

ANALYTICAL APPROACH TO DETERMINE LONGITUDINAL DEFORMATION OF THE EXISTING PRECAST TUNNEL DURING CONSTRUCTION OF A FULL-LENGTH EXCAVATION PIT

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Abstract: In the realm of urban construction employing excavation techniques, safeguarding existing underground structures from detrimental consequences arising from surface construction operations poses a formidable challenge. The reduction of loads due to excavation activities can induce unintended responses, potentially jeopardizing subterranean infrastructure, particularly high-safety-demanding structures like Tunnel Boring Machine (TBM) tunnels. This article introduces an uncomplicated method for ascertaining the axial displacement of TBM tunnels amidst concurrent surface excavation activities. Primarily, the approach entails the identification of stress variations encountered during soil excavation at the tunnel face. Subsequently, employing the solutions derived for the determination of tunnel deformation subjected to concentrated loads, the deformation incurred by the tunnel due to alterations in excavation-induced stress is quantified. The analytical outcomes are meticulously juxtaposed against results generated from a three-dimensional computational model. The comparative analysis demonstrates that the displacement values and axial deviations calculated using the proposed analytical method exhibit only marginal disparities of 4,3% and 1%, respectively, when compared to those obtained through finite element analysis. This study underscores the efficient predictive capabilities of the analytical method in assessing tunnel deformations, enabling a preliminary estimation of critical parameters associated with the excavation pit. These findings have significant implications for mitigating adverse impacts on existing subterranean infrastructure in densely populated urban areas.

Keywords: existing tunnel, TBM tunnel longitudinal deformation, analytical approach, excavation pit settlement, full-length

АНАЛИТИЧЕСКИЙ ПОДХОД К ОПРЕДЕЛЕНИЮ ПРОДОЛЬНОГО СМЕЩЕНИЯ СУЩЕСТВУЮЩЕГО СБОРНОГО ТОННЕЛЯ ПРИ ВЫЕМКЕ КОТЛОВАНА ПО ВСЕЙ ДЛИНЕ

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Анотация: При проведении земляных работ в условиях городской застройки защита существующих подземных сооружений от пагубных последствий, возникающих в результате наземных строительных работ, представляет собой сложную задачу. Снижение нагрузок в результате земляных работ может вызвать непредвиденные реакции, потенциально ставя под угрозу подземную инфраструктуру, особенно конструкции с высокими требованиями к безопасности, такие как сборные тоннели (ТБМ). В данной статье представлен простой метод определения осевого смещения тоннелей ТБМ при создании котлована над тоннелем. Прежде всего, этот подход предполагает выявление изменения напряжений, возникающего во время выемки котлована над тоннелем. Впоследствии, используя решения, полученные для определения деформации тоннеля, подвергающегося сосредоточенным нагрузкам, количественно определяется деформация тоннеля вследствие изменения напряжений, вызванного выемкой грунта. Аналитические результаты тщательно сопоставляются с результатами, полученными на основе трехмерной вычислительной модели. Сравнительные результаты показывают, что значения смещения и осевые отклонения, рассчитанные с использованием предложенного аналитического метода,

демонстрируют лишь незначительные расхождения в 4,3% и 1% соответственно по сравнению со значениями, полученными с помощью конечно-элементного анализа. Это исследование подчеркивает эффективные прогностические возможности аналитического метода при оценке деформаций тоннеля, что позволяет предварительно оценить критические параметры работ по выемке котлована. Эти результаты имеют важное значение для смягчения негативного воздействия на существующую подземную инфраструктуру в густонаселенных городских районах.

Ключевые слова: существующий сборный тоннель, продольное смещение тоннеля ТБМ, аналитическое решение, осадка котлована, по всей длине

1. INTRODUCTION

In recent times, there has been a rapid surge in construction demand in Vietnam, particularly in urban areas. Major cities have initiated the deployment of their first underground railway systems to cater to the increasing mobility needs of the populace. With a growing population and escalating transportation requirements, the expansion and development of underground railway networks within urban centers have become a top priority. Moreover, due to space constraints within cities and the trend towards developing compact urban clusters around subway stations, there has been a significant increase in the construction of commercial buildings and transportation infrastructure near subway lines. The continuous development of urban transportation infrastructure has also contributed to this phenomenon. This has led to situations where construction activities must take place in close proximity and at different elevations to intersect with the subway tunnel system.

