Planning/ Design → **Construction/ Implementation** → **Performance & Monitoring**

2013 A/E Contract

2015 Advance Work Contract T3008

2016 Main Civil Contract T301

2017

2018

2019

2020

2021

2022

2023

2024 Scheduled for completion

1

Geotechnical Design Aspects for an Underground Multilevel Train Depot
Underground Singapore 2016

W.T. Lai, **Q. Y. Ye**, C. Veeresh

2

Impact of Construction Issues on the Initial Geotechnical Design Intent for an Underground Multilevel Rail Depot
Underground Singapore 2018

W.T. Lai, **Q. Y. Ye**, C. Veeresh

3

Piling Design and Load Test Considerations in Underground Stacked Rail Depot
16th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, 2019, Taipei, Taiwan

W.T. Lai, **Q. Y. Ye**, C. Veeresh

4

Application of Observational Method in ERSS System
1st Malaysian Geotechnical Society and Geotechnical Society of Singapore Conference 2019, Petaling Jaya, Malaysia

W.T. Lai, **Q. Y. Ye**, B. Y. Lim, C. Veeresh

For Hulme Prize Submission

5

Performance of Deep Excavation Using Island Method – Temporary Berm and Buttress Wall Retaining System
20th International Conference on Soil Mechanics and Geotechnical Engineering, 2022, Sydney Australia

W.T. Lai, **Q. Y. Ye**, H. Qu, C. Veeresh

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Work in Progress

Use of Observational Method for Thomson East Coast Line Project
Underground Singapore 2023

W.T. Lai, **Q. Y. Ye**, C. Veeresh

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Performance of deep excavation using island method- temporary berms and buttress wall retaining system

Le comportement de l'excavation profonde à l'aide de la méthode de l'îlot - Bermes temporaires et système de retenue des murs de contrefort

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ABSTRACT: The East Coast Integrated Depot, a multi-billion major mass transit infrastructure project in Singapore is the world's first 4-in-1 depot housing three MRT lines and a bus depot on the same site. The depot measures over 1km in length and 360m in width and 15m deep. With various optioning study on the retaining scheme and consideration of site constraints for the vast depot area, "Island method" with the use of a temporary improved soil berms and arrays of buttress wall is selected to support the peripheral wall. Berm trimming/partially removed in conjunction with observational method (OM) have been implemented in this project to allow some part of the permanent support to be installed, this in turn enhance the productivity and time/cost saving. This paper presents the instrumentation monitoring performance from different stages of construction. Results from the back analysis and those from the initial predictions are compared to the instrumentation monitoring records. Advanced constitutive soil models and improved soil parameters are used in the prediction of soil behaviour. Lessons learnt are also included for future implementation in projects of similar scale and complexity.

RÉSUMÉ: L'East Coast Integrated Depot, un projet majeur d'infrastructure de transport en commun de plusieurs milliards de dollars à Singapour, est le premier dépôt 4 en 1 au monde abritant trois lignes MRT et un dépôt de bus sur le même site. Le dépôt mesure plus de 1 km de longueur et 360 m de largeur et 15 m de profondeur. Avec diverses études d'option sur le schéma de soutènement et la prise en compte des contraintes du site pour la vaste zone de dépôt, la «méthode de l'îlot» avec l'utilisation d'un sol amélioré provisoire de talus et de réseaux de contreforts est choisie pour soutenir le mur périphérique. L'enlèvement partiel des bermes en conjonction avec la méthode d'observation (OM) a été mise en œuvre dans ce projet pour permettre l'installation d'une partie du support permanent, ce qui à son tour améliore la productivité, gagné du temps et reduit le coût. Cet article présente les performances de surveillance de l'instrumentation à différentes étapes de la construction. Les résultats de l'analyse rétrospective et ceux des prévisions initiales sont comparés aux enregistrements de surveillance de l'instrumentation. Des modèles de sols constitutifs avancés et des paramètres de sol améliorés sont utilisés pour la prédiction du comportement du sol. Les leçons apprises sont également inclus pour la mise en œuvre à l'avenir en projets d'une échelle et d'une complexité semblable.

KEYWORDS: Deep excavation; constitutive model; ground improvement; back analysis; ERSS performance.

