DEVELOPMENT OF 3D NUMERICAL ANALYSIS FOR OPTIMIZATION OF GROUND IMPROVEMENT VOLUME IN MARINE CLAYS WITH APPLICATION IN TUNNELING

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The Thomson-East Coast Line (TEL) is a joint line between the Thomson Line and the Eastern Region Line (ERL). The ground conditions in the section along the route comprise of thick deposits of marine clays sandwiched between other layers of Kallang Formation (fluvial sands and clays) overlying the Old Alluvium deposit. The bored tunnels of diameter 6.35m will be constructed in Kallang Formation (marine clays, fluvial sands and clays) using mechanical TBMs with precast concrete segmental lining.
To minimize the impact along longitudinal axis of tunnels, provision of ground improvement (GI) by grouting is proposed. This paper discusses the methodology to define the design parameters of GI zone and optimization of its shape with the help of numerical analysis. With the optimization of ground improvement zone significant savings in terms of materials, time and cost can be achieved whilst maintaining the long-term deformation under CPRP operational limits.

INTRODUCTION

Singapore's Mass Rapid Transport (MRT) has seen an unprecedented growth in the past two decades. To handle increased ridership and cut down the travel time, new MRT lines have been proposed. Thomson-East Coast Line (TEL) is one such line which will provide connectivity between the northern regions of Singapore to the south. Upon completion, TEL will have running track length of 43km and 31 stations, all built underground. In the eastern regional segment of TEL, the tunnels are to be constructed through soft soil conditions. The tunnels will be constructed using Cut-and-Cover method and mechanized TBMs where the proposed alignment passes underneath urban infrastructure. To safeguard the roads from a future rise in sea level, the grade level of existing roads, some of which lie above the proposed alignment, will be increased.

Under the Code of Practice for Railways (CPRP), any construction adjacent to existing MRT tunnels should not cause tunnels to settle more than 15mm and should maintain longitudinal gradient of 1:1000. This paper looks into settlement of bored tunnels in soft soil due to increase in level of existing roads, longitudinal bending behavior of segmental lining and optimization of ground improvement zone geometry whilst conforming to CPRP requirements.

GEOLOGY

The geology of Singapore can be classified into four main categories:

a) Bukit Timah Granite and Gombak Norite in the north and central-north of mainly igneous origin;
b) Jurong Formation in the west and south-west of sedimentary origin;
c) Old Alluvium in the east of quaternary origin;
d) Kallang Formation in the east of marine, alluvial, littoral and estuarine origin.

Generally, the rapid transport tunnels are constructed at shallow depths such as alluvial deposits, soft clay and weathered deposits to lessen the construction costs of underground station and passenger access facilities such as escalators, lifts etc.[1]. Since the proposed alignment passes majorly through Kallang formation, only its geotechnical characteristics have been discussed here.

Kallang Formation

These deposits are of marine, alluvial, littoral and estuarine origin and are thought to have been laid down in the last 15,000 years. The deposits comprise a mixture of sands, silts and clays and most are of a soft consistency or loose density. Some notable projects have been built in the marine clays such as Marina Coastal Expressway, reclamation projects for Changi Airport, container port at Pasir Panjang, construction of Northeast MRT, Circle Line and as such, mechanical behavior of marine clays has been carefully understood by researchers as noted by Sharma et al. (1999) [e.g., Tan and Lee (197; Tan (1983); Chu and Choa (1997)]. The following sub-sections look into the engineering properties of marine clays as it is the most important member of Kallang Formation.

Index Properties

Upper Marine clay (UMC) shows mean moisture content of 59.2% and Lower Marine clay (LMC) shows mean moisture content of 49.7%. The UMC is slightly less dense and more variable than the LMC. The mean bulk unit weight of UMC is ~16.3kN/m^3 and for LMC is ~17kN/m^3. Atterberg Limits Plotted on a Casagrande A line plot the majority of the results from the UMC are spread between the high and very high plasticity clay zones. The majority of the LMC results sit within the high plasticity zone. For the UMC, a mean PL of 29% and mean LL of 67 and for the LMC, a mean PL of 28% and mean LL of 63% was observed.

