Design and Construction of Ground Improvement for TMCLKL Southern Ventilation Building

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Abstract

The Southern Ventilation Building (SVB) is located on newly reclaimed land where 15-20 m of Sand Fill overly 15m of soft to firm silty clay Marine Deposit which was improved with prefabricated band drains. The SVB sits directly above the Tuen-Mun Chek Lap Kok Tunnels (TMCLKL). The original design required installation of over 330 number of shaft grouted piles. Due to the presence of the tunnels the pile design was relatively inefficient and required 3m thick pilecaps. An alternative design was developed to delete the piled foundations by carrying out additional ground improvement within the soft soil layer using a combination of Jet Grouting and Deep Cement Mix using Cutter Soil Mix (CSM) and support the building on a raft foundation. The ground improvement scheme was also leveraged to reduce the temporary wall depth and shoring quantities. In order to rationalise and minimise the ground improvement quantities, the CSM panels formed a grid of orthogonal underground beams which allowed an efficient Area Replacement Ratio to be achieved. The excavation and building construction has been successfully completed with excellent performance in terms of both ongoing settlement of the permanent works and lateral movement of the excavation retaining walls during the temporary works.

Keywords: Ground improvement, Reclamation, Deep excavation

1 Introduction

The Tuen Mun - Chek Lap Kok Link (TMCLKL) is a new transportation route that provides a strategic connection between North West New Territories, North Lantau, the Hong Kong – Zhuhai – Macao Bridge Hong Kong Boundary Crossing Facilities (HKBCF) and the Hong Kong International Airport (HKIA) at Chek Lap Kok.

The Southern Ventilation Building (SVB) is located at the northern tip of the HKBCF ('Southern Landfall') as shown in Figure 1 and Figure 2. The SVB provides ventilation facilities to the twin 14m diameter tunnels, which are approximately 40m below ground at this location. The ground conditions include Fill overlying soft marine clay, stiff/dense alluvial deposits and completely decomposed metasiltstone.

The original design required a piled foundation with over 330 shaft grouted piles connected to 3m thick pilecaps. An alternative design was developed which involved founding the SVB on a shallow foundation sitting directly on ground improved by Deep Cement Mixing and Jet Grouting, thereby allowing all the piles to be deleted. In addition, the ground improvement was also used to enhance the stability of the temporary works, allowing a rationalization and quantity reduction for both the vertical retaining wall and the horizontal shoring system.

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This paper describes the design philosophy adopted for foundations and lateral support works. The ground improvement performance is illustrated by a comparison of predicted soil movements against monitoring data both in temporary and permanent stages.

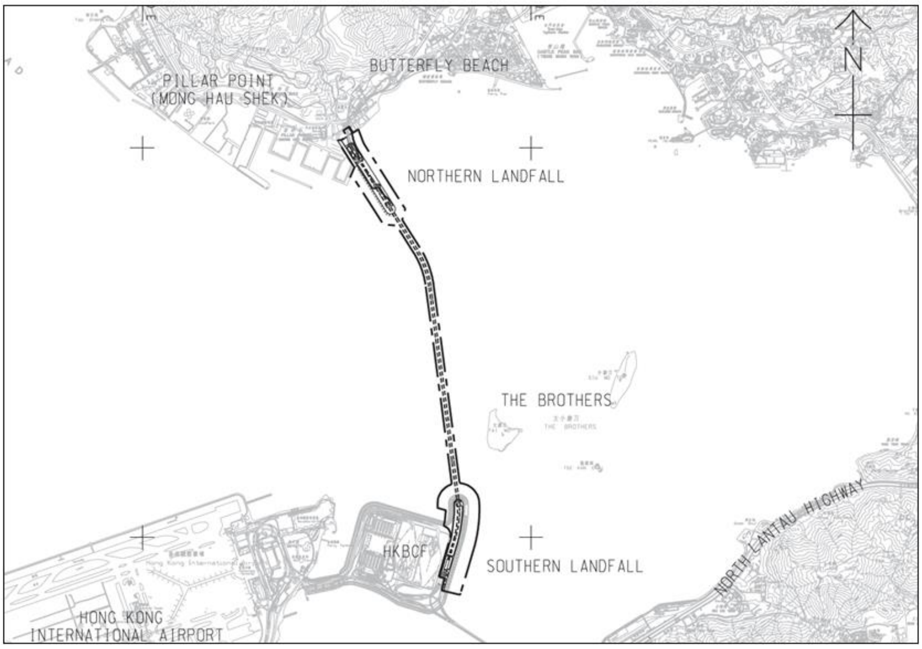

Figure 1: Site Location Plan

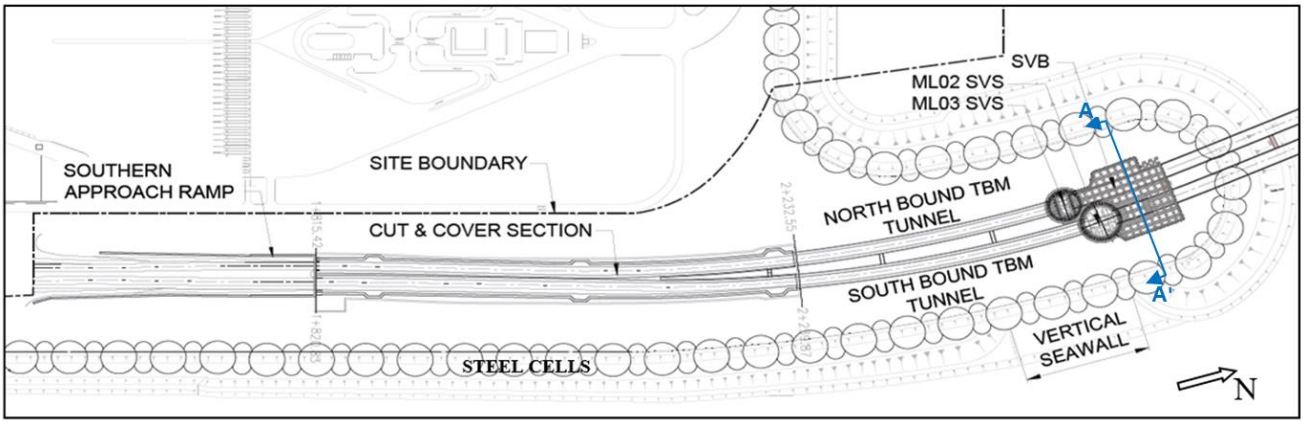

Figure 2: Southern Landfall Plan

2 Project Description

The SVB comprises 2 levels of basement and a 3 story superstructure, with a footprint of approximately 65m x 47m, housing the tunnel ventilation systems as well as electrical systems, fire services provisions and drainage system. As shown in Figure 2, the SVB is adjacent to two circular shafts MLO2 SVS and MLO3 SVS, which connect the ventilation system to the 40m deep tunnels.

