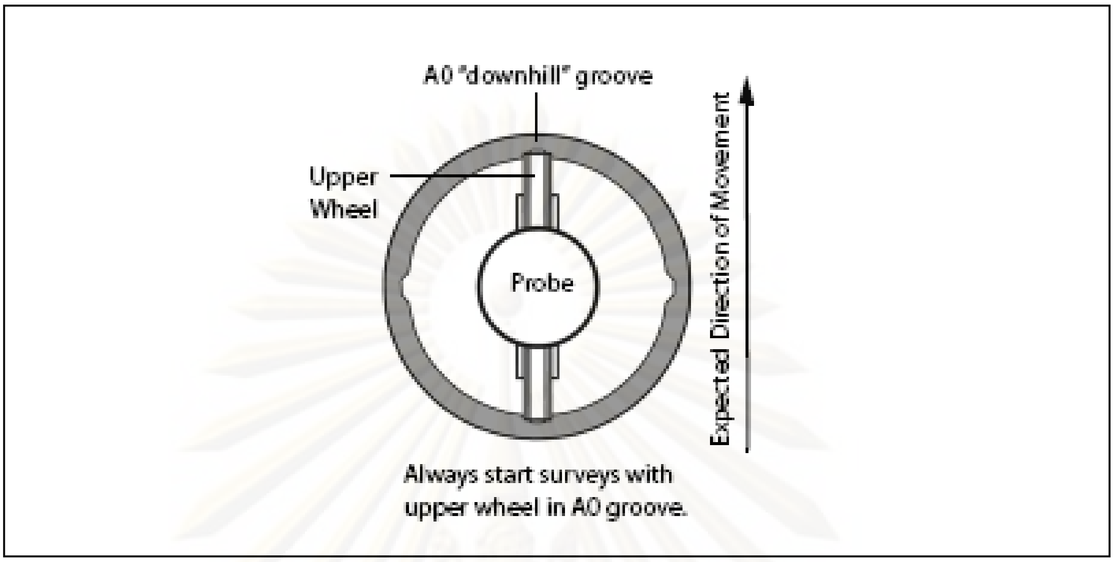


Figure 3.32 Location of groove in PVC pipe (Slope Indicator, 2005)

1. Install closed end pipe into the hole and assure that the groove is locates in the same direction of ground movement.
2. Operator shall use pier to lock the pipe while inserting the pipe into the hole, to prevent loosing of the PVC pipe.
3. Assembly next section of PVC pipe as shown in Figure 3.33. Make sure that key and key way is in position then, bolting all 4 sides of PVC pipe. Figure 3.34. Beware not to bolt the pipe at groove location.
4. Use pier to grip the upper pipe and ungrip the lower pier to prevent loosing of PVC pipe into the hole.
5. Repeat step 3-5 until finish.
6. If the hole is fully submerged with water, operator shall fill the pipe with water to increase weight of PVC pipe. Moreover, operator may add weight or steel pipe into PVC pipe during cement-bentonite grouting process, to prevent the pipe from floating. Weighting at the top of PVC pipe is not recommended as the installed pipe may be twisted.
7. Mixture of cement and bentonite shall not too thick or too light. Any admixture that may increase the temperature of mixture is prohibited as higher temperature may damage the pipe. In general, unconfined compressive strength of mixture at 28 days for soft clay and hard clay shall be 4 lbs/in^2 and 100 lbs/in^2, respectively.
8. Grouting work could be performed either before or after the installation of PVC pipe.

9. When the mixture is hardened, remove any excess pipe and install protection equipment.

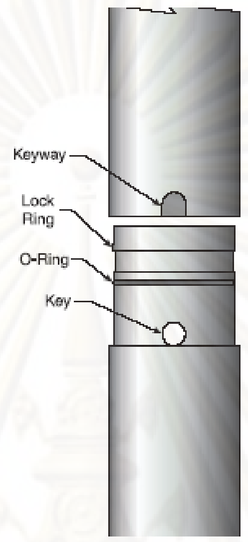


Figure 3.33 The key and key way at the end of each pipe (Slope Indicator, 1997)

Figure 3.34 Process of rivet installation

3.6.4.2 Readout Method

Monitoring can be performed by using tilt meter, comprises of waterproof Aluminum cylinder which contains two set of tilt meter, cable connector and wheels, as shown in Figure 3.35. The first tilt meter will measure angle in parallel axis to the wheel, also called axis A. The second one will measure angle in perpendicular axis to the wheel, also called axis B, with plus and minus signs.

Figure 3.35 General components of inclinometer probe

1. Assembly monitoring probe as specified by manufacturer.
2. Switch on data logger to prevent the probe from shock during insertion.
3. Insert the probe into PVC pipe, assure that the upper wheel is in downhill groove (groove A_0).
4. Slowly drop the probe to avoid hitting the end of PVC pipe. Once, the probe is at the end of the pipe, wait for 5-10 minutes to allow temperature inside the probe to match the surrounding temperature.
5. Pull the control cable to measuring depth and wait until figure on the data logger is constant. Then, record the data in both A and B axes.

6. The control cable has yellow mark at every 50 centimeters and red mark at every 1 meter, Figure 3.36. In every monitoring, operator shall use the same reference point and shall have error less than 5 millimeters.
7. Pull the cable to next level and repeat step 5 until reach the top of PVC pipe.
8. Remove the probe and re-insert the probe and assure that lower wheel is inserted into groove A_0.
9. Repeat step 4-7 again.
10. Clean the probe and put into the probe box.

Figure 3.36 Yellow and red marks on the cable of inclinometer probe

3.6.5 Pneumatic Piezometer

Various types of piezometer are available to monitor porewater pressure in soil such as Standpipe Piezometer, Pneumatic Piezometer, Vibrating Wire Piezometer. In this study pneumatic type which comprises of pressure sensor, double air tube and data logger are used.

3.6.5.1 Method of Installation

1. Drill the hole at specified location, the hole diameter shall larger than pressure sensor. If sand bag is used as filter layer and weight, the hole and casing shall be larger than diameter of sand bag, Figure 3.37.

Figure 3.37 Piezometer probe and sand bag

2. After drilling process is complete, lift casing up about 10 centimeter and fill up the hole with filter material.
3. Repeat step 2 until reach the monitoring level then, install pressure sensor.
4. Repeat step 2 again until the thickness of filter layer is more than 150 millimeters above the sensor.
5. Construct bentonite seal layer by slowly fill up the hole with granule or palette bentonite into the hole, Figure 3.38. When thickness of bentonite layer is about 300 millimeters, operator shall wait for 2-3 hours or as specified by manufacturer. Water may be added into the hole, to prevent bentonite from absorption of water from surrounding ground.

Figure 3.38 Palette bentonite used for piezometer installation

6. Backfill the hole with mixture of cement and bentonite.
7. If monitoring is carried out during installation period, readout value will not stable due to disturbance of surrounding ground and backfill pressure. The pore water pressure will decrease gradually to normal level within a few days but also depend upon thickness of filter layer and bentonite. Hence, operator shall measure the pore pressure periodically before monitoring the actual pore pressure.
8. After installation process is complete, roll the air tube and keep the connection clean and dry. Mark the location and install damage protection equipment, Figure 3.39.

Figure 3.39 Protection cap of piezometer

3.6.5.2 Readout Method

Pore pressure sensor comprises of single movement part, diaphragm, as shown in Figure 3.40. Surrounding water pressure will act on diaphragm while air pressure will act on another side of diaphragm. Operator will measure the air pressure by connecting air tube to pressure gauge and allowing nitrogen gas to enter into the air tube. When air pressure is higher than water pressure, diaphragm will deflect and allow nitrogen gas to release to outlet tube. Once, nitrogen gas is recognized from the outlet tube, operator will close the value and allow pressure to decrease. Once, water pressure is equals to air pressure, diaphragm will close and not allow any gas to release. Operator then measures the remaining air pressure.

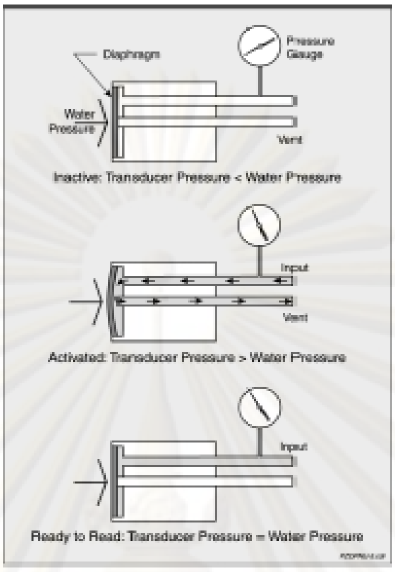


Figure 3.40 Component inside pressure sensor (Slope Indicator, 2000)

Typical monitoring method is as follow;

1. Prior to monitoring, operator shall check the pressure inside the tank. Minimum tank pressure shall be 35 bar or 500 lbs/in^2.
2. Inlet air tube is normally in black color and equipped with brass connector. Connect the hose to the monitoring device, Figure 3.41.
3. Connect clear outlet tube into the device, excess air will lift the plastic ball in the tube upward.
4. Open valve of nitrogen tank and then gradually open pressure adjustment valve.
5. Gradually open pressure valve and allow air pressure to increase at the rate of 1 lb/in^2/second or 0.05 bar/second. If the sensor is installed at very deep level, monitoring process could take long time. Anyway, operator shall not adjust pressure valve as excess gas will be loss and operator may damage the diaphragm.
6. When operator notices gas at the outlet tube, close the pressure valve.
7. Once, the figure on pressure gauge is constant, record the value.
8. After monitoring is complete, close the tank valve and open the pressure valve to release any remaining pressure. Finally, close the pressure valve.

Figure 3.41 Monitoring equipment of pneumatic piezometer

3.7 Collection of Monitoring Data

Installation of instrument and collection of data are performed, according to TBM operation. Monitoring information is then collected by subcontractor and surveyor, after the machine has reached the area of test section. All monitoring data, since the beginning of the project, will be recorded in the progress report. Specific criteria of some instruments which require attention during installation and monitoring are

- Network of reliable benchmarks (every 500m approx.) shall be set up and prevented from external disturbance. A benchmark should be installed on stable area that will not move over the period of monitoring program. For example, the benchmark should be located at least 2 times of tunnel springline level and at least 2 times of tunnel diameter below the tunnel invert (inclinometer case).

- All instruments and monitoring devices shall be calibrated by manufacturer or manufacturer’s representative within 6 months before the beginning of installation and monitoring program.

- According to monitoring result of test section 1, all pneumatic piezometers shall be installed at least 14 days prior to the arrival of TBM within a distance of springline depth, i.e. 30m, from the instrument.

- Same surveyor shall monitor same instrument with the same readout unit throughout the period of monitoring to prevent error from reading, unless the device is damaged.

- Initial reading shall be carried out at a minimum distance of 1.5 times of springline depth before arrival of TBM. At least three sets of reading shall be obtained from the initial reading.

- All geotechnical instruments shall be monitor with the following frequency.

  - 30-60m before arriving the test section, every 6 rings
  - 0-30m before arriving the test section, every 3 rings
  - 0-30m after passing the test section, every 3 rings
  - 30-60m after passing the test section, every 6 rings
  - Then, every week for two weeks
  - Then, at 3 months and 6 months
  - Surface settlement point shall be read every morning and evening of each day during period of monitoring

After receiving secondary information, data will be reviewed to assess quality of measurement and compared to theory and TBM operation in the next stage. Other related information is geological profile, ring number plan and tunnel alignment plan.

3.8 TBM Specification

The earth pressure balanced shield machine used in the project was manufactured by Mitsubishi Heavy Industries Limited in Japan. The excavation diameter is 5310mm while the overall length is about 10650mm. Three rows of wire brush are installed at the tail section. Eighteen units of hydraulic cylinder can provide a thrust force up to 27000kN. The stroke length of each cylinder is 1850mm and has an extending speed of 5.8 cm/min. The 440kW electric driven motor can rotate cutter-head either in a clockwise or counter-clockwise direction at a maximum speed of 0.94rpm and can provide a maximum torque of 5340kNm. Each spoke type cutter-head is equipped with a tungsten carbide tip.

The machine is also equipped with an articulate shield which has 16 hydraulic cylinders and can supply a maximum force of 24000kN. The maximum stroke of the cylinder is 230mm which can create ±2 degrees deviation and hence, it can provide a minimum curve of 80m. A screw conveyor with a shaft can rotate from 1.2 to 14.0rpm with a maximum torque of 30kNm. The internal diameter of the casing is 600mm and therefore, the capacity of the conveyor is about 102 m^3/hr when η = 100%. The excavation ratio is then automatically calculated from the revolutions of the screw conveyor. A soil conditioning agent is recommended to avoid blockage of high-plasticity stiff clay (Mair and Jardine, 2001). Foaming injection is typically necessary to prevent adhering on the cutter-head and chamber (Obayashi, 2006). In the project, foaming agent is mixed with water at the rate of 3% and used as slurry material. The conditioning agent is injected via four custom-made injection ports in front of the TBM. As the machine moves forward, grout material is injected via holes in the segments to fill the gap between the segments and surrounding soil. The injection hole and curve bolt are then covered by non-shrink material.

3.9 TBM Operation Record

Information of TBM along the whole route of tunnel is normally recorded by software called ARiGATAYA Viewer, as shown in Figure 3.42. The program can summarized information during each stage of operation and export to Excel format.

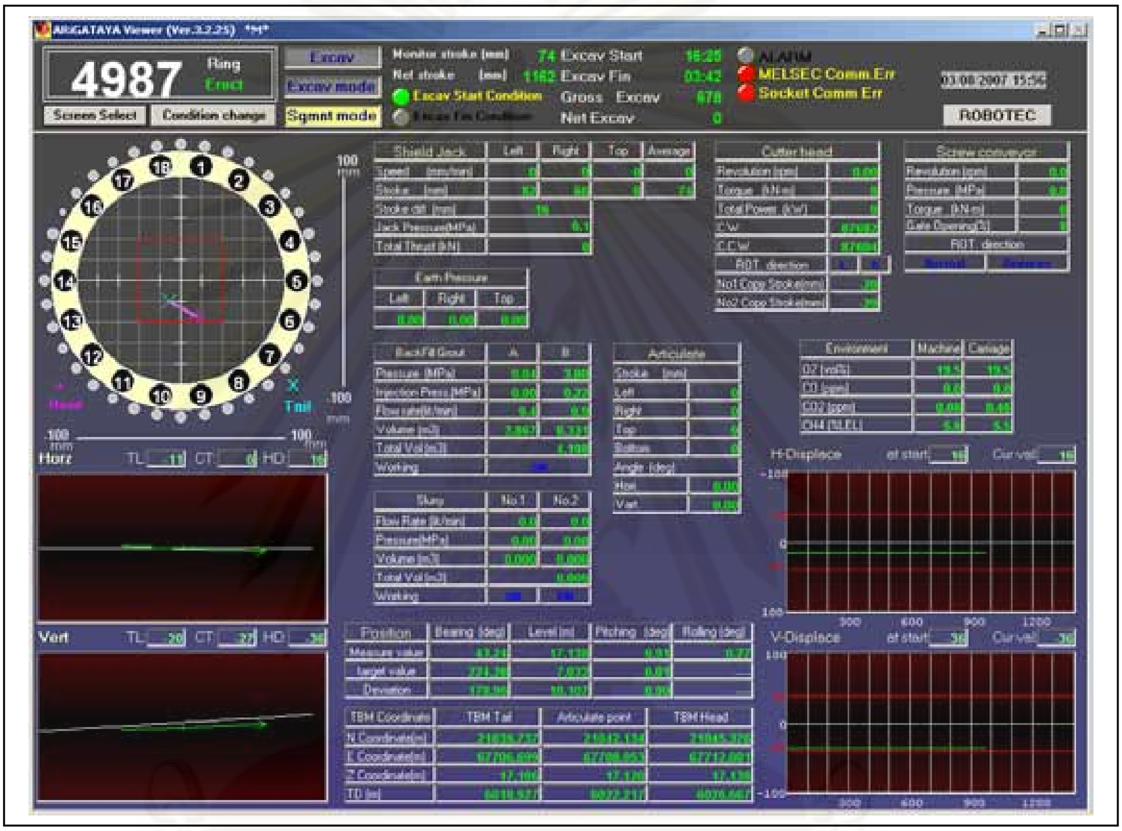


Figure 3.42 User interface of ARiGATAYA Viewer

According to Figure 3.42, the program provides basic information related to TBM operation as follow

- Ring Excavate/Erect is the current process of construction either excavation or erection at particular ring number.
- Excav Start is the start time of operation at particular ring number.
- Excav Fin is the finish time of operation at particular ring number.
- Gross Excav is the total gross volume of excavation soil up to current time.
- Thrust Jack Operating Panel presents the active thrust jack of the machine which will change to red circle during operation. The panel also presents the location of head (magenta plus sign) and location of tail (cyan X cross sign) on the screen, as shown in

Figure 4.22. The screen also presents graphic direction of TBM in both horizontal and vertical axes.

