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Numerical Analysis of the Response of Existing Tunnel Structures to Crossing Tunnel Construction in Soft Soil

Muhammad Amhar Wajdi Yanreginata, Widjojo Adi Prakoso

Department of Civil and Environmental Engineering, University of Indonesia, INDONESIA

E-mail: m.amhar@ui.ac.id, Wprakoso@eng.ui.ac.id

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ABSTRACT

Massive urban planning forces the utilization of underground space, not least in the development of transportation infrastructure. MRT Jakarta has successfully made its line in phase 1 Lebak Bulus - Bundaran HI and now is developing the line which is packaged in phase 2 to the direction of Ancol, this route reaches the south-north part of Jakarta. In recent years, there has also been a plan to add an East-West line which will be packed in phase 3, this does not rule out the possibility of a meeting between the existing tunnel and the undercrossing tunnel. This study discusses the influence of undercrossing on the existing tunnel structures, especially under Jakarta Clay, which is classified as having a thick, soft clay stratum. This research was divided into 2 steps using finite element analysis with Hardening Soil model. The first step is to conduct a back analysis to obtain the actual soil parameter value then proceed with step 2, which is to conduct an undercrossing twin tunnel analysis. Based on the results of the study showed the fittest soil parameters using the correlation E50 = 3500N by the effective soil strength parameters and the undercrossing excavation caused four stages of vertical displacement of the existing tunnel.

Keywords: tunneling; back-analysis; FEA; soft clay; hardening soil.

INTRODUCTION

This research is based on the necessity in supporting infrastructure development, especially in the city of Jakarta. According to Peng et al (2021), the concept of utilizing underground infrastructure is included in 11 of the 17 points of Sustainable Development Goals (SDGs). After successfully constructing the MRT Phase 1 line, MRT Jakarta is now constructing the Phase 2 South-North line stretching from Bundaran HI – Ancol which is the development line from phase 1. Development plans have also been echoed for MRT Phase 3 which runs from East to West of Jakarta city. The development of the East-West Phase requires an undercrossing meeting at a certain point planned in the Thamrin reference area.

Chen et al (2018) stated that undercrossing structures have a significant amount of vertical displacement induced in the EPBS tunnel undercrossing on a sand stratum. While in the other hand Liu et al (2022) also said the same as the uplift vertical deformation of a twin tunnel undercrossing has a significant impact to the Double-O Tunnel excavated using TBM in soft clay soil. This research is conducted on the effect of the undercrossing twin tunnel on the MRT Jakarta existing tunnel. The focus that will be discussed is the vertical deformation including stages that occur during the excavation.

Based on the soil data obtained, the location of Thamrin reference area has a thick very soft clay soil stratum. The construction of undercrossing tunnels in the area must get extra monitoring. The approach conducted in this study is divided into 2 steps where the first steps is the back analysis stage of soil parameters using hardening soil model compared to the results of inclinometer monitoring during the Thamrin reference area D-Wall work. Furthermore, the second step is an undercrossing twin tunnel analysis with 0.5D distance analyzed using the soil parameters from step 1.

RESEARCH METHODS

The research flow chart is shown in the figure below.

