LOAD ANALYSIS IN CONSTRUCTION OF TUNNELS BY THE PIPE UMBRELLA SYSTEM

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Summary: *With the intensive construction of the highways, intensive tunnelling works takes place. Contemporary methods of construction are applied. One of these methods is the application of the pipe umbrella system as an excavation support. This primary support system is most commonly used in weak rock masses. Considering this type of supporting system, a special problem is the stability calculation and analysis of loading.*
In this paper, the individual procedures to be used for analysis of the load when calculating the bearing capacity of the pipe umbrella are reviewed, based on the most recent researches conducted in this field. There is also one example of a load calculation.

Keywords: *rock mass, tunnel, pipe umbrella, load analysis*

1. INTRODUCTION

The steady rise of population in large cities, density of transportation, and need for storage capacity have led, inevitably, to an increased use of underground structures in modern civilisation. For the reasons of the overpopulation and the lack of space, tunnels have a significant role in the development of urban areas. In urban areas tunnels are being designed at shallow depths. In this case, a special attention should be focused on tunnelling-induced settlements of ground surface, as they have influence upon preexisting structures on the surface, and by that, may cause their damage or even failure. In non-urban areas tunnels are also often excavated in areas of week or difficult ground.

In urban areas, overburden is usually especially thin, whereasthe ground is generally comprised ofsoft soil and/or highly weathered rock mass, which are prone to large displacements during tunnel construction. In order to achieve ground settlement control (i.e., to restrict ground surface settlement), which is of paramount importance, so that the minimum disturbance is caused to surface structures, many techniques have been developed: deep slurry trenching, proprietary underpinning system, micropilling, as well as the use of compressed air or freezing in tunnel construction. In addition,there are different methods of forepoling, such as the sub-horizontal jet grouting method, the

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spiling method, and the pipe roof (pipe umbrella) method, in which case an arch-like shell is created ahead of the face prior to excavation, thus enabling tunnel excavation to be carried out safely and speedily under a protective arch.
Although the pipe umbrella systemis less stiff in comparison with other pre-supporting systems,it’s become a widely used method in tunnel construction, owing to the facts that it isless time consuming and cost intensive. A forepoling by the pipe roof method is formed in a crown of a tunnel, in weak ground conditions in conventional as well as mechanised tunneling, by installing a series of large diameter steel tubes in an arch or a ring,from the current working face out to a distance on the range of 12 m to 15 m in front of the face advance, thus providing the stability and safety in the working area. The pattern of pipes is arranged in a manner so that it outlines the tunnel extents (Fig. 1). Diameters of the pipes range from 70mm to 200mm.Typical installation methods are pipe jacking and other micro-tunneling methods. Pipe roof method pipes are designed to carry longitudinal loads only.Each pipe transfers the loads from the supported areas to the less critical areas, which are used as abutments [1]. They are typically made from open shafts and can be driven parallel to tunnel axis.This system is also applied to increase stability in portal sections, for the re-excavation of collapsed sections in underground construction, and as a ground improvement and waterproofing technique in combination with all tunnel construction methods.

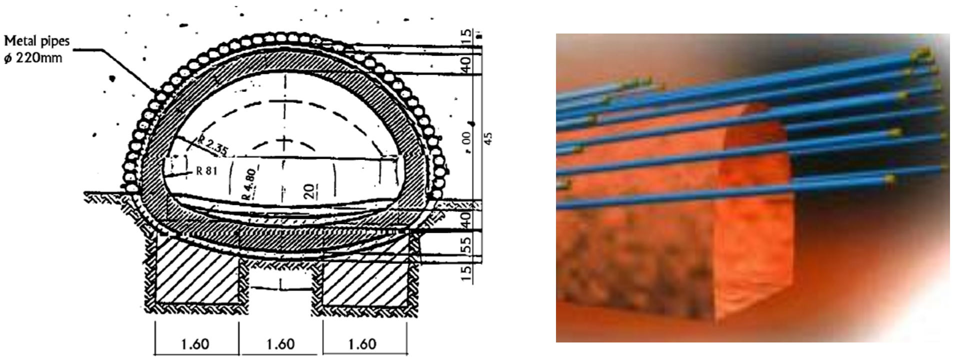

Figure 1.Schematic drawing of the pipe umbrella method [2]

The forepoling umbrella system in soft soil and weak rock tunnelling has been in the focus in a number of studies [3-7]. The main problem of application of a pipe umbrella is to determine the load of the rock mass considering the ground–support interaction that is associated with this system. The individual procedures to be used for analysis of the load when calculating the bearing capacity of the pipe umbrella are reviewed in the subsequent part of the paper, based on the most recent researches conducted in this field [8-10,12].

