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NUMERICAL ANALYSIS ON THE EFFECT OF JET GROUT PILES ON AN EXCAVATION LOCATED IN AN URBAN AREA

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ABSTRACT: The development of infrastructure in limited land space is a challenging scenario. Infrastructure in limited land space regions (e.g. Singapore) does not provide freedom to develop at favourable locations instead forces the engineer to design at the possible locations. Clay with high organic content, commonly referred as Peaty clay is predominant in coastal areas. This clay being highly acidic (PH>7) and possessing very low shear strength is a critical factor to the design of infrastructure in the vicinity. Ground improvement adopted weak strata will have varying effect and may not be able to achieve the required strength. This paper discusses the effect of ground improvement (Jet Grout Piles) on the sloped excavation predominantly in Peaty clay. A 15m deep excavation which is 60 m wide is used for the Finite element modeling. Impact study on a tunnel located 40m from the excavation is presented (The study is carried out for various achieved Jet grout piles strengths of 100, 200, 300, 400 and 500 kPa). The stability of the slopes for the various strengths is also discussed (GeoSlope was used as the medium to perform the geotechnical analysis).

Keywords: Jet Grout Piles, Peaty clay, Sloped excavation, Impact study

1. INTRODUCTION

Sloped excavation in an urban area poses threat by means of slope failure which will in turn damage nearby infrastructure. Hence, a conservative design approach has to be adopted to prevent slop failure during excavation. Slope failure is dependent on the undrained shear strength of the soil and it is favorable to execute excavation in sub soil conditions which have high strengths. In cases when the undrained shear strength is low (1-2 kPa) ground improvement techniques has to be adopted.

Peaty clay also called as organic clay (due to the presence of organic materials which starts under aerobic and anaerobic conditions through incomplete decomposition of plant and animal matter) is acidic in nature and also has low undrained shear strength. Unlike clays, in peaty deformation does not take place by pore pressure dissipation rather from a continuous rearrangement of soil particles under constant vertical effective stress after pore pressure dissipation Ground improvement technique usually adopted for peaty clay is Jet Grouted piles (JGP) or grouted stone column. Grouted stone column may not be used in the presence of critical structures nearby as it will cause unnecessary disturbances during the process. Jet grouted piles on the other hand do not cause these disturbances and hence are adopted in most of the cases. Triple tube Jet grout system is adopted for this paper. A challenging characteristic of peaty clay is that it hinders the formation of calcium silicate hydrate when mixed with cement, thus it very difficult to achieve the target undrained shear strength.

Finite element modeling (PLAXIS) and limit equilibrium model was used to study the viability of the excavation nearby critical structure (tunnel) and green field respectively. The constitutive soil models are derived based on various conservative assumptions and hence will prevent under design of the ground improvement technique adopted. Modeling the ground improvement and predicting the ground settlement are accomplished using finite element modeling

2. SUB SOIL CONDITIONS HEADINGS

Sub soil conditions are shown in Figure 1. It is evident that there is a predominant amount of peaty clay. Consists of 10m thick peaty clay layer (acidic in nature, PH>7) with a SPT value of 4-7. The organic clay layer is bedded upon a thick layer of peaty sand.

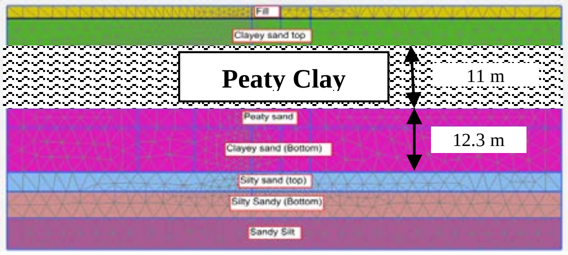

Fig. 1 Sub soil conditions

Peaty clay is a geological member of Kallang formation. Kallang formation is a recent formation and has alluvial, marine clay, Estuarine and littoral clays. The graph presented below shows variation of

Estuarine clay with depth [2]. This graph was plotted based on the shear strength encountered in various part of Singapore.