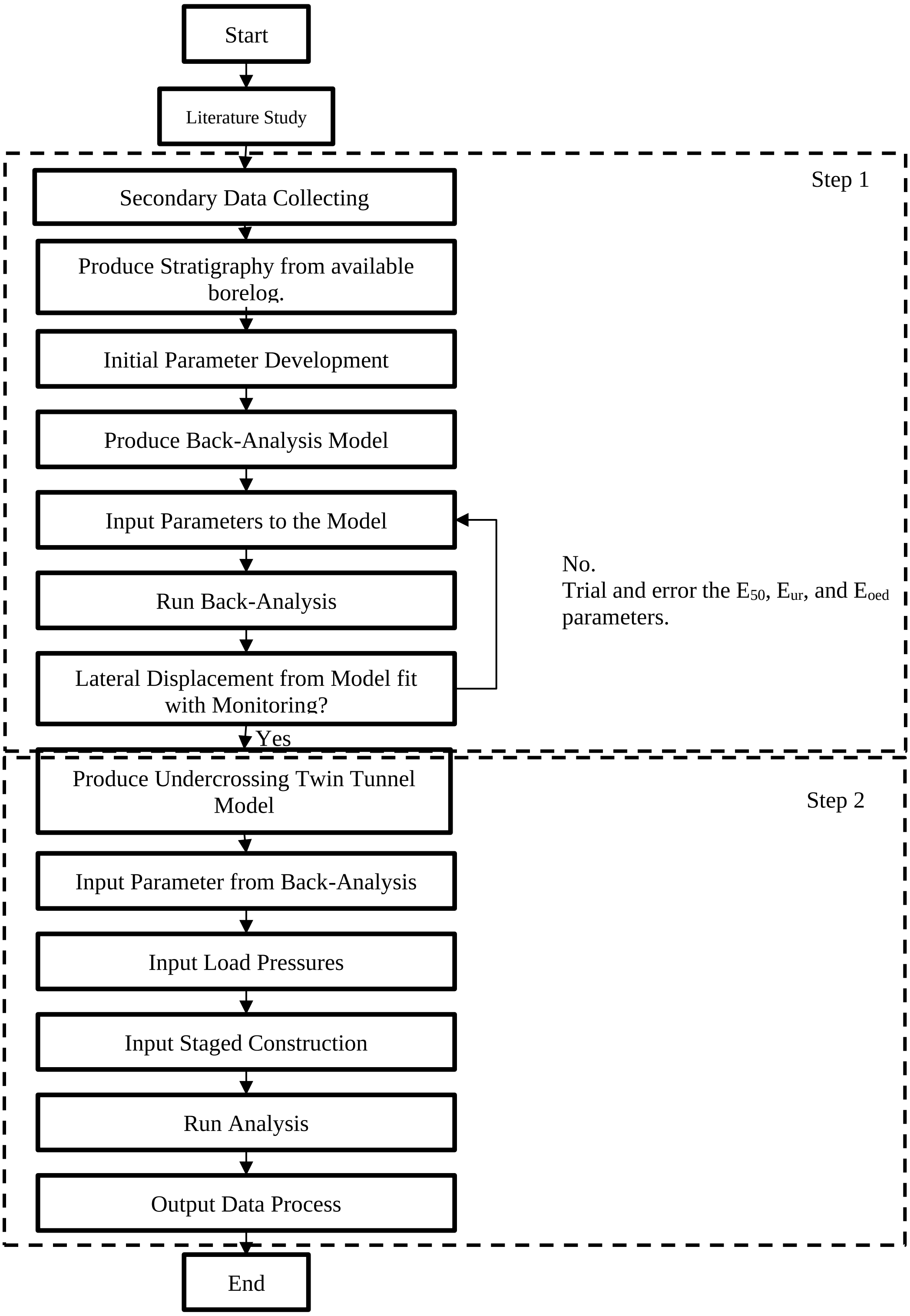

Figure 1. Flow chart

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Step 1

Soil profile obtained from three boreholes in Thamrin reference area. The inclinometer is located near one of the boreholes so the model for the back analysis is conducting the soil layer from this borehole. There is a layer of very soft clay that dominates to a depth of 20 meters followed by lenses of silty sand and furthermore continued by a very stiff layer under 30 meters.

Table 1. Soil profile on each borehole

Soil Profile	BH-03		BH-04		BH-05		BH-06	
	Top (m)	Bot (m)	Top (m)	Bot (m)	Top (m)	Bot (m)	Top (m)	Bot (m)
V. Soft Silty Clay	+3.1	-8.4	+3.2	-5.3	+2.8	-2.7	+4.5	-1.5
V. Loose Silty Sand	-8.4	-9.4	-5.3	-6.5	-2.7	-5.2	-1.5	-6.0
V. Soft Silty Clay	-9.4	-21.4	-6.5	-22.3	-5.2	-24.7	-6.0	-27.0
M. Dense Silty Sand	n.a	n.a	-22.3	-27.8	-24.7	-28.7	-27.0	-30.0
Dense Sand	n.a	n.a	-27.8	-31.8	-28.7	-31.7	-30.0	-32.6
V. Stiff Silty Clay	-21.4	-36.9	-31.8	-46.8	-31.7	-37.2	-32.6	-45.5

The inclinometer data was read during a 2.5-meters deep excavation at an MRT Station just before jet grouting work was conducted on the D-Wall structure. The D-Wall structure itself using the 32 MPa strength pre-cast concrete. The inclinometer shows the lateral deformation of D-Wall. This data will be the benchmark when doing the back analysis.

