

New Approach of Using Jacked Anchors as Reinforcements in Soil Stabilisation Works for a Cut-And-Cover Tunnel with 17m Deep Excavation

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Introduction

This paper presents the design, installation and performance of a proprietary SGE jacked anchor* system supporting both the contiguous bored pile (C.B.P.) wall and soldier pile wall for a cut-and-cover tunnel construction with a successful excavation depth down to about 17m. The design technique used in this stabilizing system lies in between the reinforced soil theory and the conventional prestressed ground anchorage design with Rankine or Coulomb earth pressure theory. As such, finite element program, PLAXIS, is deployed to model the soil-structure interaction of the jacked anchors and the overall stress and strain distribution of the retained soil mass. From the analyses, it is observed that the jacked anchor, in fact, stiffens and reinforces the retained soil mass to behave as semi rigid gravity structure, which is fairly similar to the conventional gravity wall. The shear strain distribution of the finite element mesh also indicates higher shear strain behind the reinforced soil mass. The instrumentation scheme has yielded some useful information revealing the actual behaviour of the reinforced soil mass in terms of wall movements, and the load transfer between the jacked anchor and the surrounding soil during jacking, pull out tests and excavation in front of the wall. Comparisons of this support system with the adjacent prestressed anchor wall are also presented.

The project details

To improve the infrastructure facilities, Malaysian government is constructing the Light Rail Transit (LRT) for the new government administrative centre, namely Putrajaya, in phases. Most of the LRT routes are very close to the

* Jacked anchor is a patented technology by Specialist Grouting Engineers Sdn. Bhd.
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existing buildings and often require temporary shoring system to protect the neighbouring structures during excavation. This project involves construction of a temporary shoring system for a 17m deep excavation using $\phi 750\text{mm}$ and $\phi 900\text{mm}$ contiguous bored pile (CBP) wall and soldier pile (SP) wall with maximum five rows of prestressed ground anchorage support in the original design. Due to the close proximity to the buildings, the designed ground anchorages are inclined at steep angle with relatively short anchor length to avoid hitting the building foundation, which are only 8m away from the wall. Figure 1 shows the plan view of the CBP wall.

Figure 1 Layout Plan of CBP Wall with Two Different Support Systems

Site geology and subsoil conditions

The engineering properties of the subsoil are summarised in Figure 2 and Table 1 respectively. The Young's modulus profile of subsoils interpreted from pressuremeter test results is also presented in Figure 2. The groundwater as measured from the standpipe and during subsurface exploration was about at level RL21.8m, which is 12m below the retained ground level of RL34m.

