

Article

Assessment of the Shallow Cukurcayir 1 Tunnel (NE Türkiye) Constructed in a Residential Area and in Weak Ground Using 3D Numerical Analysis

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Abstract

Tunnels excavated in weak ground can develop problems due to squeezing phenomena. This situation is much more critical in large-diameter and double-tube tunnels. The effects of the tubes on each other are also important. The NATM (New Austrian Tunneling Method) is employed in the design of the support systems for these types of tunnels, which is based on the flexible outer arch principle and allows for deformations or minimization of deformations by increasing the lining thickness and using a rigid pavement. The risk of deformations occurring in a flexible outer arch is determined by the occurrence of deformations in shallow tunnels, which can affect the surface; a ground reaction curve can quickly develop and lead to a collapse. It is also known that the stability of the tunnel whose support and excavation is completed first can be impacted by subsequent excavations for the other tunnel. The purpose of the present study was to describe the geological and geotechnical conditions of a shallow tunnel excavated in weak ground and to assess its stability using three-dimensional numerical analysis. For this purpose, the Cukurcayir 1 tunnel—a double-tube tunnel—was employed as a case study. The 3D numerical analyses were performed using the Flac3d V2.10-222 software. The results are discussed in detail and recommendations for the necessary support system for tunnel roof and face stability are given.

Keywords: twin tunnels; 3D analysis; ground reaction curve; support systems; umbrella; tunnel face stability



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1. Introduction

The problems encountered in tunnels excavated in weak ground generally occur in the tunnel roof and face. For this reason, the stability of the face and roof is very important for tunnel stability. This problem has been examined by various researchers through different case studies [1–6]. In large-diameter double-tube tunnels, the impact of the distance between two tubes causes stability problems in their support systems. In general, the distance between two tunnels is recommended to be at least 1.5 times the tunnel diameter [7]. However, this distance is often not possible due to topographic and surface constraints.

There are two main approaches in the design of support systems in weak ground. The first is an active support system approach that does not allow deformations. The other is a

passive approach that allows deformations to occur in the tunnels in a controlled manner to maintain tunnel stability [8]. For this purpose, Shubert [9] suggested deformation gaps on the steel supports in tunnel support systems. This method was used in the Bolu mountain tunnels in fault zones [10]. In addition, Hoek [11] proposed sliding steel supports (TH) in sections when deformations occur and applied this approach in the Jacombu tunnel. These deformations are within the tolerance limits and do not damage the tunnel lining. However, the main problem here is that deformation curves rapidly develop in squeezing ground, which can also cause excessive deformation.

Sharifzadeh et al. [12] used the sequence excavation method in the Shibli tunnel that was constructed in weak ground and performed a back analysis. In this study, they highlighted the importance of both deformation measurements in tunnels and monitoring systems. Klopčič et al. [13] studied displacement during the tunnel excavation progress in the Sentvid tunnel. They focused on the deformations of the rock masses in the tunnel under low overburden heights in a populated area. They found that the support system's capacity is related to the foliation orientation in the rock mass. Ozsan and Basarir [14] investigated the support capacity of diversion tunnels excavated in weak rock masses using analytical and numerical methods. Do et al. [15] examined the behavior of a tunnel excavation method using 2D and 3D numerical analyses. Han et al. [16] carried out 3D numerical analyses of double-tube tunnels excavated in weak and blocky rocks using the umbrella arch support method. They found that the deformations remained stable and the safety factor increased by 2.25 times. Marinos [17] studied the tunnel behavior and the support relationship in tunnels excavated in a flysch series with weak rock mass properties. He emphasized that flysch series have heterogeneous and tectonically disturbed structures, leading to a low strength. He also emphasized that the support demands range from light support to very rigid support under high squeezing conditions. Vlachopoulos et al. [18] investigated the effect the tubes in double-tube tunnels on each other using two- and three-dimensional analyses.

It is known that, due to the rapid development of deformation and support reaction curves in weak ground, the loads acting on the support systems can increase suddenly and result in failure. If the deformation tolerance decreases quickly due to the incoming load and the lack of arching around the tunnel, it can cause support failure. In addition, the passive approach is not recommended in shallow tunnels excavated in weak ground as it may directly damage the buildings on the surface. It is imperative to build support systems using the active approach for deep tunnel sections in ground that is weak and squeezed. In these sections, deformations can develop very rapidly due to the high velocity of displacements due to high in situ stresses.

It has been revealed that utilizing a flexible outer arch support design, according to the principles of the NATM (New Austrian Tunneling Method), in tunnels excavated in weak ground is not suitable and an alternative is needed [10,19,20]. Many researchers believe that a rigid lining is required in weak ground and that the support systems in such ground should not allow deformations [21–25]. The supports should be placed immediately after the tunnel excavation and loosening should not be allowed around the tunnel. The deformations occurring along the tunnel profile, the plastic zones around the tunnel [26–28], and the ground reaction curve (GRC) and support reaction curve (SRC) for the tunnel support systems are important considerations in the placement and timing of the placement of the support systems.

To determine the most economical support systems, it is critical to determine the deformation tunnel interactions and the deformation tolerance in order to determine the optimal dimensions. If the deformations develop very quickly, they can cause the support systems to fail. If the support systems are designed to not allow any deformation, the

support systems would be rigid. In weak ground, the rapid development of deformations makes the selection of the support system even more important.

Dong et al. [29] proposed a computational method to determine the surface settlements in parallel tunnels in layered rock medium. Huang et al. [30] investigated the surface settlements in twin-tube TBM tunnels due to different construction methods. Kumar and Jain [31] investigated the elastoplastic soil settlement response and stability of single and twin circular tunnels and the effects of tunnel depth, diameter, spacing and soil properties, and suggested stability conditions and support placement timing predictions that can be used in engineering designs. Zhang et al. [32] used 3D numerical simulation and field measurements to investigate the ground deformation properties and stresses acting to segments during the excavation of a shallow twin tunnel of Guangzhou Metro Line 18. Jamshasb and Shahrabaki [33] investigated the effect of tunnel convergence in shallow twin tunnels and presented simplified relations to help tunnel designers find the optimum tunnel center-to-center distance. Guo et al. [34] establishes a theoretical decision model based on the Mohr–Coulomb failure criterion of the middle load-bearing column to determine the safe clear distance for shallow twin TBM tunnels and validates this model with numerical simulations. The three-dimensional numerical model of the Guanyin Mountain tunnel portal section was constructed by the Rhino, AutoCAD, and FLAC 3D software, and the whole construction process of the tunnel portal was simulated by Lit et al. [35]. Mirsepahi et al. [36] employed 3D finite element analyses for investigation of the performance of parallel hypothetical twin tunnels constructed in weathered residual soil using the New Austrian Tunneling Method (NATM). Another 3D analysis simulating the surface settlement due to twin-tunnel excavations was performed by Yu Huat et al. [37]. In addition to these studies, in the recent literature, many 3D numerical analyses have been performed on the twin-tunnel constructions [38–41].

The construction of the Cukurcayir 1 tunnel requires extreme care because it is located in an urbanized area, in weak ground, and has a low overburden thickness. In addition, it is very important to determine the best support systems since it is a twin tunnel and the tunnel excavation cross-section area is large (up to 200 m²). The ground reaction curve (GRC) and support reaction curve (SRC) were drawn for the Cukurcayir 1 tunnel and its tunnel support systems were assessed by performing 3D analyses using the Flac3d software. The numerical analysis results were compared with the measured deformation results in the tunnel. This tunnel is an interesting case study for underground construction work in residential areas. In addition, Wang et al. [42] stated that the temporal and spatial effects of a complicated excavation process are vital for an ultra shallow buried large-span double-arch tunnel excavated under an expressway in service, and numerical simulations are urgent and necessary to understand the effect of the total construction process. When considering this statement [42], the importance of the present study is clarified. Consequently, it is expected that shallow tunnel constructions in urban areas will increase due to population growth. The results of the present study will contribute to the safe construction of future shallow tunnels in urban areas.

2. Properties of the Cukurcayir 1 Tunnel

The Cukurcayir 1 tunnel is within the scope of the Trabzon City (NE Türkiye) Pass Project, wherein the right tube is between Km:11 + 660.45 and 11 + 980, and the left tube is between Km:11 + 661.25 and 12 + 040. The tunnel length is 379.25 m for the left tube and 319.25 m for the right tube. The tunnel overburden thickness is 40 m at the deepest section. A map showing the location of the tunnel is given in Figure 1. The Cukurcayir 1 tunnel was designed as a double tunnel with three lanes. The final height and width of the tunnel are

8.4 and 12.5 m, respectively. The tunnel excavation width is 17 m, and the height is 13.5 m. A typical cross-section of the tunnel is given in Figure 2 [43].

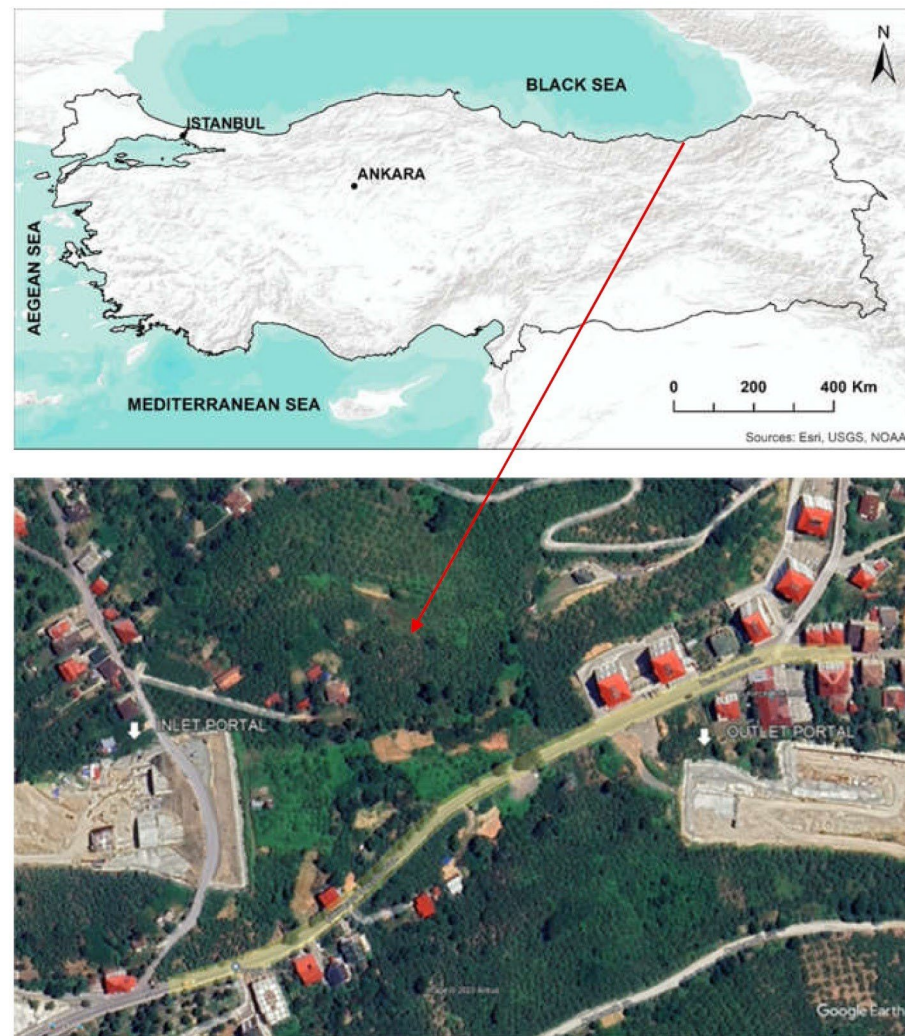


Figure 1. Location of the Cukurçayır 1 tunnel.