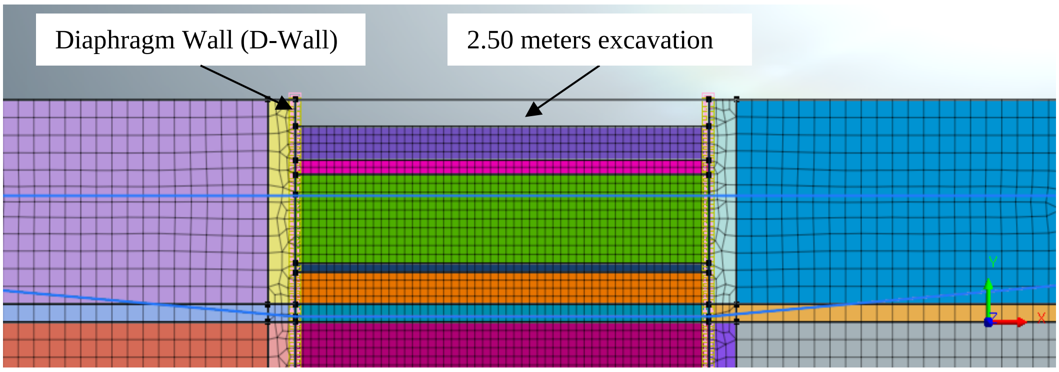

Figure 2. Excavation of the Thamrin reference area before jet grouting works.

The Hardening Soil Model was applied to model soil behavior in this study. Previously, research conducted by Hsiung (2018) used the upper and lower bounds of the E50 value which was correlated with the NSPT value. The upper bound for clay soil is E50 = 4000N and lower bound E50 = 2800N. While sand has an upper bound of E50 = 2800N and a lower bound of E50 = 2000N. Tazakka and Tirta (2023) have also conducted uniform research in the northern area of Jakarta by applying the same method, where the value of the back fitting parameter is produced adjacent to the upper bound value and uses effective soil strength.

Table 2. Correlation for elastic modulus on hardening soil model

Parameters	Correlation	Remarks
E_{50}^{ref} Upper Bound	4000 NSPT (for Clay); 2800 NSPT (for Sand)	Hsiung et al., 2018 and 2009
E_{50}^{ref} Lower Bound	2800 NSPT (for Clay); 2000 NSPT (for Sand)	Hsiung et al., 2018 and 2009
E_{oed}^{ref}	$0.7E_{50}^{ref}$	Huynh, 2022
E_{ur}^{ref}	$3E_{50}^{ref}$	Huynh, 2022

The exponential power (m) is variance regarding the soil types. Correlation from CUR (2003) used for this research.

Table 3. Correlation for exponential power (m) on hardening soil model

Soil Type	m Value
Clay – Silt (NC)	0.8 – 1.0
Clay (OCR>1)	0.5
Peat	1.0
Sand	0.55
Gravel	0.4 – 0.9

Soil strength parameters are applied using the model with the input of effective soil parameters, where the modeling using the effective soil parameters represents better result of the deformation that occurs in the soil compared to others. Correlation is made to the value of clay effective soil parameters based on the value from Burt Look (2014) and for the sand effective friction angle from Idriss and Boulanger (2008).

Table 4. Correlation for effective strength on cohesive soil

Material	Description	c' (kPa)	φ'
Clay	Soft – Organic	5 – 10	10 – 20
	Soft – non-organic	10 – 20	15 – 25
	Stiff	20 – 50	20 – 30
	Hard	50 – 100	25 – 30

Table 5. Correlation for effective strength on granular soil

Material	Relative Density	NSPT	φ'
Sand	Very Loose	< 4	<32
	Loose	4 – 10	32 – 35
	Medium Dense	10 – 30	35 – 38
	Dense	30 – 50	38 – 41
	Very Dense	>50	41 – 45

The initial parameters based on the borehole respecting the correlation summarized in table below.