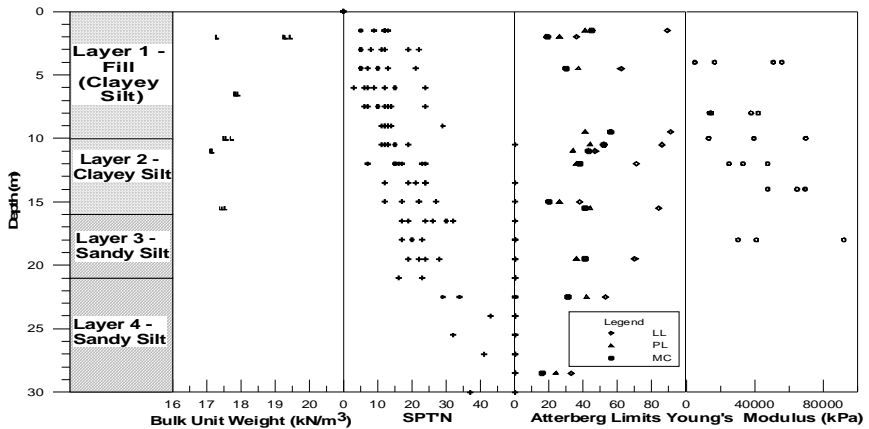


Figure 2 Interpreted Subsoil Profile

Table 1 Engineering Properties of subsoil

Layer	γ_{bulk} (kN/m ³)	γ_{dry} (kN/m ³)	ϕ' (°)	c' (kN/m ²)	ψ (°)	E' (kN/m ²)	E'_{ur} (kN/m ²)	ν_{ur}
1	18.0	14.0	30	4	0	11,130	33,380	0.2
2	18.0	14.0	32	4	2	24,500	73,500	0.2
3	18.5	14.5	32	4	2	45,390	136,170	0.2
4	19.0	15.0	34	5	4	133,500	400,500	0.2

Structural details

For the purpose of FEM analyses, the structural properties of CBP wall, jacked anchor and ground anchor are tabulated in Table 2.

The maximum tensile and compressive structural working capacity of the jacked anchor is about 180kN whereas the structural working capacity of the prestressed ground anchor is 125kN per PC strand (Grade 270 and 15.24mm).

Table 2 Engineering Properties of Structural Elements

Structural Elements	Axial Stiffness, EA	Flexural Stiffness, EI
φ900mm CBP wall	1.78×10^7 kN/m-run	9.018×10^5 kN-m ² /m-run
Mild Steel Jacked Anchor (φ114mm, 4.5mm thk.)	3.173×10^5 kN/m-run	5.367×10^2 kN-m ² /m-run
Ground Anchors	2.718×10^2 kN/PC strand	-

Test Results

Pull-out tests have been carried out at different time intervals after installing the jacked anchors to verify the development of shaft resistance with time. These pull-out test results are presented in Figure 3. Not all pull-out tests have mobilised the ultimate capacity as they were only tested to 2.2 times the designated working pull-out capacity.

From Figure 3, the mobilised shaft resistance of these pull-out tests shows an obvious increasing trend with time. This is primarily caused by the increase of effective radial stress surrounding the jacked anchor after dissipation of excess pore pressure induced by soil displacement during jacking. It is also expected that the stiffness of the soil will increase indirectly in the similar manner. Tan et al (2001) have presented a methodology using cavity expansion method to assess the excess pore pressure response of a jacked anchor inclusion and its dissipation resulting in increase of pull-out capacity.

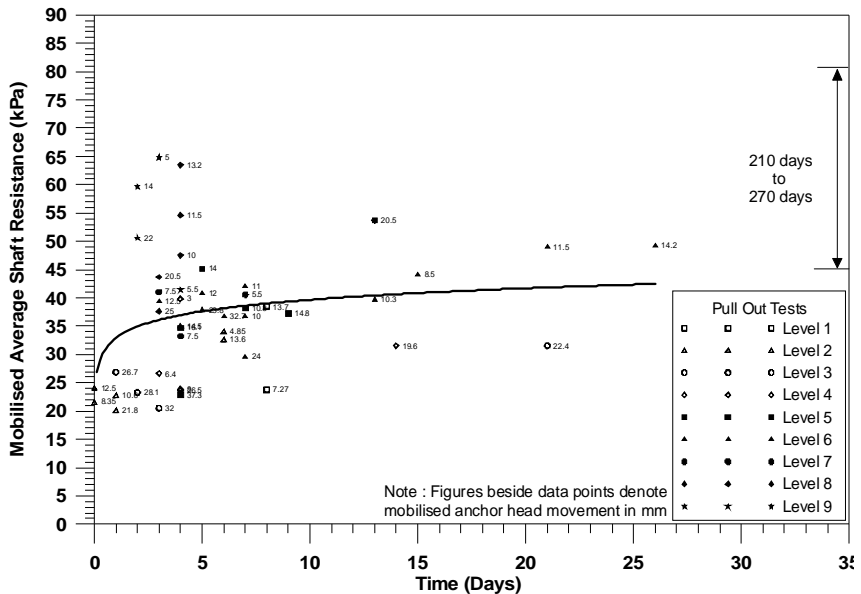


Figure 3 Mobilised Average Shaft Resistance with Time

Instrumentation

In order to verify the design performance of both the alternative and compliance support systems, the following instruments as tabulated in Table 3 have been installed. Two instrumented sections have been installed at CBP walls with jacked anchor and prestressed ground anchor respectively for performance comparison.

