Determining Optimal Thrust Force of EPB Shield Machine by Analytical Solution

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ABSTRACT

Earth pressure balanced TBM provides active support for working face by means of a pressurized earth paste with the consistency of a highly viscous liquid. The earth paste is produced from the excavated soil mixed with additives. The supporting pressure is achieved through control of the incoming and outgoing materials in the chamber. The soil-structure interaction in shield tunneling is investigated by analytical solution. The mechanism of working face stability is analyzed and the reasonable face earth pressure for EPB shield is deduced according to the active and passive earth pressure principles. The optimal thrust fore for EPB shield is proposed in different soil parameter and shield size cases. The comparison of the practical thrust forces of EPB shields with computed ones shows that the proposed computing procedure for optimal thrust fore for EPB shield can agree well with practical engineering examples. The effectiveness of the proposed computing procedure is validated.

KEYWORDS: Analytical solution; soil-structure interaction; shield tunnelling; Earth pressure balanced TBM; reasonable face earth pressure; optimal thrust force.

INTRODUCTION

The use of a tunnel boring machine (TBM) has become the first choice when planning the excavation of a circular tunnel in urban environment. The basic advantages of TBMs are high safety and rapid excavation speed, with low over-break (extra-excavation) and low manpower. TBM tunneling also has only a very small effect on the surrounding soil mass and constructions. The history of TBMs extends back to the beginning of the 19th century, and improvements have been made gradually but continually until today. However, the advantages and disadvantages mentioned above still remain to a greater or lesser degree. These features of TBMs give rise to very good excavation rates under ideal conditions, yet poor performance under adverse conditions. In the conventional tunneling method (drill and blast), high excavation rates cannot be achieved even under ideal conditions, regardless of the number of workers employed, because a consistently high work rate cannot be realized. In comparison, using a TBM, 100 meters per day is readily achievable under ideal conditions with continuous operation, however under adverse conditions; the penetration rate could fall radically. It is therefore essential to clarify in advance whether a TBM is suitable for a particular project, and in some cases a feasibility study may be necessary [1].

Shield tunneling has become a well-established tunnel construction method in various ground conditions. It is characterized by relatively complex interactions between the soil, the tunnel boring machine (TBM), the hydraulic jacks, the tunnel lining and the tail void grout [2]. It is sometimes difficult to clearly identify correlations between key parameters from measurement data due to the varying conditions of the measurements and the resulting large scatter. If realistic numerical models are used, simulations represent a useful tool to identify and quantify such correlations. To choose a constitutive model, one must verify its ability to represent the real behaviour of the soil under the considered loading. Excavating a tunnel leads to a complex loading which will generate in particular a rotation of principal stresses. Two soil constitutive models are used in past study: the elastic-perfectly-plastic Mohr-Coulomb model and an elastic-plastic model developed in Ecole Centrale de Lyon. The Duncan model for describing deformation behaviors of soils uses isotropic hyperbolic stress-strain relationship. The elastic parameters from this relationship vary according to the stress state. The Young's modulus increases with confining pressure and decreases with increasing shear stress. In a finite element simulation of a geotechnical problem, calibrations of the models used to reproduce soil behavior often pose significant challenges. Real soil is a highly nonlinear material, with both strength and stiffness depending on stress and strain levels. Numerous constitutive models have been developed that can capture many of the important features of soil behavior. An effective and more objective way to calibrate a soil model employs inverse analysis techniques to minimize the difference between experimental data (laboratory or field tests) and numerically computed results. Kasper (2006) modeled the advancement of the step-by-step tunnel construction process by using a three-dimensional finite element model, which takes into account all relevant components of shield tunneling. The material behaviour of the saturated soil and the tail void grout is modelled by a two-field finite element formulation in conjunction with an elasto-plastic Cam-Clay model for the soil and a hydration-dependent constitutive model for the grout. The analyses provide valuable information with regard to the significance of the investigated parameters and demonstrate the complexity of the various interactions in shield tunneling[2]. Bernat (1998) implemented a numerical simulation using a finite element method in the aim of developing a procedure to predict the movement induced by shield tunneling in soft soil. [3]. Plasticity models for frictional materials such as concrete, soil and rock are most conveniently represented in principal stress space. The general characteristics of the yield surface are described by its cross-sectional shape on the deviatoric plane and its trace on meridian planes. In frictional materials the form of the failure envelope is significantly affected by the value of the confining pressure. On a meridian plane the shape of the failure surface can curve like a parabola, while on the deviatoric plane the shape can vary from a curvilinear triangle at low confining pressures to nearly circular at high confining pressures. A classical plasticity model that takes into account the shape of the failure surface on the deviatoric plane is the Mohr–Coulomb (MC) model [4]. The Mohr-Coulomb criterion has been extensively used to model the static and dynamic behaviour of retaining walls in both analytical and numerical studies, the main advantage being its simplicity. In the field of computational Geomechanics, more sophisticated soil models have been proposed with take account of the inter-mediate principal stress, giving rise to an increase in the equivalent friction angle when analyzing plane strain problems [5]. The aims of the paper are to analyze the mechanism of working face stability, to deduce the reasonable face earth pressure for EPB shield, to propose the optimal thrust fore for EPB shield, to compare the practical thrust forces of EPB shields with computed ones through the practical engineering examples and to validate effectiveness of the proposed computing procedure.

