Abstract

Tunnel excavations near existing structures impose additional loads on the nearby structures and influence the stress-strain regime of the soils. In the present study, the ground response during the close construction of tunnels and the structural behavior of a segmental lining with an elliptical-shaped cross-section were investigated. The effect of nearby structures and the influence distance for a nearby excavation were discussed in terms of the changes in volume loss around the tunnel, the horizontal soil deformation, and the distribution of internal forces. Furthermore, the structural behavior of the lining was tested by showing the capacity of the reinforced concrete section and by considering the crack development in the elliptical-shaped segment. The results demonstrated that the influence distance of the elliptical-shaped tunnel excavated in coarse-grain soils was approximately four times the minor diameter of the tunnel and that the axial force increased slightly in the part of the lining close to the existing structure when the pillar width was increased. Furthermore, it was demonstrated that crack development is an important issue in tunnels with an elliptical cross-section near the spring line; and therefore, a safe zone corresponding to the onset of several cracks with an allowable width and average crack spacing was recommended.

Keywords: Elliptical cross-section lining; Close underground excavation; Influence distance; Ground surface settlement; Segment crack; Segment section capacity; Permissible crack width

1. Introduction

Tunnel excavations in close proximity to existing structures impose additional loads on the nearby structures and influence the stress-strain regime of the soils. Close underground excavations are mainly influenced by the spacing between the tunnels, the size and shape of the lining, the relative stiffness of the support to the surrounding soils, and the method of excavation.

The interaction between multiple closely spaced tunnels has been studied using different approaches, including empirical and analytical methods (Sweeney, 2006), field measurements (Yamaguchi et al., 1998; Wang et al., 2019), and physical and numerical models (Kim et al., 1996; Addenbrooke and Potts, 2001; Chapman et al., 2002; Li and Yuan, 2012; Boonyarak and Ng, 2014). Kim et al. (1996) investigated the influence of close excavations by building two sets of physical tunnel models with parallel and perpendicular alignments in clay. It was found that the interaction between the tunnels with circular cross-sections is not significant when the width of the soil column between the tunnels is greater than the tunnel diameter and that an additional bending moment induced by the close tunnel construction can lead to visible cracks in the original tunnel lining. Yamaguchi et al. (1998) conducted a study on four closely spaced tunnels excavated in Kyoto (Japan) by discussing the actual observed behavior of the tunnels.
and the ground. Their study was limited to the investigation of ground surface settlement and load variation on the segment owing to the excavation of the new parallel tunnels. Addenbrooke and Potts (2001) studied the surface and subsurface effects of the construction of twin tunnels having circular cross-sections in stiff clay. Two sets of parallel and piggyback tunnels were considered, and the influence zone of the tunneling was discussed. In their study, the interaction effect disappeared when the spacing between the tunnels exceeded seven diameters; the ground surface settlement, volume loss, and lining deformation were studied. Fang et al. (2015) presented the measured data on twin tunnels excavated beneath other existing tunnels. The new tunnels with horseshoe-shaped cross-sections were excavated in a mixture of clay, silt, fine sand, and gravel using the shallow tunneling method. Their study was limited to interpretations of the measured ground surface and the tunnel deformation of the existing tunnels. Ng et al. (2018) investigated the interaction effect of new circular tunnels on the response of an existing horseshoe-shaped tunnel in sand. Numerical analyses were used to demonstrate the stress, strain, and displacement distributions in the lining and the stress path of the soil around the existing tunnel. Wang et al. (2019) studied an excavation case of two closely spaced tunnels with horseshoe-shaped cross-sections in silty clay by interpreting the measurement data of the ground surface settlement. It was observed that the effect of the tunnel excavation in front of the tunnel face was four times that of the tunnel diameter.

In addition to the ground response during the close construction of tunnels, the structural behavior of the segmental lining with an elliptical-shaped cross-section is also important. The structural response of the lining under the loads can be investigated by knowing the distribution of axial forces and bending moments in the lining. The amounts and the distribution of the loads, the location of the segment joints, and the shape of the lining’s cross-section alter the distribution of axial forces and bending moments in the lining (Do et al., 2013). A better understanding of the cracking behavior in the concrete lining is also necessary for the proper structural design of tunnels (Yin et al., 2001). A structural crack might occur during the lining installation (e.g., machine jack forces); however, the final soil-water pressure following the lining installation represents the dominant load that normally causes cracks in a tunnel. An axial force-bending moment (P-M) interaction diagram is typically used for the structural design of segments. The diagram presents a zone outside of which segment yielding happens. Yao et al. (2018) and Spagnuolo et al. (2017) constructed a P-M interaction diagram.
diagram for a hybrid fiber-reinforcement concrete segment and reinforcement segments, respectively, and discussed the design failure capacity of these segments. Generally, reinforced concrete (RC) segments under ordinary loads do not reach the failure stage; and thus, the main issue concerning tunnels with an RC segmental lining is to control the width and the length of the cracks.

