# Singapore Case Histories on Omission of Strut by Observation Approach for Circle Line and Down Town Line Projects

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**ABSTRACT:** This paper will describe in details the issues and challenges involved in the procedures for strut omission by observational approach for two case histories from two different projects – Circle Line Contract C824 and Down Town Line Stage 3 Contract C922. The case history of Overrun Tunnel (ORT) of C922 is basically an underground facility building functions as both Railway Facility (Operation Control Centre) and Electrical Substation (ESS) which is to be built next the Expo Station. ORT is located in old alluvium (OA). The proposed underground overrun tunnel is a box structure with dimensions of approximately 23m wide, 25m deep and approximately 440m long. The proposed diaphragm wall function as the earth retaining system (ERSS), it designed for both temporary loading conditions during excavation and permanent load conditions in accordance with LTA Civil Design Criteria. Bottom-up construction sequence is adopted where lateral supports using four (S3 to S6) or six (S1 to S6) layers of steel strutting were installed as excavation progresses downward. The most challenging part is the omission of the last layer of strut S6 for the whole ORT by using observational approach. The case history of C824 Nicoll Highway Station demonstrates that Jet Mechanical Mixing (JMM), if properly installed, has major benefits in controlling the stability and movements induced by deep excavations in soft ground. The reasons can be attributed to the fact that the inner soil column is comprehensively mixed, combined with the attributes of the outer jet grouted column with sufficient overlapping. The whole process undergoes tight quality control and rigorous testing to ensure a continuous and comprehensive slab. In addition to the JMM slab, there is the major benefit of the discrete soil mixing columns formed above the JMM slab during the withdrawal of the auger. This case history also shows that with observational approach, if used appropriately, the design of temporary works can be effectively streamlined to achieve a more economical and yet safe design, This is illustrated by the approach to omit the intermediate layer of strut in the original design after observing the better than expected performance of the JMM. Based on the limited usage to date it is difficult to suggest what parameters should be used for future design. The approach to the back analyses and forward analyses in the observational approach is presented in this paper. This paper will also discuss the design and construction considerations by focusing on the challenge of strut omission by observation approach. The instrumentation monitoring results will also be presented as evaluation of the performance of the ERSS. The site observation and instrumentation result is in line with the forward analysis prediction for the omission of strut. This proposal has helped to expedite the project with a more economical design. With the implementation of observational approach, we will be able to achieve a more sustainable development of underground infrastructure projects.

**KEYWORDS:** Deep excavation, Strut omission, Back analyses, Forward analyses, Observational approach

## 1. INTRODUCTION

Design of deep excavation support system is usually designed with moderately conservative assumption of various soil parameters and loading conditions. Hence the deep excavation support system will perform well and often too well that may call timely optimization of the design on the subsequent stages of work in order to make deep excavation support system more economical and sustainable. Observational approach is a well-established method that engineers and project managers can rely on to optimize the design just in time to speed up the construction process and reduce construction cost.

Two case histories from two different projects are presented in this paper to illustrate the benefit of optimization of the design just in time to speed up construction process and reduce construction cost by omission of strut due to better than expected performance of the deep exaction support system. This paper will describe in details the issues and challenges involved in the procedures for strut omission by observational approach for two case histories from two different projects – Circle Line Contract C824 and Down Town Line Stage 3 Contract C922.

## 2. CONTRACT C922 OF DOWN TOWN LINE STAGE 3

The proposed Downtown Line Stage 3 (DTL3) will be an underground Mass Rapid Transit (MRT) System extending from Downtown Line Stage 1 (DTL1) Chinatown Station and run through MacPherson, Bedok Reservoir, Tampines and ending at the East West Line Expo Station. Contract 922, Overrun Tunnel (ORT) is part of DTL3. The ORT basically an underground facility building functions as both Railway Facility (Operation Control Centre) and Electrical Substation (ESS) which is to be built next the Expo Station.

The proposed underground overrun tunnel at Changi Business Park is a box structure with dimensions of approximately 23m wide, 25m deep and approximately 440m long. The average existing ground level is about RL106. Both the station and overrun tunnel will be constructed by conventional cut and cover method. The location plan of DTL3 and C922 is given in Figure 1. One of the major challenges in the project is the extremely tight schedule of work. Hence it is crucial to adopt omission of the last layer of strut S6 for the whole ORT by using observational approach in order to ensure that work can be completed within schedule.


Figure 1 Layout plan of DTL3 alignment and location of C922 ORT

## 3. GROUND CONDITIONS

### 3.1 Site Geological Conditions

A brief summary of the general geology of the site is presented in this section. A geological map of the site is shown in Figure 2, which is based on former Public Works Department's 1976 publication entitled 'Geology of the Republic of Singapore'. From this figure and based on the familiarity with the geology of Singapore, the geological conditions of the site could be generalized as fill, Kallang Formation and old alluvium (OA). The geology along the alignment is summarised in Table 1.

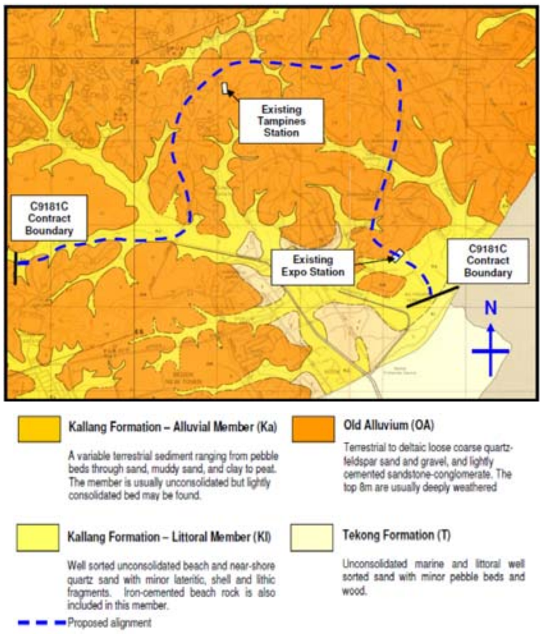


Figure 2 Geological Map of the Site for DTL3

Table 1 Anticipated Site Geology for C922

| Stratigraphic Formation | Time of Deposition | Material |
|---|---|---|
| Reclamation Fill | Recent | Man made |
| Kallang Formation | Recent Alluvium or Quaternary age deposits | Estuarine sediments-transitional member (E)<br>Fluvial sediments-alluvium members, Fluvial Sand (F1), Fluvial Clay (F2) |
| Old Alluvium (OA) | Pleistocene age | Material of Old Alluvium with varying degree of weathering OA(A), OA(B), OA(C), OA(D), OA(E) |

## 3.2 Site Investigation

Site investigation with boreholes location including existing boreholes and additional probe holes as shown in Figure 3 has been carried out to assess the ground variability across the site. A typical soil profile comprising mainly of Fill and OA is encountered along the route as shown in Figure 3. It is anticipated that the ground profile will not vary much along the station and overrun tunnel alignment for C922. Based on the results of the site investigation, the Fill consists of very soft to stiff multi-coloured, slightly gravelly to gravelly, fine to coarse sandy clay/silt and very loose to medium dense silty/clayey, fine to coarse sand. The thickness of the Fill layer ranges from 1m to 9m. The upper few meters of OA is weathered with SPT-N value of range from 5 to 15 at the depth RL105m to RL100m. Pockets of Kallang Formation are observed at the depth RL102 to 98 at the area of ORT (RF). The final formation levels at about RL81 of the tunnels generally cut through OA soil with SPT values more than 40. SPT blows of more than 40 are generally found below RL93m as shown in the Figure 4. According to the additional boreholes which performed at every 12m interval, the OA material at this site generally consist of silty sand.