Due to the presence of the twin tunnels, the original foundations design required construction of 3 large 3m thick pile caps, connected by a relatively thick slab to span over the two tunnel footprints, see Figure 3.

Grade III rock of founding quality was identified to be at a depth of over 100m below ground level over this area. As a result of the depth to rock head, 330 shaft grouted H-piles approximately 70m deep were initially proposed to support the building.

The foundation redesign using ground improvement allowed the building to be founded on an at-grade slab of variable thickness between 1.5 and 2.5 m. The underside of the basement slab is between -8.5 to -9.5 mPD, which is approximately 15m below existing ground level, as shown in Figure 4.

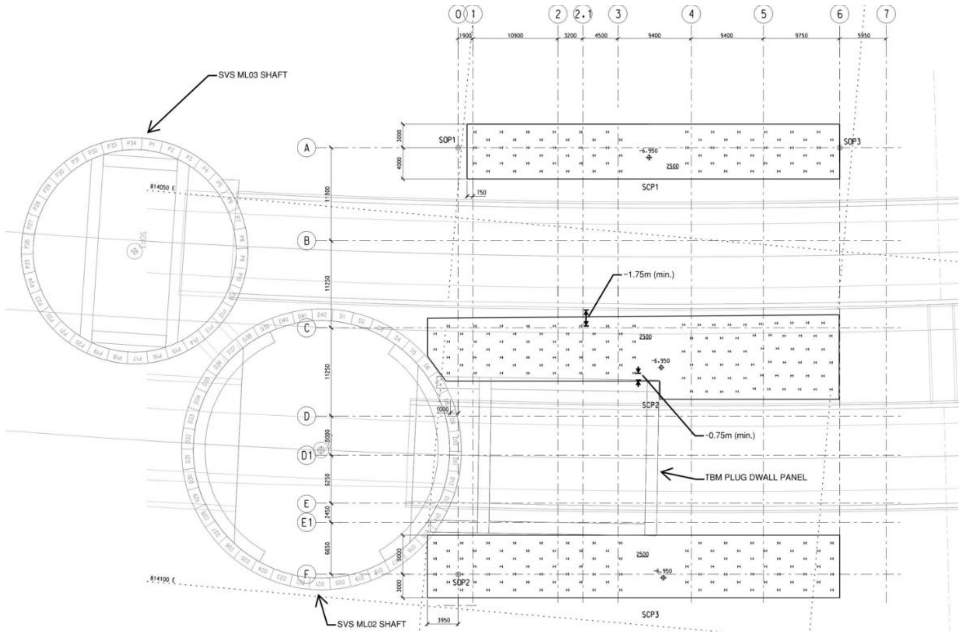

Figure 3: Layout Plan of Original Piling Scheme

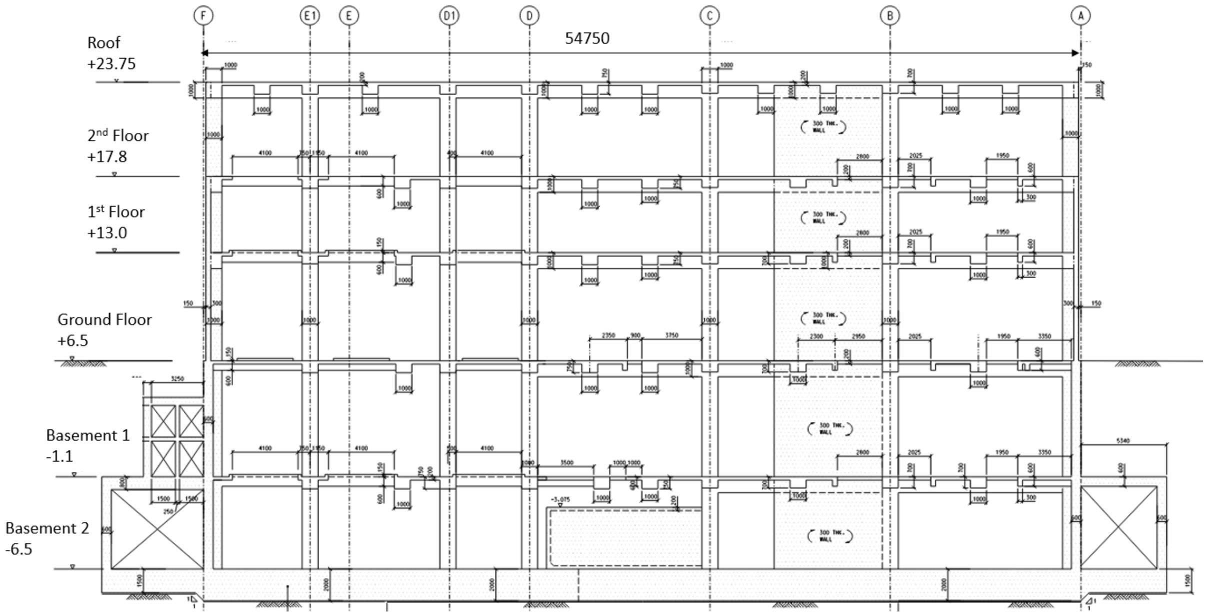

Figure 4: Cross-Section of Raft Foundation Scheme

3 Ground Conditions

The geological profile across the site has been altered by the reclamation works undertaken in the area to form the HKBCF island. The site formation level is at +5.5 mPD. The reclaimed land was constructed using sand fill placed directly over soft marine clay improved with Prefabricated Vertical Drains (PVD). The geological profile after reclamation consists of 4 main layers above engineering rock head:

a) Fill – typically medium dense to dense sand, with isolated pockets of soft disturbed clay towards the bottom of the layer
b) Marine Deposit – typically very soft to firm silty clay
c) Alluvial Deposit – typically interbedded layers of firm to stiff silty clay and medium dense to very dense silty clayey sand
d) Completely and Highly Decomposed Metasiltstone (MSLSTN)

No grade III or better rock was identified in any of the boreholes at the SVB which were drilled to a depth of approximately 100 m below ground level. The typical groundwater level is between +2 and +3 mPD, which is slightly higher than mean sea level. A geological section is shown in Figure 5. Refer to Figure 2 for the location of the geological section.