Table 6. Initial soil parameter

Geotechnical Parameter	Very Soft Silty Clay	Very Loose Silty Sand	Very Stiff Silty Clay	Medium Dense Silty Sand	Dense Sand
Average N-SPT	1.4	3.0	20.1	11.1	33.25
Saturated Weight	14.2	14.5	19.6	17.1	19.5
Effective Cohesion	6.5	n.a	30.8	n.a	n.a
Effective Friction Angle	11.5	20.7	25.1	26.3	34.9
E_{50}^{ref} Upper Bound	5641	8400	80593	31000	93100
E_{50}^{ref} Lower Bound	3948	6000	56415	22143	66500
E_{oed}^{ref} Upper Bound	3948	5880	56415	21700	65170
E_{oed}^{ref} Lower Bound	2764	4200	39490	15500	46550
E_{ur}^{ref} Upper Bound	16923	25200	241778	93000	279300
E_{ur}^{ref} Lower Bound	11846	18000	169244	66429	199500
m	0.9	0.55	0.9	0.55	0.55
νur	0.2	0.2	0.2	0.2	0.2

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The use of FEA-based Midas GTS NX software in this analysis makes it possible to see the behavior of soil and structure. Geometry models are made with two-dimensional geometric width 85 meters to avoid boundary effects on output results. The width of excavation is 26.5 meters with 30 meters deep D-Wal. A surcharge load of 10 kN/m^2 applied to represent the construction load. The groundwater occurs at 3 meters below surface.

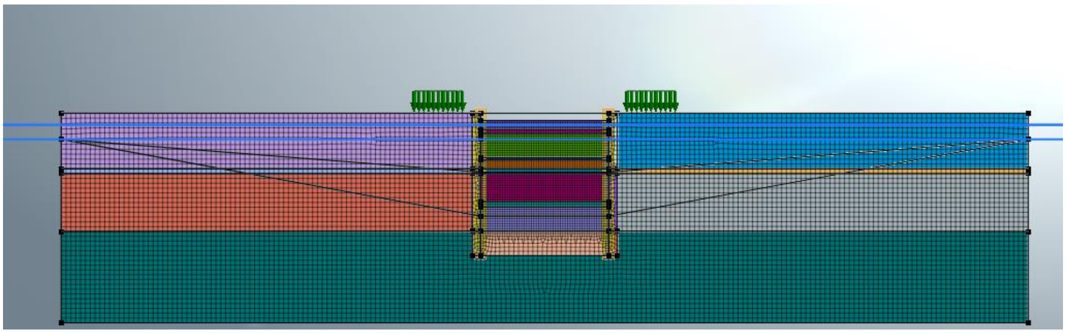

Figure 3. Boundary and load placement of the back-analysis model.

Step 2

Furthermore, from the results of the back analysis (step 1), soil parameters are applied to the undercrossing tunnel analysis. Different from step 1, this analysis approaches using a three-dimensional model to generate the effect of the undercrossing when construct beneath the existing tunnel. General data for the tunnel is taken from existing tunnel data.

Table 7. Design criteria for tunnel properties.

Parameter	Description
Outer Dimension	6650 mm
Inner Dimension	6050 mm
Lining Material	Steel
Segment Material	Pre-Cast Lining
Length per segment	1500 mm