- Shield Jack Panel shows two significant data – shield jack speed and total thrust. Shield Jack Speed is extension speed of hydraulic thrust jack which the average value also equals to TBM velocity in mm./min. Total Thrust is the total force of thrust jack in kN unit.
- Earth Pressure Panel presents the result from pressure gauges which installed at the left, the right and the top of TBM machine in MPa.
- Cutter Head Panel presents important information of cutter head including revolution (rpm) and torque (kN-m) which related to performance of TBM operation (Muangsan, 2001). Basic information such as number of rotate direction is also shown in this panel.
- Screw Conveyor Panel presents status of screw conveyor during the operation such as revolution (rpm) and torque (kN-m) which provide status screw during operation. In any case that the muck has low plastic fluidity or pass the gate as a block, the screw revolution will decrease while torque of screw will increase. The panel also presents percentage of gate opening which controls the pressure of soil (MPa) in the chamber (Muangsan, 2001).
- BackFill Grout Panel provides information of two injection grout pumps, cement milk and additive pumps, such as grout pressure (MPa), flow rate (l/min.) and total volume of grout material (m^3). The important information is grout pressure and total volume which directly relate to magnitude of ground movement around the tunnel (Chanchaya 2000 and Muangsan, 2001).
- Articulate Panel shows information of articulate jack used for sharp turning of TBM. Information is the stroke length of jack (mm) which equipped inside of the machine, and horizontal/vertical degree of the front shield.
- Slurry Panel provides information of slurry pump that used to seal the tail brush during operation.
- Position and TBM Coordinate Panel presents information used for control TBM direction including bearing of TBM (deg), current level (m), pitching and rolling of TBM (deg). Moreover, the panel also described current coordination which is measured every minute by Automatic Guidance System named Robotec.
- Environment Panel. The program also provides safety information of gas content in both machine and tunnel during operation. Significant parameters include percentage

of Oxygen (%), Carbonmonoxide (ppm), Carbondioxide (ppm) and percentage of Methane (%LEL).

3.10 Data analysis

As mentioned in the beginning of this section, after field data had been collected from first two monitoring sections, the information will then be used to adjust TBM parameters and also used to validate the FEM parameters. Result of validated analysis shall provide appropriate guidelines for tunnelling parameters as well as trigger value for the critical part of the project.

3.10.1 FEM Parameter Validation

Time limitations and MRT criteria have forced the main contractor to the edge. Therefore, numerical modelling is necessary to estimate the possible effects of construction and displacement of existing tunnels. However, available numerical software also has a limited capability to model tunnels in three dimensions. Moreover, three-dimensional analysis also requires large computer resources and calculations over a long time, while time is critical. Hence, two-dimensional analysis was originally selected for the project.

Although two-dimensional FEM has been accepted for the prediction of tunnel behaviour, simplification of the problem is concerned for this particular case. The nature of two-dimensions is absolutely limited to plane strain analysis, while the actual tunnel axis is not perpendicular to the MRT axis. The setting up of the model is completely on a single plane. The modelling of structure and activity in the third dimension has to be simplified based on input from the designer (Teparaksa, 2005b). Furthermore, the construction sequence is also simulated directly in all slices, which may provide exaggerated results in some cases. Actual construction normally varies and contains uncertainty. Therefore, simulation may be completely incorrect.

Calibrated 2D FEM with actual field information can provide good estimations of prediction (Negro and Queiroz, 1999). Hence, typical numerical parameters will have to be calibrated with the first two monitoring sections, in order to compensate for the above reasons. According to previous studies of the tunnelling in the Bangkok area, it is not necessary to concern all tunnelling parameters in the numerical process (Teparaksa et al., 2004 and

Phienwej et al., 2006). Two dimensional FEM with Mohr Coulomb failure criterion has been confirmed to be appropriate for the prediction of tunnel behaviour for the MRT project in Bangkok (Prinzl and Davies, 2006 and Teparaksa, 2005c). The non-linear relationship between soil stiffness and strain, which was considered by assuming the relationship between Young's modulus and shear strain, varies from 0.1-1%, as shown in Figure 3.43.

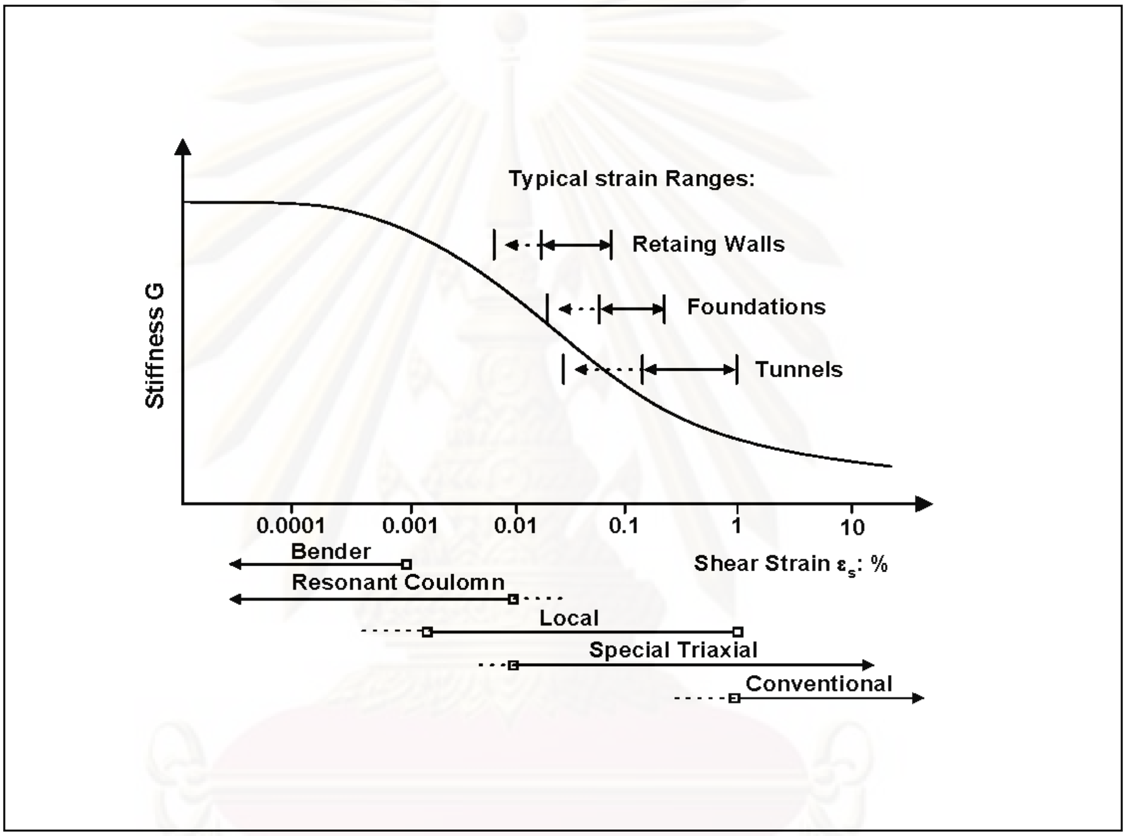


Figure 3.43 Typical shear modulus and shear strains for foundation works (Mair, 1993)

3.10.2 Numerical Prediction and Trigger Level Evaluation

After FEM parameter is validated, the prediction shall provide a close conservative result of the prediction. Negro and Queiroz (1999) shows that FEM can provide a rather accurate prediction of both surface and subsurface settlements. Figure 3.44 presents the numerical model used for displacement prediction during the boring of the diversion tunnel underneath MRT subway tunnel.

In actual conditions, the MRT tunnel is connected to the station by an omega joint which allows the tunnel to move freely. Hence, the finite element tunnel is connected with the station by a hinge connection. Khlong Toey Station and the MRT tunnel are included in the model and assigned as beam elements. The MRT tunnel is located at the tunnel spring-line, in order to represent the average vertical movement of the whole tunnel. On the boundary, the existing tunnel is allowed to move in a vertical direction only while the station is fixed in all directions. Surcharge load is also applied on the surface to represent the traffic load of Rama IV road.

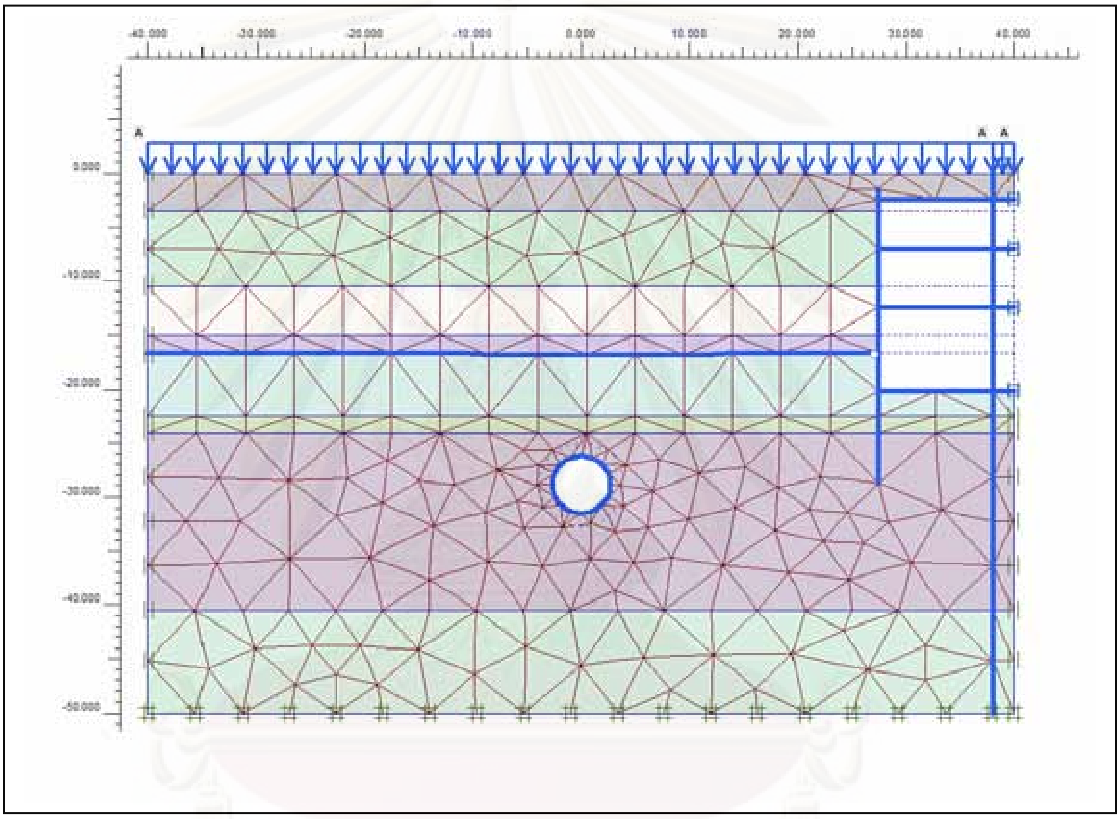


Figure 3.44 Example of numerical model for movement prediction of MRT tunnel.

After the analysis is complete, the prediction value of each measurable point will then be used to setup three levels of trigger value, called Alert, Alarm and Action level. Normally, the value of Alert, Alarm and Action level are 80%, 90% and 100% of prediction value. If actual monitoring data has reached Action level, the main contractor shall use contingency plan for the construction.

3.10.3 TBM Operation Criteria

In order to limit ground movement around the MRT tunnel, proper TBM operation is important. Thus, ranges of TBM parameters shall be specified before excavation into the critical area. Teparaksa et al., (2004) mentioned that operation parameters, such as trust force, soil excavation ratio and grouting technique, play a significant role in ground-TBM. Tunnelling parameters shall then be derived from the validated sections, e.g. Test Sections One and Two. Therefore, an average range of parameter values within ±60m from both test sections is recommended.

CHAPTER IV
RESULT OF MONITORING AND ASSESSMENT

Information of field monitoring data has been collected from first two sections for numerical validation purpose. Then, the conservative model shall be used for predict ground movement at the critical area and also provide trigger level for surveyor to monitor construction quality on site. Moreover, the information of field data and TBM record will also provide driving criteria for TBM operator for the rest section of the project. Finally, the movement of underground structure is discussed.

4.1 Field Monitoring at Validated Sections

Information of the field measurements from both Test Section One and Test Section Two are summarised in this section. Ground movement is discussed in relation to the TBM position, in order to determine the relationship between ground behaviour and the construction process. Primary factors of ground deformation, given by Mair and Taylor (1997), are examined; hence, significant construction parameters can be identified and controlled. The field monitoring results will also be used to confirmed numerical parameters and to predict ground movement at the Expressway and MRT section (Test Section 3 and 5).

4.1.1 Vertical Movement

Suwansawat (2006) studied the ground surface settlement of the MRTA Bangkok project and found that the longitudinal surface settlement can be divided into three distinct zones while Phienwej et al., (2006) reports that long term settlement is not significant in the stiff clay layer. The first zone is located in front of the TBM where the ground starts to deform and associate with driving parameters. The second zone develops over the shield body where the main ground movement occurs. Finally, the last zone, influenced by the grouting process, is located after the shield passing. Figure 4.1 shows the relationship between the TBM parameters and surface settlement along the tunnelling section. The x-axis represents the distance from the test section. Ground settlement slightly occurs from 60m, or 2 times the tunnel depth, in front of the test section. The movement substantially increases after the TBM has passed the test section. The magnitude of the settlement significantly increases until the cutter disc of the TBM has passed test section by 30m away, or equals to tunnel depth. Average ground settlement at Test Section 1 is about 3mm.

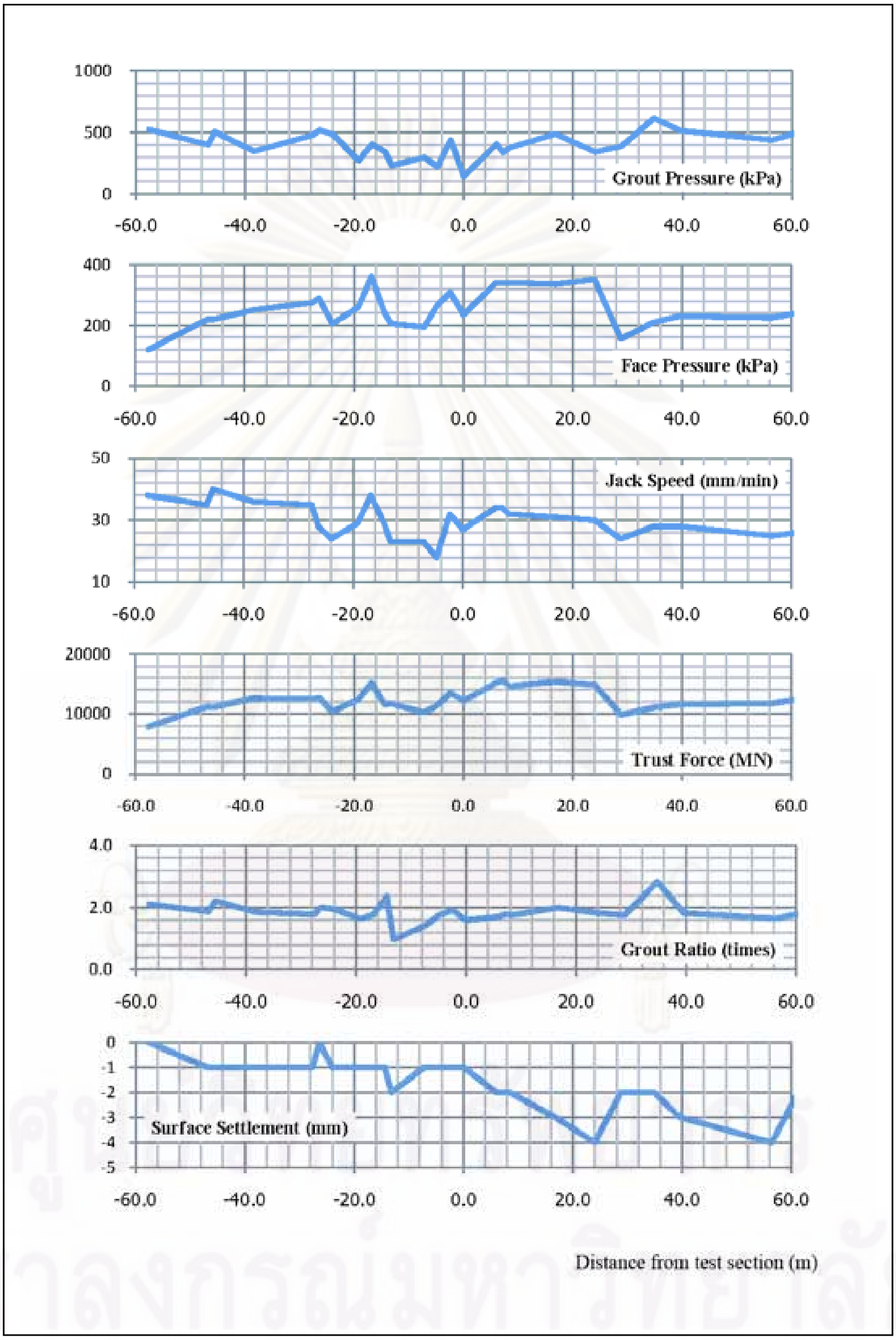


Figure 4.1 Surface settlement and TBM parameters at test section 1

Driving parameters, such as face pressure, jack speed and thrust force affect the magnitude of movement, Figure 4.1. When driving parameters increase, ground movement is decreased. Although the movement does not change as rapidly as the average face pressure, the vertical movement changes according to the pattern of face pressure. Increasing grout pressure and grout ratio also create considerable effects on settlement. When both parameters increase, ground settlement is decreased. After about twice the tunnel depth, ground settlement is rather stable. Figure 4.2 presents the results of deep settlement, installed in very stiff to hard silty clay at Test Section 1, and the TBM parameters. The anchor, 0.85m above the tunnel, slightly moves while the machine approaches within 10m or about 2 times of TBM diameter. The probe then dramatically moves after the machine has passed the test section.