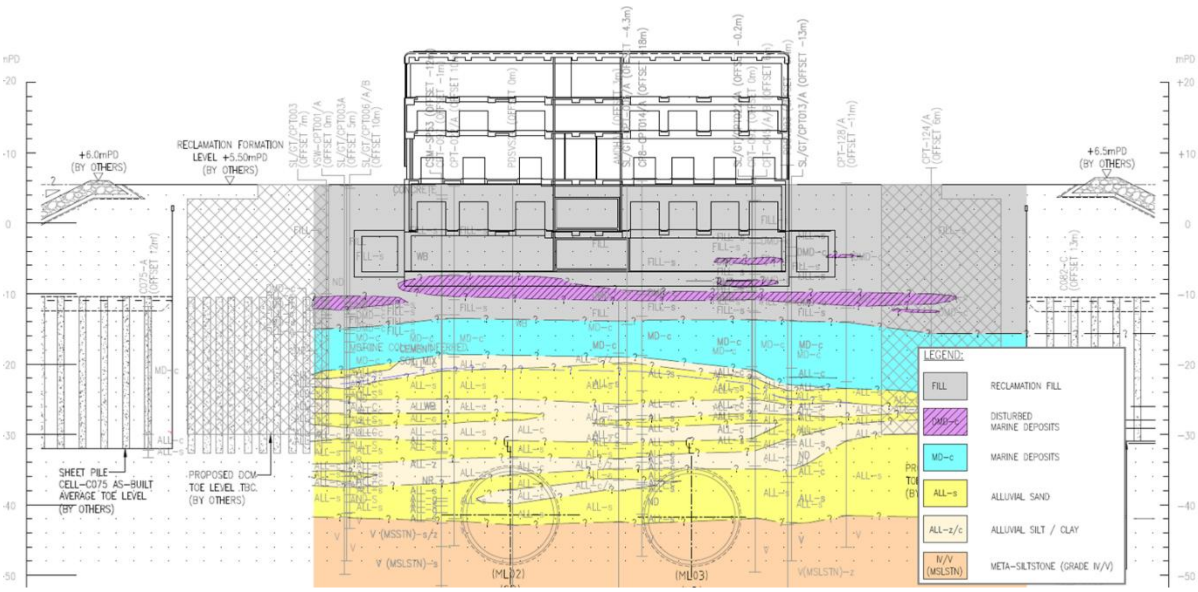

Figure 5: Geological Section

Soil parameters adopted for ELS and raft foundation design are presented in Table 1 and Table 2 respectively.

Table 1. Soil Parameters Adopted for ELS Design

Geological Stratum	γ_{sat} (γ_{unsat}) [kN/m^3]	Cu [kPa]		c' [kPa]	ϕ' [°]	Eu [kPa]	E' [MPa]	
Fill	20.0 (17.5)	-		0	33	-	10	
Marine Deposits	16.0	22 + 2.5 x (-15 -z)		0	25	300 x Cu	-	
Alluvial Clay (C1/C2)	18.5	60 60 + 2.0 x (-27 -z)	z ≥ -27 z < -27	0	28	350 x Cu	-	
Alluvial Sand	18.5	-		0	28	-	20 20 + 4.0 x (-30 -z)	z ≥ -30 z < -30
Metasiltstone	20.0	-		3	30	-	40 40 + 2.0 x (-45 -z)	z ≥ -45 z < -45

z is elevation in mPD

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Table 2. Additional Soil Parameters Adopted for Foundations Design

Geological Stratum	e_0	C_c	C_r	$C_{\alpha e}$	Ch – Cv (m2/year)	OCR
Marine Deposits	N/A, soil improved with DCM					
Alluvial Clay (C1)	0.85	0.25	0.045	0.36	5.0 – 3.0	1.2
Alluvial Clay (C2)	1.1	0.6	0.045	0.86	5.0 – 3.0	1.0

4 Ground Improvement Design

The ground improvement was designed to fulfil a dual function of vertical support for the permanent works raft foundation and lateral support for the temporary works retaining wall.

The depth of ground improvement is primarily driven by the foundation requirements, as treatment of the entire Marine Deposit was necessary to reduce long term settlements to acceptable levels. The ground improvement was therefore installed between a cut-off level of -8.0 mPD to a toe level typically between -20.0 mPD to -24.0 mPD. as shown in Figure 6. The toe level was designed to achieve a 2 m minimum embedment below the top of Alluvial Deposit, in order to cater for potential local variability of the ground profile and ensure that the ground improvement is founded on competent soil.