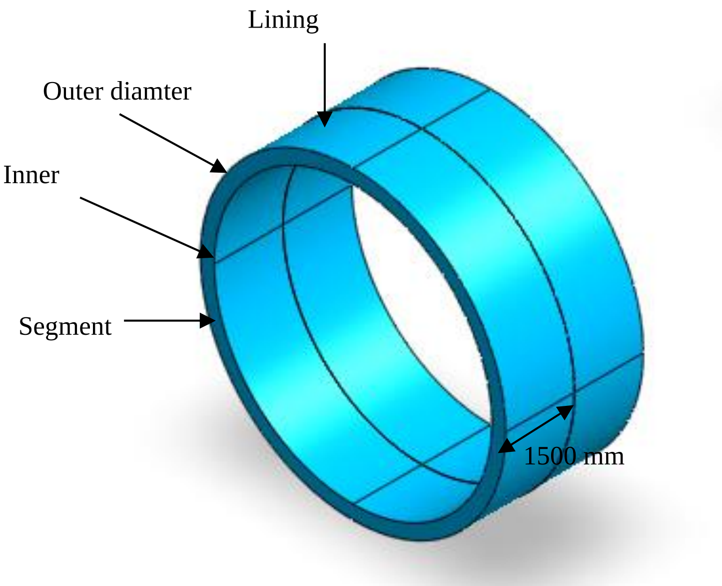

Figure 4. Description of TBM tunnel segment

The distance between the existing tunnel and the undercrossing tunnel is 0.5D. The stages of tunnel work sequentially from Existing Tunnel 1, Existing Tunnel 2, undercrossing tunnel 1, and undercrossing tunnel 2. Geometry model of 54 m x 54 m x 40 m is produced, where the length of the lining segment is 1.5 meters so that there are 36 pieces of segment lining. Determination of the length of the boundary model is conducted by considering the effect of displacement on the boundary model. Four measurement points are used for measuring the vertical displacement of the External Tunnel 1 (ET1) and External Tunnel 2 (ET 2) represented by ET1A, ET1B, ET2A, and ET2B.

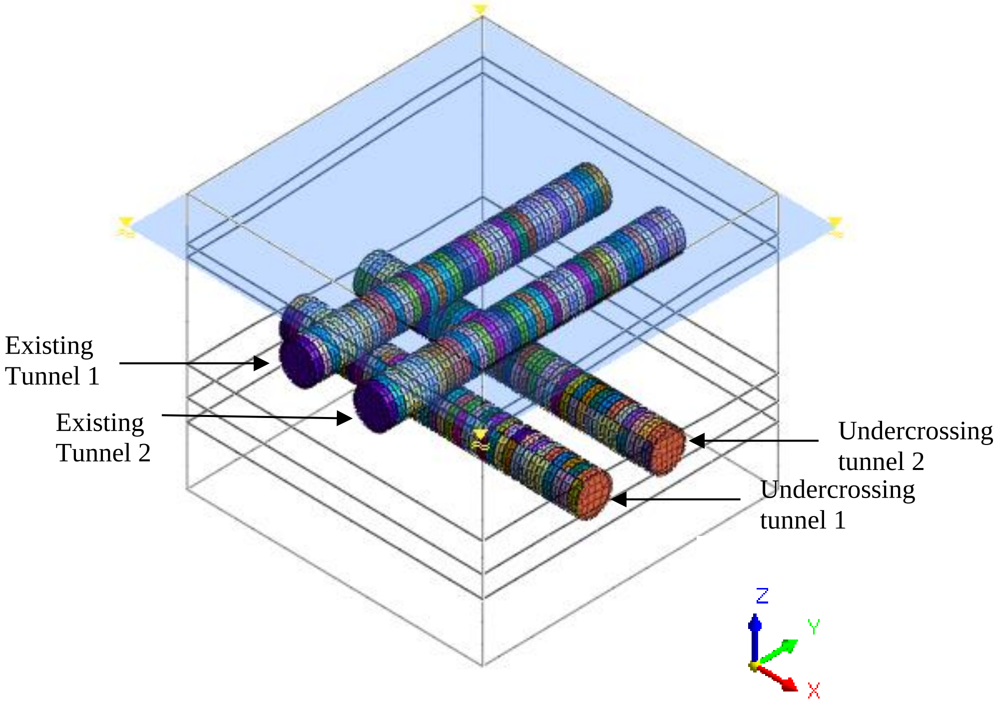

Figure 5. Undercrossing tunnel analysis isometric model

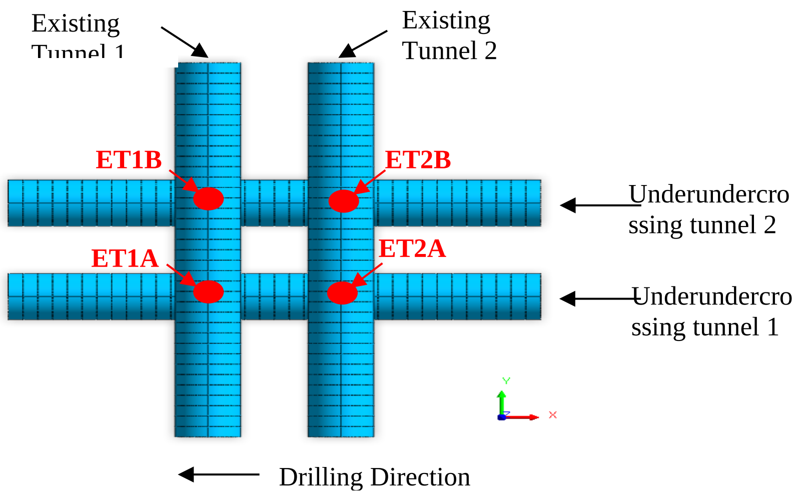

Figure 6. Measurement points for vertical displacement.

