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GEOTECHNICAL INVESTIGATION OF CAVITY AND SETTLEMENT AT BESRAYA HIGHWAY, JALAN KUCHAI LAMA

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ABSTRACT

A geotechnical investigation of cavity and settlement has been conducted at Besraya Highway, Jln Kuchai Lama due to pipe jacking works to determine subsurface condition at the proposed site and to identify the causes of settlement. In this research study, JKR Probe test and Electrical Resistivity Imaging (ERI) method were applied in order to achieve the objectives. Analyses of the settlement problem also were carried out based on soil parameters obtained from JKR Probe data using Two-Dimensional (2-D) finite element method (FEM) analyses with PLAXIS Finite Element Code. Based on JKR Probe test results, it is found that the soil layer is loose to medium dense for middle of line. While for line 1, the soil was found in fully saturated condition since there is an evidence of water pocket when the rod was pulled out from the ground. For ERI results, the soil layer was found very weak with presence of saturated condition. Besides that, there is an appearance of water pocket based on images of ERI resulting formation of cavities and voids which beneath the existing pavement structure. From the geo forensic stimulation, the development of excess pore water pressure and seepage in the loose unconsolidated deposit provide additional volume change to the material, causing excessive settlement. The results indicated that the groundwater seepage and dissipation is the main problem to the ground settlement issue with the contribution of pipe jacking work. Therefore, a mitigation step is a need in order to stop the major failure occur due to excessive settlement.

Keywords: electrical resistivity imaging, cavity, settlement, finite element, plaxis.

1. INTRODUCTION

Increasing of groundwater level is a contribution from various sources such as infiltration of rain water especially in the tropic region. The effect of wetness on shear strength of soil is characterized based on a stress state variable known as suction. The suction effect is derived from the surface tension force on the water meniscus, which clings between soil particles and generally its magnitude increases as moisture decreases (Kaye *et al*. 1973). However, shear strength does not indefinitely increase with suction since it started to decreased beyond residual suction (Escario *et al.* 1989), (Gan *et al*. 1996), Noor *et al*. 2006), (Toll *et al*. 2000). The apparent shear strength reduction due to the surface water infiltration is actually governing the failure (Rahardjo *et al.* 1991). A subgrade voids or cavity detection by drilling method experienced numbers of limitations such as high in cost and time taken as well as minimum coverage of data. Nowadays, a geophysical technique such as electrical resistivity has increasingly adopted in many projects related to engineering and environment due to its ability to be implemented less expensively, fast and can cover larger areas more thoroughly (Khatri *et al*. 2011, Liu and Evett, 2008, Godio *et al.* 2006, Cozenza *et al.* 2006) [1-4]. Rosli et al. used the resistivity technique for slope failure monitoring and they found that the factor which causes landslide is the subsurface boulders and the saturated zone which result subsidence of the surface. All methods of electrical resistivity imaging including field procedures and data conversion were referred to Instruction Manual fo Terrameter SAS 4000 (ABEM 2009) and (Loke *et al.* 1996).

Electrical resistivity imaging (ERI) method is significantly sensitive to variations in earth resistivity property with reference to water occurrences (Owen *et al*. 2006) [5]. It is the most well-liked geophysical techniques applied for subsurface mapping purposes (Hazreek *et al.* 2015) [6]. The application of ERI is vital since it has been proved to be the most successfully tools in groundwater resource mapping as the groundwater movement and existence are largely localized and difficult to predict (Juanah *et al.* 2012) [7]. ERI can detect water-saturated clay thru lower resistivity zone due to the apparent resistivity changes with depth at the measuring point (Kim *et al.* 2011) [8]. Soil resistivity value can be varied due to the variation of basic geotechnical properties such as moisture content, densities, void ratio, porosity and grain size fraction (Hazreek *et al*. 2015, Abidin *et al*. 2013, Abidin *et al*, Abidin *et al*. 2014).

Tables 1 and 2 show the resistivity and conductivity values of some of the typical earth materials (Lee, 2002) and representative values (McCarthy, 2007). Igneous and metamorphic rocks typically have high resistivity values. The resistivity of these rocks is mainly dependent on the degree of fracturing. Since the water table in Malaysia is generally shallow, the fractures are commonly filled with ground water. The higher the fractures the lower is the resistivity value of rock. Areas that contain high moisture content become saturated zone. The higher the saturation area, the lower is the resistivity value of the ground material.

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Table-1. Typical resistivity values of earth materials (Lee, 2002).

Material	Resistivity (ohm.m)
Clay and saturated silt	0 – 100
Sandy clay and wet silty sand	100 – 250
Clayey sand and wet silty sand	250 – 500
Sand	500 – 1,500
Gravel	1,500 – 5,000
Weathered rock	1,000 – 2,000
Sound rock	1,500 – 40,000

Table-2. Representative resistivity values (McCarthy, 2007).