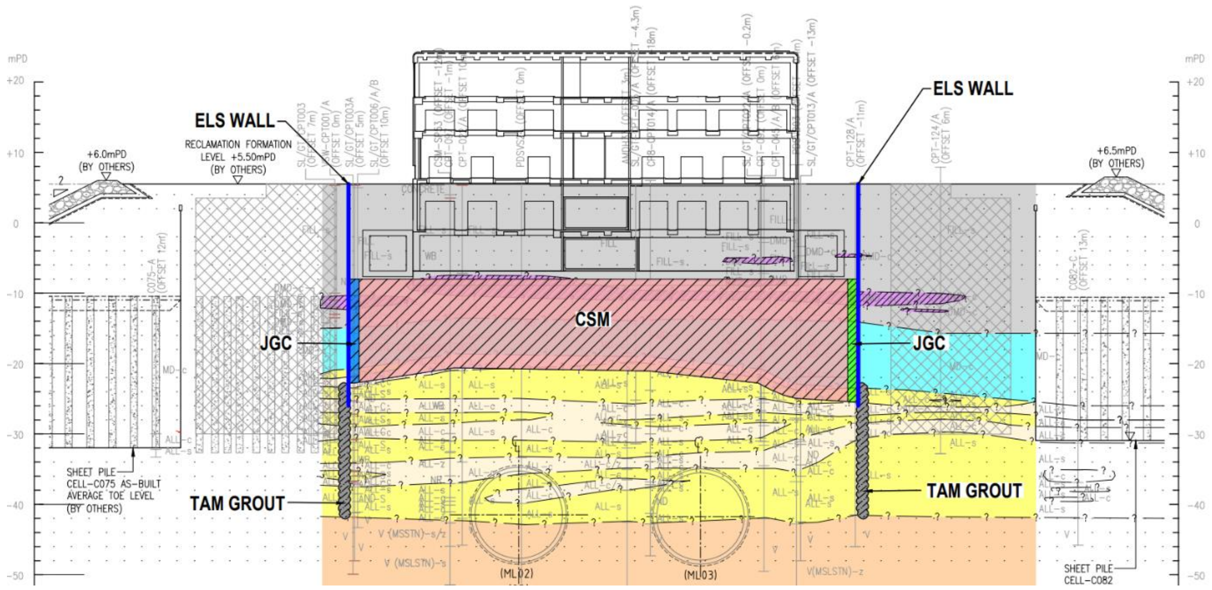

Figure 6: Ground Improvement Section

The ground improvement geometry replacement ratio was instead driven primarily by the excavation requirements. An orthogonal grid formed primarily of contiguous Cutter Soil Mix (CSM) panels was formed with an approximate replacement ratio of 55%, as shown in Figure 7. The dimensions of each panel are 1.2 x 2.4 m^2. After completion of the CSM and installation of the ELS walls, an outer ring of Jet Grout Columns (JGC) of 1.5 to 1.8m diameter was installed between the CSM and the retaining walls to ensure adequate connection between the wall and the CSM grid for toe stability.

A number of CSM panels were previously installed for the construction of the circular shaft diaphragm walls and a parking plug for the tunnel boring machine to the north of the larger shaft. The ground improvement design for the SVB therefore incorporated these existing panels.

The ground improvement grids are formed by a pair of adjacent panels and are spaced at 7.2 m. Therefore, for the purpose of ground improvement passive pressure and toe stability, the effective replacement ratio of ground improvement grids orthogonal to the retaining wall is 33% (=2.4/7.2).

Typical ground improvement solutions for retaining wall design include relatively thin underground slabs with high replacement ratio, close to 100% (Page et al.2005, Wen 2005). For the SVB case, due to the dual function requirements, deep panels of 10 to 14m thickness below excavation level with relatively low replacement ratio were instead adopted. Due to the low replacement ratio, passive pressure is concentrated into the grids in orthogonal direction to the retaining walls, therefore a minimum UCS of 1.5 MPa was specified to provide sufficient lateral capacity. A summary of minimum required ground improvement parameters is provided in Table 3.

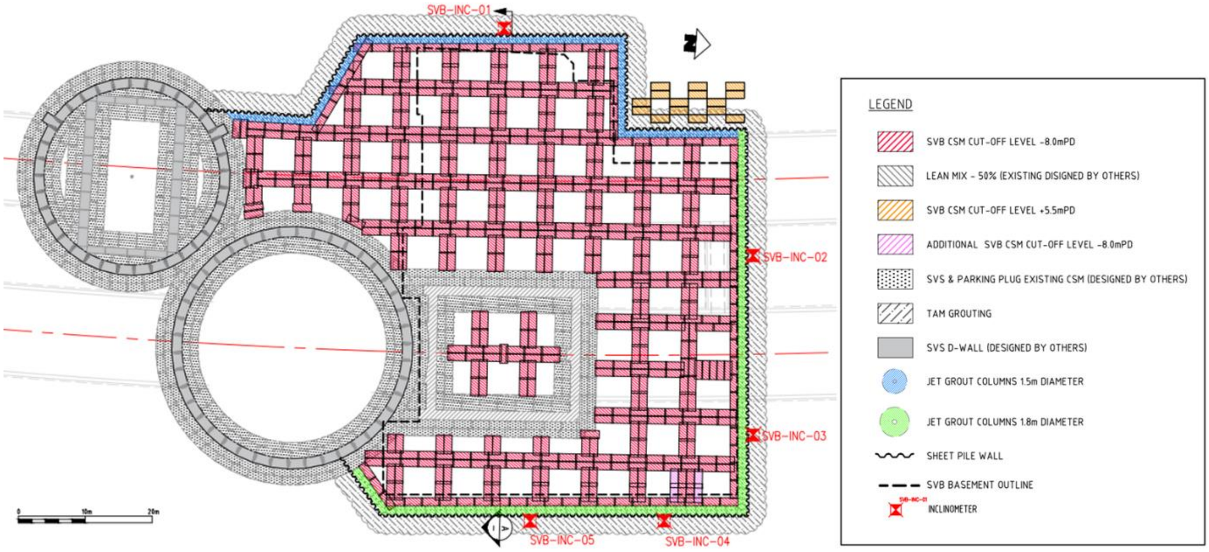

Figure 7: Ground Improvement Layout Plan

Table 3. Ground Improvement Minimum Required Parameters

Ground Improvement	**γ_{sat} [kN/m^3]**	**UCS [MPa]**	**E [MPa]**	**SCR [%]**
CSM and JGC	15.0	1.5	200	90

5 Temporary Excavation Design

The basement construction required a 15 m deep excavation from ground level of +5.5 mPD to final excavation level of -9.5 mPD.

The ground improvement grid provides a stiff passive support to the retaining wall below ground level. This allowed optimisation of the shoring design quantities in comparison to the original excavation design which had been previously developed for the piled foundation scheme. The original design

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included a combi-wall of 1.2 m diameter pipe piles connected with double AZ18 sheet piles, embedded to a toe level of -30 mPD and supported by four levels of struts.