Table 3 Instrumentation Details

Wall Type	Instrument	Location
φ900mm CBP Wall with Jacked Anchors	Inclinometer	I2 in CBP Pile A48 I3 at 1m behind jacked anchors
	Load Cell	LC3, LC5, LC7 and LC9 at anchor levels L3, L5, L7 and L9
	Strain Gauge	Vibrating wire strain gauges along L4 and L7 jacked anchors
	Settlement Marker	1m behind CBP wall
φ900mm CBP Wall with Ground Anchors	Inclinometer	I1 in CBP Pile A13
	Load Cell	LC2, LC3, LC4 and LC5 at anchor levels L2, L3, L4 and L5
	Settlement Marker	1m behind CBP wall

Mobilised Shaft Friction on Jacked Anchors during Pull-Out Tests

Strain gauges have been installed on the two selected jacked anchors at levels L4 and L7 to monitor the load transfer behaviour of the mobilized shaft resistance during jacking process and pull out test at various time intervals after installation as shown in Figure 4. As only one strain gauge was installed at each section of L4 jacked anchor, significant flexural effect can be expected to affect the strain gauge reading during jacking process. This has been verified during jacking the L7 jacked anchor, in which the coupled pair of strain gauges has indicated significant flexural effect in the jacked anchors. However, the flexural effect is much minimized during pull-out test. It was also observed that lower shaft resistance has been mobilised at the L4 jacked anchor as compared to the L7 jacked anchor. For the two instrumented jacked anchors, higher mobilized shaft resistance is observed at the middle segment of the anchor during pull out test. Most instrumented segments of jacked anchor indicate ultimate shaft resistance has been achieved when the head displacement of jacked anchor reaches 5mm to 10mm. Generally, the pull out load of the two instrumented jacked anchors show increasing trend with time indicating stiffer behaviour.

Load Cells

It is observable that the jacked anchor loads at the anchor-to-wall connection increase drastically from the nominal lock-off load with the excavation depth except the lowest jacked anchor, in which there is no significant excavation after installation of the lowest jacked anchor as compared to other jacked anchors at higher levels. Another reason for that could be due to relatively stiff soil stratum at lower level. The average excavation depth at each excavation stage is about 1.8m. Figure 5 shows the variation of jacked anchor load with time.