Shear Strength

Undrained shear strength properties for UMC and LMC are derived from the unconsolidated undrained (UU) triaxial tests and in situ vane shear tests. The results of the UU triaxial testing on UMC samples ranged from cu = 10kPa to 76kPa and on LMC material raged from cu=10kPa to 180kPa. Undrained shear strength is also obtained using shear vane testing. Based on the plasticity index of 35-40%, Bjerrum's correction factor of ~0.9 was applied to the shear strength obtained from test results. After correction, mean peak shear stress obtained for UMC is 29.25kPa and 47.25kPa for LMC. The plot of Mohr Coulomb vs. Shear Test obtained from Consolidated Undrained (CU) triaxial shear tests gave effective strength parameters as c'=0kPa and ϕ'= 22°.

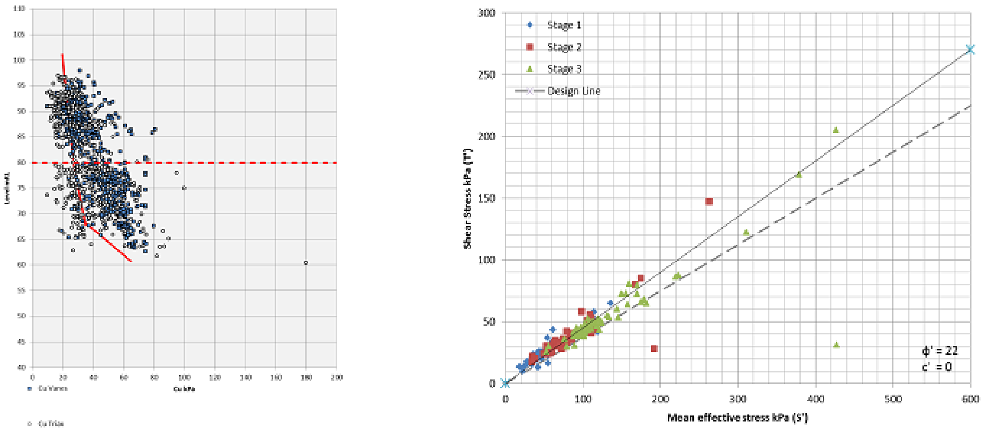
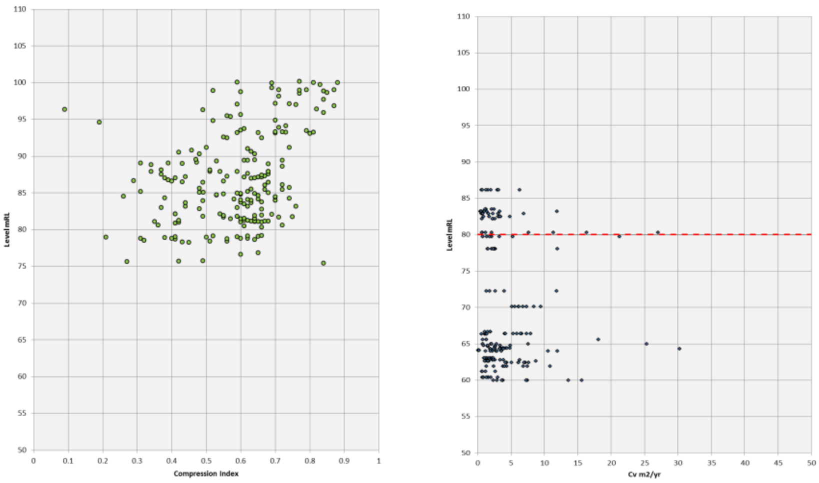


Figure 1 (a) Undrained Shear Strength from UU and VS Tests; (b) Effective Strength from CU Tests

Consolidation Parameters

Consolidation parameters were derived from oedometer tests on the UMC and LMC material. Figure 2 show plots of test results as obtained from different boreholes. Table 1 summarises consolidation parameters for UMC and LMC.