For the alternative design with ground improvement, the retaining wall size was reduced to FSP-V sheet-piles embedded to a toe level of -26 mPD. It should be noted that the toe level for the alternative design was driven by the need to form a water cut-off in the relatively thick Alluvial Sand layer identified below the Marine Deposit, in order to mitigate the risk of hydraulic uplift of the improved Marine Deposit layer and to reduce the groundwater inflow during the excavation works.

In addition, the number of strut levels was reduced from 4 to 3 levels. The strut quantities were further optimised by aligning the struts centerline in an orthogonal grid and connecting them at their intersection points. This configuration avoided using very long corner struts to deal with the particular geometry of the cofferdam and minimised the requirements for cross-bracing. The interaction between the different struts and walers generated by the grid alignment requires special consideration and therefore 3D analyses were carried out to assess the shoring performance, as shown in Figure 8.

The structural forces acting on the retaining wall and struts were assessed using both 2D and 3D Finite Element analyses, as shown in Figure 9. 3D analyses were required in particular to verify that the ground improvement capacity would be adequate to resist the horizontal stress concentration into the grids perpendicular to the retaining wall.

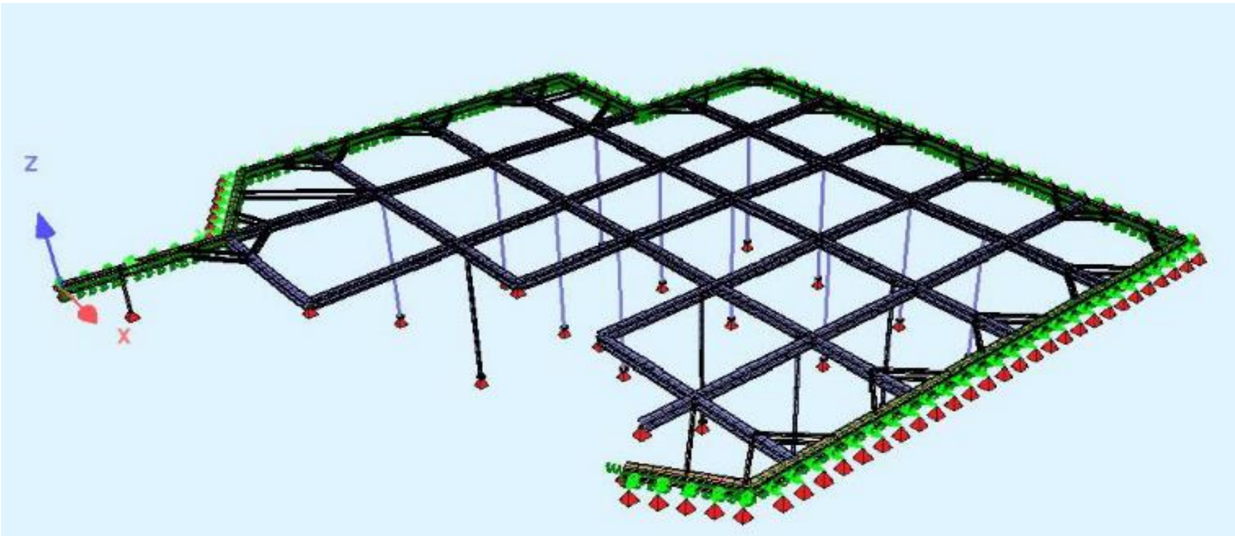

Figure 8: 3D Structural Model for Strut Design

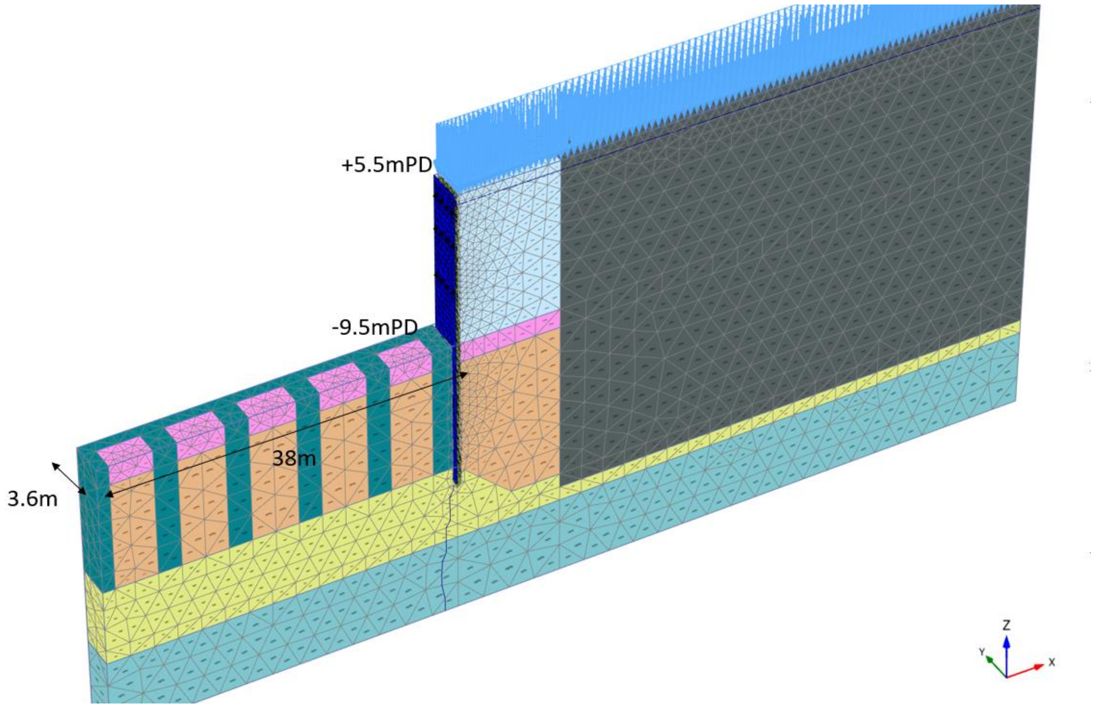

Figure 9: 3D FE Model for Temporary Excavation Design

6 Raft Foundation Design

The SVB is founded on a continuous RC slab with top level generally of -6.95 mPD and thickness varying between 1.5 and 2.5 m.

