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Tunnel Face Stability in the Soil Medium Under Collapse and Blow-up Conditions

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Abstract. Earth Pressure Balance Tunnel Boring Machines (EPB TBM) are widely used for tunnel excavation in soft cohesive soil medium. The overall safety of the tunneling operations largely depends on the stability of the excavated face. EPB shield provides the required support pressure to the excavated face. The face will collapse if the applied pressure is less than the minimum required, and the ground in front of the TBM will blow up if the support pressure is too high. As a result, correct assessment of face pressure is extremely important for face stability. A 3D wedge-prism model has been considered for the face pressure calculation under the Limit Equilibrium Method (LEM). A range of face pressure has been calculated considering the collapse of the tunnel face and blow-up of the ground. Face pressure calculated by the present method is in close agreement with other researchers. A detailed parametric study has been carried out to explore the impacts of the soil property on the face pressure.

Keywords: tunnel; face-pressure; stability; wedge-prism model; limit-equilibrium-method

1 Introduction

Earth Pressure Balance (EPB) Tunnel Boring Machines (TBM) are widely used for tunneling in the soil medium. For a safe tunneling operation in congested areas, one must ensure the stability of the tunnel face, especially in the case of a shallow tunnel. Appropriate face pressure must be applied and maintained at the tunnel face during the tunneling operation to make the tunnel face stable. The present analytical methods available for calculating face pressure can be broadly classified into two categories, (I) Limit Analysis Method (LAM) and (II) Limit Equilibrium method (LEM). Broms and Bennermark, (1967) described the stability of unsupported vertical cut in undrained cohesive material with the help of stability number (N). Atkinson et al. (1977) determined the collapse pressure using upper bound and lower bound theorems assuming a plain strain condition and found that the theoretical solution is in close agreement with the experimental study. Krause et al. (1987) provided upper bound and lower bound stability solutions for collapse under undrained conditions and showed the variation of critical stability number (N) with the depth of burial (C). Davis et al. (1980)derived the face pressure in LEM assuming circular and spherical failure mechanisms. Chambon and Corte (1994) performed a series of centrifuge model tests and found the limiting internal support pressure for different soil conditions. The authors found that in dry sand, hydrostatic pressure is sufficient to provide stability to the face of the tunnel; though the face is not self-stabilizing, a small uniform pressure of 10 kPa is required to ensure stability. It is important to note that the authors found that the pressure at the failure of the face does not vary with the different cover to diameter ratios (C/D) and soil density. Anagnostou and Kovari (1996) analyzed the stability of the face under the EPB TBM operation considering the limit equilibrium of a wedge and a prismatic body which are defined by the slip surfaces beginning at the face and reaching the soil surface, and concluded that the head difference between the ground and the chamber should be kept as small as possible to ensure face stability. Broere (2001) analyzed the face stability considering LEM for the wedge model. Non-homogeneity of the soil layers in front of the tunnel face has been taken into consideration in this model. Shahmoradi et al., (2020) extended the work of (Broere,

2001) and gave an analytical formula to determine the face pressure for the non-homogeneous soil layers at an inclination with the horizontal.

In the present work, an analytical formulation has been derived under the limit equilibrium method (LEM). To determine the range of face pressure at which the tunnel face will be stable, two conditions have been considered Collapse condition (Minimum face pressure) and blow-out condition (Maximum Face Pressure). A parametric study has been carried out to study the effect of the major parameters influencing face pressure.

2 Face Pressure Model

In the limit equilibrium method (LEM), a predefined failure surface is assumed, and analysis is carried out considering the equilibrium of the forces. To develop the face pressure model following assumptions have been made,

a) Soil medium has been considered an Isotropic, homogeneous, and linear elastoplastic material which follows Mohr-Coulomb failure criteria.
b) Length and width of the triangular wedge have been derived considering the equivalent area of the tunnel [$B = \frac{\sqrt{\pi}}{2} D$, Where B = length of the triangular wedge and D = Diameter of the tunnel]
c) The face pressure for the collapse condition has been determined considering the downward movement tendency of the triangular wedge ABCDEF and the face pressure for the blow-out condition has been determined by considering the upward movement of the triangular wedge ABCDEF.
d) The failure plane BCFE has made an angle α with the horizontal
e) For the collapse condition, the frictional force (T) and shear force (F_c) have been considered, acting in the upward direction at an inclination of α with the horizontal and in the blowout condition, these forces have been considered to act in the opposite direction.
f) For calculating surcharge load F_{σ_v} on ACFD, arching effect of soil has been taken into account
g) Water table effect has been considered by incorporating the hydrostatic pressure in front of the tunnel face.

