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ESTABLISHING THE CORRELATION OF THE VERTICAL AND HORIZONTAL PASSIVE FAILURE PRESSURE IN FRONT OF CONSTRUCTED TBM TUNNELS FACE

Nguyen Anh Tuan,
PhD

Tran Van Dung,
M.Eng.

Hochiminh City University of Transport,
No. 2, Vo Oanh St., Ward 25, Binh Thanh Dist., Hochiminh City, Vietnam

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The paper aims to investigate the relationship between factors which have the impacts on the tunnel and the ground and establish formulas to calculate the correlation of the passive failure pressure in front of tunnel face in the vertical and horizontal directions by using the Finite Element Method (FEM).

Keywords: TBM tunnel, FEM, passive failure pressure, tunnel face, sand.

1. Introduction

The Finite Element Method (FEM) was formed from the 40s of the twentieth century and is widely used up to now to find solutions to the problem of elasticity, elasticity - plasticity, ductility - plasticity. Its advantage is taking into account the discontinuity and heterogeneity of stratigraphic structures, which can solve complex boundary problems and calculate the values of stresses - their deformation and distribution. Thanks to these distribution rules, it helps analyze the mechanism of underground construction.

At the beginning of development, numerical analysis was known as a design tool, which was strongly criticized. However, the development of information technology has made a revolution in the field of underground construction. Tunnel construction are calculated with full numerical analysis.

Currently, numerical analysis methods have been simplified and tunnel design is mainly based on experience, intuition and solutions. The underdevelopment of information technology makes the great amount of data be difficult to input and analyze. The difficulty now is solved by powerful computers, user-friendly interface. It also saves time to analyze data from weekly to daily or even hours.

The convenience of numerical analysis methods has been proven. Both the behavior of the materials and boundary conditions have been included in the calculations, and the study of parameters to improve the design of tunnel construction can be done more easily.

2. Literature review

In order to avoid ground loss which can trigger collapse of tunnel face due to instability of surrounding soil, EPB-TBM machine is designed with a closed
boring chamber creating an effective surface stabilization, this makes pressure balance of soils between inside and outside boring machine. Working principle of EPB-TBM is that the excavated soils will be mixed with water, clayey slurry or additives such as foam or polymer. This mixture will form a layer pasting on the boring chamber and creates a balancing pressure versus overburden pressure of ground, and this mixture shall be removed out through conveyors upon completion of construction.

This face pressure shall be kept stably during tunneling process. It is, however that, due to the overburden and water pressures increase from the top to bottom of the tunnel face so the supporting pressure at the bottom must be higher to ensure the balancing condition.

![Schematic representation of the tunnel face pressure control](image)

Fig. 1. Schematic representation of the tunnel face pressure control [11]

Broms & Bennermark (1967) evaluated tunnel face stability by using the ratio N which is defined as: [4]

$$N = \frac{\sigma_{ob} - \sigma_T}{S_u} = \frac{\sigma_S - \sigma_T + \gamma(C + \frac{D}{2})}{S_u}.$$ (1)

Where $\sigma_{ob}$ is the overburden pressure. And $\sigma_S$, $\sigma_T$, $C$, $D$ are defined as Fig. 2.

In the undrained case, all parameters in Equation (1) shall be assumed to be constant, excepting for face pressure $\sigma_T$. Hence, $N$ value depends on $\sigma_T$. If this supporting pressure reach ultimate value $\sigma_{ob}$ then tunnel face is completely stable which corresponds to value of $N = 0$.

Davis et. al. (1980) has found four upper bound failure modes which are dependent on tunnel structure as indicated in Fig. 3. [7]
Failure modes (1) and (2) are simple mechanisms, “tunnel roof” and “roof effect and two sides”. Mode (3) is a particular case, covering both (1) and (2). Type (4) is a mechanism with 03 variable angles, saying “tunnel roof side and bottom”.

And lower bound and upper bound stability coefficients with respect to circular tunnel in plane strain condition shall be determined following Fig. 4.

