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ASSESSMENT OF THE INFLUENCE OF ADJACENT EXCAVATION STEP LENGTH ON SHIELD TUNNEL DEFLECTION

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Abstract: Protecting existing underground structures in urban areas from adverse impacts caused by surface construction activities is a significant challenge, especially when applying the open-cut sequential excavation method. This paper proposes a simple analytical method to determine the deflection of the tunnel axis induced by the sequential excavation of the overlying excavation pit segments. The method is based on identifying the changes in soil stress at the tunnel cross-section during each construction stage, followed by using analytical techniques to determine the corresponding tunnel deflection. The tunnel-soil interaction is modeled by an Euler-Bernoulli beam on an elastic foundation represented by the Winkler model. Furthermore, the paper investigates the influence of segmental excavation length on the tunnel deflection. The results show that with excavation step lengths of 3.6 m, 7.2 m, and 14.4 m, the maximum tunnel deflection increases by 135% and 193%, respectively. These results provide a basis for optimizing construction methods, minimizing risks, and effectively protecting underground structures.

Keywords: Existing shield tunnel, shield tunnel deflection, excavation step length, line 1 HCM metro, Winkler model, Euler-Bernoulli beam, Analytical approach, MIDAS GTS

ОЦЕНКА ВЛИЯНИЯ ДЛИНЫ ШАГА СМЕЖНОЙ ВЫЕМКИ НА ПРОГИБ ЩИТОВОГО ТОННЕЛЯ

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Аннотация: Защита существующих подземных сооружений в городских районах от неблагоприятных воздействий, вызванных поверхностными строительными работами, является серьезной проблемой, особенно при применении метода последовательной выемки грунта открытым способом. В данной статье предлагается простой аналитический метод определения прогиба оси тоннеля, вызванного последовательной выемкой вышележащих сегментов котлована. Метод основан на выявлении изменений напряжения грунта в поперечном сечении тоннеля на каждом этапе строительства с последующим использованием аналитических методов для определения соответствующего прогиба тоннеля. Взаимодействие тоннеля с грунтом моделируется балкой Эйлера-Бернулли на упругом основании, представленном моделью Винклера. Кроме того, в статье исследуется влияние длины шагов выемки на прогиб тоннеля. Результаты показывают, что при длине шагов выемки 3,6 м, 7,2 м и 14,4 м максимальный прогиб тоннеля увеличивается на 135% и 193% соответственно. Эти результаты создают основу для оптимизации методов строительства, минимизации рисков и эффективной защиты подземных сооружений.

Ключевые слова: Существующий тоннель, Прогиб щитового тоннеля, длина шага выемки, Линия 1 метрополитена Хошимина, Модель Винклера, : Балка Эйлера-Бернулли, Аналитический подход, MIDAS GTS

1. INTRODUCTION

In recent years, rapid urban development in major Vietnamese cities has coincided with the construction of underground metro systems. This has resulted in cases where foundation excavations of adjacent structures (such as high-rise buildings and utility infrastructure) are carried out near shield tunnels (TBM tunnels). Such adjacent excavation activities can impose significant adverse impacts on shield tunnels by altering their initial stress state, thereby posing serious risks to operational safety. Documented damages include cracking of reinforced concrete tunnel segments, displacement and detachment of the concrete invert slab from the tunnel lining [1]. Excessive deformations of this nature may reduce load-bearing capacity, induce water leakage, and even threaten the overall stability of the tunnel system, potentially leading to cracking, leakage, reduced train speeds, or in extreme cases, collapse.

To predict the behavior of shield tunnels under the influence of adjacent foundation excavation, numerous studies have been conducted, which can generally be classified into three main approaches: finite element methods, physical modeling, and analytical methods.

Three-dimensional centrifuge model tests have been employed to investigate the effects of deep excavations or twin tunnels on existing tunnels and pile groups [2-3]. This technique provides valuable insights into complex soil–structure interaction mechanisms under simulated gravity conditions. Physical modeling offers important experimental data; however, simplifications in stratigraphic conditions and construction sequences may introduce uncertainties when extrapolating results to real projects, while also being costly and time-consuming.