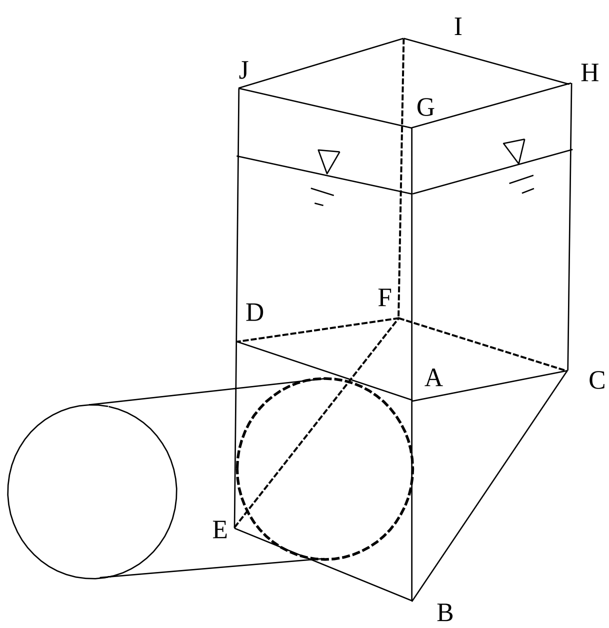

Fig. 1. Wedge model for face pressure calculation

h) The frictional forces on the triangular vertical faces (ABC and DEF) have been calculated by assuming the mean vertical stress at the tunnel center line multiplied by the coefficient of earth pressures. In collapse condition, a minimum face pressure is applied at the tunnel face, resulting in loosening soil in the tunnel face. Thus, for the collapse condition, the coefficient of active earth pressure (K_a) is used, whereas, in the case of the blowout condition, soil in the tunnel face densifies. As a result, the coefficient of passive earth pressure has been used (K_p).

3 Derivation of Face Pressure for collapse and blowout condition

Face pressure for the collapse and blowout condition have been derived by the limit equilibrium method (LEM), considering the limiting equilibrium of the triangular wedge (ABCDEF). In collapse condition, a minimum face pressure $F_{collapse}$ is applied at the tunnel face, which leads the triangular wedge to move in the downward direction. The forces acting on the wedge under collapse condition has been shown in **Fig. 2** (left). Under blowout condition, the face pressure $F_{blowout}$ applied at the tunnel face leads the triangular wedge tend to move upward direction and the forces acting on the wedge shown in **Fig. 2** (Right).