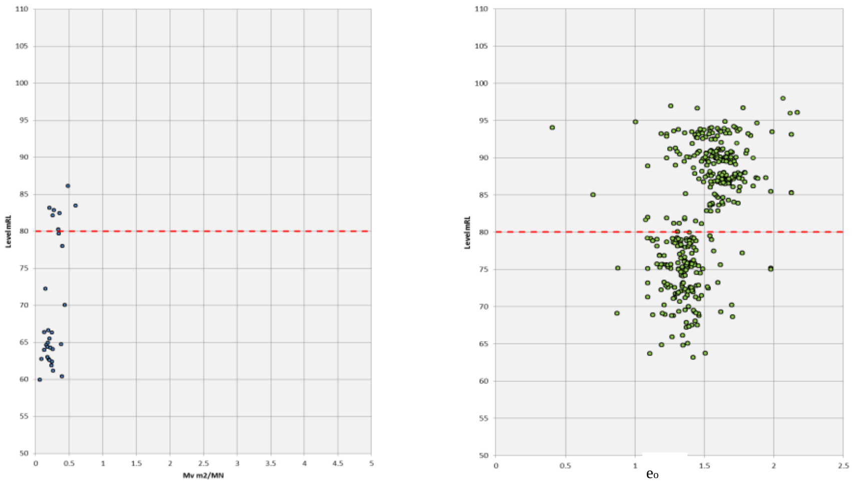


Figure 2 Consolidation Parameters as obtained from Oedometer Tests (a) Compression Index; (b) Coefficient of Consolidation; (c) Coefficient of Volume Compressibility; (d) Initial Void Ratio

Table 1 Range of consolidation parameters for UMC and LMC

Consolidation Parameter	Upper Marine Clay	Lower Marine Clay
Cc	0.09-1.1	0.21-0.84
Cv	0.45-30.48	0.003-47.5
m_v	0.03-1.53	0.03-0.78
e_0	0.41-2.17	0.88-1.98

Permeability

Permeability values for UMC and LMC are obtained from in-situ tests such as standard falling/rising head methods. The permeability of UMC ranges from 5.5×10^{-9}m/s to 6.0×10^{-8}m/s, and of LMC ranges from 1.3×10^{-9}m/s to 1.4×10^{-8}m/s. This method has difficulty in testing permeability of low permeable materials. As such, reference has also been made to past projects where permeability of UMC and LMC was taken as 1×10^{-9} m/s.

NUMERICAL ANALYSIS

For this analysis, authors have chosen to use PLAXIS 3D to simulate the longitudinal bending of tunnel lining due to settlements in soft clays. A number of different station locations were analyzed to predict the effects of tunnel alignment (Stacked and Parallel), geology and applied surcharge on settlements.

Finite Element Model

The tunnel alignment in this section is Stacked with the invert of upper tunnel lying completely within UMC. A simplified ground model is developed based on the available borehole data. Since, the loading conditions are symmetric along the tunnel axis only a half symmetric model is developed to save computational and results processing time.

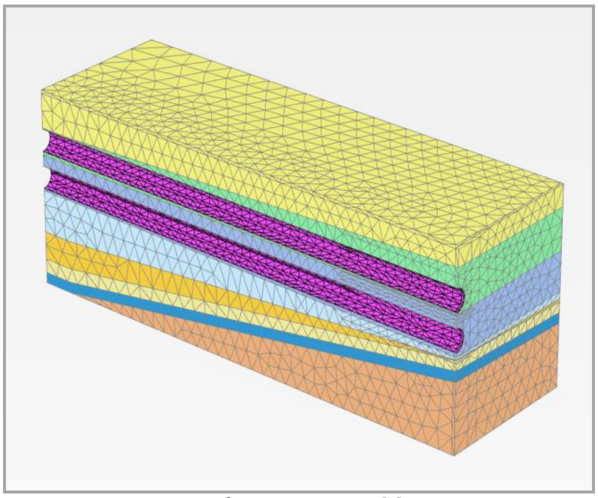


Figure 3 Isometric view of PLAXIS 3D Model

The 160.0m long, 50.0m wide and has approximately 48,000 elements. The soil clusters are meshed with standard 10-node tetrahedral elements. The surcharge at the site varies from 12kPa-30kPa. The groundwater table is considered at the surface. The geotechnical design parameters are given in Table 2.