2. BUILDING THE ANALYTICAL MODEL

One of the approaches in defining the reliable and relevant analytical models concerning the pipe umbrella system is presented in [8]. In this study, the following assumptions have been adopted:

- The force of the pipe-roof is definite, and the action between the upper soil and the pipe-roof is not considered.
- The pipe-roof is the straight beam acting on the Pasternak elastic foundation, and the uniform load $q(x)$ of the covering soil acts on the pipe-roof in the excavation section.

Owing to the deformation of the rock mass, which begins with the range in front of the tunnel face,the maximum relaxation range of the rock mass is within therange of the fracture plane in front of the tunnel face.According to the theory of rock mass pressure, the acting range of the longitudinal load of the pipe-roof in front of the tunnel face is $h \cdot (\tan 45° - \varphi/2)$. Considering the state of the pipe-roof and the rockmass during excavation, the pipe-roof can be divided into four segments in the excavation cycle as presented in Figure 2.

(1) *The support section (OA)*: this section canbe assumed as an elasticfixedendwith an initialdisplacementy_0 and an initial angle θ_0 (the initial displacement y_0 ofthe pipe-roof can be valued as themeasured settlement of the crown).
(2) *The section (AB) without support*: the pipe-roof completely bears the pressure $q(x)$ of the upper rockmass.
(3) *The section (BC) in the disturbance*: the pipe-roof is affected not only by the rock mass pressure of $q(x)$, but also by thesubgrade reaction $p(x)$.
(4) *Undisturbed section (CD)*: this section only bears the subgrade reaction $p(x)$ between the pipe-roof and the rock mass.

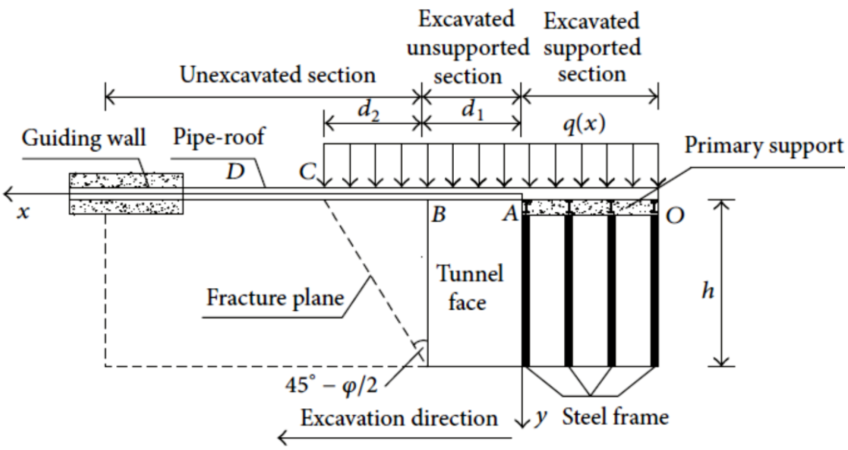

Figure 2. Force of pipe-roof in the process of tunnel construction [8]

The elastic foundation beam model of the pipe-roof is divided into two types,according to the position of the tunnel face and the pipe-roof.

(1) When the tunnel face is far away fromthe front end of the pipe-roof, the affected area of tunnel excavation does not reach the front end of the pipe-roof, which is considered an infinite elastic foundation beam (Fig.3(a)).

(2) When the tunnel face is in the proximity to thefront end of the pipe-roof, the affected area of tunnel excavation reaches the front end of the pipe-roof, which is considered a finite elastic foundation beam (Fig.3(b)).