In order to determine the geological and geotechnical conditions of the Cukurçayır 1 tunnel, a total of 6 boreholes with a depth of 35 to 50 m were drilled by the General Directorate of Highways of Türkiye. The necessary soil mechanics tests were performed on the specimens obtained from the boreholes and the material strength parameters were obtained. In addition, pressure meter tests were carried out in the boreholes and the elasticity modulus of the soil was calculated. During the field studies, yellowish brown-reddish brown clay, sandy silty clay, and clay/gravel units were encountered on the surface of the tunnel entrance portal and its vicinity. After the interpretation of the geological map and borehole data, a geological model of the Cukurçayır 1 tunnel was produced, which is shown in Figure 3 [44]. Clayey sand and weakly cemented sandstone, claystone, and tuff under the clay unit were observed on the surface. Figure 4 shows a geological cross-section of the tunnel.

Thus, it was found that the entire tunnel passes through clay and gravelly clayey sand units. A sample of the cores taken from the SK-26 borehole in the tunnel is presented in Figure 5. The samples revealed that the tunnel passes through clayey sandy units.

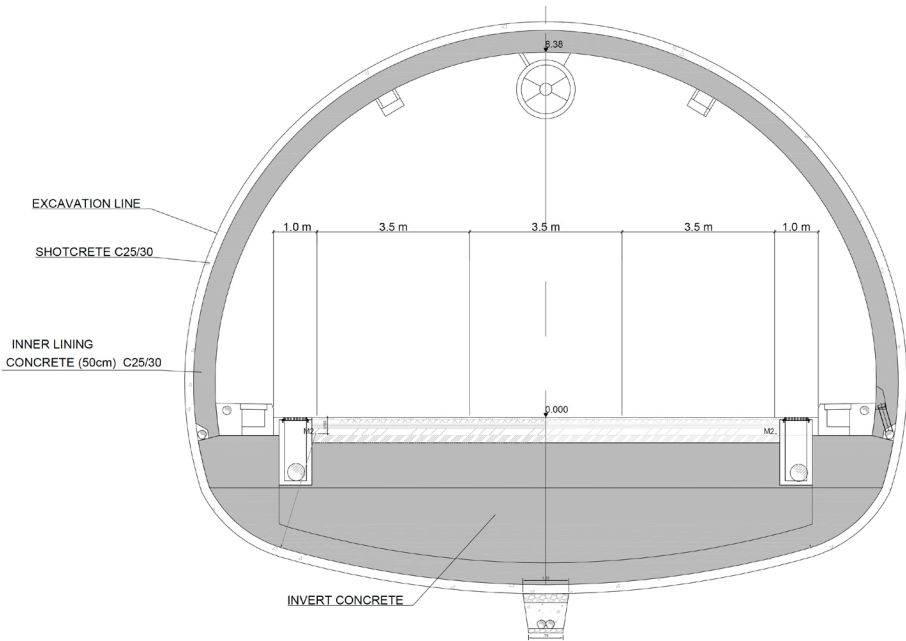
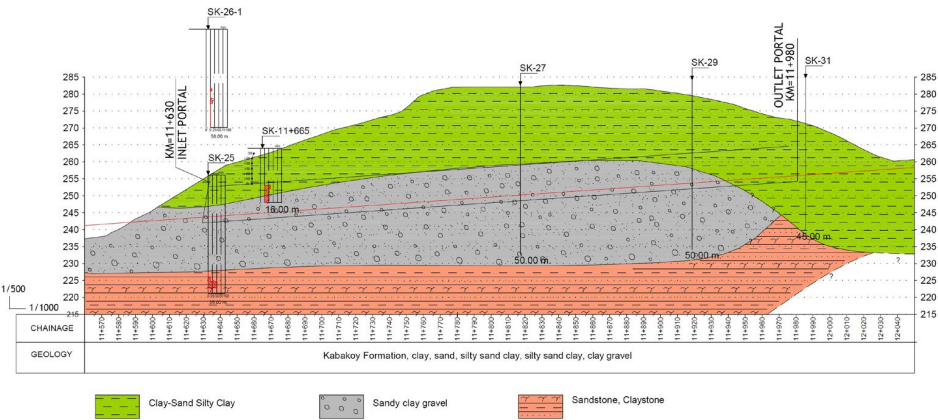


Figure 2. Typical cross-section of the Cukurcayir 1 tunnel. Reprinted with permission from Ref. [43]. 2025, General Directorate of Highways.



(a)



(b)

Figure 3. Plan (a) and geological cross-section (b) of the Cukurcayir 1 tunnel. Reprinted with permission from Ref. [44]. 2025, General Directorate of Highways.

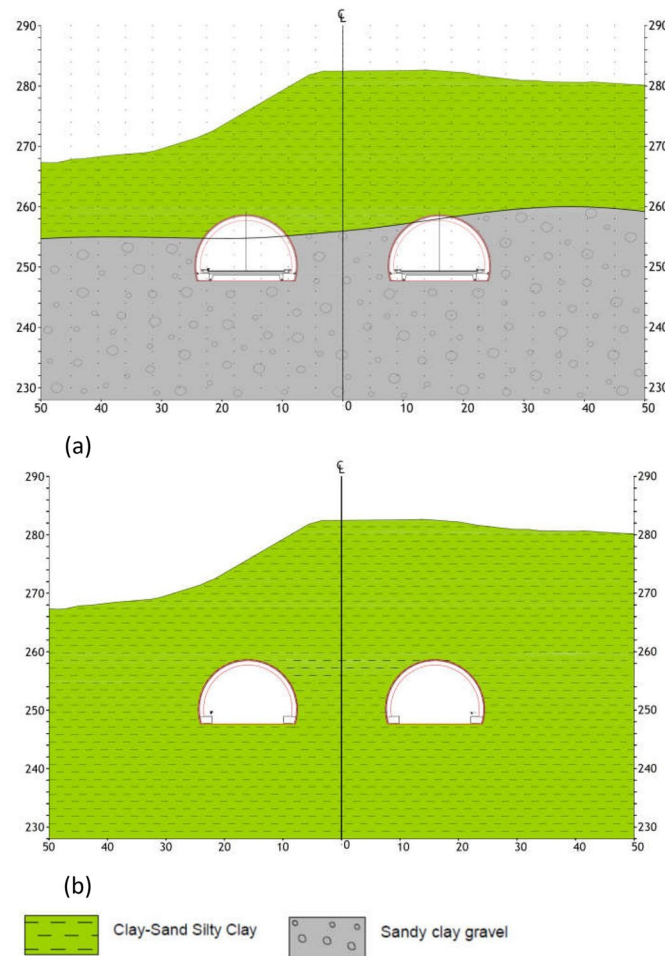


Figure 4. Geological cross-sections at Km:11 + 972 (a) and Km:12 + 040 (b). Reprinted with permission from Ref. [44]. 2025, General Directorate of Highways.



Figure 5. Core samples obtained from SK-26 borehole.

According to the triaxial compression (UU) test results, the average undrained shear strength value was $c_u = \sim 50$ kPa. This value is somewhat low, due to the sample not being saturated and due to the disturbance. In the standard penetration test (SPT) performed on the clay units in the SK-26 borehole, the test values ranged from $N = 14$ to 20 . If the mean N value was set as 17 , the corrected SPT value was $N_{60} = 0.75 \times 17 = \sim 13$, according to the method of Stroud and Butler [45]. The average plasticity index value from the Atterberg experiments on clay units can be approximated as $PI = \sim 23\%$.

After accepting the average values of $N_{60} = 13$ and $PI = 23$, the undrained shear strength value was calculated to be $c_u = 5 \times 13 = 65$ kPa for the short-term. In addition,

Atterberg limit tests were carried out on the clay unit. Using the PI (%) values obtained from the experiments, the long-term parameters for the clay levels were calculated [46]. According to the Atterberg limit values obtained from the undisturbed borehole samples, the average PI value for the unit was 23%, and the effective friction angle value was determined from the PI value using the equation presented in Terzaghi et al. [46]. The PI value was found to be 23, and the effective friction angle (ϕ) was determined to be approximately 29° . The cohesion value was predicted to be approximately 5 kPa. In the gravelly clay sand unit, the cohesion was predicted to be 5 kPa and the internal friction angle was 32° . The parameters of the soil materials are summarized in Table 1.

Table 1. Geotechnical parameters of the lithological units.

Unit	Unit Weight γ (kN/m ³)	Plasticity Index PI (%)	Cohesion c (kPa)	Friction Angle ϕ ($^\circ$)	Modulus of Elasticity E_i (MPa)
Clay (long term—drained)	18	23	5	29	30–70
Clay (short term—undrained)	18	-	65	5	
Gravel and sand	18	-	5	32	

Note: Parameters are distinguished as short term (undrained) and long term (drained) according to the drainage condition relevant to each analysis stage in FLAC3D.

3. Evaluation of the Squeezing Conditions

Since the tunnel section mainly passes through the clayey unit, the squeezing conditions in this unit were examined. The methods of Jethwa et al. [47] and Hoek and Marinos [40] were used.