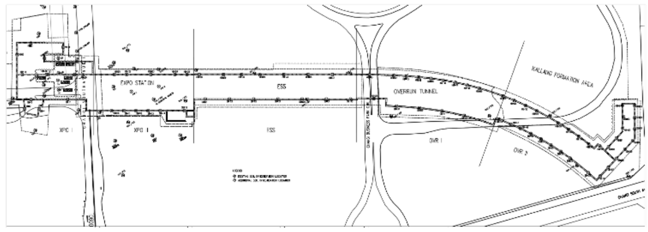

Figure 3 Boreholes Layout

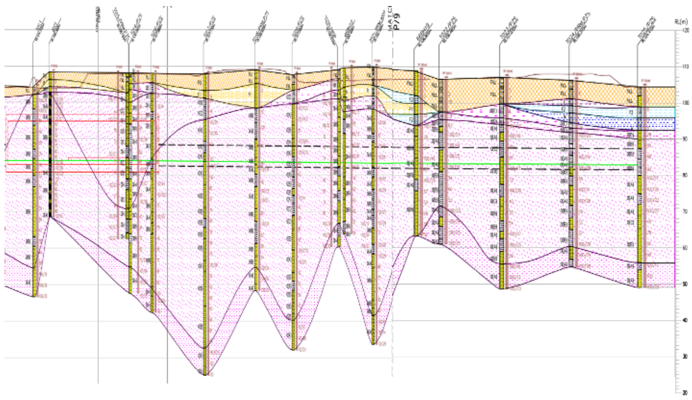

Figure 4 Soil profile based on existing boreholes

## 3.3 Engineering Properties of Old Alluvium

The engineering properties of OA material were comprehensively described by Wong et al. (2001), Chiam et al. (2003) and Chu et al. (2003). Variation of shear strength of OA within same stratum or borehole can be significant as a result of variations in density, particle size distribution and degree of cementation. In particular, cementation has major influence of the strength properties (Chiam et al., 2003). The Old Alluvium layer predominantly consists of cohesionless soils (frequently cemented), includes silty sand and clayey sand layers, as well as cohesive soils consisting of silty clay and sandy clay layers. The colour of the stratum is generally yellowish/reddish brown and light grey to greenish/bluish grey. The relative density and consistency of the soils generally increases with depth with SPT-N value ranging from 5 to more than 100. The old alluvium soil is characterized to five different categories, based on the SPT N values as shown in Table 2.

Table 2 Summary Table of Design Parameters for OA

| OA Layer | Effective friction Angle, Ø' | Effective Cohesion, Cu |
|---|---|---|
| OA(A) ($N \geq 100$)<br>OA(B) ($50 \leq N < 100$) | 34° | 35kPa |
| OA(C) ($30 \leq N < 50$) | 34° | 25kPa |
| OA(D) ($10 \leq N < 30$) | 34° | 20kPa |
| OA(E) ($N < 10$) | 32° | 15kPa |

Table 3 shows a summary of the geotechnical design parameters for the various soil materials encounter for this site that are used in the design of ERSS. The engineering behavior and design parameters of Sandy OA and Clayey OA described in Table 3 can be taken as representative to those described in published literatures as 'cemented' and 'un-cemented' OA, respectively.

Table 3 Summary of soil parameters

| Material | | Thickness (m) | Lowest Elevation (RLm) | Unit Weight (kN/m$^3$) | SPT N-Value | Strength Parameters | | | Undrained Modulus, $E_u$ (MN/m$^2$) | Drained Modulus, E' (MN/m$^2$) | Coefficient of Earth Pressure At-rest, $K_o$ | Permeability (m/s) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Total Stress | Effective Stress | | | | | |
| | | | | | | $S_u$ (kN/m$^2$) | c' (kN/m$^2$) | φ' (°) | | | | |
| 1. Fill | | 0.5~9 | 97.5 | 20 | 0~38 | 30 | 0 | 28 | - | 8.7 | 0.5 | $10^{-7}$ |
| 2. Estuarine Layer | | 2 | 99.2 | 15 | 0~5 | 0.75z + 16.25 ($20 \le S_u \le 35$) | 0 | 15 | 0.2 $S_u$ | Note (3) | 1.0 | $10^{-9}$ |
| 3. Kallang Formation Sand (F1) | | 0~6.8 | 98.4 | 20.5 | 0~16 | - | 0 | 30 | - | 8.7 | 0.7 | $10^{-5}$ |
| 4. Kallang Formation Clay (F2) | | 1.4~3 | 95.9 | 19 | 5~6 | 1.5z + 12.5 ($20 \le S_u \le 50$) | 5 | 25 | 0.2 $S_u$ | Note (3) | 1.0 | $10^{-9}$ |
| 5. Marine Clay Layer | | 3.5 | 92.4 | 15 | 0~6 | 1.28z + 8.6 ($15 \le S_u \le 60$) | 0 | 22 | 0.3 $S_u$ | Note (3) | 1.0 | $10^{-9}$ |
| 6. Clayey Old Alluvium | (a) OA (E) (N<10) | - | - | 20 | - | 5N | 0 | 28 | 1.0 | Note (3) | 0.7$^{(5)}$ 1.0 | $10^{-8}$ |
| | (b) OA (D) (10≤N<30) | | | 20 | | 5N | 5 | 28 | 2N | Note (3) | | $10^{-8}$ |
| | (c) OA (C) (30≤N<50) | | | 21 | | 5N | 10 | 28 | 2N | Note (3) | | $10^{-8}$ |
| | (d) OA (B) (50≤N<100) | | | 21 | | 250 | 10 | 30 | 1.2N+40 | Note (3) | | $10^{-8}$ |
| | (e) OA (A) (N≥100) | | | 21 | | 250 | 20 | 32 | 160 | Note (3) | | $10^{-8}$ |
| 7. Sandy Old Alluvium | (a) OA (E) (N<10) | - | - | 20 | - | 5N | 0 | 30 | 1.0 | Note (3) | 0.7$^{(5)}$ 1.0 | $10^{-7}$ |
| | (b) OA (D) (10≤N<30) | | | 20 | | 5N | 5 | 32 | 2N | Note (3) | | $10^{-7}$ |
| | (c) OA (C) (30≤N<50) | | | 21 | | 5N | 10 | 32 | 2N | Note (3) | | $10^{-7}$ |
| | (d) OA (B) (50≤N<100) | | | 21 | | 3N+100 | 10 | 35 | 1.2N+40 | Note (3) | | $10^{-7}$ |
| | (e) OA (A) (N≥100) | | | 21 | | 400 | 20 | 35 | 160 | Note (3) | | $10^{-7}$ |

Note: (1) N denotes SPT N-values (2) z = depth below ground level (3) Drained Modulus, $E' = 0.87E_u$
(4) For Estuarine Layer, $e_0 = 2.9$; $c_c = 1.5$; $c_r = 0.03\ c_c$; $c_v = 3\ m^2/yr$
(5) For Old Alluvium, a value of 0.7 and 1.0 is to be adopted for temporary work and permanent work respectively.