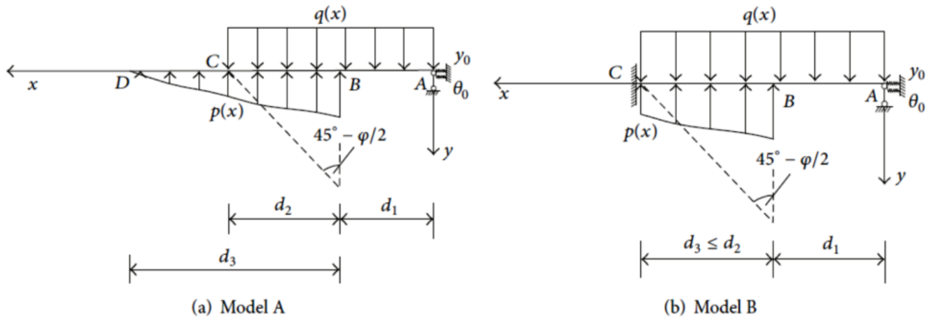

Figure3. Mechanical model of pipe-roof in the process of tunnel construction [8]

With regard to the tunnel portal construction, at tunnel exit, the tunnel face starts from inside tooutside of the tunnel, the far end of pipe-roof is restrained by the guiding wall, and the corresponding constrained condition at the far end of pipe-roof can be idealised as a fixed support as shown at the point*C*in Figure 3(b).At tunnel entrance, on the other hand, the tunnel face starts from outside to inside of the tunnel, and the constrained condition at the pointCis usually idealised as a hinged support.

3. LOAD MODELS OF ROCK MASS

As it was noted previously, application of the pipe roof system is associated with a problem of determination of the load of the rock mass considering the ground–support interaction. The models for analysis of the load in assessing the bearing capacity of the pipe umbrella, proposed by a number ofresearchers, are reviewed in detail in the studies [9,10].

In general, there are two types of semi-analytical solutions suitable for the forepole loading scenarios (Fig.4). The first type is related to a simplified beam analysis with specified reaction points and loading conditions. The second type employs elastic foundation theory.

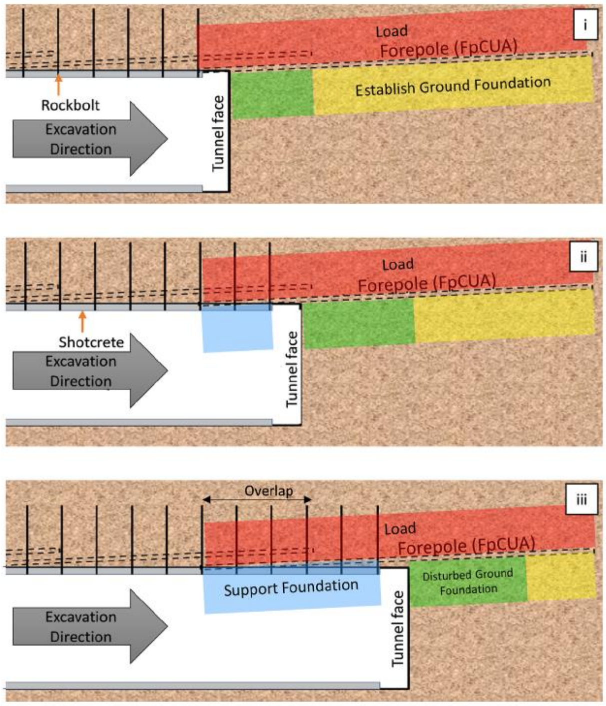

Figure 4. Illustration of the three different ways in whichthe forepole element interacts with the ground [9,10]

3.1. Harazaki et al. (1998)

In their analysis,the authors employed a simplified beam model to capture the moment profile of a forepole element. The load model, the simplified beam model, and *in-situ* results of for the alluvial soil are illustrated in Figure5. The alluvial zone consisted of uncemented sand and gravel, with a modulus of elasticity ranging from 0.5 to 5 MPa, standard penetration test values ranging from 5 to 40 (blow counts), and a final surface settlement of 100–120 mm.

