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Simulation and Settlement Monitoring from Geoforensic Investigation and Geotechnical Rehabilitation of Bridge Approach

Ismacahyadi Bagus Mohamed Jais*[1], Abdul Samad Abdul Rahman[1], Rafiee Abdul Razak[2], Saravanan Selvarajah[2], Ibrahim Husaini[2] and Mohamed Azizi Md Ali[3]

[1] GeoForenSIG, Faculty of Civil Engineering, Universiti Teknologi MARA, 40450 Shah Alam, Selangor, Malaysia
[2] SKVE Holdings Sdn Bhd, Pejabat Pentadbiran SKVE, KM 27, Arah Timur, Lebuhraya Lembah Klang Selatan Tanjung Dua Belas, 42600 Jenjarom, Selangor, Malaysia
[3] Geocon (M) Sdn Bhd, Lot 35, The Ideos Corporate Park, Lorong Sungai Puloh 8/KU6, Aman Perdana, 42200 Klang, Selangor, Malaysia

*Corresponding author: ismac821@uitm.edu.my

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Abstract

Transition approach settlement has always caused major problems, not only to the concessionaires, but also to the public. Maintenance budget of concessionaires has sky rocketed since the cost of pavement resurfacing and regulating increase every year due to routine works to improve and reduce the effect of differential settlement. The need to investigate and evaluate these transition approaches require detail preliminary and geoforensic investigation. The objectives of this investigation are a) to identify the cause of settlement at transition approach between the piled and the earth fill road embankments and b) to determine the settlement post construction. The preliminary and geoforensic investigations were conducted at South Klang Valley Expressway (SKVE) bridge BR12, specifically at abutment B and a proposed geotechnical rehabilitation, together with settlement monitoring were executed to evaluate the performance post rehabilitation. The finite element analysis is in good agreements with the monitoring data, two (2) years after the service of the expressway, hence a prediction was made to evaluate the suitability of the proposed solution, to ensure that it meets the requirements as stipulated by the Malaysian Highway Authority (MHA). In addition, settlement monitoring up to 15 months was executed to see the actual performance of the geotechnical rehabilitation solution. It is observed that the settlement post rehabilitation is within the tolerance as specified by MHA. The geotechnical rehabilitation solution does not involve major excavation and the work carried out is fast and economical, thus prolong the need to execute pavement surface maintenance by the concessionaire.

Keywords: Differential Settlement, Finite Element Analysis, Geoforensic Investigation, Transition Approach, Settlement Monitoring

1. Introduction

The South Klang Valley Expressway (SKVE) is an expressway in the southern part of Klang Valley, running through a densely populated region. It is a 51.7 km dual-carriageway providing links to the booming towns in the Klang Valley, which included the administrative capital of Malaysia, Putrajaya. The Section 2 stretch which is from Bandar Saujana Putra to Teluk Panglima Garang measuring 12.96 km was completed in 2011. (Wikipedia, 2022)

The abutment approach on the viaduct section before Teluk Panglima Garang exit experienced settlement and undulation. The section has three types of ground treatment where the first section behind the bridge approach which is approximately 8 m was constructed with stone columns in the foundation layer and Expanded Polystyrene System (EPS) with slab counterweight behind the abutment section shown in Figure 1. Then piled embankment was constructed at approximately 80 m long, 8 m away from the bridge abutment and finally, EPS with slab counterweight was installed down to the remaining road platform at a length of 110 m illustrated in Figure 2. The objectives of this investigation are a) to identify the cause of settlement at transition approach between the piled and the earth fill embankments and b) to determine the settlement post construction.

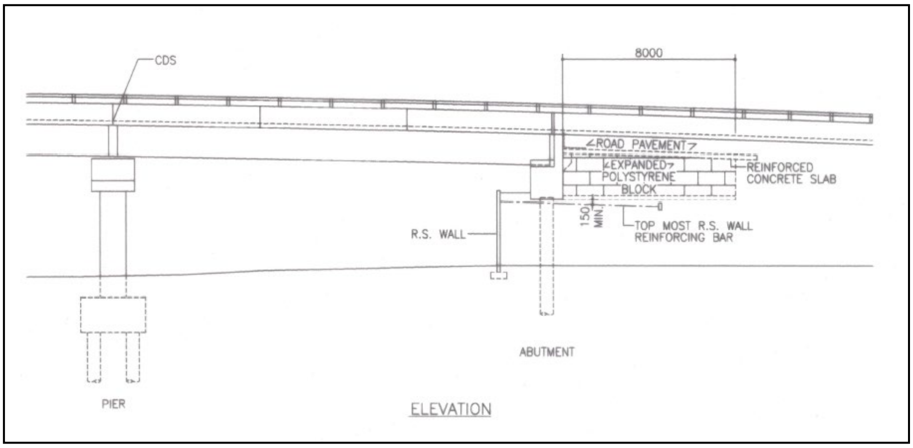

Figure 1. EPS section behind the bridge abutment B. (Courtesy of SKVE Holdings Sdn. Bhd.)

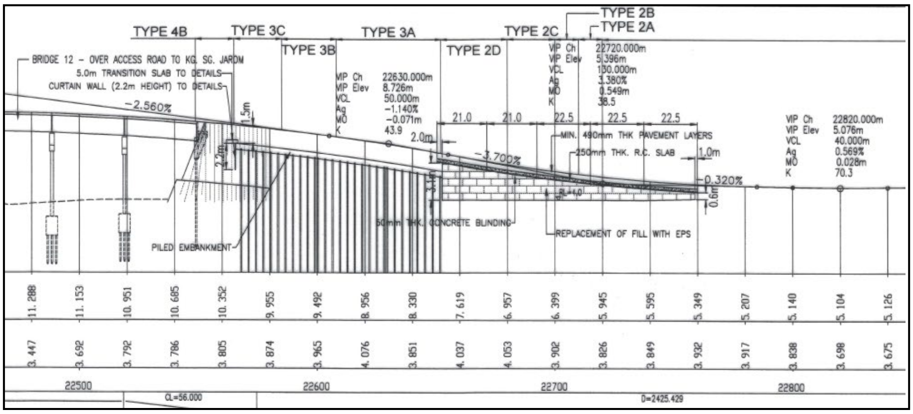

Figure 2. Full section and ground treatment for bridge approach. (Courtesy of SKVE Holdings Sdn. Bhd.)