COMPUTATION OF WORKING FACE EARTH PRESSURE ON FRONT CHAMBER OF EPB SHIELD

Earth pressure balance shields provide continuous support of the tunnel face using freshly excavated soil, which completely fills up the work chamber under pressure. The supporting pressure is achieved through control of the incoming and outgoing materials in the chamber, i.e., through regulation of the screw conveyor rotation and the excavation advance rate. The earth pressure of working face is classified into two kinds of types. The first class is active earth pressure, which is from the active action of earth pressure to the shield structure if the thrust force of shield is less than composite force of active earth pressure. The second class is passive earth pressure, which is from the active action of shield structure to the earth mass in front of working face if the thrust force of shield is greater than composite force of active earth pressure. Active earth pressure is a classical geotechnical problem, which has been investigated extensively in the literature. The methods to determine active pressure can generally be divided into three groups, i.e. limit equilibrium method, limit analysis and slip line method. In limit equilibrium method, some assumptions were introduced to make the problem determinate. Based on the limit equilibrium of the slipping wedge, lateral earth pressure was to be determined as an external force. This method can yield an analytical solution, which can be used easily by the engineers but a good precision cannot be guaranteed. In order to obtain an exact solution of the problem, a combination of stress equations, the equations of motion and boundary conditions needs to be solved.

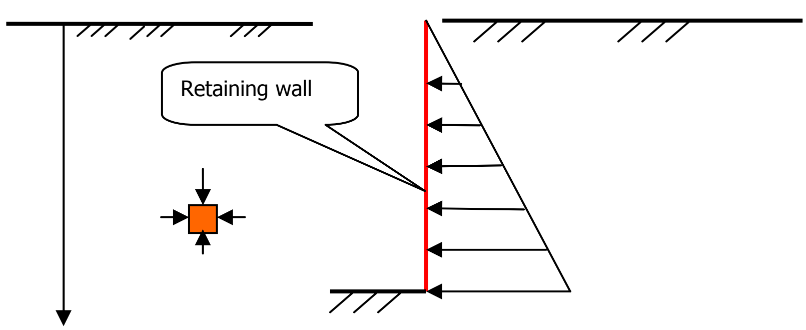

Figure 1: Stress state of earth mass and earth pressure on retaining wall

As shown in Figure 1, the vertical earth pressure is expressed as follows:

$$\sigma_z = \gamma z \tag{1}$$

Where σ_z is vertical earth pressure, z is deposit depth, γ is unit weight of soil. Horizontal earth pressure can be found