Material	Resistivity (ohm.m)
Wet-to-moist clayey soils	1.524 - 3.048
Wet-to-moist silty clay and silty soils	3.048 - 15.24
Moist-to-dry silty and sandy soils	15.24 - 152.4
Well-fractured to slightly fractured bedrock with moist soils filled cracks	152.4 - 304.8
Sand and gravel with silts	304.8
Slightly fractured bedrock with dry soils-filled cracks; sands and gravel with layers of silts	304.8 - 2438.4
Massive bedded and hard bedrock; coarse dry sand and gravel deposits	2438.4 +

This study was purposely performed to investigate the cause of settlement at BESRAYA highway at Jalan Kuchai Lama section due to the previous pipe jacking work where part of the section experienced post construction settlement after one (1) year of operation. This study was involved with ERI and JKR probe test.

2. PROBLEM OCCURRENCE AT PROPOSED SITE

A pipe jacking work was constructed to channel the sewerage pipe for a 22-storey service apartment which runs underneath the BESRAYA highway at Jalan Kuchai Lama section. One (1) year post construction, cavities are present in the middle of the tunnel crown of about 1.5 m across the highway shown in Figure-1.

Figure-1. Cavity appearing underneath the road surface.

The section was excavated, and soil was filled into the cavities and compacted shown in Figures 2 and 3. However, there is water ponding in the existing cavity. A year later, the sink hole occurred again underneath the pavement structure, but the sub base was stiff enough. Therefore, the section above sagged. Since the condition was aware the settlement might re-occur, the further investigation was carried out to identify the cause of cavity building up and settlement occurring across the highway section.

Figure-2. Excavation worlk to observe cavity beneath the pavement surface.

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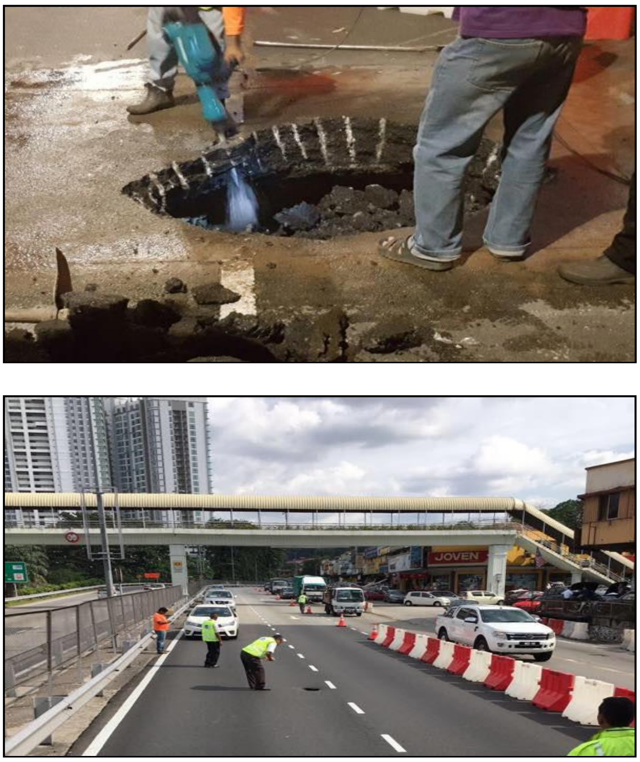

Figure-3. Filling of fill material.

3. METHODOLOGY

3.1 JKR Probe Test

The purpose of the JKR Probe test is to determine the soil profile including the overburden soils and the thickness of the bedrock. The test was conducted by compacting the rod according to the Standard number of blows of 200 within 30cm length of penetration. The equipment used for this test is such as rod of 1.2m, joint, hammer 5kg and spanner. The hammer was connected to the jointer and rod 1.2m where the rod was marked at every 30cm. The hammer was released freely while the rod is standing 90º vertically perpendicular to the ground surface. The number of blows were recorded for every 30cm of rod penetration. The test was stopped when there is no penetration or number of blows reached 100 blows per 30 cm.

3.2 Electrical Resistivity Imaging (Eri) Method

Electrical imaging was performed using the ABEM Terrameter SAS 4000, combined with ES 10-64 electrode selector shown in Figure-6. A 2-D electrical imaging survey is usually carried out with a computer-controlled resistivity-meter system connected to a multi-electrode cable shown in Figure-4. The control software automatically selects the appropriate four electrodes for each measurement to give a 2D coverage of the subsurface

Figure-4. Electrical resistivity equipment.

The sketch shown in Figure-5 outlines the use of the ABEM Terrameter SAS 4000 Lund Imaging System in a 2D survey. Each mark on the cables indicates an electrode position. The cables are placed along a single line. The total layout length depends on the spacing between the nodes, but is usually between 160 meters and 800 meters.

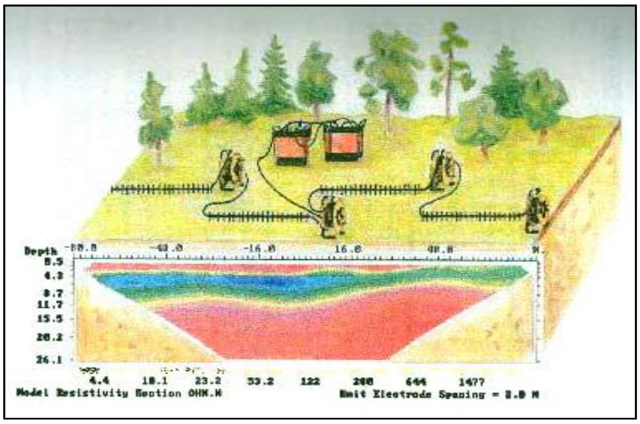

Figure-5. Field arrangement for the ABEM Lund system.