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The Tunnel Boring Machine (TBM) excavates the soil using a certain pressure and temporary lining application is conducted, this temporary lining uses steel material. After that, 9 meters behind the cutter face, there is a machine that installs segments of precast concrete (segment erectors) which then steel lining will be replaced with concrete grouting at 9 meters behind the segment erectors to fill the cavity between the ground and the precast concrete segment.

Table 8. Strength properties of tunnel lining

Parameter	Density (kN/m^3)	Elastic Modulus (kN/m^2)
Pre-cast segment	24	2.87 E07
Steel lining	78	2.50 E08
Grouting	22	10.0 E06

According to Mohammed (2017) there are four types of pressure applied in designing tunnels using the TBM method. Ma et al (2022) showed the greater drilling pressure will lead to smaller deformation obtained. Jack pressure is the force exerted by the TBM on the segment that has been installed behind the digging machine to push the machine in the direction of the drilling, the value of jack pressure depends on the surface area of the segments where the average pressure of jack thrust is 12400 kN and it evenly divided to the segment surface area, the pressure would be 1900 kN/m^2. The drill pressure is 90% from the total thrust force which is 11160 kN divided by the area of drilling (the inner circle), so the pressure would be 380 kPa. The frictional force between the steel shield and the surrounding ground (Shield pressure) of the twin tunnel applied as the value is determined by the depth of the tunnel (Ren, 2022). The grouting pressure applied at the tail of the TBM after the pre-cast segmental lining applications, the value of 1800 kN/m^2 applied for this research as this pressure ranged from 1800 to 3000 kN/m^2 (Ren, 2022).

Shield Pressure $= K_0 \gamma H$ (Ren, 2022).

Where K_0 is the static soil pressure coefficient, γ is the unit weight of the soil above the tunnel, and H is the distance between ground surface and the tunnel (depth).

RESULT AND DISCUSSION

Back Calculation Result (Step 1)

Back Analysis has been conducted using two correlation methods, the upper bound and the lower bound.

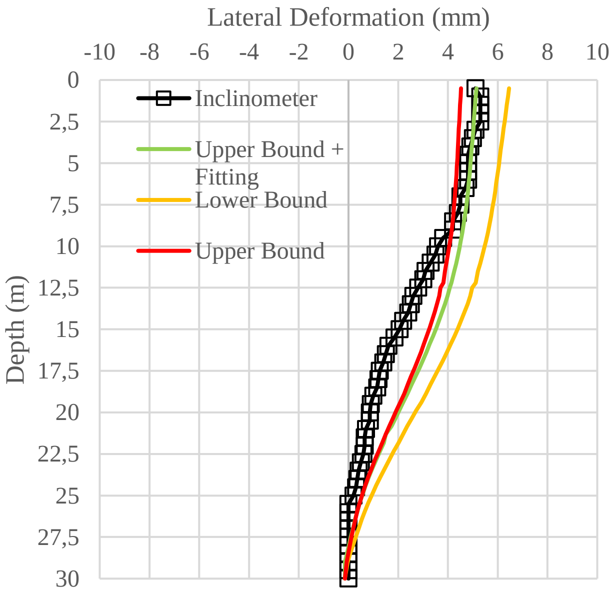

Figure 7. Back Analysis result compared to monitoring and upper-lower bound.

At the top of the D-Wall, using the upper bound value, the result of maximum lateral deformation was 4.65 mm, underestimated 12% from the recording on the inclinometer that showed a value of 5.1 mm. Using the lower bound value, the result of maximum lateral deformation overestimated 27.5% of the value read by the inclinometer. Where the maximum value of lateral deformation using lower bound value is 6.45mm.