1 INTRODUCTION

The East Coast Integrated Depot under Civil Contract T301 is a major mass rapid transit (MRT) infrastructure project in Singapore that is under construction. It combines three rail depots and a bus depot on the same site forming the world's first 4-in-1 depot. The three independently operated rail depots are stacked, with the East-West Line elevated, Thomson East Coast Line at-grade and Downtown Line underground, saving 44 hectares of land should the depots be built separately. The bus depot is an independent at-grade four-storey roofed building adjacent to the rail depot as presented in Figure 1.

Figure 1. Architectural view of the East Coast Integrated Depot

The Civil Contract also includes a 1.45km reception tracks tunnel of 22m deep leading to the rail depot for stabling. Due for completion in 2024, the depots will be able to house 220 trains and 760 buses.

This new design philosophy by the Land Transport Authority (LTA) not only make the most use of land available in Singapore but also an economically and structurally efficient concept where the infrastructure, facilities and systems common to all three lines can be shared.

1.1 *Proposed rail depot structure*

The proposed rail depot spans over 1km and has a width varying from 145m to 360m as shown in Figure 2. It has an estimated excavation area of 23 hectares and is founded on over 2700 no. of piles. The underground rail depot has a general vertical floor span of over 14m and an excavation depth of approximately 15m. The sectional view is presented in Figure 3.

In view of the vast and extensive project site, site constraints and the massive deep soft ground condition, an excavation scheme, "Island method" with the use of temporary improved soil berms and arrays of buttress wall at areas near the existing operational Changi Depot building are adopted to support the peripheral wall (Lai et al, 2016). Due to the large scale of the project, ground preparation, piling and ground improvement works (for the berm) are constructed under an Advanced Contract.

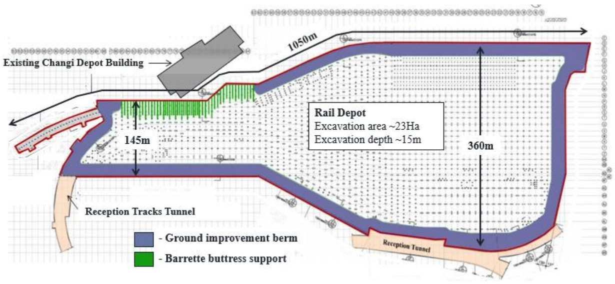

Figure 2. Rail depot layout and retaining scheme

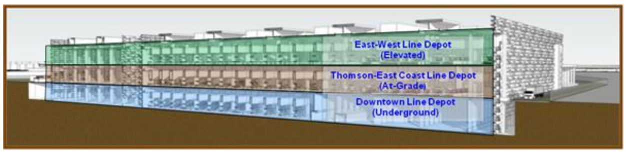

Figure 3. Rail depot cross section (LTA news release, 2018)

During the construction stage, the performance of the retaining wall is found to be better than the prediction. To enhance the productivity and time saving for the project, partial berm trimming via observational method (OM) has been implemented to allow some portion of the permanent structural elements to be cast in advance of the schedule (Lai et al, 2019).

This paper presents the instrumentation monitoring performance, results and discussion from the back analysis, i.e. comparison of initial predictions and measured monitoring records and explored the lessons learnt from the project.

2 GEOLOGICAL AND SUBSURFACE CONDITION

A total number of 210 boreholes were drilled within the rail depot footprint in the design and construction stage. Based on these boreholes, the stratigraphy encountered consists of Fill material up to 14m thick overlying the Quaternary deposits of Kallang Formation (KF) of 5m near the existing Changi Depot and 20m towards the two canals (Sungei Bedok and Sungei Ketapang). This is followed by the late tertiary Old Alluvium (OA) deposit of varying degrees of weathering and cementation with SPT-N values ranging from 5 to more than 100 blows per 300mm penetration.

The most prominent member of the KF which is encountered in almost all the boreholes is the very soft and highly compressible Marine Clay member while the remaining, are the fluvial sand (F1) and fluvial clay (F2) deposits with occasional pockets of estuarine peaty clay (E) sandwiched between the Marine Clay layer . A typical subsurface profile is presented in Figure 4.