The finite element method is a widely used approach that enables the modeling of complex soil-structure interactions, including the elasto-plastic behavior of soil and construction sequencing [2, 4-10]. Software such as FLAC3D, PLAXIS3D, and Midas GTS NX are commonly employed to analyze the impacts of excavations on existing tunnels. The finite element method provides in-depth analysis of deformations and internal forces, with the capability to simulate intricate processes in detail. However, this method requires significant computational resources, specialized software, and a considerable number of assumed parameters, making the modeling process complex.

The analytical method provides theoretical formulas that allow rapid estimation of tunnel deformations. In this approach, tunnels in soil are commonly idealized as beam-on-elastic-foundation problems, employing the Euler–Bernoulli or Timoshenko beam models in conjunction with Winkler, Pasternak, or Vlazov foundation models [9], [11-17]. Due to its simplicity, cost-effectiveness, and the ability to produce quick forecasts, the analytical method is considered a valuable tool in the early stages of tunnel design and construction.

However, current analytical studies still have limitations, as they have not fully considered the load changes over a large range such as the entire length of the excavation, nor have they evaluated the effects of different excavation steps on tunnel deformation. Meanwhile, optimizing construction parameters such as depth and length of excavation steps plays a key role in ensuring the safety and efficiency of underground works.

In this paper, the authors propose an analytical framework in which the shield tunnel is idealized as an Euler–Bernoulli beam, while the surrounding ground is simulated using a Winkler elastic foundation model. For validation, the analytical outcomes are benchmarked against a three-dimensional finite element model, calibrated with monitoring and design data from Metro Line No. 1 of the Ho Chi Minh City Urban Railway.

2. ANALYTICAL APPROACH

To approach the problem analytically, it is necessary to accept the following assumptions. The surrounding ground of the tunnel is considered as an elastic foundation characterized by a Winkler model coefficient. The TBM tunnel excavated in the soil environment is considered as a beam on elastic foundation problem. The consideration of internal forces in the cross-section is ignored. And the beam is considered to have infinite length. In addition, the deflection of the tunnel is within the elastic limit of the structure.

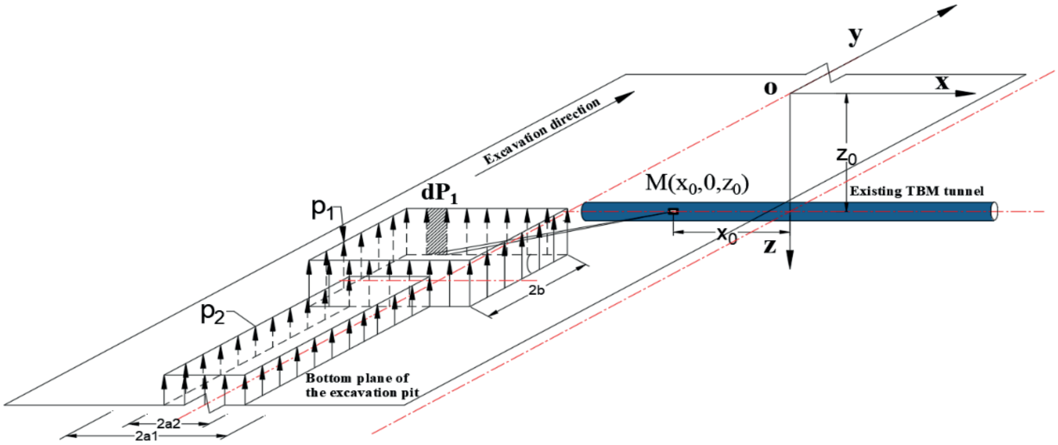

Figure 1. Force Distribution in a homogeneous elastic half-space

In this problem, the influence of the tunnel on the stress generation, as well as the longitudinal and shear stresses in the tunnel, is ignored.