For both the Clayey and Sandy OA layer, the proposed effective stress parameters generally lay on the lower bound of the results from the triaxial test data. Orihara and Khoo (1998) proposed values for the effective angle of internal friction (φ') and cohesion (c') for a slope design project, using the lower quartile lines of t'-s' plots, which are summarized in Table 3. The proposed effective stress parameters especially the effective cohesions are generally lower than that proposed by Orihara and Khoo. There is no undisturbed sample obtained from borehole due to the difficulties in obtaining undisturbed samples especially in very dense Sandy OA(A).

## 4. PROPOSED CONSTRUCTION METHOD AND INITIAL ERSS DESIGN

### 4.1 Proposed Construction Method

The Overrun Tunnel for this Contract C922 (ORT) was designed as a bottom up construction with a 1.0m diameter thick permanent diaphragm wall anchored into the Old Alluvium strata. The exaction depth is 18m to 25m and the width of excavation is 23m. The embedded depth of diaphragm wall is range 0.42H to 0.5H (H=total excavation depth). Bottom-up construction sequence is adopted where lateral supports using four (S3 to S6) or six (S1 to S6) layers of steel strutting were installed as excavation progresses downward. The typical cross sections of the ORT are shown in Figure 5.

### 4.2 Original Design Assumption

The initial design of ORT was carried out using moderately conservative parameters as per Table 3. The geotechnical analysis of the excavation sequence was performed using finite element program, two-dimensional PlAXIS Version 9.02. non-linear and stress-dependent stress-strain properties of the soil are modeled as elastic perfectly plastic using the Mohr Coulomb model. For the initial analysis and design of the ORT ERSS, two cases of analysis were considered as listed below and designed for the governing case: 1) Drained analysis and 2) Undrained analysis

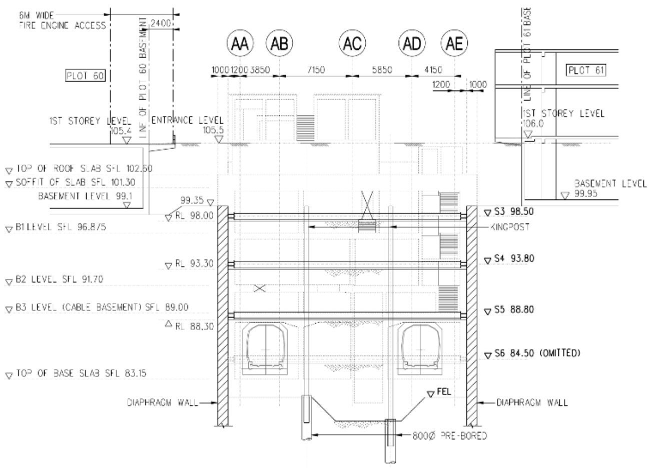


Figure 5 Typical cross sections of the ORT

## 5. BACK AND FORWARD ANALYSES

### 5.1 Observation Approach

The schedule for the completion of this project is very tight and the progress of work is delayed. Therefore it is necessary to carry out the study to omit last layer of strut due to insignificant wall deflection observed from the instruments records. To be successful in today's competitive consultancy service industry, engineers must deliver intelligent design on time to meet the required dateline. The observation approach discussed by Peck (1969) is one of the common practice in Geotechnical Engineering to achieve economical design provided the design can be modified and improved as construction progressed with careful timely review of monitoring results.

When excavation reached to third or fourth layer strut, contractor has approached designer to review and study the possibility of omission of strut S6 based on observational approach. Then back analyses were carried out based on actual site conditions and results compared with the monitoring data. After that, forward analysis carried out to study the feasibility of omitting Strut S6. Back and forward analysis results show that it is possible to omit S6 without affecting current design of installed diaphragm wall and remaining struts.

Observational approach is adopted for the excavation and construction of ESS tunnel to monitor the performance of the ERSS design and study for possibility of optimization of design. In this project, the measured wall deflection is smaller than the predicted wall deflection from the original design. Hence it is an indication that the actual site condition is better than the original design assumptions. There is a possibility that the ERSS design can be reviewed and optimized further based on the better than expected site and ground conditions. The observational approach carried out for this section of ERSS is shown in Figure 6. Similar approach has been successfully implemented in other projects to omit a layer of strut as a result of the better than expected site and ground conditions. One of the examples is the omission of strut S3 in Singapore Circle Line Contract C828 Nicoll Highway Station.

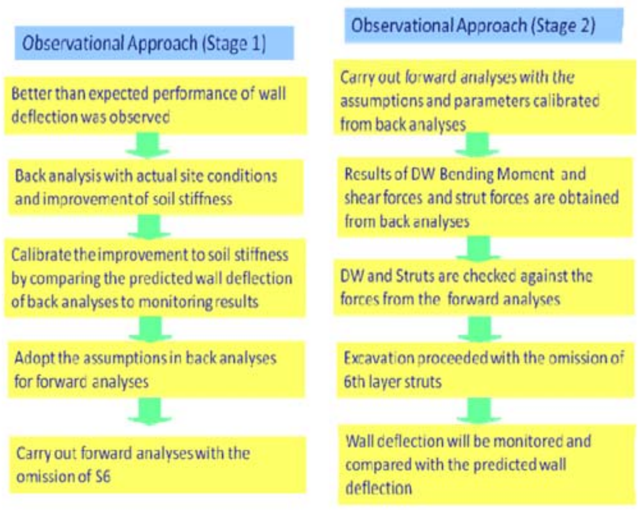


Figure 6 Observational Approach for review and optimization of ERSS design

### 5.2 Assumption for Back and Forward Analyses

The assumptions and parameters for the back analyses have been based on the actual site condition to obtain a closer design prediction to the observation on site. Table 4 shows the summary table of comparisons between parameters in original design and back analyses. Figures 7 are the example model for revised PLAXIS analysis based on the assumption showed in Table 4. The wall deflections comparison for this feasibility study is included in Figure 8.

## 6. INSTRUMENTATION AND PERFORMANCE OF ERSS AFTER OMISSION OF S6 STRUT

An instrumentation monitoring programmed is proposed for ORT ERSS to monitor various parameters that may affect the proposed ERSS during actual work execute. The proposed instrument to access the performance of the excavation works is showed in Figure 9.

The monitored wall deflection are well within the work suspension level (WSL). The maximum recorded ground settlement and wall deflection is 30mm and 20mm, respectively. The recorded wall deflections for excavation of ORT are well within the 0.5% of the wall retained height implying the successful performance of the adopted ERSS system.