Finally, the movement is quite stable when the machine has passed the anchor by 40m or about 33 grouting cycles. However, the deep settlement point, installed at 0.5m above springline level, has shown different characteristics. The second deep settlement point was installed at springline level, 3 m away from the edge of the TBM. The excavation causes the probe to move directly after the TBM has passed the test section. The vertical movement stops at about 10m or after the machine has passed the test section and the grouting process has started. Movement of both anchors could be justified by the ground on the side wall of excavation which has moved into the void before the soil above the machine. Vertical movements of both anchors are highly affected by grout pressure while TBM parameters are slightly affected, as shown in Figure 4.1 and 4.2.

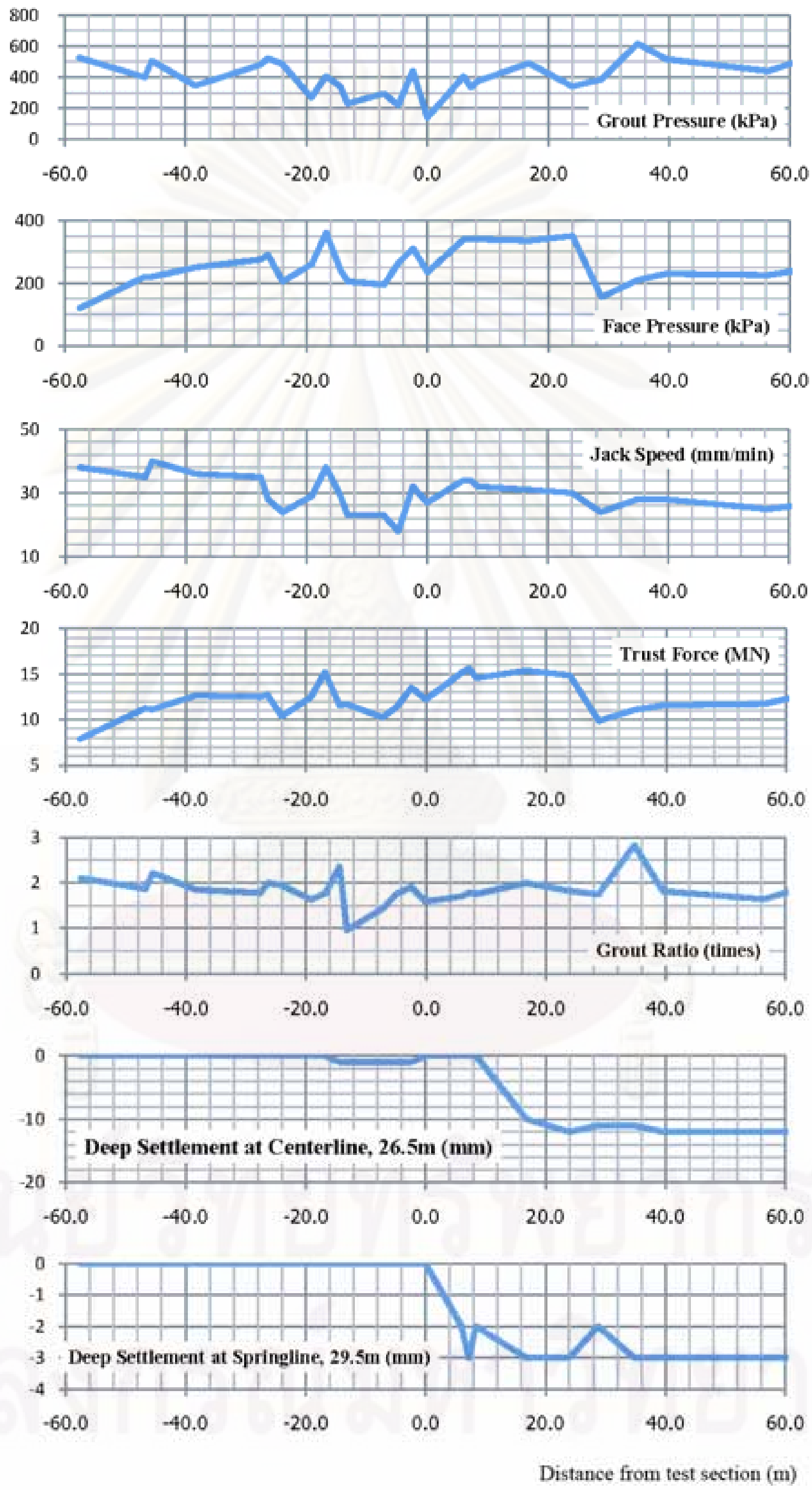
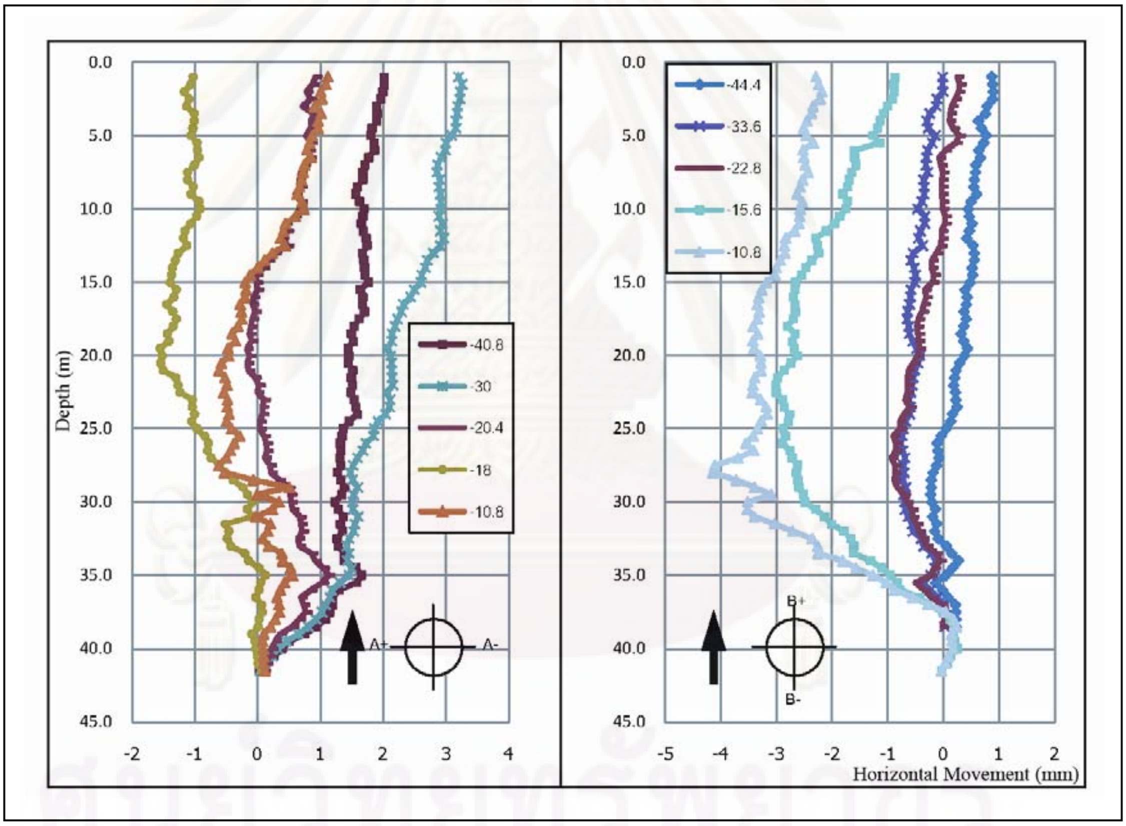


Figure 4.2 Results of deep settlement point and TBM parameters at test section 1.

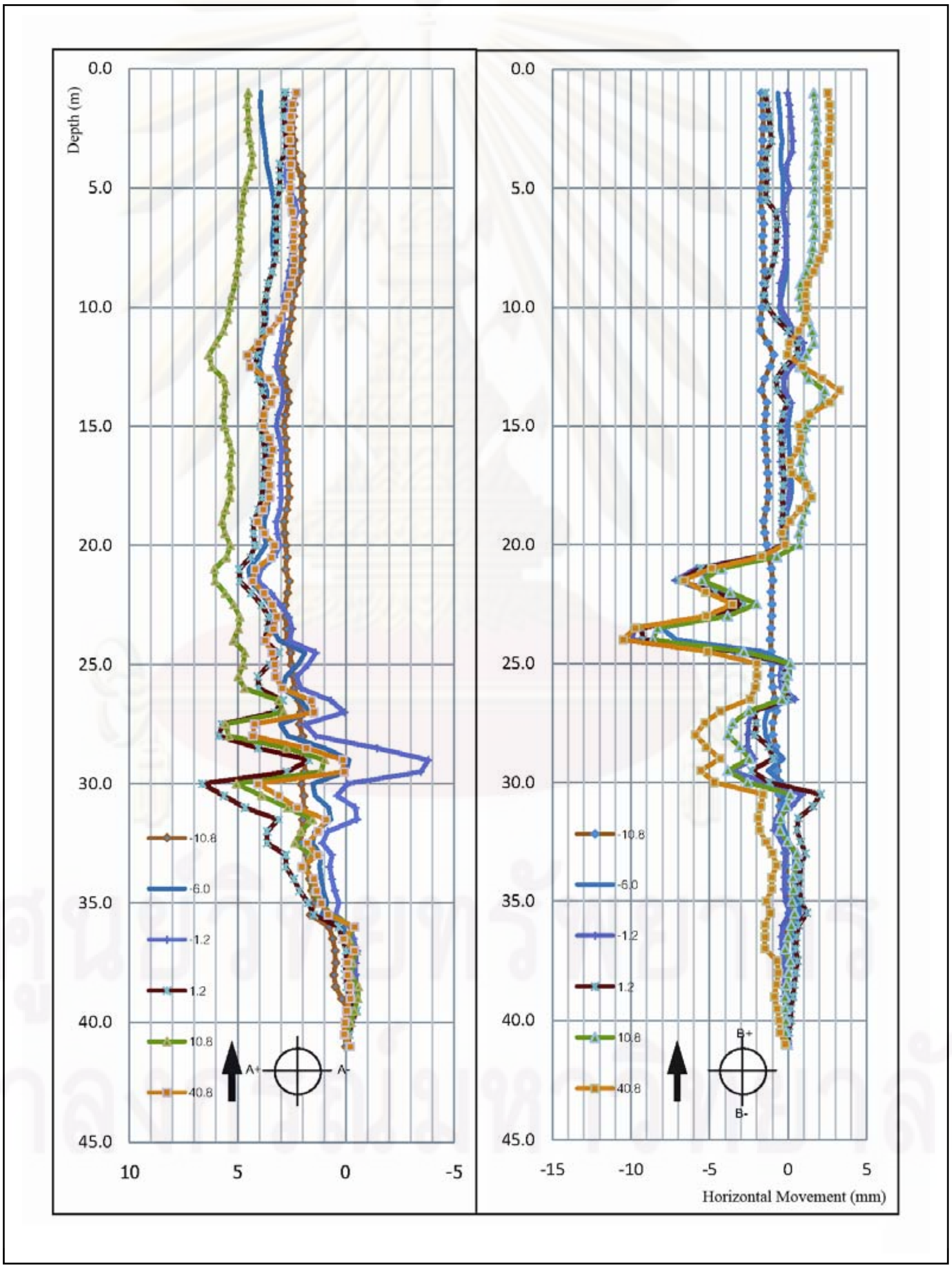
4.1.2 Horizontal Movement

Figures 4.3a and 4.3b illustrate the horizontal movement of the ground in front of the TBM prior to passing the test section. Horizontal movement, perpendicular to the tunnel direction, is minimal. However, the movement is likely to reflect the rotating direction of the machine cutter, Figure 4.3a. On the other hand, the horizontal movement, parallel to the axis of the tunnel, has actually moved forward to the TBM face, Figure 4.3b. The ground moves dramatically when the TBM cutter is about 15m away or three times of TBM diameter from the instrument. The profile of horizontal movement extends further below the invert level of the tunnel by 1.2 times of TBM diameter.



Figures 4.3a and 4.3b Horizontal movement of ground in perpendicular and parallel to tunnel axis at centre-line of tunnel.

Figures 4.4a and 4.4b show monitoring results of the inclinometer installed at 3.5m from the tunnel centre-line. Figure 4.4a presents the soil that moves forward to the TBM position when the cutter disc is about 10m away (about 2 times of TBM diameter). Most displacement occurs on the side wall and above the spring-line level. The movement stops after the cutter face has passed by 20m or about 16 cycles of the boring process. Soil particles have been pushed away from the TBM before moving back again after the cutter disc has passed, Figure 4.4b. The grouting process also causes the instrument to move outward by 2mm. Horizontal displacement of the ground that is parallel to the tunnel axis occurs dramatically when the TBM cutter is about 6m away. Movement of the soil particles is then very small after the arrival of the equipment.



Figures 4.4a and 4.4b Horizontal movement of inclinometer at 3.5m from tunnel axis

Figures 4.5a and 4.5b illustrate the horizontal movement of the soil from the inclinometer which is installed at 5.6m from the tunnel axis. The result from the A-A axis also confirms the characteristics of the previous instrument, i.e. the ground has clearly moved when the cutter face is about 1.2m away. However, the horizontal displacement on the parallel axis is decreased substantially when compared to previous instruments, Figure 4.5b.

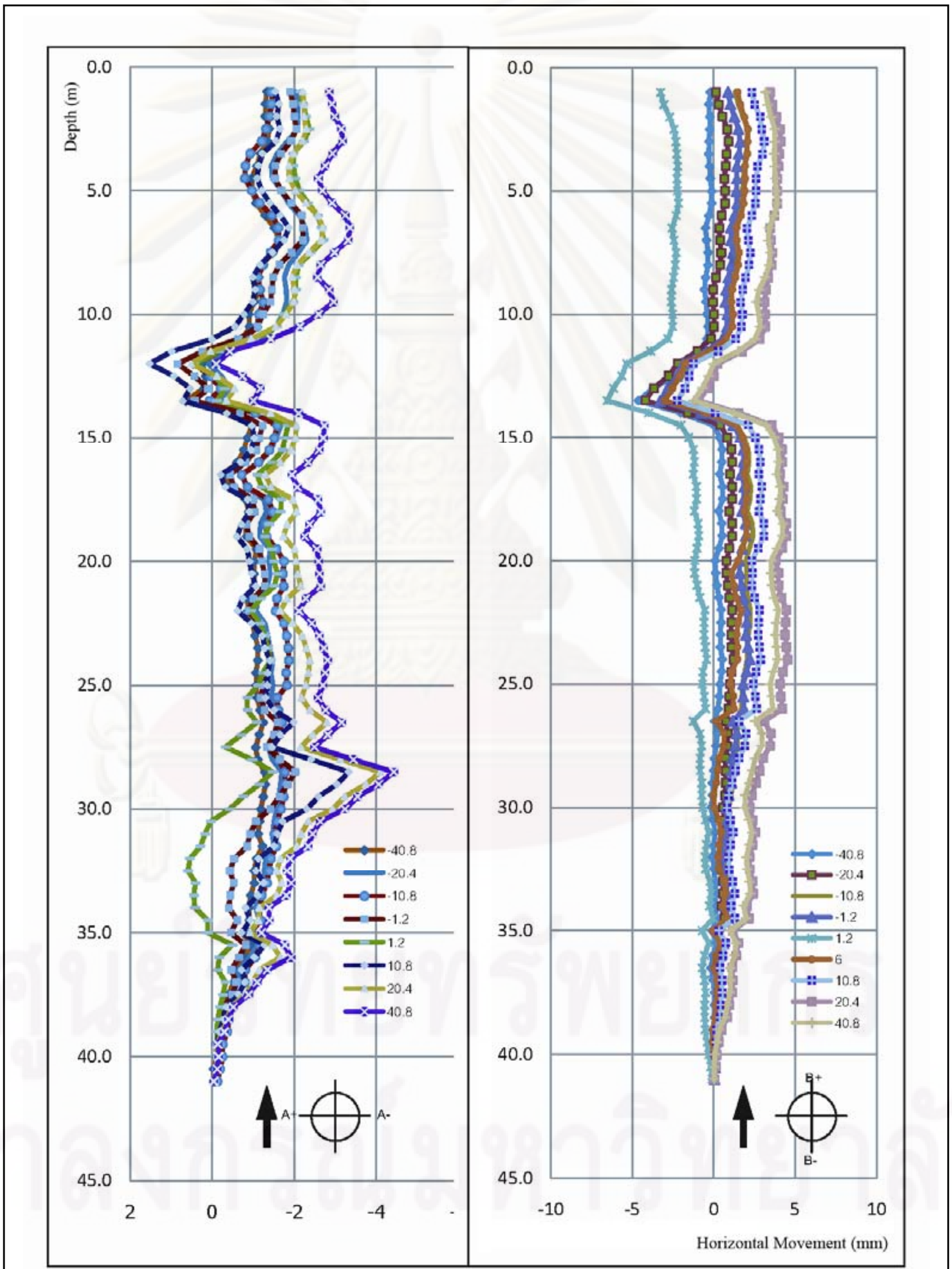


Figure 4.5a and 4.5b Horizontal movement of inclinometer at 5.6m from tunnel axis

4.1.3 Pore-water Pressure

Figure 4.6 presents the changing of pore-water pressure at the spring-line level and the TBM parameters of Test Section One. Pore-water pressure is substantially increased when the TBM is about 15m, about 3 times the TBM diameter, in front of the test section. The positive pore-water pressure then dissipates immediately after the cutting tool of the machine has passed the probe. The grouting process also causes pore-water pressure to slightly increase. Pore-water pressure gradually dissipates after the machine has passed by 30m or 25 boring cycles.