During the construction of excavation pits above tunnels, changes in load due to excavation activities can have significant and even damaging effects on underground tunnels. A notable example is documented in the Pachio project in Taiwan [1], where the construction of a nearby building caused damage to the TBM (Tunnel Boring Machine) tunnel lining. Therefore, assessing tunnel deformation and predicting tunnel behavior when subjected to changes in surface loads is crucial, particularly in the context of today's large cities.

The behavior of existing tunnels below has attracted the attention of many researchers and has been approached through various methods. Among these methods are monitoring methods [2], physical modeling methods [3,4], numerical methods [5,6,7,8,9], and analytical methods [10,11,12].

Among these methods, conducting field experiments or using centrifuge machines has the drawback of long cycles and high costs. The utilization of finite element software methods can model complex intersection structures and depict tunnel-terrain interactions. However, it demands expenses for commercial software and expertise in handling intricate models. With such models, it consumes a significant amount of time and requires high-performance computers.

An analytical approach is a fast and cost-effective method to predict the deformation of tunnels subjected to surface excavation. This can be considered as a suitable approach in the early stages of project design and construction. In the published studies, the tunnel within the terrain is simplified into a problem of a beam on an elastic foundation, employing either the Euler-Bernoulli or Timoshenko beam models on a Winkler or Pasternak foundation model. In this paper, the authors present an analytical approach, with the tunnel modeled using the Euler-Bernoulli beam, and the surrounding soil modeled using a Winkler foundation model. For comparison, the analytical results will be compared with a 3D finite element model based on data from the urban railway line No.1 in Ho Chi Minh City.

2. DEFORMATION PROCESS OF TUNNELS DUE TO FOUNDATION EXCAVATION

Before excavation, the interaction between the tunnel and the subsurface is stable, and the tunnel is in a state of force equilibrium, as shown in Fig. 1a. When excavating the soil above the tunnel, the stress state of the subsurface changes, affecting the structure of the tunnel below. Vertical excavation-induced loading results in the uplift of the soil beneath the excavation pit and an upward displacement of the tunnel below due to a reduction in the overlying soil pressure. The lateral stress on both sides of the tunnel is increased due to the confining effect of the soil. The influence of the excavation-induced loading on the overlying soil pressure of the tunnel is reduced, and the cross-sectional shape of the tunnel appears as a vertically elongated ellipsoid, as shown in Fig. 1b.

3. ANALYTICAL APPROACH

In the analytical approach, several assumptions are made to simplify the problem: 1. The soil environment is considered as a linearly elastic, isotropic, and homogeneous medium; 2. The tunnel is treated as a beam on an elastic foundation (1D problem), without considering deformations within the cross-section, and it is assumed to have infinite length.

3.1. Load acting on the Tunnel lining

After excavation, an uplift pressure, $p(x)$, directed upward, is generated at the bottom of the excavation pit. $p(x)$ represents the change in soil load removed per unit area. In the case of a full-length excavation pit, the 3D environmental problem becomes a 2D deformation problem (Figure 2) with a uniform distributed load, p_0 , within the segment $2a$, corresponding to the boundary conditions:

$$p(x) = \begin{cases} -p_0, & |x| < a \\ 0, & |x| > 0 \end{cases} \quad (1)$$

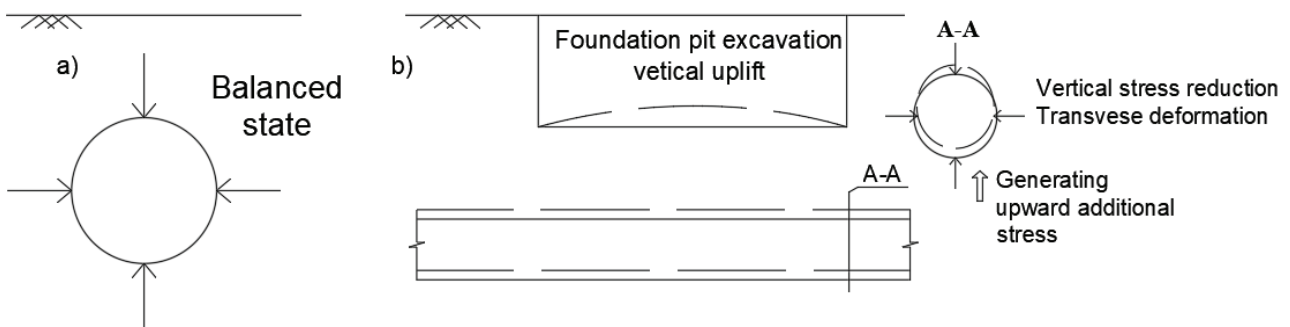


Figure 1. a) Pre-Excavation balanced State; (b) Changes in Stress-Strain State during Excavation