Few studies have paid attention to the effect of close tunneling on both ground responses around the tunnel and the structural behavior of the tunnel segments. In addition, the effect of close underground excavations for tunnels having an elliptical-shaped cross-section, especially in coarse-grain soils, remains unknown. Herein, the ground response during the close construction of a tunnel and the structural behavior of the segmental lining with an elliptical-shaped cross-section are investigated using measurement data and a numerical investigation. The effect of nearby structures and the influence distance for the close-proximity excavation are discussed in terms of the changes in volume loss around the tunnel, the horizontal soil deformation, and the distribution of internal forces.

Attention is given to the development of cracks in the RC-segmented lining with an elliptical shape using an axial force-bending moment capacity diagram. As a new attempt, a safe zone with the combined loads of axial forces and bending moments, corresponding to the onset of several cracks with allowable widths and average crack spacing, is recommended. Two representative new tunnels with elliptical-shaped cross-sections in close proximity to an existing tunnel in coarse-grain soils in Japan are chosen as examples.

2. Closely spaced tunnels with elliptical cross-sections

The two new tunnels with elliptical-shaped cross-sections excavated near an existing box-type (EBT) tunnel are located in Tokyo (Japan). Fig. 1 presents the plan view and the location of the ground surface monitoring points, while Fig. 2 presents a schematic cross-section and the soil stratification of the site. The major and minor diameters of the new tunnels are 6.6 and 5.5 m, respectively, and the width and the height of the EBT tunnel are 18.4 and 9 m, respectively. The tunnels are located in dense coarse-grained sandy and gravelly soils with standard penetration test (SPT) values of greater than 50 (as illustrated by the Mg and Tos layers in Fig. 2). Above the coarse-grained soils are soft clay and loam layers with SPT values of 2–5. Different laboratory and in-situ tests were performed to obtain the geotechnical parameters for each soil layer (Table 1). The generally recommended values are used in the case of the non-availability of geotechnical test results. Fig. 3 presents the measured ground surface settlement along the A and B lines. A limited settlement is observed after the machine tail passes through the sections, and the maximum settlement in both lines is limited to 4 mm. The elliptical-shaped linings present signs of multiple cracks in the longitudinal direction of the tunnel after the completion of the construction of the lines. There is no instrumentation above the tunnels on the ground surface in the transverse direction and the settlement trough is not measured in that direction.

3. Analysis method

The excavation is modeled using the convergence-confinement method (Panet, 1995) under 2D plain-strain conditions and with the finite element code GTS-NX (2018). The excavation of the tunnel before the lining installation is simulated by reducing the normal internal pressure at the wall of the opening from the in situ state of stress to a specified value using Eq. (1): 
\[
\sigma = (1 - z)\sigma_0
\]

where \(z\) is the confinement loss factor simulating the incremental excavation. The value of \(z\) is determined by introducing the amount of volume loss equivalent to the measured ground surface settlement above the A line after the machine tail passes (0.5 mm in Fig. 3(a)). By changing
the value of the confinement loss factor, the volume loss prescribed for the A line excavation is chosen to be approximately 0.03%. The same confinement loss factor used for the A line is employed for the B line; therefore, the volume loss of the B line is not controlled but predicted by numerical analysis. After applying the volume loss for both lines, the tunnel lining and the grouting pressure are activated. According to Franza et al. (2019), the magnitude of ground loss can be expressed with two parameters: tunnel volume loss ($V_{LT}$) and soil volume loss ($V_{LS}$). The soil volume loss is $V_{LS} = (V_s/V_0) \times 100$, where $V_s$ is the volume of the settlement trough per unit length of the tunnel and $V_0$ is the area of the tunnel cross-section. The tunnel volume loss is $V_{LT} = (\Delta V/V_0) \times 100$, in which $\Delta V$ is the ground loss in the tunnel periphery. For tunnels constructed in clayey grounds under undrained (constant volume) conditions, $V_{LT} = V_{LS}$; and thus, surface measurements can be used to evaluate the volume loss at any depth. However, for tunnels constructed in drained granular soil, $V_{LT}$ might not be equal to $V_{LS}$, and the relationship between the two is affected by the volumetric strain of the soil. In this study, the tunnels are excavated in sandy soil (Mg and Tos layers; Fig. 2), and there are 10.2 m of cohesive soil above them (6.8 m of loam and 3.4 m of soft clay; Fig. 2). The developed ground loss around the tunnel reaches the ground surface; and therefore, the effect of all the soil lay-

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Table 1
Parameters obtained from geotechnical tests performed on site.