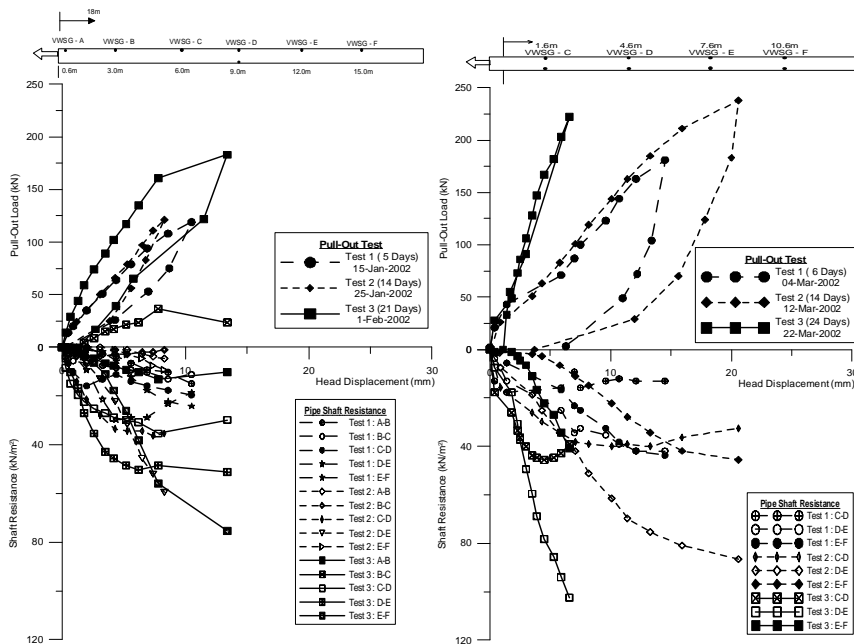


Figure 4 Pull Out Test Results for Instrumented Jacked Anchors L4 and L7

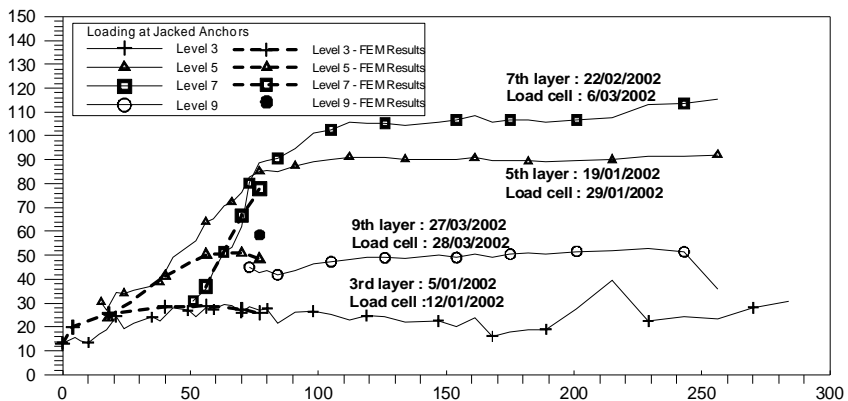


Figure 5 Jacked Anchor Load with Time

As for the prestressed ground anchor, designated prestress loads have been applied to the respective ground anchors during lock-off. However, some ground anchors have shown reduction of prestress as in Figure 6, which could be due to creeping of the relatively short fixed anchor length and potential relaxation of prestress as a result of vertical movement of CBP piles under high vertical load component from the prestressed ground anchor.

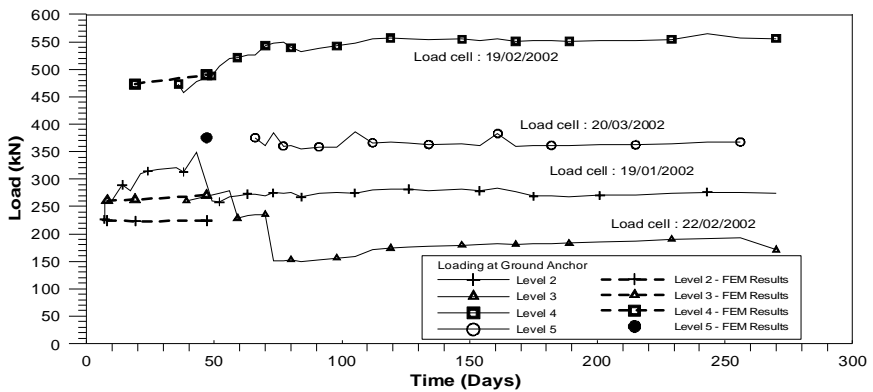
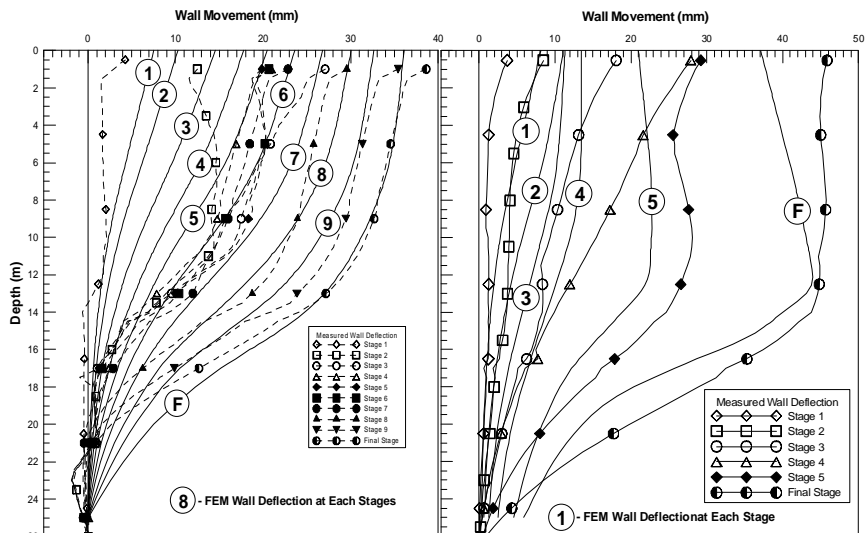