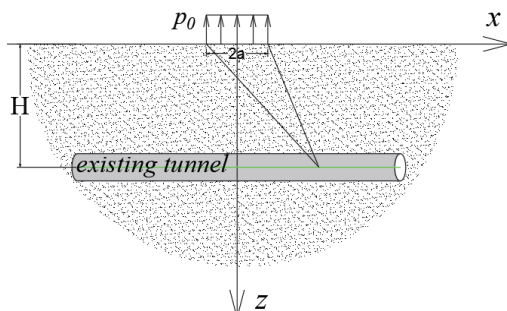


Figure 2. Force Distribution in a homogeneous elastic half-space

The problem of a uniformly distributed pressure on an infinite half-space surface was solved by Sneddon using Fourier transforms in 1951 [13], with the stress distribution determined as:

$$\begin{aligned}\sigma_{xx} &= -\frac{p_0}{\pi} \left(\arctan\left(\frac{x+a}{z}\right) - \arctan\left(\frac{x-a}{z}\right) \right. \\ &\quad \left. - \frac{(x+a)z}{(x+a)^2+z^2} + \frac{(x-a)z}{(x-a)^2+z^2} \right) \\ \sigma_{zz} &= -\frac{p_0}{\pi} \left(\arctan\left(\frac{x+a}{z}\right) - \arctan\left(\frac{x-a}{z}\right) \right. \\ &\quad \left. + \frac{(x+a)z}{(x+a)^2+z^2} - \frac{(x-a)z}{(x-a)^2+z^2} \right) \\ \sigma_{xz} &= -\frac{p_0}{\pi} \left(\frac{z^2}{(x+a)^2+z^2} - \frac{z^2}{(x-a)^2+z^2} \right)\end{aligned}\quad (2)$$

In this problem, the influence of the tunnel on stress generation, as well as the longitudinal and shear stresses within the tunnel, are neglected. In this case, the vertical stress at a depth H, induced by the distributed load placed at the bottom of the excavation pit in an upward direction, is determined by the following formula:

$$[\sigma_z(x)]_{z=H} = -\frac{\gamma_0 H_0}{\pi} \left(\arctan\left(\frac{x+a}{H-H_0}\right) - \arctan\left(\frac{x-a}{H-H_0}\right) \right. \\ \left. + \frac{(x+a)(H-H_0)}{(x+a)^2+(H-H_0)^2} - \frac{(x-a)(H-H_0)}{(x-a)^2+(H-H_0)^2} \right) \quad (3)$$

where: γ_0 - the unit weight of the soil layer within the excavation pit; H_0 - the distance from the bottom of the excavation pit to the ground surface; H- the distance from the existing tunnel axis to the ground surface.

3.2. Tunnel deformation equation

The existing tunnel is considered as an Euler-Bernoulli beam resting on a Winkler foundation model. The Winkler foundation model is regarded as a one-dimensional spring, with individual springs acting independently of each other. The differential equation for the beam on the elastic foundation [11] is expressed as:

$$EI_{eq} \frac{d^4 w(x)}{dx^4} + KDw(x) = D\sigma_{zz}(x) \quad (5)$$

where: D- the outer diameter of the existing tunnel; E- the elastic modulus of the tunnel material; I_{eq} - the equivalent inertial moment of the

existing tunnel accounting for the influence of longitudinal connections in the tunnel; K- the subgrade coefficient of the surrounding soil environment; $w(x)$ - the displacement of the tunnel axis, $\sigma_{zz}(x)$ - the vertical stress acting on the tunnel. The equation (5) is solved through the solution of the homogeneous equation:

$$EI_{eq} \frac{d^4 w(x)}{dx^4} + KDw(x) = 0 \quad (6)$$

Due to the symmetric distribution of the load applied to the tunnel through the excavation pit, the calculation can be simplified by considering only a portion of $x \geq 0$. With the condition as x approaches $+\infty$, $w=0$, the homogeneous solution of Eq. (6) takes the following form:

$$w(x) = e^{-\beta x} (C_1 \cos \beta x + C_2 \sin \beta x) \quad \text{for } x \geq 0 \quad (7)$$

where: $\beta = \sqrt[4]{\frac{KD}{4EI_{eq}}}$

To analyze the impact of excavation above the tunnel below, the tunnel with its concentrated load on its Winkler foundation is considered initially. The total vertical stress, P, is simplified to act on the neutral axis of the tunnel, and it can then be directly applied to the foundation as a concentrated load, as illustrated in Fig. 3.

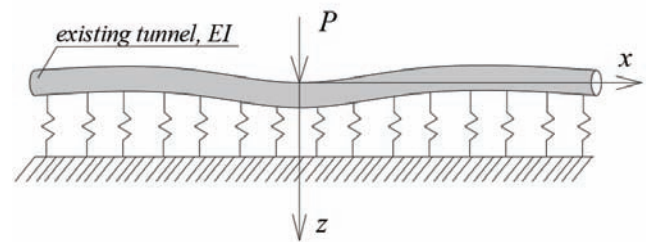


Figure 3. The tunnel on an elastic foundation is subjected to the action of a concentrated load

By applying boundary conditions for the case of a concentrated load: shear force $Q = -PD/2$ and angular deflection $w' = 0$ at $x=0$, we can derive the deflection equation of the tunnel:

$$w(x) = \frac{PD}{8EI\beta^3} e^{-\beta|x|} (\cos \beta|x| + \sin \beta|x|) \quad (8)$$

Once we obtain the deflection of the tunnel under the influence of the concentrated load, we can use this result to deduce the deformation of the existing tunnel under a uniformly distributed load. As depicted in Fig. 4, the load $q(\xi)d\xi$ placed at the position ξ , based on Eq. (8), the derivative of the deflection $dw(x)$ at any position x within the tunnel can be determined through a coordinate shift as follows:

$$dw(x) = \frac{q(\xi)D}{8EI_{eq}\beta^3} e^{-\beta|x-\xi|} (\cos \beta|x-\xi| + \sin \beta|x-\xi|) d\xi \quad (9)$$

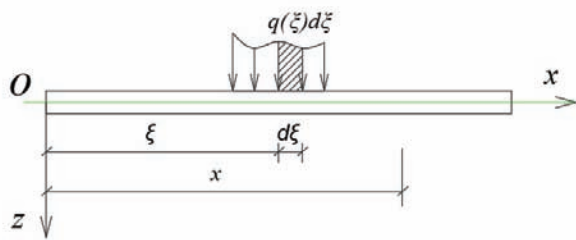


Figure 4. A uniformly distributed load acts on the beam

$$w(x) = \int_{-\infty}^{\infty} \frac{\sigma_{zz}(\xi)D}{8EI_{eq}\beta^3} e^{-\beta|x-\xi|} (\cos \beta|x-\xi| + \sin \beta|x-\xi|) d\xi \quad (12)$$

$$\varphi(x) = \frac{dw(x)}{dx} = - \int_{-\infty}^{\infty} \operatorname{sgn}(x-\xi) \frac{\sigma_{zz}(\xi)D}{4EI_{eq}\beta^2} e^{-\beta|x-\xi|} \sin \beta|x-\xi| d\xi \quad (13)$$

The convolution integrals Eq. (12) and Eq. (13) for determining displacement and angular deflection will be implemented in MATLAB with a sufficiently large integration range. Typically, the region of significant influence when excavating a pit is about 5 times the width of the excavation pit, so the integration limits for the displacement Eq. (12) and angular deflection Eq. (13) of the tunnel axis will be taken from -10a to 10a.