<table>
<thead>
<tr>
<th>Parameter (unit)</th>
<th>Symbol</th>
<th>Lm</th>
<th>Lc</th>
<th>Mg</th>
<th>Tos</th>
<th>Test method or recommended values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet unit weight (kN/m$^3$)</td>
<td>$\gamma$</td>
<td>12</td>
<td>13.3</td>
<td>$\approx 20$</td>
<td>18.6</td>
<td>18.6</td>
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<tr>
<td>Cohesion (kPa)</td>
<td>$c'$</td>
<td>$\approx 60$</td>
<td>$\approx 90$</td>
<td>$\approx 0$</td>
<td>$\approx 0$</td>
<td>$\approx 0$</td>
</tr>
<tr>
<td>Internal friction angle (°)</td>
<td>$\phi'$</td>
<td>$\approx 0$</td>
<td>$\approx 0$</td>
<td>$\approx 35$</td>
<td>35</td>
<td>35</td>
</tr>
<tr>
<td>Compression index (-)</td>
<td>$C_r$</td>
<td>3.04</td>
<td>1.31–2.29</td>
<td>1.31</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Preconsolidation pressure (kPa)</td>
<td>$P_c$</td>
<td>173.2</td>
<td>210–344.7</td>
<td>210.1</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Uniaxial compressive strength (kPa)</td>
<td>$q_u$</td>
<td>126</td>
<td>156.4–228.6</td>
<td>156.4</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Deformation coefficient (MPa)</td>
<td>$E_{50}$</td>
<td>16</td>
<td>8.54–28.43</td>
<td>8540</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Earth pressure at rest (kPa)</td>
<td>$P_{o}$</td>
<td>32.7</td>
<td>49.2</td>
<td>1201</td>
<td>1183.3</td>
<td>1183.3</td>
</tr>
<tr>
<td>Yield pressure (kPa)</td>
<td>$P_y$</td>
<td>104.8</td>
<td>96.2</td>
<td>1801.5</td>
<td>1881.2</td>
<td>1881.2</td>
</tr>
<tr>
<td>Ground coefficient (MN/m$^3$)</td>
<td>$K_m$</td>
<td>12.43</td>
<td>127.6</td>
<td>488</td>
<td>2150</td>
<td>2150</td>
</tr>
<tr>
<td>Deformation modulus (MPa)</td>
<td>$E_{sm}$</td>
<td>7.4</td>
<td>8.04</td>
<td>24.3</td>
<td>93.9</td>
<td>93.9</td>
</tr>
</tbody>
</table>

$: For the Mg and Tog layers, the generally recommended values are used.

$: For the Lm and Lc layers, the values are obtained from uniaxial compression tests assuming that $c' = q_u/2$; for the other coarse grain soil layers, $c'$ is assumed to be approximately zero. In the Tos layers, a triaxial consolidated drained (CD) test is performed; however, according to the generally recommended values, $c' \approx 0$ kPa is used instead.

$: $\phi'$ is assumed to be approximately zero for the fine grain layers.

$: $E_{sm}$ is obtained from the results of the Lateral Load Test (LLT) and the SPT test. For the SPT test, $E_{sm} = 670 \times N_{98.6}^{0.986}$ is used.

$: All the parameters of the Tog layers are assumed to be identical to those of the Tos layers.
ers above the tunnel invert should be taken into account for the calculation of the volume loss in the layered soil.

In this study, the true nature of the ground volume loss is unknown; and therefore, it is assumed that \( V_{LT} = V_{LS} = V_L \). In the numerical analyses, the ground loss is calculated through the tunnel volume loss, while in the field study, the ground loss is calculated through the soil volume loss. The settlement trough volume is obtained using the Gaussian distribution curve (Peck, 1969; Schmidt, 1969) as

\[
V_s = \sqrt{2\pi} \sigma_{max},
\]

where \( i \) is the trough width and \( \sigma_{max} \) is the maximum ground surface settlement. In the case of layered soils, the trough width is calculated as follows:

\[
i = K_1z_1 + K_2z_2
\]

