Design Challenges and Performance Evaluation of Temporary Removable Ground Anchor in Old Alluvium – A Singapore Case History

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Abstract
Contract 937B, design and construction of Tai Seng Facility Building (TSFB), is part of Downtown Line Stage 3 (DTL3) of the Singapore Mass Rapid Transit System. The proposed TSFB will provide maintenance, operation and staff facilities for DTL3. It is a 52m wide and 295m long underground building which consists of two basement levels and entrance structure at ground level. The depth of excavation for the construction of basement is approximately 20m. Contiguous Bored Pile (CBP) wall retaining system supported by temporary removable ground anchorage and a top-down roof slab is proposed as the alternative design of Earth Retaining and Stabilizing System (ERSS) for the excavation and construction of the underground TSFB. Partial top-down construction method is adopted where the roof slab with access opening will be cast first. Excavation will proceed and ground anchors will be installed as excavation progress downward. A total of 534 numbers of ground anchors are proposed to be installed on site and 16% of all ground anchors are monitored for load using load cells. The monitoring results for installed ground anchors were well within the work suspension level (WSL). This paper presents the design challenges and performance evaluation for the temporary removable ground anchor. Some site monitoring data compared against design prediction is discussed in this paper to evaluate the effectiveness of the ground anchor design for the ERSS of this project. This paper also discusses the behaviour of the ground anchor and the practicality of the recommendation of BS8081 and TR26:2010 in terms of lock-off load for the ground anchor.

1. Introduction
The proposed DTL3 will be an underground Mass Rapid Transit (MRT) System extending from Downtown Line Stage 1 (DTL1) Chinatown Station running through MacPherson, Bedok Reservoir, Tampines and ending at the East West Line Expo Station. Contract 937B, design and construction of TSFB is part of DTL3. The proposed TSFB will provide maintenance, operation and staff facilities for DTL3.
TSFB is a 52m wide and 295m long underground building which consists of two basement levels and entrance structure at ground level. The locality of DTL3 and C937B is given in Figure 1. This design and build contract has been constructed with cut and cover construction method with a maximum excavation depth of about 20m. The site works of this project commenced on November 2010 and target to be completed by March 2013. One of the major challenges in the project is the extremely tight schedule of work. Hence it is crucial to optimize the design in order to ensure that work can be completed within schedule.

2. Ground Condition

The ground conditions, i.e. Kallang Formation and Old Alluvium (OA) Formation, encountered at the site are generally in line with those anticipated from available published Defence, Science and Technology Agency (DSTA), Geological Map of Singapore, 2009.

The Geological survey by PWD (1976) subdivides Kallang Formation into five members namely, Marine, Alluvial, Transitional, Littoral and Reef. The Kallang Formation commonly encountered are Estuarine, Fluvial Sand, Fluvial Clay and Marine Clay. The OA consists of alluvial deposit that has been variably cemented, often to the extent that it has the strength of a very weak or weak rock. The upper zone of the OA has typically been affected by weathering and has typically penetrated as a discernible front from the surface. All five classes of weathering classification of the OA according to BS5930:1999 are encountered at this site.
Site investigation with boreholes location as shown in Figure 2 has been carried out for both sides of the walls 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. The interpretative subsurface profiles have four soil layers, namely Fill, Kallang Formation, OA (N<50) and OA (N>50). The soil layer boundaries were interpolated in between two boreholes to develop the subsurface profile. The subsurface profiles indicate that Fill and OA layers consistently exist along the alignment but in varying thickness. The surface of the OA is encountered between +96mRL and +109mRL which is above formation of the TSFB. The material below the formation level is expected to be Class A, B or C of OA. Pockets of Kallang Formation are shown to be present in at least two locations. The upper few meters of OA is weathered with SPT-N value of less than 50. The lower OA material has SPT-N of more than 50. The engineering properties of OA material were comprehensively described by Wong et al. (2001), Chiam et al. (2003) and Chu et al. (2003). Table 1 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.

From the soil investigation report, the variability of the OA material is apparent in the particle size distribution curves for the various OA of different degree of weathering and depth. The clayey OA has a very wide range of particle size distribution (PSD) curves and the data reveal that the material ranges between 40% and 80% fines. The sandy OA has a tighter band of PSD curves, showing the material to be silty or clayey sand with a clay/silt content of about 20%.