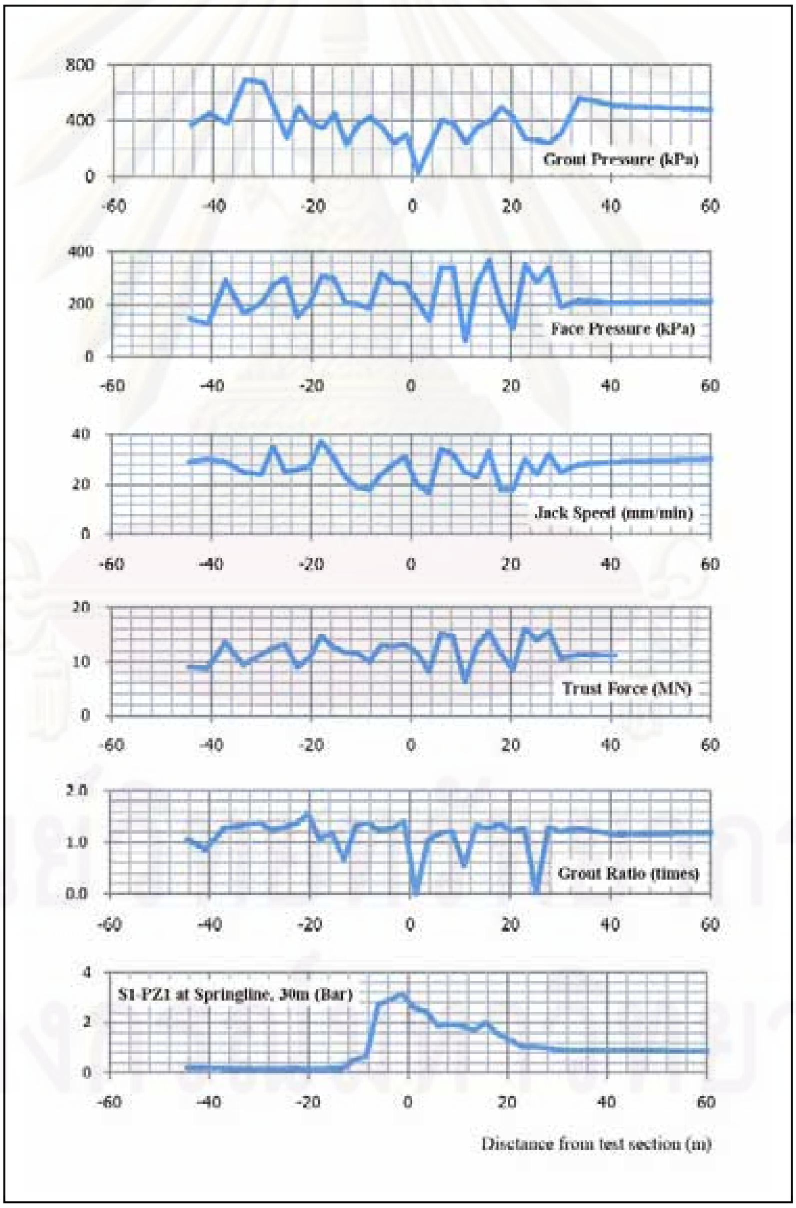


Figure 4.6 Measurement of pore-water pressure at test section 1

The pneumatic piezometer of Test Section One has provided good information of porewater dissipation after the installation process. Therefore, monitoring of the pore-water pressure around the tunnel is more comprehensive in Section Two. The piezometer on the side wall indicates that pore-water pressure is substantially increased, when the TBM is about 15m in front of the test section, Figure 4.7. The positive pore-water pressure is mainly caused by the increasing of the TBM parameters. The positive pore-water pressure then dissipates immediately after the cutting tool of the machine has passed the probe. The grouting process also causes the pore-water pressure to increase slightly. After the arrival of the TBM, any tunnelling activities beyond 30m from the piezometer also do not affect the dissipation of porewater pressure.

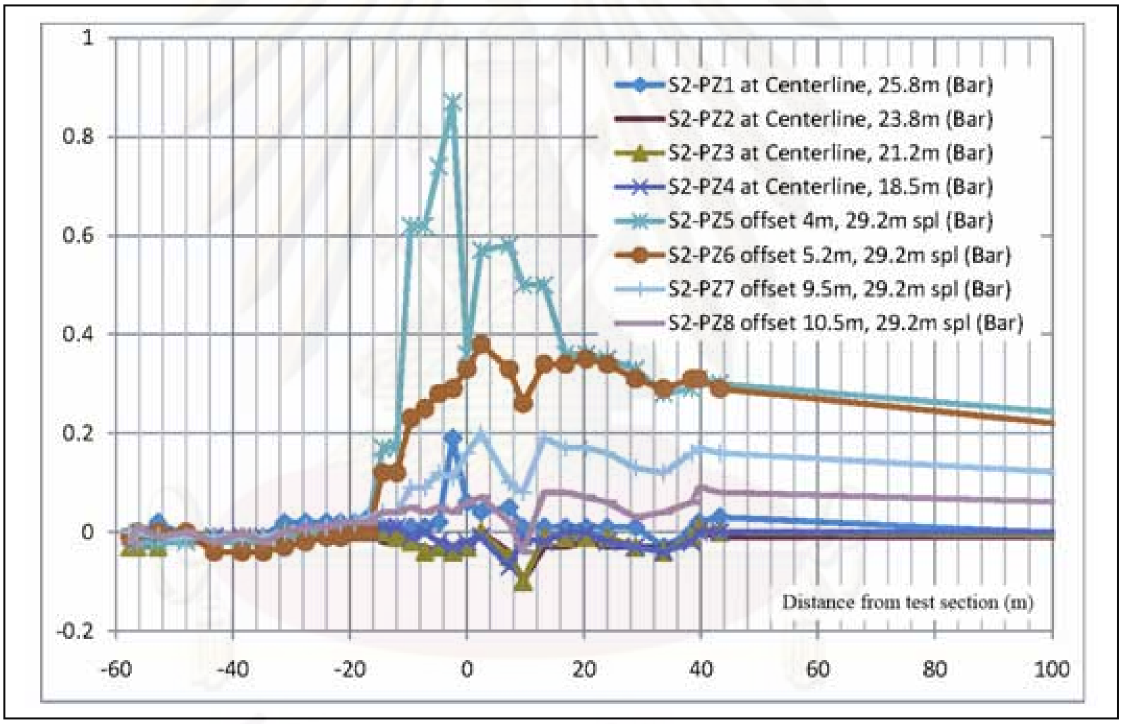


Figure 4.7 Changing of pore-water pressure along tunnel axis

According to Figure 4.7, the piezometer indicates that the positive pore-water pressure at 5.2m from the tunnel axis is only about 50% of positive pore-water pressure at 4m. Figure 4.8 presents maximum positive pore-water pressure at various distances from the TBM perimeter. Please note that all piezometers are installed at the spring-line level.

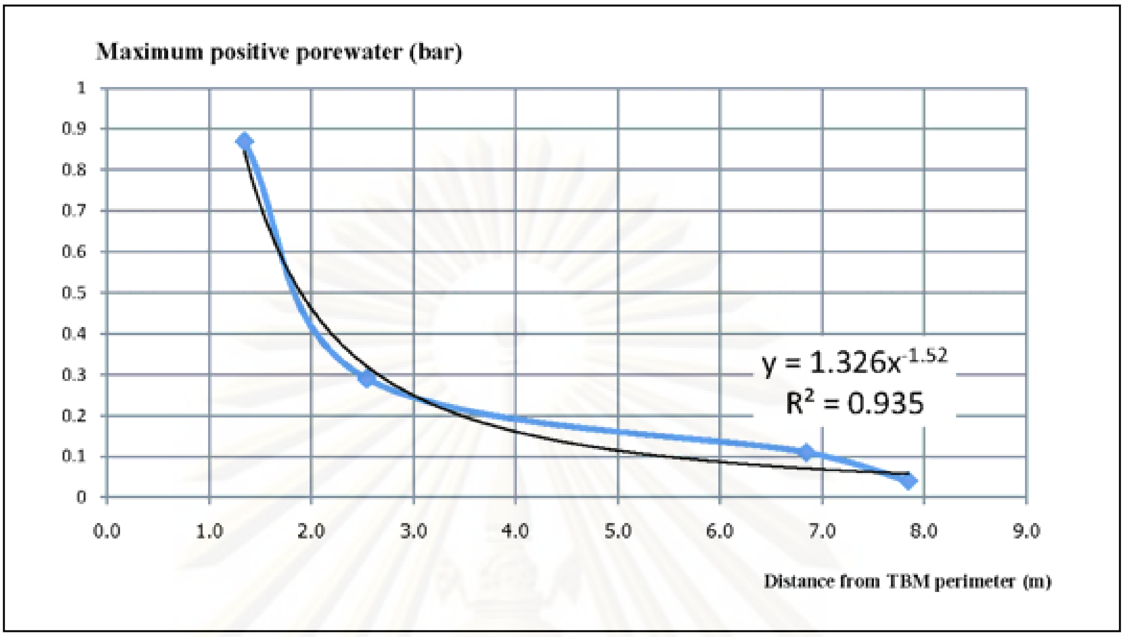


Figure 4.8 Distribution of positive pore-water pressure at tunnel spring-line level

Based on Figure 4.7, positive pore-water pressure, which is created by the TBM operation in Bangkok clay, can be calculated by the powered equation

$$u = 1.326x^{-1.52} \qquad (1)$$

where u is the positive pore-water pressure (in bar) at distance x from the TBM perimeter and x the distance (in meters) from the TBM perimeter to the location of piezometer.

Surprisingly, the TBM activities have less effect on pore-water pressure in a vertical direction, as presented in Figure 4.7. The piezometer above the crown of the tunnel indicates that pore-water pressure increases when the TBM face is about 8m from the instrument or nearly 50% of the trigger distance for pore-water pressure at the spring-line level. Moreover, the positive pore-water pressure at the crown is only about 25% of pore-water pressure at the springline level. Pore-water pressure mostly does not change at 0.5 times the TBM diameter above the crown.

According to all monitoring information, both initial and final locations of movement are plotted and compared to the TBM position. The typical influence zone of tunnel construction by TBM in green field of Bangkok subsoil could be defined as shown in Figure 4.9.

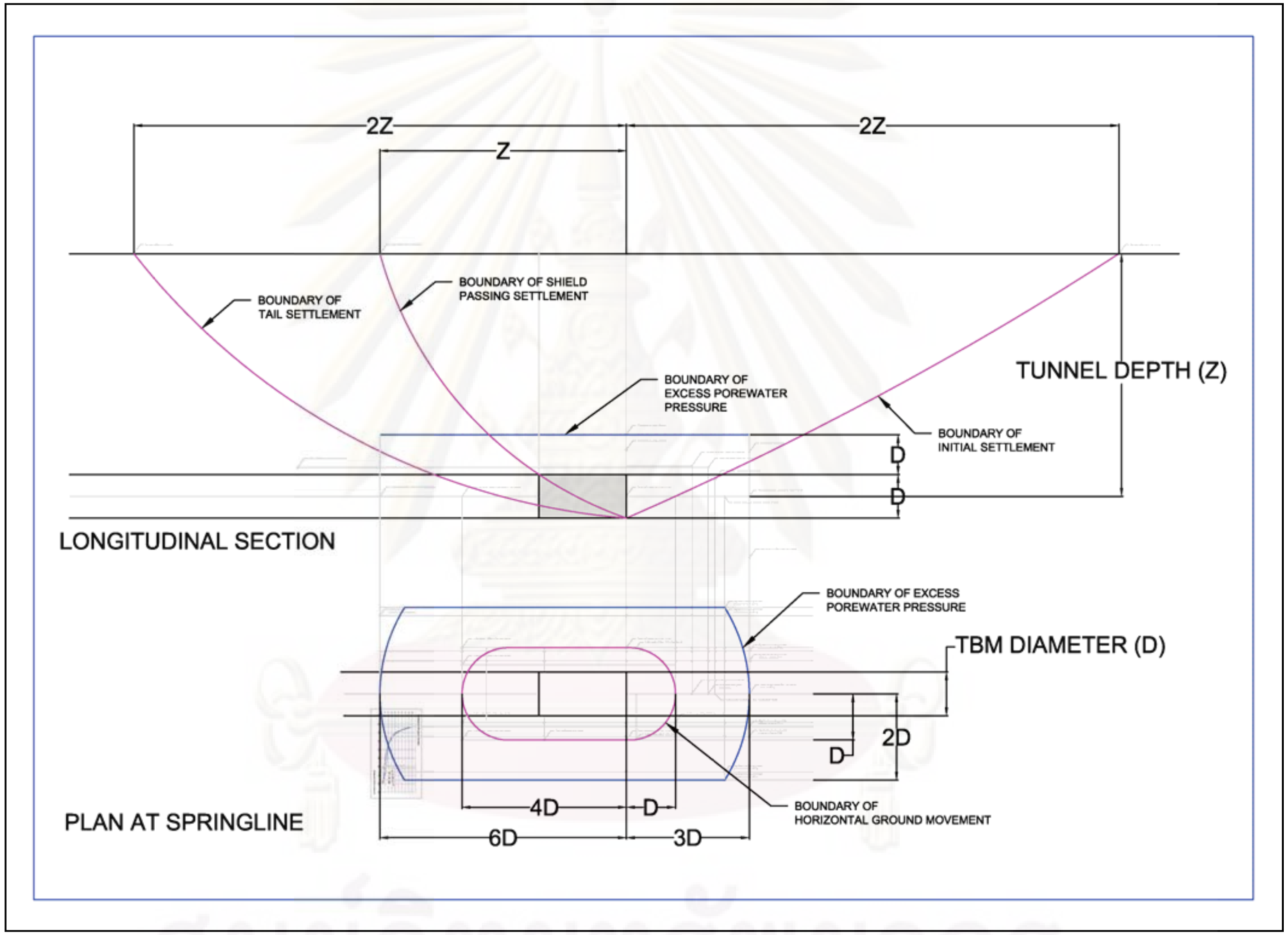


Figure 4.9 Influence zone of TBM operation in Bangkok subsoil

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According to Figure 4.9, boundary of initial settlement could be defined as two times of tunnel depth in front of TBM face, while the boundary of shield passing settlement could be described as one time of tunnel depth behind the TBM face. The boundary of tail settlement could be located at about two times of tunnel depth behind the TBM face. The boundary of ground movement, at spring line level, is about one time of TBM diameter in the front of TBM face, while the boundary behind the TBM is about four times of TBM diameter. The influence zone of horizontal ground movement is limited to about 0.5 time of TBM diameter around the TBM perimeter.

Furthermore, the influence zone of excess porewater pressure is limited to about one time of TBM diameter in vertical axis and limited to about two times of TBM diameter in horizontal axis. Moreover, similar to ground movement, most excess pressure is also fully dissipated over a short term period. The influence zone extends to about three times of TBM diameter, in front of TBM face, while the excess pressure could be fully dissipated within about six times of TBM diameter.