In case of layered soils or soil properties changing over depth, stability coefficient is followed underneath formula:

\[ N = \frac{\sigma_S - \sigma_T + \gamma(C+(D/2))}{c_{u0}[1+\rho(C+(D/2))]}, \quad (2) \]

If assuming stress closed to tunnel crown to be larger than calculated value, then stability coefficient will be:

\[ N_{\text{ground}} = \frac{\gamma(C+(D/2))}{c_{u0}[1+\rho(C+(D/2))]}, \quad (3) \]
According to Davis et al. (1984), with $C/D = 1.5$, lower bound and upper bound limits of $N$ vary from 2.9 to 3.4, while $N_{\text{ground}} = 1.67$.

These authors also proposed an upper bound failure mechanism of ground in front of tunnel face based on angles as $\theta_1, \theta_2, \text{ and } \theta_3$.

Where:

\[
\tan \theta_1 = \tan \theta_2 = 2\sqrt{C/D + 1/4}, \quad (4)
\]
\[
\theta_3 = \pi/2. \quad (5)
\]

### Summary of tunnel face supporting pressure [9]

| Tunnel diameter, m | Soil type | Applicable supporting pressure |
|-------------------|-----------|--------------------------------|
| **Earth Pressure Balance (EPB)** | | |
| 7.45 | Soft mud | Earth pressure |
| 8.21 | Sand, cohesive soil | Earth pressure + water pressure + 20kPa |
| 5.54 | Fine sand | Earth pressure + water pressure + surplus pressure |
| 4.93 | Sand, cohesive soil | Earth pressure + 30 to 50 kPa |
| 2.48 | Gravel, bed rock, cohesive soil | Earth pressure + water pressure |
| 7.78 | Gravel, Cohesive soil, soft soil/mud | Passive earth pressure + water pressure |
| 7.35 | Soft soil/mud | Earth pressure +10kPa |
| 5.86 | Soft cohesive soil | Earth pressure +20kPa |
| **Slurry Pressure Balance** | | |
| 6.63 | Gravel | Water pressure + 10 to 20kPa |
| 7.04 | Cohesive soil | Earth pressure |
| 6.84 | Soft cohesive soil, sandy diluvium soil | Passive earth pressure + water pressure +20kPa |
| 7.45 | Sandy soil, cohesive soil, gravel | Water pressure +30kPa |
| 10.00 | Sandy soil, cohesive soil, gravel | Water pressure +40 to 80kPa |
| 7.45 | Sandy soil | Pressure of lost soil + water pressure + surplus pressure |
| 10.58 | Sandy soil, cohesive soil | Passive earth pressure + water pressure +20kPa |
| 7.25 | Sand, gravel, soft soil | Water pressure + 30kPa |

Kanayasu (1995) [9] has summarized many tunneling projects using various methods of creating different supporting pressure of tunnel face. When
EPB shield is used, supporting pressure of tunnel face will depend on geological condition, water pressure and surplus pressure. Mortar EPB shield will be controlled by water pressure to act on active earth pressure and excess pressure. This additional pressure is to prevent unfavorable change of pressure during tunneling works. From Table 1, it can be seen that this additional pressure will be 20 kPa.

Mair (1981) [12] has presented and defined Load Factor (LF) as a ratio of stability coefficient in working condition (service state) over failure state. This factor essentially correlates to stability coefficient against failure:

\[
LF = \frac{(\sigma_{V0} - \sigma_i)}{(\sigma_{V0} - \sigma_{ic})}. \tag{6}
\]

In which \(\sigma_{ic}\) is value of \(\sigma_i\) at failure moment.

Using this coefficient \(LF\) is convenient to define safety level of tunnel face [18].

Pavlos Vardoulakis et al. (2009) [24] also implemented scaled-down models to investigate failure mechanism of front soil mass. There were 9 experiments with respect to \(C/D\) ratios of 0.5; 1 and 2 in dry sand. These experiments models a half of circular-shaped tunnel which is originally 7 m diameter. During experiment process, piston moves backward with constant speed and failure mechanism of ground in front of tunnel face is illustrated in Fig. 6. All failure modes have cylindrical-shaped and runs up to ground surface.