Table 2 Geotechnical parameters for PLAXIS Analysis

Material	Top (mRL) @ L=0.0	Top (mRL) @ L=160.0m	Mohr-Coulomb Parameters			Soft Soil Parameters			Permeability (m/s)
			E' (kPa)	c' (kPa)	phi' (deg)	Cc	Cr	e_o	
Fill	102.8	102.8	12E+3	0.0	30.0	-	-	-	2.0E-5
UMC	97.10	84.2	-	0.0	22.0	0.76	0.074	1.6	1.0E-9
Upper F2	84.8	84.2	-	0.0	24.0	0.36	0.069	1.0	1.0E-8
LMC	73.5	74.0	-	0.0	22.0	0.59	0.010	1.4	1.0E-9
Lower F2	72.7	62.0	-	0.0	24.0	0.51	0.10	1.2	1.0E-8
OA(D)	72.6	54.0	25E+3	0.0	32.0	-	-	-	1.0E-8
OA(C)	69.8	50.0	66.7E+3	5.0	35.0	-	-	-	1.0E-8
OA(B)	66.8	47.0	125E+3	5.0	35.0	-	-	-	1.0E-8

The lining is modeled with 5-node plate elements. The tunnels have an external diameter of 6.35m and lined with precast concrete panels of thickness 275mm. The invert level for the tunnels is at 85.31mRL and 74.71mRL. The segments are rotated in the longitudinal direction to prevent the joints from lining up along the tunnel axis. Modeling and analysis of jointed behavior of lining is quite complex for a model of this scale. A simplified approach to simulate lining stiffness accounting for joints has been adopted based on the works of Chen and Wen (2002). Based on input parameters given in Table 3, an equivalent bending stiffness of 12% of the original bending stiffness has been used for analyses.

Table 3 Input parameters for calculation of equivalent bending stiffness

Parameter	Value	Units
Tunnel Outer Diameter, Do	6350	mm
Tunnel Inner Diameter, Di	5800	mm
Length per Ring, ls	1400	mm
Young's Modulus of Concrete, Ec	17200	N/mm^2
Young's Modulus of Bolt, Es	200000	N/mm^2
Cross Sectional Area per Bolt, As	353	mm^2
Length per Bolt, l	350	mm
Yield Strength per Bolt, py	560	N/mm^2
No's of Bolt between ring to ring, n	16	-

Two finite element analysis cases were set up with a varying degree of uniform surcharge – 10kPa and 20kPa. Figure 4 shows long-term settlement and gradient after application of surcharge. It can be inferred that without any ground improvement, CPRP requirements would not be satisfied.

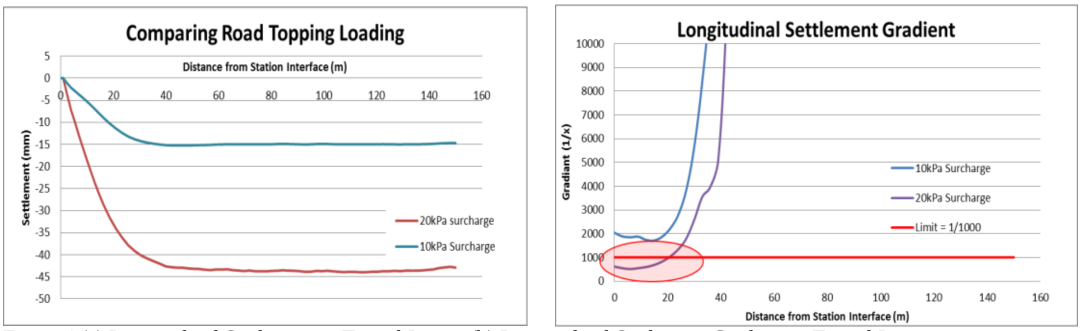


Figure 4 (a) Longitudinal Settlement at Tunnel Invert; (b) Longitudinal Settlement Gradient at Tunnel Invert