3.5. Determine the parameters

The deformation behavior of the tunnel below is influenced by the interaction between geological layers and the structural capacity of the existing tunnel to resist deformation. Therefore, determining the relevant parameters needs special attention, such as the

Based on Eq. (9), the deformation of the tunnel caused by excavation within the range (x_1, x_2) is determined:

$$w(x) = \int_{x_1}^{x_2} \frac{\sigma_{zz}(\xi)D}{8EI_{eq}\beta^3} e^{-\beta|x-\xi|} (\cos \beta|x-\xi| + \sin \beta|x-\xi|) d\xi \quad (10)$$

According to Eq. (10), the deflection of the existing tunnel due to excavation above is determined as follows:

$$\varphi(x) = \frac{dw(x)}{dx} = - \int_{x_1}^{x_2} \operatorname{sgn}(x-\xi) \frac{\sigma_{zz}(\xi)D}{4EI_{eq}\beta^2} e^{-\beta|x-\xi|} \sin \beta|x-\xi| d\xi \quad (11)$$

$$\text{where: } \operatorname{sgn}(x-\xi) = \begin{cases} 1 & \text{for } x-\xi \geq 0 \\ -1 & \text{for } x-\xi < 0 \end{cases}$$

Considering the problem of full-length excavation pit, the displacement and axial deflection of the tunnel are determined as follows:

flexural stiffness EI , reflecting the deformation resistance of the assembled tunnel, and the subgrade coefficient K , determining the level of foundation settlement.

3.5.1. Equivalent Flexural Stiffness of the Assembled Metro Tunnel Lining

Due to the segmented tunnel connections spaced 1,2 or 1,5 meters apart in the longitudinal direction, the overall flexural stiffness of the shield tunnel is significantly lower compared to a continuous concrete pipe structure. Therefore, some authors have proposed the concept of equivalent flexural stiffness in the longitudinal direction of the shield tunnel. Liao [14] concluded that the approximate equivalent flexural stiffness in

the longitudinal direction of a shield tunnel is about 1/5 to 1/7 compared to the flexural stiffness of a continuous tunnel, which has been verified and is consistent with on-site tests. Therefore, the equivalent flexural stiffness in the longitudinal direction EI_{eq} in this paper can be calculated as follows:

$$EI_{eq} = \frac{E_c}{5 \times 7} \frac{\pi(D^4 - d^4)}{64} \quad (16)$$

where E_c is the Young's modulus of shield tunnel segments; D is the outer diameter of the shield tunnel; and d is the inner diameter of the shield tunnel.

3.5.2. Subgrade Modulus coefficient

Determining the parameter of the subgrade modulus coefficient is complex and affected by the size and distribution of the foundation pressure, soil compressibility, buried depth, field test conditions and other factors. For deep shield tunnels, this paper estimates the subgrade modulus coefficient k from an empirical formula in the reference literature [15].

$$K = \frac{1,3E_s}{B(1-\mu^2)} \sqrt[12]{\frac{E_s B^4}{EI}} \quad (17)$$

where E_s is the elastic modulus of the soil, μ is Poisson's ratio, B is the width of the beam section ($B=D$ in this paper), and EI is equal to EI_{eq} .

Then, the influence of the embedment depth was quantified by Yu et al. [16], where the following coefficient η was proposed:

$$\eta = \begin{cases} 2,18 & H/D \leq 0,5 \\ 1 + \frac{1}{1,7H/D} & H/D > 0,5 \end{cases} \quad (18)$$

where H is the buried depth of the existing tunnel in this paper.

Thus, the calculation formula of K is updated as follows:

$$K = \frac{1,3E_s}{\eta D(1-\mu^2)} \sqrt[12]{\frac{E_s D^4}{EI}} \quad (19)$$

4. RESULTS AND DISCUSSION

In order to provide visual insights from the analytical results obtained, the author's team considered an example from a real project in Ho Chi Minh City and compared it with the results from a 3D finite element model.

4.1 Case study

The Ho Chi Minh City Urban Railway Line 1, with a total length of 19,7 km, consists of 2,6 km underground and 17,1 km elevated tracks. The underground section includes two bored shield tunnels: The Eastbound (EB) and the Westbound (WB) tunnels, constructed using a pressure-balanced Tunnel Boring Machine (TBM).

According to the geological survey results from the Urban Railway Line 1 Ben Thanh - Suoi Tien project, hydrogeological data used in the model were obtained from experiments conducted at borehole U-175 (km 1+553), located approximately 2 meters along the tunnel alignment of the TBM tunnel. Through analysis, the soil layers were categorized into three basic types: Fill, Alluvial Soil, and Flood Deposit Soil. More specifically, the layers at the intersection area consist of five main soil layers: Fill, AC2, AS1, AS2, DC. The distribution of these layers is shown in Fig. 5, with the TBM tunnel situated within layer 3-AS1.

