Optimization of deep soil mixing mass for construction of tunnel cross-passage in Bangkok

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Abstract. A cross-passage from a tunnel to an intervention shaft in Bangkok subsoils was modelled using three-dimensional finite element analysis. The optimization was performed for reducing the volume of soil-cement mass while maintaining ground stability. The result showed that the factor of safety increases linearly with the thickness of improved ground. The relationship between the improved groundmass and ground surface settlement curve was made. It was found that the predicted settlement profile can be captured by the formula proposed by Peck and O’Reilly, providing that the empirical coefficient of 0.6 is fit well with the results from the 3D simulation. This value was obtained from the average of empirical coefficient value 0.4 and 0.7, which are the empirical coefficient values of stiff clay and soft clay, respectively based on the overburden thickness.

1. Introduction

The ground stabilized method has to be used to reduce the large deformation and cross-passage stability when cross-passages were constructed in Bangkok. The deep soil mixing method is an effective method used widely for excavation support, particularly for soft clay deposits. The deep soil mixing method enhances the soft ground into soil-cement, which is durable and can be employed at low cost.

Currently, the improvement scheme of the deep soil mixing is not optimal since the improvement zone was conducted from the surface to reach the cross-passage leading to waste much cement consumption. The construction cost can be reduced further by optimizing the deep soil mixing mass.

In this paper, the optimization of deep soil mixing was conducted to achieve stability with deformation requirement using 3D finite element method. Furthermore, the effect of improvement pattern on the tunnel cross passages is also evaluated.

2. Methodology

2.1. Pattern arrangement of soil-cement mixing

There are four types of pattern mentioned: single type, wall type, grid type, and block type [1]. The block type of the pattern was selected for this study (Figure 1) since block type is safer and suitable for permeable prevention since it contains the overlapping of each column in all direction which is acted as water barrier [2]. The diameter of 0.7m with the overlaps of columns is about 0.1m, which is connected according to the research on road tunnel and soil-cement wall in Bangkok.
2.2. Optimizing reinforcement

The optimization was based on the stability of the cross-passage where safety factor of 1.40 was considered. The improvement material's optimization is conducted by selecting the optimal improvement radius ($R_i$) of the ground stabilization, ranging from between $1.2R$ and $3.0R$ while the radius $R$ is 2.25m. The octagonal shape was adopted; however, soil-cement mixing cannot make a perfect oblique pattern (Figure 2).

2.3. Simulation of cross-passage

2.3.1. Subsoil and groundwater condition in Bangkok. The study is based on the subsoil profile in section A of the MRT blue line in Bangkok (Figure 3) by simplified into three layers:

- Layer (1) is denoted as Bangkok soft clay – medium clay layer from the surface to -14m long.
- Layer (2) is denoted as a stiff clay layer from depth -14m to -24m long.
- Layer (3) is denoted as the first sand layer which is from -24m to downward.

Figure 4 indicates the most recent groundwater level in Bangkok subsoil. Since the groundwater in Bangkok is undergone drawdown incredibly, although the current groundwater is increased, it has not fully recovered.
2.3.2. Model geometry and boundary condition. Möller proposed the guideline for 3D modelling in the tunnel for the tunnels diameter range between 4 ~ 12m, the width of the geometry can be suggested in equation (1) [5]. Meißner recommended the bottom boundary is followed by equation (2) [6].

\[ w = 2D \left( 1 + \frac{H}{D} \right) \] (1)

\[ h = (1.5 \sim 2.5) \times D \] (2)

where
- \( w \) = width of mesh generation
- \( H \) = distance from the ground surface to tunnel crown
- \( D \) = tunnel diameter
- \( h \) = bottom boundary (Starting from the invert of the tunnel down to the considerable length)

Figure 5. Dimension of the IVS and cross-passage
The layout of the cross-passage, D-wall and TBM tunnel in the model have been illustrated in Figure 5. The cross-passage's width and height are about 3.85m and 4.5m respectively, with 12m long. Cross-passage was modelled at -24m from the surface to bottom of cross-passage while -21.75m from the surface to the centre of the cross-passage and it was connected between TBM tunnel and IVS (16m × 16m).

2.3.3. **Soil constitutive and input parameters.** Hardening soil model (HSM) has been selected for this research, and the input parameters are adopted from [7], as shown in Table 1.

| Soil type | $\gamma_b$ (kN/m$^3$) | $c'$ (kPa) | $\phi'$ ($^\circ$) | $\psi'$ ($^\circ$) | $E_{50}^{ref}$ (MPa) | $E_{v50}^{ref}$ (MPa) | $E_{ur}^{ref}$ (MPa) | $v_{ur}$ | $m$ | $K$ | $Rf$ | Analysis Type |
|-----------|----------------|-------------|-------------------|-------------------|----------------------|----------------------|----------------------|-------|---|---|---|---------------|
| BSC       | 16.5           | 1           | 23                | 0                 | 0.8                  | 0.85                 | 8.0                  | 0.2   | 1 | 0.7 | 0.9 | Undrained (A) |
| 1st SC    | 19.5           | 25          | 26                | 0                 | 8.5                  | 9.0                  | 30                   | 0.2   | 1 | 0.5 | 0.9 | Undrained (A) |
| CS        | 19.0           | 1           | 27                | 0                 | 38                   | 38                   | 115                  | 0.2   | 0.5| 0.55| 0.9 | Drained       |

Mohr-coulomb soil model has selected for the soil-cement material model, and the input parameters are adopted from [8] as shown in Table 2.

| Material Type | Analysis Type | $\nu$ | $\gamma_b$ (kN/m$^3$) | $C$ (kPa) | $E$ (MPa) |
|---------------|---------------|-------|----------------------|-----------|-----------|
| Soil-cement   | Undrained (B) | 0.33  | 15                   | 400       | 0.8       |