Mitigation Procedure

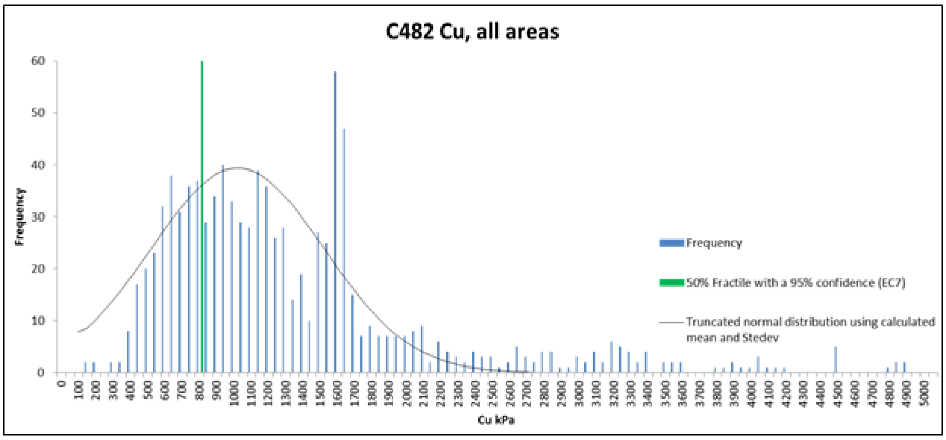
As a mitigation measure, it has been proposed to extend (or, modify) the shape of ground improvement block used for break-in/break-out. The grout block enhances the bending stiffness of the tunnel lining to provide an equivalent to a Transition Slab. This is a well-known[4] concept for dealing with the transition from natural ground to a hard point such as bridge abutment, building foundations etc. In order to take advantage of higher average properties achieved for ground improvement on Marina Coastal Expressway (MCE), Cu of 800 kPa and Eu of 280 MPa for grout have been adopted as design values.

In the following section, these properties will be considered within the context of EC7 and propose appropriate acceptance criteria. As a comparative study, characteristic values for the MCE dataset were determined and then compared with the design values. In geotechnical engineering, the characteristic value is a cautious estimate of the mean, i.e. 50% fractile (mean) at 95% confidence level. This follows that the acceptance criteria for MCE datasets correspond to minimum Cu of 400kPa and Eu of 140 MPa with 95% confidence. Table 4 shows the number of area and data points available for statistical analysis.

Table 4 List of MCE Data Points for Ground Improvement Area

Contract Area	No. of Areas	Data Points
C482 DCM	13 (A1-G)	890
C483 DCM	6 (Ai-Dii)	649
C486 DCM	4 (C1-E)	246
C486 JGP	9 (JP1-JP11)	99

For the sake of brevity, details of statistical analysis and output have been omitted from discussion. Figures 5-6 show the frequency diagram of Cu and Eu values for C482 data sets. The green vertical bar represents the 50% fractile with 95% confidence.