Geological and hydrogeological data were derived from the geological reports of the Metro railway line, as summarized in Table 1. The intersection point of the underground drainage system project and the TBM tunnel Line 1 is located at km 1+555 (according to the urban railway system's alignment). The relevant parameters related to the tunnel lining are presented in Table 2.

Table 1. Properties of soils

Parameter	Layer				
	Fill	AC2	AS1	AS2	DC
E (kPa)	10000	3000	12500	37500	136000
ν	0,3	0,3	0,3	0,3	0,3
γ (kN/m ³)	19	16,5	20,5	20,5	21
K_0	0,577	0,8	0,5	0,455	0,5
γ_{sat} (kN/m ³)	19	16,5	20,5	20,5	21
k (m/s)	1×10^{-6}	1×10^{-9}	2×10^{-5}	2×10^{-5}	1×10^{-8}
c (kPa)	10	0	0	0	170
ϕ (°)	25	24	30	33	35

Table 2. Parameters for tunnel lining

Parameter	Tunnel lining
Tunnel Inner Diameter, D_{int} (m)	6,05
Tunnel lining thickness (m)	0,3
Tunnel center line position, H (m)	11
Equivalent Elastic Modulus, E^* (kPa)	$7,2 \times 10^6$
Poisson's Ratio, ν	0,2
Bulk Density, γ (kN/m ³)	24

The values E^* in the table are equivalent values, indicating the reduction in the flexural stiffness of the shield tunnel in the model.

The excavation pit has corresponding dimensions: width $2a = 4,68\text{m}$; depth of the pit: $H_0 = 1,6\text{m}$; the pit is located in the Fill surface soil layer, $\gamma_0 = 19 \text{ kN/m}^3$.

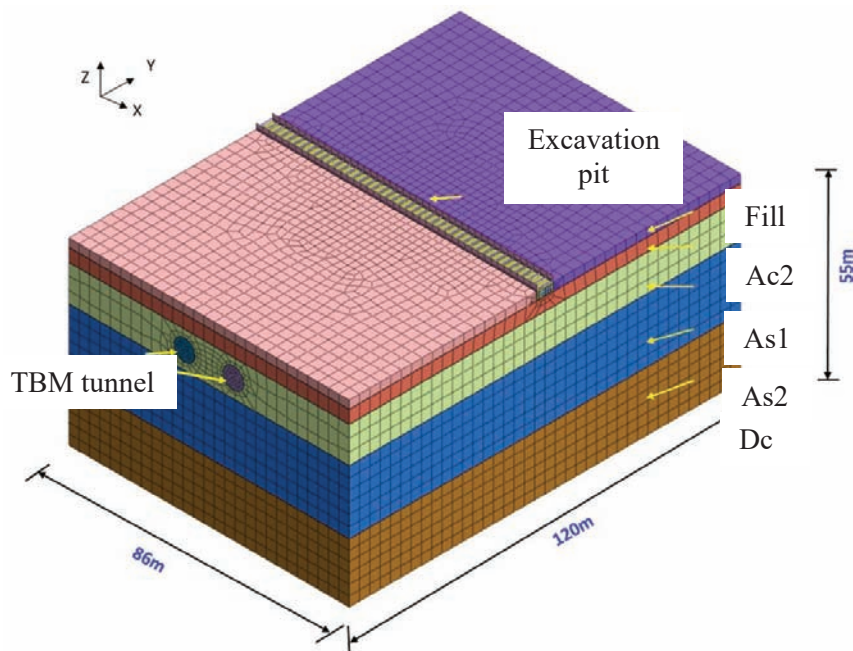


Figure 5. Dimensions of the 3D Model

The value of uniformly distributed load due to the process of unloading the soil mass in the pit is: $p_0 = \gamma_0 * H_0 = 30,4 \text{ (kPa)}$.