The load-settlement behaviour of the building was modelled using 3D Finite Element soil-structure interaction analyses. The slab detailed geometry with variable thickness is modelled using continuum elements. As shown in Figure 10, the loads from each column, load bearing wall, and the distributed loads on the base slab are modelled individually in order to determine a realistic load distribution at the underside of the slab. The bearing pressure is typically between 100 and 250 kPa, as shown in Figure 11.

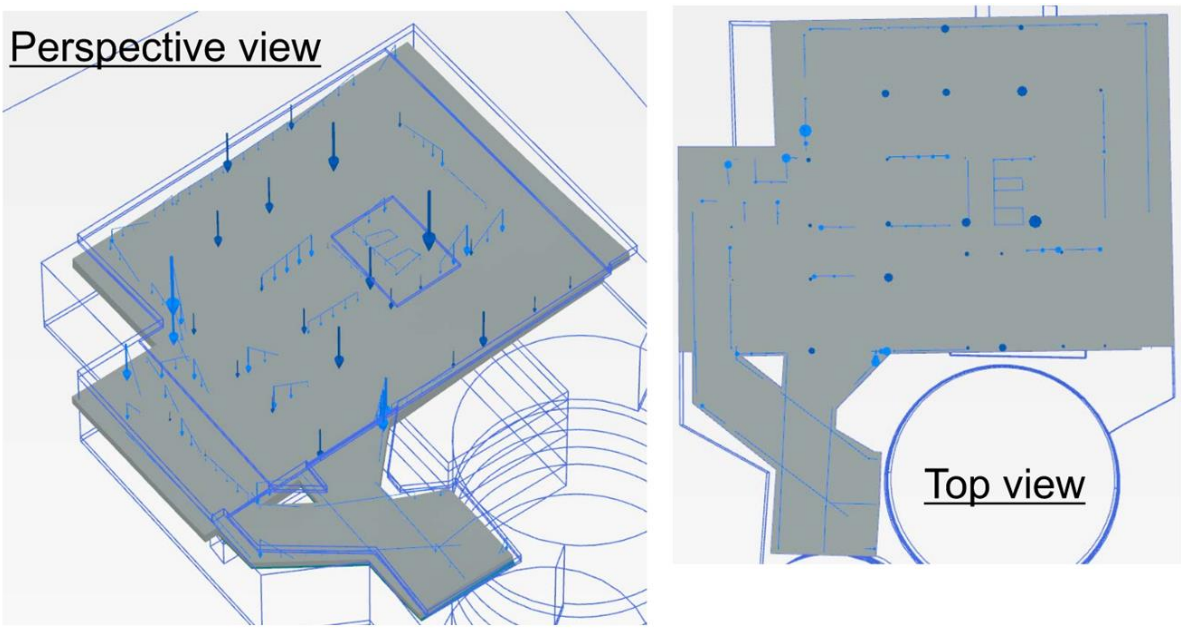

Figure 10: 3D FE Model for Foundations Design

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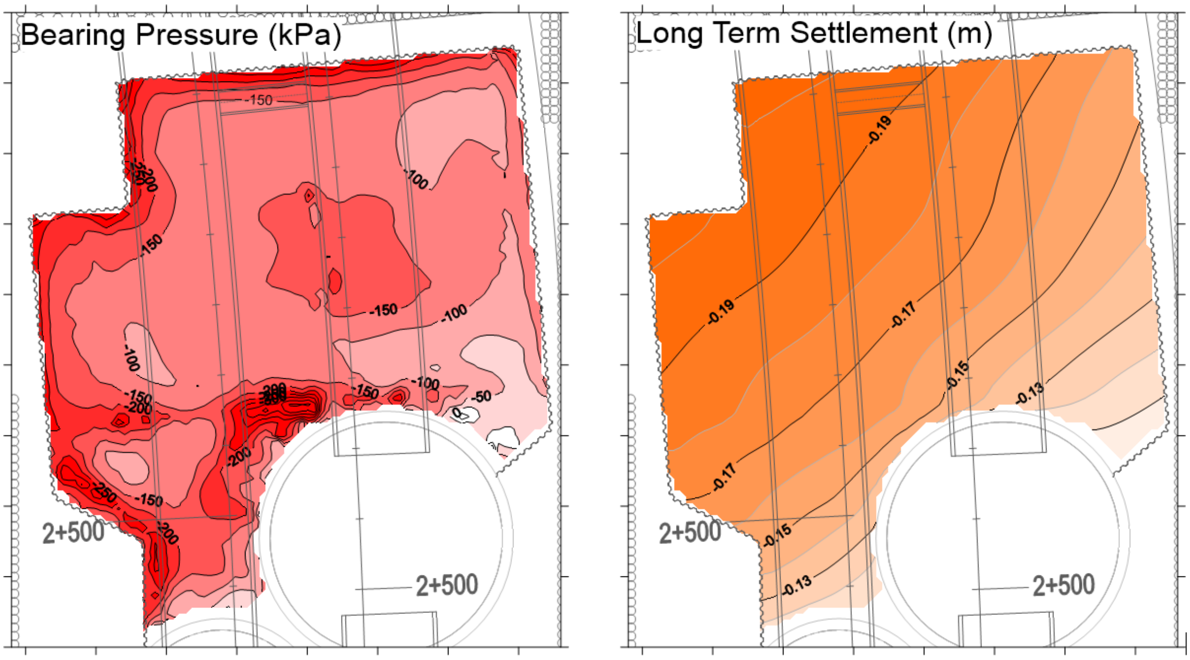

Figure 11: Raft Foundation Bearing Pressure and Predicted Long Term Settlement

The Alluvial Sand and CDM materials are modelled using the Mohr-Coulomb soil model, whilst the Alluvial Clays C1 and C2 are modelled with the 'Soft Soil Creep' model (Stolle et al. 1999). The effects of the reclamation additional overburden and unloading caused by the temporary excavation are considered in order to estimate a realistic vertical stress distribution within the underlying Alluvial Clays. For long term settlement assessment, the beneficial contribution of the groundwater uplift pressure at the underside of the slab was considered assuming a mean low water level of +0.3 mPD.

The analyses predicted a long term settlement of approximately 110 to 190 mm over 120 years design life. Approximately 50 mm of the total settlement were expected to occur in short term during construction.