3.2.1 Data acquisition and processing

The resistivity lines were set out along the proposed alignment. A number of 41 electrodes were pinned into ground in a straight line with equal spacing. A computer-controlled system is then used to automatically select the active electrodes for each measure. A fixed single line of electrical resistivity survey was performed across the target of interest zone. Testing configuration was based on Wenner array given in Table-3. Raw data obtained from data acquisition was processed using commercialized RES2DINV software to provide an inverse model that approximates the actual subsurface structure. The inversion algorithm of RES2DINV was used to process the data, as proposed by, in order to obtain the 2-D electrical results.

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Table-3. Test configuration.

No	Setting	Description
1	Array	Wenner
2	Electrode spacing	5 m
3	Total number of electrodes	41
4	Total number of small jumper cable	40
5	Total length of 2D resistivity test	200 m

3.3 Geoforensic Method Analysis

Finite element analysis was carried out to evaluate conditions of site for geotechnical forensics investigation. Back analysis for existing conditions was performed in order to simulate failure behaviour of the proposed site. This is important to trace the main cause leading to the problem. Analyses of the settlement problem was carried out using Two-Dimensional (2-D) finite element method (FEM) analyses with PLAXIS Finite Element Code. The sophisticated FEM is necessary because of the complexity of soil-structure interaction problem, involving subsoil-slab-fill and vice versa.

3.3.1 Stages of construction in FEM analyses

In order to investigate the settlement problem, the stages of construction for the FEM analyses are shown in Table-4.

Table-4. Stages of construction for FEM 2D analysis

Stage of Construction	Description
0	Initial condition of the subsoil (before construction)
1	Existing traffic loading on the road embankment
2	Existing road embankment and pre-tunnelling assessment
3	Post-construction settlement 2 years after tunnelling work

All of the parameters used in the FEM model are in drained condition. The results of analyses from Stages 1 and 2 would represent the actual condition after the pipe jacking procedure.

3.3.1.1 Stages 0: Initial condition of the subsoil (before construction)

Initial condition of the subsoil refers to the condition where there is no construction was carried out. It is based solely on the subsoil strata of the original soil. The subsoil parameters were based on the SI carried out after the defects of the structures were analyzed as shown in Figure-6.

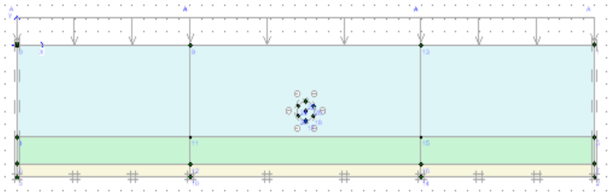

Figure-6. Initial condition.

3.3.1.2 Stages 1: Existing traffic loading on the road embankment

The existing road embankment was analyzed taking into account the traffic loads experience by the pavement before the construction. This is to assess the behavior of the road embankment before the pipe jacking procedure is executed as shown in Figure-7.

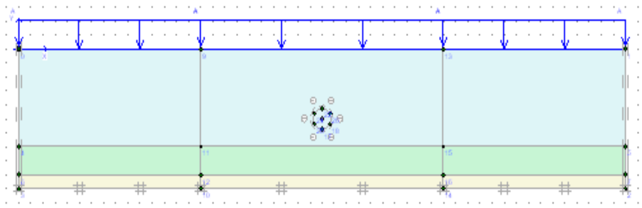

Figure-7. Existing traffic load on the road embankment.

3.3.1.3 Stages 2: Existing road embankment and pre-tunneling assessment

The existing road embankment was analyzed considering the traffic loads experience by the pavement during construction. At this stage, settlement during construction is also analyzed and the construction period is about one year as shown in Figure-8.

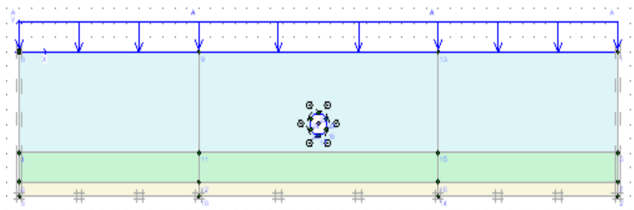

Figure-8. Pipe jacking procedure.

3.3.1.4 Stages 3: Post-construction settlement after tunnelling work

This stage refers to the calculation of settlement about 2 years after completion of the construction. The duration taken as 2 years based on the age of the constructed structures before defects as shown in Figure-9.