Table 1  Summary of soil parameters

<table>
<thead>
<tr>
<th>No.</th>
<th>Material Name</th>
<th>Type</th>
<th>Unit Weight</th>
<th>Effective Angle of Friction</th>
<th>Effective Cohesion</th>
<th>Undrained Shear Strength</th>
<th>YIELD IMPRACTICABLE</th>
<th>YIELD RATIONAL</th>
<th>Ks</th>
<th>Kt</th>
<th>n</th>
<th>Eref</th>
<th>cref</th>
<th>phi</th>
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<tr>
<td>0</td>
<td>Backfill</td>
<td>Drained</td>
<td>12.4</td>
<td>0.00866</td>
<td>0.086466</td>
<td>0.33</td>
<td>8700</td>
<td>0.2</td>
<td>30</td>
<td></td>
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<td></td>
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<tr>
<td>1</td>
<td>TSF-Fill</td>
<td>Drained</td>
<td>12.4</td>
<td>0.008649</td>
<td>0.08649</td>
<td>0.33</td>
<td>8333</td>
<td>0.2</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>2</td>
<td>TSF-OA(D)-SPT~10</td>
<td>Drained</td>
<td>12.4</td>
<td>0.00864</td>
<td>0.0864</td>
<td>0.33</td>
<td>13333.3</td>
<td>5</td>
<td>32</td>
<td></td>
<td></td>
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<td>0.08655</td>
<td>0.33</td>
<td>22700</td>
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<tr>
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<td>Drained</td>
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<td>0.00865</td>
<td>0.0865</td>
<td>0.33</td>
<td>50000</td>
<td>10</td>
<td>34</td>
<td></td>
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<td>5</td>
<td>TSF-OA(C)-SPT~40</td>
<td>Drained</td>
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<td>0.00865</td>
<td>0.0865</td>
<td>0.33</td>
<td>66700</td>
<td>10</td>
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<td>6</td>
<td>TSF-OA(B)-SPT~60</td>
<td>Drained</td>
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<td>0.00865</td>
<td>0.0865</td>
<td>0.33</td>
<td>100000</td>
<td>15</td>
<td>35</td>
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<td>7</td>
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<td>Drained</td>
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<td>0.00865</td>
<td>0.0865</td>
<td>0.33</td>
<td>125000</td>
<td>15</td>
<td>35</td>
<td></td>
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<td></td>
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<tr>
<td>8</td>
<td>TSF-OA(A)-SPT~100</td>
<td>Drained</td>
<td>12.4</td>
<td>0.00865</td>
<td>0.0865</td>
<td>0.33</td>
<td>167000</td>
<td>20</td>
<td>35</td>
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</table>
The input soil parameters for Finite Element Modelling (FEM) for each layer of soil are tabulated in Table 2. The input soil parameters for OA is based on the SPT-N value of the OA. Figure 4 shows a plot of SPT-N value for few boreholes located at section A1-A1.

**Figure 4** Plot of SPT-N value of OA material against depth for section A1-A1

### 3. ERSS Design and Proposed Construction Method

The original base design of the ERSS was bottom up construction method with steel strutting support. The contractor has won the contract with the currently adopted alternative design using removable ground anchor and partial-top down construction method which is more efficient. The schedule for completion of this project is very tight and therefore it is necessary to optimize the design of ERSS. CBP wall retaining system supported by temporary removable ground anchorage and a top-down roof slab is proposed as the alternative ERSS scheme for the excavation and construction of the underground TSFB. The gaps between the bored piles were sealed by the jet grout piles (JGP) up to moderately stiff soil where the SPT-N value is about 30. The subsequent gaps was sealed by 150mm thick grade 30 shotcrete to prevent water ingress into excavation area.

Partial top-down construction method is adopted where the roof slab with access opening will be cast first before excavation below the roof slab. Excavation will proceed and ground anchors will be installed as excavation progress downward. The use of partial top-down construction method and incorporating the roof slab as part of the support system to the excavation help to reduce one layer of temporary support system and hence cut down on the construction period.