Table 4 Summary table of comparisons between parameters in original design and back analyses

| **Parameters** | **Original Design** | **Back Analyses (feasibility Study)** |
|---|---|---|
| Ground Level | RL99.95m | RL99.35m |
| Ground Water Table | GWT at GL (RL99.95m) | GWT at 3m below GL |
| Final Excavation Level | RL79.80m | RL80.81m |
| Soil Profile | Based on original boreholes | Based on additional boreholes |
| Soil Type | Clayey Old Alluvium | Sandy Old Alluvium |
| Soil stiffness | E = 2N (MPa) | E = 3N (MPa) |
| Unplanned Excavation | 0.5m unplanned excavation assumed in the normal case analyses (ULS FOS = 1.4) | 0.5m unplanned excavation assumed in the accidental case analyses(ULS FOS = 1.05) |
| Number of layer of struts | 4 layers of struts (S3, S4, S4 & S6) | 3 layers of struts (S3, S4 & S5) |

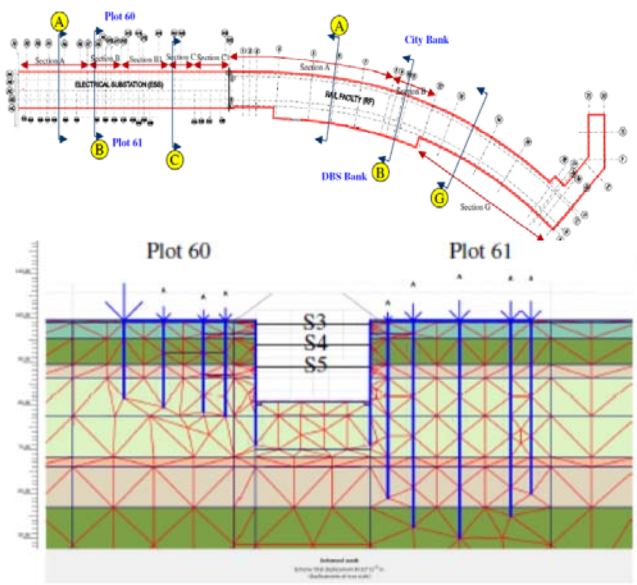


Figure 7 Plan and Section A-A ORT (ESS) used in analysis

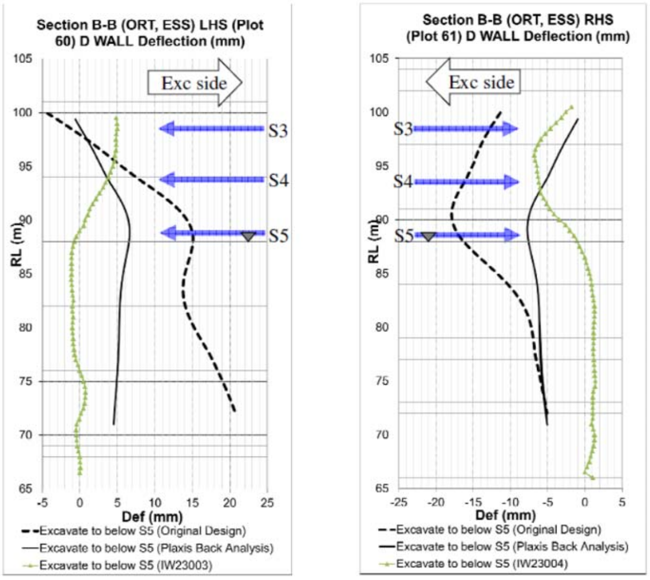


Figure 8 Wall deflection comparisons for feasibility study of strut omission

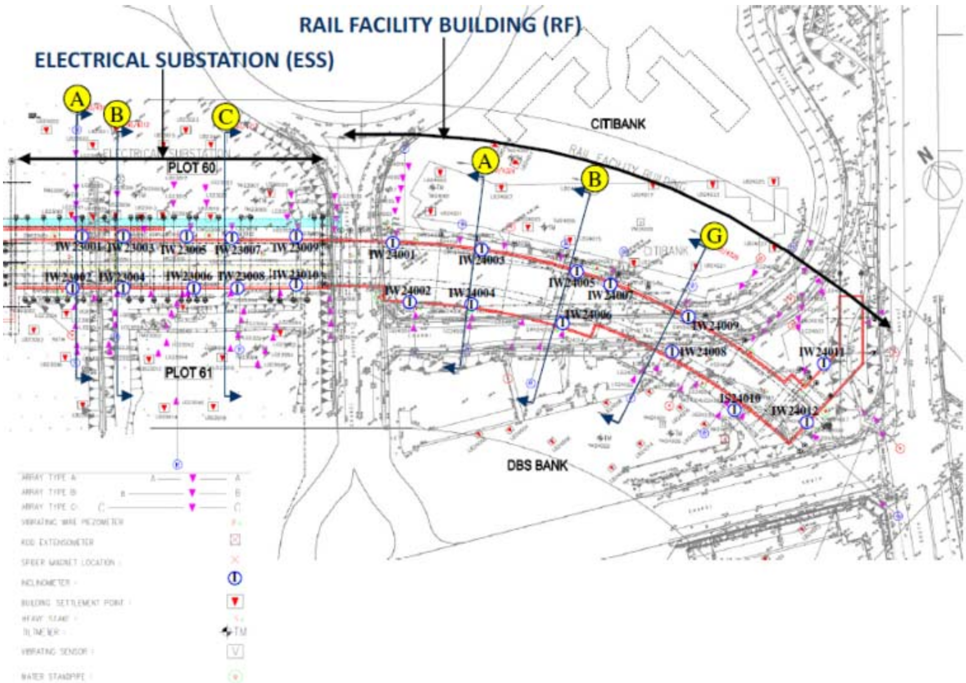


Figure 9 Proposed Instrumentation Layout

## 7. COMPARISON OF ACTUAL PERFORMANCE WITH BACK ANALYSIS USING MOHR COULOMB MODEL DRAINED AND UNDRAINED ANALYSIS

In this project, the total number of struts S6 omitted successfully is 23nos at ORT (ESS) and 37nos at ORT (RF) in the end. Figures 10 and 11 show the comparison between the initial predicted wall deflections using MC drained parameters and wall deflections from back analysis using MC undrained parameter againstt the actual wall defletions measured by in place wall inclinometers at different construction stages. The results show that the wall deflection are well within the allowable limit for the original design of the diaphragm wall.

From these two sections of back analysis, they show that the predicted wall deflection using MC drained parameter has a very significant difference of wall movement as compare to the actual performance. A maximum measured deflection of 10mm compared to a predicted 35mm wall deflection is observed for section A-A, ORT (ESS), as shown in Figure 10. While for section A-A, ORT (RF) the wall lateral wall deflection monitored by inclinometers installed in both side of diaphragm walls are only about 25% of the predicted deflection suing MC drained parameter. Therefore, the finite element using MC drained parameters is found to be conservative approach for OA materials in this case study.