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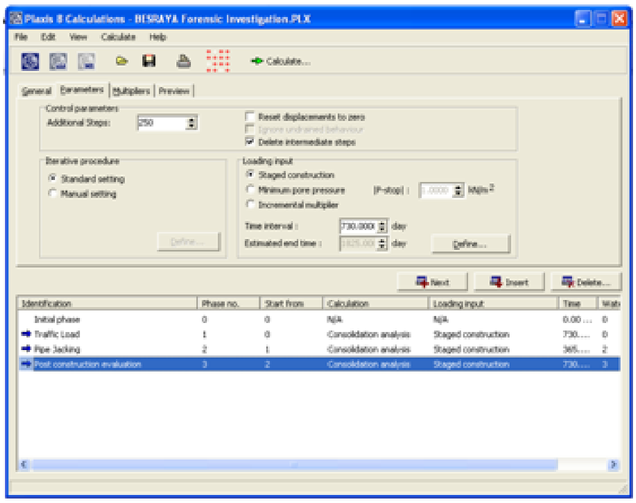

Figure-9. Construction stage.

4. RESULTS AND ANALYSIS

4.1 JKR Probe Analysis

Results of JKR Probe for the proposed above mentioned project were stipulated in JKR Probe MP1 – MP5 for Lines 1, 2, 3 and 4. Based on the results of the JKR probes, for the first 0.3 m, the soil is considered to be medium dense with the no. of blows ranging between 30 to 80 blows/300 mm. However, as it reaches only 0.6 m the soil becomes very dense with blows exceeding 200/300 mm. This indicates that the platform is founded on very dense silty SAND and it seems that the water table is 1.2 m below the fill profile. Figure-10 through 24 illustrate the ground profile and the expected groundwater beneath the pavement.

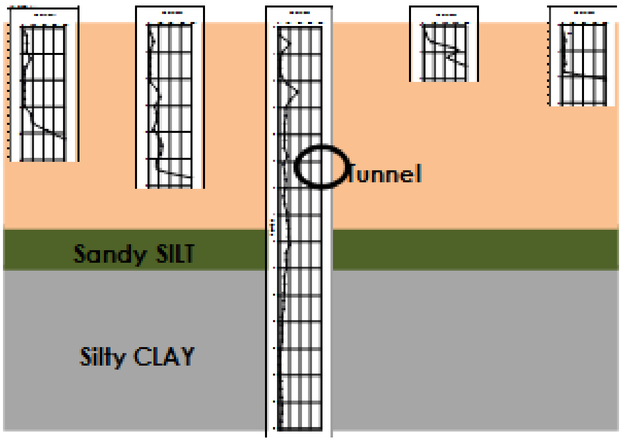

Figure-10. Subsoil profile for Line 1.

As shown in Figure-11, the center profile where the tunnel is installed showed loose to medium dense silty SAND. The groundwater table is expected at 1 m below the pavement surface due to the sudden reduction of the number of blows from the JKR probe. In addition, the values from the probe indicate low values hence, this promote to the weakening on the soil layer below the tunnel. There is a possibility of the silt/clay layer experiencing additional settlement since the bearing resistance is low.

Figure-12 shows the control line to observe the initial condition of the fill, whereby the silty SAND is very dense. The groundwater table is expected at 0.9 m below the pavement surface due to the sudden reduction of the number of blows from the JKR probe. This shows that the existing fill was compacted to the specification in accordance to the PWD standard.

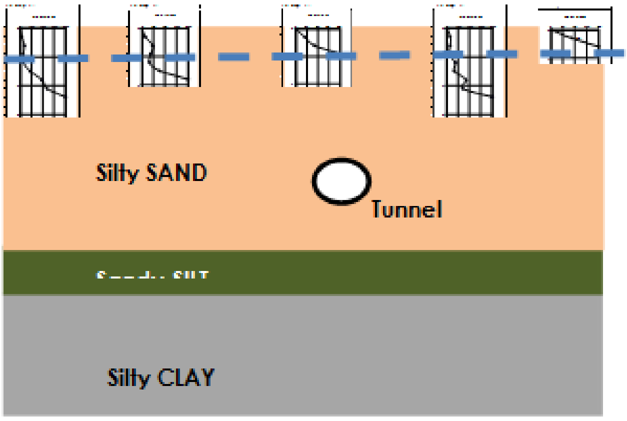

Figure-11. Subsoil profile for Line 2.

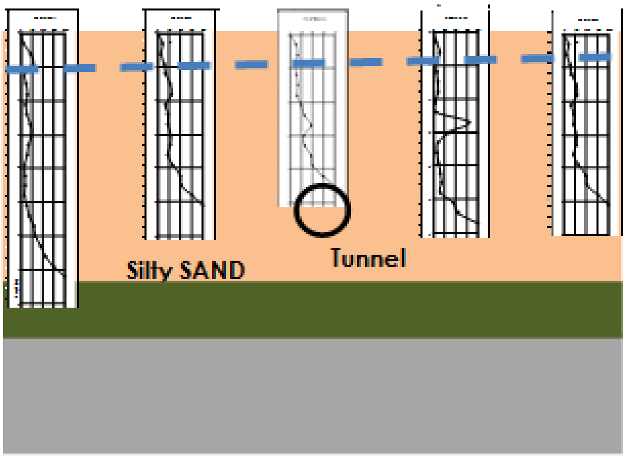

Figure-12. Subsoil profile for Line 3.

Based on the probe readings, it is observed that the water table varies from 0.5 m below the pavement surface down to 1.2 m below the pavement surface. This indicates variation of hydrostatic pressure regime; hence the weakening layer will be located at the downstream of this line. In fact, the last JKR probe point indicates lower probe value at the upper layer and the values increase down to 7.2 m. At the center of the line, the probe detected the pipe crest; therefore, the value is extremely high. The weakening at point 4 shows that there is seepage eminent, hence at this section, there is sagging of the pavement structure.