Figure 5 shows the layout of the ERSS. One major constraint to the ERSS design is that the excavation is in close proximity to the existing viaduct piers of Bartley Flyover as shown in Figure 6. Partial top-down construction method is provided between grids 3 to 16. Figure 7 shows a typical section of the ERSS with partial top-down construction. The roof slab will be connected to the CBP wall and supported by bored pile during temporary stage. Once the excavation reaches the formation level, base slab will be cast. Walls, columns and basement 1 structure will be cast from bottom up.
Figure 5  Layouts of ERSS for C937B

Figure 6  Photo showing existing viaduct in close proximity to the excavation.

Figure 7  Typical section of ERSS using partial top-down construction method with ground anchors for Section A1-A1
4. Finite Element Analysis

Numerical modeling was carried out for the design of the earth retaining structure and structural elements for the underground station. In this project, the behaviour of the ERSS was analyzed using geotechnical finite element software PLAXIS Version 9.02, software commonly used by geotechnical engineers in Singapore.

Figure 8  Typical model of the Plaxis analysis for Section A1-A1

Figure 9  Result of numerical analysis for wall bending moment
The soil behavior was modeled using Mohr- Coulomb Model. Tension cut-off option was activated to cut off all possible developed tension points in the soil elements. The CBP wall was modeled as beam elements with the corresponding axial and bending stiffness. The ground anchor was modeled as fixed end anchor in the PLAXIS model. The stiffness of the fix end anchor is calculated based on the design force per ground anchor divided by the total length and the spacing of ground anchor. Figure 8 show the typical finite element model of the PLAXIS analyses. The results of analyses for wall bending moment and deflection are shown in Figures 9 and 10, respectively. The groundwater level contour is generated by SURFER based on the readings measured from boreholes and water standpipes. Groundwater level prior to construction was between 1m to 2m below ground surface. The adopted design groundwater level are divided into three zones as shown in Figure 11.

In this project, the wall deflection for south wall is limited to 50mm as required by the contractual requirement of Land Transport Authority (LTA). Thus aims to limit movement of viaduct structure as shown on plan in Figure 5 and photo in Figure 6. The allowable deflection for north wall is 140mm or 0.7% of total excavation depth as required by Building and Construction Authority (BCA).
5. Design Approach and Consideration for Temporary Removable Ground Anchor

Ground anchors have been commonly used in Singapore as temporary and permanent support system for retaining walls. The materials, design and execution have developed and evolved to be consistent with international practice. As the application of ground anchors increases, the necessity for more rational guidelines on the design, construction, testing and monitoring of anchors will be required. Since early 1980’s ground anchor codes of practice and published guidelines such as BS 8081: 1989, CIRIA: 580, BS EN 1537: 2000, BS EN 1997-1: 2004 and Eurocode 7 have emerged throughout the world. In Singapore, Technical Reference, TR26:2010 has been published with the aim to develop a more standardized design and construction approach applicable to local condition on ground anchor works.

There are a number of methods that used in Singapore for removable temporary ground anchors, namely Dummy Strand, Explosive, Water Jet Cutting, Coupling, U-turn (Chua and Lai, 1994 and Chua and Prasanthee, 1997) and single bored multiple anchor (SBMA) (Chua, 2003). Figure 12 shows the typical details of the temporary removable ground anchor. This ground anchor system is a removable ground anchor tie back system using the U-turn system. Figure 13 shows the typical strand and anchor block for the ground anchor U-turn system.

U-turn anchors use sheathed strands that loop around a cast iron U-turn anchor holding piece. The holding piece serves as the medium to transfer load from the strands to the grout then to the soil. Figure 14 shows the schematic diagram of main components of U-Turn ground anchors and forces in free-body. The wire strands can finally be removed by pulling one end of the loop as shown in Figure 15. The strands will then slide inside the sheath through the loop until the whole wire strand is removed.