1.1 Site Geomorphology and Geology

This geoforensic investigation was performed at bridge BR12 at KM31.00 East Bound and West Bound (Abutment B) at SKVE Highway near Teluk Panglima Garang, Klang as shown in Figure 3. The location of the project is situated at 02°54’09.7” N 101°29’33.2” E along the SKVE highway. Generally, the site topography consists of swampy and flat terrain surrounded by palm oil plantations and housing areas (Manap *et. al.*, 2012). The geological formation found in most of the site location situated in Teluk Panglima Garang and Jenjarom areas is dominated by phyllite, slate, shale, sandstone, peat, humic clay and silt (Department of Mineral and Geosciences, Malaysia). Moreover, the site location also consists argillaceous rocks which are mostly carbonaceous. Based on geological map shown in Figure 4, the soil variation consists of peat, humic clay and silt. The elevation of the surrounding areas between 20 to 177 m is typically flat. In general, the present of this type of materials will exhibits soft soil phenomenon due to its high-water content derived from high water table of the lowland areas (Manap et al., 2012).

Figure 3. Location of the site.

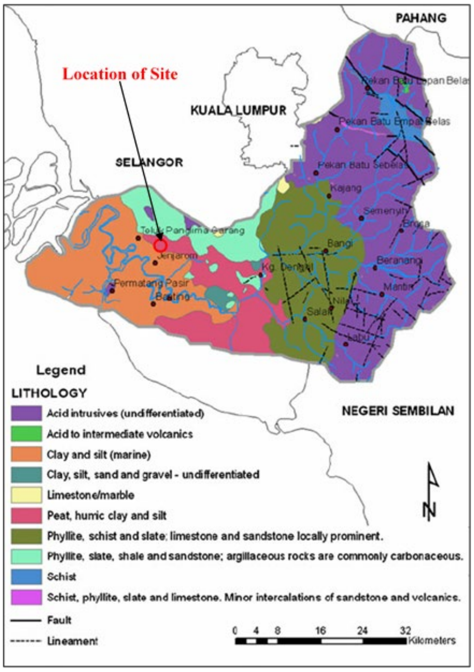

Figure 4. Geological formation of Selangor Darul Ehsan
(Department of Mineral and Geosciences, Malaysia)

It is observed that the proposed project site is located on the Gula formation (Quaternary deposit) based on the geological map of Peninsular Malaysia, 1985 and Quaternary geological map of Peninsular Malaysia, 1989. Towards the coast, quaternary marine deposits formed during the period of Holocene. This is likely to be areas with original ground reduced level generally less than 10 m.

2. Research Method

2.1 Soil Profile and Parameters

Based on the previous Site Investigation (SI) record, very soft organic CLAY was encountered from 0 m down to 19 m where the vane shear tests yielded undrained shear strengths from 9.71 kPa to 29.43 kPa. The correlated strength based on BS5930 (British Standard Institution, 2015) is extremely low to low strength and could cause massive settlement if not treated. Between 19 m down to 21 m, the layer consists of medium dense sand with SPT N value at 18 and 25 blows/300 mm. Then, between 22 m to 28 m very soft clay was encountered again with SPT N value at 1 to 4 blows/300 mm. Then between 28 m down to 53 m very thick medium dense to dense SAND was encountered with SPT N value ranging from 22 to 38 blows/300 mm. BH11 was terminated at 55.95 m and the water level recorded was at shallow depth between 1 – 1.5 m. Table 1 and Table 2 provide the soil parameters used with the reinforced earth (RE) wall and EPS properties adopted in PLAXIS Finite Element Code (FEC). Figure 5 shows the geometry of the abutment section simulated in PLAXIS FEC.

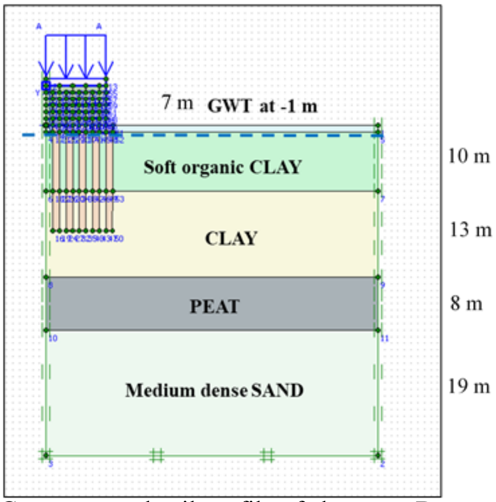

Figure 5. Geometry and soil profile of abutment B representing 2D plane strain geometry from Figure 1.

Table 1. Soil parameters used in PLAXIS FEC. Model

Name	Type	γ_{unsat} [kN/m^3]	γ_{sat} [kN/m^3]	k_x [m/day]	k_y [m/day]	E_{50}^{ref} [kN/m^2]	E_{oed}^{ref} [kN/m^2]	E_{ur}^{ref} [kN/m^2]
Sandy SILT	UnDrained	16.0	17.5	0.0100	0.0100	2500.0	2500.0	7500.0
Organic CLAY	UnDrained	14.0	17.0	1.0000E-4	1.0000E-4	1250.0	1377.0	3750.0
CLAY	UnDrained	16.0	18.5	1.0000E-5	1.0000E-5	4500.0	4331.6	13500.0
PEAT	UnDrained	14.0	19.0	1.0000E-3	1.0000E-3	6000.0	6000.0	18000.0
SAND	UnDrained	19.0	21.0	1.0000	1.0000	20000.0	20000.0	60000.0
Stone column	Drained	19.0	19.5	10.0000	10.0000	25500.0	25000.0	76500.0
Compacted Fill	Drained	18.5	21.0	0.1000	0.1000	25000.0	25000.0	75000.0

Name	Type	c_{ref} [kN/m^2]	φ [°]	ψ [°]	ν_{ur} [-]	p_{ref} [kN/m^2]	Power [-]	K_0^{nc} [-]
Sandy SILT	UnDrained	0.0	25.0	0.0	0.20	100	0.500	0.577
Organic CLAY	UnDrained	0.0	15.0	0.0	0.15	100	1.000	0.589
CLAY	UnDrained	0.0	19.0	0.0	0.20	100	1.000	0.509
PEAT	UnDrained	0.0	10.0	0.0	0.20	100	0.500	0.826
SAND	UnDrained	0.0	35.0	7.0	0.20	100	0.500	0.426
Stone column	Drained	0.0	40.0	9.0	0.20	100	0.500	0.357
Compacted Fill	Drained	0.0	30.0	5.0	0.20	100	0.500	0.500

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Table 2. Structural material properties.