From the probe values shown in Figure-13, the groundwater is presumed to be located at 0.9 m from the ground surface since there is a reduction in value for the particular points from points 2 and 5. However, the fill is

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compacted to the specification in accordance to PWD except for point 2 where the readings are medium dense to dense.

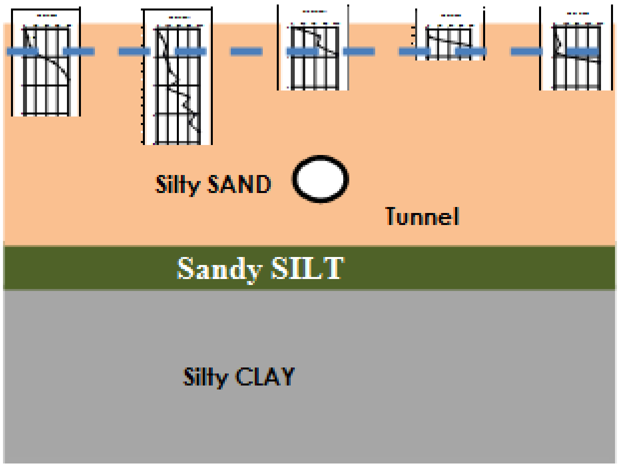

Figure-13. Subsoil profile for Line 4.

According to the observation and probing executed, the summary of geotechnical subsurface can be divided into 4 lines. As for line no. 2, the subsurface were very dense and suitable to carry load. However, in the middle of the line which in this case for MP3, the subsurface were very weak and even when the JKR probe reaches 15-meter depth, the soil layer is still weak with the result of 20 numbers of blows per feet. It shows that on that particular location, the soil layer is very weak due to the occurrence of groundwater beneath the pavement structure. Results of the probing for the rest of the lines, especially in the middle of the line showed that the soil layer is very weak. In addition, there was also evidence of water pocket as the rod was pulled out from the ground.

4.2 Electrical Resistivity Imaging Analysis

The problematic subsurface profile due to ground settlement was successfully being investigated using electrical resistivity imaging (ERI). The geometry and electrical resistivity anomaly distribution has been determined by analyzing ERI data obtained along the settlement zones and the result has shown a good correlation with the JKR probe data. Figure-14 until Figure-17 provide the ERI for the lines as indicated previously. Figure-17 provides the ERI for the lines as indicated previously. The intepretation of resistivity index is shown in Table-5 for Line 1,2,3 and 4. Table 5 shows the intrepretation results of resistivity for Line 1.

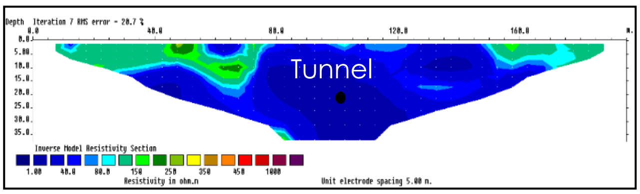

Figure-14. Electrical resistivity image for Line 1.

Table-5. Interpretation of electrical resistivity imaging for Line 1.

Data Name	Colors	Resistivity (ohm.meter)	Descriptions (after Lee, 2002)
Besraya		0 - 1.00	Clay and saturated silt
		- 20.5	Clay and saturated silt
		20.5 - 40.0	Clay and saturated silt
		40.0 - 60.0	Clay and saturated silt
		60.0 - 80.0	Clay and saturated silt
		80.0 - 115	Clay and saturated silt, sandy clay and wet silty sand
		115 - 150	Sandy clay and wet silty sand
		150 - 200	Sandy clay and wet silty sand
		200 - 250	Sandy clay and wet silty sand
		250 - 300	Clayey sand and wet silty sand
		300 - 350	Clayey sand and wet silty sand
		350 - 400	Clayey sand and wet silty sand
		400 - 450	Clayey sand and wet silty sand
		450 - 725	Clayey sand, wet silty sand and sand
		725 - 1000	Sand
		>1000	Sand
		>1000	Sand

According to the image given in Figure-14, it clearly showed that 1 m below the ground surface, the soil is fully saturated and loose. However, below the tunnel, since the sandy SILT and silty CLAY layer is an impermeable boundary with water flow restriction, it retains its moisture capacity and could easily cause consolidation settlement if the excess pore water pressure is present due to the loading capacity from the traffic above. The tunnel above may experience changes due to these conditions and if the settlement is huge enough, it would realign the tunnel vertical alignment and could cause leaking to the tunnel. In fact, from the probe data observed, the layer below the tunnel at this section is weak and loose which could cause probable settlement issues to the tunnel, hence the ground above. This is also proof of internal erosion occurring below the tunnel section due to the saturated condition of the silt or clay layers. Based on visual inspection, the settlement occurred at about the tunnel crest, slightly eccentric from the alignment at about 90 m from the left end.