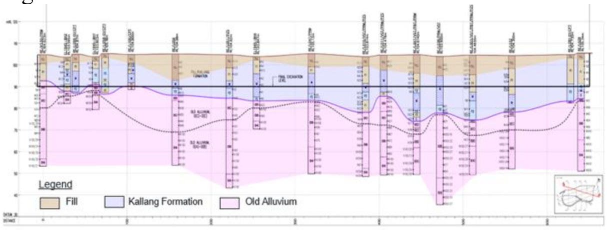

Figure 4. Typical subsurface profile

3 EARTH RETAINING STABLISING SYSTEM (ERSS)

The retaining system (ERSS) consists of a 1.2m thick diaphragm wall with the passive resistance derived solely from the 10m tall trapezoidal berm and arrays of 1m thick buttress wall skirting around the peripheral wall, thus providing ample space for bulk excavation and allowing commencement of structural works to be built bottom-up from the central portion of the rail depot based on Island method sequence.

Figure 5 and 6 shows the ERSS cross sections of the two schemes. The berm with a slope of 2V:1H, is improved using Deep Soil Mixing (DSM) columns formed by cement grout and soil mixing technique and is designed to set in 1 to 2m into OA to provide a better "interlocking" interface as to prevent possible risks of sliding should the toe level terminates in the KF layers. The buttress wall (1m thick unreinforced) is cast with concrete grade C16/20 and has a minimum width of 27m. The buttress walls are spaced at 5.7m c/c or 3.8m c/c at regions in proximity to the existing Changi Depot building.

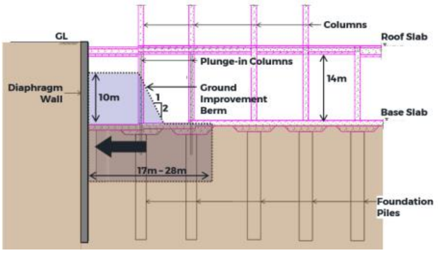

Figure 5. Typical ERSS section with temporary DSM berm

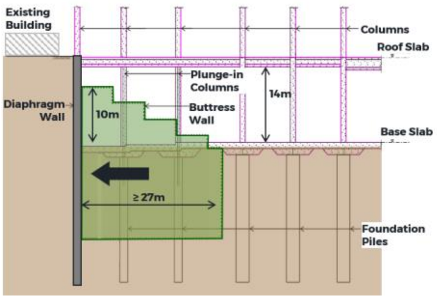

Figure 6. Typical ERSS section with buttress wall

3.1 *Construction Sequence*

Owing to no intermediate slabs below the underground depot, the diaphragm wall would behaved like a cantilever wall until the roof slab is cast and propped against the diaphragm wall. The general construction sequence for both ERSS scheme are as follows:

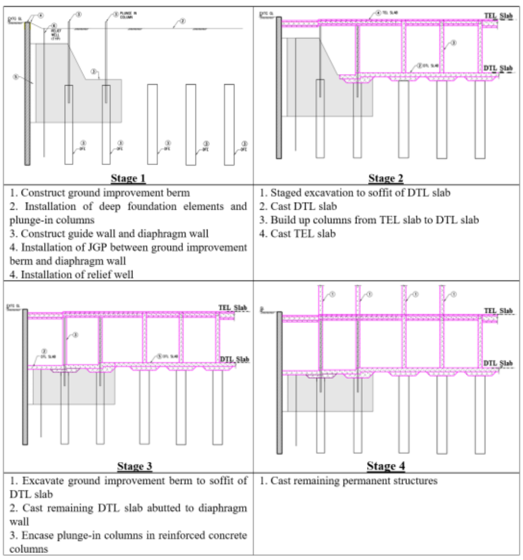

Figure 7. Construction Sequence (Temporary DSM berm)

3.2 *ERSS design*

Plane-strain model using finite element (FE) program PLAXIS 2D are used to simulate the staged construction and soil-structural interaction to obtain the ground movements and induced forces for the structural elements' design.

Hardening soil (HS) model is adopted for KF and OA soil where the stress and strain changes account for the stress-dependency of stiffness modulus. Mohr-coulomb soil model is adopted for man-made Fill and DSM material. The geotechnical design parameters correspond to the Geotechnical Interpretative Baseline Report (GIBR) whereby the characteristic values are established in accordance to Eurocode 7 (EC7).

