



Article

Evaluating Minimum Support Pressure for Tunnel Face Stability: Analytical, Numerical, and Empirical Approaches

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Abstract: Tunneling in loose soil and urban areas presents numerous challenges. One effective solution is the use of Earth Pressure Balance Shields (EPBSs). Maintaining the correct balance of pressure at the tunnel face is critical, as applying too little pressure can cause a collapse, while excessive pressure may result in a blow-out. Therefore, a key aspect of using EPBSs in urban environments is determining the optimal pressure required to stabilize the tunnel face, taking into account the existing soil in the excavation chamber and controlling the screw conveyor's rotation rate. This study focuses on a section of the second line of the Tabriz subway to evaluate the minimum pressure needed for tunnel face stability using empirical, analytical, and numerical approaches. The analytical methods involve evaluating the limit equilibrium of forces and considering soil buckling due to overburden, while the numerical methods employ 3D finite element analysis. Additionally, a sensitivity analysis of the parameters affecting the required pressure was conducted and compared across the three approaches. The results revealed that the formation of a pressure arch mitigates the full impact of overburden pressure on the tunnel face. For soil cohesion values below 20 kPa, the numerical results aligned well with the empirical and analytical findings. For a tunnel depth of 22.5 m and a water table 2 m below the surface, the estimated minimum pressure ranged from 150 to 180 kPa. Moreover, the analytical methods were deemed more suitable for determining the required support pressure at the tunnel face. These methods considered wedge and semi-circular mechanisms as the most probable failure modes. Also, for cohesive ground, the pressure from the finite element analysis was found to be almost always equal to or greater than the values obtained with the analytical solutions.

Keywords: stability of tunnel face; EPB method; minimum support pressure; sensitivity analysis



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1. Introduction

With the increasing population in urban areas and limited surface space, the demand for underground construction, particularly tunnels, has become more significant. Among the various excavation methods available, full-face mechanized boring stands out as a technically and economically favorable choice due to its high speed and precision. In urban areas with loose ground conditions, Earth Pressure Balance Shields (EPBSs) are commonly employed to ensure worker safety and minimize surface settlements. A critical aspect of tunneling with EPBSs is maintaining the stability of the tunnel face and preventing ground

surface settlement. This involves accurately determining the temporary and permanent support pressures for the tunnel face and walls. In the EPB method, a balance must be achieved between the pressure exerted at the tunnel face and the excavation and removal of materials. This balance is crucial because the loose soils and groundwater present around the tunnel face pose a risk of soil and water ingress, which can lead to ground settlement and its associated problems [1,2].

The tunnel face is a high-risk area during construction because collapse failure there can occur simply due to stress release. To ensure the stability of the tunnel face during closed-face tunnel boring machine drilling, the shield machine can apply constant supporting pressure. For example, the traditional silo wedge model [3], the triangular base prism model [4] within the limit equilibrium method, cone models [5], multi-block models [6,7], and sophisticated 3D rotational models [8] within the limit analysis theory are just a few of the theoretical models that have been developed to predict the necessary face pressure. A new yield function that permits asymmetric yielding is proposed and discussed by Elkayam and Klar [9]. Only three-dimensional, continuous velocity fields can benefit from this kind of yielding.

In the absence of horizontal forces on the tunnel face during excavation, collapse occurs when the earth's strength is insufficient to counteract the vertical stress. To address this, fiberglass dowels can be employed to reinforce the tunnel face. These dowels exert an equivalent pressure on the tunnel face, helping to stabilize it and significantly reduce deformation. By simulating the effect of minor horizontal stress, akin to that observed in traditional triaxial shear tests, the reinforcement provided by the fiberglass dowels enhances the failure strength of the tunnel face, ensuring greater stability during excavation.

Tunnel face stability is a topic of significant practical importance, garnering extensive attention in global research. Analytical methods, such as those developed by [7,10], estimate the factor of safety for a carefully selected failure surface based on the stress equilibrium of a critical ground wedge at the excavation face. However, these methods are often constrained by their simplifying assumptions, which stem from the predetermined wedge geometry, limiting their accuracy and applicability. An alternative approach is the Strength Reduction Method, introduced by Zienkiewicz et al. [11], which offers a numerical solution to this problem. This method systematically reduces the ground strength until tunnel face collapse occurs, signaled by the divergence of the numerical integration scheme. It provides a more robust analysis by overcoming some of the limitations inherent in purely analytical techniques.

The stress history of in situ soil surrounding the excavation area undergoes significant changes during tunnel construction. This alteration triggers deformation in the soil mass, leading to a redistribution of stress along the tunnel face. The mechanism responsible for this redistribution is known as the arching phenomenon, first identified by Terzaghi [12]. This process reduces vertical stress in the soil. A series of physical model studies conducted by [13–15] investigated tunnel face stability and documented the effects of soil arching, highlighting its crucial role in maintaining tunnel face stability during excavation.

Adequate support pressure at the tunnel face is essential to prevent collapse during excavation, achieved through methods like compressed air, slurry, or earth pressure support. The soil arching effect, which reduces the required support pressure, plays a key role in this process. Analytical approaches are commonly utilized to account for the 3D arching effect when determining the necessary support pressure. The two primary analytical methods are the Limit Equilibrium Method (LEM) and the Limit Analysis Method (LAM). The LEM is used to theoretically evaluate tunnel face stability by analyzing the static equilibrium of forces acting on the soil mass. For critical collapse mechanisms, the LEM considers the balance of forces to predict stability. Various 3D wedge silo models have been developed

using the LEM to calculate the required support pressure effectively, as demonstrated in studies by [5,7,16,17].

Numerous studies have explored surface settlement resulting from EPB tunneling using numerical, analytical, and empirical approaches [18,19]. Hrubesova et al. [20] employed 3D finite element modeling to determine the minimum face pressure required during the excavation of a circular tunnel with a fully excavated face. Additionally, several researchers have investigated the influence of water pressure on tunnel face stability, highlighting its critical role in maintaining structural integrity during excavation [21–23].

One of the primary causes of increased displacements during urban excavation is the application of inadequate pressure on the tunnel face. Insufficient support pressure below the allowable limit can lead to the uncontrolled collapse of tunnel face soils into the excavation chamber, resulting in significant ground settlement. Conversely, applying excessive pressure to the tunnel face may cause substantial surface deformation, leading to blowouts that damage the support system and surface structures. To address these challenges, the tunnel face support pressure must balance minimizing the collapse risk while optimizing the friction and excavation rates. Over the years, researchers have developed various methods to evaluate the minimum support pressure for tunnel faces. These include empirical, experimental, analytical, and numerical approaches [3,7,15,16,24–28]. These approaches aim to ensure tunnel stability while minimizing the impact on surrounding infrastructure.

Moreover, resilience in tunneling refers to the system's ability to withstand, adapt to, and recover from challenges such as ground instability, groundwater effects, or construction-related disturbances. It encompasses ensuring safety, optimizing resource use, and maintaining structural integrity under varying and unpredictable conditions [29]. By emphasizing resilience, this study addresses the critical need for robust design and operation strategies in tunneling, particularly for projects in urban or complex geological settings. This study contributes to resilient tunneling practices by examining the sensitivity of tunnel face stability to key parameters, including cohesion, the internal friction angle, and groundwater table variations. The findings align with recent advancements in understanding hydro-mechanical interactions, as discussed by [23], who explored the influence of permeability anisotropy on tunnel face stability. By integrating these insights, the study offers adaptable strategies to optimize support pressures and enhance the safety and sustainability of tunnel construction projects.

The Earth Pressure Balance Shield (EPBS) method offers significant advantages in maintaining groundwater and soil stability during tunneling projects, particularly in urban areas. EPBSs stabilize the tunnel face by balancing the excavation pressure with the surrounding soil, which minimizes ground displacement and prevents surface settlement. This is especially important in loose or saturated soils, where traditional tunneling methods might lead to significant subsidence or water ingress. The method is designed to control the effect of groundwater fluctuations, reducing the risk of destabilizing the surrounding soil and groundwater table. This results in safer tunneling operations, minimizing environmental disruption and the impact on urban infrastructure.

However, there are challenges to consider when using EPBSs, particularly concerning groundwater management and soil integrity. While the method effectively minimizes groundwater inflows, over-pressurization can occur if not carefully controlled, which may destabilize the soil around the tunnel. Additionally, the disturbance to soil layers can still affect underground water flow, potentially leading to localized groundwater contamination if leaks or faults occur in the shield. EPBSs also require careful monitoring of soil and water conditions to ensure that pressure levels are adjusted correctly throughout the excavation process. Despite these challenges, EPBSs remain a sustainable and efficient method for urban tunneling when managed appropriately.

The following section explores general approaches to determining the minimum support pressure required for stabilizing a tunnel face, with a focus on techniques utilized by Earth Pressure Balance (EPB) boring machines. The analysis included modeling the specified segment using a 3D finite element numerical method to evaluate the minimum support pressure accurately. Finally, the study examined how variations in key factors, such as the internal friction angle, cohesion, and groundwater table levels, influence the tunnel face's support pressure, providing valuable insights through comparative analyses of different methods.

This study focused on the intersection of the Mehran River (Ghouri Chay) and the second line of the Tabriz metro, which passes near the Ghari Bridge and includes the BH-12 borehole. The investigation emphasized identifying the numerical method most accurately representing the failure mechanism and pressure values. To achieve this, a detailed comparison was conducted between analytical and numerical approaches for calculating the minimum support pressure of the tunnel face, with particular attention given to a sensitivity analysis of key influencing parameters.

2. Approaches to Determination of the Minimum Support Pressure of Tunnel Face

2.1. Empirical Approaches

In empirical approaches, relationships are established between the tunnel face and one or more parameters based on observations made during excavation. As shown in Table 1, these approaches do not provide specific minimum or maximum support pressures. Instead, they suggest a pressure range that ensures optimal working conditions, with a general focus on determining the minimum pressure required.

Table 1. Experimental approaches to determining minimum support pressure of tunnel face.

Approach	Description
Dutch Center Underground Bowe (COB)	See Equation (1)
Terzaghi [13]	See Figure 1

2.2. Analytical Approaches

These methods can be broadly classified into two categories: limit analysis stress methods and global limit equilibrium methods. In limit equilibrium approaches, the process involves three key phases: identifying the critical slip surface, assuming the stress distribution along the slip surface, and solving the problem using global equilibrium equations for a rigid body such as soil. Limit analysis stress methods primarily focus on stress analysis through two types of solutions: upper bound solutions (which define a failure mechanism consistent with the velocity field but do not analyze the equilibrium of internal and external forces) and lower bound solutions (which define an assumed stress field without solving for strains) [1]. A detailed description of these strategies is provided in Table 2.