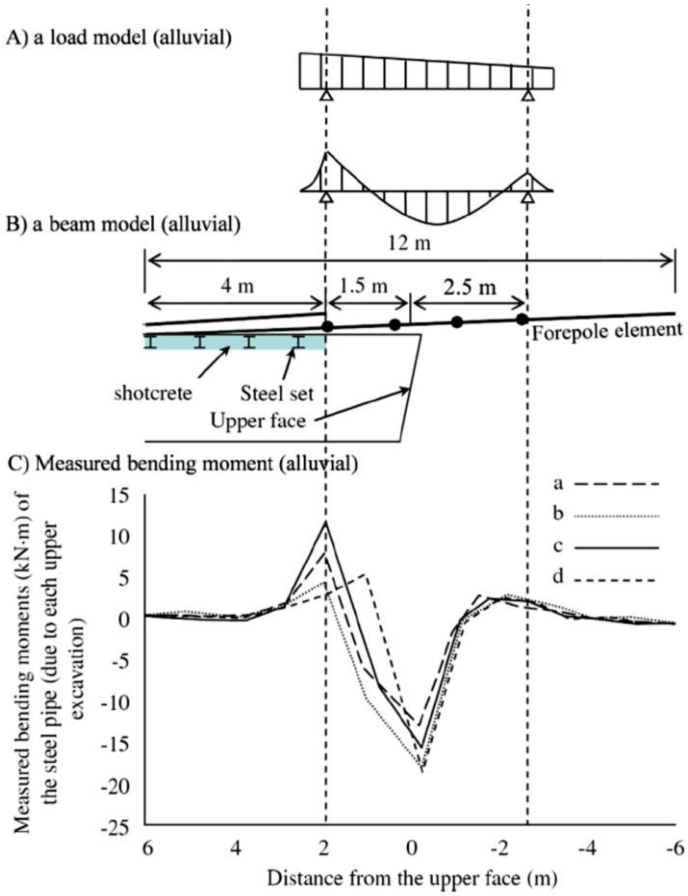

Figure 5. Illustration of the model proposed by Harazaki et al. (1998)based on measured bending moments in alluvial ground conditions [9,10]

3.2.John and Mattle (2002)

John and Mattle (2002) also performed a simplified beam analysis (Fig. 5C)) in order to determine an approximation for the maximum applied bending moment (Fig. 5D)) of the forepole structural element. The analysis was based on the results of a numerical axisymmetric finite element method. In accordance with the axisymmetric assumption, the forepole elements were simulated as a closed ring of horizontal supports (Fig. 5A)). The analysis simulated excavation advance rate of 1 m (Fig. 5B).Based on the uniformly distributed loading condition, the maximum bending moment always occurs at the fixed reaction point, within the unexcavated tunneling section or at the tunnel face, which is not in line with the findings of Harazaki et al. (1998).

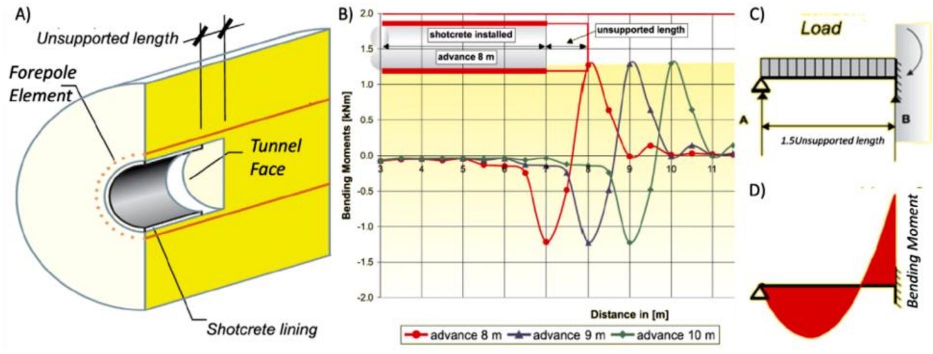

Figure 6.Illustration of the creation of the model after John and Mattle (2002) [9,10]

3.3. Peila and Pelizza (2003)

These two authors proposed a similar model to Harazaki et al. (1998) and John and Mattle (2002), based on their empirical experience (Fig. 7). In their analysis, the following assumptions were made:

- The concrete (grout), filling and surrounding the pipe (i.e. forepole), is not considered in the calculation.
- The analysis is performed for the most critical stage – just before the installation of the steel rib, whereas the free span is the longest.
- The length ahead of the tunnel face, which is not acting as support for the pipes (Fig. 7), is usually empirically chosen and very often the value of 0.5 m is assumed.
- The loading condition is defined by a fraction (0.5–0.75) of the well-known formulation of Terzaghi.