4.2 FEM Parameter Validation

The model was based on the 2-D FEM and the Mohr-Coulomb failure criterion. Teparaksa (2002) has analysed the results of the self-boring pressure meter test and developed the E_u/S_u ratio, based on the relationship between soil stiffness and the degree of shear strain. The modulus parameters of all soil layers were presented in terms of the ratio between modulus and shear strain (E_u/S_u), see Table 4.1.

Table 4.1. Geological condition and soil parameters along tunnelling path (Teparaksa, 2005b)

Depth (m)	Soil Description	Unit Weight (kN/m^3)	Su^a (kN/m^3)	$SPT\text{-}N^b$ (Blows/ft)	Eu^c/Su	E'^d/N (kN/m^2)
0.0-12.0	Soft Clay	16.0	15	-	240	-
12.0-15.0	Medium Clay	17.0	28	-	240	-
15.0-19.5	Stiff Clay	20.0	110	16	480	-
19.5-25.5	Very Stiff Clay	20.0	170	25	480	-
25.5-28.5	Very Dense Sand	20.0	-	50	-	2000
28.5-40.0	Hard Clay	20.0	250	30	480	-

Note: [a] Undrained Shear Strength [b] SPT-N Value [c] Undrained Young's Modulus [d] Effective Young's Modulus

In Mohr-Coulomb's model, cohesion and stiffness are normally constant values; however, in reality, both parameter values increase with depth. The functions of E_{inc} and C_{inc} are activated to compensate for stress increases per unit depth, based on reference points in the model, as shown in Table 4.2.

Table 4.2. Soil parameters for numerical prediction at MRT intersection area

Depth (m)	Soil Description	Unit Weight (kN/m^3)	Su[a] (kPa)	Cinc[b] (kPa)	N[c] (Blows/ft)	Eu[d]/Su	E'[e]/N (kPa)	Eref[f] (MPa)	Einc[g] (MPa)	Friction Angle (Deg)
0.0-1.1	Lateritic Soil	17.0			5		2000	10000		30
1.1-5.0	Silty Clay (CH)	17.1	16	-		240		3840		-
5.0-15.0	Clay (CH)	16.4	16-33	1.7		360		5760	612	-
15.0-22.0	Silty Clay (CH)	19.7	64-157	1.3		480		30720	640	-
22.0-39.0	Silty Clay (CH)	20.2	169	-		480		81120		-
39.0-43.0	Silty Sand (SM)	20.1			24		2000	48000		28

Note: [a] Undrained Shear Strength [b] C increment [c] SPT-N Value [d] Undrained Young's Modulus
[e] Effective Young's Modulus [f] Young's Modulus at reference point [g] E increment

Based on field monitoring, the adopted parameters are validated before applying them to numerical prediction. A critical criterion of the MRT tunnel is the displacement limitation. Therefore, the finite element model shall be verified by ground displacement (Teparaksa, 2004 and 2005d). Validation will concentrate on vertical movement as the settlement value is predominant displacement on site. Lateral displacement is then automatically validated at the same time (Teparaksa, 2005c). Hence, numerical results will be compared with surface settlement and the deep settlement point. Figures 4.10 and 4.11 present the vertical displacement of numerical analysis and field measurements at various depths.

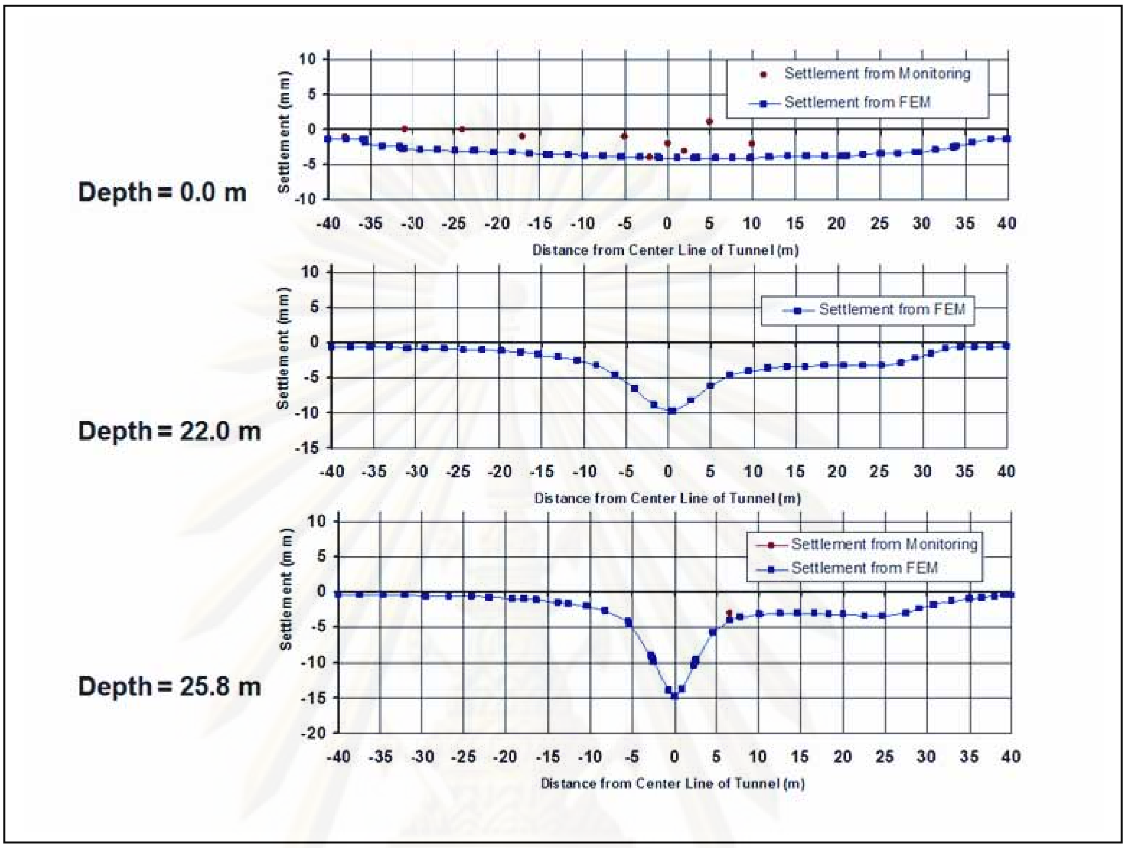


Figure 4.10 Settlement prediction by numerical analysis and field measurement at test section 1.

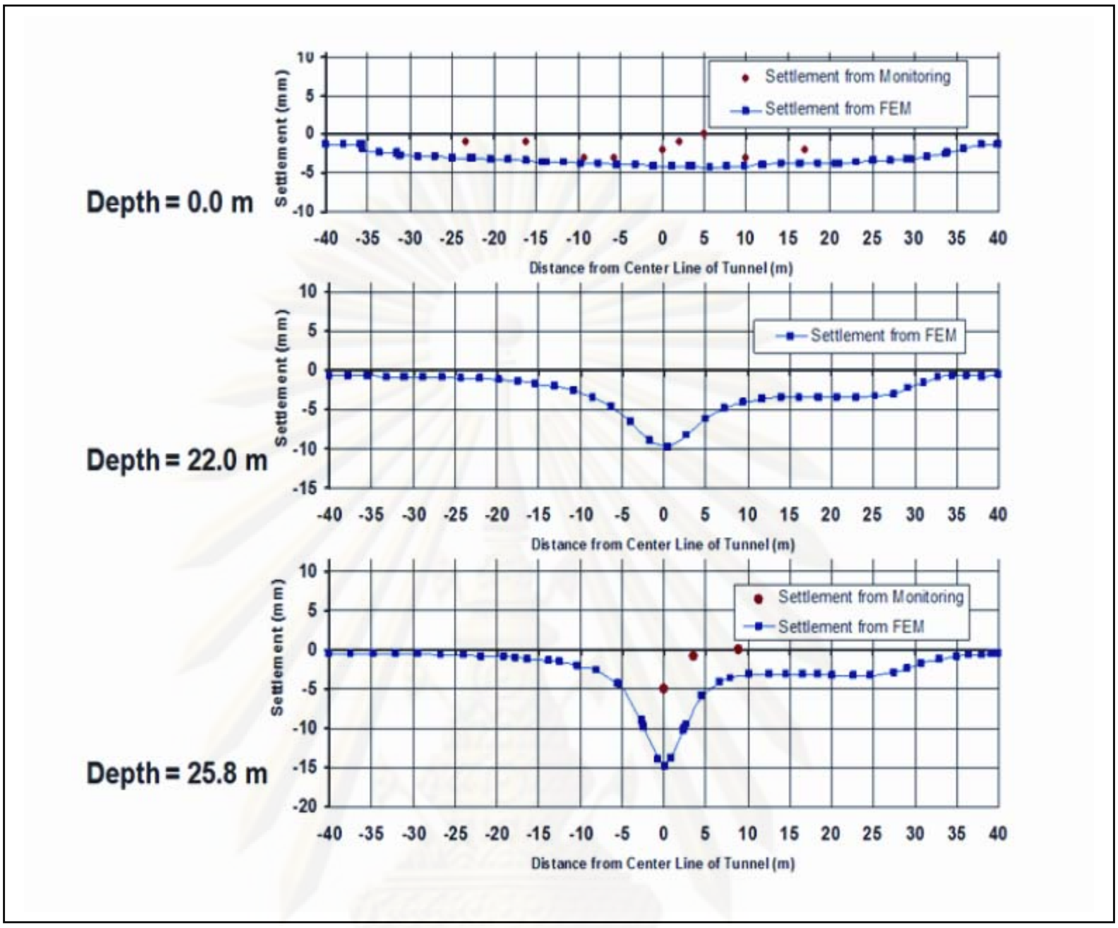


Figure 4.11 Settlement prediction by numerical analysis and field measurement at test section 2.

According to Figs. 4.10 and 4.11, the adopted soil parameters produce slightly larger settlement than actual measurement in both cases and hence, are considered conservative.

4.3 Numerical Prediction and Trigger Level Evaluation

4.3.1 The Expressway

Two-dimensional numerical study was carried out to assess effect of tunnelling on the pile foundation and MRT tunnel. Site characteristic is simplified to cope with capability of numerical software. The constitutive soil model is based on the elasto-plastic failure criteria of Mohr-Coulomb. Critical geological section was used for the analysis. Two sections of the pile foundation were examined, also shown in Figure 4.12a and 4.12b.

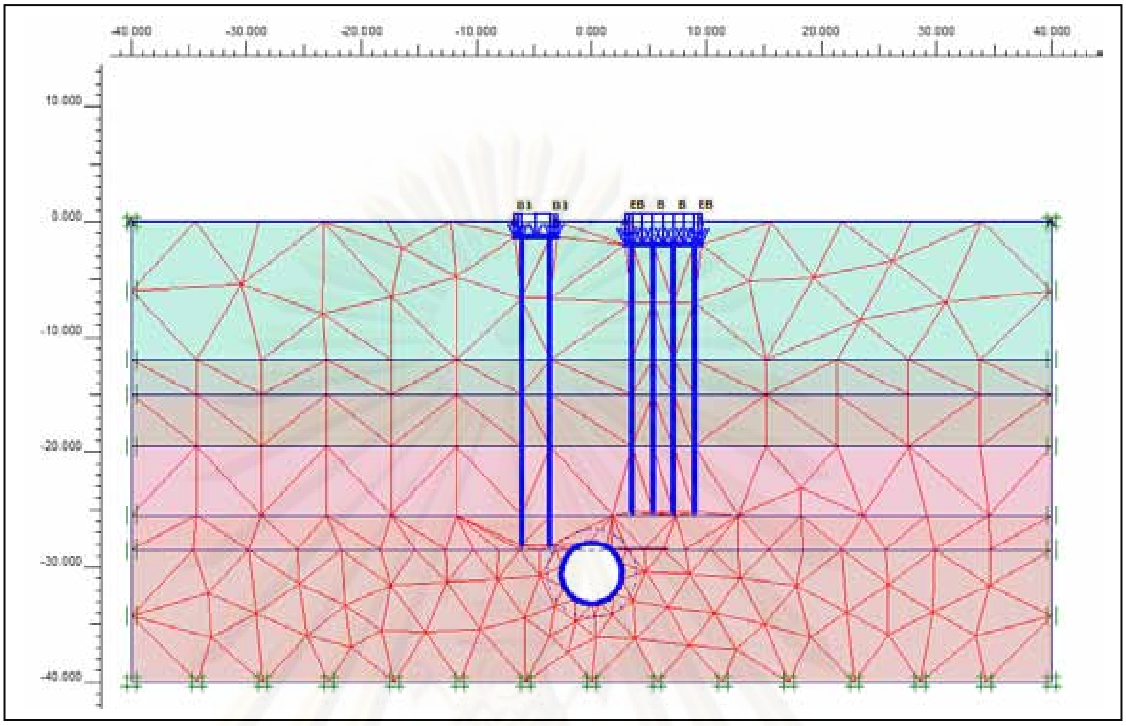


Figure 4.12a FEM at the expressway (from left: pile group no. X1-2 and pile group no. Y1-4)

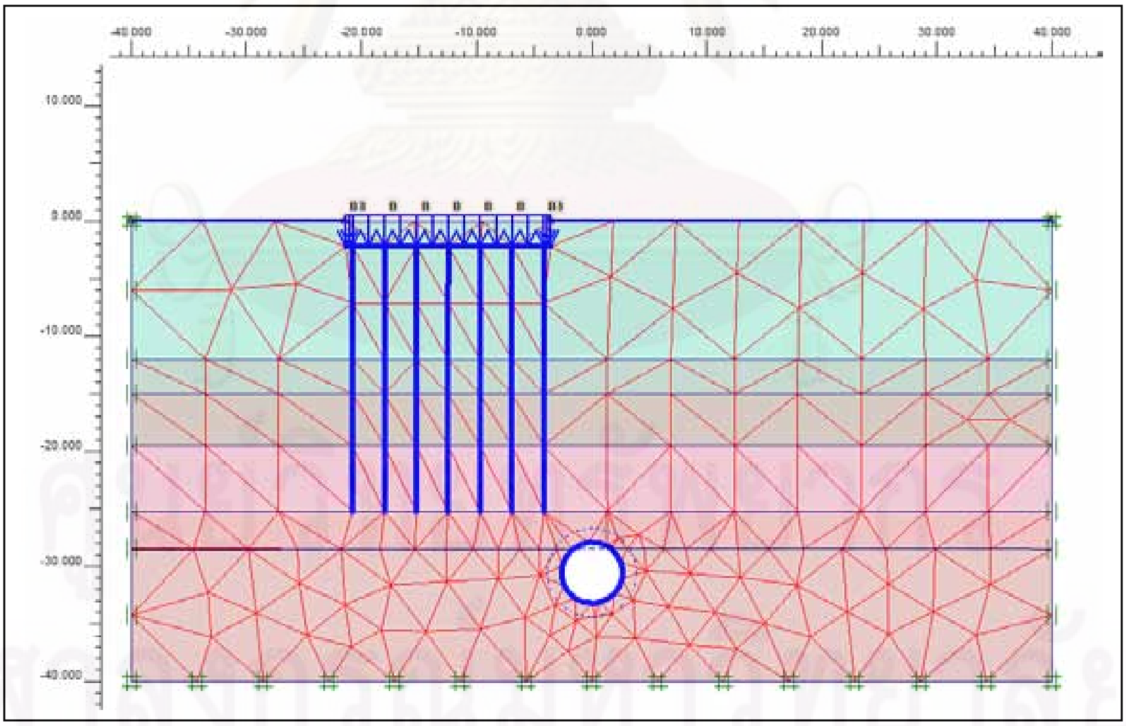


Figure 4.12b FEM at the expressway (from left: pile group no. Z1-7)

Assuming that ground loss is about 1% and the surcharge load of 10 kN/sq.m. is applied on the foundation; the maximum vertical movement at the pile tip is varied, as shown in Table 4.3. Based on the numerical analysis, the authority has approved the construction process however; the construction shall be carried out under close operation control. Trigger values for monitoring section is proposed as shown in Table 4.4.

Table 4.3 The vertical movement of pile foundation, obtained from the analysis (Teparaksa, 2005b)

Pile Row no.	Foundation		
	X	Y	Z
1	7	10	1
2	9	8	2
3	-	7	2
4	-	5	3
5	-	-	5
6	-	-	7
7	-	-	9

Table 4.4 Trigger value of monitoring section (Teparaksa, 2005b)

Monitoring Point	**Trigger Level**		
	Alert	**Alarm**	**Action**
Ground surface settlement	7 mm	8 mm	9 mm
Structural settlement of pile cap	7 mm	8 mm	10 mm
Extensometer at 19m	5 mm	6 mm	7 mm

4.3.2 MRT Intersection

Numerical model of MRT section has been simplified and analysed, as shown in Figure 4.13. According to the numerical analysis, vertical movements of the MRT and the station have been summarized and shown in Table 4.5. The total displacement of the MRT tunnel is about 5mm whereas the total displacement of the station wall is about 1mm. Both values are less than MRTA criteria and therefore, the MRTA has allowed the TBM to pass underneath the MRT tunnel with close control of the construction process. The results of the FEM analysis are then used for the setting of the trigger level for instrumentation, as presented in Table 4.6. The predicted result will also be used to provide operations guidelines for tunnelling under the MRT tunnel.