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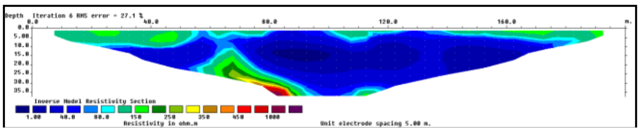

Figure-15. Electrical resistivity image for Line 2.

Figure-15 shows the control Line 2 where the layer closed to the surface is well compacted. However, from 80 m to 100 m distance, the section is fully saturated and this causes the surface to settle at this section. At this section, the saturated zone is about 5 m below the ground surface, the soil is loose. Moreover, below the tunnel, since the sandy SILT and silty CLAY layer is an impermeable boundary with water flow restriction, it retains its moisture capacity and could easily cause consolidation settlement if the excess pore water pressure is present due to the loading capacity from the traffic above. However, the hard layer is encountered below the level which provides additional resistance to settlement. In fact, from the probe data observed, the layer below the tunnel at this section is dense to very dense hence could restrain the ground from excessive settlement.

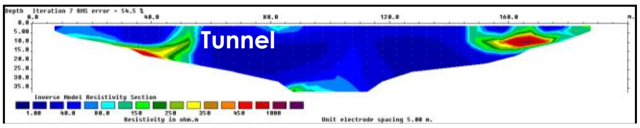

Figure-16. Electrical resistivity image for Line 3.

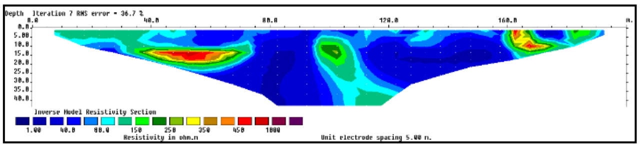

Figure-17. Electrical resistivity image for Line 4.

Based on the image shown in Figure-17, this section is stable since the hard layer is below the tunnel horizon. The resistivity image showed that part of the ground is fully saturated approximately 1.2 m below the surface and there is water pocket on the left side and right side of the diagram. It is seen that the hard layer is encountered below the ground level which provides additional resistance to settlement. From the probe data observed, the layer below the tunnel at this section is dense to very dense hence could restrain the ground from excessive settlement.

According to the image and probing executed, the summary of geotechnical subsurface can be divided into 4 lines. As for line no. 2, the subsurface were very dense and suitable to carry load. However, in the middle of the line which in this case for MP3, the subsurface were very weak and even when the JKR probe reaches 15 meter depth, the soil layer is still weak with the result of 20 numbers of blows per feet. In addition, there is a presence of water pockets at all of the lines which, indicates saturation and potential internal erosion. Results of the image for the rest of the lines, especially in the middle of the line showed that the soil layer is very weak and presence saturated condition. This justified, the existence the of water pocket from the ERI.

4. GEOFORENSIC SIMULATION ANALYSIS

This section provides simulation and settlement analysis for the pipe jacking work pre and post construction conditions. The simulation is based on the data obtained from all aspects, which are previous borehole records, ERI and JKR probe interpretation. Figure 18 shows the deformation of the existing fill during traffic. The maximum settlement observed due to traffic is 469 mm. This indicated that the loose deposit settled in time due to self-compacting of the silty SAND layer. As it is dense, the fill platform becomes stable and the groundwater table is at hydrostatic condition.

Based on the image shown in Figure-18, this is the most critical line where the settlement occurred excessively eccentrically from the tunnel crest. The resistivity image even showed that the ground is fully saturated approximately 1.2 m below the surface and there is water pocket on the left side of the diagram. Moreover, below the tunnel, since the sandy SILT and silty CLAY layer is an impermeable boundary with water flow restriction, it retains its moisture capacity and could easily cause consolidation settlement if the excess pore water pressure is present due to the loading capacity from the traffic above. However, the hard layer is encountered below the level which provides additional resistance to settlement. From the probe data observed, the layer below the tunnel at this section is dense to very dense hence could restrain the ground from excessive settlement.

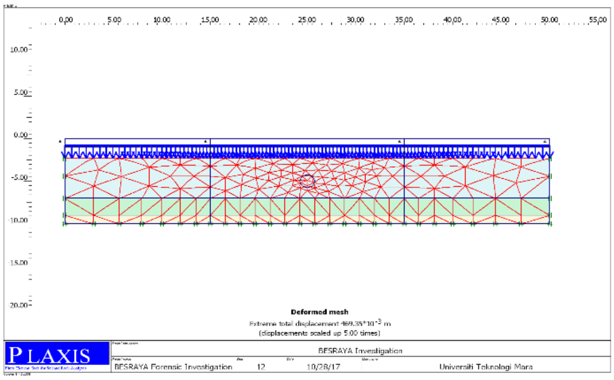

Figure-18. Deformation of the ground due to existing traffic load.

Figure-19 provides the deformation of the ground due to pipe jacking work. It is observed that the settlement is 529 mm at the crest of the tunnel from the ground surface. The deformation pattern followed the expected settlement trough due to micro tunneling work executed underneath and existing road structure. Figure-20 shows the settlement trough which, provided discomfort to the traffic users. Figure-21 shows the development of excess pore water pressures during the pipe jacking work which triggered settlement at the crest of the tunnel. Hence, due to the dissipation of the excess pore water pressure, the

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pavement structure could have experienced settlement during the pipe jacking work if temporary support work was not planned.