Name	Type	γ_{unsat} [kN/m^3]	γ_{sat} [kN/m^3]	k_x [m/day]	k_y [m/day]	ν [-]	E_{ref} [kN/m^2]
Bituminous Pavement	Drained	21.0	21.0	1.0000	1.0000	0.20	30000.0
EPS	Non-porous	0.8	0.8	0.0000	0.0000	0.30	15000.0

Name	Type	EA [kN/m]	EI [kNm^2/m]	w [kN/m^2]	ν [-]
RC Wall	Elastic	2.358E6	17685.0	24.0	0.30
Concrete Slab	Elastic	1.965E6	10250.0	24.0	0.30

Electrical Resistivity Imaging (ERI) was used to evaluate the ground settlement in local scale at bridge BR12 at KM31.00 East Bound and West Bound (Abutment B) at SKVE highway near Teluk Panglima Garang, Klang. ERI and existing borehole results were used to interpret the condition of the problematic subsurface profile due to its different stiffness. The heterogeneous of the subsurface material presented using integrated analysis of ERI and borehole data enabled ground settlement in this area to be evaluated. There were 3 lines of resistivity used namely Line 1 for East Bound (Slow Lane bound to Putrajaya), Line 2 for East Bound (Fast Lane bound to Putrajaya) and Line 3 for West Bound (Slow Lane bound to Teluk Panglima Garang) and each line consists of 5 points of JKR Probe except for Line 2 where only 3 points of JKR Probe were executed. A total of thirteen (13) points of JKR Probe were carried out at the proposed site. The positions of the probes are shown in the layout plan in Figure 6.

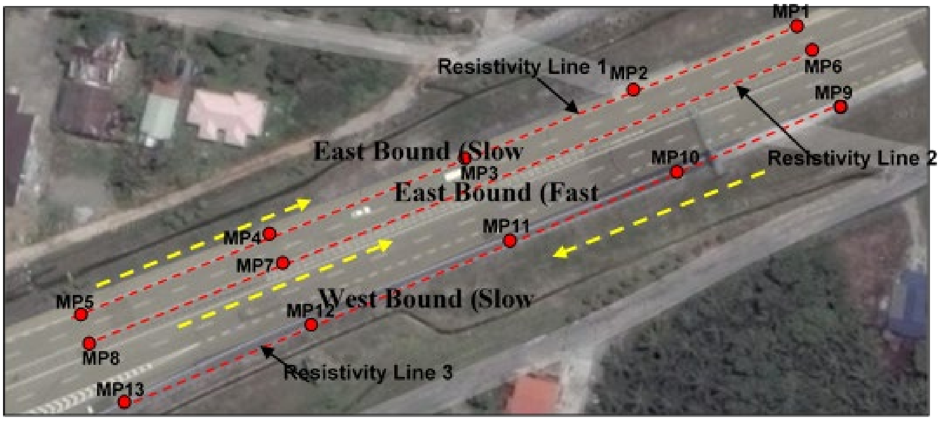

Figure 6. Layout of testing executed at the site location.

The purpose of the subsurface exploration is to determine the soil profile including the overburden soils and the thickness of the bedrock. Table 3 and Table 4 provide reference of undrained shear strength and relative density correlated with JKR or Mackintosh probe results based on Jabatan Kerja Raya (2016). A 2-D electrical imaging survey is usually carried out with a computer-controlled resistivity meter system connected to a multi-electrode cable. The control software automatically selects the appropriate four electrodes for each measurement to give a 2D coverage of the subsurface. To interpret the data, a 2D model referring to Figure 1 and Figure 2 is used where the resistivity changes in the vertical direction and one horizontal (x-) direction, but does not change in the other horizontal direction (y-direction).

Table 3. Correlation between MP values and related densities. (After Terzaghi and Peck, 1967, Jabatan Kerja Raya, 2016)

N (Blows/ft)	Relative density	Allowable soil pressure (kPa)	JKR or Mackintosh Probe (Blows/ft)
0 - 4	Very loose	Not suitable	0 - 10
4 - 10	Loose	0 - 80	10 - 30
10 - 30	Medium	80 - 280	30 - 80
30 - 50	Dense	280 - 470	80 - 110
Over 50	Very dense	470	110

Table 4. Correlation between MP values and soil strengths. (After Terzaghi and Peck, 1967, Jabatan Kerja Raya, 2016)

N (Blows/ft)	Consistency	Unconfined Compressive strength (kPa)	JKR or Mackintosh Probe (Blow/ft)
2	Very soft	0.0 – 25	0 – 10
2 – 4	Soft	25 – 50	10 – 20
4 – 8	Medium (firm)	50 – 100	20 – 40
8 – 15	Stiff	100 – 200	40 – 70
15 – 30	Very Stiff	200 – 400	70- 100
Over 30	Hard	400	100

The resistivity lines were set out along the proposed alignment. The topographic profile of each resistivity line has been considered as flat. The resistivity method basically measures the resistivity distribution of the subsurface materials. Table 5 and Table 6 show the resistivity and conductivity values of some of the typical earth materials (Lee *et. al.*, 2002) and representative values (McCarthy, 2001). Areas that contain high moisture content become saturated zone. The higher the saturation area, the lower is the resistivity value of the ground material.

Table 5. Typical Resistivity Values of Earth Materials. (Lee *et. al.*, 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

2.2 Geoforensic Method of Analysis

Finite element analysis was carried out to evaluate conditions of site for geotechnical forensics investigation. Back analysis for existing conditions were 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 were carried out using Two-Dimensional (2-D) finite element analysis (FEA) with PLAXIS FEC. The sophisticated FEA is necessary because of the complexity of soil-structure interaction problem, involving subsoil-slab-fill

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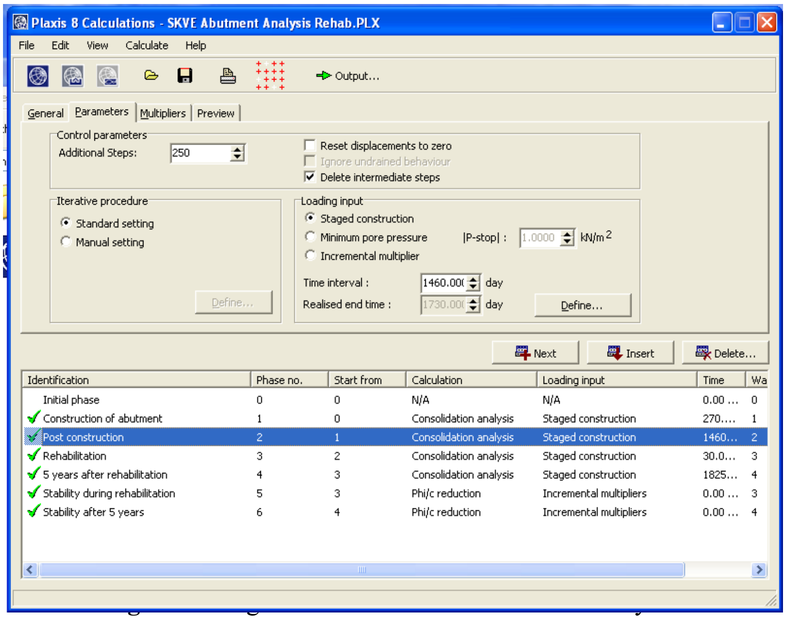
and vice versa.