Table 2. Limit approaches to determination of minimum support pressure of tunnel face.

Approach	Description
Krause [30]	See Figure 2
Jancsecz and Steiner [5]	See Figures 3 and 4
Anagnostou and Kovári [7]	See Figure 5
Broere [16]	See Figure 6
Atkinson and Potts [8]	See Figure 7
Leca and Dormieux [24]	See Figure 8

2.3. Experimental Approaches

Field measurements are essential for validating tunnel face stability model formulations. Due to the high cost and uncertainties associated with field experiments, some researchers opt for laboratory trials to assess the support pressure required for tunnel faces. Experimental stability models are valuable as they allow the measurement and recording of the appropriate pressure needed to stabilize the tunnel face, particularly in the absence of sufficient data from actual failures. However, these tests can only determine the necessary pressure to maintain stability and cannot establish the minimum or maximum allowable pressures. Using a geo-centrifuge, several researchers have studied tunnel face stability through two experiments conducted in loose ground with clay and sandy soils, both under conditions of support pressure and without any support pressure [16].

This method, based on Equation (1), recommends that the support pressure of the face is more than the earth active pressure.

$$\sigma_T = K_a \sigma'_v - 2c\sqrt{K_a} + K_a q + P + 20 \text{ kPa} \quad (1)$$

where P is the water pressure, q is the surface load, C is cohesion, and K_a is the index of the earth active pressure.

According to this approach, if some part of the soil mass above the tunnel gives way during excavation, the soils in this area will collapse. In shallow zones, the loosening reaches to the ground surface, which means no cases of arching occur. But in deep zones, it forms an arch above the tunnel, as shown in Figure 1. The generally vertical pressure on tunnel is obtained by Equation (2). In this equation, c is the soil cohesion, K_0 is the soil lateral pressure coefficient with a stationary status, H_c is the overburden height, B is half the length of the loosening zone, H_w is the water level, q_0 is the surface load, γ is the density of the soil, φ is the soil friction angle, and σ_v is the average vertical pressure on the tunnel heading [13].

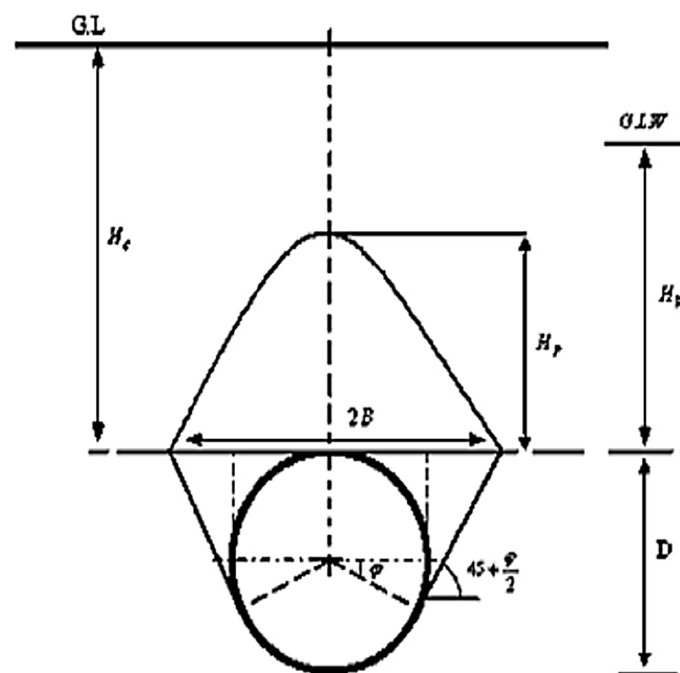


Figure 1. Soil arching of tunnel heading.

$$\sigma_v = \frac{B\gamma - c}{K_0 \tan \varphi} \left(1 - e^{-K_0 \tan \varphi \frac{H_c}{B}} \right) + q_0 \left(e^{-K_0 \tan \varphi \frac{H_c}{B}} \right) \quad (2)$$

The length and height of the loosening zone are obtained by Equations (3) and (4), respectively:

$$B = R \left(\tan(45 - \varphi/2) + \cos(45 - \varphi/2)^{-1} \right) \quad (3)$$

$$H_p = \sigma_v / \gamma_d \quad (4)$$

Regarding the effective support pressure of the tunnel face when the tunnel is excavated at above the level of the underground water table, the total pressure is obtained by Equation (5) and the sum of the water pressure and soil pressure:

$$\sigma_T = K_a \times \gamma_d \times (H_p + R) \quad (5)$$

In this approach, the required minimum support pressure is evaluated in three slide modes and by using shear stresses on the slide planes (Figure 2). The minimum support pressure of the tunnel face for the semi-circular and semi-spherical slide mechanism is obtained by Equations (6) and (7), respectively. Assumptions such as a constant value of pressure on the face and type of slide mechanism are the disadvantages of this approach [12,16].

$$S_{min} = \frac{1}{\tan \varphi} \left(\frac{1}{3} D Y' - \frac{1}{2} \pi c \right) \quad (6)$$

$$S_{min} = \frac{1}{\tan \varphi} \left(\frac{1}{9} D Y' - \frac{1}{2} \pi c \right) \quad (7)$$

According to Figure 3, the failure model in this approach is three-dimensional and consists of a soil wedge (lower part) and soil silo (upper part). Indeed, it is the first practical performance demonstrated by [31]. The vertical pressure resulting from the soil and acting on the soil wedge is calculated according to Terzaghi's solution. These two researchers (Jancsecz and Steiner) studied the effect of soil buckling over the TBM and presented the results as a three-dimensional coefficient (K_{a3}), which is shown in Figure 4 for different overburdening and internal friction angles. By multiplying this pressure coefficient by the vertical effective stress, the value of the soil pressure on the tunnel face is estimated by Equation (8) [5,16]:

$$\sigma_T = K_{a3} \sigma'_v + P \quad (8)$$

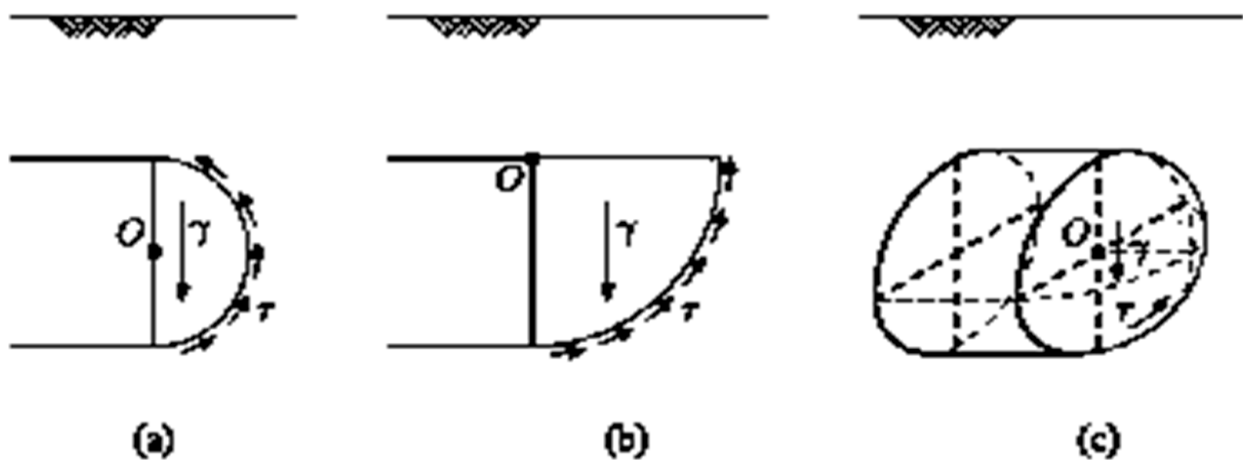


Figure 2. Different failure mechanisms of tunnel face using Krause method, (a) Shear stress distribution along the curved boundary of a semicircular region. (b) Variation of shear stress under vertical loading at the center. (c) Three-dimensional representation of stress distribution in the semicircular geometry [30].

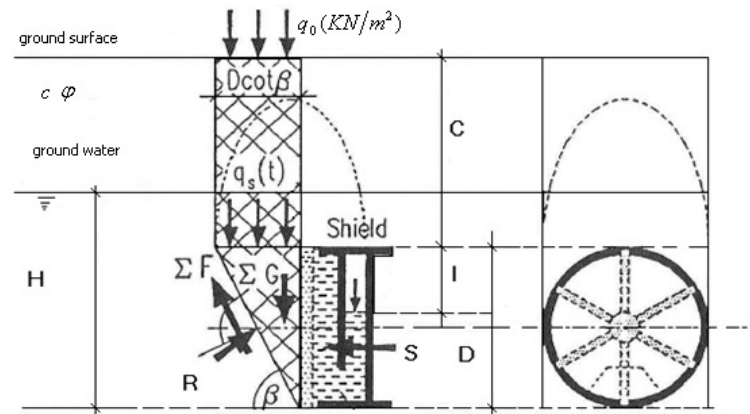


Figure 3. Mechanism of load distribution on tunnel face using Jancsecz and Steiner method [5].

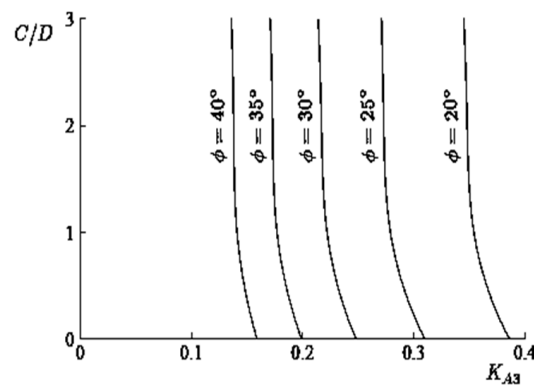


Figure 4. Three-dimensional earth pressure coefficient (K_{a3}) obtained [5].