Although originally proposed for rock masses, the criteria of Jethwa et al. [47] and Hoek and Marinos [48] were used here for comparative purposes to illustrate the similar compaction tendencies observed in overconsolidated or weakly clayey formations. To ensure conceptual consistency, soil-specific approaches (e.g., Anagnostou and Kovári [49]; Mair and Hight [50]) were also considered.

Jethwa et al. [47] classified the degree of compression depending on the ratio of the compressive strength of the rock mass to the in situ stresses. In Equation (1), the formula for compressive strength is given.

$$N_c = \frac{\sigma_{cm}}{P_0} = \frac{\sigma_{cm}}{\gamma \times h} \quad (1)$$

where σ_{cm} is the uniaxial compressive strength of the rock mass, h is the height, and γ is the unit weight.

According to Jethwa et al. [47], squeezing conditions is divided into 4 main categories: if the N_c value is less than 0.4, it indicates a high squeezing level; an N_c value between 0.4 and 0.8 indicates moderately squeezing behavior; an N_c value between 0.8 and 2.0 indicates mildly squeezing behavior; and if the N_c value is greater than 2.0, it indicates non-squeezing behavior.

Hoek and Marinos [48] defined squeezing based on the relationship between the compressive strength of the rock mass and the in situ stresses. Here, strain is given by Equation (2).

$$\varepsilon = 0.2 \times (\sigma_{cm}/p_0)^{-2} \quad (2)$$

The squeezing status was determined according to Jethwa et al. [47] and Hoek and Marinos [48], and is summarized in Table 2. According to Jethwa et al. [47], the de-

gree of squeezing was determined to be high, and according to Hoek and Marinos [48], severe squeezing problems were revealed. According to Hoek and Marinos [48], the support systems should be placed quickly, and the construction quality should be carefully controlled.

Table 2. Evaluation of the squeezing conditions.

σ_{cm}	N_c	Jethwa et al. [47] Degree of Squeezing	ε_t (%)	Hoek and Marinos [48] Degree of Squeezing
0.20	0.27	High	2.6	Severe squeezing problems

Evaluation of the Tunnel Support System Using Analytical Methods

The most important factors to be considered in weak ground tunneling are the plastic zone thickness and the deformations in the tunnel face. In addition, the behavior of the ground surrounding the tunnel is one of the most important factors in the design of the tunnel support system. In the design of the tunnel support system, the tunnel overburden thickness (h), the deformation modulus of the lithological unit (E_m), the strength parameters (c , ϕ and ν), and the tunnel radius (r) are important parameters. It is necessary to calculate plastic deformations (u_{ip}), plastic zone radius (r_p), elastic deformations (u_{ie}), and critical tunnel pressure (p_{cr}) that may occur in the tunnel under unsupported conditions. Closed-form equations are commonly used for these calculations, which are explained in more detail by Hoek and Brown [26] and Hoek [27] and are summarized in Table 3. In these calculations, it was assumed that the tunnel is circular and under hydrostatic stresses and that the soil is isotropic. The Mohr–Coulomb failure criterion was used as the failure criterion. In the selection of the support system of the tunnel, a general analysis was performed by examining the closed-form equations to check the tunnel and the ground interaction behavior.

Another important factor is the displacements occurring in front of and behind the tunnel face. Panet and Guenot [51], Panet [52], Unlu and Gercek [53], Carranza-Torres and Fairhurst [54], Vlachopoulou and Diedrichs [55], and Oke et al. [56] have conducted studies on the installation and timing of installation of support systems. The soil reaction curve (GRC) and support system curves (SRCs) should be determined, which are then used to determine the deformations that will occur and the correct time to apply the supports. The determination of the deformations in the tunnel face immediately after excavation is a critical step. If a high amount of deformation is allowed, situations that cause collapse in the tunnel may be encountered due to the rapid development of deformations in the ground reaction curve [27].

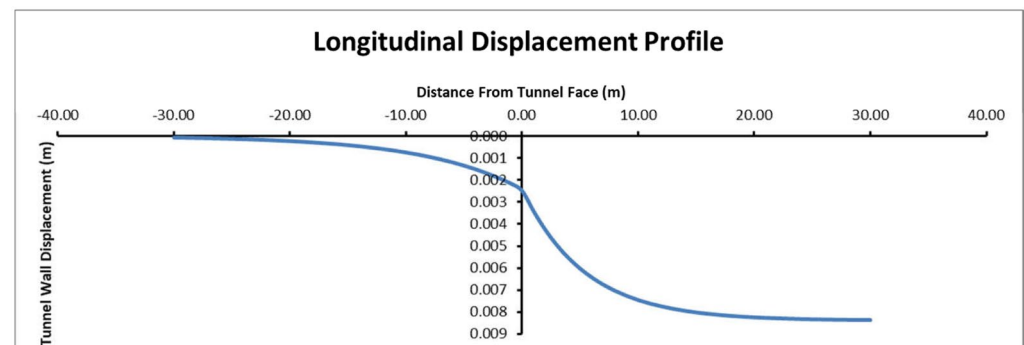
The longitudinal displacement profile is a graphical representation of the displacements in the unsupported tunnel along the tunnel. The longitudinal deformations occurring in the tunnel face, in front of the tunnel, and in the excavated tunnel sections are shown in Figure 6. The horizontal axis displays the longitudinal distance in the tunnel and the vertical axis shows the displacements [52].

Table 3. Closed-form solution equations [26,27].

Mohr–Coulomb Criteria (3)	$\sigma'_1 = \sigma_{cm} + k\sigma'_3$	Radius of the plastic zone r_p when $p_i = 0$ (8)	$r_p = r_o \left[\left(\frac{2(p_o(k-1) + \sigma_{cm})}{(1+k)((k-1)p_i + \sigma_{cm})} \right)^{\frac{1}{k-1}} \right]$
Uniaxial compressive strength of rock mass σ_{cm} (4)	$\sigma_{cm} = \frac{2c' \cos \phi'}{(1 - \sin \phi')}$	inward radial displacement u_{ip} (9)	$u_{ip} = \left(\frac{r_o(1+\nu)}{E_m} \right) [2(1-\nu)(p_o - p_{cr}) \left(\frac{r_p}{r_o} \right)^2 - (1-2\nu)(p_o - p_i)]$
Slope k of $\sigma'_1 - \sigma'_3$ (5)	$k = \frac{(1 + \sin \phi')}{(1 - \sin \phi')}$	Percent strain ε (10)	$\varepsilon\% = \left(\frac{u_i}{r_o} \right) \times 100 = [0.2 - 0.25 \left(\frac{p_i}{p_o} \right) \left(\frac{\sigma_{cm}}{p_o} \right)^{2.4 \left(\frac{p_i}{p_o} \right) - 2}]$
Critical support pressure p_{cr} (6)	$p_{cr} = \frac{2p_o - \sigma_{cm}}{1+k}$	Radius of plastic zone (11)	$\left(\frac{r_p}{r_o} \right) = (1.25 - 0.625 \left(\frac{p_i}{p_o} \right) \left(\frac{\sigma_{cm}}{p_o} \right)^{\left(\frac{p_i}{p_o} \right) - 0.57})$
Radial elastic displacement u_{ie} (7)	$u_{ie} = \frac{r_o(1+\nu)(p_o - p_i)}{E_m}$		

r_p = Plastic zone radius
 u_i = Tunnel sidewall deformation
 r_o = Original tunnel radius in meters
 p_i = Internal support pressure
 p_o = In situ stress = depth below surface · unit weight of rock mass = $p_o = \gamma * h$
 σ_{cm} = Rock mass strength = $2 \cos \phi' / (1 \sin \phi')$
 E_m = Young's modulus or deformation modulus
 ν = Poisson's ratio

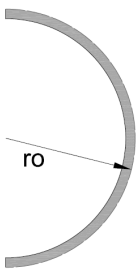
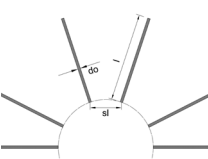
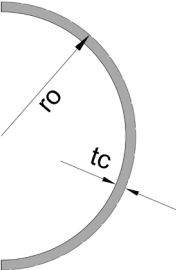
$\sigma_{\geq 1}$ = Axial stress at which failure occurs
 σ'_3 = Confining stress
 c' = Cohesive strength
 ϕ' i ϕ' = Angle of friction of rock mass

**Figure 6.** Longitudinal displacement profile.

Tunnel excavation deformations affect the ground around it up to a distance of 1.5 times the tunnel diameter. After this point, the displacements caused by the excavation are assumed to be stable [27].

The equations used to calculate the pressures of the tunnel support systems are given in Table 4. These equations were proposed by Hoek and Brown [26] and Brady and Brown [57] for shotcrete/concrete and bolt and circular steel supports. It is assumed that the steel shoring is applied in a circularly closed section, the bolts are applied along the tunnel perimeter in a regular pattern, and the shotcrete or concrete is applied as a full section ring [11].

Table 4. Support capacity equations [26,57].

Steel Set		
	σ_{ys} is the yield strength of the steel (MPa) E_s is the Young modulus of the steel (MPa) A_s is the cross-sectional area of the section (m ²) s_l is the set spacing along the tunnel axis (m) r_0 is the radius of the tunnel (m) P_{ssmax} is the maximum support pressure K_{ss} is the stiffness	$P_{ssmax} = \frac{A_s \cdot \sigma_{ys}}{s_l \cdot r_0} \quad (12)$
		$K_{ssmax} = \frac{E_s \cdot A_s}{s_l \cdot r_0^2} \quad (13)$
Rock Bolts		
	d_b is the rockbolt or cable diameter (m) l is the free length of the bolt or cable (m) E_s is the Young modulus of the bolt or cable (MPa) s_c is the circumferential bolt spacing (m) s_l is the longitudinal bolt spacing (m) T_{bf} is the ultimate bolt or cable load P_{sbmax} is the maximum support pressure K_{ss} is the stiffness	$P_{sbmax} = \frac{T_{bf}}{s_l \cdot s_c} \quad (14)$
		$K_{sb} = E_s \cdot \pi \cdot \frac{d_b^2}{4 \cdot s_l \cdot s_c} \quad (15)$
Concrete or Shotcrete		
	σ_{cc} is the uniaxial compressive of the concrete or shotcrete (MPa) E_c is the Young modulus of the concrete or shotcrete (MPa) ν is the Poisson ratio of the concrete or shotcrete t_c is the thickness of the lining (m) r_0 is the radius of the tunnel (m) P_{scmax} is the maximum support pressure K_{ss} is the stiffness	$P_{scmax} = \frac{\sigma_{cc}}{s} \left[1 - \frac{(r_0 - t_c)^2}{r_0^2} \right] \quad (16)$
		$K_{sc} = \left(E_c \cdot \frac{r_0^2 - (r_0 - t_c)^2}{2 \cdot (1 - \nu^2) \cdot (r_0 - t_c) \cdot r_0^2} \right) \quad (17)$

The Cukurcayir 1 tunnel was designed using the NATM [58–60]. The tunnel excavation was divided into top heading, bench, and invert stages. A rigid support system was chosen due to the low overburden height and the weak soil conditions. In addition, the presence of buildings and highway on the surface was another critical factor for choosing the rigid support system. Details of the support system that is used in the Cukurcayir 1 tunnel are presented in Table 5 and Figures 7 and 8. The support system was designed to be an 88.9 mm thick umbrella and the tunnel face is bolted to prevent possible slips and collapse in the tunnel face. In addition, the bolts installed in the bench sections were designed with completely dry drilling and 8 m of PG bolts were applied. A 40 cm layer of shotcrete (C25/30) and HEB200 steel rib were used in the tunnel [44]. In addition, a temporary invert was applied in the top heading to prevent deformations.