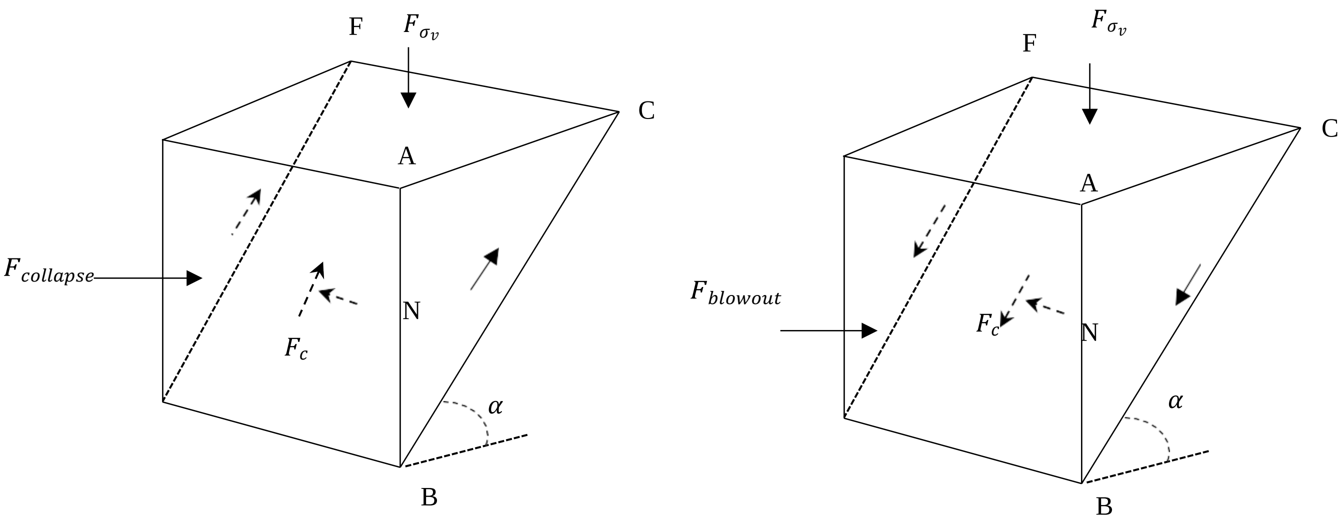

Fig. 2. Forces acting on the triangular wedge for collapse condition (left) and Blowout condition (right)

3.1 Collapse Condition

Considering the horizontal equilibrium of the triangular wedge $\sum F_h = 0$

$$F_{collapse} + (2T + F_c)cos\alpha - Wp - Nsin\alpha = 0 \tag{1}$$

Considering the vertical equilibrium of the wedge $\sum F_v$ = 0

$$Ncos\alpha + (2T + F_c)sin\alpha = W + \sigma_v \tag{2}$$

Where, Shear force (F_r) acting on the slating face EBFC, can be represented by equation (3)

$$F_r = C + Ntan\varphi \tag{3}$$

Putting the value of F_r from equation (3) into equation (2), The normal reaction force acting on the inclined face EBCF, has been determined as shown in equation (4)

$$N = \frac{\sigma_v + W - sin\alpha(2T + C)}{cos\alpha + sin\alpha.tan\varphi} \tag{4}$$

Thus, using equations (1) and (4), the face pressure for the collapse condition can be determined.
Where, $F_{collapse}$ = Face pressure acting on the tunnel face σ_v = Vertical load of the prism acting on the face ACFD, W = Self weight of the triangular wedge ABCEFD, α = Wedge angle with the horizontal, T = Frictional forces acting on the vertical triangular faces, W_p = Hydrostatic pressure force acting on the tunnel face, φ = Friction angle of soil, C = Cohesive force acting on the inclined face, F_r = Shear force

$$\sigma_v = (B^2.cot\alpha)[\frac{B(\gamma - \frac{c}{B})}{k_0.tan\varphi}(1 - \exp(-\frac{z.k_0.tan\varphi.}{B})) + (H - z).\exp(-\frac{k_0.tan\varphi.(H - z)}{B})] \tag{5}$$

$$T = \frac{B^2 (h + \frac{D}{2}).\gamma.K_a.cot\alpha}{2} \tag{6}$$

$$W_p = \frac{h_w\gamma_w.\pi.D^2}{4} \tag{7}$$

$$C = \frac{c.B^2}{sin\alpha} \tag{8}$$

Where, γ = Unit weight of soil, h = Tunnel cover, γ_w = Unit weight of water, c = Cohesion K_a = Coefficient of active earth pressure, k_0= Coefficient of earth pressure at rest

3.2 Blowout Condition

Considering the horizontal equilibrium of the triangular wedge $\sum F_h = 0$

$$F_{blowout} = N.sin\alpha + (2T + F_c).cos\alpha + W_p \tag{9}$$

Considering the vertical equilibrium of the wedge $\sum F_v = 0$

$$N.cos\alpha = (W + \sigma_v) + (2T + F_c).sin\alpha \tag{10}$$

From equations (8) and (9) the normal vertical force acting on the inclined plane EBCF can be found as,

$$N = \frac{(\sigma_v + W) + sin\alpha(C + 2T)}{cos\alpha - tan\varphi.sin\alpha} \tag{11}$$

From equation (9) and (11), face pressure for the blowout condition can be determined.

4 Validation of the present model with the existing model

For the tunnel face stability two face pressures have been calculated using collapse and blowout condition of the tunnel face. Thus, in this study a range of face pressure have been calculated, within this range TBM operation can be considered as safe. The parameters used to calculate the face pressure has been shown in **Table 1**. Same soil parameters have been used to calculate the face pressure for the existing models of the earlier researchers. The details of the existing models for calculating face pressure have been illustrated in **Table 2**.