The wall deflection from back analysis using MC undrained parameters is able to reasonably match the reading of the in wall inclinometers at the same construction stage. This suggests that this back analysis study support the use of MC undrained parameters is most appropriate to model the OA soils.

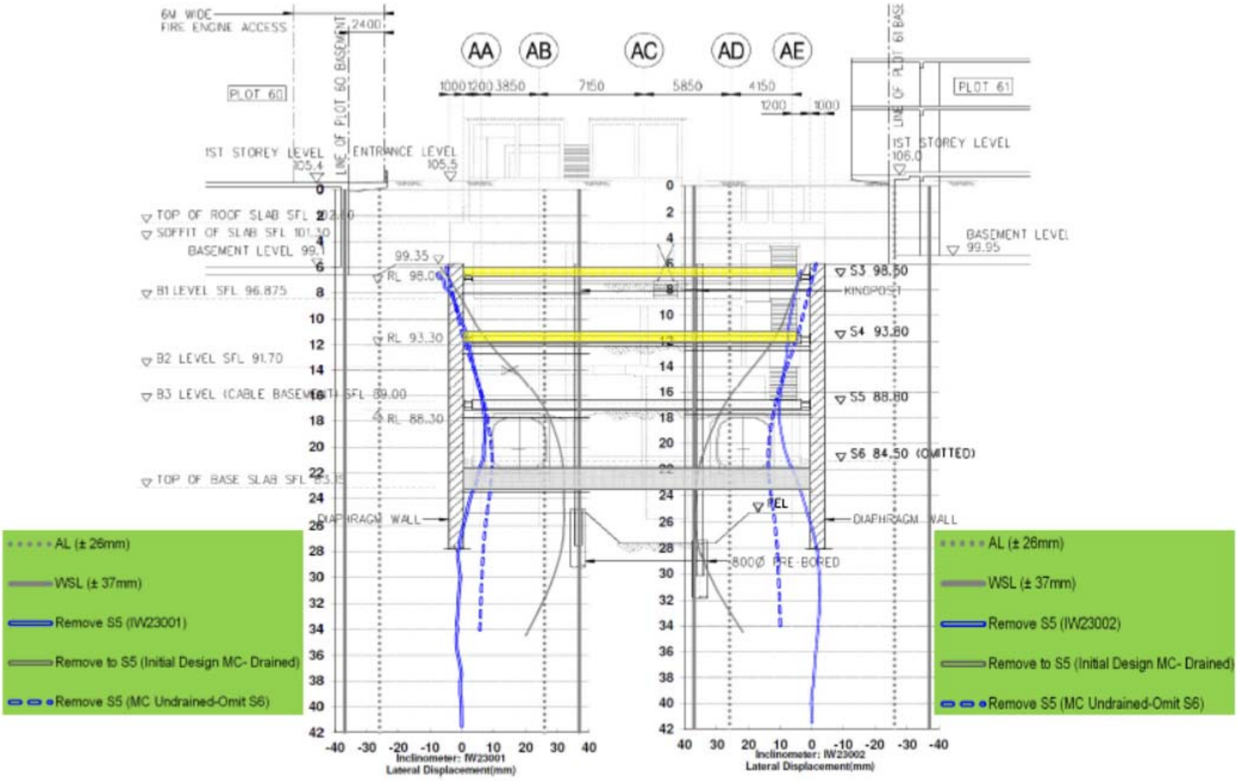


Figure 10 Comparison Wall deflections for the back analyses (Section A-A ORT, ESS)

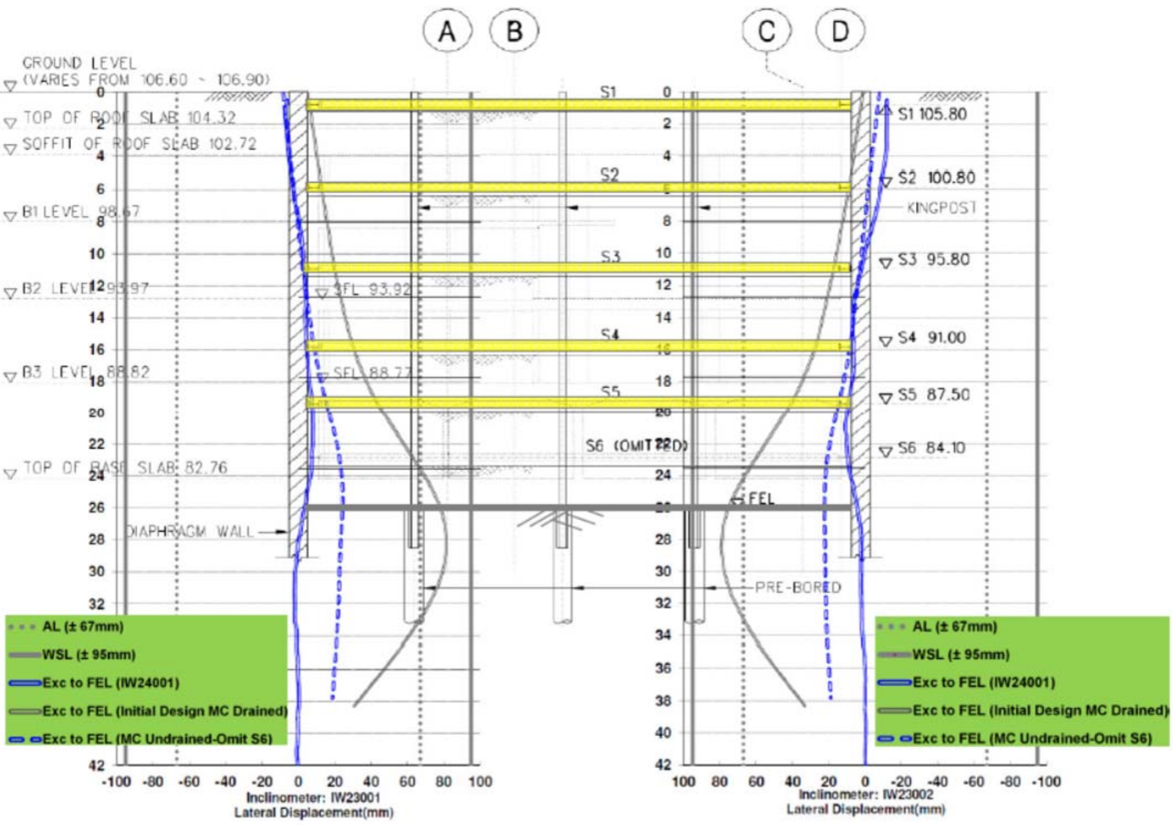


Figure 11 Comparison Wall deflections for the back analyses (Section A-A ORT, RF)

## 8. RECONSTRUCTION OF NICOLL HIGHWAY STATION AFTER NICOLL HIGHWAY COLLAPSE

The construction works for the original Nicoll Highway Station (NCH) on the Circle Line Project (CCLP) was halted when a collapse of the cut and cover tunnels leading to the station occurred in April 2004. Following the collapse, several options were studied for the recommencement of the works. The option to realign part of the project to avoid the collapsed site was eventually adopted. As a consequence of this realignment, NCH was relocated approximately 100m to the south, as shown in Figure 12, with the station design and excavation restarting afresh.