Table 5. Tunnel support details.

Shotcrete (cm) C25/30	Steel Rib	Wire Mesh Q589/443	Forepoling	Round Length	Bolts
40	HEB200B	2 layers	Diameter Ø 88.9 mm, L = 9 m, 4.5 m overlap	0.75–1.0 m	PG, L = 8 m, 28 mm

The determined tunnel support system pressures are given in Table 6. The longitudinal displacement profile (Figure 9), ground reaction curve and plastic zone radius plot (Figure 10), and support reaction curve (Figure 11) are given as well.

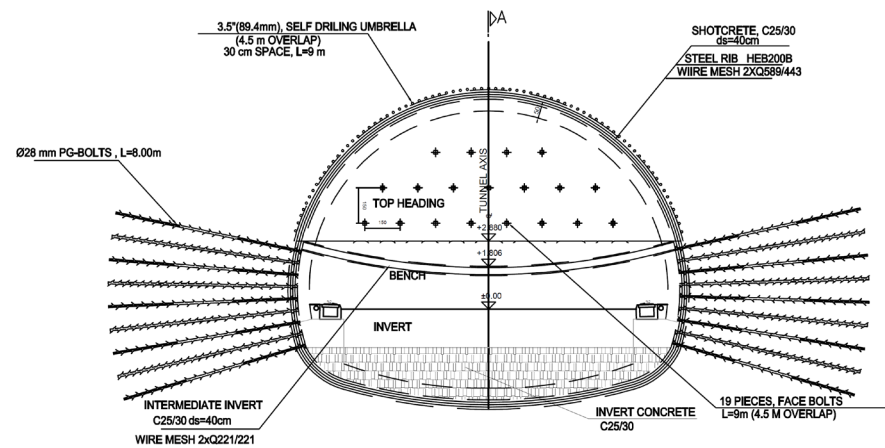


Figure 7. Tunnel cross section support details. Reprinted with permission from Ref. [44]. 2025, General Directorate of Highways.

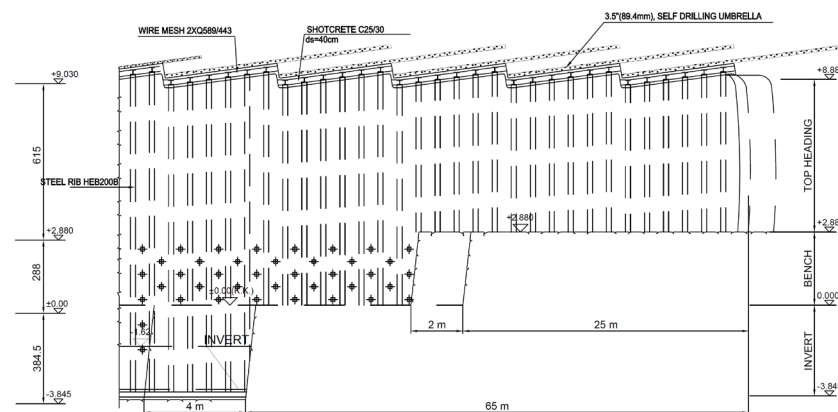


Figure 8. Tunnel support details longitudinal profile. Reprinted with permission from Ref. [44]. 2025, General Directorate of Highways.

Table 6. Support pressure between entrance portal and Km:210 + 150.

	p_{scmax} (MPa)	K_{sc} (MPa/m)	u_{icmax} (m)
Shotcrete (ds = 40 cm)	1.148	178.76	0.0074
Steel Rib (HEB 200)	0.45	29.83	0.016
Rock Bolts	0.373	214.6	0.0027

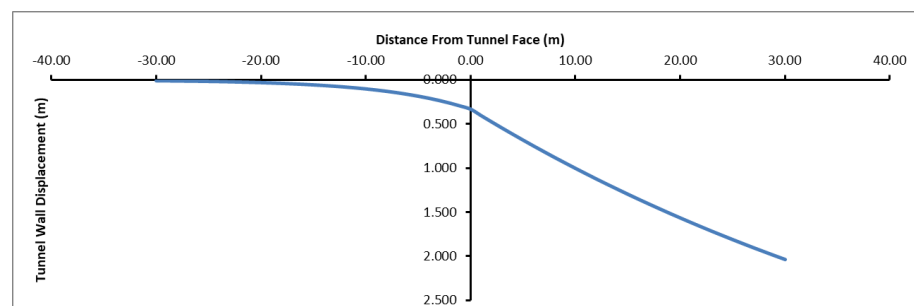


Figure 9. Longitudinal displacement profile for gravel sandy unit.

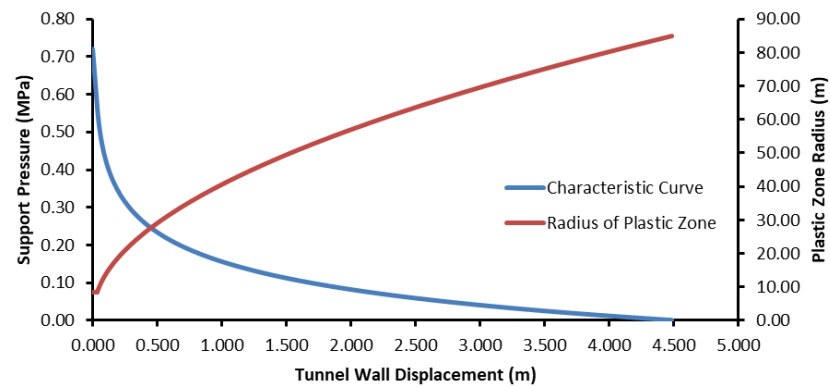


Figure 10. Ground reaction curve for gravel sandy clay unit.

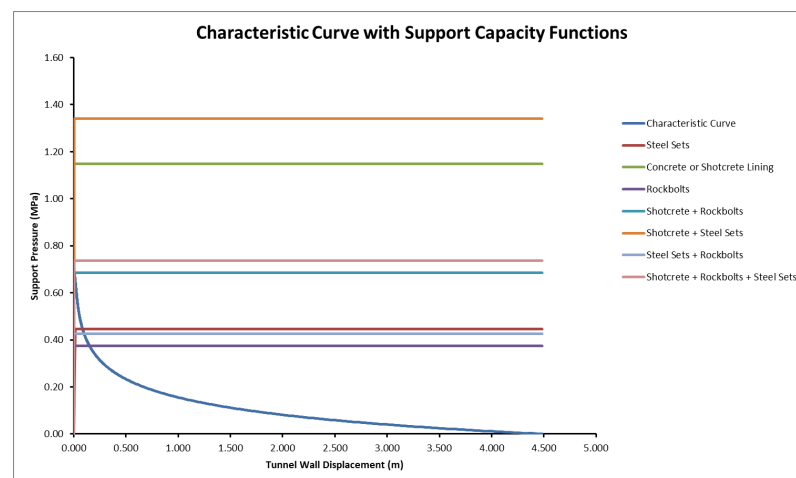


Figure 11. Characteristic curve with support capacity functions for support system elements.

When the longitudinal displacement profile was examined, deformations of 33.4 and 40 cm were observed at the tunnel face and in a 1.0 m section at the back of the tunnel face, respectively. These values continued to increase along the tunnel and reached up to 2.042 m at 30 m behind the tunnel due to the very weak ground conditions (Figure 9). This result demonstrates that the tunnel should always be supported to maintain the stability of the tunnel face and roof.

The ground reaction curve shows that the displacements developed very rapidly in the tunnel. The deformations occurring at the point where the critical pressure was 0.57 MPa was calculated to be 3.4 cm, whereas the radius of the plastic zone was 8.5 m (Figure 10). In the unsupported condition, the plastic zone developed to cover the entire overburden thickness.

On the other hand, in the support reaction curves for the tunnel with the specified support systems (Figure 11), it can be seen that no stability problem occurred in the tunnel if the support systems did not allow deformations in the tunnel.

In the calculations, the critical pressure (P_{cr}) was 0.57 MPa, the elastic deformation (u_{ie}) was 0.16 m, the radius of the plastic zone (r_p) was 88 m, the plastic deformation (u_{ip}) was 4.48 m, and the deformation of the tunnel face (u_{if}) was 0.33 m (Table 7).

Table 7. Analysis of tunnel behavior for gravel–sandy–clay and metasiltstone units.

p_{cr} (MPa)	r_p (m)	U_{ie} (m)	r_{pm}/r_0	u_{ip} (m)	u_{if} (m)
0.57	88.84	0.16	9.98	4.48	0.33

4. Three-Dimensional (3D) Numerical Analysis

Recently, 3D numerical analyses have been used extensively for simulation of tunnel excavations. Zhang et al. [61] provided a detailed comparison with evaluation of three design alternatives through 3D numerical analyses on the stress distribution inside the advance core, ground deformation, and soil arching over the tunnel crown, and they [61] stated that the numerical results agreed well with the field monitoring data. Falcone et al. [62] considered 3D modelling of construction sequences and subsoil heterogeneity effects on the seismic response of shallow tunnels in complex topographical settings, and the results of the numerical investigations performed by Falcone et al. [62] highlighted a clear correlation of the tunnel lining force distribution with the overburden thickness variation along the longitudinal direction of the tunnel. Jafari et al. [63] proposed a failure mechanism for evaluating active tunnel face failure, using the upper-bound theorem of limit analysis and they [63] used 3D numerical analyses during the validation stage of the proposed approach. A FLAC3D-based numerical study was performed by Li et al. [64] for spatiotemporal stability responses of tunnel excavation under cyclical footage impact while Saif et al. [65] employed 3D parametric analyses of cross-passages in shallow tunnels within noncohesive soils. Habib and El-Khouly [66] examined the behavior of a tunnel face reinforced by longitudinal pipes, using a 3D finite element model.