Table 6. Representative Resistivity Values. (McCarthy, 2001)

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 +

2.3 Stages of Construction in Finite Element Analysis (FEA)

In order to investigate the settlement problem, the stages of construction for the FEA analyses are shown in Figure 7. It should be noted that for moderately conservative condition, all the parameters used in the FEA model are in drained condition. The results of analyses from Stages 1 and 2 would represent the actual condition before the rehabilitation works (assuming five (5) years post-construction period). Stages 3 and 4 represent the condition immediately after the rehabilitation works and condition in the long term, respectively.



Stage 0: Initial condition of the subsoil (before construction)

Initial condition of the subsoil refers to the condition where there is no construction carried out. It is based solely on the subsoil strata of the original soil. The subsoil parameters and the abutment geometry based on the SI carried out after the defects of the structures will be analyzed as shown in Figure 8.

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Stage 1: Construction of the abutment

The construction of the abutment is simulated according to the history of the construction. The consolidation analysis was performed with the duration of 270 days since there is the history of six (6) months consolidation period during construction (3 months of construction period).

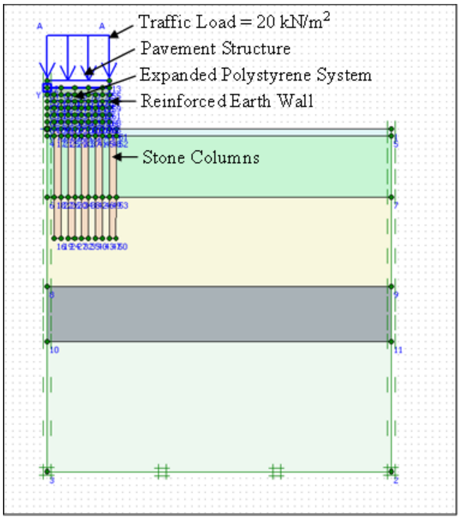

Figure 8. Stages of construction for FEA 2D analysis.

Stage 2: Post Construction Deformation Analysis

In this stage, the period of post construction is simulated according to the service life that the structure has performed before the geoforensic investigation. The consolidation analysis was selected for the period of four (4) years, according to the working life estimated after the construction in 2014. The traffic load during operation is estimated at 20kN/m^2.

Stage 3: Proposed rehabilitation using polyurethane (PU) foam/resin

Polyurethane (PU) foam/resin injection introduced by Mohamed Jais *et al.* (2015) is to plug and fill the voids, hence reducing the settlement of the structure to a tolerable settlement value as stipulated by the Malaysian Highway Authority (MHA). The maximum settlement allowed by MHA is 400mm/seven (7) years, which is equivalent to 5 mm per month (Jabatan Kerja Raya, 1997). The consolidation analysis was selected with the rehabilitation duration of one (1) month. The polyurethane foam/resin properties established by Mohamed Jais (2017) are given in Table 7.

Stage 4: Five (5) years after rehabilitation

In the design of the proposed rehabilitation method, estimation of settlement after five (5) years is essential to predict the value of settlement and whether it has achieved to the specification in accordance to MHA. The consolidation analysis was performed for the duration of five (5) years in order to evaluate the performance of the PU injection system.

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Stage 5: Stability analysis post construction

Shear strength reduction was performed to evaluate the stability of the PU injection system after rehabilitation using effective shear strength parameters of the soil. The proposed Factor of Safety (FOS) is as stipulated by GCO (1991) given in Table 8.

Stage 6: Stability analysis prediction five (5) years after rehabilitation

The stability performance after five (5) years was simulated to project the increase FOS. Shear strength reduction calculation was selected with the FOS tolerance of 0.01. The rehabilitation works will be expected to eliminate or reduce the excessive settlement and angular distortion.

Table 7. Design properties of the polyurethane foam/resin (Mohamed Jais, 2017)

Description	**Value**	**Unit**
Unit weight of the polyurethane foam/resin, γ	3	kN/m^3
Stiffness modulus, E	15,000	kN/m^2
Poisson's ratio, ν	0.3	
Compressive strength, σ	2.2	MPa
Permeability, k	1×10^{-12}	m/s

Table 8. Recommended FOS for new slopes of ten-year return of rainfall. (GCO, 1991)

Risk of life / **Economic**		**Recommended FOS against loss of life for a ten-year return of rainfall**		
		Negligible	**Low**	**High**
Recommended FOS against economic loss for a ten-year return of rainfall	Negligible	> 1.0	1.2	1.4
	Low	1.2	1.2	1.4
	High	1.4	1.4	1.4

2.4 Settlement Monitoring

Deformation survey or deformation monitoring is the systematic measurement and tracking of the alteration in the shape or dimension of an object as a result of stresses induced by applied loads. Deformation survey is a major component of logging measurement values that may be used for further computation, deformation analysis, predictive maintenance and alarm. Deformation survey is performed to detect and measure movements of the structure where it can detect relative or absolute movements toward the structure. Deformation survey is used to (Okiemute et al., 2018):

1. ascertain if movement have taken place;
2. to assess whether a structure is stable and safe;
3. analyzed the movement factors;
4. and its use to predict the future behavior of a structure.

Deformation survey requires accuracy to millimetres which depends on many factors which include from the techniques of deformation, type of measuring devices, the type of monitoring structures and requirements of adjustment use in detecting structure movements. Figure 9 illustrates the close-up location of the settlement marker for settlement monitoring.