2.1 Load acting on the tunnel lining

The process of excavating the upper excavation segment, changes the load above the tunnel from the complete unloading (state 1) of an excavation segment of size $2a_1 * 2b$, with the value of $p_1(x)$, to the completed state (state 2) which will be the value of $p_2(x) = p_1(x) - \Delta p(x)$, $\Delta p(x)$- the load difference between the unloaded soil weight and the completed stage load. The change in load on the tunnel structure during excavation is shown in Figure 1. The value of the bottom plane load of an excavation element is shown by the following formula (1):

$$p(x) = \begin{cases} p_1, & |x| \le a_1, |y - y_0| \le b \\ p_2, & |x| \le a_2, y - y_0 < -b \end{cases} \quad (1)$$

in which, y_0- coordinates of the center of the excavation pit along the y-axis; $2a_1$- width of the excavation pit; $2a_2$- width of the permanent structure placed in the excavation pit; $2b$- length of the excavation step.

The problem of concentrated loads in half-space was solved by Sneddon using the Fourier transform in 1951 [18], the formula for the vertical stress change at depth z under the action of a concentrated load P, is determined by the following formula:

$$\sigma_{zz} = \frac{3P}{2\pi}\frac{z^3}{R^5}, R = \sqrt{x^2 + y^2 + z^2} \quad (2)$$

The generalization for vertical stress acting on the tunnel axis due to uniformly distributed load with varying intensity is determined as follows:

$$\sigma_{zz} = \int_{-a_1}^{a_1}\int_{y_0-b}^{y_0+b} \frac{3p_1}{2\pi}\frac{z^3}{R^5}dxdy + \int_{-a_2}^{a_2}\int_{-\infty}^{y_0-b} \frac{3p_2}{2\pi}\frac{z^3}{R^5}dxdy \quad (3)$$

With the initial assumptions when approaching the analytical method, the vertical stress at depth H, due to the distributed load placed at the bottom of the excavation hole in the upward direction as shown in Figure 1, is determined by the following formula:

$$\sigma_{zz} = \int_{-a_1}^{a_1}\int_{y_0-b}^{y_0+b} \frac{3}{2}\frac{\gamma_0 H_0}{\pi}\frac{(H-H_0)^3}{\left[(x-x_0)^2 + y^2 + (H-H_0)^2\right]^{5/2}}dxdy + \int_{-a_2}^{a_2}\int_{-\infty}^{y_0-b} \frac{3p_2}{2\pi}\frac{(H-H_0)^3}{\left[(x-x_0)^2 + y^2 + (H-H_0)^2\right]^{5/2}}dxdy \quad (4)$$

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.

2.2 Tunnel deflection 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 [14] is expressed as:

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

where: D - the outer diameter of the existing tunnel; EI_{eq} - the equivalent bending stiffness 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 total solution of equation (5) is determined based on the solution of the homogeneous equation and the boundary conditions. With the initial assumption, we consider this as an infinite beam problem on an elastic foundation, from which the deflection of the tunnel is determined by the following equation:

$$w(x)=\frac{PD}{8EI\beta^3}e^{-\beta|x|}\left(\cos\beta|x|+\sin\beta|x|\right), where: \beta=\sqrt[4]{\frac{KD}{4EI_{eq}}} \tag{6}$$

To analyze the effects of excavation above the tunnel below, first consider the tunnel with a concentrated load placed on the Winkler foundation. The total vertical stress P is simplified to act at the neutral axis of the tunnel and can then be transmitted directly to the foundation as a concentrated load, from which the deflection of the tunnel under the force can be determined. Once the deflection of the tunnel under the concentrated load has been determined, this result can be used to deduce the deflection of the existing tunnel under the uniformly distributed load. Based on the transformations, the deflection of the tunnel due to the change of load in the range $(-\infty;+\infty)$ can be determined by formula:

$$w(x)=\int_{-\infty}^{+\infty}\frac{\sigma_{zz}(\xi)D}{8EI_{eq}\beta^3}e^{-\beta|x-\xi|}\left(\cos\beta|x-\xi|+\sin\beta|x-\xi|\right)d\xi \tag{7}$$

The convolution integral in equation (7) used to determine the displacement will be performed in MATLAB with a sufficiently large integration interval.

2.3 Define the parameters

The tunnel axis deflection below depends on the interaction between geology and the bearing capacity of the existing structure. Therefore, special attention should be paid to parameters such as equivalent bending stiffness EI_{eq} and the subgrade modulus coefficient K, as they determine the resistance to deflection and settlement of the foundation.