3.3 *ERSS performance*

Temporary berm and buttress wall have been removed/excavated which is considered the completion of underground works. Based on the inclinometer readings, the diaphragm wall deformations are approximately 25% and 50% of the predicted values at the North and South side of the depot respectively.

4 BACK ANALYSIS VALIDATION

To understand the soil behavior in terms of deformation and stability of the retaining structure, back analysis has been carried out by considering the actual ground condition near the instruments' readings and the as-built berm's DSM column toe levels. The scheme using DSM berm is studied and the findings are discussed in the aspect of wall deflection and forces.

Two important factors in the numerical analysis are evaluated, i.e. constitutive soil models and geotechnical design parameters. The following summarizes various cases that have been assessed in this paper:

a) HS model with GIBR parameters (base case)
b) Hardening Soil Small strain (HSS) model with GIBR parameters
c) HS model with revised parameters
d) HSS model with revised parameters
e) HSS model with revised parameters and 10kPa surcharge and steady-state seepage condition

The purpose of Case (a) to Case (d) is to ascertain the sensitivity of the stress-strain moduli in relation to changes in effective stresses characterized by different constitutive models under ultimate limit state (ULS) design. Construction surcharge of 20kPa and the onerous condition with full water pressure is accounted for in these cases. Case (a) is with the original design assumptions (as mentioned in section 3.2), thus referenced as the base model.

Case (e) is intended for the serviceability limit state (SLS) condition whereby the surcharge is reflected actual imposed loading of 10kPa with steady state groundwater flow. This case is generated, based on HSS model with revised parameters after reviewing the findings from Case (a) to (d).

In all cases, drained condition for all soil types, except the clay members (E, F2 and Marine Clay) is considered. While it is understandable that pore water pressure built up is unlikely for the top Fill layer and F1, it might be uncertain for OA soil where its coefficient of permeability from the field and laboratory tests is in the range of 10^{-7}m/s. From the review of index tests carried out from the project site, OA soil is found to be more silty than clayey and the excavation period will be more than a year, with these considerations, drained condition is deemed to be reasonable for OA strata.

As per the design specifications, the DSM berm shall be embedded into OA soil. Hence, based on the actual ground condition, two scenarios are encountered on-site. One, DSM berm resting at the FEL due to high OA level and second, DSM berm toe level goes as deep as 10m below FEL as thicker KF layer is encountered. All the five cases will be simulated for these two scenarios.

4.1 *Constitutive Soil Models*

As opposed to the elastic-perfectly plastic Mohr-Coulomb soil model, HS model with its hyperbolic stress-strain relationship and stress-dependent soil stiffness will be able to simulate a more realistic soil behavior on the unloading/reloading stress paths during excavation. Hence, this model was adopted during the design stage.

Despite the advanced soil model, the prediction are still four to five times larger than the measured readings. Based on the measured wall deflection as shown in Figure 13 and 14, it is less than 20mm over 15m depth excavation, and according to Wong et al (2001), wall deflection within 0.2%H indicates the small strain behavior of the retaining walls. Therefore, HSS model, an extension of the HS model, was chosen to improve the prediction since small strain soil deformation is compatible under the current performance of the ERSS scheme. In addition to the parameters in HS model, G_o^{ref}, small strain shear modulus and $\gamma_{0.7}$, shear strain at which the shear modulus is decayed to 70% of the initial value are required in the FE program using HSS model.

4.2 *Geotechnical design parameters*

The parameter, G_o^{ref} for HSS model are obtained through bender element test and PS logging. Bender element test, a laboratory triaxial test in which a pair of bender elements (i.e. piezoelectric plates placed at top and bottom of the test sample), where time interval of transmission and reception of shear waves velocity propagated through the sample can be measured. PS logging, in-situ field test where the propagation of shear wave velocity is obtained via geophysical methods.

As there are limited tests carried out for this project, the test results from another LTA project with similar geological condition are used. The data are plotted in Figure 8 and Figure 9.