$$\sigma_x = k_0 \gamma z \tag{2}$$

$$K_o = 1 - \sin \varphi \tag{3}$$

Where K_o is the coefficient of earth pressure at rest, φ is drained friction angle. As approximate expression, the coefficient of earth pressure can be determined as follows: K_o = 0.34～0.45 for sandy soil, and K_o =0.50～0.70 for clay soil. Consider a semi-infinite mass of soil with a horizontal surface and having a vertical boundary formed by s smooth wall surface extending to semi-infinite depth, as shown in Figure 1. The soil is assumed to be homogeneous and isotropic. A soil element as ant depth z is subjected to a vertical stress σ_z and a horizontal stress σ_x and, since there can be no lateral transfer of weight if the surface is horizontal; no shear stresses exist on horizontal and vertical planes. The vertical and horizontal stresses are principal stresses. If there is now a movement of wall away from the soil, the value of σ_x decreases as the soil dilates or expands outwards, the decrease in σ_x being an unknown function of the lateral strain in the soil. If the expansion is large enough the value of σ_x decreases to a minimum value such that a state of plastic equilibrium develops. Since this state is developed by a decrease in the horizontal stress σ_x, this must be the minor principal stress (σ_3). The vertical stresses σ_z is then the major principal stress (σ_1).

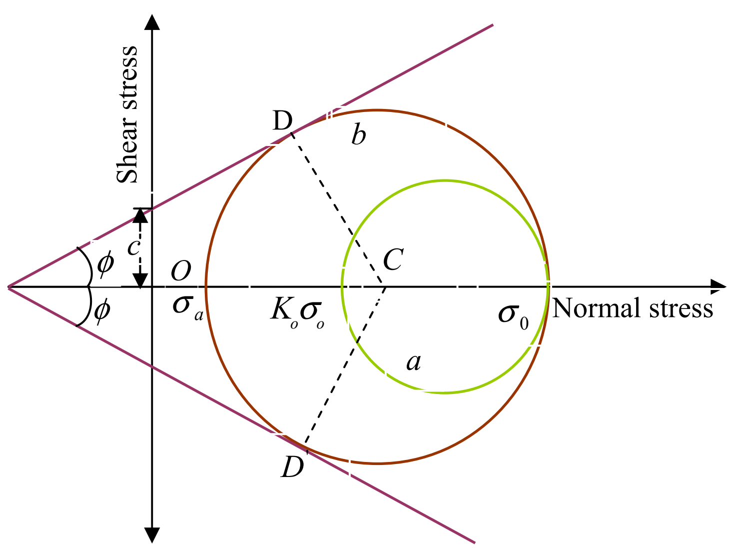

Figure 2: Diagram for computing active earth pressure

As shown in Figure 2, stress condition for a smooth and vertical retaining wall is given by circle “a” before the wall moves. State of Plastic equilibrium is represented by circle “b”. This is the “Rankine’s active state.” For a vertical and smooth retaining wall with a horizontal backfill, the Rankine’s active earth pressure σ_a, which depends on the internal friction angle φ and the cohesion c of the backfill soil, can be expressed as

$$\sigma_a = \gamma z \tan^2(45° - \varphi/2) - 2c\tan(45° - \varphi/2) \tag{4}$$

$$\sigma_a = \gamma z K_a - 2c\sqrt{K_a} \tag{5}$$

$$K_a = \tan^2(45° - \varphi/2) \tag{6}$$

Where σ_a is active earth pressure, K_a is the coefficient of active earth pressure. The angle between the failure planes slip planes and major principal plane (horizontal) is

$$\vartheta = \pm\left(45 + \tfrac{\varphi}{2}\right) \tag{7}$$

If the wall is moved against the soil mass there will be lateral compression of the soil and value of σ_x will increase until a state of plastic equilibrium is reached. For this condition σ_x becomes a maximum value and is the major principal stress σ_1. The stress σ_z equal to the total overburden pressure, is then the minor principal stress. The maximum value $\boldsymbol{\sigma_1}$ is reached when the Mohr circle through the point representing the fixed value σ_3 touches the failure envelope for the soil. In this case the horizontal stress is defined as the passive earth pressure, representing the maximum inherent resistance of the soil to lateral compression.