2.4 Equivalent bending stiffness of TBM tunnel

Due to the segmental joints, the longitudinal bending stiffness of the TBM tunnel is significantly smaller than that of the continuous concrete tunnel. According to Liao [15], the equivalent longitudinal bending stiffness of the shield tunnel is only about 1/5 to 1/7 of that of the continuous tunnel, which has been experimentally verified.

$$EI_{eq}=\frac{E_c}{5\div 7}\frac{\pi\left(D^4-d^4\right)}{64} \tag{8}$$

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

2.5 Subgrade modulus coefficient

The foundation modulus factor in Winkler models for analyzing pipe–soil interactions was originally proposed by Attewell et al. [19] based on the expression of Vesic [20], usually with a double value for buried pipes. To improve this method, Yu et al. [13] developed a modified

Winkler foundation modulus factor expression, applicable to arbitrarily buried pipes subjected to free ground displacements of any curve shape, while considering the buried depth ratio (H/D) of the pipe. The modified subgrade modulus coefficient according to Yu et al., which is employed in this study, is expressed as follows:

$$K = \frac{1,3E_s}{\eta D\left(1-\mu^2\right)}\sqrt[12]{\frac{E_s D^4}{EI}},\qquad (9)$$
$$where: \eta = \begin{cases} 2,18 & H/D \le 0,5 \\ 1+\dfrac{1}{1,7H/D} & H/D > 0,5 \end{cases}$$

where: E_s - the elastic modulus of the soil, μ - Poisson's, D - the outer diameter of the existing tunnel, H - the distance from the existing tunnel axis to the ground surface, and EI is equal to EI_{eq}

3. EXAMPLE VALIDATION

To provide a visual insight from the obtained analysis results, the authors considered an example from a real project in Ho Chi Minh City and compared it with the results from the 3D finite element model.

3.1 Case study

Ho Chi Minh City Urban Railway Line 1, with a total length of 19.7 km, includes 2.6 km underground and 17.1 km elevated. The underground section includes two shield tunnels: the East Tunnel (EB) and the West Tunnel (WB), constructed by a pressure balance tunnel boring machine (TBM). According to the geological survey results of the Ben Thanh - Suoi Tien Urban Railway Line 1 project, the hydrogeological data used in the model were taken from experiments at borehole U-175 (km 1+553), located about 2 meters from the center of the TBM tunnel [4]. Through analysis, the soil layers were classified into three main groups: Fill soil, alluvial soil and alluvial soil. More specifically, the soil layers at the intersection area include five main layers: Fill soil, AC2, AS1, AS2 and DC.

The geological and hydrogeological data were obtained from the geological reports of the urban railway, summarized in table 1. The intersection point between the underground drainage system project and the TBM tunnel of Line 1 is located at km 1+555 (according to the centerline of the urban railway). The parameters related to the tunnel lining structure 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)	1x10^{-6}	1x10^{-9}	2x10^{-5}	2x10^{-5}	1x10^{-8}
c (kPa)	10	0	0	0	170
φ (^{o})	25	24	30	33	35

Table 2. Parameters for tunnelling

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)	7,2x10^6
Poisson's Ratio, ν	0,2
Bulk Density, γ (kN/m^3)	24

The E^* values in the table are equivalent values, indicating the reduction in the bending stiffness of the shield tunnel in the model. The excavation pit has the following dimensions: width $2a_1 = 5m$; excavation pit depth: $H_0 = 1.6m$;the excavation pit is located in the topsoil layer, $\gamma_0 = 19kN / m^3$. The value of the uniformly distributed load due to the process of unloading the soil mass in the excavation pit is: $p_1 = \gamma_0 * H_0 = 30.4(kPa)$. The load value after the soil is placed $p_2 = 0.5 * p_1$; the width of the culvert considered is $2a_2 = 3.5m$.

3.2 Analysis results and discussion

The finite element model and tunnel deflection for comparison, are taken from the paper Nguyen, et al. [4]. The finite element software used is MIDAS GTS, with the properties of the soil layers determined by the Mohr-Coulomb model, the tunnel lining is a plate element, the tunnel material is considered as isotropic elastic. Interface elements are used to model the interaction between the soil and the tunnel lining, with the strength reduction factor Rinter=0.67. The tunnel axis deflection is determined by the average value of the displacement at the crown and bottom of the tunnel.