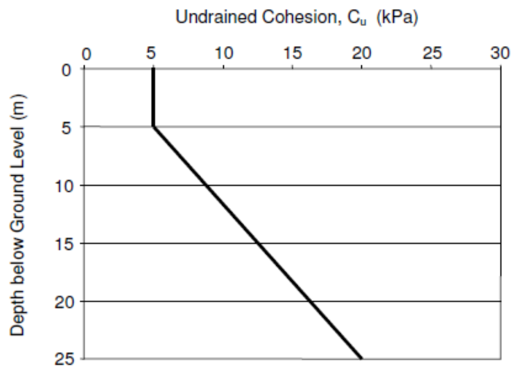

Fig. 1 Undrained Cohesion of Estuarine Clays (Source: LTA CDC, Singapore)

3. EXCAVATION GEOMETRY

The excavation geometry is shown in Figure 3. The geometry of the excavation is adopted from the general sloped excavation scheme is Singapore. The excavation is 60 m wide and 30 m deep. Peaty clay is encountered from 8 to 18 m overlaying 5m deep 5m peaty sand. Two 10 m wide promenades is to be provided on either side of the excavation Slope of 1:1.5 is considered on either side. The slopes are interrupted by two berms so as to increase the stability of the Slopes. Due to the sloped profile it is not possible to provide Earth retaining stabilizing scheme (struts and walers).

3.1 Side A: Near Critical Structure

A Tunnel 20m away is considered as shown in Figure 3. The axis of the tunnel is 25m below ground level Peaty clay. It will be influenced by the excavation and hence advanced finite element modeling techniques is considered to study the impact of the excavation on the tunnel embedded in peaty clay. In order to reduce the influence of the excavation on the tunnel it is necessary to provide a Diaphragm wall 1m thick.

3.2 Side B: Near Green Field

Impact of the excavation to green field areas is not a critical issue due to the absence of infrastructure and utilities in the vicinity. The Diaphragm wall provided for Side A will cut of the impact of excavation but due to the absence of Earth Retaining Stabilizing Structure (ERSS) for side B Slope stability is an important factor and it is necessary to maintain a factor of safety of greater than 2 (Global and local slope stability).

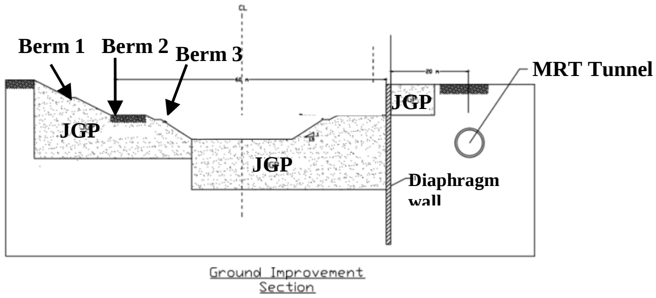

Fig. 3 Geometry of the excavation

4. GROUND IMPROVEMENT SCHEME

The presence of weak sub soil strata (low undrained shear strength) such as peaty clay and peaty sand indicates the need for ground improvement before the start of the excavation [6]. Jet grout piles 1m c/c spacing with varying depths of treatment as shown in Figure 3. The varying depth of treatment was derived based on the minimum depth of Improvement required to satisfy the allowable deflection criteria on the tunnel (Horizontal displacement of 25 mm)