In order to investigate the interaction between the ground and support system in the tunnel, 3D numerical analyses were performed using the Flac3d software [67]. The Flac3d program was developed by Itasca and uses the finite difference method. Field conditions and tunnel support systems can be input into the model and tunnel excavation stages can be introduced in stages as top heading, bench, and invert. The tunnels were modeled as double tubes and the overburden thickness was assumed to be 40 m. In the model, the length was 100 m in the Y direction, 215 m in the X direction, and 120 m in the Y direction (Figure 12). The Mohr–Coulomb failure criterion was used in the model, which was solved using the gravity method. The model was divided into top heading, bench, and invert stages. The excavation process was carried out primarily in the left tunnel, and then the excavations continued for the right tunnel. In order to provide the necessary advance distances in the left tunnel, it was assumed that the first 60 m of excavation at the top heading, up to 40 m at the bench, and 36 m at the invert section were excavated. After this stage, the displacements were reset and then the excavation process continued through the top heading, bench, and invert stages. The tunnel was excavated between 60 m and 66 m at the top heading, 40 and 48 m in the bench section, and between 36 and 44 m in the invert section. After the excavation of the left tunnel was completed, the excavation of the first 10 m section of the right tunnel was conducted, and then the excavation progress continued in the bench and invert sections (Figures 13 and 14). The excavation stages are given in Table 8. In order to observe the displacements in the tunnels, measuring points were arranged around the tunnel. The changes in these points were obtained graphically depending on the number of steps.

The numerical model was calibrated by comparing the predicted tunnel convergence and ground surface subsidence with measured field data. Initial estimates of the elastic modulus (E) were obtained from empirical correlations with SPT-N values [45,68] and the plasticity index of soils. Poisson's ratio (ν) values were selected from within the typical range for cohesive soils (0.30–0.38) [69]. These parameters were iteratively adjusted until the numerical results reproduced the observed deformation magnitudes with an accuracy of $\pm 10\%$. This procedure ensured that the adopted stiffness parameters represented realistic ground response properties for the Cukurcayir 1 tunnel alignment.

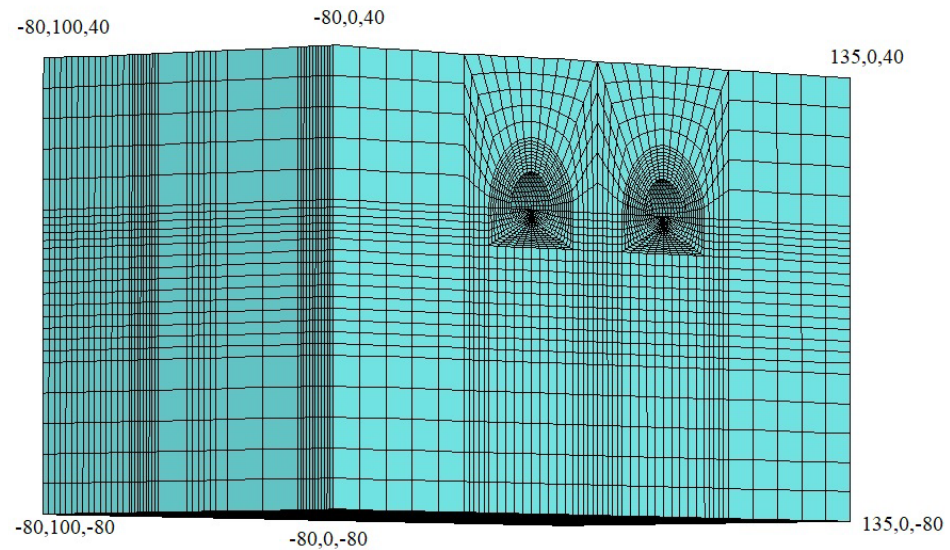


Figure 12. Three-dimensional model conditions.

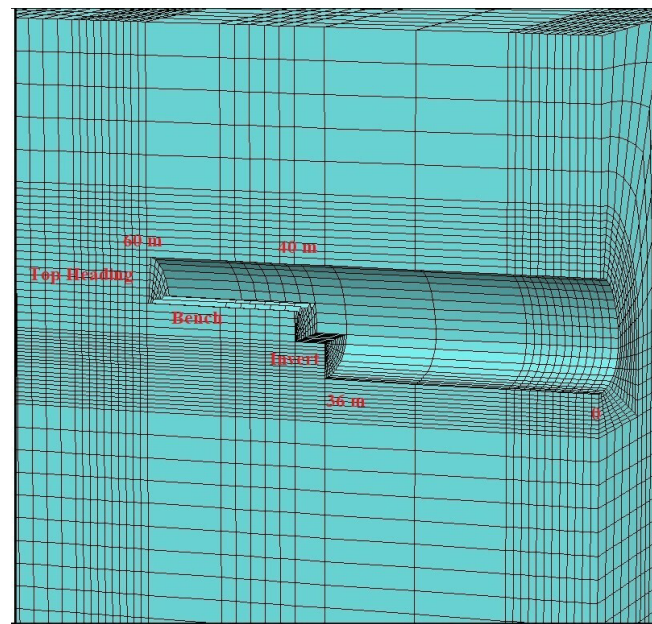
Table 8. Modeling stages.

Stage	Excavation Status
1	In situ stress
2	Top heading excavation between 0 and 60 m, Bench excavation between 0 and 40 m, invert excavation between 0 and 36 m, reset to displacement
3	Top heading excavation between 60 and 61 m (left tunnel)
4	Top heading excavation between 61 and 62 m (left tunnel)
5	Bench excavation between 40 and 42 m (left tunnel)
6	Bench excavation between 42 and 44 m (left tunnel)
7	Invert excavation between 36 and 40 m (left tunnel)
8	Top heading excavation between 62 and 63 m (left tunnel)
9	Top heading excavation between 63 and 64 m (left tunnel)
10	Bench excavation between 44 and 46 m (left tunnel)
11	Top heading excavation between 64 and 65 m (left tunnel)
12	Top heading excavation between 65 and 66 m (left tunnel)
13	Bench excavation between 46 and 48 m (left tunnel)
14	Invert excavation between 40 and 44 m (left tunnel)
15	Top heading excavation between 0 and 1 m (right tunnel)
16	Top heading excavation between 1 and 2 m (right tunnel)
17	Top heading excavation between 2 and 3 m (right tunnel)
18	Top heading excavation between 3 and 4 m (right tunnel)
19	Top heading excavation between 4 and 5 m (right tunnel)
20	Top heading excavation between 5 and 6 m (right tunnel)
21	Top heading excavation between 6 and 7 m (right tunnel)
22	Top heading excavation between 7 and 8 m (right tunnel)
23	Top heading excavation between 9 and 10 m (right tunnel)
24	Bench excavation between 0 and 2 m (right tunnel)
25	Bench excavation between 2 and 4 m (right tunnel)
26	Bench excavation between 4 and 6 m (right tunnel)
27	Invert excavation between 0 and 4 m
28	Bench excavation between 6 and 8 m (right tunnel)
29	Bench excavation between 8 and 10 m (right tunnel)
30	Invert excavation between 4 and 8 m

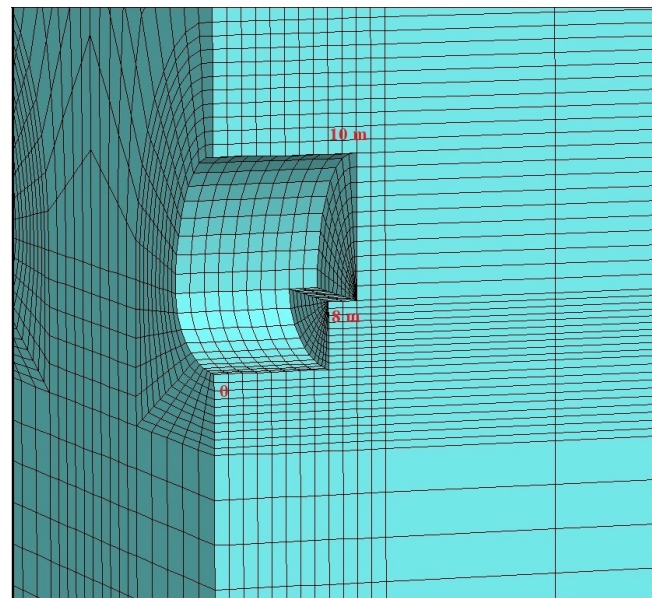
Shotcrete was defined as a shell element, face bolts were defined as cable bolts and the umbrellas were defined as pile elements. The shotcrete, bolt, and umbrella properties are given in Tables 9–11.

Table 9. Parameters used for shotcrete and interior lining concrete.

Element	Ei (GPa)	✓	Y (kg/m ³)
Shotcrete	25	0.25	2500



(a)



(b)

Figure 13. Excavation sequence: (a) left tunnel; (b) right tunnel.

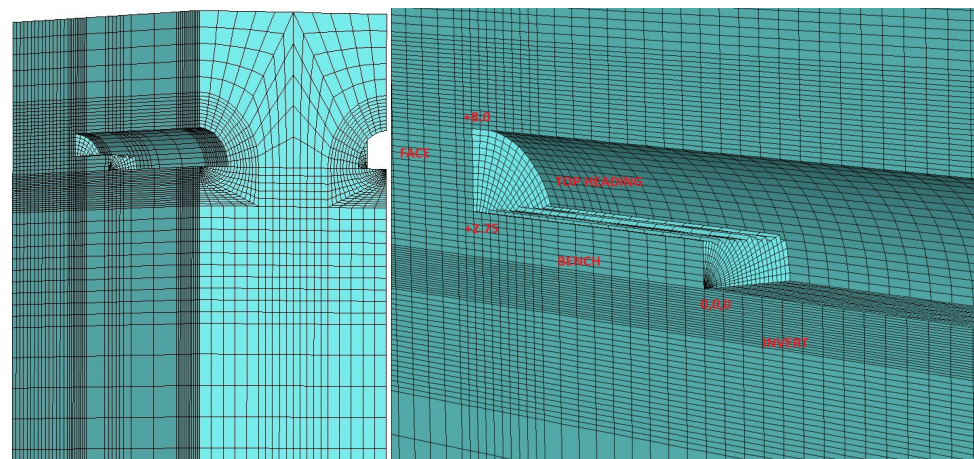


Figure 14. Top heading, bench, and invert excavation sequence.