These two researchers, as shown in Figure 5, used a wedge model for face stability in an EPBS and, with the assumption of Horn's sliding theory and Terzaghi's pressure arching, presented a relationship. In this method, the effective pressure in the chamber in the general form of limit equilibrium is obtained by Equation (9) and in this equation Δh is a distinction of the water head in the chamber and in the ground that should be kept at the minimum value. F_0 to F_3 are dimensionless factors from nomograms:

$$S' = F_0 Y' D - F_1 c + F_2 Y' \Delta h - F_3 c \frac{\Delta h}{D} \quad (9)$$

For creating a stability model using a multilayer wedge, as shown in Figure 6, the failure wedge is divided into N similar pieces with different thickness, with homogeneous soil conditions established within them. It is possible that the soil conditioning between the pieces and also the wedge angle between the slide plane of each piece is changed $\theta^{(i)}$ with horizon. Each of the i pieces is loaded by forces resulting from the upper and lower pieces, which are called $Q_b^{(i)}$ and $Q_a^{(i)}$, and also by the effective weight of the piece itself, $G_w^{(i)}$, and the overburden force, $G_s^{(i)}$. In the slope slide plane, both the cohesion force ($K^{(i)}$) and friction force ($R^{(i)}$) due to the normal force perpendicular to the failure plane ($N^{(i)}$) acted parallel to the plane. We assumed that each of the forces around the wedge is being loaded by shear force, which acted contrary to the deformation of the wedge. The force balance yields the earth effective force ($E^{(i)}$), which, with the water force ($W^{(i)}$), will be equal to the support force of the face ($S^{(i)}$) that is obtained by Equation (10):

$$S = \eta_E E + \eta_W W \quad (10)$$

$$(\eta_E = 1.05, \quad \eta_W = 1.5), \quad \sigma_T = 4S / \pi D^2$$

$$N_s Q_s + N_Y Q_Y \geq Q_T \quad (13)$$

where N_s and N_Y are the dimensionless weighting factors from the nomograms, where one is for the maximum support pressure and the other is for the minimum support pressure.

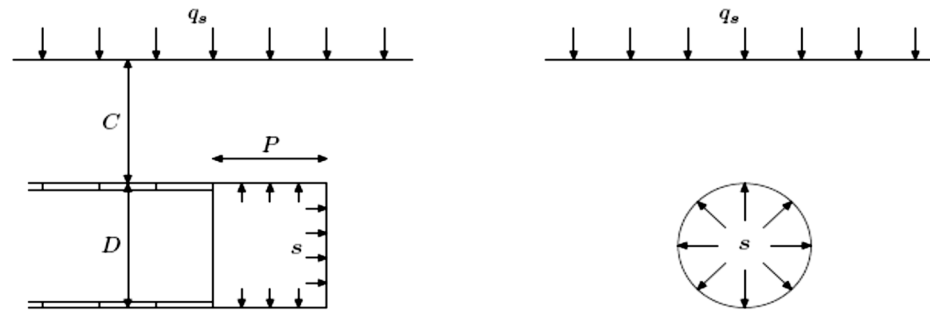


Figure 7. Scheme of tunnel without any support in Atkinson and Potts method [8].

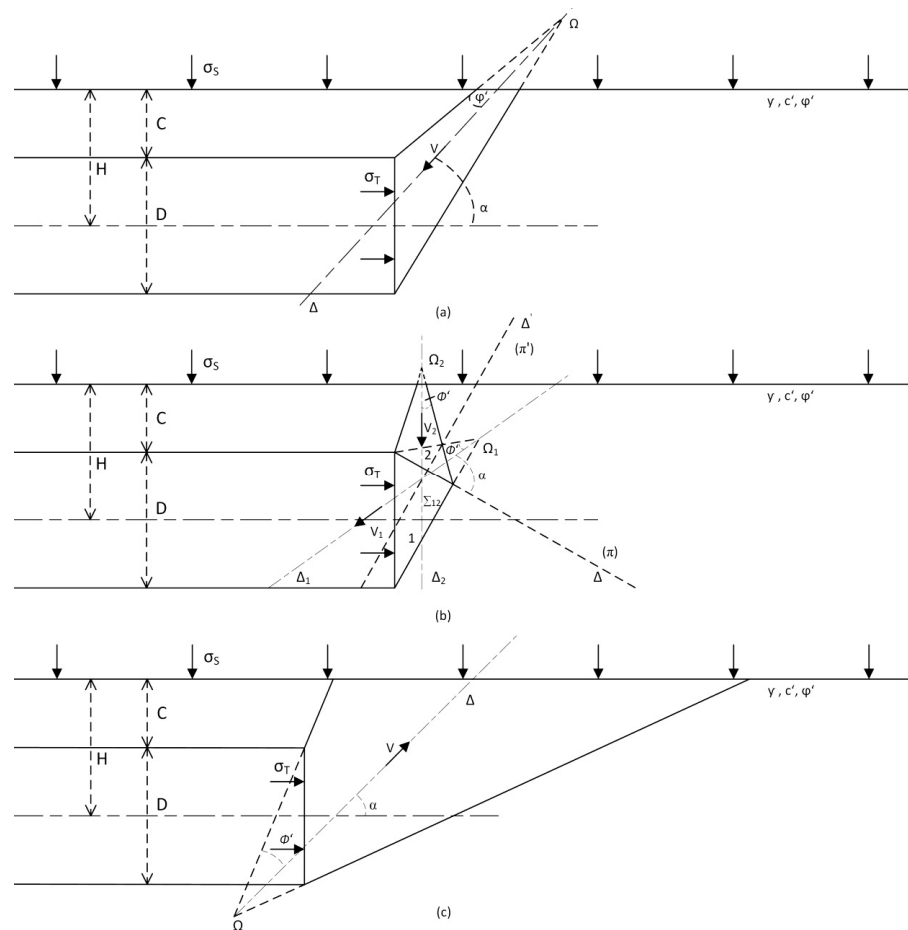


Figure 8. Conical failure mechanism in upper limit solution [24].

The wedge stability models have limitations due to the heterogeneity of the tunnel face, grout infiltration into the TBM's soil front, and the additional pore pressure induced by this grout penetration. To address these issues, Broere's [16] multilayer wedge stability model offers a more effective solution. However, analytical methods are not ideal for assessing the long-term stress–strain behavior of both the tunnel's surrounding ground and the ground itself. Despite this, these analytical approaches still provide valuable design insights. Moreover, numerical methods are often validated through these analytical results.

The critical evaluation of existing methods highlights the limitations of the analytical, empirical, and numerical approaches for determining the minimum support pressure at the tunnel face. The analytical methods oversimplify the failure mechanisms and soil behavior, leading to inaccuracies, while the empirical methods rely on generalized relationships that lack site-specific precision. The numerical methods, although powerful, are computationally intensive and depend heavily on appropriate material models and boundary conditions. Additionally, the combined effects of groundwater variations, soil heterogeneity, and overburden depth have often been overlooked in past studies [7,32–34]. This work addresses these gaps by integrating site-specific soil properties, conducting a detailed sensitivity analysis, and validating the results against multiple analytical approaches. The study provides a robust framework for accurately estimating tunnel face support pressures, emphasizing the importance of accounting for the pressure arching effect and site-specific conditions to optimize stability and cost-efficiency in EPB tunneling.

3. Studied Area

The construction of an underground railway or subway is an ideal solution to the transportation and traffic challenges faced by large cities. With a population exceeding 2 million, Tabriz is grappling with significant traffic congestion, prompting the implementation of a subway project to alleviate these issues. The second line of the Tabriz subway spans 22 km, connecting the Gharamalek district to the Tabriz International Exhibition Center, with 20 stations along the route. Figure 9 illustrates the intersection between the Mehran River (Ghouri Chay), which flows around the Ghari Bridge, and the second subway line. The selection of the BH-12 borehole for assessing tunnel face stability was primarily due to the erosion and sedimentation caused by the river, as well as the seasonal fluctuations in the water table.

The subsurface in this region predominantly consists of coarse alluvial sediments, mainly sandy gravel. Based on field and laboratory data, the excavated soil comprises two gravel layers (GPs) ranging from 2 to 6 m in thickness, and two layers of sandy-silty soil with thicknesses between 11 and 16 m. Table 3 details the geomechanical properties of these layers, as observed in the BH-12 borehole. The second line of the Tabriz subway, excavated using an Earth Pressure Balance Shield (EPBS), has dimensions of 10.5 m in height and 9.45 m in width, and it spans a total length of 22 km. Table 4 illustrates the features of EPB machine in this project.

Table 3. Geomechanical parameters of existing layers in BH-12 borehole.

Parameter	Symbol	Dimension	Depth of Layer	
			GP	MS
			0–2 (m) 18–24 (m)	2–18 (m) 24–35 (m)
Material Model	-	-	Mohr–Coulomb	Mohr–Coulomb
Material Behavior Type	-	-	drained	drained
Dry Density	γ_{unsat}	KN/m ³	17.5	18
Wet Density	γ_{sat}	KN/m ³	21	22
Young’s Modulus	E_{ref}	KN/m ²	45,000	50,000
Poisson’s Ratio	ν	-	0.3	0.3
Cohesion	C_{ref}	KN/m ²	1–10	10–30
Angle of Internal Friction	Φ	Deg	30–40	23–35
Dilation Angle	ω	Deg	0	0
Interface Resistance	R_{inter}	-	0.9	0.8
Lateral Compression Index	K_0	-	1-sin ϕ	1-sin ϕ

Table 4. Features of EPB machine.

Parameter	Symbol	Dimension	Value
Material Behavior Type	-	-	Elastic
Axial Rigidity	EA	KN/m	12,600,000
Momenta Rigidity	EI	KNm ² /m	85,000
Equivalent Thickness	D	M	0.285
Weight	W	KN/m/m	50.77
Poisson's Ratio	ν	-	0