The face pressure calculated by the present model has been compared with the existing model as shown in **Fig. 3**. It can be seen that the face pressure calculated by the other researchers are within the range of calculated face pressure for collapse condition and blow-out condition of the present model for different C/D ratio. The face pressure calculated by the present model for collapse condition is well comparable with the existing model though the difference with the blow-out condition is very high for a higher C/D ratio. It can be noted that the face pressure calculated for the blow-out condition by the present model is in good agreement with the blow-out pressure predicted by (Mollon et al., 2013)

Table 1. Parameters used for calculating the face Pressure

Diameter (m)	Depth (m)	Unit weight (kN/m^3)	Cohesion (kN/m^2)	Friction Angle (°)	Wedge Angle (°)		Coefficient of Earth Pressure	
					Collapse	Blowout	Collapse (K_a)	Blowout (K_p)
6	9	18	40	30	60	30	0.33	3

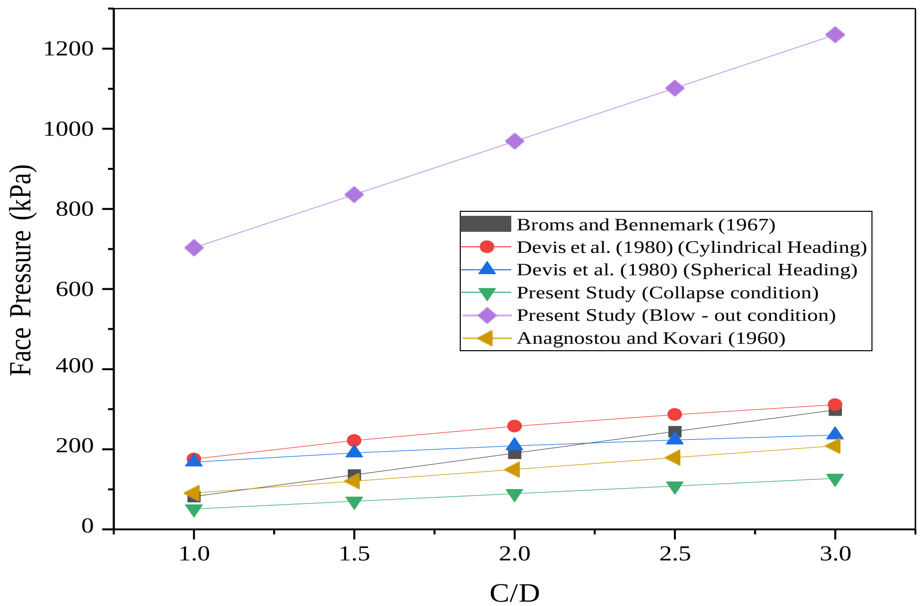

Fig. 3 Validation of the Present model with existing model

Table 2. Models of the earlier researcher to calculate face Pressure

Researchers	Method	Face Pressure Equation	Remarks
Broms and Bennemark (1967)	Observation from experiment, collapses in building pits and Tunnel construction	$F_p = \gamma(C + R) - NC_u$	For stability N<6
Davis et al. (1980)	Limit Analysis Method (Lower Bound solution)	$Fp = C_u(2 + 2\ln(\frac{C}{r}) + 1)$	For Cylindrical stress field
		$Fp = C_u[4\ln(\frac{C}{r}) + 1]$	For Spherical stress field
Anagnostou and Kovari (1996)	Limit equilibrium Method and dimensional analysis	$F'_p = F_0\gamma' D - F_1 c + F_2\gamma'\nabla h - F_3 c\frac{\nabla h}{D}$	F_0, F_1, F_2 and F_3 are the constants

5 Parametric Study

A parametric study has been carried out to know the effects of important parameters such as cover to diameter ratio (C/D), the diameter of the tunnel (D), cohesion (c), friction angle (φ) and Depth of water table (h_w)on the face pressure. The following sections illustrate the effects of these parameters in detail.