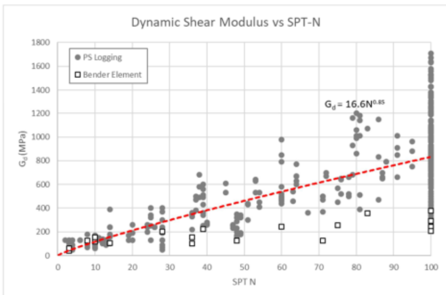

Figure 8. Dynamic shear modulus from PS logging and bender element test with SPT N value for OA soil

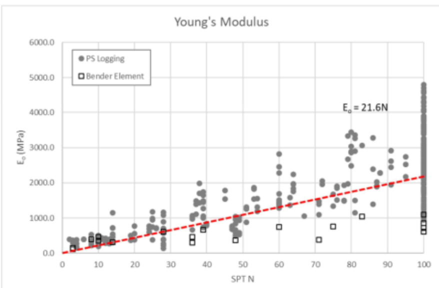

Figure 9. Young's modulus derived from PS logging and bender element test with SPT N value for OA soil

A characteristic value of G_o =16.6$N^{0.85}$ and E_o=21.6N is adopted for OA soil. The study by Veeresh et al (2015) shows correlation of the very small strain Young's modulus with SPT N is E_o=12N and 15N for bender element test and PS logging respectively. The correlation falls in the lower bound of the test results in Figure 9, where it is observed that Bender Element tests generally yields a lower modulus possibility due to sample extraction disturbance at the range of SPT N values more than 30 blows count.

For the KF members comprises of soft cohesive soil and loose granular material, the derivation of the G_o value is based on the published empirical co-relationship after Alpan (1970) (Figure 10) relating dynamic soil stiffness to static soils stiffness. The derived G_o (MPa) value is 9.1$c_u^{0.48}$ and 18.2$N^{0.6}$, for cohesive and granular material, respectively. The G_o= 9.1$c_u^{0.48}$ of cohesive soil is counterchecked with the Seismic Cone Penetration Test (SCPT) test results from LTA's past projects and it appeared to fall in the region of the test results of Marine Clay of KF as shown in Figure 11. While the G_o=18.2$N^{0.6}$ for granular soil (F1 material) is generally two times lesser than the G_o value of OA material which is considered acceptable in term of soil consistency.

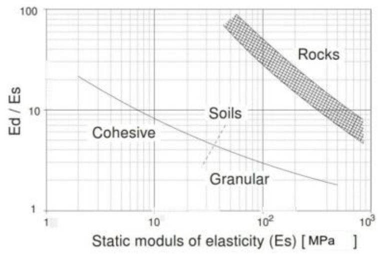

Figure 10. Relationship between Dynamic and Static Soil Stiffness (Alpan, 1970)

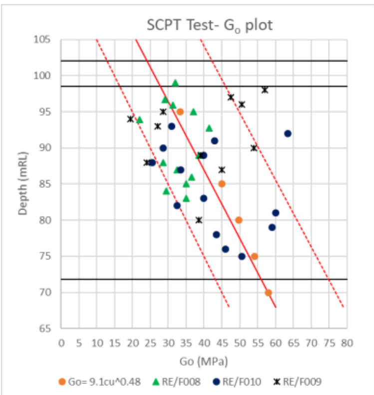

Figure 11. Measured G_o from SCPT test extracted from LTA past project

The shear strain $\gamma_{0.7}$ is derived from the influence of plasticity index on stiffness reduction after Vucetic and Dobry (1991) shown in Figure 12. Due to the limitation of local strain measurement in laboratories in Singapore, empirical correlation chart is used in the derivation of the local strain values.

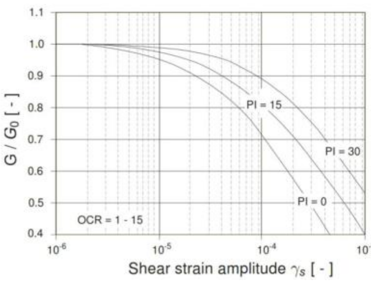

Figure 12. Influence of Plasticity Index on Stiffness Reduction (Vucetic and Dobry, 1991)

The revised soil parameters were determined after a review on the laboratory tests results for a set of more probable values rather than the GIBR's characteristic values.

The strength and stiffness of the DSM were also updated in the back-analysis. During the design stage, a minimum strength and stiffness values of 600kPa and 230MPa were specified. Since the core test data are generally above the minimum values, location specific characteristic values are determined using the approximately 4000 DSM core test data gathered during the Advance Contract. The GIBR and revised parameters are shown in Table 1.