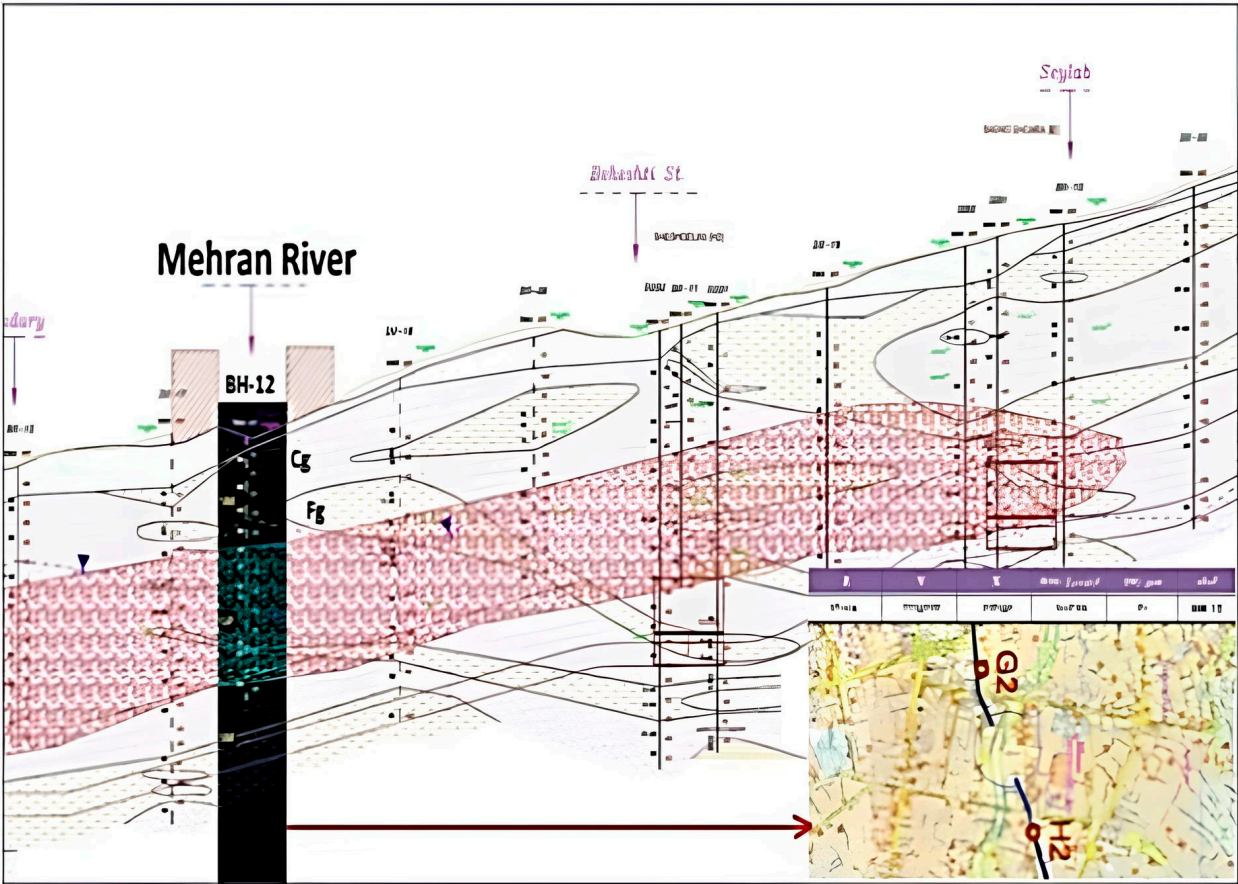


Figure 9. Geological map of 2nd line of Tabriz subway (red arrows indicate the direction of the subway line alignment. Purple text labels streets and key locations along the alignment. Black lines represent borehole locations and vertical sections (BH-12). Shaded pink area denotes a geological feature or formation, such as a sedimentary deposit or aquifer. Green contour lines depict elevation changes or the topography of the area. Blue section highlights the location of a significant underground feature or water body).

4. Numerical Modeling of Support Pressure of the Face

This paper utilizes Plaxis 3D (2020) Tunnel software to analyze the stability of the tunnel face. The process consists of four key steps: input, calculation, output, and curve generation. Each of these steps requires specific performance stages, as illustrated in Figure 10, for the appropriate model. In the finite element simulations, the soil was modeled as an elastic perfectly plastic (elasto-plastic) material, adhering to the Mohr–Coulomb failure criterion. Despite its limitations, this constitutive model has been previously evaluated and found sufficiently accurate for determining the tunnel face support pressure at failure [35,36].

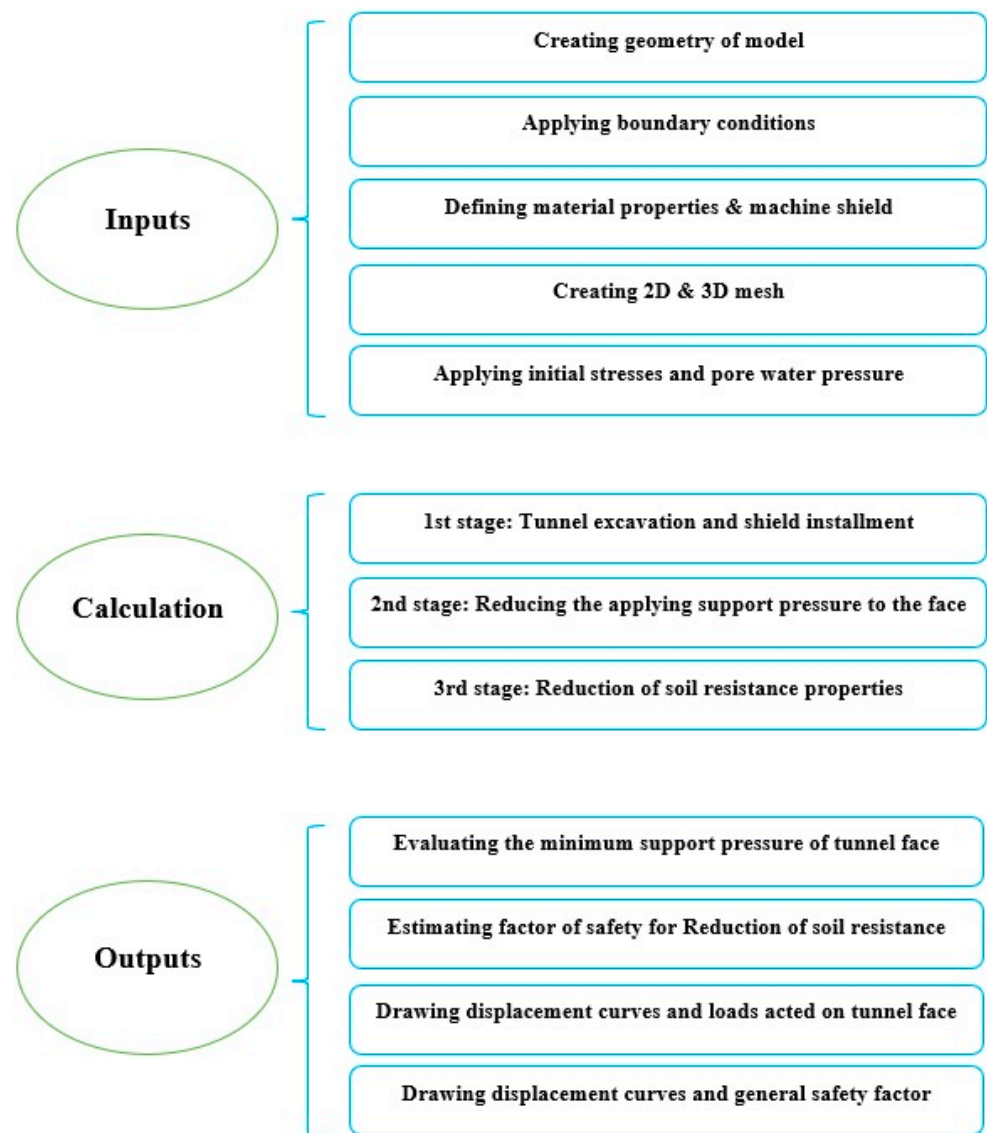


Figure 10. Numerical modeling stages.

This section discusses the geometry, boundary conditions, meshes, and construction stages used in all the numerical analyses. The model geometry, as shown in Figure 11, considers a height of 35 m (comprising four layers), a width of 30 m, and a length of 30 m in the Z direction. These dimensions ensure that the model is large enough to accommodate any failure mechanism while minimizing the boundary effects. To optimize the model volume and calculation time, only half of the tunnel and shield machine were modeled, leveraging symmetry in both geometry and boundary conditions. The model's boundary dimensions were carefully selected by analyzing different boundary sizes and comparing the results, in line with Ruse's recommendations, to ensure they did not impact the face collapse pressure. The tunnel, with a diameter of 9.45 m and an axis located 22.5 m below the surface, was designed with an overburden-to-tunnel-diameter ratio of 2. The water table is positioned two meters below ground level, and the surface load is assumed to be zero. Soil resistance properties were considered for the gravel layers ($c = 2$ kPa, $\varphi = 35^\circ$) and sandy-silty layers ($c = 10$ kPa, $\varphi = 30^\circ$). The bottom face of the model is completely restricted, preventing any movement, while the ground surface is left unrestricted and free to shift. The vertical boundaries on the sides of the soil box are constrained solely in the direction perpendicular to their surface, with horizontal displacements controlled

using roller supports. The Mohr–Coulomb failure criterion is utilized for the soil modeling. The soil's elastic characteristics, represented by Young's modulus and Poisson's ratio, are incorporated. However, it is important to note that these elastic parameters have minimal influence on the stability analysis outcomes. Based on the elevation of the layers, appropriate material groups are assigned to each corresponding layer. The tunnel lining is modeled using shell elements, while interface elements adhering to Coulomb's law establish the connection between the lining and the surrounding soil. Normal stiffness (K_n) and shear stiffness (K_s) are attributed to the interface, determined by the stiffness of adjacent elements. These parameters have minimal impact on the precision of this type of stability analysis, ensuring reliable results. Moreover, symmetry was used to optimize the computations, and roller supports constrained displacements at vertical boundaries. Additionally, the boundary size was analyzed following Ruse's recommendations to ensure it did not impact the calculated collapse pressure, confirming that the edge influences were effectively controlled.

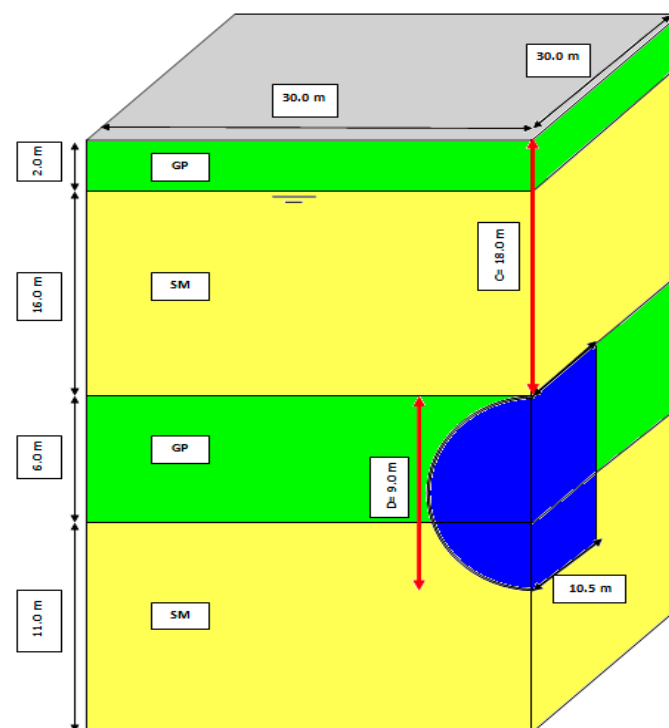


Figure 11. Three-dimensional finite element model. (Yellow (SM): Represents silty sand; Green (GP): Denotes poorly graded gravel; Blue (Tunnel Section): Represents the tunnel excavation area; Gray (Surface): The ground surface).