Structures were modelled in plate elements, and structural parameters which are used in this model are adopted from [9] [10] (Table 3).

| Parameters | $d$ (m) | $\gamma$ (kN/m$^3$) | $E1$ (GPa) | $v$ |
|------------|--------|---------------------|-----------|-----|
| Tunnel lining | 0.30   | 24                  | 31        | 0.20|
| D-wall     | 1.00   | 16.5                | 28        | 0.15|
| Slab       | 1.00   | 25                  | 28        | 0.15|
| Base slab  | 1.80   | 25                  | 28        | 0.15|

2.3.4. **Construction sequences.** The simulation was categorized into six main stages — first, the initial phase where $K_0$ is selected (a). Secondly, soil-cement mixing columns were constructed around the excavation area (b). Then the TBM tunnel was activated (c). Next, the intervention shaft was activated (d). After then the excavation was operated every -1.5m to final excavation (e) (f). Full face excavation is conducted in this study, and face pressure is not applied to the tunnel (Figure 6).
Figure 6. The process of cross-passage in PLAXIS 3D: (a) Initial Phase, (b) Soil-cement mixing, (c) TBM tunnel was constructed, (d) IVS was constructed, (e) Cross-passage open face and excavation, and (f) Final excavation.

2.4. Model Validation

2.4.1. Ground movements induced by tunnelling. An immediate ground surface settlement profile above a single tunnel can be represented with the empirical equation below [11] [12]:

\[ \delta = \delta_{\text{max}} \cdot \exp \left( -\frac{x^2}{2i} \right) \]  

(3)

where
- \( \delta \) = surface settlement at a lateral distance
- \( \delta_{\text{max}} \) = maximum surface settlement at the centerline
- \( x \) = distance from the tunnel centerline

The settlement trough equation was updated which is respective to the depth of the tunnel and the type of the ground as following in equation (4). O'Reilly proposed that the empirical coefficient \( (K) \) is selected based on the types of soil overlaid on the tunnel [13].

\[ i = KZ_o \]  

(4)
2.4.2. Effect of ground improvement on tunnel stability. The ground improvement design was conducted in the plastic region of the soil where the soil was enhanced by increasing the shear strength [14]. Figure 7 (a) shows that when the shear strength was increased, there is no ground failure unless the achieved shear strength touches a failure line of the Mohr circle of the original ground. The radial and tangential stress are equal to each other at the elastic region’s boundary from the plastic region, as shown in Figure 7 (b).

Figure 7. (a) Mohr circle of the improved ground and original ground; (b) Cross-section of improvement zone based on the plastic theory [14]

The equation of the radial plastic region can be obtained lead to the following:

\[
\ln R + \frac{R \cdot \gamma_t}{2C} = \frac{H \cdot \gamma_t}{2C} + \ln R_0
\]

(5)

\[
t = FS \cdot (R - R_0)
\]

(6)

where
- \(R\) = radius of plastic region
- \(R_0\) = radius of tunnel
- \(\gamma_t\) = average unit weight of soil
- \(C\) = shear strength of improved soil
- \(H\) = depth to the centre of the tunnel
- \(FS\) = factor of safety
- \(t\) = thickness of the improvement zone

3. Results and discussion

3.1. Stability of cross-passage
Figure 8 shows the results of the safety factor, which increase linearly with the improvement thickness. The round marks are denoted as the PLAXIS 3D model results while the triangle marks are denoted as the improvement thickness of improvement based on the plastic theory [14]. The analytical formula of Shibazaki using for improvement in the TBM tunnel with circle improvement zone while the pattern of the soil improvement in this research is octagonal. From the graph, the safety factor of the 1.6R case is 1.70, which is acceptable since the safety factor is more than 1.40.
3.2. Behaviour of ground surface displacement

The result of the settlement profile from FEM is agreed well with the empirical equation of O'Reilly and New [13]. In the figure, three types of line represented the empirical coefficient varying depending on the soil types. The $K$ value ranges from 0.4 (stiff clay) to 0.7 (soft clay) for the cohesive soil. In this study, $K = 0.6$ fits well with the results of 3D simulation where $K = 0.6$ is obtained from the average of $K$ value based on the thickness of each type of soil above the crown of cross-passage. Overall, the empirical coefficient $K = 0.6$ is the suitable value for this case, and it also can be proved that the displacement from the 3D simulation is accurate.

![Figure 8. Stability of the cross-passage](image)

![Figure 9. Ground surface profile induced by tunnel excavation](image)
Figure 10. Comparing the surface settlement of improvement schemes 1.6R to 3.0R

Figure 11. Comparing the surface settlement of improvement 1.2R and 1.4R.

Figure 10 indicates the ground surface behaviour after the soil being improved with the improvement thickness increased from 1.6D to 3.0D. The settlement of the 1.6D improvement scheme is about 4.5mm while the settlement is decreasing significantly to less than 1.5mm in the 3.0D scheme. However, the improvement scheme of 1.2R and 1.4R shows that the vertical displacement is extensive, about ten times those, see Figure 11. The improvement scheme of 1.2R found that soil collapsed at 4.5m exaction.
4. Conclusion

Optimization based on the safety factor indicated that the ground improvement 1.6R scheme was the best choice because the safety factor is more than 1.40.

The ground surface displacement of the 1.6R improvement scheme was about 4.57mm. The ground surface settlement can be reduced by increasing the improvement radius. The validation of ground surface displacement of the finite element model and the empirical equation of O’Reilly was found to fit well when the empirical coefficient $K = 0.6$ was used.

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