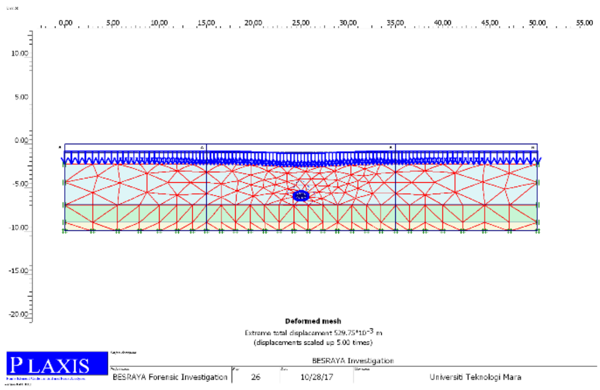

Figure-19. Deformation of the ground surface due to pipe jacking work.

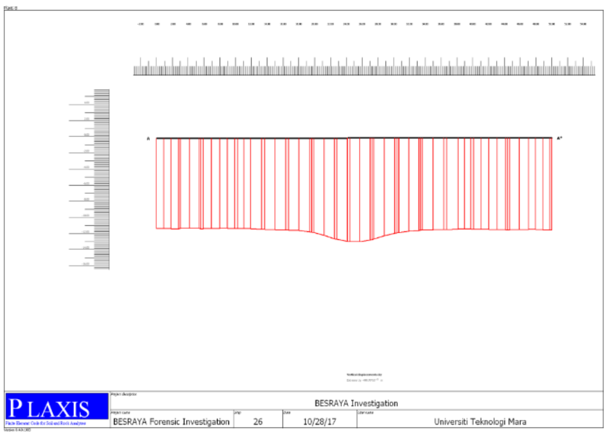

Figure-20. Settlement trough due to pipe jacking work.

Figure-21 shows the deformation of the ground surface after two (2) years of construction. Due to continuous traffic load, the ground has settled about 531 mm at the crest of the tunnel from the ground surface. This deformation does not include settlement due to erosion within the soil layer due to the presence of the water pockets and seepage. Integrating ERI and JKR probe, the settlement of the ground becomes excessive as the excess pore pressure dissipated, washed away all of the fines in the sand layer, introducing cavity and voids underneath the pavement structure. There are evidences that showed the presence of cavities and voids due to inundation, seepage and dissipation. Figure-22 illustrates the settlement through two (2) years post construction causing riding discomfort to the traffic users. Hence, if remediation work is not proposed, there is a possibility that the cavities and voids would reappear and cause more problems.

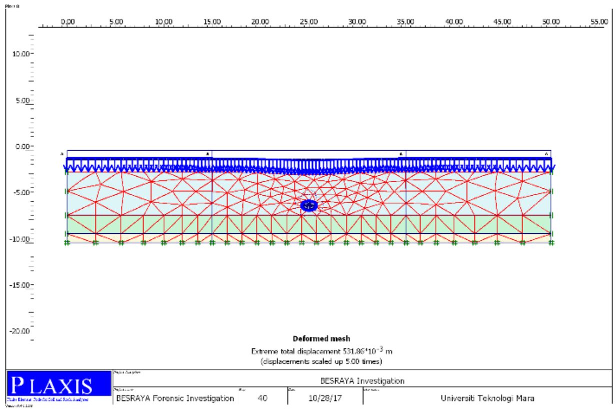

Figure-21. Deformation of the ground surface two (2) years post construction.

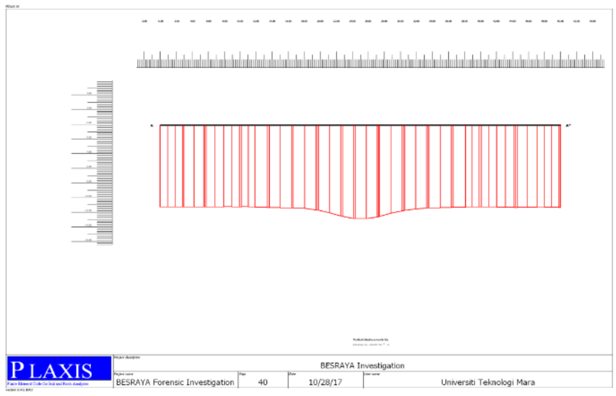

Figure-22. Settlement trough two (2) years post construction.

Figures 23 and 24 provide the development of excess pore water pressures during the pipe jacking work and two (2) years post construction. It is observed that the excess pore water pressure developed underneath the tunnel which could cause settlement as it dissipates. In fact, as the excess pore water dissipated two (2) years post construction, the pressure around the tunnel reduced; hence the deformation of the tunnel is less. Integrating ERI and JKR probe data, this showed significantly that the dissipation would trigger seepage and washed away of fines within the silty SAND layer. The issue caused the development of cavities and voids underneath the pavement structure as observed on site.