where \( K_i \) is the trough width factor (\( K = 0.5 \) for tunnels in clay and 0.35 for tunnels in sand and gravel) of soil layer 1 with a thickness of \( z_1 \), and \( K_2 \) is the trough width factor of soil layer 2 with a thickness of \( z_2 \) (Selby, 1988). Here, \( z_1 = 10.2 \) m (loam and soft clay), \( z_2 = 5.7 \) m (sand and gravel layers to the tunnel spring line level), \( K_1 = 0.5 \), and \( K_2 = 0.35 \); and therefore, the trough width becomes \( i = 7.1 \). By setting \( i = 7.1 \) and \( \sigma_{max} = 0.5 \) mm (surface settlement above the A line after the machine tail passes), and knowing the original cross-section of the new tunnel, \( V_L \approx 0.03\% \) is obtained for the A line. According to Fig. 3(a), the settlement above the A line at the section under study, after the second tunnel installation, is taken to be \( \sigma_{max} = 3 \) mm, which is also equal to the settlement above the A line after 3 months. By knowing the values of \( i = 7.1 \) and \( \sigma_{max} = 3 \) mm, the volume loss of the first tunnel after the lining installation of the second tunnel is calculated to be \( V_L \approx 0.17\% \).

For stress-path problems related to unloading and reloading events, an advanced soil model is required. For instance, the hardening soil (HS) model (Schanz et al., 1999) provides realistic displacements in the case of an excavation (Stallebrass et al., 1994; Grammatikopoulou et al., 2008; Obrzud, 2010). Here, the HS model was adopted to represent the soil layers. In this model, the stress-strain behavior during loading is highly non-linear and loading stiffness is stress-dependent, as follows:

\[
E_{50} = E_{50}^{ref} \left( c \cos \varphi - \sigma'_s \sin \varphi \right)^m
\]

where \( E_{50}^{ref} \) is a reference stiffness modulus corresponding to reference stress \( \sigma_{ref} \), \( c \) and \( \varphi \) are strength parameters, and \( m \) is a coefficient that determines the amount of stress dependency.

Unloading and reloading also use the stress-dependent stiffness as follows:

\[
E_{ur} = E_{ur}^{ref} \left( c \cos \varphi + \sigma'_{s} \sin \varphi \right)^m
\]

Based on the values listed in Table 1, the parameters for the soil layers are selected. Table 2 presents the parameters used in the numerical model. Four-node quadrilateral solid elements are employed to represent the soil and the beam elements used for the lining. The segmental ring consists of six pieces of reinforced concrete segments.
Fig. 4 represents the two types of segmental rings that are applied in a staggered arrangement to offset the longitudinal joint. The stress-strain relationship for the RC segments and steel is assumed to be linearly elastic. The lining properties are listed in Table 3.

4. Influence distance between tunnels

4.1. Effect of nearby structures

The excavation-induced ground surface settlements of tunnels are affected by the existing nearby structures. Fig. 5 presents the predicted and measured surface settlements after the excavation of the second tunnel for two cases, namely, with and without consideration of the EBT tunnel. Without consideration of the EBT tunnel, the predicted settlements above both tunnels are symmetrical ($S_{max}$ eccentricity is negligible) around the tunnel centerline. The center-to-center distance between the two new tunnels is approximately 4.7 times that of the minor diameters and no interaction is seen between them. With consideration of the EBT tunnel, the value of $S_{max}$ above the second tunnel decreases by 0.32 mm and the eccentricity of $S_{max}$ from the tunnel centerline to the right-hand side of it becomes 1.42 m as the EBT tunnel confines the movement of the soil layers in proximity to the second tunnel. Measured data were obtained from the longitudinal ground settlement observed in Fig. 3 for Section 7k210m (ring 76 in the B line). Relatively small values are recorded after the face pass in the first and second tunnels. The measured settlement after the tail pass in the second tunnel corresponds to the predicted value. The extra settlement occurring a few months after the machine tail passes is not considered in the model; and thus, the final measured values are slightly larger than the predicted ones.

4.2. Influence distance of tunneling

The interaction between tunnels becomes influential if they are located close enough to each other. Fig. 6(a)
presents the predicted horizontal movement of section A–A during the construction steps. Part of the A–A section includes the EBT tunnel wall. After the excavation and lining installation of the first line, the section is drawn toward the center point of the first tunnel, while after the excavation and lining installation of the second line, the section is drawn back toward the center point of the second tunnel. The maximum horizontal movement of the section with a value of 0.8 mm occurs at the bottom of the soft clay layer (Lc). The maximum horizontal deformation of the EBT tunnel wall occurs at the bottom of it. Fig. 6(b) shows the horizontal movement of the A–A section obtained by changing the distance of the pillar width by 0.3Dmin, 1Dmin, 3Dmin, and 5Dmin, in which Dmin represents the minor diameter of the new tunnels. The value of 0.3Dmin represents the measured pillar width in the field. As the width of the pillar increases, the horizontal movement of the section approaches zero. When the second tunnel is far from the EBT tunnel (i.e., 3Dmin and 5Dmin), the EBT tunnel wall displays very small negative horizontal movements under the influence of the first tunnel excavation.