5.1 Cover to Diameter ratio (C/D)

The effect of cover to diameter ratio (C/D) on Face pressure has been illustrated in **Fig. 4**. A linear relationship can be observed between the face pressure and cover to diameter (C/D) ratio. In the collapse and the blowout for both cases, the face pressure increases linearly with the cover to diameter ratio. It can be observed that the face pressure is higher in the blow-up condition than in the collapse condition for a fixed cover to diameter ratio.

5.2 Friction Angle (φ)

Friction angle (φ) is one of the major soil parameters that significantly influence face pressure. **Fig. 5** illustrates the effect of friction angle on face pressure for both collapse and blowout condition. It can be observed that the face pressure decreases rapidly with increasing the friction angle from 5° to 15°; beyond 20°, the friction angle is found to have a negligible effect on the face pressure. The collapse and blowout both the cases friction angle show similar effects on face pressure. The centrifuge and numerical analysis by Weng et al., (2020) have found a similar variation in limiting face pressure with the friction angle.

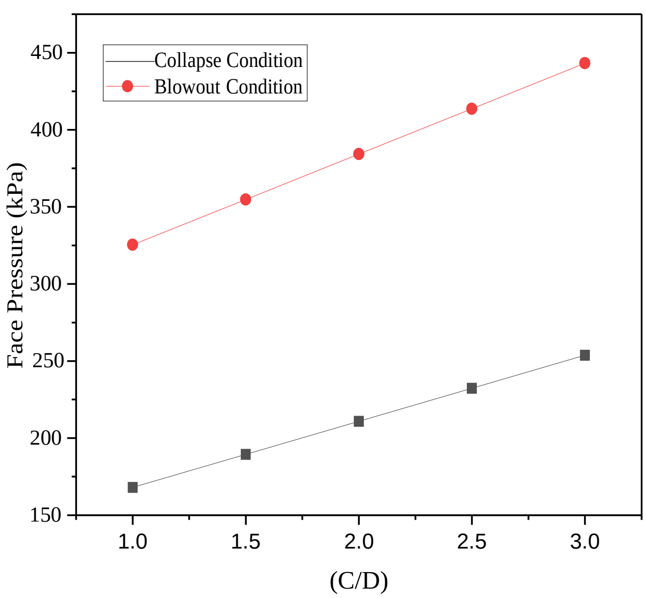

Fig. 4. Variation of face pressure with cover to diameter ratio

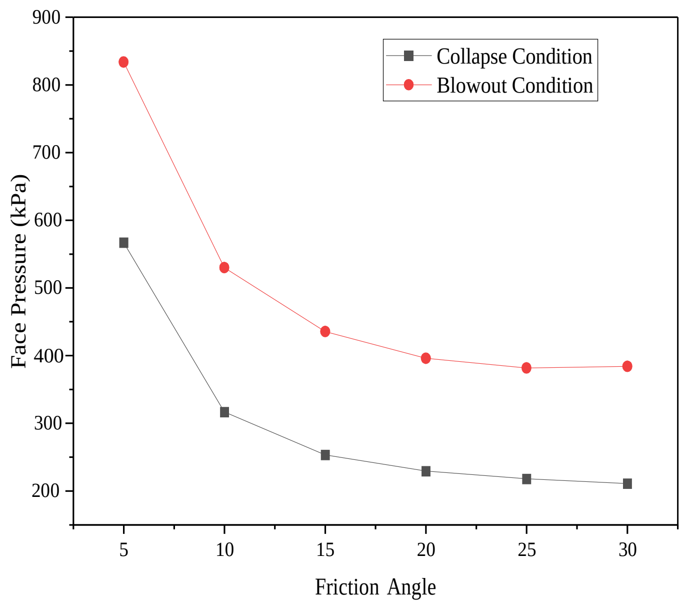

Fig. 5. Variation of face pressure with friction angle

5.3 Cohesion (c)

The effect of cohesion on face pressure has been illustrated in the **Fig. 6**. Collapse and blowout for both the cases of tunnel face stability, face pressure decreases with increasing soil cohesion. It can be seen that, for a fixed cohesion value of soil, the face pressure required for the collapse condition is much lesser than the blowout condition.

5.4 Tunnel Diameter (D)

For studying the effect of tunnel geometry on face pressure, tunnel diameter (D) has been varied, as shown in **Fig. 7**. For the collapse as well as blow-up condition, both show an increasing trend of face pressure with the diameter, but the rate of increment is more significant for the blowup condition. Thus, the risk of the blowup of the tunnel face is higher for the small tunnel diameter.