The first stage involves soil excavation to allow the shield to penetrate, lowering the water level in the machine, applying pressure to the tunnel face, and simulating convergence to model the conical shape of the shield (due to the reduction in shield diameter from the face to the rear). This stage focuses on the stability of the tunnel face heading, considering the shield's advancement over a length of 10.5 m. At this point, the support pressure, determined through empirical or analytical methods—277 kPa from the COB—is applied as the base pressure to the tunnel face. Figure 12 depicts the general displacements following this initial stage of tunnel excavation, showing that the applied pressure was sufficient to cause only partial displacements in the face, with the conical shield's effect on ground convergence clearly visible.

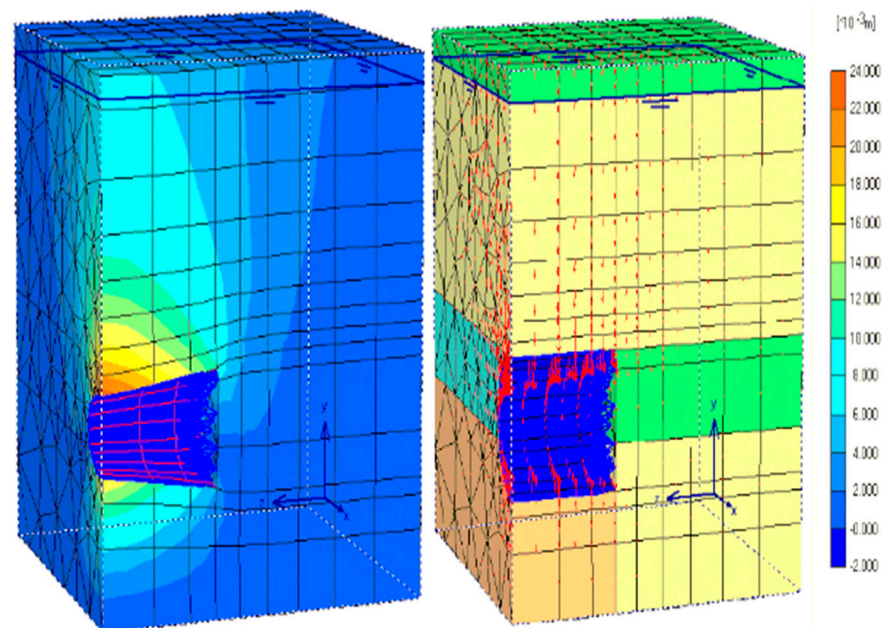


Figure 12. General displacements at the end of first stage.

In the first stage, the applied pressure was gradually reduced until the tunnel face collapsed during the second stage, allowing the minimal pressure required to support the tunnel face to be determined. As the total loads on the face reached zero, the displacement of the face reached its maximum, resulting in failure. By reducing the pressure applied to the tunnel face near the end of the second stage, the total acting loads on the face were reduced to 0.616. Given the initial pressure of 277 kPa, the minimum pressure required to support the tunnel face was calculated to be 170.6 kPa. Figure 13 illustrates the local soil displacement into the face at the end of the second stage (as the pressure was reduced). It shows that the compression developed by the initial pressure decreased as the pressure was lowered, causing soil to move into the tunnel face and leading to a localized collapse. The maximum displacement of the face at this point was 70.14 mm.

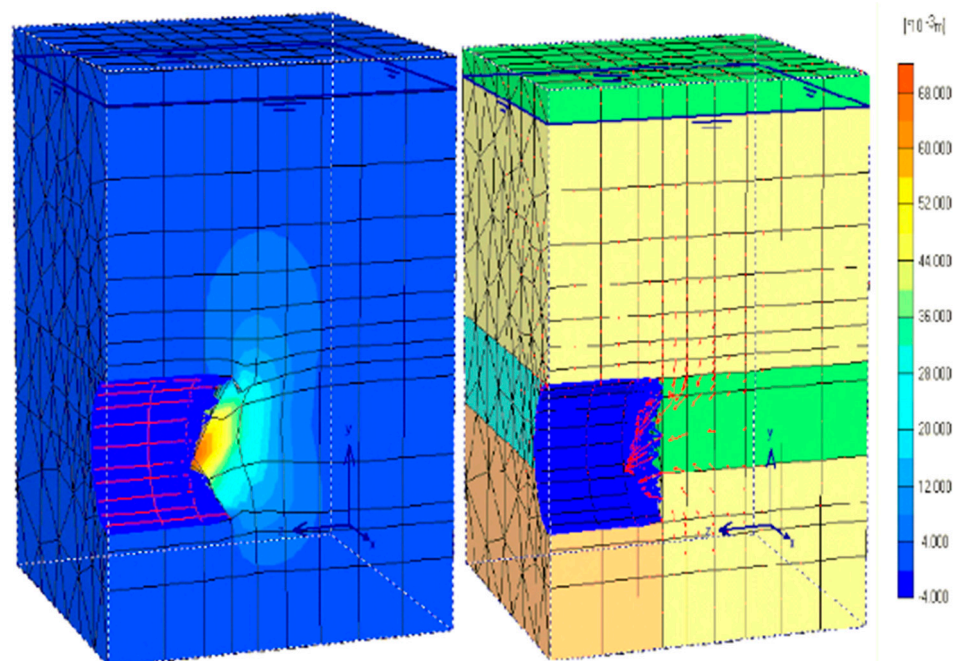


Figure 13. Total displacements at the end of second stage.

Figure 14 compares the pressure values obtained using the empirical, analytical, and numerical modeling methods. The results show good agreement between the numerical method and the approaches of Jansecz and Steiner [5], Anagnostou and Kovári [7], and Broere [16]. Furthermore, the Broere [16] method, which incorporates the effect of three-dimensional pressure arching, yields lower pressure values compared to the method using two-dimensional pressure arching. According to this figure, the COB and Terzaghi approaches yield the highest support pressures (297 kPa and 282.5 kPa, respectively), indicating conservative predictions that ensure safety but may overestimate the required support. Approaches like Janbu and Senneset [37] (175.2 kPa), Anagnostou and Kovári [7] (163 kPa), and Atkinson and Potts [8] (120.3 kPa) predict significantly lower values compared to the empirical methods, suggesting optimization in support design while maintaining stability. However, the range of the values highlights the variability in the assumptions. The numerical analysis (170.6 kPa) aligns closely with the analytical predictions, such as Janbu and Senneset [37] and Anagnostou and Kovári [7], reinforcing its reliability. These methods account for site-specific conditions, enhancing accuracy compared to general empirical rules. Broere's 2D and 3D models [16] (161 kPa and 215.3 kPa, respectively) show variability within the numerical methods, highlighting the influence of modeling dimensions. Therefore, Figure 14 supports the conclusion that numerical methods provide a balanced approach by bridging the gap between conservative empirical estimates and analytical predictions, optimizing tunnel face stability design while avoiding over-design.

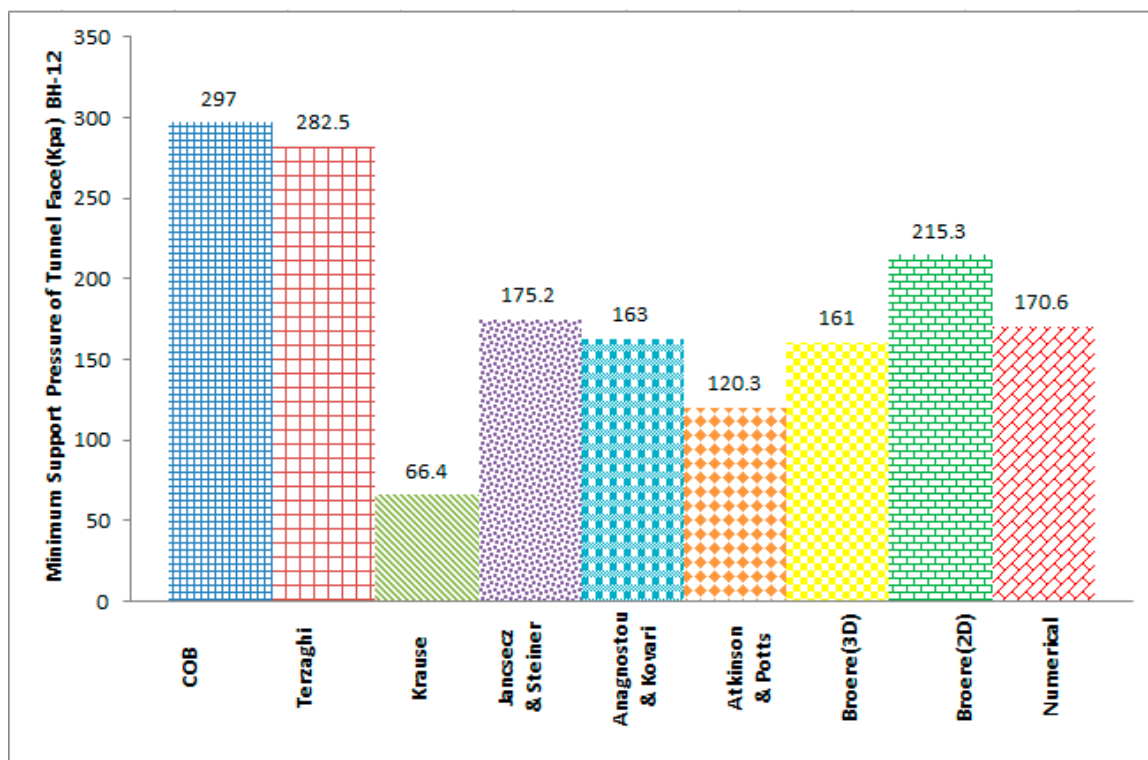


Figure 14. Comparing the values resulting from empirical, analytical, and numerical approaches for the support pressure of tunnel face in BH-12.

5. Sensitivity Analysis of Supporting Pressure of Tunnel Face

Soil cohesion (c) and the internal friction angle (φ) are key factors influencing the support pressure at a tunnel face. The following sections explore how these parameters impact the support pressure and how various methodologies estimate it. A 3D finite element model (using Plaxis 3D Tunnel software) was employed to simulate the tunnel face and its surrounding soil. The model incorporated the Mohr–Coulomb failure criterion

to assess the stability across these variations. Four distinct models were considered, each representing a different ratio of overburden to tunnel diameter. Furthermore, for each model, the water table was factored in, with its level set 2 m below the ground surface. For each scenario, the minimum pressure was determined by progressively reducing the applied pressure at the tunnel face until collapse conditions were identified, allowing precise sensitivity evaluation.

5.1. Effect of Cohesion Variations on the Support Pressure of Face

Figure 15 illustrates the relationship between cohesion and minimum support pressure at the tunnel face for different depths ($C = D$, $1.5D$, $2D$, and $2.5D$) in EPB tunneling. As the cohesion increases, the support pressure decreases significantly for the numerical and advanced analytical methods (e.g., [16,30]), especially at greater depths. In contrast, traditional methods like COB and Terzaghi remain nearly constant and conservative, neglecting the beneficial effect of cohesion. The results emphasize that cohesion plays a more critical role in deeper tunnels, where numerical approaches provide the most realistic reductions in support pressure, making them more reliable for practical design.