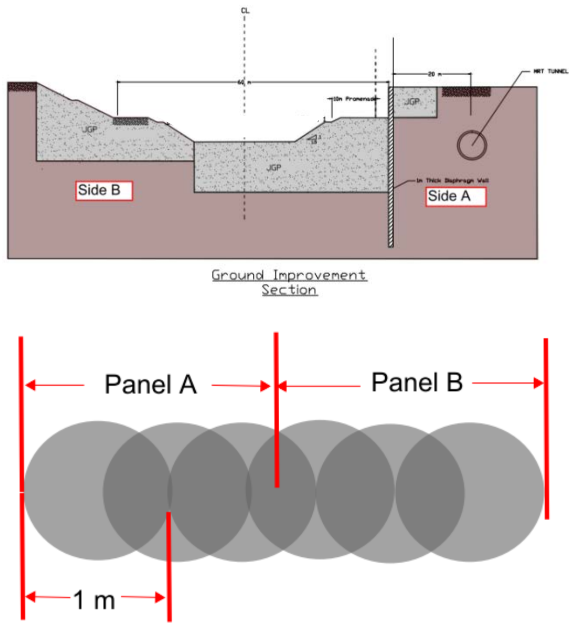

Fig. 4 Jet Grout pile – Plan and Elevation view

Triple tube coring can be used to carry out the ground improvement scheme as it reduces the time required and also attains a uniform mix of JGP. The uniformity can be checked by taking core samples at the overlap as shown in figure 4. The samples should be tested for 7 day and 28 day strengths.

4.1 Ground Improvement Methodology

Methodology for the design used is an important factor in determining the efficiency of the model. The flowchart presented in figure 5. The governing criteria are the stability of slopes and impact of excavation on critical structures.

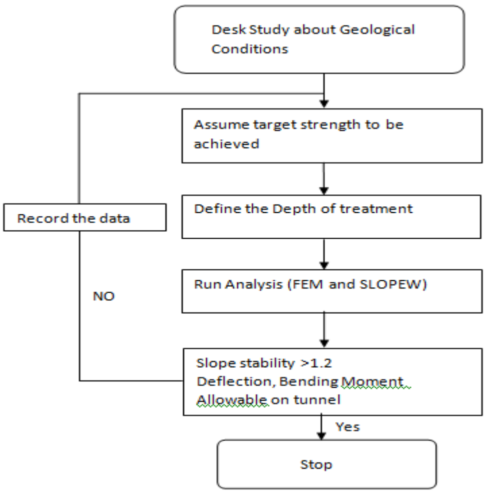

Fig. 5 Design Flowchart

5. FINITE ELEMENT MODEL

The numerical tool used in this work is the commercial PLAXIS finite element program. PLAXIS 2D V12 is a 2 Dimensional Finite element analysis. PLAXIS allows using two elements with 6 or 15 node triangles. All analysis in this paper use 15 node analysis unless otherwise stated. Mohr Coulomb model (linear soil model) was adopted.

5.1 Overview of Soil Model

Mohr Coulomb model (Method B - Stiffness and strength are defined in terms of effective property) was used for modeling the geological strata. MC model as shown in Figure 6 behaves elastically before failure. The important parameters used were E', Cu or C' for Undrained and Drained cases respectively. Peaty clay was assumed to be normally consolidated (OCR=1). The drained stiffness of JGP was based on E'=0.5Cu. Moderatively, conservative parameters were taken from the LTA Civil design criteria thus preventing under design of the excavation.

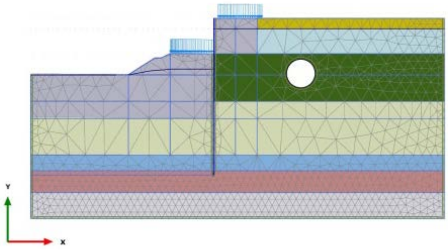

Fig. 6 – FEM Model (PLAXIS)

6. Limit state Model

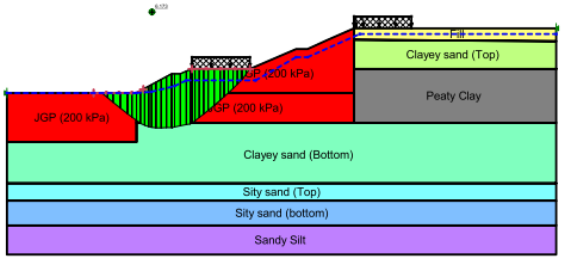

Fig. 7 Limit state model (GEOLSOPE)