Figure 6 Prestressed Ground Anchor Load with Time

Wall Movements

The wall movements at both the jacked anchors and prestressed ground anchor walls have been monitored using the inclinometers installed inside the CBP wall. Figure 7 shows the monitored wall movements during various excavation stages at both walls. The jacked anchor wall has moved about 35mm at the final excavation with relatively fixed toe embedment, whereas the prestressed ground anchor wall has moved about 46mm with the wall toe rotated, which implies the overall wall movement could be more and the more passive resistance at the wall embedment has been mobilised to maintain overall wall



stability.

Figure 7 Wall Movements at Jacked Anchor Wall and Ground Anchor Wall

Settlement Markers

The ground settlement behind the CBP wall at various construction stages is shown in Figure 8. The ground settlement results indicate the performance of jacked anchor support system is far better than the prestressed ground anchor system. Generally, the ratios of ground settlement to wall movement of jacked anchor wall and ground anchor wall are 1.57 and 3.37 respectively.

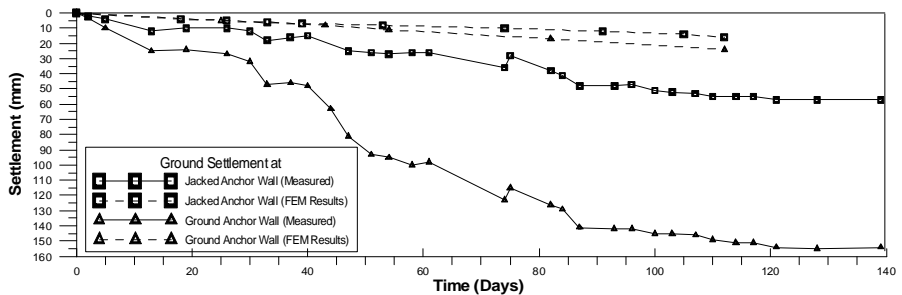


Figure 8 Ground Settlement behind the CBP Wall

FEM modelling

Two dimensional finite element method (FEM) with 6-node elements was used to model and back-analyse the performance of the CBP wall supported by both jacked anchors and prestressed ground anchors. “Hardening Soil” model (Brinkgreve, 2002), which is suitable for residual soil, was used as soil model. The input parameters for the “Hardening soil” model are summarised in Table 1. The $\phi 900\text{mm}$ CBP wall and jacked anchors were modelled as beam element whereas the prestressed ground anchors were modelled as elastoplastic spring node-to-node element for the free length and geotextile interface element for the fixed length. Interface elements were also applied to the wall-soil and anchor-soil contacts. In numerical modelling, the CBP wall was assumed as “wished-in-place” condition before the excavation started, and the undrained analysis incorporated with groundwater calculation was performed under 2-D plane strain condition for the expected short construction period.