3. Results and Discussion

Results of the JKR Probe were stipulated in JKR Probe MP1 – MP13 for Lines 1, 2 and 3. Figure 10 illustrates the ground profile. Based on the results of the JKR probes, for MP1 to MP5 (Line 1), the first 0.3 m, the soil is considered to be dense with the no. of blows ranging between 68 to 220 blows/300 mm especially for MP2, MP4 and MP5. However, as it reaches only 0.6 m the soil becomes very dense with blows exceeded 400/300 mm for MP4. For MP1 and MP5, the soil is likely to be in the medium dense state where at depth of 2.8 and 1.5 m the blows are at 200/300 mm. This indicates that the platform was compacted to the specified standard whereby the material is very dense SAND and there is no occurrence of water table below the fill profile. The fill material encountered is above the piled embankment and concrete slab counter weight.

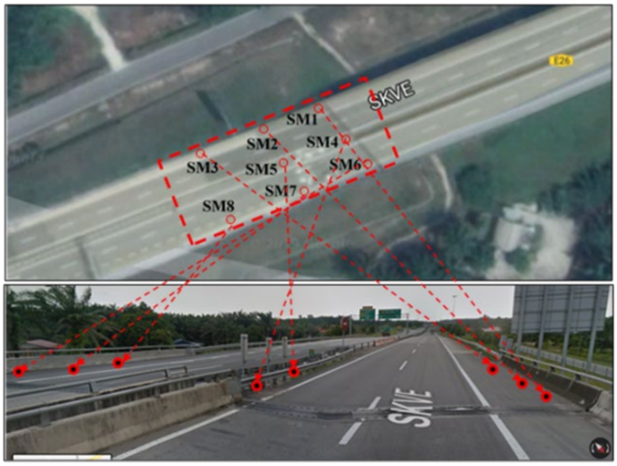
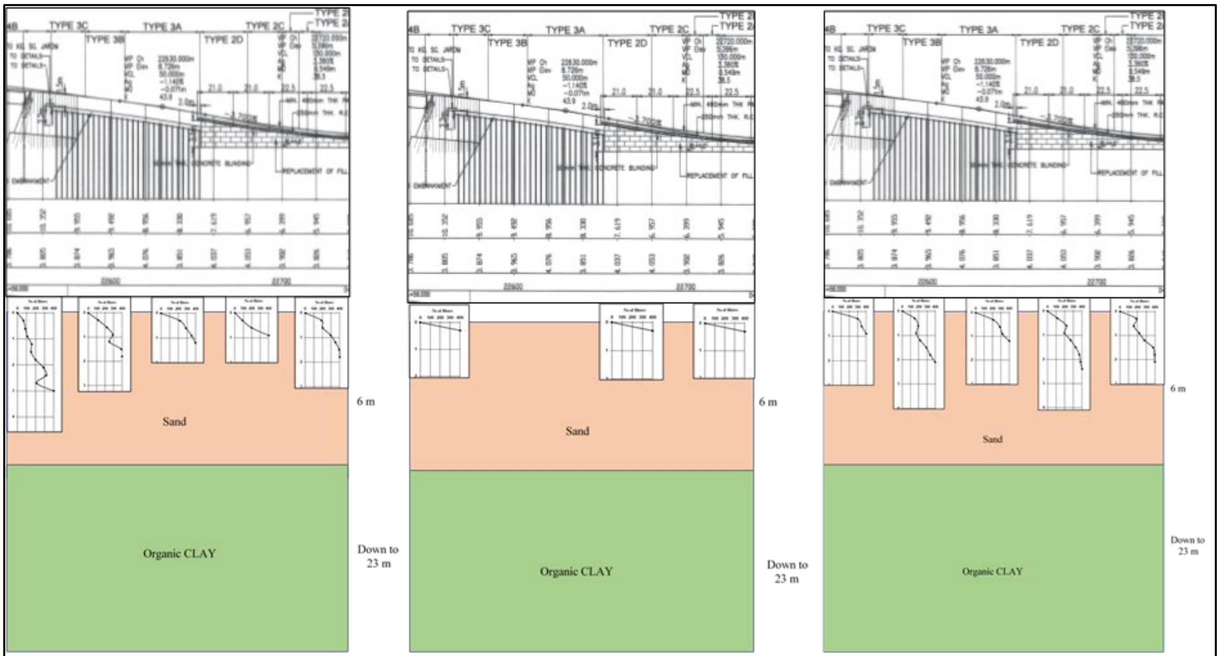


Figure 9. Location of the settlement marker for settlement monitoring.

The JKR Probe data for Line 2 located at the East Bound Fast Lane to Putrajaya. About three (3) points were recorded and the results found that overall, of the subsurface layer was very dense. The penetration was only at a depth of 300 mm where the blows exceeding 400/300 mm.

Based on the JKR probe results for MP9 to MP13, the soil is considered to be dense with the no. of blows ranging between 117 to 395 blows/300 mm. However, for MP12 and MP13, the reading suddenly dropped at about 1.2 m and then increased again until it reaches 400/300 mm blows at a depth of 2.1 m. There is a slight weakening to this layer which might due to water seeping from the surface into the subgrade layer.



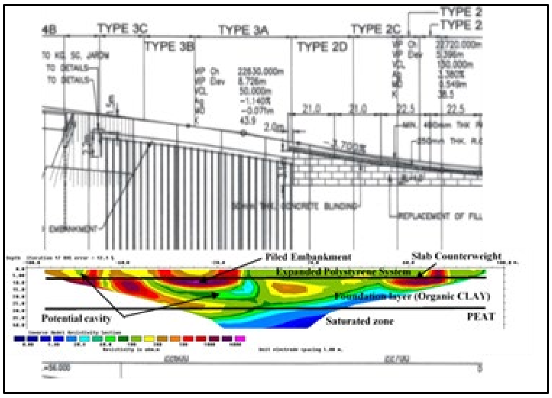
Subsoil profile for Line 1 at MP1 to MP5

Subsoil Profile for Line 2 at MP6 to MP8

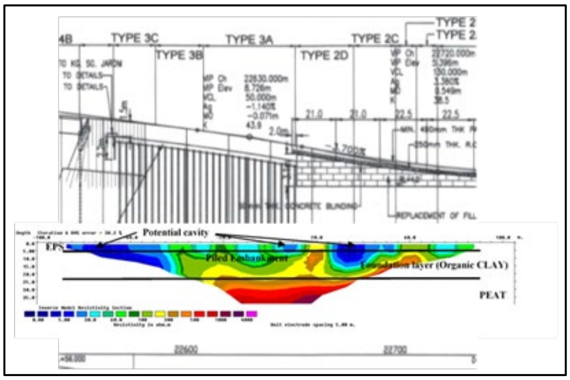
Subsoil Profile for Line 3 at MP9 to MP13

Figure 10. JKR Probe readings for 13 JKR probe locations.