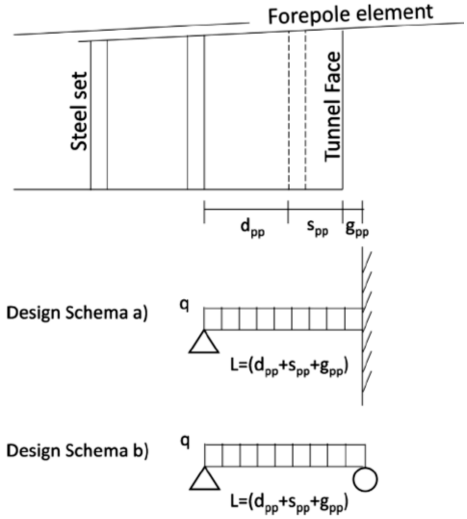

Figure 7.Illustration of the model proposed by Peila and Pelizza (2003) [9,10]

According to the authors, the length g_{pp} is directly related to the geomechanical properties of the ground and to the presence of tunnel-face reinforcements.For design schema a (Fig. 7), the maximum moment will always occur at the fixed reaction boundary (within the unexcavated portion), as in the case of John and Mattle (2002). For design schema b (Fig. 7), however, the section of occurrence of the maximum moment will depend on the length g_{pp}. In case when g_{pp} is greater than the sum of d_{pp}ands_{pp}, the maximum moment will occur within the unexcavated portion of the forepole element, similar to the alluvial zone from Harazaki et al. (1998). On the other hand, if g_{pp} is less than the summation, the maximum moment will occur within the excavated portion for the forepole element, similar to the diluvial zone from Harazaki et al. (1998). For that reason, the authors point out to the further research in determining this length.

3.4. Oreste and Peila (1998)

In distinction from the previously described models, the model proposed byOreste and Peila (1998) is based on an elastic spring foundation in order to represent the support systems and ground conditions of the forepole element response. In comparison to the simplified beam models, this model allows for a more realistic deformation profile for the entire forepole element. Nevertheless, the results indicate thata maximum positive moment will occur a fair distance ahead of the face (>2m, the middle of Fig. 8), whereas a large shear force is found ahead of the face (>1.5m, the bottom of Fig.8) at the transition point from a load being applied to no load being applied to the forepole. The main reason for such results is the independency of the springs used in the proposed model. One of the options for overcoming this issue is establishing spring dependency through the inclusion of a shear component.

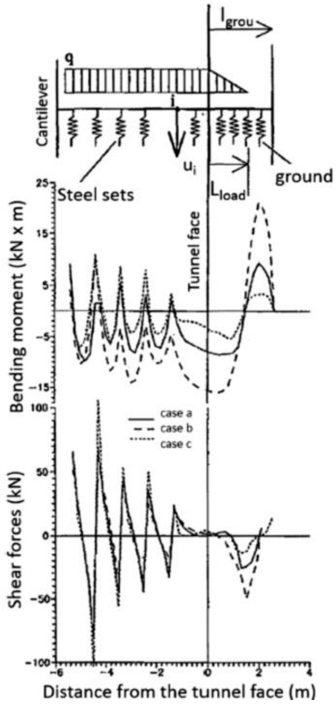

Figure 8. Proposed model of Oreste and Peila (1998) and resulting bending moment and shear force from a parametric analysis on the moment of inertia (J) and support stiffness (k) [9,10]

3.5.Wang and Jia (2008)

This model employs a Pasternak foundation, which is essentially a Winkler spring model with an additional shear component.The model assumes that a distance of 1.5 times the unsupported span from the tunnel face can be simulated as a fixed reaction to represent the support system, (portion from A’ to B, Fig. 9).