The ground anchor design and testing is according to TR 26:2010, BS 8081:1989 and CIRIA Report 580 (Gaba et al., 2003). Each anchor was tested to prevent over-stressing of the strands by measuring the load increments and strand displacements adhering to the proposed load cycle as per TR 26:2010 Section 5 and BS 8081:1989. All tendons were stressed at one end of the anchor head to the test load of 125% of working load (WL) and lock-off at 110% of the preload value, which was about 70% of WL of the ground anchors, based on the Plaxis analysis results. This is in accordance with TR 26:2010 Clause 5.11 which states that the preloading values of the ground anchors should be based on the design values of preload assumed in the ERSS analysis by finite element modelling and the anchor lock-off load should be increased by 10% of the design preload value to allow for losses. A similar rationale for adopting about 70% of WL can also be found in Chua et al (2006). This is a deviation from BS8081:1989 Clause 11.4 which states that each anchor shall be lock-off at 110% of the anchor’s WL. We reckon that in this aspect, TR 26:2010 is more appropriate as the lock-off loads or preloading values of the ground anchors are obtained from the finite element analyses. This allows some room of increment for the ground anchor forces from the preload values to the design values. The design values are usually set as the WSL a review level lodged to BCA for stopping work if the level is breached.

Two trials were carried out for the removable ground anchors. Both trial ground anchor was tested up to 150% of WL and lock-off at 125% for a period of 24 hours. The allowable elongation for the trial ground anchor test is 90% to 110% of the theoretical elongation.

After completion of construction and backfilling has reached approximately 0.5m below the ground anchor level, the strands were de-stressed and removed. For extraction of the strands, the tendon is initially de-stressed with the help of a mono-jack or first cut one end of the strand behind the bracket and then pulled in the other end.
Figure 12  Photo of typical ground anchor strand and U-turn anchor block

Figure 13  Typical section and details of the removable ground anchor
The strands of the ground anchor using U-turn system can have different lengths from the anchor block to the anchor head where prestressing is applied. The load transfer mechanism of the applied load is therefore more complicated. Steps have to be taken to ensure that applied load is distributed evenly to each holding piece which is the U-turn anchor block. This can be achieved by taking care of the initial differential elongation of each pair of strands. Each pair of strands has to be stressed sequentially, from the longest to the shortest, to overcome the differential elongation between them so that they can be stressed together and acts as a group. The elongation for the shortest pair is the baseline against which the remaining pair must be altered to suit. In this project, the total length of ground anchors ranges from 21m to 42m with the design capacity range from 500kN up to 1200kN. The strand diameter is 12.7mm and is grouted by 250mm diameter grout. There can be up to 7 loops of strands per ground anchor. Figures 16 and 17 show the comparison of bond stress distribution between conventional ground anchors and U-turn ground anchors.
The ground anchors were designed to satisfy both structural and geotechnical requirement based on the design load from the analysis. The temporary removable ground anchor, usually with it strands in loops system, will be removed after the service life. The bending of the strand at the end of such loop will result in reduction of strength of the strand. Hence the design of calculating the number of strands required need to consider the reduction factor to bending of strand. Based on Table 3 from TR26:2010, structural factor of safety, $F_s = 1.6$ and reduction factor due to bending strand, $R_d = 0.8$ were adopted for design of temporary removable ground anchors.

The number of strands can be calculated using the equation below, Equation 5c from TR26:2010:

$$N = \frac{(F_s \times WL)}{UTS \times R_d}$$  \hspace{1cm} (1)

where $N$ = Number of strand;

$F_s$ = Factor of safety, structural (see Table 3);

$UTS$= Ultimate tensile strength of strand, kN;

$R_d$ = Reduction factor due to bending of strand;

$WL$ = Working load of anchor, kN.

Table 3  Minimum safety factors recommended for design of individual anchorages (TR26:2010)

<table>
<thead>
<tr>
<th>Anchorages Category</th>
<th>Minimum safety factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tendon failure</td>
</tr>
<tr>
<td>Temporary anchorage with a service life of, say, up to two years where although the consequences of failure are quite serious, there is no danger to public safety without adequate warning*</td>
<td>1.6</td>
</tr>
<tr>
<td>Permanent anchorages and temporary anchorages where corrosion risk is high and/or the consequence of failure is serious*</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The removable ground anchor is using loop system and usually consists of one or more unit anchors. Based on Table 3, the minimum geotechnical safety factor of 2.5 was used since the ground anchor is only design as temporary ground anchor. The fixed length for each unit anchor should be calculated using the following equation, Equation 5d from TR26:2010:
\[ L_{fix,i} = \frac{(F_g \times WL_i)}{\pi \times D \times f_{s,i}} \]  \hspace{1cm} (2)

where

- \( L_{fix,i} \) = Required fixed length for each unit anchorage, m;
- \( D \) = Diameter of borehole, m;
- \( F_g \) = Factor of safety, geotechnical (see Table 3);
- \( f_{s,i} \) = Ultimate skin friction at the location of the anchor holding piece, kPa;
- \( WL_i \) = Working load of each unit anchor, kN.