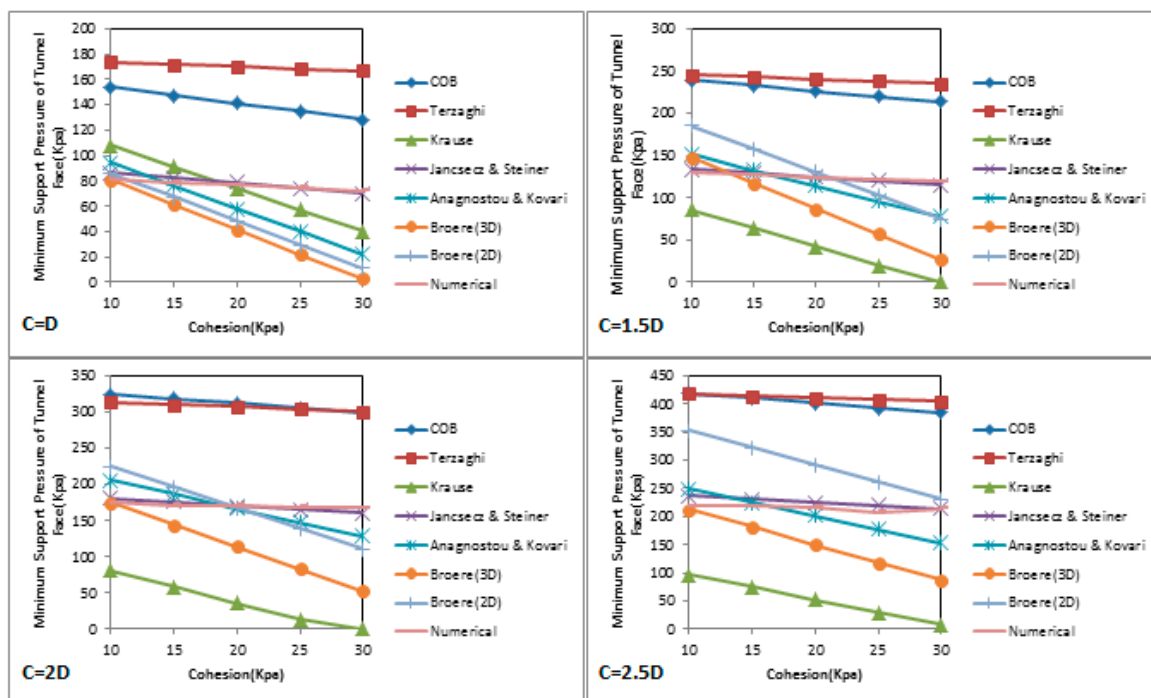


Figure 15. Relationship of cohesion with the support pressure of tunnel face for $\phi=25^\circ$.

Figure 16 demonstrates that increasing the cohesion (10–30 KPa) significantly reduces the minimum support pressure required at the tunnel face, especially for the advanced analytical methods [7,16,30] and numerical simulations. In contrast, the traditional methods like COB and Terzaghi remain conservative and unaffected by the cohesion. As the tunnel depth increases (from $C = D$ to $C = 2.5D$), the overall support pressure rises, but the reduction due to the cohesion becomes more pronounced, particularly in the numerical results. This highlights the importance of incorporating cohesion effects and utilizing numerical methods for accurate and efficient support pressure estimation in EPB tunneling.

Figure 17 shows the effect of cohesion (10–30 KPa) on the minimum support pressure required at the tunnel face for different depths ($C = D$, $C = 1.5D$, $C = 2D$, and $C = 2.5D$) in EPB tunneling. As the cohesion increases, the support pressure consistently decreases for the numerical methods and advanced analytical models [7,16,30], with the numerical approach

showing the most significant reductions. In contrast, the COB and Terzaghi methods remain nearly constant and conservative, ignoring the cohesion's influence. At greater depths ($C = 2.5D$), the required support pressure is higher, but the effect of the cohesion becomes more substantial, further emphasizing the importance of accurate methods like numerical simulations for practical and optimized tunnel face support design.

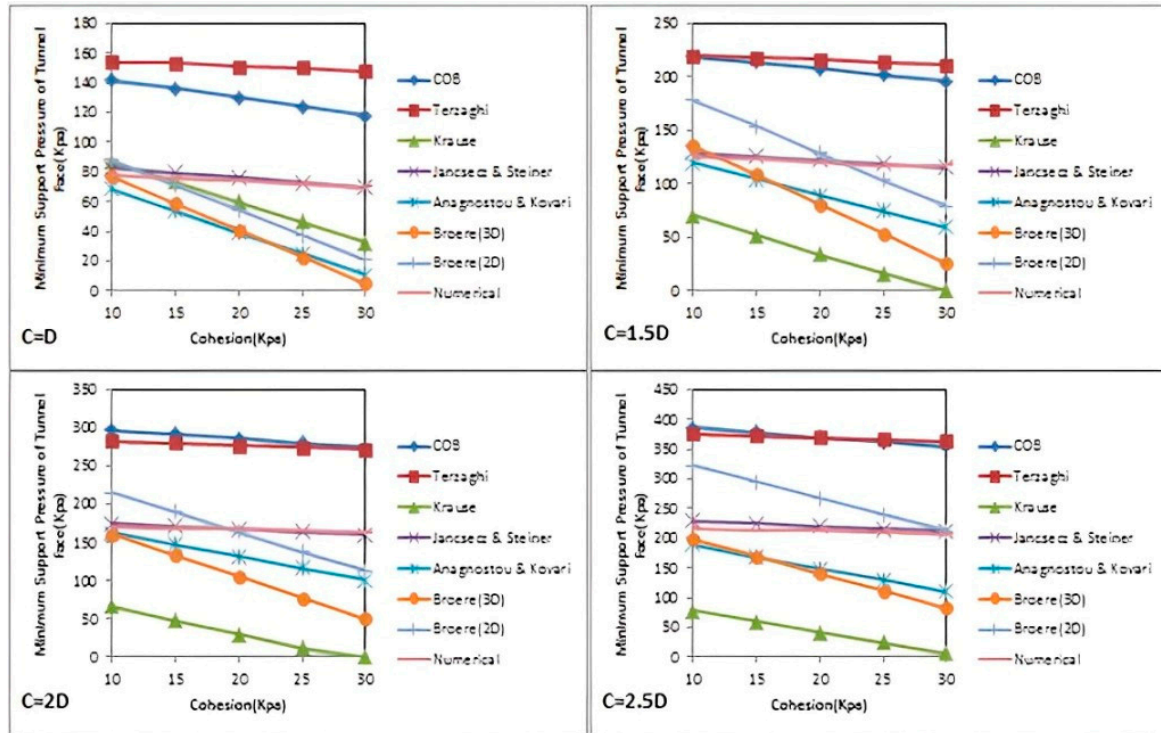


Figure 16. Relationship of cohesion with the support pressure of tunnel face for $\phi = 30^\circ$.

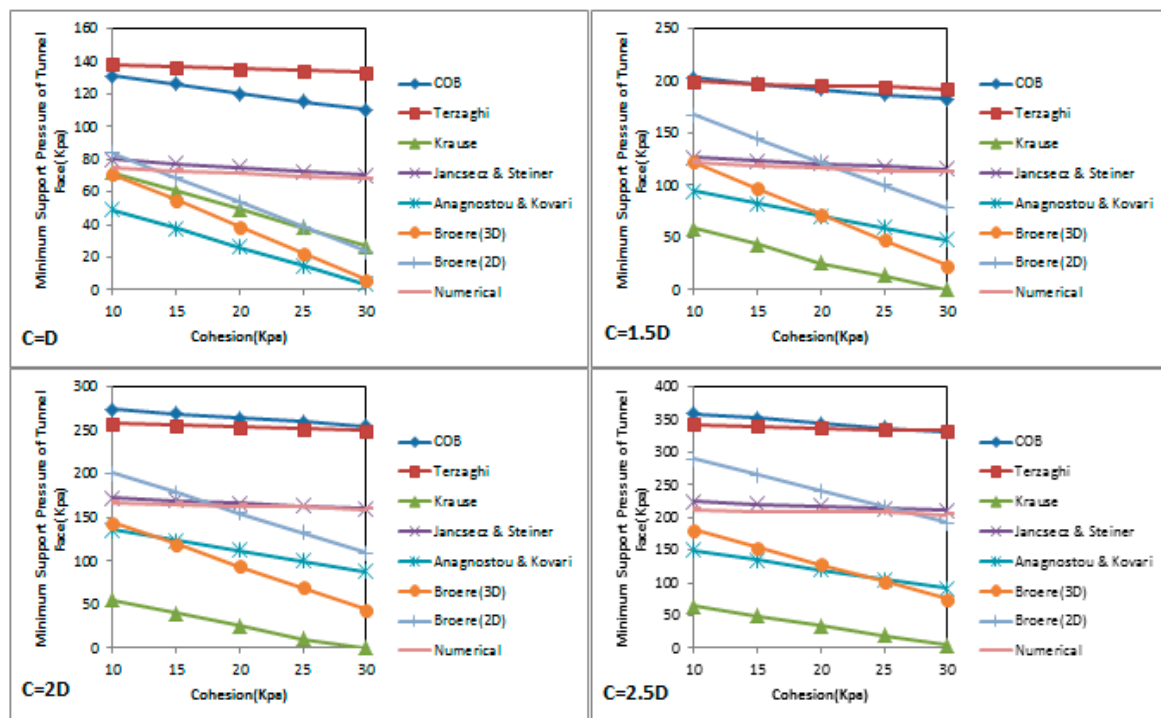


Figure 17. Relationship of cohesion with the support pressure of tunnel face for $\phi = 35^\circ$.

5.2. Effect of Friction Angle Variations on the Support Pressure of Face

Figure 18 depicts the effect of the friction angle (25° – 35°) on the minimum support pressure required at the tunnel face for various depths ($C = D$, $C = 1.5D$, $C = 2D$, and $C = 2.5D$) in EPB tunneling. As the friction angle increases, the support pressure decreases across all the methods, with more pronounced reductions in the numerical and advanced analytical models [7,16,30]. Traditional methods like COB and Terzaghi exhibit higher and less sensitive support pressures, remaining conservative. At greater depths (e.g., $C = 2.5D$), the required support pressures increase, but the benefit of the higher friction angles becomes more significant, particularly in the numerical methods, which show the steepest decline. This highlights the importance of incorporating accurate soil strength parameters (e.g., friction angle) into advanced models for optimizing face support design.