Chambon and Corte (1994) [6] studied failure mechanism and failure active pressure for tunnels embedded at various depth ratio \(C/D\) ranging from 0.5 - 4.0 by centrifuge tests. Tested dry unit weight of sand was 15.3-16.1 kN/m\(^3\), equivalent to void ratios of 0.65-0.92. Prototype diameter of tunnel corresponding to 5m, 10m and 13m were modeled by changing spinning speed of centrifuge machine. Fig. 7. shows the active failure mechanism of tunnels with \(C/D\) ratios of 0.5; 1.0 and 2.0. With respect to \(C/D\) of 0.5, failure mechanism climbs up to ground surface. Fig. 8. shows failure mechanism of 13m-diameter tunnel with \(C/D\) of 4.0. The observed internal collapse actually caused ground subsidence. It was found that tunnel diameter has a close relationship with failure pressure.
However, surveys on soil around the tunnel are limited. Their disadvantages are not to focus on the soil failure pressure in front of the tunnel face and provide the relationship between factors which have the impacts on the tunnel and the ground. Therefore, the paper helps establish formulas to calculate the correlation of the passive failure pressure in front of tunnel face in the vertical and horizontal directions.

2. Numerical simulation

2.1. Material parameters

Soil parameters for sand and clay and TBM is shown as Table 2.

| Geological parameters | Sand | Clay |
|-----------------------|------|------|
| Saturated unit weight, $\gamma_{sat}$ (kN/m$^3$) | 20.3 | 21.1 |
| Unsaturated unit weight, $\gamma_{unsat}$ (kN/m$^3$) | 19.5 | 20  |
| Cohesion intercept, $c$ (kPa) | 1.0  | 300  |
| Angle of friction, $\varphi$ (degree) | 30°  | 1.0  |
| Angle of dilation, $\psi$ (degree) | 0°   | 0    |
| Young modulus, $E_{50}$ (MPa) | 27   | 100  |
| Unloading and reloading modulus, $E_{ur}$ (MPa) | 81   | 300  |
| Oedometer modulus, $E_{oed}$ (MPa) | 27   | 100  |
| Poisson’s ratio, $\nu$ | 0.3  | 0.3  |
| $m$ | 0.5  | 1.0  |
| $R_f$ | 0.9  |      |

2.2. Analysis

The tunnel with the 5m diameter is stimulated for cases with $C/D$ ratio of 1.5; 2.0; 2.5; 3.3 and 4.0 respectively by using PLAXIS 3D TUNNEL software.
Due to symmetry, only a half of the tunnel was stimulated in this model. The model extended 20m in the z-direction, with the width and depth of 30m and 50.5m respectively. This model is large enough to allow any collapse mechanism to evolve and avoid significantly influence on the boundary of the model.

The interaction between the TBM and soil is defined by the boundary. During excavation, the tunnel pressure is put in the z-direction.

2.3. Establishing the correlation of passive failure pressure between vertical and horizontal stress in front of the tunnel face

The calculated figures are shown in Table 2.

| C/D | \(\sigma_{zz}/\sigma_{yy}\) ratio |
|-----|-------------------------------|
| 1.5  | \(\sigma_{yy}\) | \(\sigma_{zz}\) |
| 14.7 | 11.0 | 45.5 | 33.1 |
| 71.0 | 49.0 | 34.6 | 22.8 |
| 60.2 | 41.5 | 92.4 | 62.2 |
| 65.3 | 44.4 | 94.8 | 63.0 |
| 153.3 | 105.6 | 158.2 | 107.9 |
| 152.6 | 108.5 | 185.4 | 128.3 |

| 2.5  | \(\sigma_{zz}/\sigma_{yy}\) ratio |
|-----|-------------------------------|
| \(\sigma_{yy}/\sigma_{zz}\) | \(\sigma_{zz}/\sigma_{yy}\) |
| 0.749006 | 0.728074 |
| 0.690680 | 0.660592 |
| 0.688416 | 0.672616 |
| 0.679980 | 0.664796 |
| 0.689154 | 0.682144 |
| 0.711103 | 0.691804 |

Let \(K\) be a variable depending on the depth of tunnel \(C\) and diameter of tunnel \(D\), the relationship of stresses in two directions is shown through \(K\) as follows:

\[
\sigma_T = K \sigma_z. \tag{7}
\]

To determine the relationship between \(K\) and \(C/D, K_1, K_2, K_3, K_4, K_5\) and \(K_6\) is seen as the sections at 8.38406 m, 9.97974 m, 11.38978 m, 13.64642 m, 15.64052 m, 18.83188 m respectively in front of the tunnel face in Z axis.