The theoretical elongation of each pair is calculated as follows:

\[ \text{Theoretical elongation, } e = \frac{(P \times L)}{(A \times E)} \]  \hspace{1cm} (3)

where

- \( P \) = applied load on each pair of strands, kN
- \( L \) = total length of strand, m
- \( A \) = area of strands, m²
- \( E \) = Young’s modulus, kN/m²

The applied load, \( P \) is the ultimate load divided by the number of holding pieces. After tabulating the elongation, the values are subtracted from each other to find the difference in elongation. This differential elongation is used to calculate the load required by each pair to achieve balanced or equal load at the maximum test load of 125% WL. The elongation during the test is compared with the theoretical values specified by the code. If these criteria are not met, the affected ground anchor will be downgraded or replaced.

Pre-load affects the behavior of ground anchor during subsequent excavation work. For this project, the preloading of ground anchor is 75% for south wall and 60% for north wall based on the results of the analysis to satisfy the wall deflection requirement of north and south wall. These pre-load values deviated from the recommendation of preloading 110% of its WL from BS8081:1989. Preloading for the ground anchor should be based on the design value of preload assumed in the earth retaining wall analysis and the anchor lock-off load should be increased by additional 10% from the design preload value to allow for losses as recommended by TR26:2010. The magnitude of the grout/tendon interface ultimate bond stress is assumed to be uniform over the tendon bond length and is equal to 2.0 N/mm² for clean strand for minimum grout compressive strength of 30N/mm² prior to stressing and clear spacing between tendons is not less than 5mm.
6. Comparison between Predicted Results and Field Monitoring Results

At this project, a total of 534 numbers of ground anchors were installed successfully on site and 16% of all ground anchors are monitored for load using load cells. The monitoring results for installed ground anchors were well within the WSL. Figures 18 and 19 show the view of ground anchors on site. Figure 20 shows the comparison of measured and predicted load from the ground anchors based on Plaxis analysis. The monitored ground anchors force was only available for up to stage of excavation to below last layer of ground anchors at the time when this paper was written for north side of section A1-A1 is presented in this paper for discussion. The load trends for first layer ground anchor are comparable between measured and predicted readings. From the FEM analysis, the forces of ground anchor below the roof slab increase gradually during subsequent stages of excavation. However, the measured loads do not increase during the subsequent excavation stages as predicted by the finite element analysis. This deviation is probably attributed to the soil properties in this area being better than the design assumptions.

Comparison between the analytical and measured wall deflections for section A1-A1 (north), B-B (north) and A-A (south) are shown in Figure 21. It can be observed that the finite element analysis has over-predicted the actual wall deflection at all the three sections. The shape of wall deflection for predicted and observed are similar. However, the observed maximum deflection values are lower than the predicted values at various stages. The performance is notably better than the predictions. This is probably due to the finite element analysis being carried out using moderately conservative soil parameters. Another reason for the over-estimation of the wall deflection could be due to the cracked section properties assumed for the CBP wall in the FEM, which is 70% of the original stiffness. The axial stiffness of the ground anchor could also be underestimated due to the use of total length in the stiffness calculation. To date, the monitored movement induced on the adjacent viaduct structure during ground anchor installation and basement excavation is less than 2mm. This is less than allowable movement of 10mm.
Figure 19  Photo showing the installed ground anchor at south side of section A1-A1

Figure 20  Comparison between measured and predicted ground anchor force development and distribution for north wall of section A1-A1
Figure 21 Comparison of wall deflection at (a) A1-A1 (North) Inclinometer, IW115; (b) section B (North) Inclinometer, IW117; (c) section A-A (South) Inclinometer, IW107
7. Performance Evaluation

As shown in Figure 22, the ground anchor force can vary by 10%-15% of WL after lock-off. Hence it is recommended to set the Alert Level (AL) with 15% of WL more than lock-off load. When acceptance tests were carried out at site, there were few numbers of ground anchors not able to achieve the specified capacity. In order to make use of these defective ground anchors, the WL was downgraded and they were finally lock-off at 90% of WL. In addition, the WL of the adjacent ground anchors was upgraded to compensate for the load of the defective ground anchors.