7 Ground Improvement Performance Review

7.1 Ground Improvement Quality Control

Over 600 CSM panels and 150 JGCs were installed under the SVB. The strength and quality of the ground treatment was verified by carrying out a total of 30 and 8 cores in the CSM and JGCs respectively. For each core, four samples were selected for UCS testing. The test results are summarised in Figure 12. Approximately 98% and 92% of the tests demonstrated higher UCS than the minimum specified 1.5 MPa for the CSM and JGC respectively, which met the minimum 90% passing criterion. The mean UCS is approximately 4.6 MPa for both CSM and JGC, whilst the standard deviation is 1.3 MPa and 1.9 MPa for the CSM and JGC respectively.

It is considered that the ground improvement strength achieved in-situ exceeded the design requirement, in particular for the CSM panels which form the critical grid at a lower replacement ratio within the footprint of the excavation.

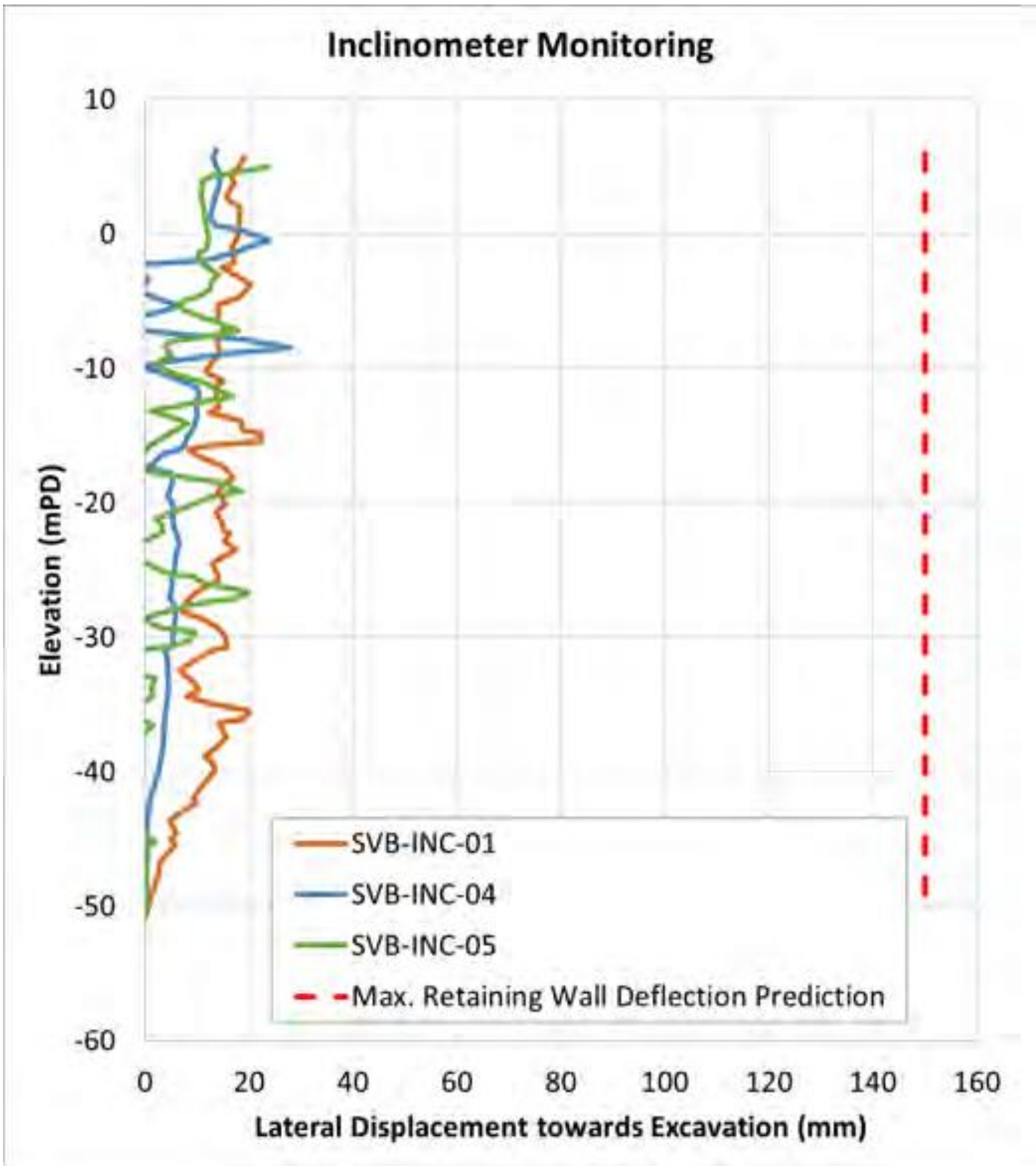

Figure 12: UCS and Young's Modulus QC Testing Results

7.2 Excavation Performance

A total of 5 inclinometers were installed directly behind the sheet pile wall to monitor the lateral movements during the excavation and construction of the basement, as shown in Figure 7. A plot of available data after completion of the basement is presented in Figure 13. Typically less than 20 mm lateral movement was measured, which is significantly smaller than the predicted 150 mm maximum retaining wall deflections, indicating that the ground improvement and strutting system was very effective in supporting the 15m deep cofferdam excavation.