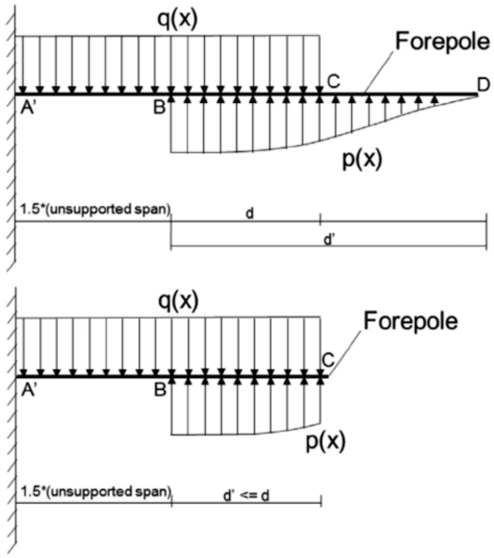

Figure 9.Illustration of the model proposed by Wang and Jia (2008) [9,10]

3.6. Song et al. (2013)

The authors proposed a model (Fig. 10) based on the *in situ* data from the analysis of Harazaki et al. (1998) and numerical model analysis. The model simplifies most of the distributed loading condition to point loads at the location of foundation springs (i.e., steel sets and shotcrete). This simplification results in quite low values of induced moments and low shear forces within the supported section of the analysis.

3.7. Volkmann and Schubert (2010)

This model, depicted in Figure 11,is based on the numerical analysis and *in situ* data collected by Volkmann and his collaborators for considered shallow-laid tunnel excavations. The authors did not propose any specific method for solving this model. However, they emphasisedthe difficulty of approximating the loading conditions. Taking into consideration that this model was proposed for the case of shallow excavations, the question that arises iswhether the mechanical response will be the same for the case of deep excavations related to squeezing-ground conditions.

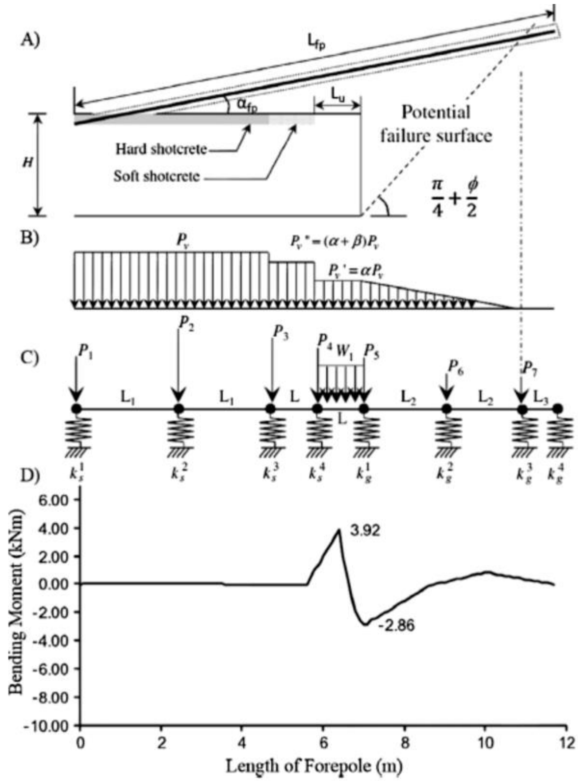

Figure 10.Illustration of the model proposed by Song et al. (2013):
(A) Diagram of setup of model. (B) Illustration of distribution of load. (C) Analytical model. (D) Example of results from a parametric analysis by Song et al. (2013) [9,10]