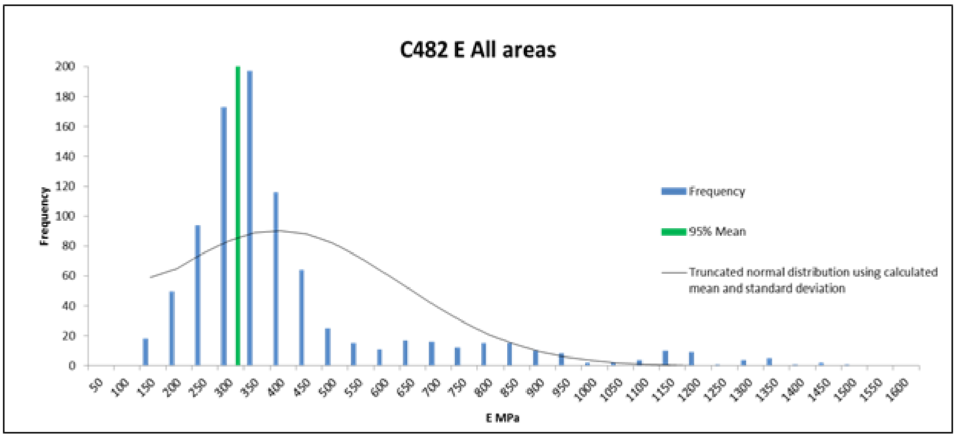


Figure 5-6 Frequency diagram of Cu and Eu values for all areas of C482

The exercise was repeated for the remaining contracts – C483 (DCM), C486 (DCM and JGP). Table 5 shows the output of this analysis in terms of mean to characteristic ratio. A value greater than 1 validates the chosen values of Eu and Cu for ground improvement.

Table 5 Mean to Characteristic ratio for MCE data sets and TEL

Contract	GI Type	c_u (kPa)			E_u (MPa)		
		Mean	Characteristic	Ratio $m_x/X_{k,inf}$	Mean	Characteristic	Ratio $m_x/X_{k,inf}$
C482	DCM	1185	1072	1.105	401	352	1.139
C483	DCM	2042	1863	1.096	362	320	1.131
C486	DCM	2114	1935	1.093	289	270	1.070
C486	JGP	2581	2315	1.115	327	305	1.072
TEL	(MCE mean)	(1981)	800	(2.476)	(345)	280	(1.231)

The adopted ground improvement properties are then used to simulate ground improvement (GI) zone around bored tunnels-station interface. The process of arriving at an optimum GI block dimensions involved several iterations, out of which the results from three extreme cases as shown in Figure 7 are briefly discussed. Case 1 represents the minimum provisional GI block dimensions for break-in/break-out; Case 2 represents a conservative GI block dimensions and Case 3 represents optimized dimensions based on Case 1 and Case2. For all the three cases, it has been assumed that the width of GI zone is 3m measured from the extrados of tunnel lining. The tunnels are assumed to be rigidly connected to the station D-Wall.

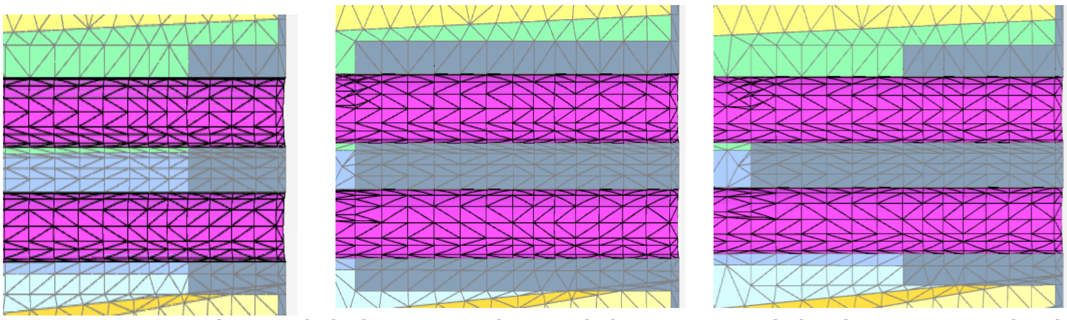


Figure 7(a) Case1: 12m long GI Block; (b) Case2: 40m long GI Block; (c) Case3: GI Block with 20m Outer Length and 40m Inner Length