Table 10. Parameters used for cable bolts.

Cable Modulus (GPa)	Cable Modulus (m ²)	Cable Ultimate Tensile Capacity (kN)	Grout Bond Stiffness (N/m/m)	Grout Cohesive Strength (N/m)
45	1.57×10^{-3}	250	1.75×10^7	2.0×10^5

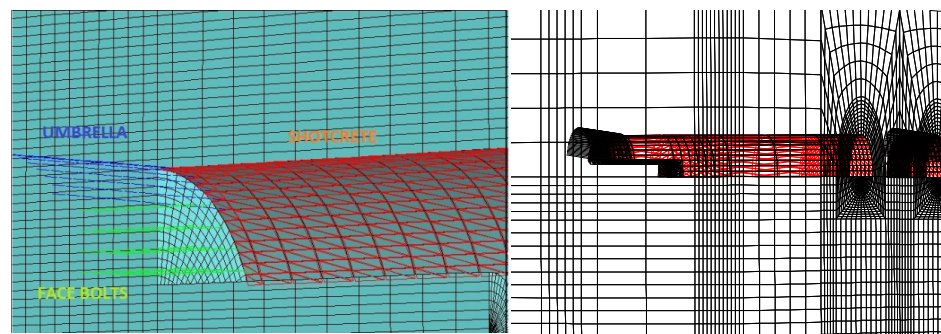
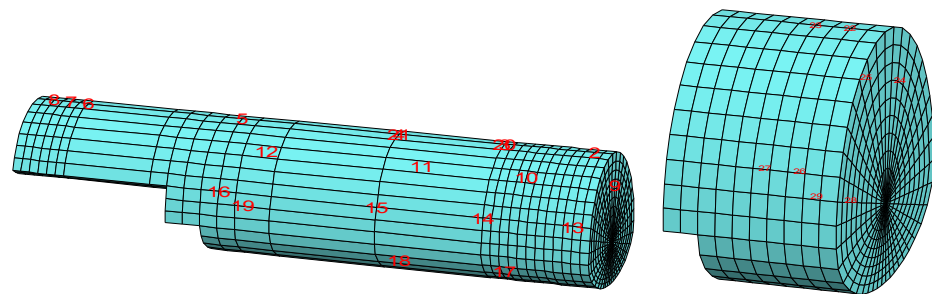
Table 11. Parameters used for umbrellas.

Diameter of Pipe (m)	Thickness of Pipe (m)	ν	E (GPa)	Yield (N)
0.089	0.0065	0.3	200	200,000

The support elements used in the model are presented in Figure 15.

The measurement points along the tunnel for measuring the displacements are shown in Figure 16.

The Flac3d program uses the bulk and shear moduli in its analyses. Equations (18) and (19) were used to calculate the bulk and shear moduli. The calculated values are given in Table 12.

**Figure 15.** Support elements in tunnel.**Figure 16.** Displacement measurement points in the tunnel.**Table 12.** Bulk, shear, and deformation modulus parameters.

E (Deformation Modulus, MPa)	ν (Poisson Ratio)	K (Bulk Modulus, MPa)	G (Shear Modulus, MPa)
50	0.3	40	20

Bulk Modulus

$$K = \frac{E}{3(1 - 2\nu)} \quad (18)$$

Shear Modulus

$$G = \frac{E}{2(1 + \nu)} \quad (19)$$

Evaluation of the Numerical Analysis Results

In order to evaluate the numerical analysis results, the displacements in the tunnel at the pre-defined points were examined. The deformation in the roof section of the left tunnel at 60 m was 3.86 cm at maximum, 4.11 cm at 62 m, and 3.56 cm at 64 m (Figure 15). When the deformation trend was examined at 60 m, a deformation of approximately 1.0 cm occurred after the first excavation, while the deformations at 60 m increased by 0.6 cm to 1.6 cm when the excavation reached between 61 and 62 m. The deformations remained stable at 60 m when the bench excavations were carried out between 40 and 44 m. Thus, the distance between the top heading and the bench appears to be sufficient. When re-excavating between 62 and 63 m in the top heading, the deformations increased by 1 cm and reached up to 2.7 cm. After this stage, the bench was excavated between 44 and 46 m and the deformations continued until 60 m. In the upper half, while the excavations continued between 64 m and 65 m and between 65 m and 66 m, the deformations increased by 1 cm at 60 m in each 1 m of excavation, and the deformations at 66 m increased to 3.6 cm. At the point where the invert excavation started (between 40 and 44 m), the deformations increased by 0.2 cm and stabilized at 3.86 cm (Figure 17). After this stage, excavation was started in the right tunnel and no impact was observed on the left tunnel. Thus, at this point, the stability of the tunnel had been ensured and the inner lining concrete can be installed. The changes at 60 m were similar to those observed at 62 m and 64 m (Figure 17).

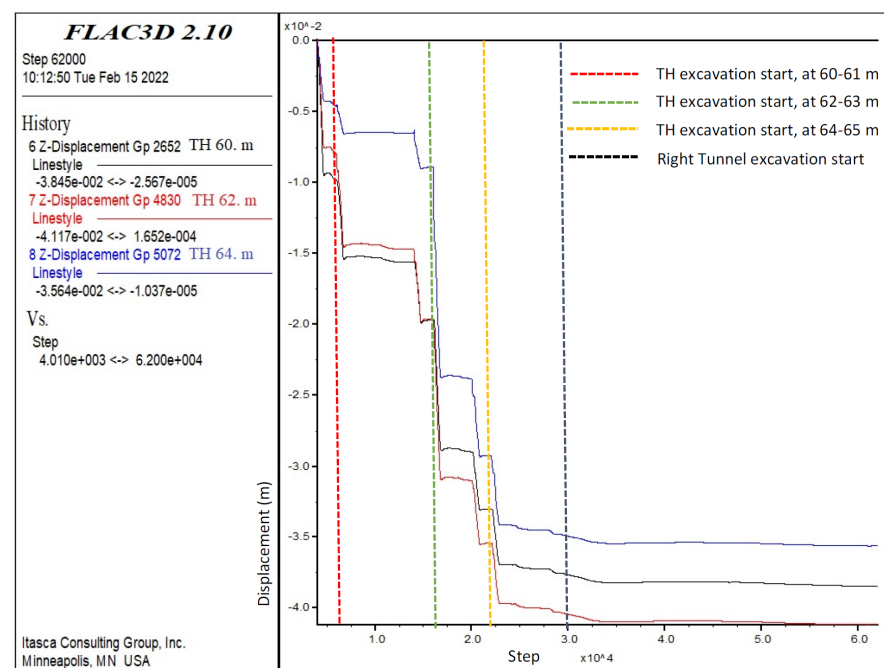


Figure 17. Left tunnel top heading displacements in Z-direction at 60, 62, and 64 m.

When the displacements occurring during the bench excavations were examined (Figure 18), the horizontal displacements were found to be below 1 cm. The horizontal displacements at 2, 10, 12, 24, and 42 m were used in the analysis of the deformations. At this location, an increase in horizontal displacements was only observed at 42 m, which is close to the tunnel excavation face, while it was fixed below 0.1 cm at the other points. Changes in the points close to the tunnel entrance (2, 10, 12, and 24 m) were observed until the top heading was excavated up to 62 m, and then they became fixed. Thus, the excavation of the right tunnel seemed to have no effect on the left tunnel in the first 10 m. At 42 m, while the deformations increased during the top heading, bench and invert excavations, all the deformations remained below 1 cm. No major impact was observed.

The changes at 42 m increased up to 0.4 cm when the top heading was excavated between 60–61 m and 61–62 m. Afterwards, it remained stable, and the horizontal displacements increased to 0.65 cm at the point where the bench excavations started (between 42 and 44 m), and the bench deformations remained stable during the top heading excavations between 64–65 m and 65–66 m. Therefore, the results indicate that the distance between the top heading and bench is sufficient.

The displacements in the invert section are given in Figure 19. Next, the displacements in the invert section at 12, 24, and 42 m were investigated. The maximum displacements of 3.4 cm in the invert section occurred at 42 m. The displacements at 42 m showed an increase of 4 mm during top heading excavation between 60–61 m and 61–62 m, indicating that the top heading excavations did not have a large effect on these displacements. However, after the start of the bench excavations (between 40–42 m and 42–44 m), an increase in deformations of up to 3 cm was observed. When the top heading excavations started again (between 62–63 m and 63–64 m), no significant increase was observed. At the point when the excavation of the lower half started again, the displacement increased again but was at the 0.2 cm level. In the next stage, the 40–44 m excavation of the invert section showed the same displacements as those after the construction was completed. At this stage, stability is ensured at the invert due to the pouring of invert concrete immediately after excavation. The displacements at 12 and 24 m of the tunnel remained at the 1 mm level and continued to be stabilized.