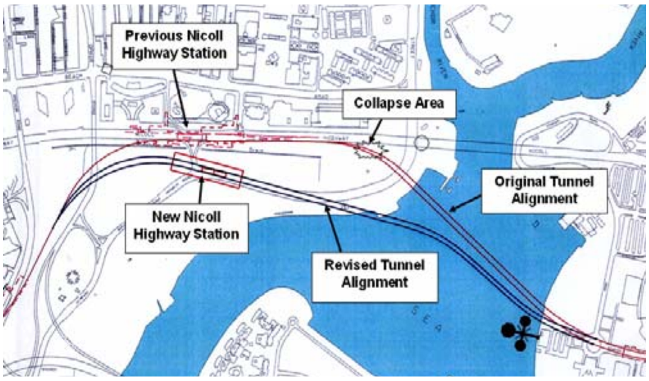


Figure 12 Tunnel Alignment Drawing

The realigned NCH Station and tunnels presented a number of unique challenges to be overcome during the planning, design and construction stages. These included tunnelling through the previously constructed bored tunnels and tunnelling in the soft marine clay, which in some locations was still undergoing consolidation settlement, and the control of ground movement due to deep excavation in the soft Marine Clay.

These potential problems were identified well in advance and the risks were mitigated or managed by putting appropriate solutions in place. These solutions utilized a combination of previous Singapore experience and new techniques, allowing the tunnelling & excavation works to progress as planned. The challenges are described in details in the following sections.

## 9. GROUND CONDITION

At NCH, the ground consists of man-made fill, fluvial sands, fluvial clay and the Marine Clay of the Kallang formation, underlain by the Old Alluvium, as shown in Figure 13. The thickness of the fill is typically 3 to 6 meters. Underlying the fill is a layer of fluvial sand. Beneath this, it is the very soft to soft Marine Clay. The thickness of the sand layer is 3 to 7m. The depth of the Marine Clay varies from 30m to 40m below ground level.

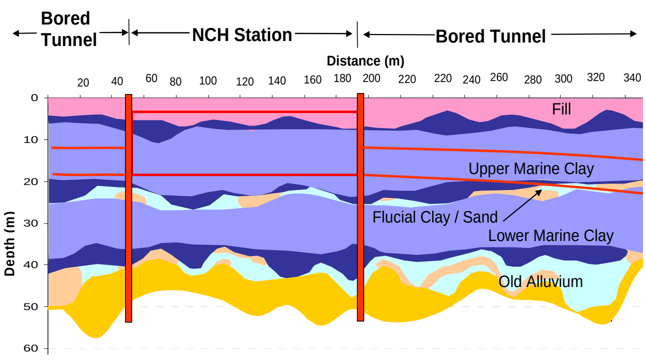


Figure 13 Ground conditions at the new NCH Station

Locally, the Marine Clay is separated by a layer of laterally discontinuous fluvial deposits. The fluvial sands found at NCH are typically described as loose to medium dense gray sands or silty sands. The properties of the fluvial sands are described by Chu, et al (2000). The properties of the Singapore Marine Clay and problems associated with it from a tunnelling and deep excavations perspective have been well established in Singapore; see Tan (1972), Shirlaw & Copsey (1987), Chang (1991) and Tanaka et al (2001), and generally relate to its softness. The Marine Clay is normally consolidated or slightly over-consolidated; with an undrained shear strength (Cu) starting at about 20kPa and increasing slowly with depth. The compression index is typically in the range of 0.6 to 1.0. The permeability is low and is in the order of $10^{-9}$ to $10^{-10}$ m/s. The Old Alluvium is typically described as sandy silt or clayey silt. At depth the material is generally found to have some cementation. However much of the cementation has been lost due to weathering at shallow depth. The permeability of the Old Alluvium depends on weathering and grain size distribution. It typically ranges between $10^{-6}$ to $10^{-9}$ m/s.

## 10. HYBRID TYPE GROUND IMPROVEMENT FOR EXCAVATION IN SOFT MARINE CLAY

Ground treatment underneath the base of the station is often used to limit the wall deflection, act as a working platform and prevent uplifting of these soft clayey soils. The application of ground treatment such as jet grout piles (JGP) for deep excavations in Marine Clay in Singapore has been presented by Page et al. (2006). For this project, the ground treatment option was Jet Mechanical Mixing (JMM), a hybrid of jet grouting and deep soil mixing. A proprietary name, RASJET is given to it by the specialist contractor from Japan, Raito Kogyo. This was the first time such a system had been used in large scale in Singapore.

JMM is a combination of soil mixing and jet grouting that produces overlapping columns with an internal column of mixed soil by the auger and an external column created by a slurry jet into the in-situ soil. The process of forming the columns is similar to the method of forming JGP columns with the addition of dual and counter rotation mixing blades on the drill rod to ensure intensive soil mixing. Figure 14 show the JMM machines, the drilling rod and the mixing arm of the machine. The rod/auger had a large diameter of 457mm as compared to the traditional JGP rod of 200mm. The high stiffness of the drill rod contributes to a more accurate drilling verticality. Combined with the rod were the mixing blades which created an inner mechanical soil mixing column of 1.6m diameter. A jet grout nozzle on the mixing blade introduces cement slurry mix with pressurized air into the soil and adds a further 0.6m of jet grouting around the soil mixing column, creating a 2.8m column within the ground. These columns are then designed with appropriate overlap to provide a full coverage of the treated areas.

Figure 14(a) JMM machines

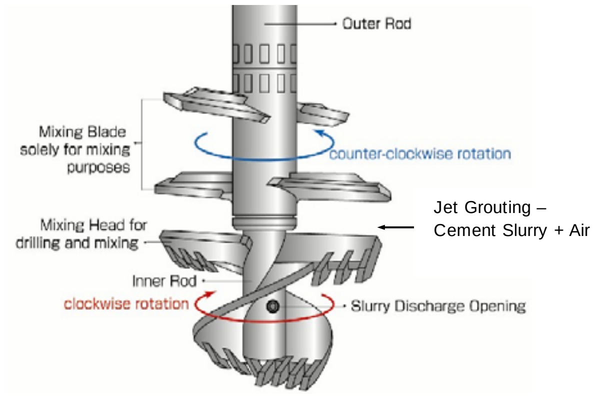


Figure 14(b) Schematic diagram of the drilling rod

Figure 15 shows the layouts of the mechanical mixing part and jet grouting part of the JMM column. There are numerous advantages to this system (Page et al 2006 & Ueda et al 2007), principally the benefits of mixing and grouting are experienced. From the mechanical soil mixing, a known treated area is assured and from the jet grouting, a sizeable overlap and penetration into any shadow areas close to the retaining system is achieved.