After performing calculations using MATLAB software with the input data considered in section A, the tunnel axis deflection determined by the two methods is shown in Figure 2, with the maximum displacement value of the tunnel according to the analytical method being 4.7 mm, while according to the finite element method it is 4.41 mm.

The displacement value of the tunnel is about 6.2% larger according to the analytical method than that of the finite element method (FEM), and the deflection area according to the analytical method is also larger. The area affected by the displacement of the tunnel extends about 40 meters (about 10 times the width of the excavation) on both sides. Beyond this 40m range, the displacement value becomes negligible.

From the analysis results, this analysis method provides a higher margin of safety 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-factor soil model), (2) The influence of different soil layers is not considered in the model, affecting the determination of the foundation coefficient.

4. PARAMETRIC SURVEY

To examine the influence of the excavation step length on the displacement of the tunnel, the authors conducted a survey with different excavation lengths, respectively 2b=3.6m; 7.2m & 14.4m. The displacement of the tunnel when moving with different segments is shown in Figure 3. When the excavation step changes from 3.6m to 7.2m, the maximum deflection of the shield tunnel changes from 1.92 mm to 2.59 mm; the maximum displacement value increases to 3.71mm when the excavation step increases to 14.4m. The choice of excavation length greatly affects the maximum displacement value of the tunnel. When the excavation step is doubled, the maximum deflection of the tunnel axis increases by 135% & 193% respectively compared to the maximum displacement value when the initial excavation step length is selected. The graph also shows that when the excavation/filling takes place within a range of 15m or more from the tunnel centerline, the deflection of the tunnel axis does not change significantly. This range can be the basis for determining the scope of the model size survey when modeling with finite elements. The residual deflection of the shield tunnel centerline when the excavation and filling process is completed is recorded according to the model as ~1.6mm.

The change in the maximum deflection of the tunnel when changing the width of the excavation pit is shown in Figure 4. The change of the maximum deflection of the tunnel when the width of the excavation pit changes is shown in Figure 4. The maximum deflection value changes rapidly when the width of the excavation pit changes from 4-15m. Then the maximum deflection value increases gradually. When the width of the excavation pit is from 25m or more, the maximum displacement of the tunnel has a stable value, even though the width of the excavation pit still increases. The maximum deflection of the tunnel axis currently is 4.7mm.