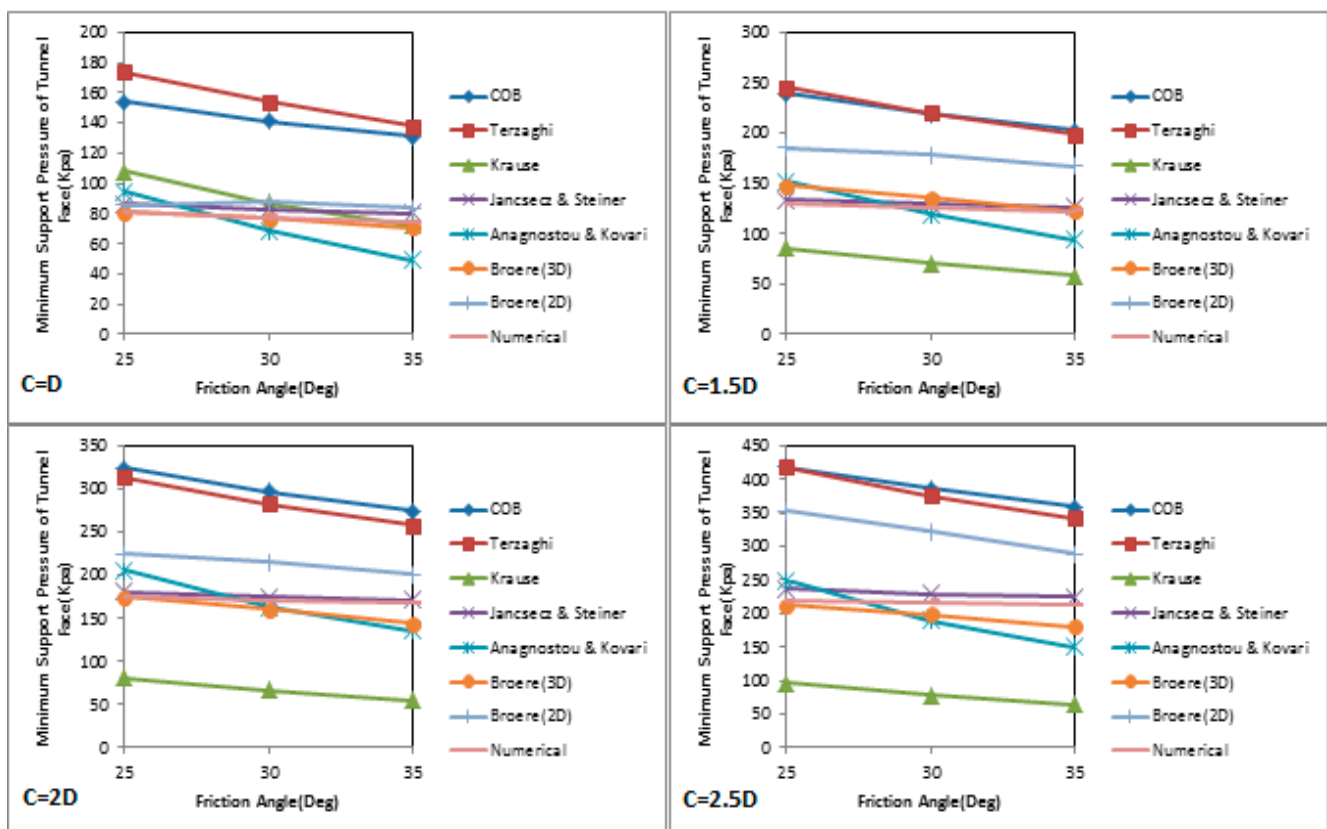


Figure 18. Relationship of friction angle with the support pressure of tunnel face for $c = 10$ kPa.

Figure 19 demonstrates the effect of the friction angle (25° – 35°) on the minimum support pressure at the tunnel face for different depths ($C = D$, $C = 1.5D$, $C = 2D$, and $C = 2.5D$) in EPB tunneling. As the friction angle increases, the minimum support pressure decreases across all the depths, with the numerical method and advanced approaches (e.g., [7,16,30]) showing significant reductions. The COB and Terzaghi methods remain conservative and less sensitive to the friction angle. At greater depths ($C = 2.5D$), the overall support pressures increase, but the effect of the friction angle becomes more pronounced, highlighting its role in reducing the required support. This emphasizes the importance of using advanced analytical and numerical methods that account for the friction angle's contribution to tunnel stability.

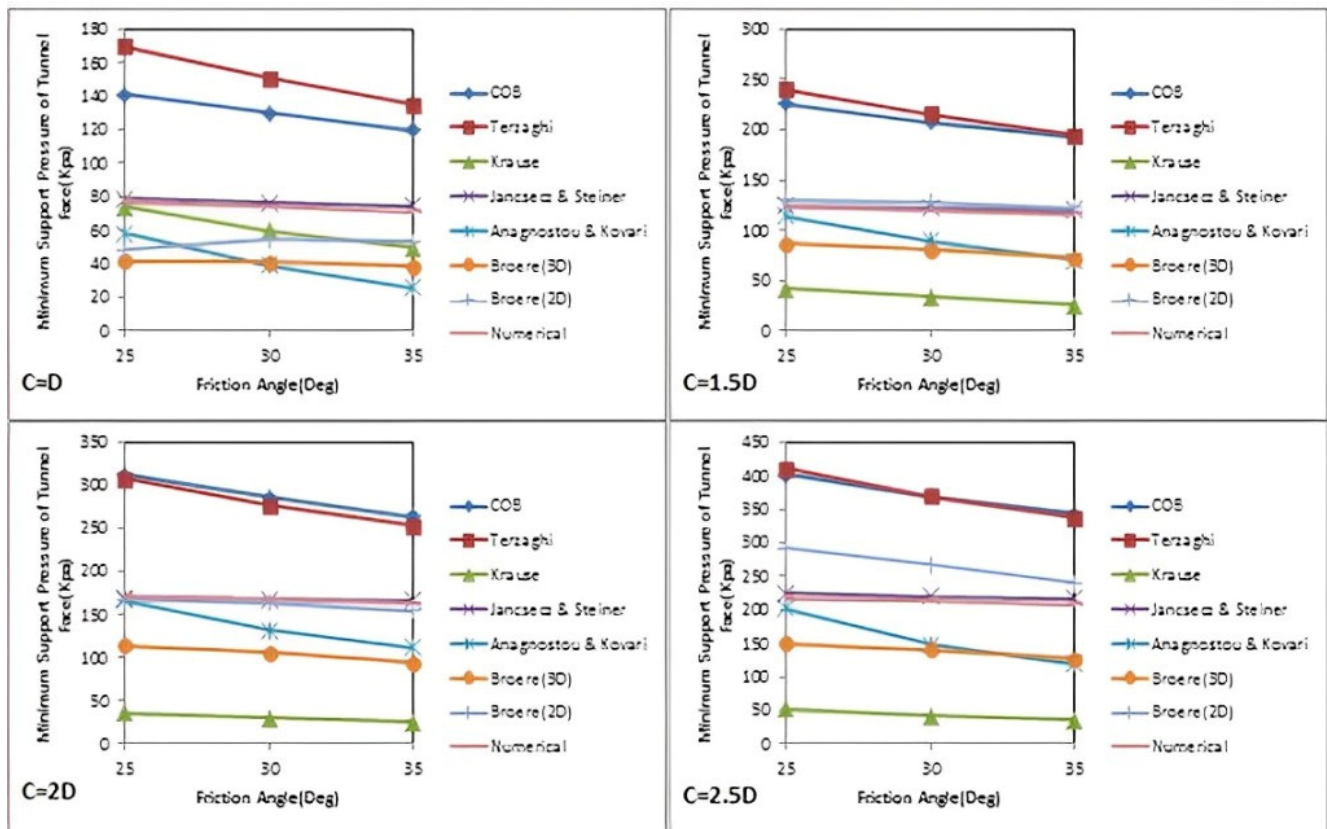


Figure 19. Relation of friction angle with the support pressure of tunnel face for $c = 20$ kPa.

5.3. Effect of Water Table Variations on the Support Pressure of Face

Figure 20 illustrates the relationship between the minimum support pressure of a tunnel face and varying water table depths for a sandy layer with cohesion of 30 kPa and a friction angle of 20° . As the water table drops from +4 m to -35 m (dry conditions), the pore water pressure decreases, increasing the effective stress and friction between the soil grains. This enhances the soil's arching ability, reducing the support pressure required at the tunnel face. The results consistently show that tunnels at shallow depths ($C = D$) require lower support pressures compared to deeper tunnels ($C = 2D$ and $C = 2.5D$) due to the increasing overburden stress with depth. The deepest tunnels ($C = 2.5D$) exhibit the highest support pressures, even as the water table is lowered, indicating the compounded effect of depth and stress. The comparison of the results from the numerical simulations, theoretical methods (COB, Terzaghi, Janacek & Steiner), and Broere's models [16] (2D and 3D) reveals distinct trends. Terzaghi's method consistently predicts higher support pressures, particularly for deep tunnels, making it more conservative. The numerical simulations align closely with Broere's 3D results, especially for greater depths, and show good agreement with the findings of Anagnostou and Kovári [7]. As the tunnel depth increases, the dependency on accurate methods becomes more critical, as deeper tunnels require significantly higher support pressures. The study highlights the importance of both tunnel depth and water table position in determining support pressures, emphasizing that reduced pore water pressure due to a lower water table can improve soil stability and minimize the necessary support forces.