4.2 Finite element model

Using the finite element method based on the MIDAS GTS software platform, with soil layers modeled using the Mohr-Coulomb model with

Hexadretrol elements, the tunnel lining is simulated as a thick plate. Interface elements are used to model the interaction between the soil and the tunnel lining, with a reduction factor $R_{inter} = 0,67$. The model dimensions are shown in Figure 5 [7]. The displacement of the tunnel axis is determined by the average of the displacements at the top and bottom of the tunnel.

4.3 Results of analysis and discussion

After performing calculations using the MATLAB software with the input data considered in section 4.1, the vertical stress distribution values are shown in Fig. 6. The load applied at the corresponding floor is $p = -30,4$ kPa, and the maximum stress value at the tunnel axis position is $(\sigma_{zz})_{\max} = -9,68$ kPa.

The displacement of the tunnel axis is shown in Fig. 7, with the maximum displacement value of the tunnel axis by analytical method being 4,6 mm, while according to the finite element method, it is 4,4 mm. The displacement value of the tunnel axis from model is determined by the average value of the displacement at the tunnel crown and the displacement at the tunnel invert.

The vertical deflection angle of the tunnel axis is shown in Fig. 8, with the maximum deflection angle by the analytical method being 0,165 ‰, and by the finite element method, it is 0,178‰. The corresponding bending point for the analytical method is located about 16 meters

away from the excavation center, while according to the finite element method, it is approximately 10 meters away.

4.4 Discussion

The displacement value of the tunnel axis is greater by approximately 4,3% according to the analytical method compared to the finite element method (FEM), and the deformation zone according to the analytical method is larger. The area of influence affecting the displacement of the tunnel axis extends about 40 meters (approximately 10 times the width of the excavation) on either side. Beyond this 40-meter range, the displacement value becomes negligible.

The deflection angle along the tunnel axis is nearly the same for both methods, differing by only 1%. However, the location of the bending point differs by approximately 40% between the two methods, with the analytical method indicating a point farther from the excavation site than the FEM.