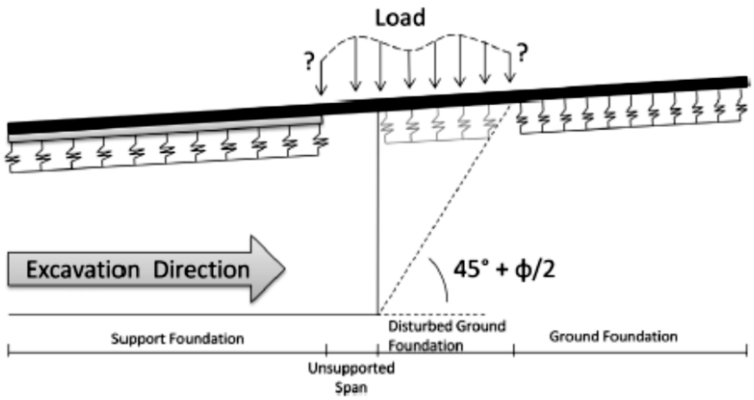

Figure 11. Illustration of the proposed model of Volkmann and Schubert (2010) [9,10]

3.8. Terzaghi method

The Terzaghi model of loading imposed to the support system of an rectangular-shapedunderground structure is illustrated in Figure 12. This model is applicable for the case of shallow-laid underground structures ($D \leq 5B_1$) [11].

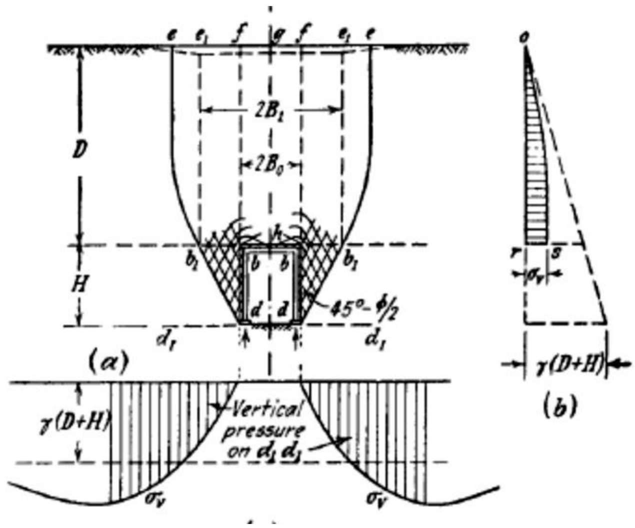

Figure 12.Schematic representation of the Terzaghimodel[12]

$$\sigma_v = \frac{B_1 \times \gamma - c}{K \times \tan\varphi}\left(1 - e^{-K\frac{D}{B_1}\tan\varphi}\right) \tag{1}$$

$$2B_1 = 2B_0 + 2 \times H \times \tan\left(45 - \frac{\varphi}{2}\right) \tag{2}$$

where:

- $2B_1$ is the width of the rock mass that moves after the excavation of the tunnel profile;
- $2B_0$is the width of the tunnel;
- $H$is the height of the tunnel;
- Disthe overburden depth;
- φ isthe angle of internal friction;
- c iscohesion;
- γ is the volume weight;
- K= 1– sin φstands for the ratio of horizontal and vertical pressures.

4. NUMERICAL EXAMPLE

In this part, an example of a load calculation based on the Terzagi model is presented. The entire length of the tunnel is required to be supported by the pipe umbrella system, whereas an excavation advance rate is 1.0 m.

- The properties of the rock mass: γ= 18 kN/m^3; φ= 19°; c=16kN/m^2;
- Data for thetunnel geometry: $2B_0$=7.0 m; H=5.0 m; D =20.0 m.

1) **Calculation of load σ_v:**

$2B_1$ = 7.0+2×5.0×tan(45°–19°/2) = 14.13 m $\rightarrow B_1 \approx$ 7.1 m;

K= 1 – sin19° = 0.6744; $-K \times D \times \tan\varphi / B_1$ = –0.6744×20.0×tan19°/7.1= –0.654;

$\sigma_v = (B_1 \times \gamma - c) \times (1 - e^{-0.654})/(K \times \tan\varphi)$= 231.32 kN/m^2.

Based on the model presented inFigure 12, a static model of the forepole-element loading condition is given inFigure 13.