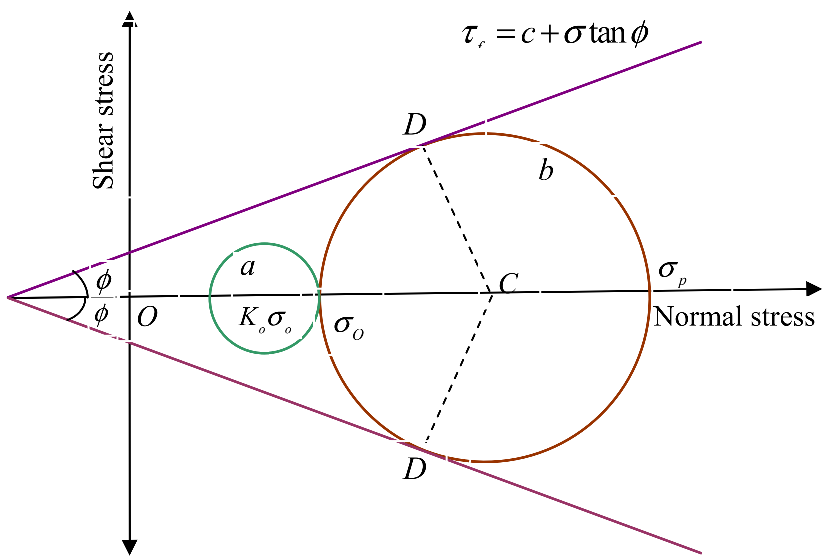

Figure 3: Diagram for computing passive earth pressure

As shown in Figure 3, Circle "*a*" gives initial state stress condition, "Rankine's passive state" is represented by circle "*b*". The Rankine's passive earth pressure is given by:

$$\sigma_p = \gamma z \tan^2(45° + \varphi/2) + 2c\tan(45° + \varphi/2) \tag{8}$$

$$\sigma_p = \gamma z K_p + 2c\sqrt{K_p} \tag{9}$$

$$K_p = \tan^2(45° + \varphi / 2) \tag{10}$$

Where σ_p is passive earth pressure, K_p is the coefficient of passive earth pressure. The angle between the failure planes slip planes and major principal plane (horizontal) is:

$$\vartheta = \pm\left(45\text{-}\tfrac{\varphi}{2}\right) \tag{11}$$

In order to control effectively ground surface settlement, the optimal face earth pressure on front chamber of EPB shield should is larger than active earth pressure and less than passive earth pressure. The optimal face earth pressure on front chamber of EPB shield can be determined by

$$\sigma_a \le \sigma_{opt} \le \sigma_p \tag{12}$$

Where σ_{opt} is optimal face earth pressure.

COMPUTATION OF OPTIMAL THRUST FORCE OF EPB SHIELD

According to the mechanical balance principle, as shown in Figure 4, the thrust fore of EPB shield is mainly balanced by the active or passive earth pressure and friction resistance force.

$$F = F_p + F_f \tag{13}$$

Where F is the thrust force of EPB shield, F_p is composite force of earth pressure, F_f is friction resistance force. The minimum composite force of earth pressure, which is balanced by the active earth pressure as shown in Figure 5, can be expressed as follows

$$\begin{aligned} F_{p\min} &= 2\int_0^R \int_0^\pi [\gamma z k_a - 2c\sqrt{k_a}] r d\theta dr \\ &= 2\int_0^R \int_0^\pi [\gamma(H + R + r\cos\theta)k_a - 2c\sqrt{k_a}] r d\theta dr \\ &= \pi R^2[\gamma(H+R)k_a - 2c\sqrt{k_a}] \end{aligned} \tag{14}$$

Where L is the length of EPB shield, R is the radius of shield, H is the deposit depth of shield. The maximum composite force of earth pressure, which is balanced by the passive earth pressure as shown in Figure 5, can be expressed as follows:

$$F_{p\max} = 2\int_0^R\int_0^\pi [\gamma z k_p + 2c\sqrt{k_p}]r d\theta dr$$
$$= 2\int_0^R\int_0^\pi [\gamma(H + R + r\cos\theta)k_p + 2c\sqrt{k_p}]r d\theta dr \qquad (15)$$
$$= \pi R^2[\gamma(H+R)k_p + 2c\sqrt{k_p}]$$