Table 1 GIBR and revised geotechnical design parameters

Geotechnical Unit	Parameters	c' (kPa)	ϕ' (°)	c_u (kPa)	HS Model	HSS Model	
					E_{50}^{***} (MPa)	G_0 (MPa)	$\gamma_{0.7}$
Fill	GIBR*	0	30	-	10	-	-
	Revised**	-	-		30	-	-
F1	GIBR*	0	32	-	15	-	-
	Revised**	-	33	-	20	18.2$N^{0.6}$	1.1 x 10^{-4}
M	GIBR*	0	24	17+1.5(z-5)	300c_u	-	-
	Revised**	-	-	20+1.75(z-5)	400c_u	9.1$c_u^{0.48}$	7.5 x 10^{-4}
OA(E)	GIBR*	2	30	5N	2.6N	-	-
	Revised**	3	31	-	4N	16.6$N^{0.85}$	3.3 x 10^{-4}
OA(D)	GIBR*	3	32	5N	2.6N	-	-
	Revised**	5	33	-	4N	16.6$N^{0.85}$	3.3 x 10^{-4}
OA(C)	GIBR*	10	35	5N	2.6N	-	-
	Revised**	12	-	-	4N	16.6$N^{0.85}$	3.3 x 10^{-4}
OA(B)	GIBR*	15	35	4.2N	2.6N	-	-
	Revised**	-	-	-	4N	16.6$N^{0.85}$	3.3 x 10 -4
OA(A)	GIBR*	20	35	4.2N	2.6N	-	-
	Revised**	-	-	-	4N	16.6$N^{0.85}$	3.3 x 10^{-4}
DSM	GIBR*	-	-	600	230	-	-
	Revised**	-	-	1165	370	-	-

*GIBR: characteristic values

**revised parameters: more probable values

***E_{50}^{ref}=1.15E_{oed}^{ref}; E_{50}^{ref}=3E_{ur}^{ref}

4.3 *Concept of mid-point in wall deflection*

Ideally, the back-analysis results should give a close match (possibly <5%) to the measured readings. However, it is realized that the parameters required to achieve this close match might not be sensible based on experience and published literature. Besides, there is a lack of soil test data to substantiate the required parameters. Hence, instead of targeting for a close match, the wall deflection from the back-analysis is compared to the mid-point of the base case and the measured value. The soil model and parameters adopted are at the best knowledge of the soil behavior and site condition. The purpose of using the mid-point is also to have some conservatism in design, and the findings will be for the use in future projects.

4.4 *Results and discussion*

4.4.1 *Scenario where DSM toe level terminates at FEL ("shallow DSM")*

The retaining wall's deflection and induced wall forces for the case where the OA level is high is presented in Figure 13. The measured wall deflection along this section is approximately 10mm. It is five times lesser than the estimate of 54mm in the base case (Case (a)). Even when HSS small strain model is considered under Case (b), the deflection is still four times, at 40mm, exceeding the midpoint of measured and base case.

Understandably, the wall deflection for both HS model and HSS model with revised parameters are lower, at 32mm and 34mm respectively due to a higher shear strength. Both values fall in the midpoint range. Although HSS model is giving a slightly higher wall deflection magnitude (at the top of the retaining wall), the profiles using HSS model is considered to be more representative, with the wall deflection tapering below FEL, approaching the stiffer OA layer.