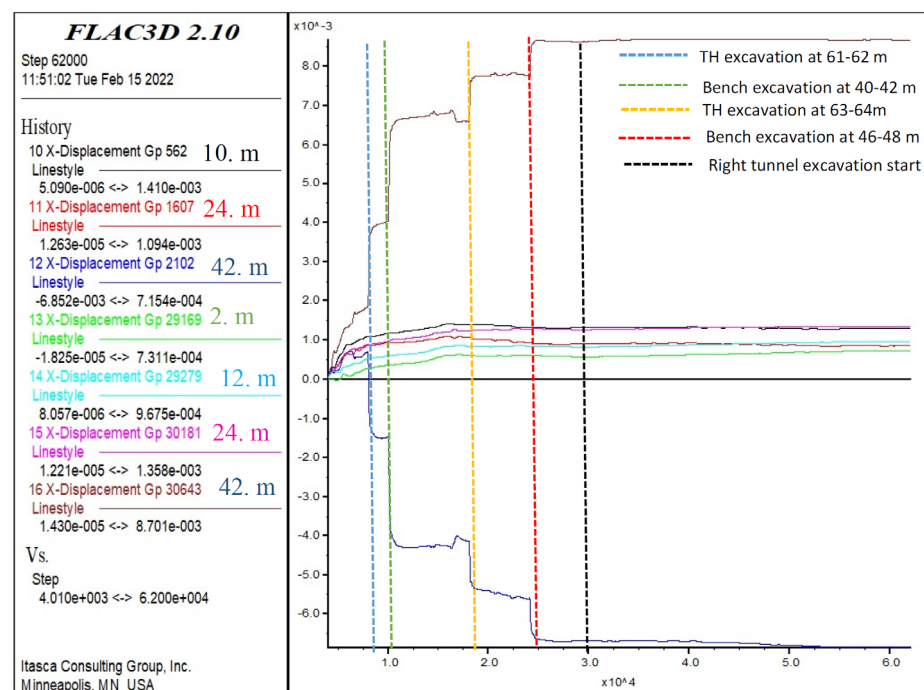


Figure 18. Bench displacements in X-direction at 2, 10, 12, 24, and 42 m.

At the top heading of the right tunnel (Figure 20), deformations occurred at the 7.3 cm level. In the first stage, excavation was carried out in the 10 m section in the top heading and then the lower half, and finally, the invert excavations were carried out. The deformations continued to increase after each stage of excavation of the top heading. While the deformations increased by 0.5 cm in the first 5 m section, the deformation increased to 1.0 cm at each level of the excavations after 5 m. This situation reveals that in the case of progressing without ring closing in the upper half section, the increase in deformation increases. Bench and invert excavations of the tunnel should be carried out sequentially. It

was observed that the deformations in the bench and invert excavations remained constant in the top heading.

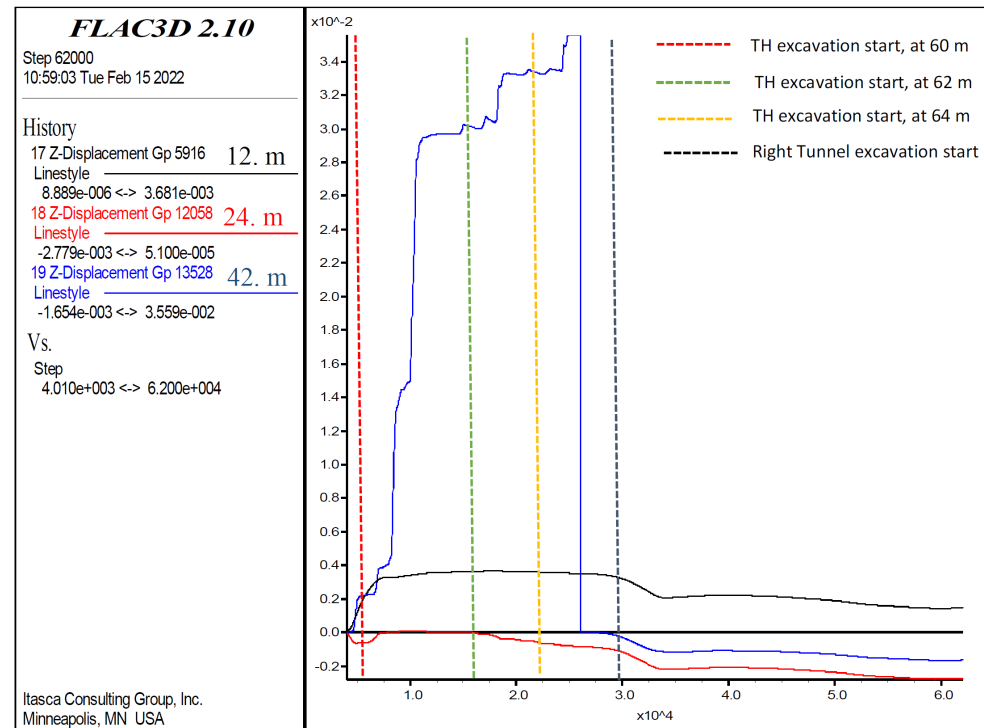


Figure 19. Left tunnel invert section displacements in Z-direction at 12, 24, and 42 m.

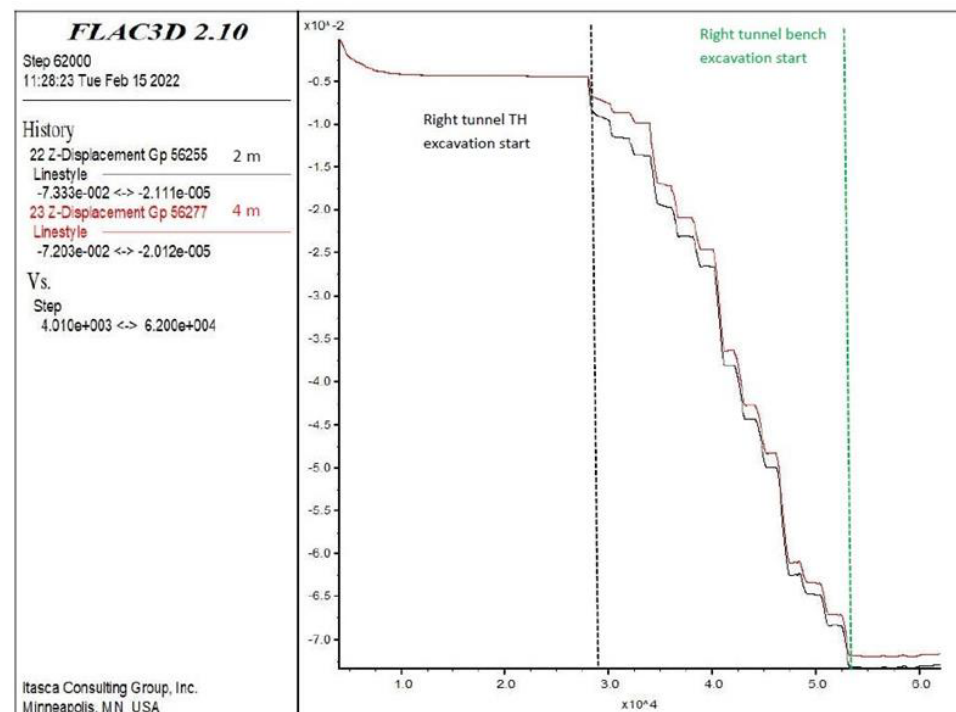


Figure 20. Right tunnel vertical displacements (Z-direction) in top heading at 2 and 4 m.

The deformations that occurred at 2 and 4 m in the bench were examined (Figure 21). The deformations occurring in the bench continued to increase by 0.5 cm during the excavations of the top heading, whereas the deformation in the bench increased to 3.0 cm

after the completion of the first 6 m of excavation in the bench. After the invert excavations, the deformations in the bench became stable.

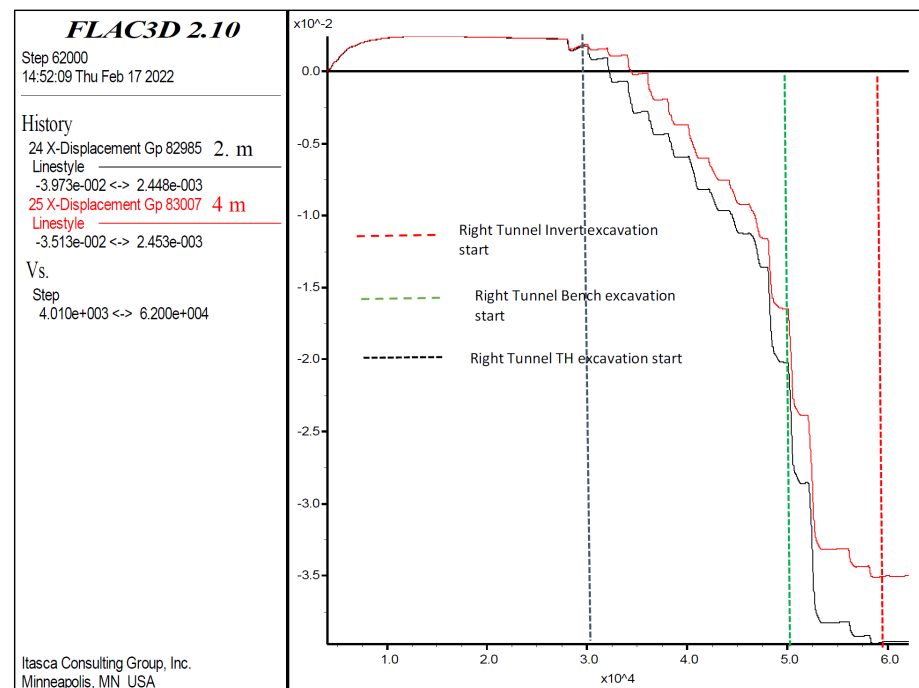


Figure 21. Right tunnel horizontal displacements (X-direction) in the bench at 2 and 4 m.

On the other hand, the deformations at the invert region of the right tunnel increased the deformations in the tunnel roof after the excavation in the top heading (Figure 22). Deformations in the form of swelling occurred at the base section and deformations of up to 18 cm occurred. During the first 10 m of excavation in the top heading, an average of 1.0 cm of swelling occurred at each level, and an increase of 1.5 cm was observed in the bench of the excavation. After the invert excavations, the deformations were fixed by defining the invert concrete in the model. The most important result was that invert excavations should be carried out in shortest time interval possible in tunnels excavated on weak soils in order to ensure stability.

The progressive development and stabilization of displacements that were observed in the numerical analyses can also be interpreted within a granular-creep and particle-packing framework. In weakly cemented or partially saturated clayey–sandy materials, the time-dependent response was not solely governed by macroscopic stress redistribution but also by the micro-mechanical rearrangement of particles and contact evolution under sustained loading. The gradual reduction in deformation rate observed during the excavation and support installation stages can therefore be attributed to an increase in packing density and the progressive mobilization of inter-particle friction. Similar findings were reported by Tong et al. [70,71], who demonstrated that particle shape and packing fraction significantly influence the creep and fluidity of granular materials. This interpretation provides a micro-mechanical rationale for the delayed deformation stabilization trends observed in both the numerical analysis and field monitoring results.