Limit state model as shown in Figure 7 was used for finding the global and local factor of safety for the slopes near green field. Morgenstern and Price (1965) [13] method is used to iterate the global and local factor of safety. Morgenstern and Price [13] method is based on combined equations of force and moment equilibrium equations. A modified Newton-Raphson numerical technique was used to solve for factor of safety satisfying force and moment equilibrium equations.

$$X / E = \lambda f(x) \tag{1}$$

Where f(x) is a function that describes the manner in which X/E varies across the slope and λ a constant representing a portion of the function used when solving for factor of safety. Limitation of this method is that the direction of the resultant is arbitrary assumed.

7. RESULTS AND CONCLUSIONS

7.1 Slope Stability

Berm 1, Berm 2 and Berm 3 as indicated in Figure 4 were analysed for slope stability. A factor of safety greater than 1.2 was set as minimum criteria for the improvement strength to be considered. Figure 8 indicates the increase in global Factor of Safety (FOS) is insignificant and increase in undrained shear strength of the JGP did not affect the Global FOS. This is due to the formation of slip circle beyond the treatment zone.

The inverse phenomenon is observed in the stability of Berms. Berm 1 and Berm 3 are intercepted by untreated Fill and Peat hence the increase in factor of safety is lower than berm 2 which is embedded in treated peaty clay. The trend indicates that the liner relationship between the undrained shear strength and the factor of safety is satisfied for the global and local slip surfaces and also is in accordance with Morgenstern and Price (1965) limit equilibrium formula

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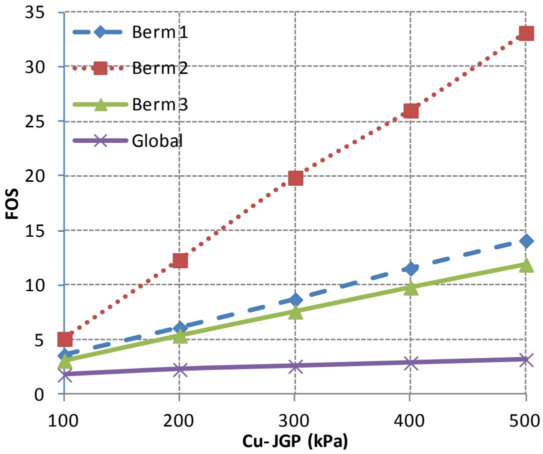

Fig. 8 – Slope stability (FOS) vs Cu – JGP

7.2 Finite Element model

Finite element modeling was carried out to study the excavation impact on the diaphragm wall and tunnel located at 20m from the excavation.

7.2.1 Impact on Diaphragm Wall

The toe stability of the diaphragm wall is the governing factor and hence sufficient toe embedment to prevent rigid body movement is to be designed [7]. Figure 9 plots the deflection Vs depth (mRL) for different strengths of JGP. From the graph, we observe that the toe of Dwall has a lateral displacement of less than 10 mm and thus ensuring a factor of safety > 1 for toe stability.

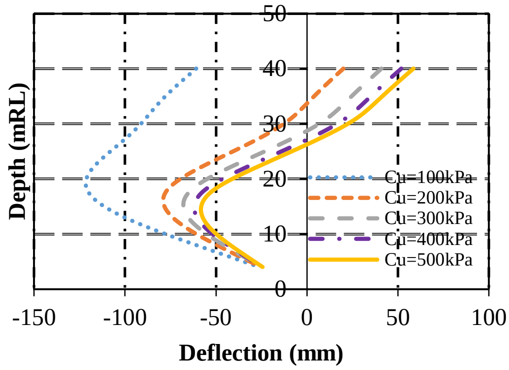

Fig. 9 Depth (mRL) vs. Deflection (mm)

Maximum deflection of 125 mm (towards the excavation) is observed at a depth of 20m due to insufficient strength of JGP. Deflection can be controlled by means of providing struts. The deflection decreases as the strength of JGP increases which substantiate the validity of this model.