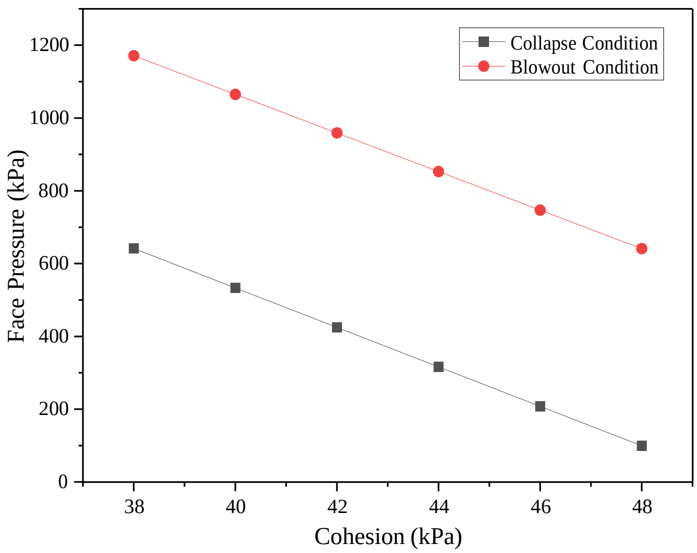

Fig. 6. Variation of Face Pressure with Cohesion

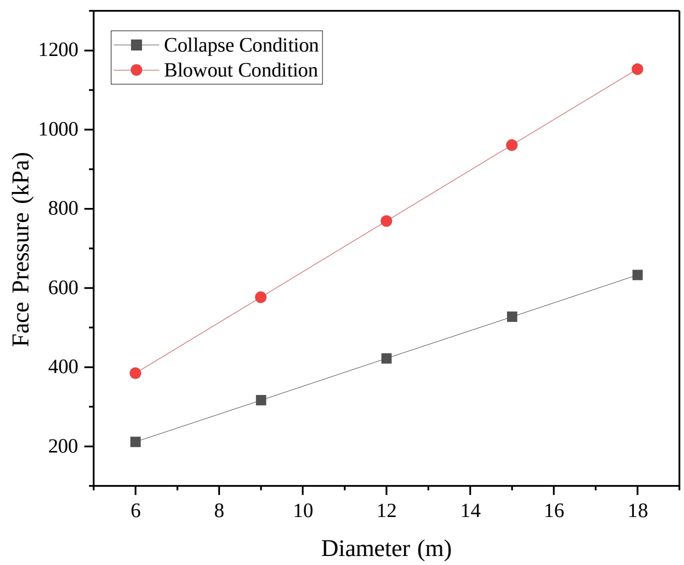

Fig. 7. Variation of Face Pressure with tunnel diameter

5.5 Water Table Effect

The effect of depth of water table on face pressure for collapse and blow-out condition has been illustrated in **Fig. 8** and **Fig. 9**. For collapse and blowout in both the cases, the presence of water table has similar effects on face pressure. The face pressure has been found to decrease rapidly with the drawdown of the water tables. Thus, in the collapse