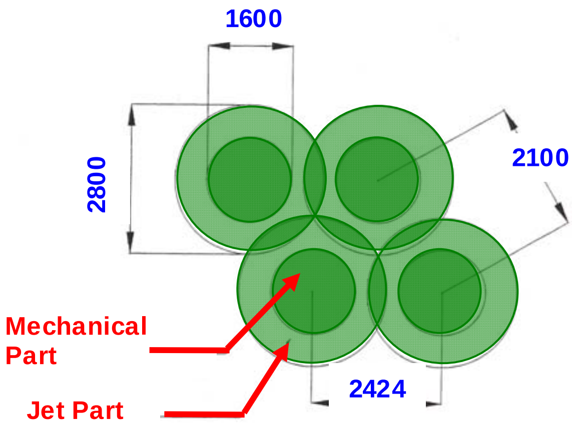


Figure 15 Typical layouts of the JMM columns

To install a JMM column, the auger is first drilled to the base level of the JMM column with water injection, and withdrawn to the top level of the JMM column with mechanical mixing without any injection. It then descends with slurry injection and mechanical mixing to form the internal soil mixing column up to base level. After which, it ascends with jetting to form the external jet grouting perimeter. The whole process is automated and monitored real time by data loggers to ensure that a high level of quality control. It should be noted that a further benefit is gained during the withdrawal the auger after completing the JMM slab. As the auger is withdrawn, lower quantities of cement are added and the ground is mixed. This creates a 1.6m diameter treated column all the way to the surface, as shown in Figure 16. Consequently the strength of the soil above the treated ground is significantly enhanced. There were comprehensive quality control procedures implemented during the construction of the JMM columns, as described by Ueda et al (2007).


Figure 16 Columns created during withdrawal

## 11. ACTUAL STRENGTH PARAMETERS FOR JMM AND SOIL CEMENT MIX ABOVE

### 11.1 JMM Strength and Parameters

Post installation and prior to the commencement of excavation, an extensive quality check on the strength parameters of the JMM layer as well as the soil cement mix above was carried out. Figure 17 shows the summary of the test results from the 7m thick JMM layer directly beneath the base slab. It shows the comparison between the actual average strength of the JMM layer and the parameters assumed in the original design. Also included is a strength factored by a mass correction factor of 0.725 to account for any variation caused during the construction process. The average strength, Cu, of the JMM layer is about 1845kPa, with very consistent strengths achieved in the samples tested, ranging from a lowest of 1150kPa to the highest of 2370kPa.

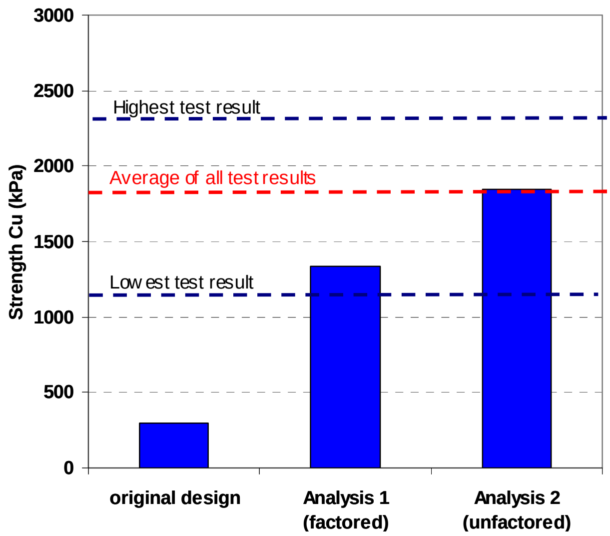


Figure 17 Comparison of JMM strength used for initial design and actual values from test data

The average strength is more than five times higher than that originally assumed value of 300kPa in the design. Similar findings are experienced on the results of the stiffness testing, Figure 18 shows the comparison of the stiffness of JMM from test results, the original design value and the factored value taking into account the mass correction factor. The average stiffness of the JMM from test results is 572MPa, with a lower bound of 400MPa and an upper bound of 700MPa. Again the tested average stiffness is significantly higher than the 90MPa assumed in the original design. The original design parameters are adopted from commonly accepted values as outlined by Page et al (2006).

## 12. SOIL CEMENT MIX STRENGTH PARAMETERS

The test results for the soil cement mix above the JMM show that the average strengths, Cu are 178kPa for the upper layer (94.6m < RL < 102.5m) and 339kPa for lower layer (81.3 < RL < 94.6). The average stiffness of the soil cement mix from test results are 24MPa for the upper layer and 44MPa for lower layer. These average strength and stiffness parameters are significantly higher than those assumed in the original design. The various strength of the treated ground with JMM and soil cement mix above the JMM is summarized in Figure 19 and Table 5.

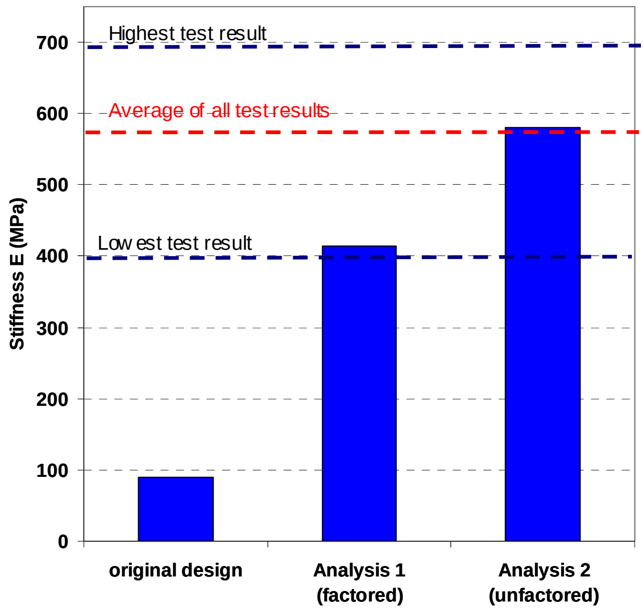


Figure 18 Comparison of JMM stiffness used for initial design and actual values from test data