At the bottom of the D-Wall, the discrepancy was not truly clear captured by the fitting. Although the discrepancy might seem at the middle of the wall, this result was not affecting the fitting result. The use of E_{50}^{ref} values with a correlation of 3500N for clay and 2800N for sand produced maximum lateral deformation value that were aligned with the results of the inclinometer reading. Then, the soil parameter updated to those correlations (Table.9)

Table 9. Result of back-fitted elastic properties of hardening soil model

Geotechnical Parameter	Very Soft Silty Clay	Very Loose Silty Sand	Very Stiff Silty Clay	Medium Dense Silty Sand	Dense Sand
E_{50}^{ref} Back-Fitted	4936	8400	70518	31000	162400
E_{oed}^{ref} Back Fitted	3455	5880	49363	217000	113680
E_{ur}^{ref} Back Fitted	14808	25200	211556	93000	4878200

Under Crossing Analysis Result (Step 2)

After obtaining the actual soil parameter value from step 1, tunnel undercrossing analysis of the existing tunnel was conducted. The result of the twin tunnel undercrossing with 0.5D (diameter) showed the movement of vertical displacement in the existing tunnel. The behavior of the model with a distance of 0.5D undergoes uplift vertical deformation that occurs due to TBM drilling at UT 1 and UT 2. There are 4 phases in vertical deformation that occur in undercrossing tunnel drilling against vertical deformation of existing tunnels.

The first phase is when the TBM surface approaches the existing tunnel, the vertical deformation increases when the pressure comes from the machinery that drives the ground lift and the peak is when the TBM face reaches 1D – 1.5D after the center line of the existing tunnel.

The second phase occurred 9 meters after the first phase, vertical deformation decreased due to the installation of segmental lining that previously had gaps after excavation resulting in ground loss under the existing tunnel. The third phase occurred about 9 meters after the second phase, this is the second uplift that occurred during construction due to grouting to fill the gap between the soil and the segment layer. The fourth phase is when the TBM moves away from the existing tunnel, the vertical deformation slowly decreases again.

The behaviour on ET1A produce the largest vertical deformation in the form of uplift accumulation was obtained when grouting work was carried out during UT2 drilling with a distance of 0.5D whose value reached +3 mm. in ET2A, the vertical deformation change is greater when drilling UT1 compared to UT2 where the difference is about 1 mm, this is due to its closer distance to UT1. It is clearly seen in the model, after the uplift there is land subsidence due to a gap between the segmental lining and the ground which results in significant soil loss causing a decrease in the ET1 structure.

ET1B shows the largest uplift vertical deformation accumulation value compared to other reference points of +4mm obtained in UT2 drilling. In terms of uplift vertical deformation changes, this reference point also undergoes the most significant change of +3 mm. This is because this reference point is at the last stage of construction, namely when UT2 drilling is carried out at the end of the construction stage.

Conversely, ET2B shows a value of vertical deformation change at UT2 drilling which is greater than when drilling UT1, which is around 2 mm value. The cause is similar because the ET2B reference point is closer to UT2.

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The reference points ET1A and ET1B experience peak values (in model 1) and lowest values (in models 2 and 3) after ET2A and ET2B, this is because ET2A and ET2B are affected by TBM construction first. This also results in greater deformation of ET1B and ET1A compared to ET2A and ET2B due to the deformation influence of ET2A and ET2B which pushes towards ET1A and ET1B.

These results show that each reference point behaves almost the same. Liu et al. (2022) when conducting research on the effects produced by the undercrossing tunnel of the existing Double-O Tunnel (DOT) which is 3 meters away on soft cohesive soil, showed the same trend line and phase results.