Back-Analyses and discussions

Figure 7 shows the lateral wall movements of the CBP wall supported by jacked anchors and prestressed ground anchors. In general, reasonably close agreement in the lateral wall movement profile of jacked anchor wall and prestressed ground anchor wall has been achieved during the back-analyses except at the final stage of excavation for the prestressed ground anchor wall where the wall movement is slightly underestimated. This could be due to the load relaxation and creeping of some prestressed ground anchors with time, particularly those at levels 2 and 3 as shown in Figure 6, that are not taken into account in the FEM modelling. Generally, the FEM results match well with the measured lateral

wall movement profile in most excavation stages for both jacked anchor and ground anchor walls. On the other hand, ground settlement markers installed at about 1m behind the CBP wall indicate larger settlement magnitude compared to the FEM results as shown in Figure 8. Figure 9 shows the dimensionless plot of the analysed and measured ground surface settlement profiles as recommended by Clough & O'Rourke (1990). The maximum wall movement of CBP wall at final excavation is about $0.002H$, which is tally well with the average maximum movement of cast-in-situ rigid wall on hard clay and sandy soil layers presented by Clough & O'Rourke (1990). However, Ou et al (1993) has also presented the ratio of maximum wall deflection to excavation depth in the range from 0.002 to 0.005 in numbers of deep excavation in Taipei basin.

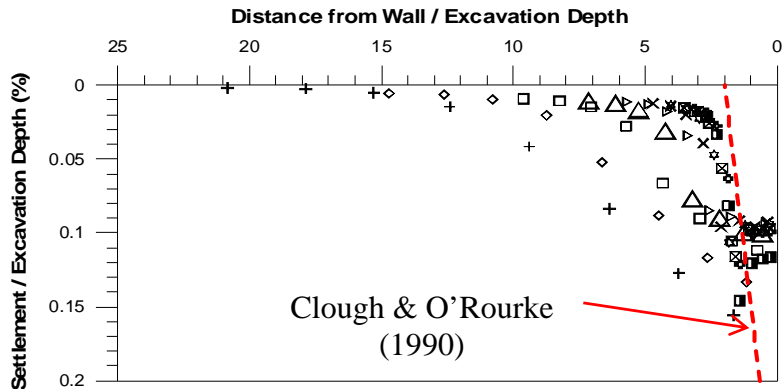


Figure 10 Dimensionless Ground Surface Settlement Profile.

In Malaysia, it is a common practice to use correlation of Standard Penetration Test (SPT) to obtain both strength and stiffness parameters of the residual soils for geotechnical design, despite some researchers disagree one in-situ test can derive both parameters simultaneously. In this case study, the FEM back-analysis results show that effective Young's modulus (E') of the upper clayey silt and lower sandy silt subsoils are about $2150 \times \text{SPT}'N$ and $2600 \times \text{SPT}'N$ respectively. The first correlation agreed well with the correlation as proposed by Tan et al. (2002) for 27m deep excavation in the weathered meta-sedimentary Kenny Hill formation. The suggested unloading/reloading stiffness (E'_{ur}) used in the 'Hardening Soil' Model is about three (3) times of effective Young's Modulus (E'). Figure 10 shows the back-analysed effective Young's Modulus in the FEM analysis for the prestressed ground anchor wall superimposed over the interpreted effective pressuremeter (PMT) stiffness modulus, in which the back-analysed E' is generally at lower bound of the interpreted pressuremeter stiffness modulus. Whereas the back-analysed E' for the jacked anchor wall is generally about 30% more than that for the prestressed ground anchor wall.