It is nominal to achieve strength of 200-300 kPa cause for strengths greater than 300 kPa the head of the wall moves towards the soil side which in turn will cause movement on the tunnel which is unfavourable.

It is evident from the figure 10 that the peak bending moment of 1100 kN/m is obtained at a depth of 30m (Cu = 500 kPa). At a depth of 15m the same strength of JGP causes the lowest bending moment of 430 kNm. The trend signifies that greater the Cu of the JGP greater the Bending moment at 30m and the inverse trend in observed at 15m.

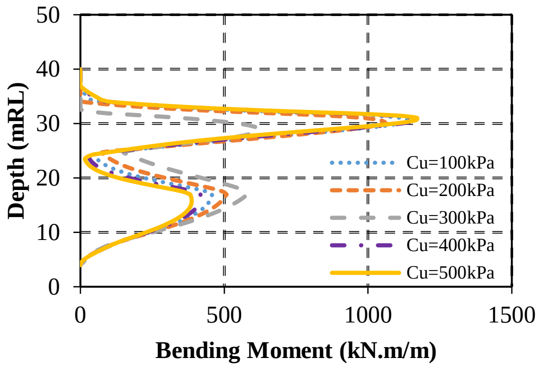

Fig. 10 Bending Moment vs. Reduced level

7.2.2 Impact on Tunnel

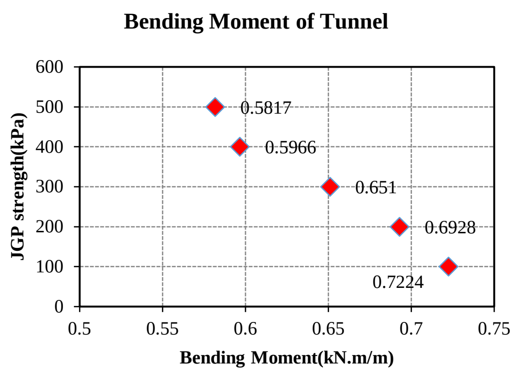

Fig 11. Bending moment vs Cu-JGP

A tunnel of 6.6m radius is introduced to see the effect of JGP. The impact of the ground improvement on the tunnel bending moment is compared. Figure 11 shows the comparison of bending moments for various strengths of JGP. The JGP had helped in reducing the Bending moments of the tunnels. It can be observed the bending moments are indirectly proportional to the JGP strength that is higher the JGP strength lower is the bending moment. There is a 25% decrease in bending moment on an average between JGP strength of 100kPa & 500kPa.

7.2.3 Soil movement

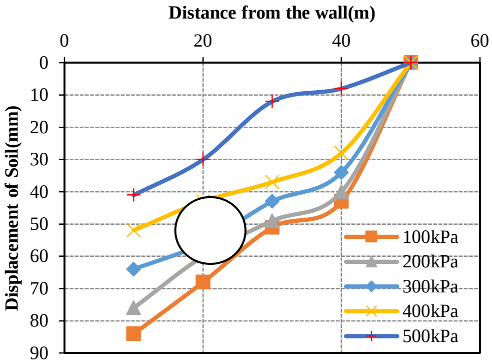

Fig. 12 comparison of various displacement behind the Diaphragm wall

Figure 12 shows the soil displacement behind the wall for a JGP strength of 500kPa. It is clearly inferred that soil movement reduces as we go farther from the diaphragm wall. There was an impact of tunnel on the displacement of soil but that was not very high. Maximum displacement of about 40mm was observed nearer to the wall. All the measurements were taken along a line at 32mRL.

8. ACKNOWLEDGEMNET

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