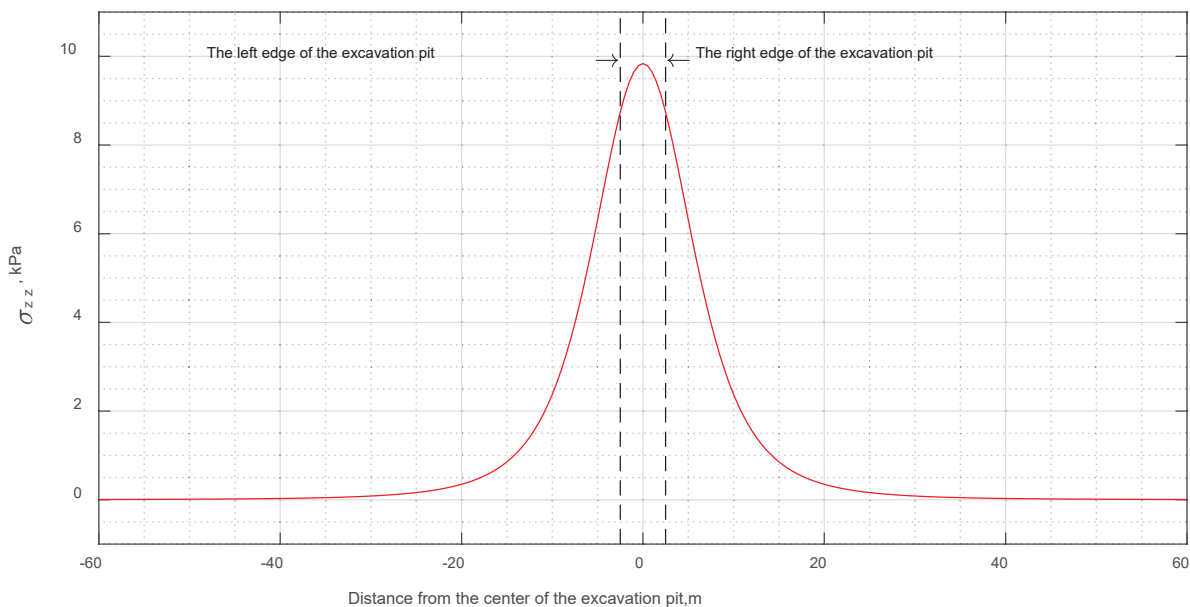


Figure 6. Vertical Stress Distribution

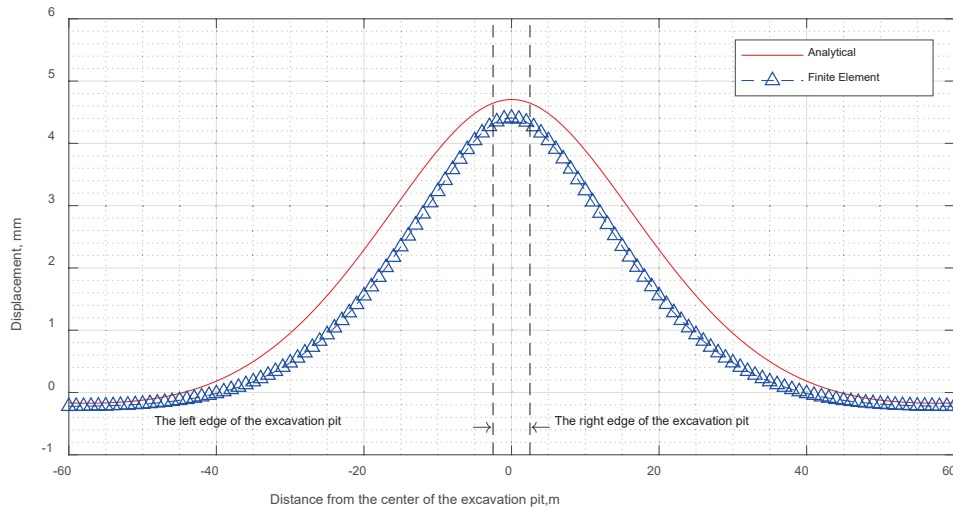


Figure 7. Displacement of the Tunnel Axis

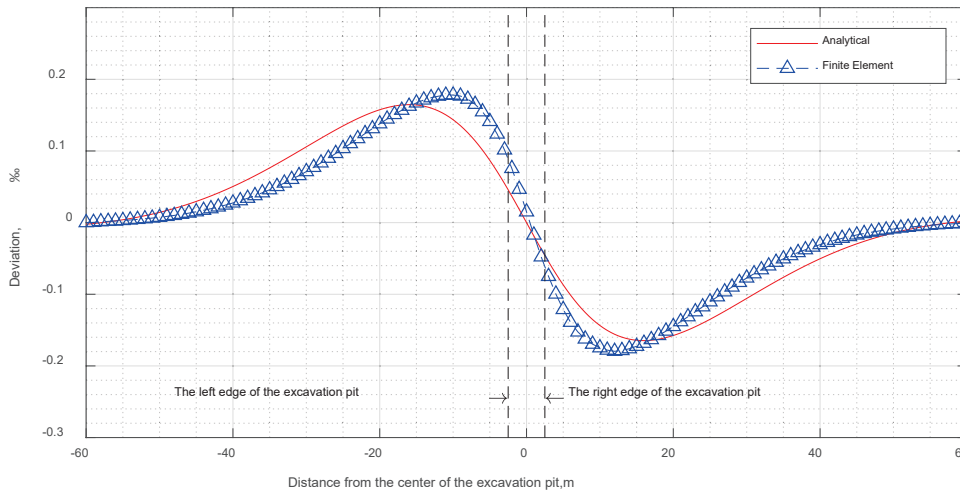


Figure 8. Vertical Deflection Angle of the Tunnel Axis

From the analysis of these results, it is observed that the analytical method provides a higher safety margin in terms of displacement. This can be explained by the following reasons: (1) The soil model does not consider the sliding interaction between layers (two-coefficient soil model), (2) The influence of different soil layers is not accounted for in the model, affecting the foundation coefficient determination.

5. CONCLUSIONS

The analytical approach to determine the displacement of the tunnel axis during the construction of full-length underground culverts

is a practical problem with initial evaluation significance and serves monitoring purposes during the culvert construction process.

The analytical approach simplifies the process of determining the displacement and deflection of the tunnel axis, making it straightforward and efficient, without the need for complex tools such as the finite element method. However, the results obtained through the analytical method tend to be more conservative compared to the finite element method because they do not consider all influencing factors, such as the foundation stiffness and the two-coefficient soil model.

Therefore, further research is needed to address the problem while taking into account factors

that influence the soil stiffness, such as the two-coefficient soil model and various soil layers.

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