![Figure 22](image)

Figure 22 (Maximum GA Force / WSL)x100% against depth of GA below ground level for (i) GA with 60% lock-off, (ii) GA with 75% lock-off and (iii) GA with 90% lock-off

Majority of the instrumented ground anchors experience initial loss of load after lock-off as shown in the trend plot of measured load against time in Figure 23. This provides evidence to support the recommendation by BS8081 and TR26:2010 where the ground anchor lock-off has to be increased by 10% of the design pre-load value to allow for losses. As such, the engineer need to be aware of the final lock-off force in tendon and should allow for the losses of the tension resulting from the following factors:

1) Losses due to friction of the tendon in its duct
2) Losses due to instantaneous deformation of the concrete during non-simultaneous tensioning of the several tendons (elastic loss)
3) Losses due to anchorage pull-in
4) Losses due to deferred concrete shrinkage
5) Losses due to concrete creep
6) Losses due to relaxation of the prestressing

From Figure 24, it can be seen that the loads have picked up again after initial loss when excavation progresses.
Figure 23 Trend plots showing measured ground anchor (GA) force against time
As shown in plot and bar chart in Figure 24, the majority of the ground anchors fall within the category of having 0-5% initial losses from lock-off load. In general, the percentages of initial losses to the lock-off force are within 10%. Only a small percentage of ground anchors exceed 10% of initial losses from lock-off force.

This statistics based on 97 number of monitored ground anchor has shown evidence that it is appropriate to adopt 110% of lock-off force to allow for the possible losses that could be occur due to reasons discuss earlier.

8. Conclusion & Recommendations

In conclusion, this case history has demonstrated the optimization of the ERSS design with partial top-down method and removable ground anchors is successful in meeting the design objective and ensuring safety in construction. This case study has also demonstrated that by using partial-top down construction method with the permanent roof slab as part of the support system for the excavation, one layer of temporary support system can be omitted and thus construction period can be cut down substantially. The ERSS design is also a more robust design for the project site which has major constraint due to the adjacent viaduct structure.

Ground anchor are usually installed as critical elements of the structural support system of earth retaining structure and are normally covered over and rendered inaccessible following completion of construction. It is for these reasons that it is imperative that anchor should be well designed from the outset and executed in a manner which will ensure that the performance expectations are satisfied. If there are any problems with the anchoring system at
some time in the future, it is difficult to carry out repair or replace defective anchors. The design of ground anchors should be undertaken by engineers with necessary geotechnical knowledge and experience.

Currently, there are few ground anchor codes of practice available although there are recommendation and guides given in these codes for the design of ground anchors. However, Clause 11.4 of BS8081:1989 may not be appropriate for the design of the temporary removable ground anchors in the Singapore context. It is more practical, the preloading for ground anchor to be based on ERSS analyses instead of fixed 110% of WL as stated in Clause 11.4 of BS8081:1989. TR26:2010 on the other hand has provided a more practical approach to determine the pre-loading values for the ground anchors, which is based on the results of the ERSS analysis. This basis for the pre-load values makes construction control more practical as the WSL will not be breached from the point of stressing the ground anchor. The monitoring results of this case study have shown that the approach of applying pre-loading values on the ground anchors based on the ERSS analysis results, which is about 60% to 75% in this case, is practical and the site performance is in line with prediction.

Based on the monitored ground anchor load cells data, it is recommend to set the AL by a markup of 15% of WL from the lock-off load to avoid the subsequent load increase of ground anchor to breach the AL.

The statistics of the monitoring results has provided good evidence that the ground anchors are likely to experience initial losses of 10% of preload force due to various reasons. Hence it is reasonable to adopt for 110% of the preload value as the lock-off force to allow for initial losses of preload.

This case history has shown that the performance of the ERSS in terms of ground anchor forces and wall deflection is better than prediction but the design of the ERSS with ground anchor should be review on case by case basis specific to the site and ground conditions. The benefit of the removable ground anchor in terms of providing greater site accessibility and construction safety should not be ignored as it will be future benefit to the industry in enhancing safety in deep excavation project.

9. Acknowledgement

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