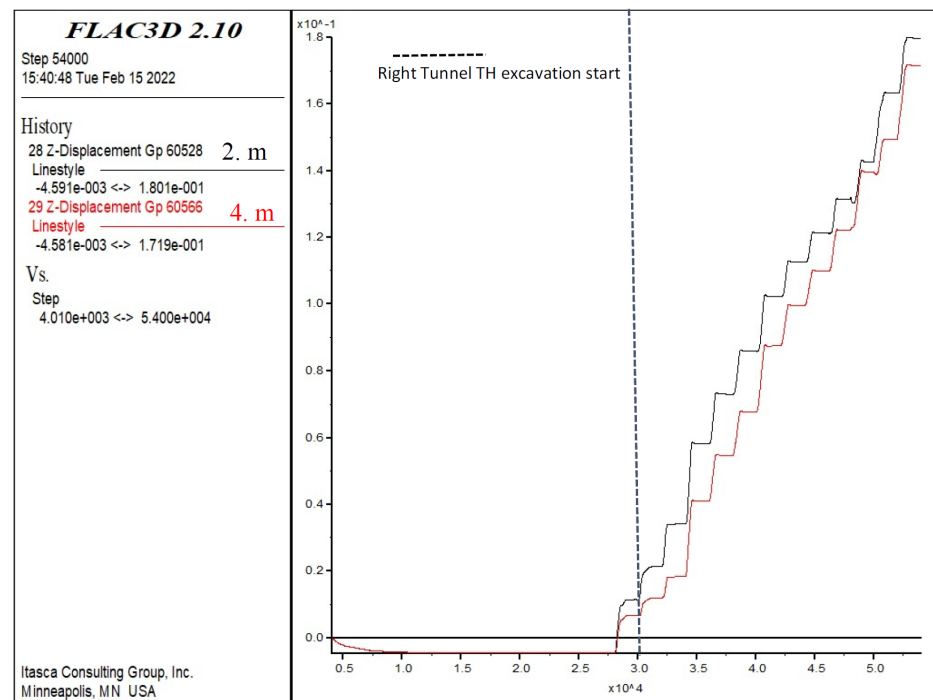


Figure 22. Right tunnel vertical displacements (Z-direction) in the invert at 2 and 4 m.

5. Comparison of Measured Deformations and Analysis Results

One of the most important issues to be considered in tunnel excavation in weak clay and sandy units is keeping water away from the tunnel. In such soils, when clayey and sandy units interact, excess water adds an additional load depending on the swelling feature of the soil, and it triggers slippage by decreasing the cohesion due to undrained conditions. For this reason, one of the most important steps during the construction phase is installing bolts and umbrellas in a dry environment. All the boreholes in Cukurcayir 1 tunnel were dry-drilled and contact with water was prevented. As can be seen from Figure 23, the tunnel face remained stable immediately after the excavation.



Figure 23. Tunnel excavation face.

In order to observe the tunnel stability, deformation measurements were carried out continuously. The deformation measurement points are shown in Figure 24. The measurement results are presented in Figures 25 and 26.

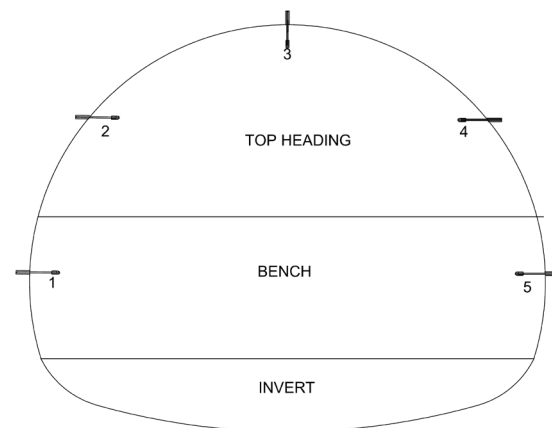


Figure 24. Deformation measurement points.

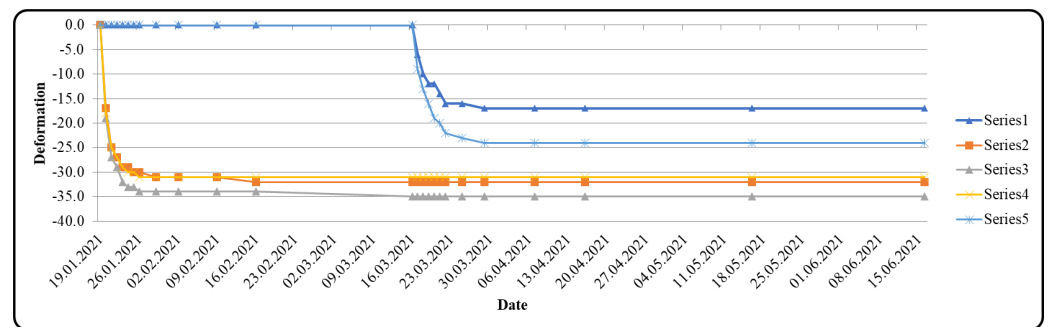


Figure 25. Vertical displacement at Km:11 + 677 (deformation unit is mm).

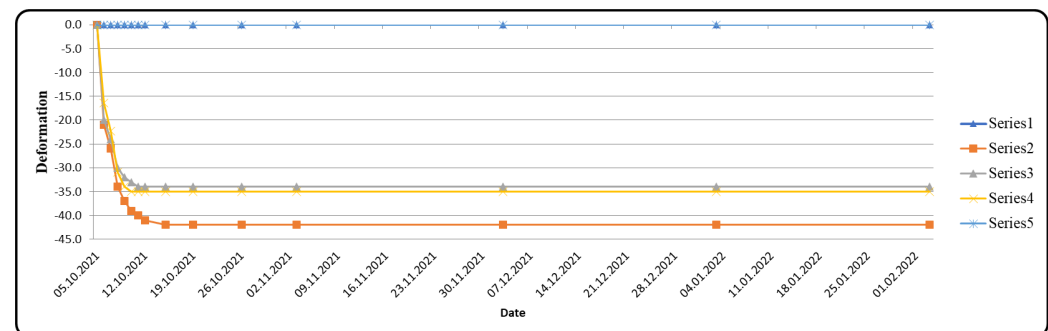


Figure 26. Vertical displacement at Km:11 + 864 (deformation unit is mm).

The maximum deformations were 4.2 cm at Km:11 + 864 and 3 cm at Km:11 + 796. Compared to the deformations on the left tunnel roof given in Figure 22, the measurements were at the 4.0 cm level. These measured values are in agreement with those from the numerical analysis.

In the numerical analysis results for the right tunnel (Figures 20–22), the deformations continued to increase in every round if only the 10 m interval advance was carried out in the top heading without bench or invert excavation.

No stability problems were encountered in the tunnel when the tunnel excavation and support system installation were performed sequentially. In other words, if the top heading, bench, and invert excavations are carried out consecutively, the deformations remain stable.

6. Conclusions

In this study, the Cukurcayir 1 tunnel, which was excavated in weak ground units under a shallow overburden close to a residential area, was studied using 3D numerical analyses and the results were compared with the deformations measured during the excavations. As stated by Namlı [72], ground surfaces settlements are the most significant restriction when constructing shallow metro station tunnels in urban areas. This statement is valid for each shallow underground opening excavated in urban areas. Due to the tunnel's location, it was excavated using a rigid support system that does not allow for deformation. Collapses and excessive deformations were prevented due to the implementation of an umbrella system in order to support the tunnel roof. The conclusions of this study are as follows:

- (a) The results of numerical analyses and the deformation measurements performed in the tunnel were similar. Due to the closeness of the tunnel to a residential area, the internal tunnel deformations were limited and the deformations were stabilized at the 4 cm level.
- (b) One of the most important factors when excavating tunnels in weak ground is tunnel roof and face stability. The design of the Cukurcayir 1 tunnel ensured this stability.
- (c) Tunnel top heading, bench, and invert excavation phases should be kept as short as possible, and the ring should be closed immediately. In weak ground excavations, the distance between the top heading and bench should be a maximum of 20–25 m. During the analyses and excavation stages, this behavior was clearly observed.
- (d) During tunnel excavations, squeezing should be expected in the tunnel invert during the excavation process in the top heading. In these cases, the tunnel invert should be immediately excavated and the ring should be closed with invert concrete.
- (e) In tunnels excavated as double tubes, the distance between the excavation face of the tunnels is critical. For the tunnels to not affect each other, a distance of at least 4 times the tunnel diameter should be left between the tunnel excavation faces. During the excavations of the Cukurcayir 1 tunnel, this principle was adhered to and successful results were achieved.
- (f) The creation of arching around the tunnel by allowing deformation in weak ground often causes a very rapid increase in deformations. In this case, it caused failures in the tunnel. Therefore, support systems should be sized by prioritizing the prevention of excess deformation on weak ground.
- (g) It is worth noting that the FLAC3d analyses presented in this study included several simplifications. Groundwater level changes were represented using steady-state pore pressure conditions rather than transient flow conditions. Time-dependent soil behaviors, such as consolidation and creep, were not explicitly modeled and were analyzed qualitatively based on previous studies. Furthermore, the tunnel lining was assumed to exhibit linear elastic behavior. While these assumptions are limited in scope, they were considered acceptable for comparing deformation mechanisms and support performance.
- (h) The tunnel roof must be stabilized with a long umbrella. Otherwise, possible roof slips could cause failures, leading to collapse. Deformations should not be allowed, especially in shallow tunnels excavated in a residential area. A collapse in the tunnel is likely to reflect up to the surface and cause damage.
- (i) Future research can enhance the predictive and interpretive capabilities of such analyses by integrating multimodal and coupled hydromechanical frameworks. Recent advances, such as multi-channel fusion prediction of TBM thrust based on multimodal decomposition and reconstruction [73] and analytical modeling of nonlinear seismic metasurfaces of saturated porous media [74], offer complementary perspectives for

linking numerical simulations, in situ monitoring, and fluid–solid interactions. Integrating these data-driven analytical approaches with numerical modeling could significantly improve the reliability of tunnel deformation predictions and improve performance under complex ground conditions. It will be useful to consider the following points in future studies. In particular, further studies of twin-tube tunnels and the detailed studies of their interaction conditions are crucial. Determination of the effects of top heading, bench and invert distances on tunnel deformations is important and necessary revisions should be performed. In addition, the optimum ring closure distance should be determined. As the most important recommendation, the additional supports that can be applied in addition to the temporary invert to prevent vertical and horizontal deformations in the upper half should be re-investigated.

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