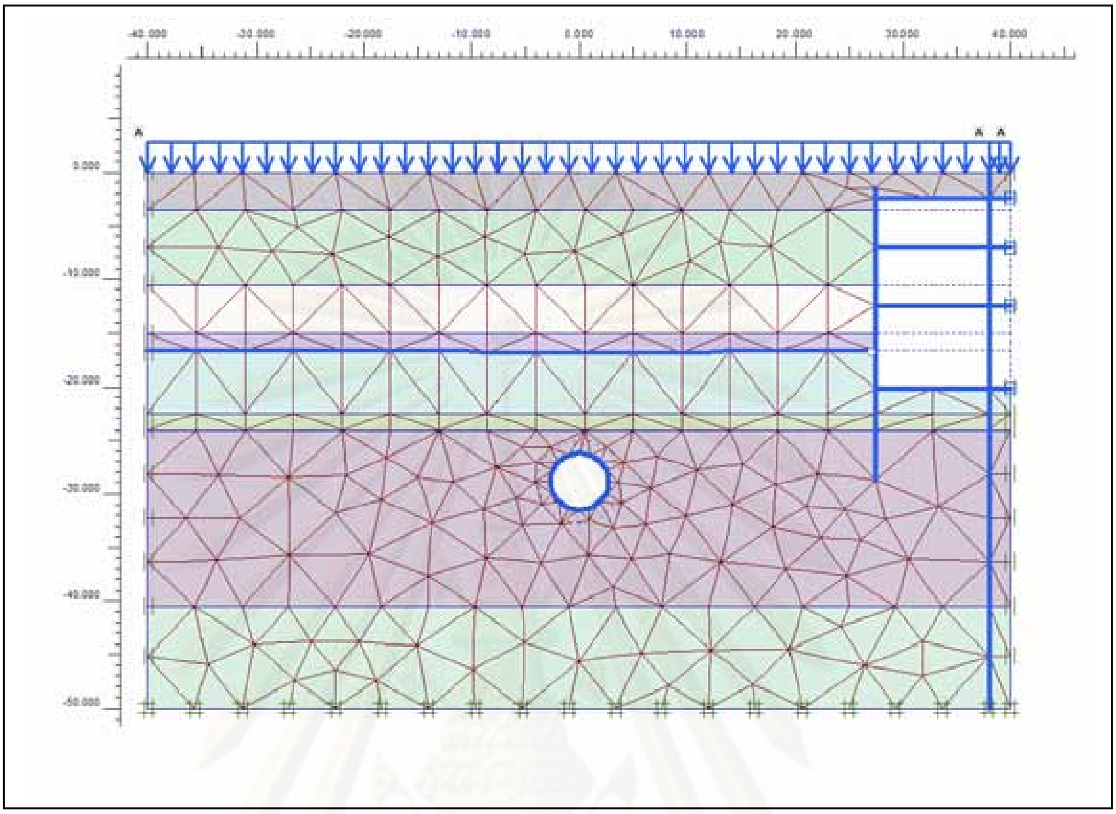


Figure 4.13 Simplified numerical model of test section 5 (MRT intersection)

Table 4.5 Displacement prediction by numerical analysis.

Effects from the Construction of BMA Flood Diversion Tunnel		Additional Pressure (kPa)		Differential Movement		Total Movement (mm)	
		FEM	Allowable	FEM	Allowable	FEM	Allowable
Surcharge	MRT Tunnels	21.3	25.0	-	-	-	-
	Station Roof	4.2	20.0	-	-	-	-
	Station Wall	24.1	25.0	-	-	-	-
Movement	MRT Tracks	-	-	1:8,600	1:2,000	-	-
	MRT Tunnels	-	-	-	-	5.01	6.00
	Station Wall	-	-	-	-	1.12	6.00

Table 4.6 Levels of response used during the construction at MRT tunnel intersection.

Monitoring	Station	Trigger Level (mm)		
		Alert	Alarm	Action
Ground Surface Settlement	1+613.611	3.68	4.20	4.73
	2+019.401	3.68	4.20	4.73
Structural Settlement	2+112.043	3.51	4.01	4.51
Vertical Displacement at -16m	All	4.46	5.10	5.73

4.4 TBM Operation Criteria

Appropriate TBM driving parameters are important, in order to limit ground movement around the MRT tunnel. Avoiding personal experience which may vary from person to person, suitable ranges of the operation parameters are specified. According to Teparaksa et al., (2004), only significant parameters, such as trust force, soil excavation ratio and grouting technique are emphasized. Based on tunnelling parameters from the validated sections, e.g. Test Section 1 and 2, an average range of parameter values within ±60m from both test sections is recommended. Table 4.7 presents the average range of TBM parameters from each section and a suggested range used for construction under the MRT area.

Table 4.7 Recommended ranges of TBM parameters for construction under MRTA area.

Operational Parameter	Test Section 1	Test Section 2	MRTA Tunnel
Earth Pressure (Bar)	2.2-2.5	2.4-2.8	2.0-2.5
Average Jack Speed (mm/min)	11-13	12.5-13.2	12-13
Grout Pressure (kPa)	300-500	400-700	400-700
Grout Ratio (x times)	1.5-2.0	1.1-1.5	>1.1

4.5 Field Monitoring at Expressway Section

4.5.1 Vertical Movement

Monitoring of surface settlement reveals that the ground has settled when the TBM is actually 10m away from the monitoring point, Figure 4.14. Magnitude of settlement is increasing until the face of TBM is about 60m away. The maximum settlement is about 5mm. The series of extensometer has detected ground movement; even through the face of TBM was about 40m away, also shown in Figure 4.15. The ground has lifted up while the TBM was about 20m away. Once, the face of TBM is about 8m, the extensometer above the TBM has moved downward, while the instrument below the TBM has moved upward. Maximum settlement near the pile tip is about 6mm, which is less than the prediction. The maximum vertical movement is about 7mm near the crown of the tunnel.

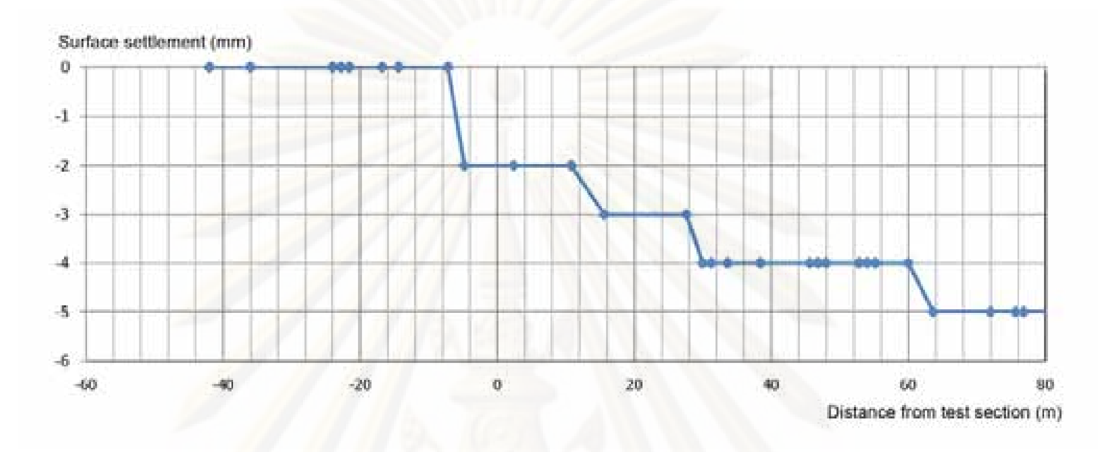


Figure 4.14 Ground surface settlement along tunnel route at test section 3

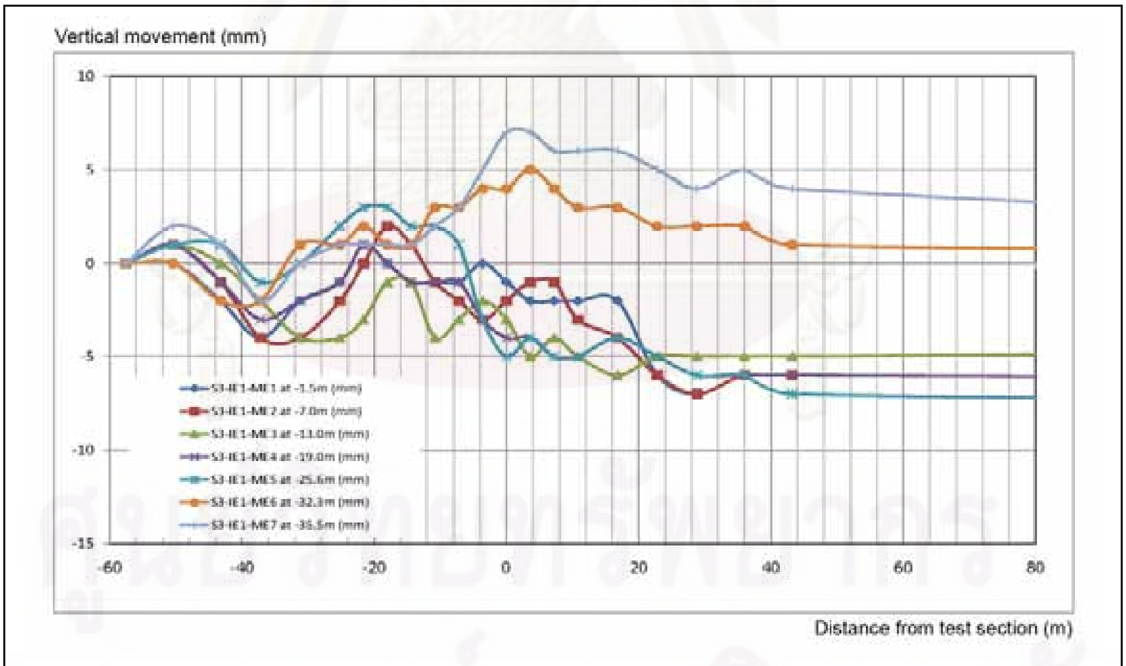


Figure 4.15 Vertical ground displacement at test section 3

4.5.2 Horizontal Movement

The result of inclinometer shows that the ground has started to move inward to TBM face when the machine is about 4m in front of the test section. When the TBM has approached the test section, the instrument shows dramatically horizontal movement of the ground, up to 12mm, as shown in Figure 4.16. Grouting does not show any significant effect on horizontal ground movement.

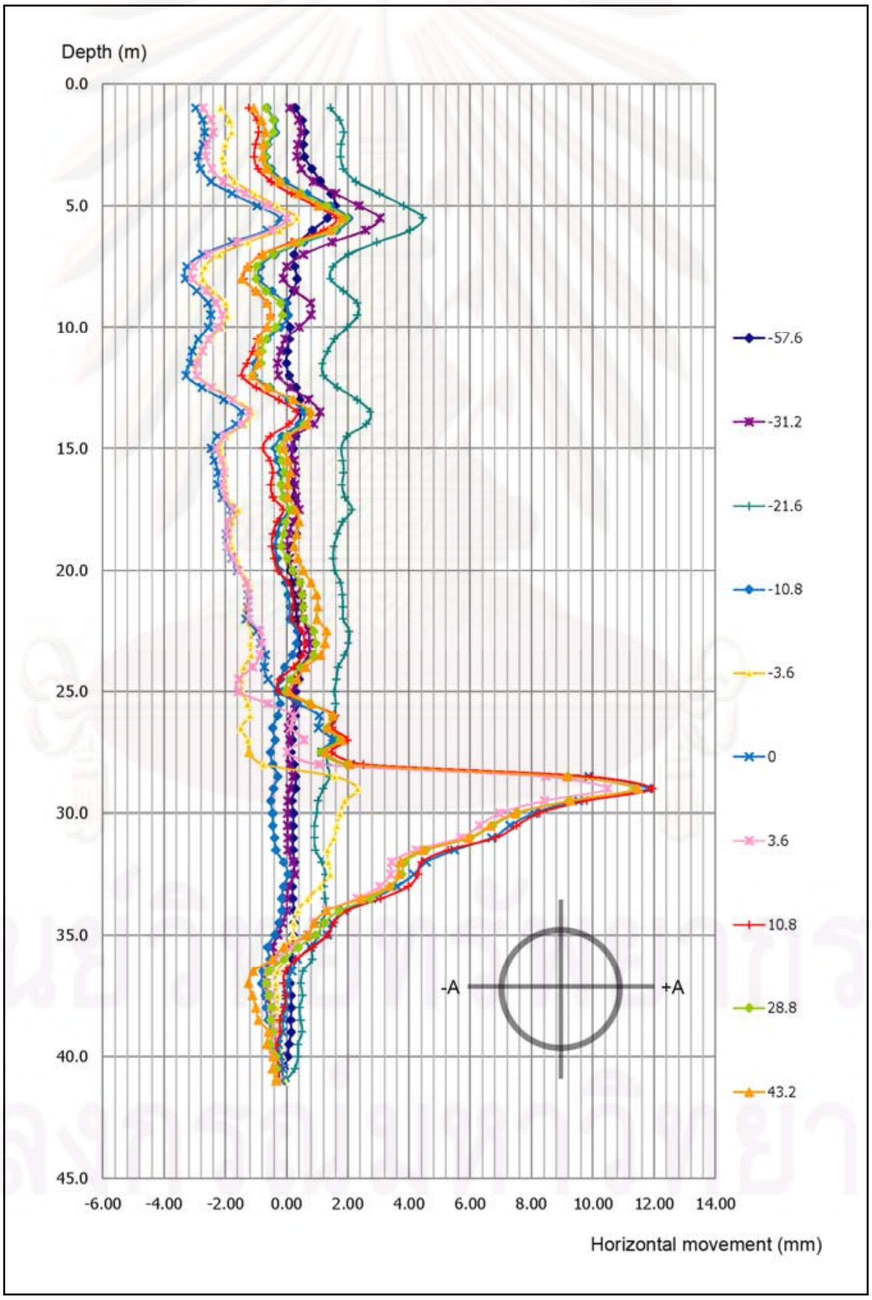


Figure 4.16 Horizontal ground displacement at test section 3

4.5.3 Porewater Pressure

According to Figure 4.17, fluctuation of porewater pressure has been detected when the face of TBM is about 30m away. The pressure (PZ1) is dramatically increases when the machine has reached the test section. Increasing of porewater pressure tends to follow the variation of average earth pressure however; once the TBM has passed the pile foundation, the driving parameter is not significant. The grout ratio then have important role on the porewater pressure. The porewater pressure at PZ2 which has been installed behind the pile foundation provides much lower value than PZ1. Porewater pressure becomes stable when the TBM is about 60m behind the test section. Both maximum surface settlement and structure settlement are about 5mm, less than the prediction. All tiltmeters had not shown any significance movement.

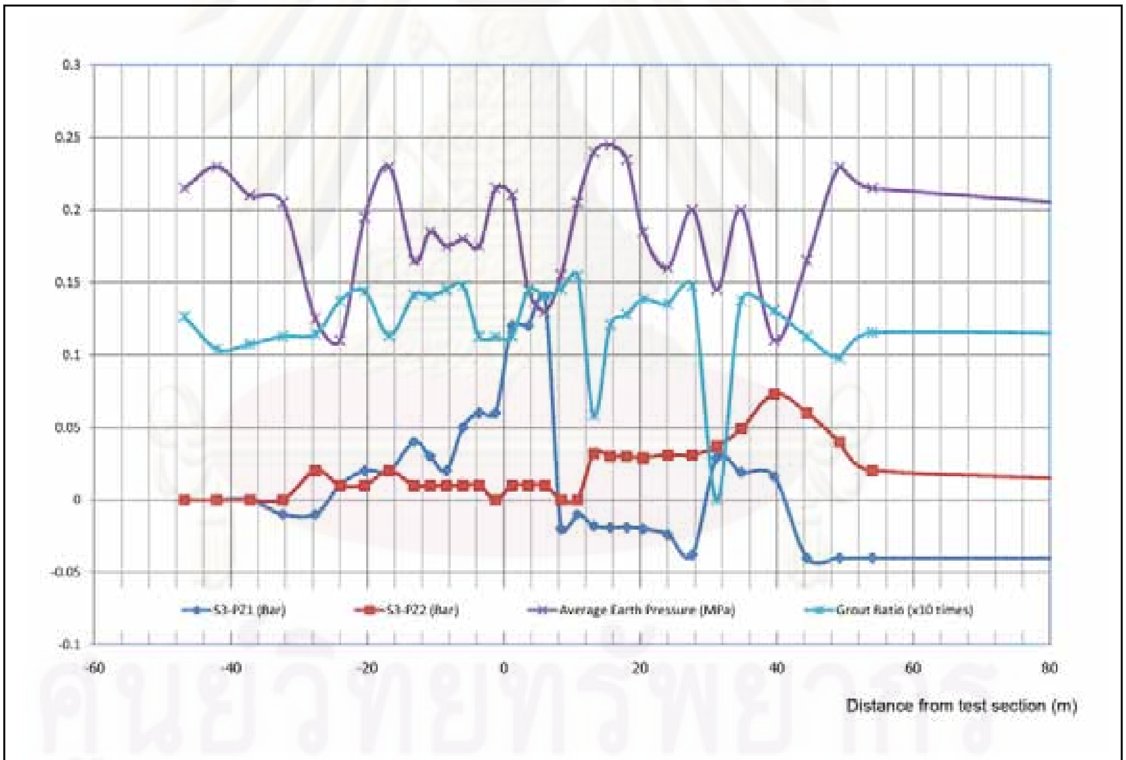


Figure 4.17 Changing of porewater pressure at test section 3

4.6 Field Monitoring at MRT Section

Before approaching the MRT tunnel, the fifth test section was set up to reconfirm suggested TBM parameters and also to ensure that all equipment is in good functional condition. Field measurement at Test Section 5 is discussed below.