Figure 8-9 show longitudinal settlement profile at tunnel invert level and Figure 10-11 show the corresponding gradient for all three cases. Case 1 has insufficient length to enhance the longitudinal stiffness of tunnel lining whereas Case 2 can be considered as ‘overdesign’. Due to high overburden, the lower tunnel is less likely to experience high magnitude of settlement hence it is possible to reduce the GI block length surrounding it. However, reduction of GI block length around the upper tunnel is sensitive to surcharge, overburden, thickness of marine clays and any relatively hard stratum such as F2 underneath it. It can be inferred from the plots that Case 3 (Optimized GI block) satisfies CPRP requirements

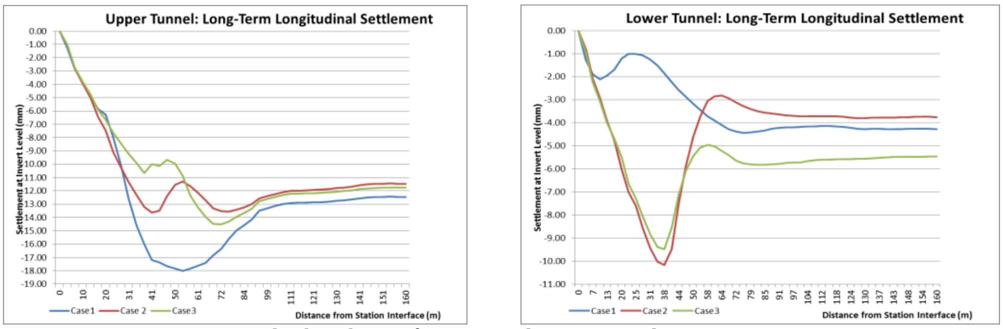


Figure 8-9 Long Term Longitudinal Settlement for Upper and Lower Tunnel

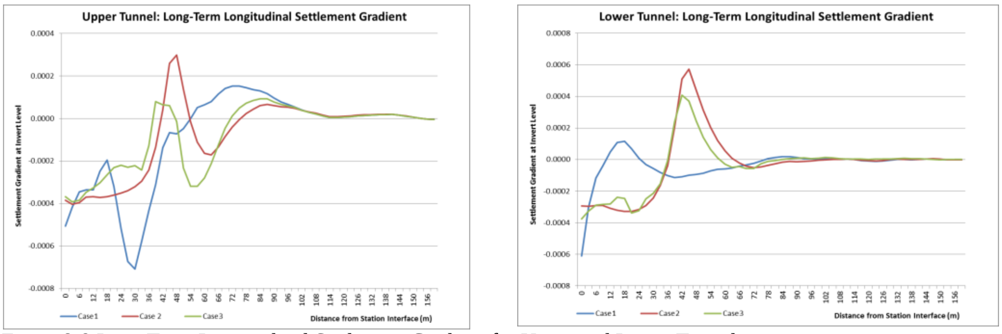


Figure 8-9 Long Term Longitudinal Settlement Gradient for Upper and Lower Tunnel

CONCLUSION

This paper presents a way to predict settlement of tunnel lining in consolidating soft clays. Built-in advanced constitutive models such as Soft Soil in PLAXIS 3D provide a reliable way of simulating the behaviour of marine clays. Using empirical equations, it is possible to calculate the longitudinal bending stiffness of tunnel lining accounting for the presence of ring-to-ring joints without the need to consider orientation of rings and joints in the longitudinal direction explicitly in a finite element model. Ground improvement by grouting has been proposed to reduce settlement caused by future road topping surcharge. The design values for grouted zone are chosen based on the past field experience and has been verified using the EC7 approach to determine characteristic values form existing MCE datasets. It is noted from the analyses that the criterion of maximum settlement of 15mm is much more stringent to meet as compared to maximum longitudinal gradient of 1:1000. Any numerical analysis must consider accurate geological profile and load profile to predict accurate settlement values. The role of finite element analysis is pivotal in such cases. Furthermore, the use of finite element analysis helped in carrying on an iterative design procedure to optimise the dimensions of GI zone thereby saving time, material and construction costs.

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