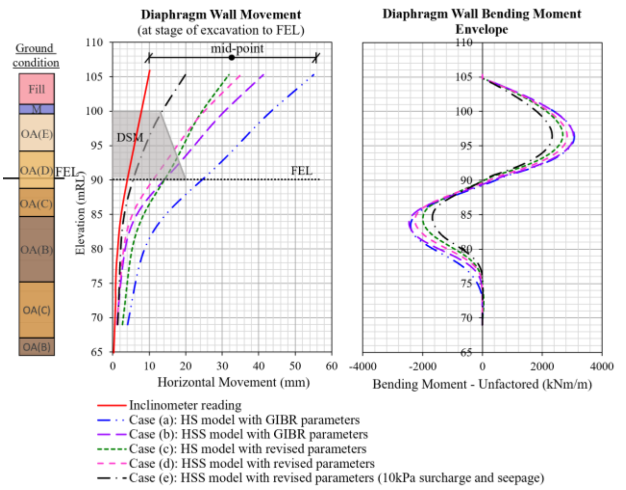

Figure 13. Wall deflection and forces for shallow DSM

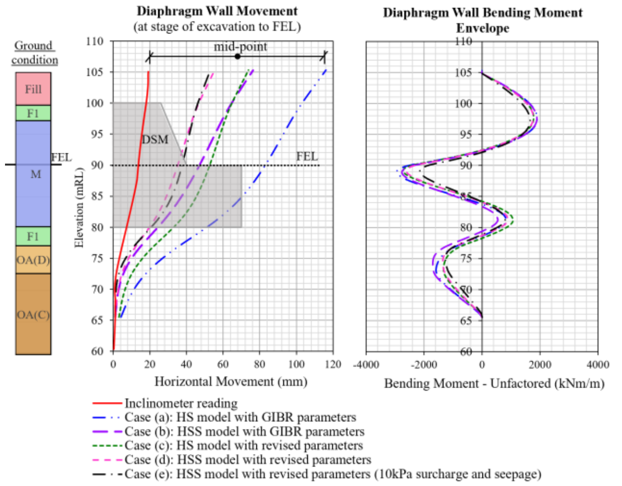

Figure 14. Wall deflection and forces for deeper DSM

In light of the above findings, Case (e) is developed, using HSS model with revised parameters based on steady-state flow seepage analysis and 10kPa surcharge. Despite that the maximum computed wall deflection of 19mm is approximately twice the measured readings, the deflection profile is observed to be similar to the measured one in the stiffer OA layer.

Further refinement of this case can be done by further adjusting the parameters such as using values from the higher bound, but this is not the intention for this study without justification from field or laboratory test results.

The forces from HS and HSS model with GIBR parameters produced forces of similar magnitude. Nonetheless, HSS model seem to produce a more realistic bending moment profile particularly at the stiffer OA strata i.e. slightly higher at the peak and reduced sharply compared to HS model as replicating the curvature from the wall deflection profile.

It is noted that for the case of HSS model with revised parameters, the wall forces is reduced by in the order of 10% to 15% which might have some savings in rebar quantity but the crucial point is that the model gives a realistic prediction of the ground movement. This in turn, will be helpful when assessing the impact of deep excavation to surrounding buildings and structures.

4.4.2 *Scenario where DSM toe level terminates way below FEL ("deep DSM")*

For the case where the geological condition is predominantly KF material, it is noted that the wall deflection from HSS model is very close to the HS model with revised parameters and is much better in terms of curvature wall profile and fixity towards the more competent OA strata as presented in Figure 14.

The wall deflection from Case (b) is close to the mid-point. This indicates that the HSS model with GIBR parameter showed vast improvement in terms of wall deflection prediction which is comparable to HS model with revised parameter. The response from Case (e) is relatively identical as per Case (d), indicating that the shear strength of KF material has been mobilised. Therefore, there is minimal impact on wall deflection when the imposed surcharge and water pressure are reduced.

For the induced forces, except for the portion closer to the toe of retaining wall, the forces magnitude is within 10%. In contrary to some practices that uses a different soil model to be set up to assess the serviceability limit state (SLS), the findings enhance the confidence of using the same model, i.e. HSS model for both ULS and SLS in the semi-bottom-up structure.

5 LESSONS LEARNT

From the designer point of view some of the lessons learnt are summarised below:

a) *Erosion control for the DSM berm*
Despite having already divided the construction into Zones due to the large project site, the DSM berm is left exposed on-site for more than a year. There have been instances where portion of the berm is eroded and rectification measures such as shotcrete is applied to the berm surface. In retrospect, though the berm is cement-treated, slope protection measures such as shotcrete or covering sheets would have prevented the erosion.

b) *Proper drainage system*
There are occurrences where rainwater collected from the ground surface pours through the drainage points located at the top of the diaphragm wall, directly onto the DSM berm. Proper drainage paths should have been thought through and set up, so that no excess rainwater will flow directly onto the DSM berm which eventually infiltrates the berm through the weak points i.e. the joints between each DSM column, causing erosion not just on the berm surface, but also within the berm. Until tension cracks are observed on top of the berm, the weakened locations can be difficult to trace. Moreover, as the retaining system is solely dependent on the DSM berm, the damage can be catastrophic should there be no rectification measures done in time.