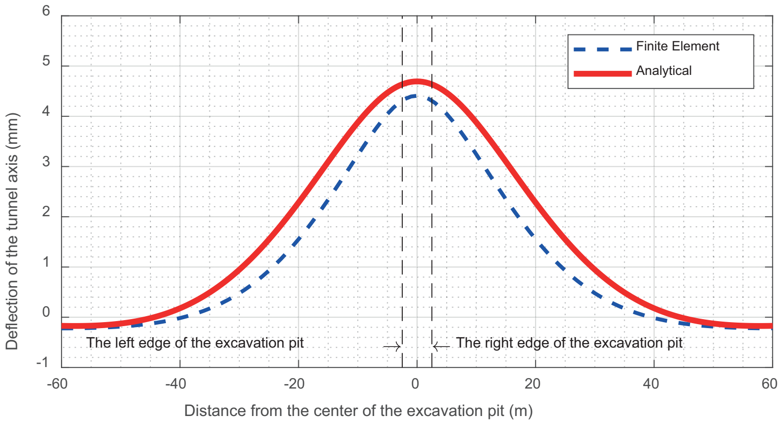

Figure 2. *Deflection of the tunnel axis due to full-length excavation*

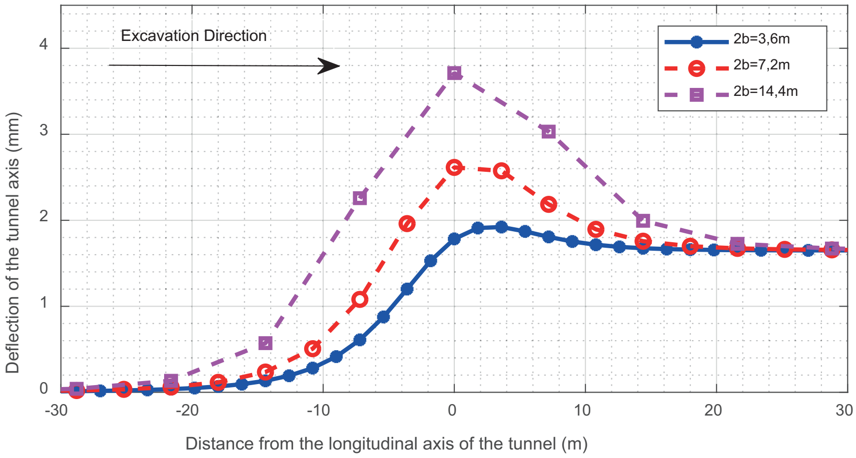

Figure 3. *Maximum deflection of the tunnel axis with changing excavation position along the excavation direction and varying excavation step lengths*

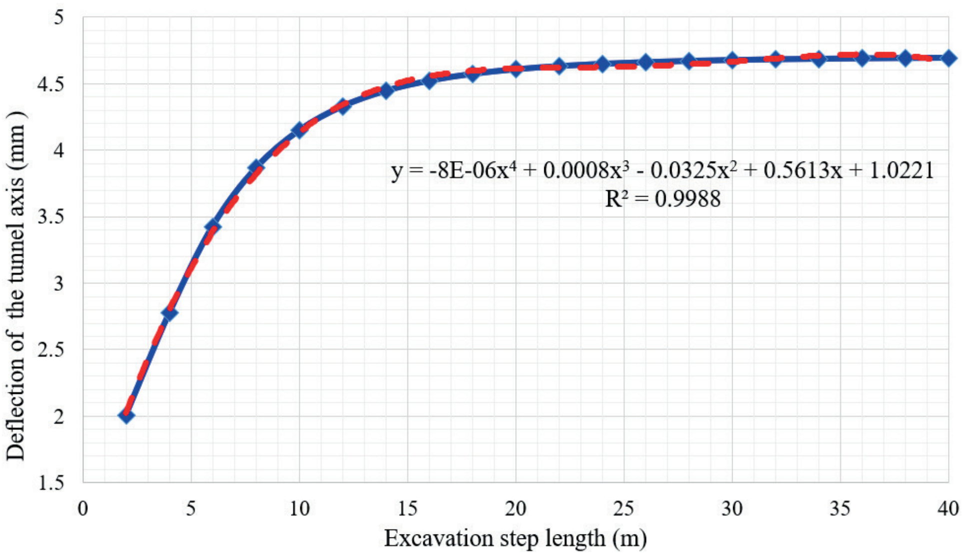

Figure 4. *Maximum deflection of the tunnel axis with varying excavation step length*

From the survey results, the relationship between the width of the excavation and the maximum deflection value of the tunnel centerline is established, the results are shown through the fourth-order function in Figure 4.
With the variance percentage R^2=0.9988, showing a good fit of the regression function, it can be used to predict values in the middle of the surveyed points.

5. CONCLUSIONS

The analytical approach for determining tunnel axis displacement during staged excavation of a foundation pit provides an important preliminary assessment tool, supporting the selection of suitable excavation step lengths. The results indicate that when excavation step lengths of 3.6 m, 7.2 m, and 14.4 m, the maximum tunnel deflection increases by approximately 135% and 193%, respectively, highlighting the significant influence of excavation step size. Based on these findings, an appropriate excavation step length can be selected to optimize construction performance.
This analytical method offers a simplified and efficient means of estimating tunnel displacement without the need for complex numerical tools such as the finite element method (FEM). However, the results obtained tend to be more conservative compared to FEM, as the analytical method neglects several influencing factors, including foundation stiffness and advanced soil constitutive models. Further research is therefore required, particularly with the incorporation of soil–structure interaction models such as the Pasternak two-parameter foundation model, as well as advanced structural models like the Timoshenko beam theory.
Upon completion of the excavation process, the maximum residual deflection observed at the tunnel crown is approximately 1.6 mm, regardless of excavation step length. Thus, when assessing safety with respect to deflection limits, both cumulative construction-induced deflections and long-term residual deflections during operation must be taken into consideration.

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