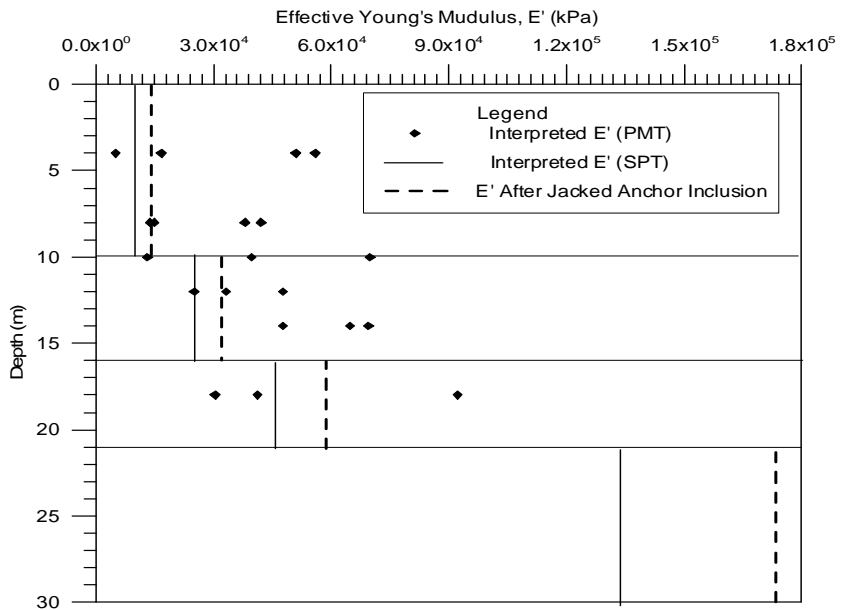


Figure 10 Interpreted E' from SPT'N and Pressuremeter Test (PMT)

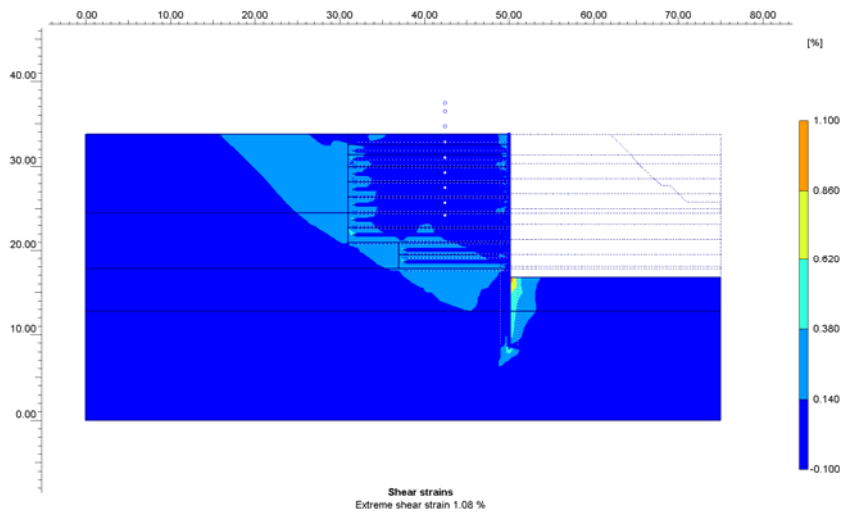


Figure 11 Soil Shear Strain within the Jacked Anchor Retaining System

From Figure 11, the shear strains within the reinforced soil mass as a result of jacked anchor inclusion are mostly less than 0.15%. There is a relatively larger shear strains ranging between 0.26% and 0.38% developed along the

potential slip surface of the active zone behind the reinforced soil mass. This potential band of slip surface appears to suggest that the reinforced soil mass tends to slide laterally along the base under the huge active earth pressure behind the reinforced mass. The inclusion of jacked anchors has restricted the development of active zone within the reinforced soil mass. In accompany to the wall geometry and the potential failure mechanism, it is expected that huge shear force and flexural stresses would be induced at the embedded wall and the passive zone will be highly mobilized, particularly at the excavation level. In fact, the highest shear strain in the FEM analysis is actually located at the passive zone in front of the wall embedment.