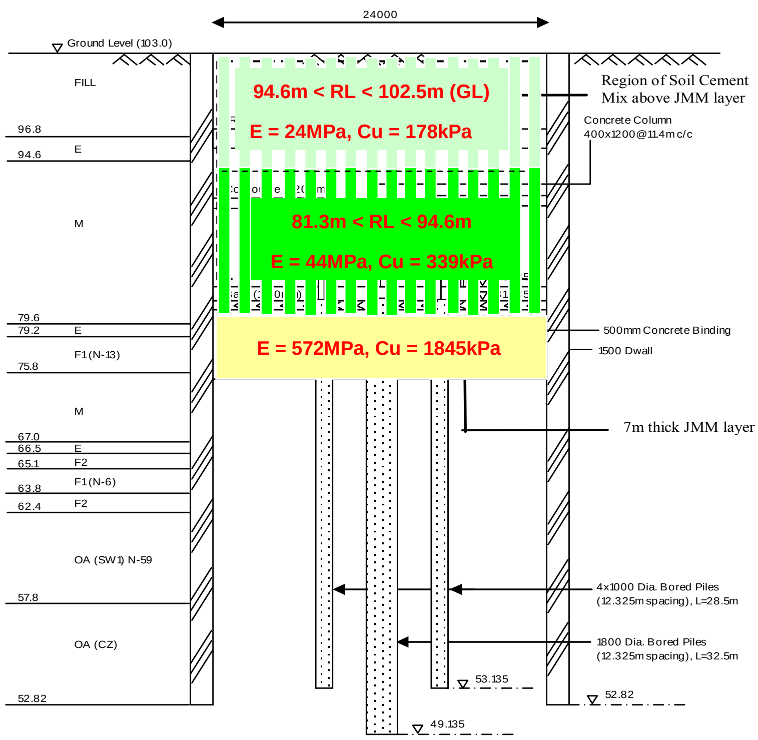


Figure 19 Average parameters of JMM and Soil Cement Mixing above the JMM

Table 5 Most probable JMM parameters for back analysis

| Most probable JMM parameters (Correction Factor = 0.725) | | Most probable parameters for soil cement mix above the JMM (Correction Factor = 0.725) | |
|---|---|---|---|
| | | 94.6m<RL<102.5m(GL) | 81.3m<RL<94.6m |
| Cu' (kPa) | 0.725×1845 = 1337.6 | 0.725 × 178 = 129 | 0.725 × 339 = 245.8 |
| $E_{50}$ (MPa) | 0.725 × 572.1 = 414.8 | 0.725 × 24 = 18 | 0.725 × 44 = 31.9 |

## 13. INTERPRETATION OF BACK ANALYSIS RESULTS

The results from the various back analyses are summarized in Figure 20 The wall deflection from the back analysis (Analysis 1) using the most probable parameters at second stage excavation, with the mass correction, is able to reasonably match the reading of the in-wall inclinometers at the same stage. This is supported by the fact that the reading of the first level strut force is quite similar to that obtained from the back analysis. However, the wall deflection obtained from the Analysis 2 using unfactored parameters gives an even much closer match to the actual wall movements. This suggests that the quality, strength and stiffness of the JMM are consistent throughout the entire JMM slab and correction factor may not be needed to be applied. These most probable parameters of JMM, with the mass correction factor, were used in the analysis to predict the performance of the temporary works in the subsequent stages of excavation with the omission of the third layer of struts originally proposed between the concourse and the base slab.

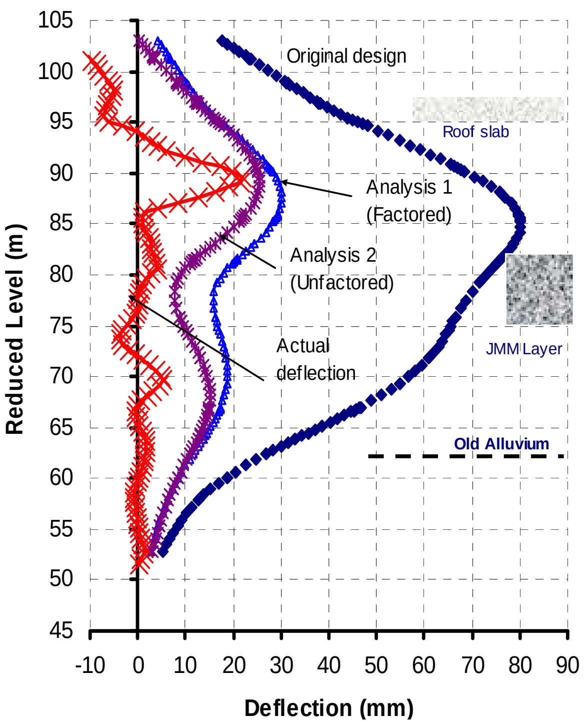


Figure 20 Comparison of measured and predicted wall deflection by the back analyses up to final stage of excavation

The analysis results also show that the bending moment of the walls was within the capacity, the strut forces in the first and second layers struts were smaller than the original design and the locked-in stresses in the permanent walls, roof slab, concourse slab and base slab were smaller as compared to the original design. Hence, the excavation proceeded with the omission of third layer strut. The measured wall deflection by inclinometer for the subsequent stages of excavation was found to be close to the predicted wall deflection.

## 14. CONCLUSION

In conclusions, OA soil is cemented and its engineering properties are complex and more effort shall be put in to understand and obtain better parameters for design purpose. Based on the site observation of C922 and C937B, the actual wall deflection is only about 30% of the predicted wall deflection using MC drained parameters. Hence back analysis was carried out to optimize the design. Various approaches have been performed to try to model the excavation in PLAXIS 2D to obtain the actual site performance. OA material using Mohr Coulomb drained analysis would be quite conservative. Backanalysis supports the use of Mohr Coulomb undrained analysis is more realistic to predict the OA soil behavior. The recorded wall deflections for excavation of ORT are well within the 0.5% of the wall retained height which implied that the performance of the ERSS system is much better than prediction. Generally the computed predictions from back analysis were in reasonable agreement with the measured results.

The collapse of Nicoll Highway, coming only months before the World Tunnelling Congress in Singapore, served as a reminder to all those involved in underground construction both in Singapore and throughout the world of the hazards and challenges in deep excavations. The resulting recommendations from the COI will reduce the risks and likelihood of a reoccurrence of such an incident in Singapore. The revised construction methods utilised in the re-aligned tunnel and station works are both challenging and appropriate for the very soft ground conditions.

This case history demonstrates that JMM, if properly installed, has major benefits in controlling the stability and movements induced by deep excavations in soft ground. The reasons can be attributed to the fact that the inner soil column is comprehensively mixed, combined with the attributes of the outer jet grouted column with sufficient overlapping. The whole process undergoes tight quality control and rigorous testing to ensure a continuous and comprehensive slab. In addition to the JMM slab, there is the major benefit of the discrete soil mixing columns formed above the JMM slab during the withdrawal of the auger.

This case history also shows that with observational approach, if used appropriately, the design of temporary works can be effectively streamlined to achieve a more economical and yet safe design, This is illustrated by the approach to omit the intermediate layer of strut in the original design after observing the better than expected performance of the JMM. Based on the limited usage to date it is difficult to suggest what parameters should be used for future design. However it is clear that the key is in the quality control of the process in ensuring a total and uniform treatment. With today's engineering sophistication, this can be achieved. The strength, stiffness and quality of the JMM are significantly higher than those of jet grouting, but the choice of actual design parameters to be used required careful consideration. It is recommended that they should be determined on a case by case basis with local trials specific to the ground conditions, but considering the strengths and stiffness already achieved in past projects in similar ground conditions. The benefits of JMM should not be ignored and this technique will be a future benefit to the industry in controlling ground movements.

It is important and beneficial to implement a comprehensive instrumentation and monitoring program to the excavation projects as this will allow the contractor and designer to have adequate and sufficient information in a timely manner to optimize the design by observational approach. Effective and sufficient instruments monitoring scheme with early warning features allows the engineer to gather information so that the timely review on the performance of the proposed system can be performed and the design modification or improvement deviated from original design can be timely carried out whenever necessary for assurance of safety. It is strongly recommended that observational approach to optimize the design should be encouraged for excavation site especially for sites with geological formation of OA formation.

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