Based on the data in Table 3, the charts between the parameters are shown in Fig. (10-15).
Table 3

Coefficients $K_1$, $K_2$, $K_3$, $K_4$, $K_5$ and $K_6$

| $C/D$ | $K_1$  | $K_2$  | $K_3$  | $K_4$  | $K_5$  | $K_6$  |
|-------|--------|--------|--------|--------|--------|--------|
| 4     | 0.622753 | 0.597708 | 0.605556 | 0.606562 | 0.592043 | 0.587409 |
| 3.3   | 0.675334 | 0.661056 | 0.665371 | 0.667535 | 0.683163 | 0.658096 |
| 2.5   | 0.71722  | 0.664892 | 0.681638 | 0.683047 | 0.692097 | 0.686197 |
| 2     | 0.728074 | 0.660592 | 0.672616 | 0.664796 | 0.682144 | 0.691804 |
| 1.5   | 0.749006 | 0.69068  | 0.688416 | 0.67998  | 0.689154 | 0.711103 |

Fig. 10. Relationship chart between $K_1$ and $C/D$

Fig. 11. Relationship chart between $K_2$ and $C/D$

Fig. 12. Relationship chart between $K_3$ and $C/D$
Let the general formula $K$ be:

$$K = A_1 \frac{C}{D} + A_2.$$  \hspace{1cm} (8)

$K$ will change with each cross-sectional area in front of the tunnel face with the coefficients $A$ and $B$ in the graphs in Fig. 10 to Fig. 15, we sum them up into Table 4.
Table 4

Coefficients $A_1$, $A_2$ and $K_1$-$K_6$

| Coefficient | $A_1$   | $A_2$   | $z$, m  |
|-------------|---------|---------|---------|
| $K_1$       | -0.0492 | 0.8294  | 8.38406 |
| $K_2$       | -0.0298 | 0.7343  | 9.97974 |
| $K_3$       | -0.0285 | 0.7385  | 11.38978|
| $K_4$       | -0.0241 | 0.7246  | 13.64642|
| $K_5$       | -0.0324 | 0.7538  | 15.64052|
| $K_6$       | -0.0456 | 0.7882  | 18.83188|

Based on the data in Table 4, we use the chart to show the relationship between the coefficient $A_1$, $A_2$ and the sections in front of the tunnel face. The charts are shown in Fig. 16 and Fig. 17.

![Chart of $A_1$ along the vertical axis](image1.png)

Fig. 16. Chart of $A_1$ along the vertical axis

![Chart of $A_2$ along the vertical axis](image2.png)

Fig. 17. Chart of $A_2$ along the vertical axis

The relationship between $K$ and $C/D$:

$$K = (-0.0008x^2 + 0.0214x - 0.1697)\frac{C}{D} + 0.0029x^2 - 0.0795x + 1.2719. \quad (9)$$

After changing the numbers, we have a table of $K$ coefficient corresponding to the different depths of tunnels as in Table 5.
Table 5

Coefficients $K_{1.5}$, $K_2$, $K_{2.5}$, $K_{3.3}$ and $K_4$

| $z$, m  | $K_{1.5}$ | $K_2$  | $K_{2.5}$ | $K_{3.3}$ | $K_4$  |
|---------|-----------|--------|-----------|-----------|--------|
| 8.38406 | 0.739443  | 0.716185 | 0.692928  | 0.655716  | 0.623155 |
| 9.97974 | 0.713622  | 0.695717 | 0.677812  | 0.649165  | 0.624098 |
| 11.38978| 0.69801   | 0.68314 | 0.66827   | 0.644748  | 0.62366 |
| 13.64642| 0.687092  | 0.67369 | 0.660445  | 0.639128  | 0.620476 |
| 15.64052| 0.691853  | 0.67607 | 0.66116   | 0.636605  | 0.615119 |
| 18.83188| 0.727606  | 0.702402| 0.67197   | 0.636869  | 0.601583 |