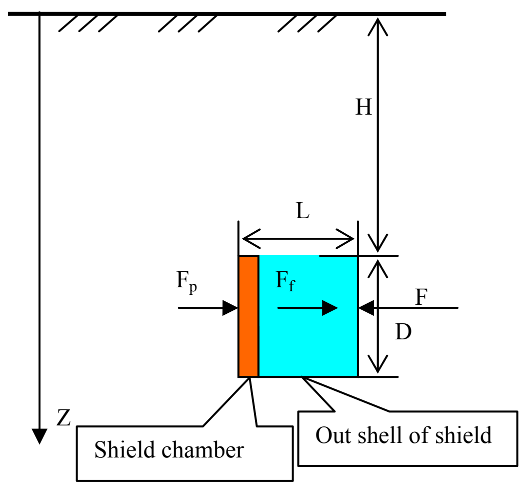

Figure 4: Diagram for computing force balance of EPB shield

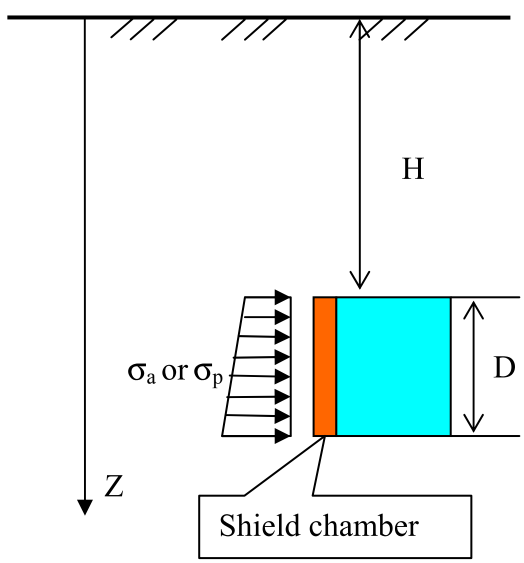

Figure 5: Earth pressure on working face of EPB shield

Table 1: Comparison of the practical thrust forces (F_{prac}) of EPB shields with computed ones (F_{comp})

No	c (kPa)	φ/°	H (m)	D (m)	L (m)	F_{prac} (MN)	F_{comp} (MN)
1	11.7	16.4	8.4	6.34	6.54	14.00	5.90-16.38
2	46.3	30.0	13.8	6.4	6.54	31.56	5.09-40.64
3	68.0	30.3	24.0	6.25	7.74	33.00	7.85-62.42
4	11.9	13.8	11.9	6.34	6.54	14.00	8.42-19.56

As shown in Figure 6, resistance force from fraction between the shell of EPB shield and soil mass can be expressed as follows:

$$\begin{aligned} F_f &= 2\int_0^L\int_0^\pi f(\sigma_z+\sigma_x)Rd\theta dy \\ &= 2\int_0^L\int_0^\pi f[\gamma(2-\sin\varphi)(H+R+R\cos\theta)]Rd\theta dy \\ &= 2\pi f\gamma LR(2-\sin\varphi)(H+R) \end{aligned} \tag{16}$$

Where f is fraction coefficient between the shell of EPB shield and soil mass, f=0.2-0.3. Table 1 lists the comparison of the practical thrust forces (F_{prac}) of EPB shields with computed ones (F_{comp}).

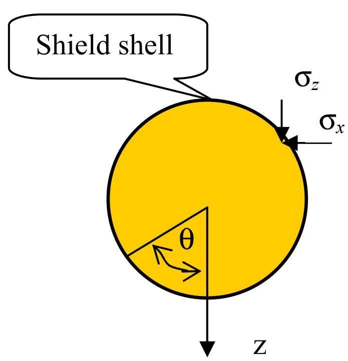

Figure 6: Earth pressure on the shell of EPB shield

CONCLUSION

Earth Pressure Balance shield tunneling has been successfully adopted in urban environments in recent years for very different ground conditions and at present it can be considered the most commons used mechanized tunneling technology. It is important to evaluate accurately the load acting on a shield machine to facilitate its economical and rational design. The reasonable face earth pressures calculated by using Rankine's active and passive earth pressure are generally adopted for the design of tunnel excavation in order to ensure face stability conditions. The proposed analytical procedure for choosing reasonable face support pressure and thrust force proved to be a very effective improvement of the actual applied EPB shield technology.

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