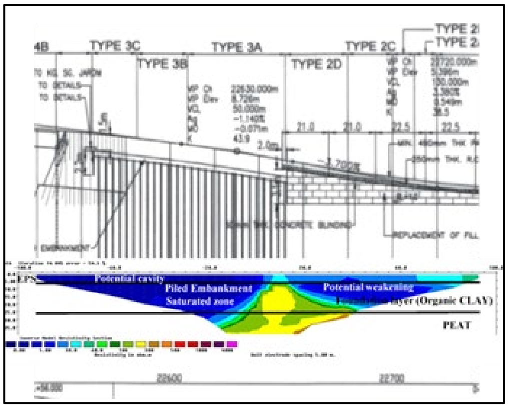
The problematic subsurface profile due to ground settlement was investigated using electrical resistivity imaging (ERI). The geometry and electrical resistivity anomaly distribution has been determined by analysing ERI data obtained along the settlement zones and the results show good correlation with the JKR probe data. ERI successfully mapped the ground instability which is able to extend the surface information observed during the physical mapping. The information from the ERI provides suggestions for decision making with regards to the location of the weak layer underneath the pavement structure. The application of ERI in conjunction with the JKR probe points and geological information was effectively being applied to the evaluation of the ground settlement due to its ability to detect the weak layer within the subsurface profile. Figure 11 provides the ERI for the lines as indicated previously.



(a) Resistivity Image for Line 1.



(b) Resistivity Image for Line 2



(c) Resistivity Image for Line 3

Figure 11. Electrical resistivity image generated for Lines (a) 1, (b) 2 and (c) 3.

According to the image given in Figure 11, at the section closed to the bridge abutment, there is a potential cavity and since EPS was installed behind the abutment with stone column supporting the EPS, the settlement of the EPS still occurred due to the segregation of the stone columns, creating voids underneath the counterweight slab. Eight (8) m away from the EPS section, another method of ground treatment using piled embankment was installed and the ERI managed to capture the density change. However, below the piled embankment, there is potential weakening and cavity caused by the settlement of the soil, which led to the over hanged section of the piled embankment. Since below six (6) m is organic CLAY, the resistivity value reduced due to saturated conditions below the ground water level. Therefore, it is advised that the section where the cavities are present, rehabilitation works need to be executed immediately to ensure that the approach does not settle excessively. It is suggested that if any rehabilitation work is to be conducted, the voids must be plugged to avoid migration of the soil underneath the piled embankment.

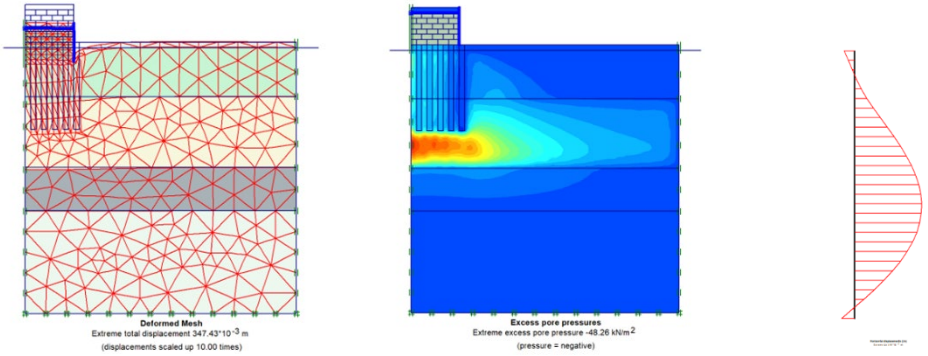
As a summary, the ERI shown critical section of the settlement occurrence, which is at the EPS section 8 m length from the bridge abutment to the piled embankment. At this section ground treatment using stone columns seems experienced settlement. Beneath the piled embankment, cavities start to appear causing the fill material at the side of the slope migrated into the cavities, creating voids on the verge and at the line of the guard rail for about 80 m. However, at the transition section between piled embankment and EPS with counterweight, settlement is slowly occurring causing discomfort during driving.

3.1 Geoforensic Simulation

The simulation is based on the data obtained from all aspects, which are previous site investigation report, settlement monitoring data, ERI and JKR probe interpretation. The geometry and soil parameters used are based on the correlation as mentioned previously. In this simulation, the construction stages given provide the basis for modelling deformation of the abutment section, four (4) years after construction and the settlement for the proposed rehabilitation works using polyurethane foam/resin injection.

3.2 Construction of the Abutment

Figure 12 shows the deformation 270 days after construction of the bridge abutment section using EPS and counterweight slab system. The simulated settlement is 347.43 mm which is more than 250 mm as stipulated by PWD with the effective stress increase of 308.49 kPa. The lateral movement of the wall is 4.58 mm which is less than 1/800 per m height as stipulated by PWD which is 8.75 mm and the built up of excess pore water pressure is 48.26 kPa. The built up of excess pore water pressure during construction will induce settlement as the excess pore water pressure dissipates, which occurs underneath the stone columns.



(a) Deformed mesh (b) Excess pore pressure

Figure 12. Deformation of the ground immediately after construction (a) Deformed mesh (b) Excess pore pressure

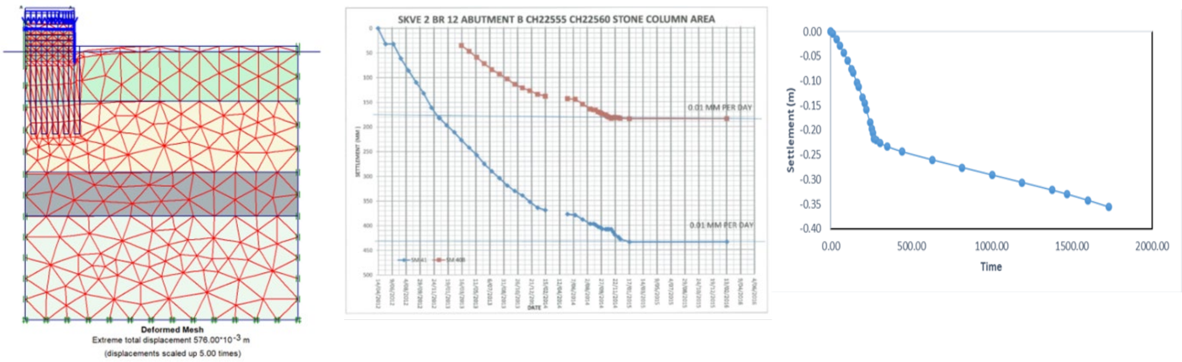
3.3 Post Construction Deformation Analysis

Figure 13 shows the deformation of the existing bridge abutment four (4) years of service. The maximum settlement observed during its service life was 576 mm which is more than the tolerable settlement of 400 mm in seven (7) years as stipulated by MHA. This showed that the stone column segregated causing the EPS system and its counterweight to settled excessively. In fact, the settlement trend showed good agreements with the monitored settlement data shown in Figure 13.