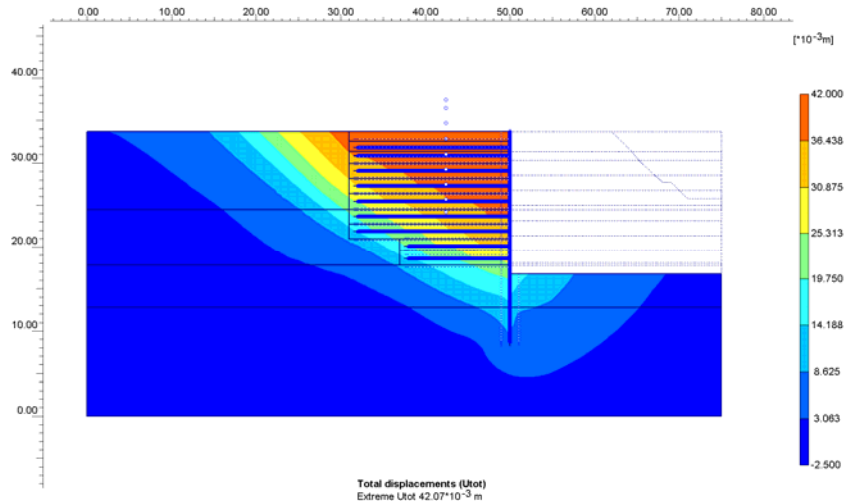


Figure 12 Total Ground Displacement of the Jacked Anchor Retaining System

From the total ground displacement contour as shown in Figure 12, it can be observed that the reinforced soil mass has more displacement at the upper portion with gradually reduced trend towards the lower portion, in which the displacement is cumulative.

Conclusions and recommendations

Based on the discussions in the earlier sections, the following conclusions can be made:

- i. The jacked anchor wall behaves as a semi reinforced soil wall with better overall performance as compared to the prestressed anchored wall. Intensive soil-structure interaction can be observed between the soil and the jacked anchors. As a result, earth pressure immediately behind the jacked anchor CBP wall is much less as compared to the one with prestressed ground anchors. This is because part of the resistance to the active zone within the reinforced area has been

provided through the interfacial resistance of the jacked anchors before it is fully transferred to the wall. Therefore, it is conservative to use conventional Rankine or Coulomb earth pressure for assessing the bending stresses of the wall.

- ii. Observable shearing zone, which could be developed into slip surface defining the failure mechanism of the retaining system forming the active wedge, has occurred behind the jacked anchor wall.
- iii. From the observation of the initial pull out test results, the pull out capacity of the jacked anchor does not appear to be related to the effective overburden stress at the initial stage, rather its mobilised shaft resistance is constricted to a narrow range from 20kPa to 30kPa. However, the thixotropy effect of soil has shown increased pull out capacity of the jacked anchor with time, but remains constant after probably 120 days. The effective increase of pull out capacity is generally in the range of 70% to 90%.
- iv. There is also significant stiffening effect after the jacked anchor installation, which significantly improve the overall performance of the wall. In this case study, there is an increase in stiffness by about 30%. Therefore, selection of design parameters shall take into consideration of such effect.
- v. The backed-calculated engineering parameters between the ground anchor wall and the jacked anchor wall can give indication of the stiffening effect.
- vi. The instrumentation and back-analyses have yielded very useful information in this study.
- vii. Finite element method can be successfully deployed to analyse the complicated interaction of the entire soil-structure system and therefore to assess the ultimate and serviceability conditions of the retaining system.

There are also recommendations for the future research of this support system as follows:

- i. Strain gauges shall be installed in pairs at the jacked anchor section to avoid flexural effect in the interpretation.
- ii. Settlement profile behind the wall and even behind the end of jacked anchors shall be monitored to confirm the settlement trough above the active wedge, which could be a concern to the structures sit on top of the active wedge.
- iii. More inclinometer results are needed behind the jacked anchors to indicate the formation of the active wedge.
- iv. More researches on the generation of excess pore water pressure and its dissipation around and along the jacked anchor during inclusion to the retained soil shall be carried out to assess the set-up of interfacial resistance.

References

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