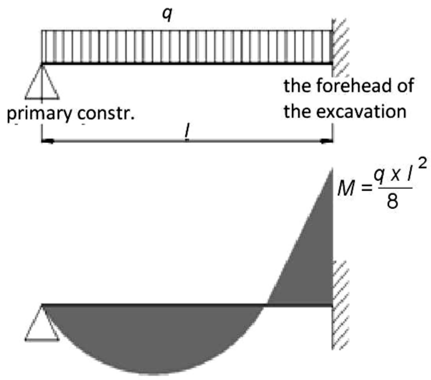

Figure 13. Static model[12]

2) **Calculation of load q and moment M:**

$$q = p_v \times i \tag{3}$$

where: - p_v= 0.65 ×σ_v (assuming that 65% of the total vertical load determined by the application of the Terzaghi method is at the forefront of the excavation);
- i is the distance of the pipesin the pipe umbrella system(0.20 m);
- ℓ= 1.5 ×unsupported span(in this example 1.0 m);

σ_v = 231.32 kN/m^2
p_v = 0.65 × 231.32 = 150.36 kN/m^2
q = 150.36× 0.20 = 30.07 kN/m
ℓ = 1.5 × 1.0 = 1.5 m

$M = (q \times \ell^2)/8 = 8.457$ kNm

Pipediameter is ∅114.3mm, wall thickness is of 8.0 mm filled with concrete.

$$I = I_{steel} + \frac{1}{n} I_{concrete} = 379 + \frac{1}{\frac{210000}{31500}} \times \frac{(11.83 - 1.6)^4 \pi}{64} = 379 + \frac{458.34}{6.67} = 447.72\ cm^4$$

$$W = \frac{447.72}{11.43 \times 0.50} = 78.34\ cm^3$$

$\sigma = M / W = 8.457 \times 10^2 / 78.34 = 10.80$ kN/cm^2;

$F_s = \sigma_{all} / \sigma = 16.0$ kN/cm^2/10.8 kN/cm$^2 \Rightarrow F_s \approx 1.50$ (safety coefficient);

where is $\sigma_{all} = 16.0$kN/cm^2 (allowed stress in steel);

5. CONCLUDING REMARKS

The pipe umbrella support system, also known as the pipe roof method, is commonly used forepoling system in tunnelling operations and is generally employed under the conditions of the existence of shallow overburden above the tunnel, the need to restrict ground surface settlement, and poor ground conditions. Pipe umbrellas serve as a method of pre-support in underground excavations to increase stability in the working area and decrease deformations due to construction.A forepoling by the pipe roof method is formed by installing a series of large diameter steel tubes from the current working face in front of the face advance, with the pattern of pipes arranged in a manner so that it outlines the tunnel extents. Pipe roof method pipes are designed to carry longitudinal loads only.

The main problem associated with the application of the pipe umbrella system is the stability calculation and analysis of loading. Many authors have been dealing with this issue, and as a result, numerous procedures and models for analysis of the load when calculating the bearing capacity of the pipe umbrella are proposed.In general, two types of models prevail the analyses of forepole element loading scenarios: the simplified beam model and the elastic spring foundation model. The proposed modelsare developed with certain assumptions and simplifications, and in many cases, based on case studies and/or on empirical experience of the authors. Although a major step has been made in resolving the problem related to the loading conditions of forepoling elements, however, the general conclusion that could be drawn is that there is no single model that would cover all relevant cases, and accordingly,further research in this area would definitely be needed.

ACKNOWLEDGEMENTS

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АНАЛИЗА ОПТЕРЕЋЕЊА КОД ГРАЂЕЊА ТУНЕЛА СИСТЕМОМ ЦЕВНОГ КИШОБРАНА

Резиме: *Са интензивном изградњом аутопутева одвија се и интензивна изградња тунела. Примењују се савремене методе изградње. Једна од тих метода је примена система цевног кишобрана као осигурања ископа. Овај систем примарне подграде најчешће се користи у слабим стенским масама. При томе, посебан проблем представља прорачун стабилности и анализа оптерећења.*
У оквиру овог рада приказују се поједини поступци који се користе за анализу оптерећења при прорачуну носивости цевног кишобрана, а на бази најновијих истраживања у овој области.Takођеје приказан и један пример прорачуна оптерећења.
Кључне речи: *стенска маса,тунел,цевни кишобран, анализа оптерећења*