3.4 Proposed Rehabilitation using PU Foam/Resin

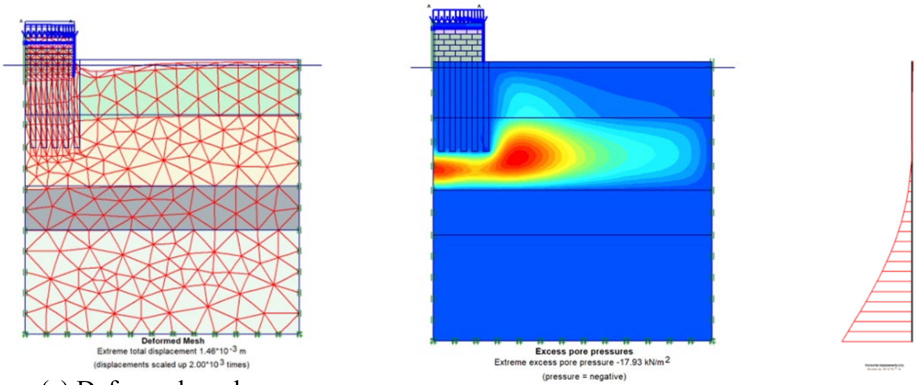
From the settlement analysis conducted using PLAXIS FEC, PU foam injection method was proposed to stabilized and plug the voids in the soil so that the fill material does not migrate in to cavity area. From the analysis, the settlement was estimated about 1.46 mm immediately after construction, which is less than 5 mm estimated per month according to MHA as shown in Figure 14. This is fascinating, since the PU foam/resin is light weight, thus it does not increase the overburden pressure to the existing foundation soil. The lateral movement of the wall is estimated at 0.09 mm inward and the excess pore water pressure built up immediately after the rehabilitation work is 17.93 kPa. Therefore, during the rehabilitation there is no excess pore water pressure increase in the soil, since the injection material is lightweight and does not add loading to the existing condition.

A five-year projection of settlement was predicted to obtain the settlement tolerance according to MHA requirements. In this case, MHA requirement is 400 mm per seven (7) years, which is equivalent to 5 mm/month. Based on the prediction for five (5) years after rehabilitation, the settlement was estimated at 25 mm, which is 0.4 mm/month as shown in Figure 15. The wall moved laterally at a value of 0.6 mm at the toe after five (5) years, which is less than the tolerable rotation value specified by PWD. In fact, after five years of the rehabilitation work, the excess pore water pressure was reduced to 9.58 kPa, gradually decreasing to ensure that minimal disturbance would not affect the performance of the bridge abutment. Table 9 shows the summary of the results for the analysis executed.



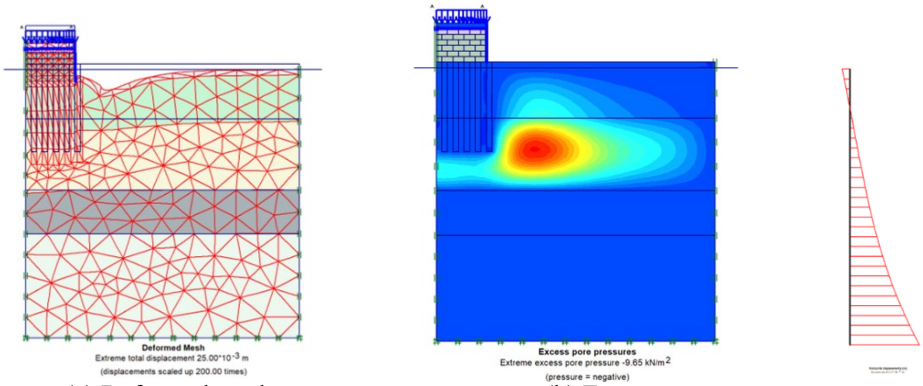
(a) Deformation of the ground (b) Settlement monitoring

Figure 13. (a) Deformation of the ground four (4) years of service life (b) Settlement monitoring data against the simulation modelling data.



(a) Deformed mesh (b) Excess pore pressure

Figure 14. Deformation of the ground immediately after rehabilitation using PU foam/resin injection (a) Deformed mesh (b) Excess pore pressure.



(a) Deformed mesh (b) Excess pore pressure

Figure 15. Deformation of the ground five (5) years after rehabilitation using polyurethane foam/resin injection (a) Deformed mesh (b) Excess pore pressure.

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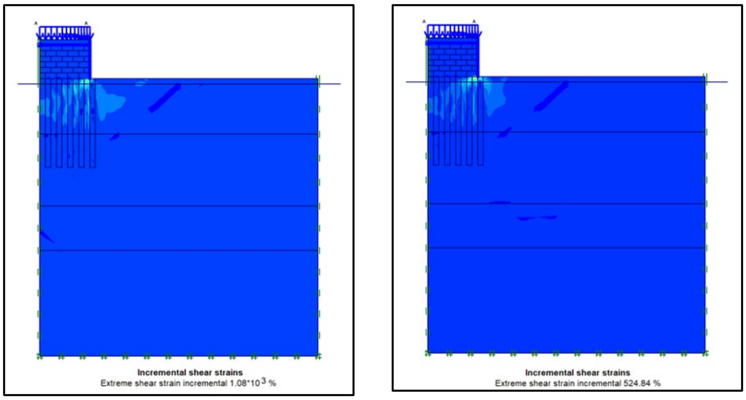
Table 9. Summary of deformation values for settlement, lateral movement and generation of excess pore water pressures for various stages.