c) *Calibration of DSM machine torque to soil types*
The DSM toe levels could be highly variable due to actual ground condition. Although cone penetration tests (CPT) spaced 20m apart are carried out along the DSM berm prior to DSM installation to have a rough gauge on the toe levels, the toe levels could be still far from the gauge for a 2.3m width DSM column (Lai et al, 2016). To avoid disputes and eliminate the ambiguity as far as practically possible, calibration of DSM machine torque to different soil types should be established during a trial test to determine the 1m penetration into OA and this protocol should be used as a control parameter and standard during DSM

installation on-site especially when there are different operators involved.

d) *Configuration of DSM berm*
Instead of striving to achieve fairly, the same toe level for the DSM columns, an alternate long-short configuration may be feasible to provide lateral restraint while saving the amount of ground improvement required.

e) *Backfilling material*
The voids created from the boring for plunge-in column's installation are backfilled with sand. As the berm surface is not protected, it is observed that fines are lost around the plunge-in columns due to rain exposure as shown in Figure 14. The voids in turn creates a flow path and causes discontinuity in the ground improvement berm. Liquified soil stabilizer would have prevented such voids.

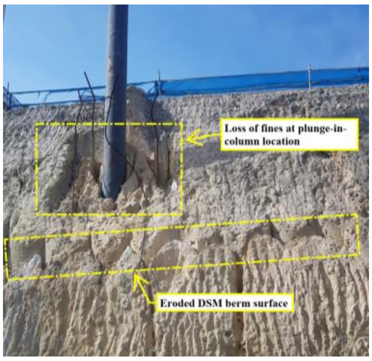

Figure 14. Close-up view of eroded DSM berm surface and the loss of fines at plunge-in column location

f) *Orientation of DSM columns*
Each DSM column is orientated with its width aligned along the diaphragm wall. In view of the excavation behaviour, a stiffer configuration, the major axis of the DSM columns perpendicular to the diaphragm wall may be more efficient as shown on the right of Figure 15.

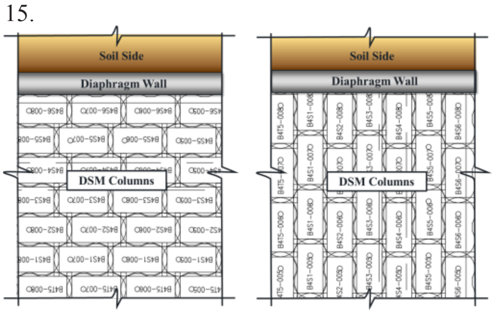

Figure 15. DSM columns orientation with respect to the diaphragm wall

5 CONCLUSIONS

The sensitivity of using a separate constitutive soil model and a set of probable parameters rather than the characteristic values are studied in this paper. Based on the findings, HSS model seems to be a better fit in capturing the unloading/reloading behavior for OA soil in particular for this case study. It yields a closer retaining wall deformation compared to the measured readings while having induced forces of similar magnitude.

Hence, it is recommended that HSS model can be used for projects in similar ground condition, on the premise that enough reliable site investigation and appropriate testing to determine the soil parameters are carried out. Though it must be emphasized that all constitutive soil models have its own limitations in reproducing accurately the stress-strain changes in all loading conditions.

Nevertheless, the findings could act as a basis and foresight in allocating budget to cater for certain targeted testing (in this case, PS logging and bender element tests to obtain information on the small-strain deformation behavior) which might require advanced equipment and is thus, not as readily accessible and available compared to the conventional triaxial tests.

Furthermore, in the local industry practice where the magnitude of retaining wall's movement is used as a mandatory limiting criterion by the Authority, the outcome where the HSS model is seen to be better in predicting the wall deformation and to comply the limiting criterion. In other words, measures such as a stiffer retaining wall, additional levels of temporary restraints especially in a built-up area which are necessary previously, to reduce the deformation of retaining wall to comply to the Authority's regulations could be evaded. This economic and robust (not over-designing) design approach will also be in-line with the goals of achieving sustainable development.

The lessons learnt from Section 5 mainly pertaining to the challenges dealing with DSM such as erosion protection of berm surface, interface between plunge-in columns and DSM block under exposure, installation constraints due to actual site conditions can be applied to future projects with similar ground condition and site geometry.

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