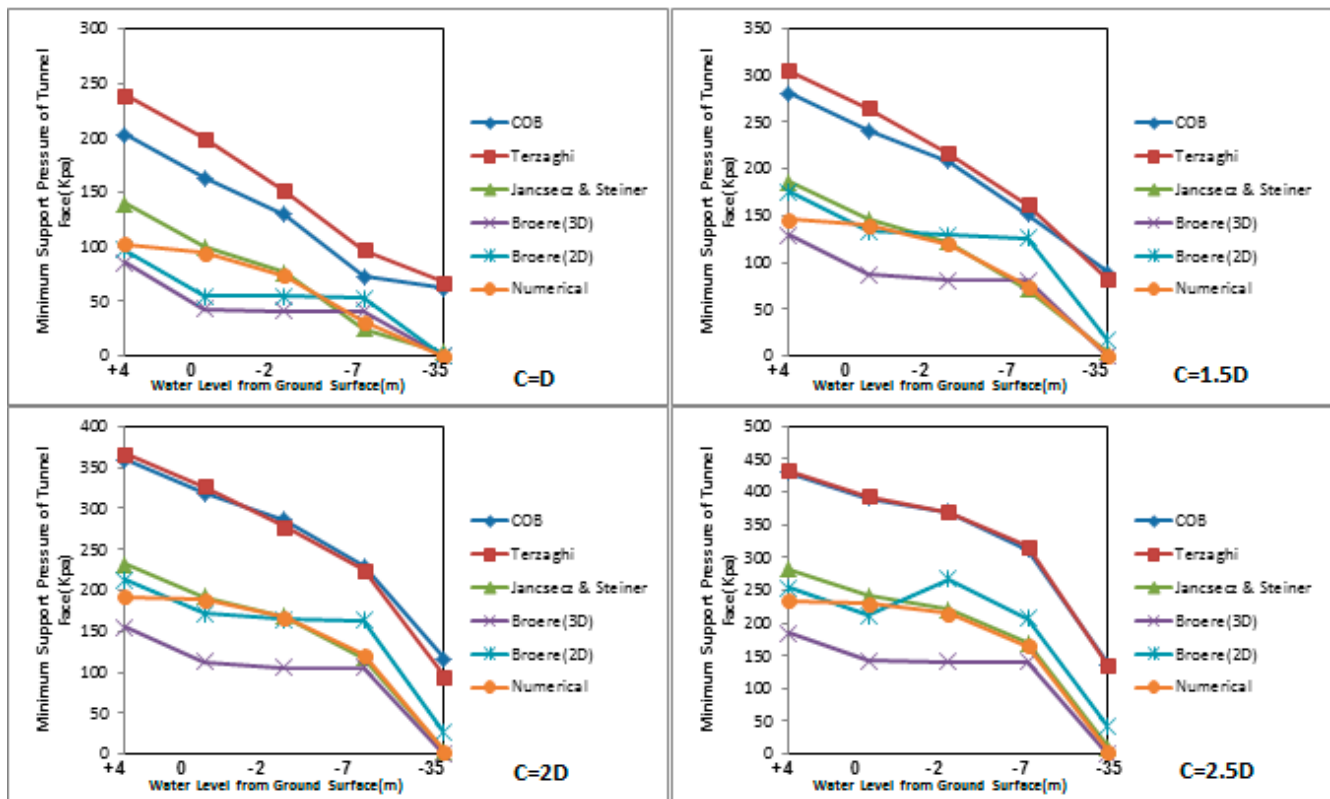


Figure 20. Relationship of water table with the support pressure of tunnel face for $c = 30$ kPa and $\phi = 20^\circ$.

5.4. Effect of Variation of Proportion of Overburden to Diameter on the Support Pressure of Face

Figure 21 shows the effect of the proportion of the overburden to the tunnel diameter on the minimum support pressure required at the tunnel face for two soil conditions: $c = 10$ kPa, $\phi = 25^\circ$ (left) and $c = 30$ kPa, $\phi = 35^\circ$ (right).

1. Left Graph (Weak Soil: $c = 10$, $\phi = 25^\circ$):

The support pressure increases significantly with increasing overburden proportion, particularly for COB, Terzaghi, and Broere (3D), which yield the highest pressures. The Kruse method shows relatively low and stable pressures, while the numerical and advanced approaches [7,16] provide more moderate predictions.

2. Right Graph (Strong Soil: $c = 30$, $\phi = 35^\circ$):

The overall support pressures are lower compared to the weak soil conditions. While the COB and Terzaghi methods still produce conservative values, and the Kruse method and numerical solutions show minimal increases with overburden, reflecting the influence of higher cohesion and friction angle on reducing pressure demands.

3. Key Insight:

In weak soils, the support pressure rises sharply with depth, highlighting the need for conservative methods. In strong soils, the increased strength parameters (higher cohesion and friction angle) significantly reduce the support pressures, particularly when advanced analytical or numerical methods are employed.

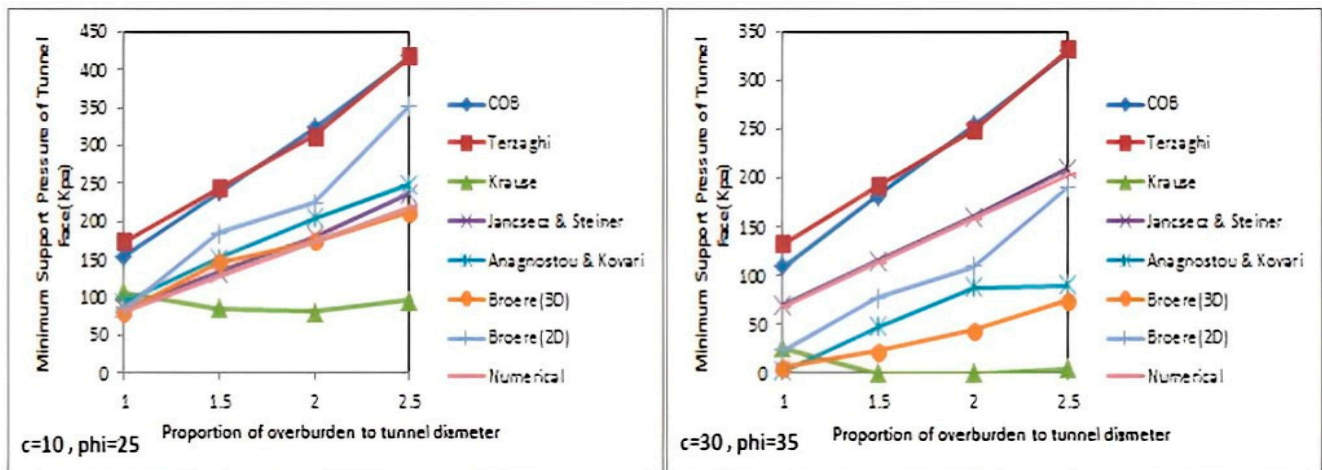


Figure 21. Relationship between proportion of overburden to tunnel diameter and the support pressure of tunnel face for $c = 30$ kPa and $\phi = 25^\circ$, and $c = 30$ kPa and $\phi = 35^\circ$.

6. Conclusions

This study references the Botlek rail tunnel in Rotterdam, excavated using an EPB shield with a diameter of 9.75 m and a cover depth of 22.7 m, as a case for verifying the estimated minimum support pressure at the tunnel face. The estimated range for the minimum support pressure was between 180 and 250 kPa.

The sensitivity analysis showed that as cohesiveness increased, the required support pressure to stabilize the tunnel face decreased. However, the evaluated support pressure did not align consistently with this trend. Similarly, as the friction angle increased, the support pressure also decreased, although it stabilized and showed minimal change across the range of friction angles considered. Ultimately, the support pressure needed for the tunnel face increased with the depth-to-diameter ratio. Under high resistance conditions, the pressure increase was gradual, owing to the enhanced effect of soil pressure arching.

The Broere [16] method, which incorporates three-dimensional pressure arching, yielded lower values compared to two-dimensional pressure arching. This is because the development of pressure arching over the tunnel face prevents the stresses from the overlying soil weight from reaching the tunnel face. Consequently, this difference becomes more pronounced as tunnel depth increases, due to the enhanced effect of soil pressure arching.

The validation of the numerical methods in this study was conducted through comprehensive comparisons with both analytical and empirical approaches. Key aspects of the validation include:

1. Benchmark Comparisons:

The numerical results were compared against established analytical methods, such as those by Jancsecz and Steiner [5] and Broere [16], which incorporate the pressure arching effects in tunnel face stability. This comparison demonstrated good agreement, particularly for cohesive soils, where the numerical and analytical pressures aligned well.

2. Empirical Correlation:

The empirical methods provided a range of minimum and maximum pressures for tunnel face stability. The numerical results obtained using Plaxis 3D (Version 2020) Tunnel software, fell within this range, validating their consistency with field observations.

3. Sensitivity Analysis:

Sensitivity studies on parameters such as cohesion, internal friction angle, and depth-to-diameter ratio were conducted. The trends observed in the numerical simulations

matched the expected behavior described by the analytical models, further supporting their validity.

4. Progressive Load Reduction:

A staged reduction in applied face pressure was modeled to replicate failure conditions. The calculated minimum pressure required for stability (170.6 kPa) aligned closely with the analytical estimates and empirical observations for similar conditions.

This multi-faceted approach ensured that the numerical methods effectively replicated real-world conditions and reliably predicted tunnel face stability.

The findings from the study contribute to sustainable tunneling practices in several ways, particularly in minimizing over-excavation and optimizing construction costs. The study establishes precise minimum support pressures (e.g., 170.6 kPa for the Tabriz subway section) through numerical, analytical, and empirical approaches. This ensures the tunnel face remains stable without excessive pressure, reducing over-excavation and associated soil displacement risks, which could otherwise lead to increased material handling and waste production. By determining accurate support pressures for Earth Pressure Balance Shields (EPBSs), the study facilitates optimized machine operation. Balancing pressure prevents tunnel face collapse and excessive deformation, improving operational efficiency and lowering energy consumption and repair costs. Incorporating the soil arching effect into pressure calculations allows for a reduction in support pressures. This effect minimizes the vertical stress transfer to the tunnel face, thereby reducing the amount of material that needs to be handled and optimizing resource use. The sensitivity analysis of the cohesion, friction angle, and water table variations enables better prediction of the necessary support pressures under various conditions, leading to tailored designs that avoid excessive conservatism and material usage. Therefore, these practices align with sustainable goals by reducing material waste, lowering energy demands, and preventing unnecessary environmental disturbances during tunnel construction.

Future research could investigate how grouting pressure, jack pressure, and the distribution of grain size at the tunnel face affect the required support pressure by modeling the installed segments. Furthermore, by deploying pressure sensors in the machine chamber and utilizing existing extensometers in the surrounding ground, a back analysis could be conducted to determine the accurate parameters for stabilizing the tunnel face. Moreover, the study would benefit from broader field data validation, especially from tunnels with different diameters, depths, and operational conditions to strengthen the generalizability of its conclusions. Future studies should address these limitations by including more comprehensive modeling of influencing factors and expanding the dataset to cover a wider range of soil conditions and construction scenarios.

The failure mechanism at the tunnel face predominantly exhibited a wedge mode, with the pressure disparity between the tunnel's top and bottom—caused by the density of the drilling mud—significantly influencing the stress distribution, as indicated by the numerical modeling. Consequently, analytical methods were deemed more suitable for determining the required support pressure at the tunnel face. These methods considered wedge and semi-circular mechanisms as the most probable failure modes. Therefore, the analytical approaches developed by Jancsecz and Steiner [5], Anagnostou and Kovári [7], and Broere [16] were well-suited to addressing these conditions.

The findings highlight the importance of balancing safety and efficiency. Optimized pressure calculations and settlement control reduce material waste and energy consumption, aligning with sustainable construction practices. These insights help in designing safer, more efficient tunnels that minimize disruption to urban infrastructure and enhance the economic and environmental feasibility of urban tunneling projects.

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