4.6.1 Vertical Movement

Figure 4.18 shows the relationship between the TBM parameters and surface settlement along the tunnelling section. Ground settlement occurs slightly at 2 times of the tunnel depth or about 60m in front of the test section. The movement increases constantly after the TBM has passed the test section. The magnitude of the settlement decreases again at about 24m from the test section, mainly due to rapid increase in the thrust force.

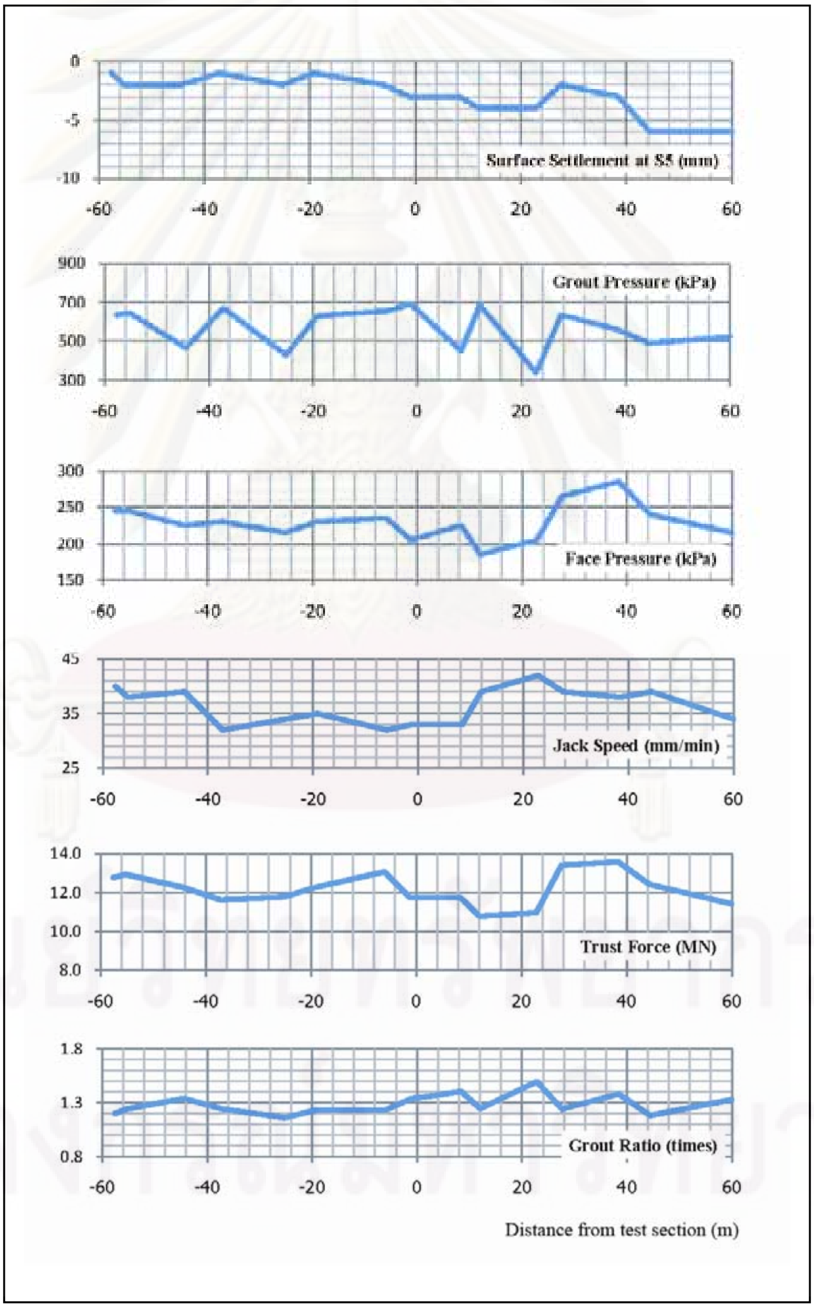


Figure 4.18 Ground settlement and TBM parameters at test section 5

Figure 4.19 presents the results of deep settlement and the TBM parameters. The anchor, 0.35m above the tunnel, moves as the machine approaches. The instrument then moves dramatically after the machine has reached the test section. The deep settlement point, installed at 5.5m above spring-line level, also shows similar characteristics.

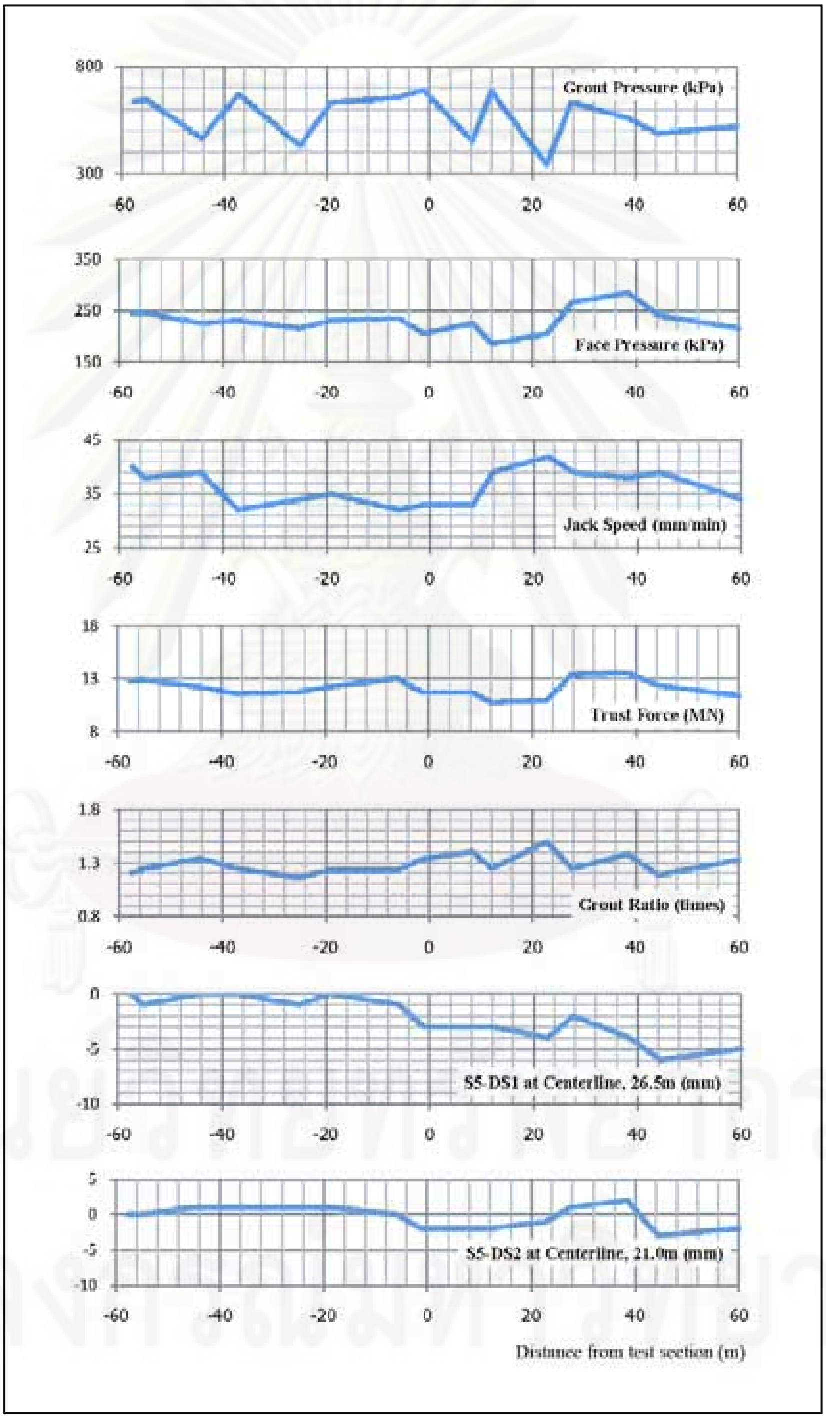
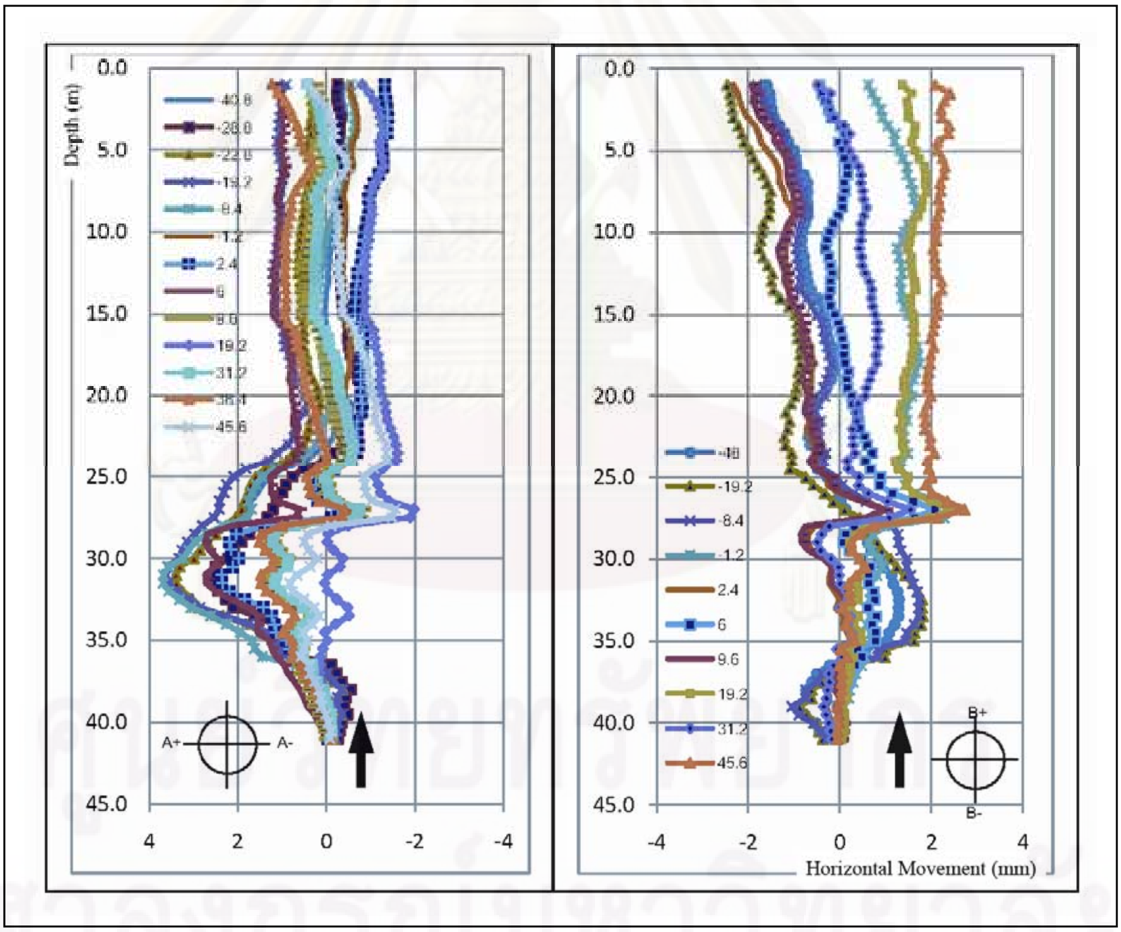


Figure 4.19 Result of deep settlement and TBM parameters at test section 5

4.6.2 Horizontal Movement

Figures 4.20a and 4.20b present horizontal displacement on perpendicular and parallel axes at 5.6m from the tunnel axis, respectively. As mentioned previously, the inclinometer shows that on a perpendicular plane, soil moves to the tunnel axis when the cutter disc is about 6m away (about 1 time of TBM diameter). The maximum outward displacement occurs around the tunnel shape when the TBM cutter is about 19.2m away, i.e. 16 boring cycles. Soil particles around the TBM face have been affected by the cutter disc and grouting process. Figure 4.20b presents that soil has also shifted outward before the TBM passes and then shifted inward to the TBM position again when the cutter disc has passed. Most displacements occur on the side wall and above the spring-line level.



Figures 4.20a and 4.20b Horizontal displacement of test section 5 at 5.6m from tunnel axis.

4.6.3 Pore-water Pressure

Figure 4.21 presents the pattern of pore-water pressure during tunnel construction at Test Section Three. As shown in Figure 4.21, the piezometer on the side wall indicates that pore-water pressure substantially increases, 15m or 3 times of the TBM diameter, prior to the machine's arrival. Unfortunately, the closer piezometer was damaged and hence, read-out data could not be obtained. As stated previously, the positive pore-water pressure then dissipates directly after the face of the TBM has passed the instrument. The grouting process also causes additional pore-water pressure around the tunnel. After the arrival of the TBM, any tunnelling activities beyond 30m from the piezometer do not affect dissipation of porewater pressure.

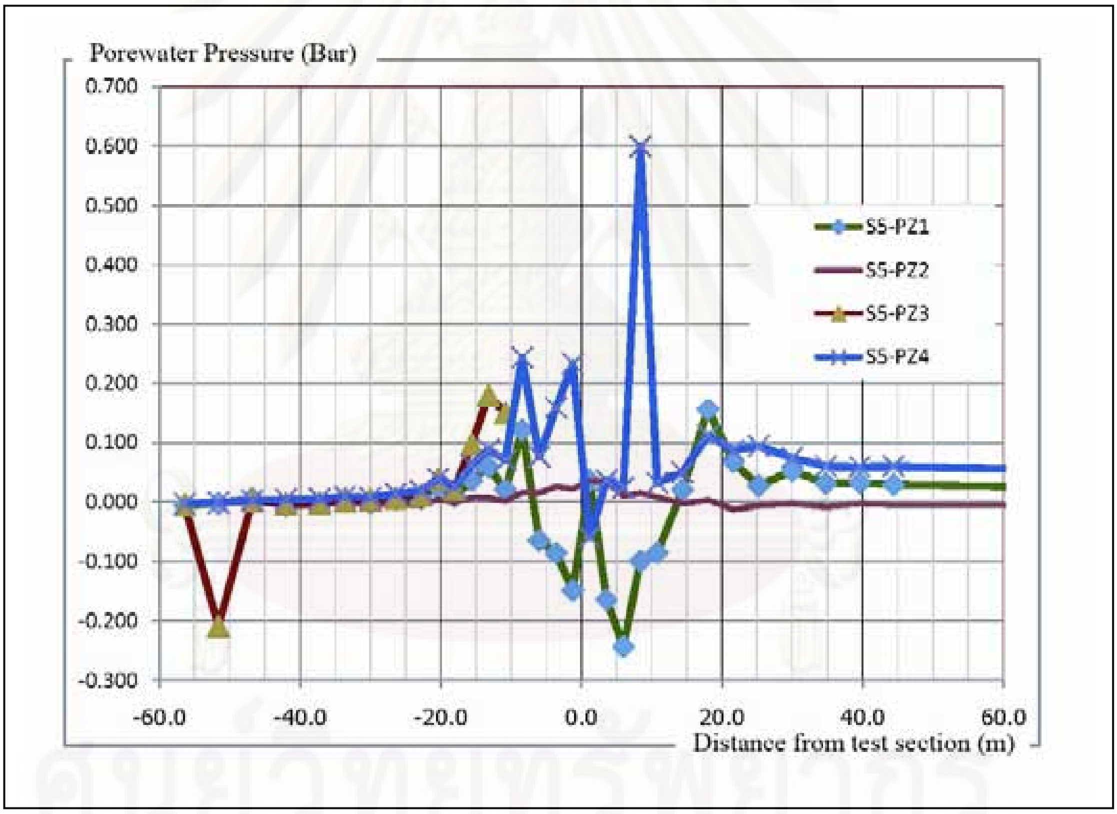


Figure 4.21 Changing of pore-water pressure during tunnel construction at test section 5

Figure 4.21 indicates that positive pore-water pressure above the tunnel decreases substantially when the TBM arrives. Moreover, the instrument which is installed at 1 time of TBM diameter above the crown, also shows that the construction process has hardly affected pore-water pressure in the soil layer. Most of the excess pore-water pressure is fully dissipated within a short period of less than two months.