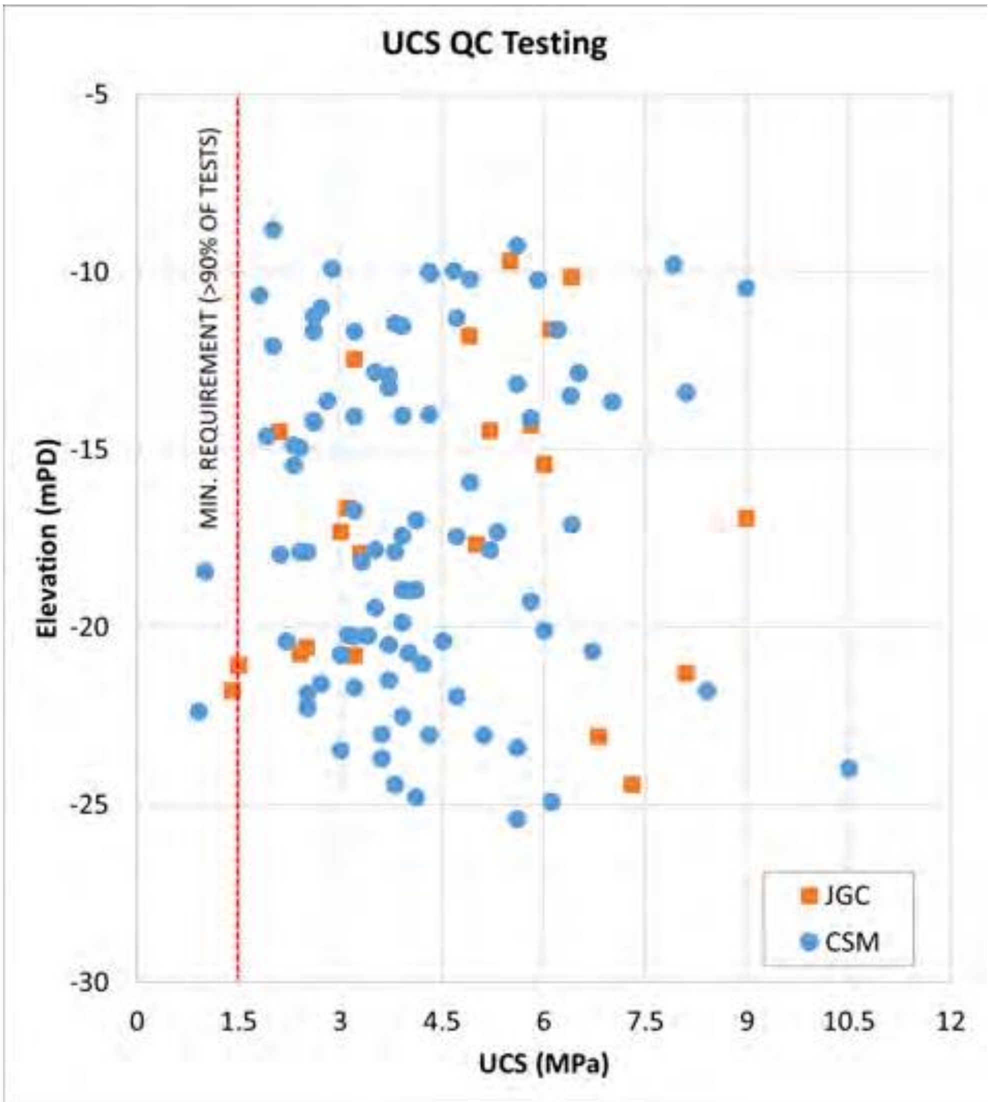

Figure 13: Inclinometer Monitoring

7.3 Foundations Settlement Performance

A total of four building settlement markers were installed at the ground level slab to verify the long-term performance of the raft foundations. The available data is presented in Figure 14. Less than 10 mm settlement were recorded over approximately 18 months between January 2019 and May 2020, with a stable trend of negligible settlement after October 2019. As the building main reinforced concrete works were completed in August 2019, the data indicates that the settlement performance is satisfactory and likely to be within the 190 mm predicted long term settlement over 120 years design life.

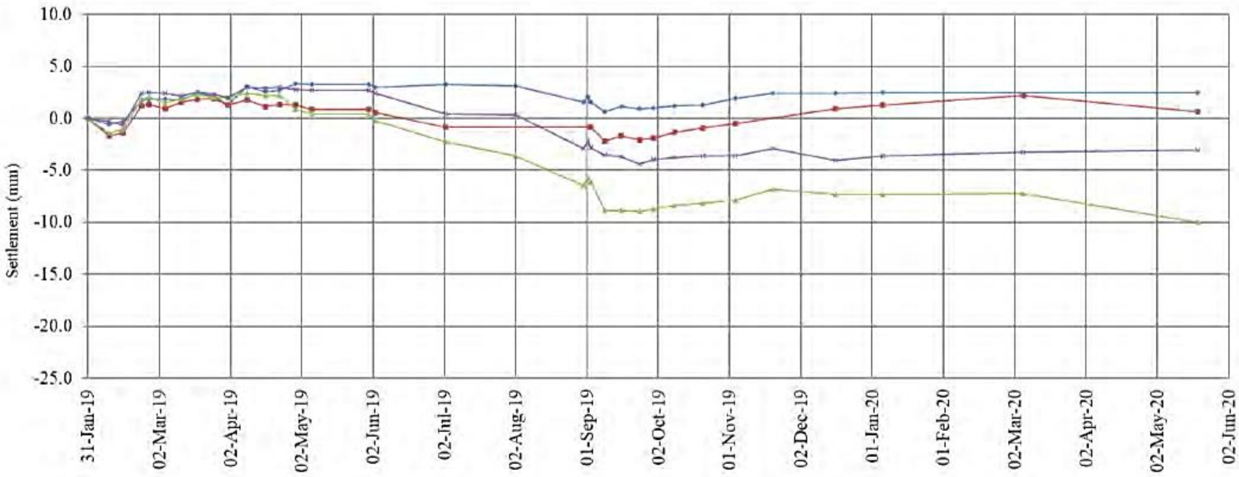

Figure 14: Building Settlement Monitoring

8 Conclusions

The construction of the Southern Ventilation Building provided a challenge due to the presence of a layer of soft clay directly underlying the basement slab, depth of rockhead exceeding 100m below ground level and construction of two large diameter twin tunnels at approximately 20m below the basement. An effective solution was developed to found the building on a raft sitting directly on deep soil mixing ground improvement. This alternative design replaced over 300 shaft grouted piles. In addition, the ground improvement was leveraged to significantly optimise the shoring design for the 15m deep excavation, deleting one level of struts and reducing the size of the retaining wall from combi-wall to FSP V sheet pile. The installation of the ground improvement on the field was successful. The minimum design criteria in terms of unconfined compressive strength have been achieved with more than 90% of the tested samples exceeding the required 1.5 MPa strength with an average exceeding 4 MPa. Very limited lateral movement during excavation and post-construction settlement of the buildings have been measured, indicating that the performance ground improvement scheme has been highly satisfactory.

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