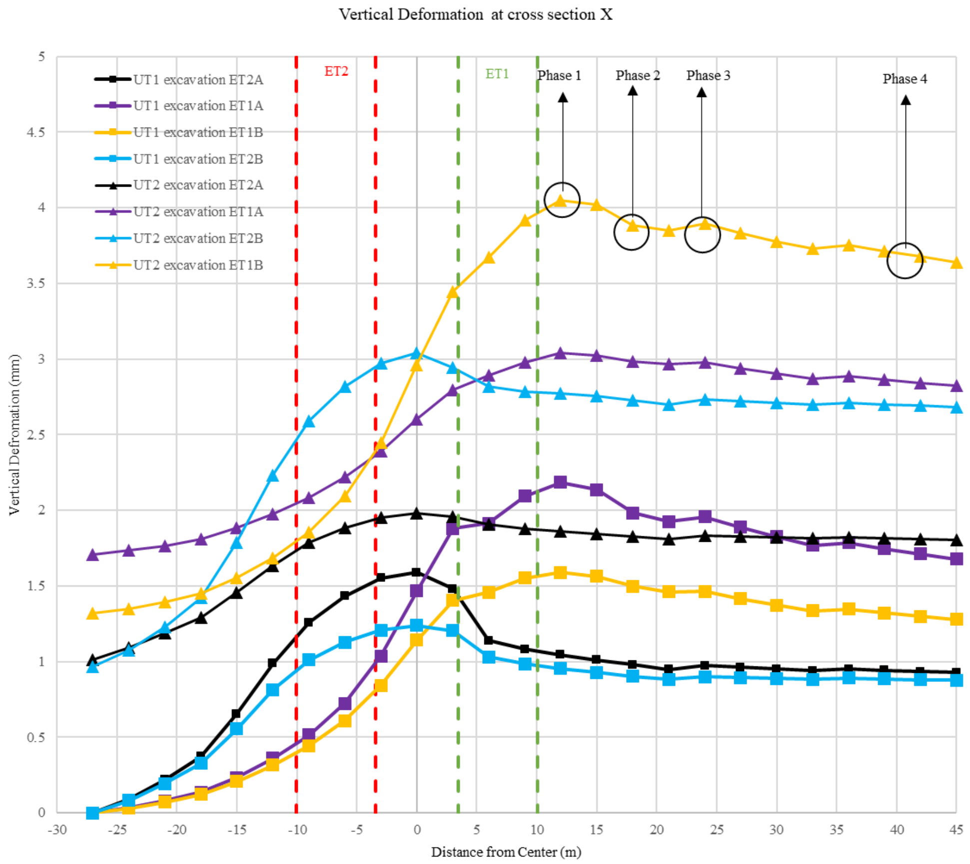

Figure 8. Vertical deformation at X Section.

CONCLUSION

This paper discussed the effect of the construction of twin undercrossing tunnel Earth Pressure Balance Shield (EPBS) structures, one of the Tunnel Boring Machine (TBM) methods, on existing twin tunnel structures at Jakarta area. The vertical distance between the undercrossing tunnel (UT) and the existing tunnel (ET) is set at 0.5D or 3.325 m. The overall structure of ET and UT is at a depth of -10 m and -13 m in a very soft cohesive soil profile about 20 meters thick.

The research was divided into two steps, all of the steps are using Finite Element Analysis Midas GTS NX. Soil model uses hardening soil and soil strength parameters using Undrained A with effective parameters. Step 1 is a back-analysis of soil parameters with the back-fitting method of hardening soil model elasticity values (E_{50}^{ref}, E_{oed}^{ref}, E_{ur}^{ref}) which are correlated to NSPT values. The results of the back analysis are compared to the results of lateral deformation read by monitoring inclinometer on the D-Wall excavation work of MRT Jakarta Station. Step 2 is an undercrossing tunnel analysis using soil parameters from the back analysis in step 1. The emphasis of the analysis is on the results of vertical deformation that occurs when the construction of UT1 and UT2 against ET1 and ET2.

The results of step 1 back-analysis showed a maximum lateral deformation movement of 5.1 mm at the inclinometer reading. The results of the analysis using the upper bound parameter (E_{50}^{ref} = 4000 N for clay; E_{50}^{ref} = 2800 N for sand) produce a maximum lateral deformation value that underestimates about 12% of the monitoring reading value, where the upper bound parameter produces a maximum lateral deformation of 4.65 mm. As for the analysis using lower bound parameter (E50 = 2800 N for clay; E50 = 2000 for sand) values, an overestimate of 27.5% was obtained, where the maximum result of lateral deformation was 6.45 mm. The back-fitting results stated that the correlation of 3500N for cohesive soil and 2800N for sand soil produced values that matched the lateral deformation pattern and lateral deformation values.

The results of step 2 undercrossing tunnel analysis show results where the vertical displacement of the existing tunnel due to undercrossing has four stages. The first stage of uplift occurs when the TBM face tunnel passes through the 1D-1.5D reference point. The second stage decreased the vertical deformation due to the installation of segments by segment erectors at 9 meters behind the drilling face which resulted in ground loss. The third stage is the second uplift due to grouting about 9 meters behind the segment erectors. The fourth stage decreases in vertical displacement as the TBM moves away. The largest vertical displacement occurs after the excavation of UT2 reaching 4 mm.

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