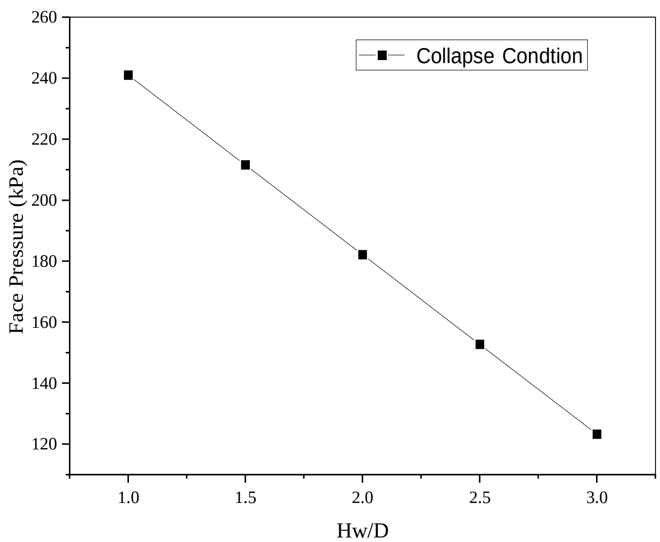

Fig. 8 Water table effect on collapse condition

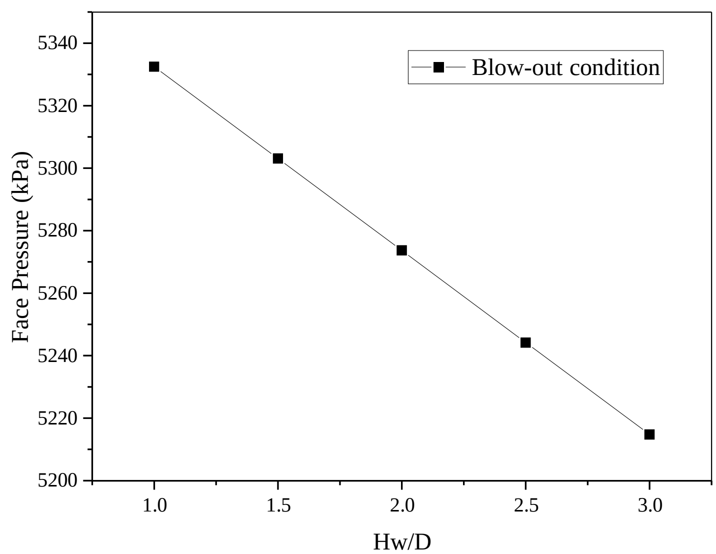

Fig. 9 Water table effect on blow out condition

condition of the tunnel face, less face pressure is required to apply in case of higher depth of water table whereas, in the blowout condition, less pressure leads to a blowout of the tunnel face.

6 Significance of the present study

A closed form analytical solution has been derived using limit equilibrium method. The face pressure for both collapse and blow-out condition have been calculated. The present model gives the minimum pressure, that must be maintained to ensure the face stability under collapse condition. Variation of the face pressure with important parameters can also be noted from the present study. In case of shallow tunnel, if the applied face pressure is more than the blow-out pressure, will lead to blow-out of soil in the TBM face and may cause severe damage of the structures on the ground. Thus, the present study provides a range of face pressures considering collapse and blow-out condition of the soil, within this range of face pressure TBM can be operate safely.

7 Conclusions

The analytical model developed for predicting the collapse and blowout face pressure is applicable for homogeneous, isotropic soil. The face pressure model has developed using the limit equilibrium method (LEM) and incorporated soil arching effects. Two extreme cases of face pressure, such as collapse and blowout condition, has been studied, and flowing points can be concluded from the analysis.

1. The face pressures calculated using the present model provides the range of face pressure within which the tunnel face will not collapse and blow out. Comparing the face pressures for collapse and blow-out with the existing model, it has been found that the face pressure of the existing model lies within the range provided by the present model. It has been found that the collapse pressure provided by the present model is in good agreement with the existing model.
2. Two different earth pressure coefficients have been used to predict the face pressure in the collapse and blowout conditions. In the case of collapse condition, soil in front of the tunnel face expands; as a result, active earth pressure coefficient (k_a) has been used, whereas, for blowout condition, the soil at the tunnel face compressed. As a result, a passive earth pressure coefficient (k_p) has been used.
3. The face pressures have been found to increase linearly with increasing the cover to diameter ratio (C/D). An increment of (C/D) by 33.33 % leads to an increment in face pressure by 11.32 % for collapse condition, whereas for blowout condition, it shows an increment of 8.29 %. To find the effects of tunnel diameter in face pressure prediction, Tunnel diameter (D) has been varied with a constant cover (C), which also shows a linear increment in the face pressure with tunnel diameter for both collapse and blow-out condition.
4. A non-linear relation has been found between the face pressure and friction angle (φ). The face decreases rapidly with increasing the friction angle of the soil. The face pressure remains almost constant for the friction angle greater than 25°. The face pressure decreases linearly with increasing the cohesion (c) of the soil
5. The face pressure decreases with increasing the water table depth for both collapse and blowout condition. The difference between the face pressures in blow-out and collapse condition is very high for a fixed (H_w/D) ratio.

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