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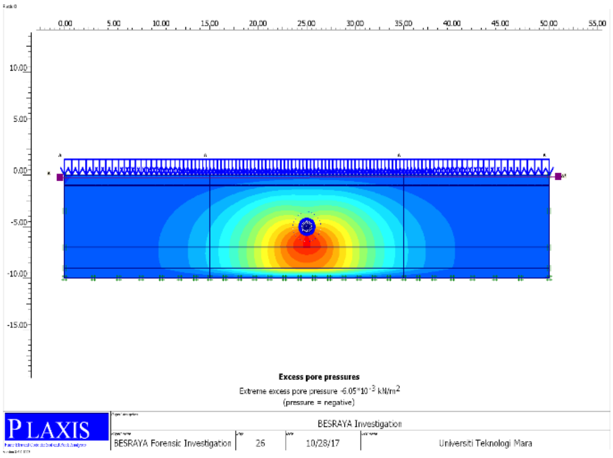

Figure-23. Excess pore water pressure development during construction.

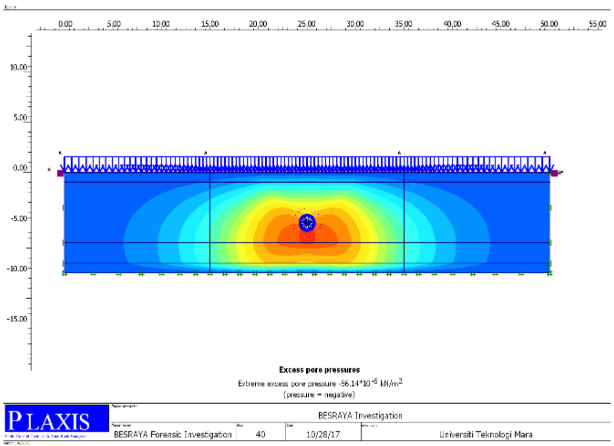

Figure-24. Excess pore water pressure dissipation two (2) years post construction.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Based on the geoforensic investigation conducted, the conclusions are given below:

a) The existing ground condition consist of 7.5 m thick silty SAND layer, underlain by 1.5 m thick of sandy SILT layer and finally at the end of the existing borehole results showed a hard-silty CLAY layer. The groundwater table is located about 1.5 m below the ground surface and part of the pipe jacking work undergo through a suitable fill platform. Therefore, the presence of water pocket in the loose unconsolidated material would cause the loss of fines due to internal erosion in the water pocket.

b) Results of the probing for the rest of the lines, especially in the middle of the line showed that the soil layer is loose to medium dense. The probe values for the line 1 showed low values of JKR probe suggesting that the soil is fully saturated. In addition, there was also evidence of water pocket as the rod was pulled out from the ground.

c) Results of the ERI for the rest of the lines, especially in the middle of the line showed that the soil layer is very weak and presence saturated condition. This justified, the existence the of water pocket from the ERI. The presence of the water pocket promotes to internal erosion that could wash away the fines, developing cavities and voids beneath the existing pavement structure.

d) From the geoforensic simulation, the development of excess pore water pressure and seepage in the loose unconsolidated deposit provide additional volume change to the material, causing excessive settlement. At the crest of the tunnel, the settlement trough is very large up to 0.5 m of settlement, hence providing riding discomfort to the traffic users.

As a summary, the pipe jacking work contributed part of the settlement issue. However, from the geoforensic investigation conducted, it is observed that the groundwater seepage and dissipation is the main problem to the ground settlement issue. If this element is not mitigated, future settlement and cavities will develop and cause more problems, not only to the concessionaire but the public as a whole.

5.1 Recommendation for the Proposed Remediation Works

For the geoforensic investigation conducted for the BESRAYA highway, Jalan Kuchai Lama section, the proposed remediation work is given below:

a) For stabilization of the loose ground, it is suggested to inject polyurethane (PU) foam/resin providing buoyancy and stability of the loose deposit. In addition, the impermeable characteristic of the PU foam/resin will divert water from flowing underneath the ground surface, reducing the seepage. Moreover, subsoil drainage is to be provided, to reduce the internal erosion occurring in the water pocket and providing better flow of water in the ground. Using polyurethane foam injection method, no excavation is required and small diameter holes are drilled to perform the work. The setting and curing time for PU foam is very fast, whereby it takes only 24 hrs. for the foam to harden. In addition, since PU foam is an expansive material and lightweight, the volume of liquid material before mix is less and acts as a buoyant layer for soil stabilization.

b) Another solution is by lightweight cementitious grout into the weak layer, to improve the density and bearing capacity of the weak layer. Cement increases the strength and durability of the platform, however increases the weight of the ground providing additional overburden for the underlain layer. The time of curing and hardening is approximately three (3) days but the volume of cement use varies, depending on the ground conditions. Subsoil drains are to be installed as well for providing better flow of water in the ground, reducing internal erosion and wash away of fines.

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c) The final solution is excavating and replacing the fill with lightweight material i.e. Expanded Polystyrene System (EPS) of polyurethane (PU) flatbed to reinstate the ground and reduce the effect of settlement. This method is not recommended since the traffic closure would be massive and time consuming whereby, the pavement structure is to be demolished, excavate the weak layer, replacing partly with the light weight material, then the suitable compacted fill and reconstructing the pavement structure. Subsoil drains are to be installed as well for providing better flow of water in the ground, reducing internal erosion and wash away of fines.

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