Stage	Settlement (mm)	Lateral wall movement (mm)	Excess Pore Water Pressure (kPa)
Construction	347.43	4.58	48.26
Post construction	576.00	8.70	18.18
Rehabilitation	1.46	0.09	17.93
Post Rehabilitation	25.00	0.60	9.58

3.5 Stability After Rehabilitation

Figure 16 shows the global stability of the wall immediately after rehabilitation and five (5) years after rehabilitation. The global factor of safety (FOS) is 1.57, which is more than the stipulated value by PWD of 1.5. For the projection of five (5) years, the FOS increased to 1.84 illustrated. These show that after the rehabilitation work using polyurethane foam/resin injection, the stability of the existing system has increased.



(a) immediately after rehabilitation (b) five (5) years after rehabilitation.

Figure 16. Predicted plane underneath the wall (a) immediately after rehabilitation (b) five (5) years after rehabilitation with FOS of 1.57 and 1.84, respectively.

3.6 Settlement Monitoring Post Geotechnical Rehabilitation

Based on the monitoring data recorded for 15 months, the settlements at all locations are still within the tolerance of 5 mm/month at a total of 75 mm/15 months. Therefore, for the recorded settlements, the percentage of settlement at the rehabilitation location in accordance to the specification. Therefore, the injection procedure has reduced the effect since the PU foam provides a barrier between the voids and the abutment structure. Figure 17 shows the settlement markers values recorded within the period of 15 months. Table 10 provides the settlement monitoring results after 15 months of observation.

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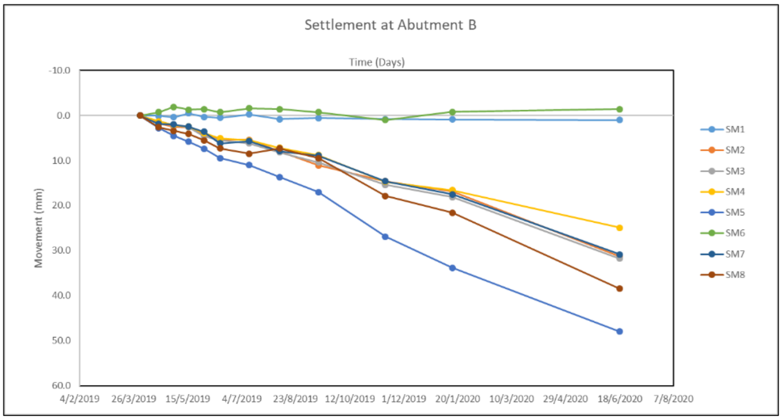

Figure 17. Recorded settlement from the markers during the 15 months' period.

Table 10. Settlement monitoring results after 15 months of observation.

Trip	**Date**	**SM1**	**SM2**	**SM3**	**SM4**	**SM5**	**SM6**	**SM7**	**SM8**	**Remarks**
	Distance	14.0952	13.9106	13.7632	14.3082	14.1243	14.6257	14.3842	14.2324	Reading
		0.0000	8.0000	15.0000	6.2000	15.2000	0.0000	7.0000	14.0000	
1	1/4/2019	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	Baseline
2	18/42019	0.1	1.8	1.3	1.3	2.8	- 0.7	1.9	2.6	1st Reading
3	2/5/2019	0.4	2.5	2.2	2.1	4.5	- 1.9	2.0	3.4	2nd Reading
4	16/5/2019	- 0.5	2.6	2.6	2.5	5.8	- 1.3	2.5	4.1	3rd Reading
5	30/5/2019	0.3	4.3	4.6	3.9	7.4	- 1.4	3.6	5.5	4th Reading
6	13/6/2019	0.5	5.4	5.5	5.1	9.5	- 0.7	6.2	7.3	5th Reading
7	11/7/2019	-0.2	5.4	6.2	5.5	11.0	-1.6	5.7	8.5	6th Reading
8	8/8/2019	0.8	7.9	8.2	7.2	13.7	-1.4	7.9	7.3	7th Reading
9	13/9/2019	0.6	11.1	10.4	8.8	17.0	-0.7	8.9	9.5	8th Reading
10	14/11/2019	0.8	14.6	15.4	14.7	26.9	1.0	14.6	17.9	9th Reading
11	15/1/2020	0.9	16.8	18.1	16.6	33.8	-0.8	17.5	21.6	10th Reading
12	18/6/2020	1.0	31.3	31.8	24.9	48.0	-1.4	30.8	38.5	11th Reading

4. Conclusion

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

i. As a summary, the settlement occurred due to the presence of voids and cavities beneath the piled embankment and the segregation of the stone columns. The EPS and its counterweight slab should not settle since it is lightweight and reduces the overburden pressure from the fill material. However, from the geoforensic investigation conducted, it is observed that the main problem is the failure of the stone columns and settlement of the existing foundation soil that created cavities beneath the rigid structure and EPS system. 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.

ii. For stabilization of the weak material at the abutment, the PU foam injection re-compacts and stabilize the material. In addition, the impermeable characteristic of the PU foam/resin diverts water from flowing underneath the ground surface, reducing the seepage. For cavity filling at the piled embankment, PU foam injection will plug the void area and prevent migration of the fill material into the cavities created from the settlement of the ground. Since PU foam/resin is lightweight, it does not impose additional overburden pressure to the soil.
iii. Based on the monitoring data recorded, the settlements at all locations are still within the tolerance of 5 mm/month at a total of 75 mm/15 months. Therefore, for the recorded settlements, the percentage of settlement at the rehabilitation location in accordance with the specification.

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Author Contribution

Ismacahyadi Bagus Mohamed Jais contributed to conceptualization, software, methodology, investigation, formal analysis and writing original draft.
Abdul Samad Abdul Rahman contributed to writing review and editing, investigation, data curation and methodology.
Rafiee Abdul Razak contributed to resources, project administration and funding acquisition.
Saravanan Selvarajah contributed to resources, project administration and funding acquisition.
Ibrahim Husaini contributed to resources, project administration and funding acquisition.
Mohamed Azizi Md Ali contributed to conceptualization, investigation, resources, project administration and funding acquisition.
All authors read and approved the final manuscript.

Declaration of Conflicting Interests

All authors declare that they have no conflicts of interest.

Declaration of Generative Ai in The Writing Process

During the preparation of this manuscript, the author(s) used generative artificial intelligence (AI) tools to assist with language editing and improving the clarity of the text. The author(s) reviewed and edited the content as necessary and take full responsibility for the content of the publication.

Data Availability/Supplementary Materials

The data supporting the findings of this study are available within the article. Additional supporting information may be provided by the corresponding author upon reasonable request.

Ethics Statement

This study did not involve human participants, animals, or any ethical issues requiring institutional review board approval.

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