The formula shows the relationship of stress in vertical and horizontal directions:

$$ \sigma_z = \left[ (-0.0008x^2 + 0.0214x - 0.1697) \frac{C}{D} + 0.0029x^2 - 0.0795x + 1.2719 \right] \sigma_y. \quad (10) $$

Table 6

Values $\sigma_y$ in FEM

| $z$, m  | $\sigma_y$ kN/m² |
|---------|------------------|
|         | $C/D=1.5$ | $C/D=2$  | $C/D=2.5$ | $C/D=3.3$ | $C/D=4$  |
|---------|-----------|----------|-----------|-----------|----------|
| 8.38406 | 14.70697  | 45.48453 | 51.75698  | 33.42773  | 30.72395 |
| 9.97974 | 70.99307  | 34.55252 | 31.36566  | 66.1367   | 69.47639 |
| 11.38978| 60.24304  | 92.43649 | 99.00351  | 98.25886  | 117.2908 |
| 13.64642| 65.28522  | 94.81899 | 120.7609  | 153.3606  | 213.5641 |
| 15.64052| 153.2471  | 158.2327 | 169.1545  | 208.3287  | 296.7107 |
| 18.83188| 152.6218  | 185.3962 | 218.5622  | 278.1142  | 338.1551 |

Based on Table 6 and combined with Equation (10), we find $\sigma_z$ as Table 7.

Table 7

Stress in horizontal direction $\sigma_z$ calculated by Equation (10)

| $z$, m  | $\sigma_z$ kN/m² |
|---------|------------------|
|         | $C/D=1.5$ | $C/D=2$  | $C/D=2.5$ | $C/D=3.3$ | $C/D=4$  |
|---------|-----------|----------|-----------|-----------|----------|
| 8.38406 | 10.87496  | 32.57353 | 35.86385  | 21.91909  | 19.14578 |
| 9.97974 | 50.66223  | 24.03879 | 21.26     | 42.93614  | 43.36007 |
| 11.38978| 42.05027  | 63.14709 | 66.16108  | 63.32565  | 73.14954 |
| 13.64642| 44.85694  | 63.88606 | 79.75595  | 98.01708  | 132.5113 |
| 15.64052| 106.0245  | 107.0455 | 111.8382  | 132.6231  | 182.5125 |
| 18.83188| 111.0486  | 130.2226 | 148.0096  | 177.1224  | 203.4282 |

After having results from Equation (10), we compare the figures with the results from FEM. The comparison are shown in Fig. 18 to Fig. 22.
Fig. 18. Deviation $\sigma_z$ by FEM and Equation (10) at $C/D=1.5$

Fig. 19. Deviation $\sigma_z$ by FEM and Equation (10) at $C/D=2.0$

Fig. 20. Deviation $\sigma_z$ by FEM and Equation (10) at $C/D=2.5$

Fig. 21. Deviation $\sigma_z$ by FEM and Equation (10) at $C/D=3.0$
It can be seen that the calculated results of stress in vertical and horizontal direction in front of tunnel face from Formula (10) and results from FEM are approximately the same. Thus, under similar geological conditions, we can use the proposed formula by the author to calculate the passive failure pressures in front of the tunnel face.

3. Conclusion
From the results of the calculation, the following conclusions can be drawn:
- It is important to determine the tunnel pressure during the construction process, the greater the depth of tunnel is, the greater the stress in front of tunnel face is. Therefore, it is necessary to calculate the minimum amount of bentonite to avoid the instability of tunnel face.
- For the small errors, it can refer to the given formula to determine the stress in front of tunnel face in the vertical and horizontal directions. However, it is also necessary to refer to results from other methods to obtain the most accurate data because this study only consider a certain geological form, which can result in some mistakes. In addition, the paper has not mentioned changes in groundwater level, rainfall and the existing load on the ground.

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Fig. 22. Ref. 27.