4.6.4 MRT Movement

The surveyor monitored the deformation of the MRT tunnel for 7 consecutive days before and after the arrival of the TBM. However, the monitoring process is only allowed between 2 am and 5 am. The vertical movement along the longitudinal section is shown in Figure 4.22.

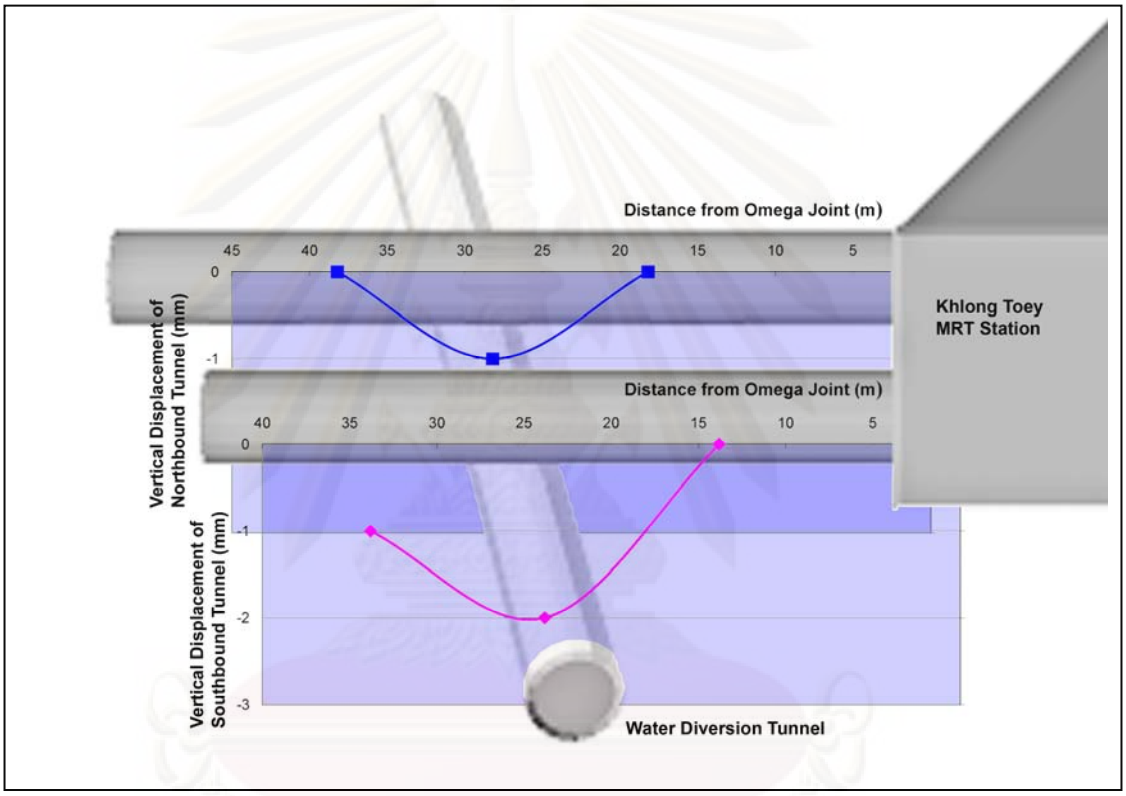


Figure 4.22 Vertical displacement of MRT tunnel on longitudinal section

According to Fig. 4.22, the Southbound tunnel seems to move more than the Northbound tunnel. The maximum settlement of the Southbound tunnel at S2 is only 2mm, while movement at S3 is about 1mm. The vertical displacement at S1 is zero which may be affected by the stiffness of the tunnel and the station. The maximum settlement of Northbound tunnel at N2 is only 1mm, while at other locations it is zero. Based on the direction of the TBM driving, the Southbound tunnel seems to be more affected by construction than the Northbound tunnel. Figure 4.23a and 4.23b present the relative deformation of the MRT tunnel at the intersection point (S2 and N2), during the construction of the water diversion tunnel.

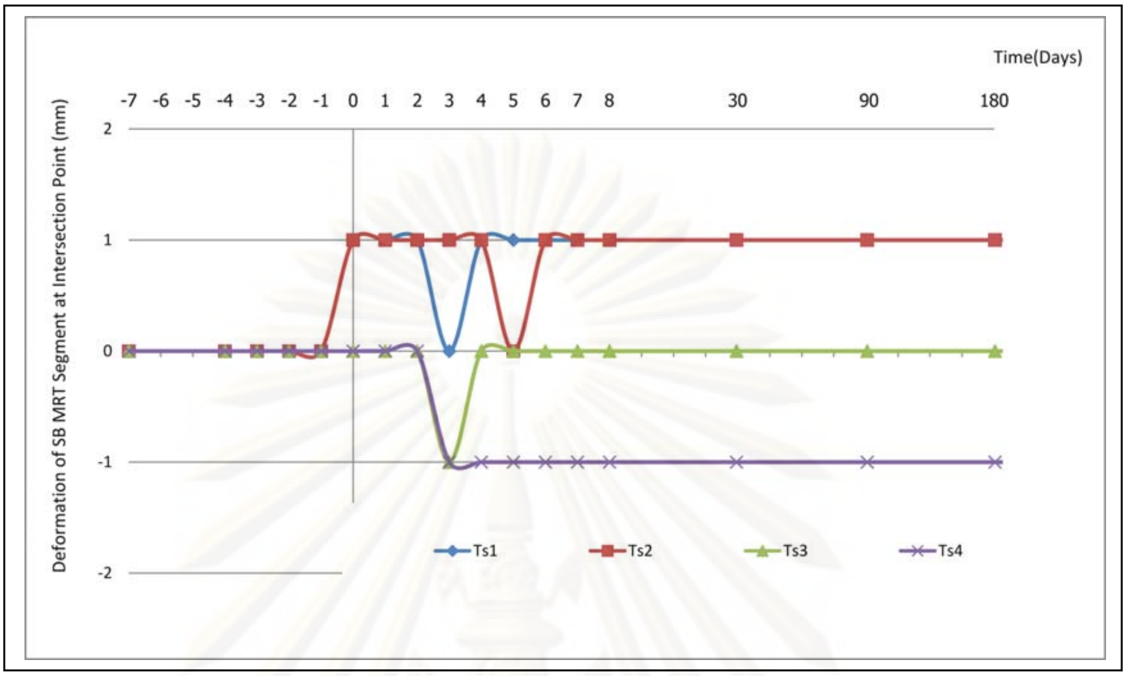


Figure 4.23a Relative deformation of Southbound MRT tunnel at intersection point (S2)

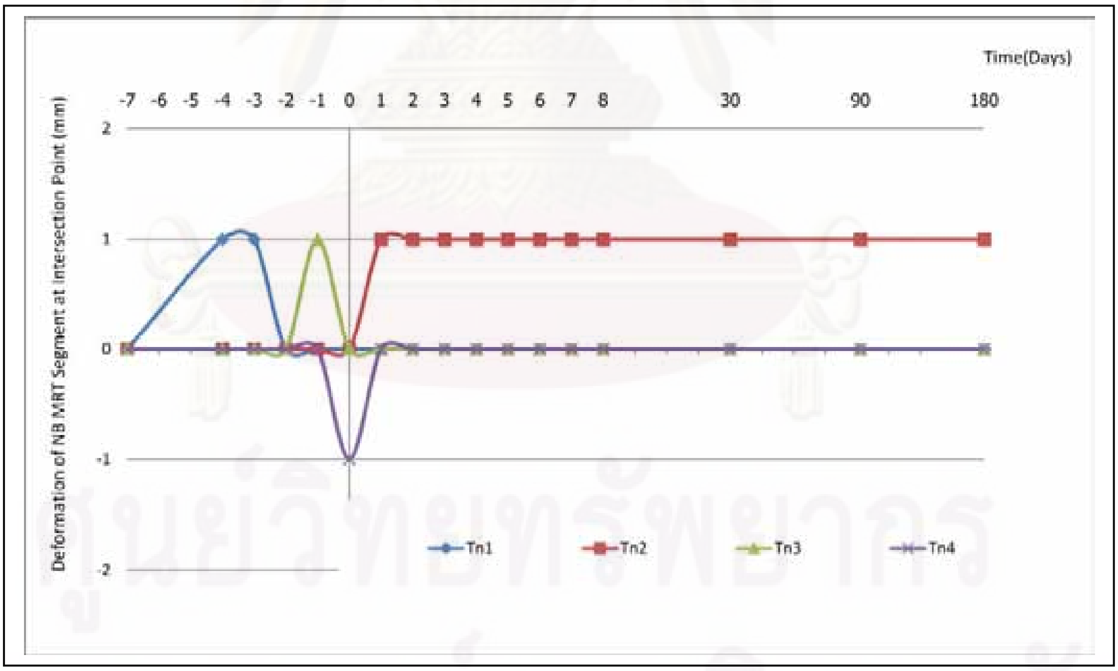


Figure 4.23b Relative deformation of Northbound MRT tunnel at intersection point (N2)

According to Figure 4.23a, the diameter of the tunnel in a vertical direction (TS3) is still the same while lengths on TS1 and TS2 axes are wider by 1mm. Field measurement also indicates that the length on TS4 axis is shorter by 1mm. In the Northbound tunnel, the deformation is different. In fact, the whole tunnel has barely moved. The length on TN2 axis has enlarged by only 1mm, Figure 4.23b. The measurement was carried out for a further 180 days. The result shows that the deformation of the tunnel is stable at about 6 days after the TBM passing. The MRTA will perform a scheduled check after this period. The relative deformation of the Southbound and Northbound tunnels can be simulated by numerical analysis, as shown in Figure 4.24.

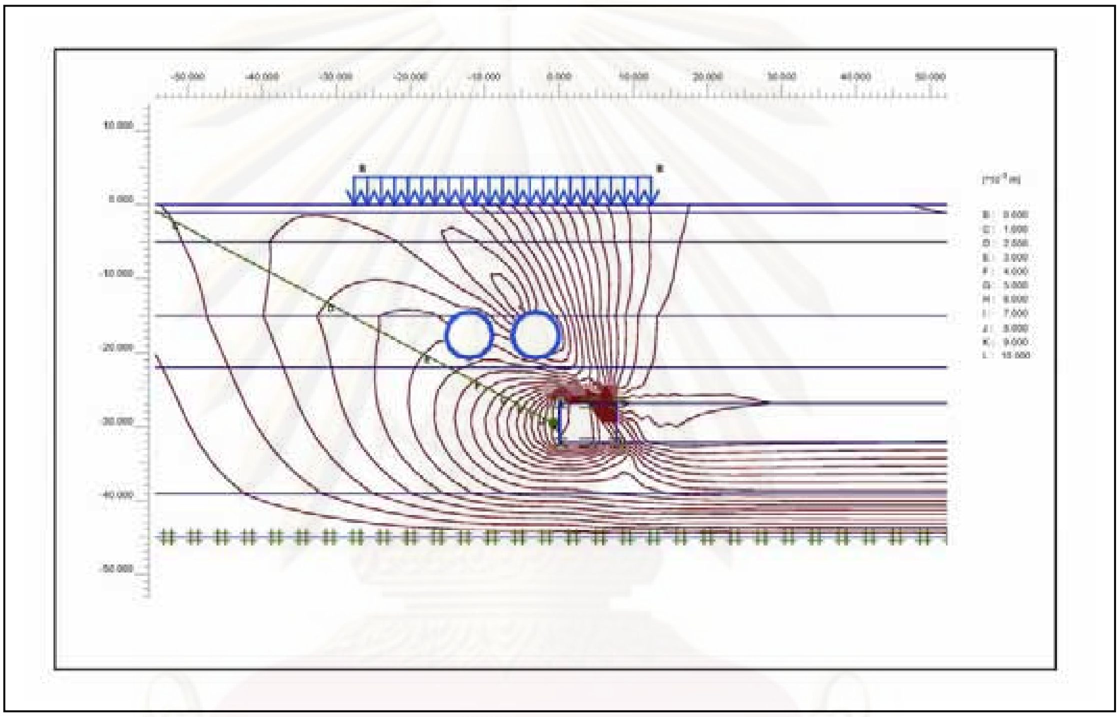


Figure 4.24 Numerical analysis of ground movement around MRT tunnel

According to the information on the ground movement around the tunnel, field measurement is input into the model. The result shows that the deformations of both tunnels are similar to deformations in actual conditions. Tunnel deformation actually moves in the same pattern in the surrounding ground, as found in Figure 4.23a and 4.23b.

CHAPTER V
CONCLUDING REMARKS

Prior to the actual tunnel construction, setting up the test sections can provide validation of numerical analysis and prevent possible risk during construction. Test sections also encourage all parties to keep alert, before approaching the actual intersection. Moreover, the TBM operator could also use appropriate suggested TBM parameters from test sections, as a guideline for ease of operation, while maintaining settlement in the construction criteria.

Assessment of construction performance by using two-dimensional finite elements method shall be carried out. Based on similar research or construction performance in the past, this will compensate for tunnel parameters and workmanship. Two dimensional numerical modelling tends to provide a conservative solution. However, by performing the trial process, the result is still acceptable to represent the complex three-dimensional problems.

Most of the tunnel deformation and ground settlement, about 90%, has occurred over a short term period and is likely to achieve full displacement within two months or about 60m after tunnelling has passed. Long term movement rarely occurs, as the construction was carried out in a stiff clay layer. According to field monitoring of TBM operation in Bangkok green field area, boundary of initial surface settlement could be defined as two times of tunnel depth in front of TBM face, while the boundary of shield passing settlement at the surface could be described as one time of tunnel depth behind the TBM face. The boundary of tail settlement at the surface could be located at about two times of tunnel depth behind the TBM face. The boundary of horizontal ground movement, at spring line level, is about one time of TBM diameter in the front of TBM face, while the boundary behind the TBM is about four times of TBM diameter. The influence zone of horizontal ground movement is limited to about 0.5 time of TBM diameter around the TBM perimeter.

Fluctuation of excess pore-water pressure surprisingly changes only in the horizontal plane. Changing pore-water pressure in a vertical direction is limited, probably by the characteristics of clay particles. The influence zone of excess pore-water pressure is limited to about one time of TBM diameter in vertical axis and limited to about two times of TBM diameter in horizontal axis. Moreover, similar to surface ground movement, most excess pressure is also fully dissipated over a short term period. The influence zone of excess pore water pressure extends to about three times of TBM diameter, in front of TBM face, while the excess pressure could be fully dissipated within about six times of TBM diameter.

Finally, suggested tunnelling parameters have proven to be useful as reference information for both numerical analysis and operation guidelines for future projects. Tunnel records show that additional face pressure and grout filling ratios could minimize ground surface movement, and also decrease the horizontal displacement around the TBM. Tunnelling, with ground loss of 0.6% at about one time of TBM diameter, underneath the MRT tunnel in the stiff Bangkok clay is applicable and causes only 1 to 2 mm settlement in the MRT tunnel. The stability of pile foundation of the expressway where tunnelling was bored, with clearance of 2m or 0.4 times of TBM diameter, can be controlled by measuring the inclinometer installed in front of the pile foundation.

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BIOGRAPHY

Kullapat Phisitkul was born on August 18, 1979 in Chiangmai, Thailand. The author was graduate in Bachelor degree of Environmental Engineering from Chiangmai University in 1999. The author then continued his Master Degree of Geotechnical Engineering at University of New South Wales (UNSW) in Sydney, Australia and graduated in 2001. After graduation, he had worked with Snowy Mountain Engineering Corporation (SMEC) in Sydney for three years as a geotechnical engineer. Then, he had returned to Thailand and worked with Kenber Geotechnic (Thailand) Co., Ltd. since 2003. In 2005, he has graduated in Master of Public and Private Management (MPPM) at National Institute of Development Administration (NIDA). He then continued his doctoral degree at Chulalongkorn University in 2005. In 2009, he has also received Certificate of Business Advisor from Kasetsart University.

The author is experienced engineer who is specialist in geotechnical, environmental engineering programmes, such as: Plaxis, Plaxis 3D Tunnel, Autoslope, Slope W, Wallap, Help, as well as setting of geotechnical instrument layout. The author also has experience in rotary, percussion and Down The Hole (DTH) drilling, conventional/slush/chemical cement/resin grouting, soil nailing, rock bolt, shotcrete and gabion. Moreover, the author also has knowledge in application of foam concrete and Expand Poly-Styrene in geotechnical remedial work. Furthermore, the author is also an experienced engineer in environmental laboratory analysis and testing, water and wastewater plant design, air pollution and solid waste management. International work experience of the author includes Thailand, Laos, and Australia. The author is currently works as deputy managing director of Kenber Geotechnic (Thailand) Co., Ltd.