During the sequence excavations, the internal forces of the lining in the existing tunnel can change. From a practical point of view, it might be of interest for engineers to know the extent of the variations in axial force and bending moment by the change in the pillar distance between tunnels. Fig. 7 shows the predicted axial forces and bending moments in the lining of the second tunnel after the lining
installation of the second tunnel by an angle from the tunnel crown in the clockwise direction. In this figure, a lining with six joints is considered to show the internal forces for two cases of pillar widths equal to 0.3D and 5D. The 0.3D case shows an excavation near a relatively rigid EBT tunnel, while the 5D case represents a case without interaction with surrounding structures. The full details of the type and specifications of the joints are given in Section 5. The maximum changes in the amounts of axial force and bending moment in the tunnel lining for the two pillar widths are 80 kN and 10 kN.m, respectively. According to Fig. 7(a), the axial force increases slightly in the lining between joints 3 and 6 (shown by arrows) by the increase in pillar width. The section of the lining between joints 3 and 6 is close to the EBT tunnel, and by eliminating the confining effect of the EBT tunnel in the case of the pillar equal to 5D, the lining bears a somewhat larger axial force in comparison to the pillar case of 0.3D.

Fig. 7. Predicted (a) axial forces and (b) bending moments in the second tunnel after the lining installation of the second tunnel for pillar widths equal to 0.3D and 5D.

Fig. 8 presents changes in $S_{\text{max}}$ eccentricity and volume loss created by the excavation of the second tunnel against the normalized pillar width. The $S_{\text{max}}$ eccentricity starts from 1.42 m for a pillar width of 0.3D (see Fig. 5) and is reduced to zero as the pillar width exceeds 4D. The volume loss starts from $V_L = 0.17\%$ (i.e., $V_L$ after the lining installation of the second tunnel for a pillar width of 0.3D) and reaches the value of volume loss for a single tunnel excavation without any interaction by the increase in pillar width. This is due to the soil around the second lining having more freedom to move toward the center of the tunnel without the EBT tunnel. The influence width of the pillar ($4D_{\text{min}}$) for the elliptical cross-section ($V_L = 0.17\%$) in this study can be compared with the predicted interaction distance of 7D ($D = 4.15$ m and $V_L = 1.4\%$) between two tunnels with a circular cross-section in London clay (Addenbrooke and Potts, 2001) and approximates the predicted interaction distance of 10–12D ($D = 7$ m and $V_L = 2\%$) between two tunnels with a circular cross-section in Singapore Marine Clay (Sweeney, 2006).

*The N and M are read after lining installation of the second tunnel.

Fig. 8. Changes in $S_{\text{max}}$ eccentricity and volume loss in the second tunnel by changing the normalized pillar width.
In the convergence-confinement method, the ground reaction curve (GRC) is an important tool that can be used in the design of tunnels. To further examine the effect of the pillar width, GRC graphs for the two cases of pillar widths (0.3D and 5D) are prepared and compared. According to Eq. (1), the internal support pressure is reduced from an in-situ value of \( \sigma_0 \) to zero in the second tunnel and internal radial displacement at the crown (\( u_{r-c} \)); the induced volume loss (\( V_L \)) and radial support pressure at the crown (\( \sigma_{r-c} \)) are calculated and plotted in each case.

Fig. 9 shows the GRC graph of the second tunnel for the two cases by changing the confinement loss factor from 0.0 to 0.4. In both cases, because of large radial displacement, the analyses do not converge for confinement loss factors of \( \alpha \) greater than 40%. For 0.3D, by relaxing the initial support pressure by 40% from \( \sigma_{0-c} = 163 \) kPa to \( \sigma_{r-c} = 102 \) kPa, the internal radial displacement reaches \( u_{r-c} = 31 \) mm at the crown, which is equivalent to a volume loss of \( V_L = 0.81\% \). For the 5D case, after a 40% reduction in the internal pressure, \( \sigma_{r-c} = 104 \) kPa, \( u_{r-c} = 37 \) mm, and \( V_L = 1.21\% \). The initial stress at the lining crown for the 0.3D case (\( \sigma_{0-c} = 163 \) kPa) is slightly smaller than that for the 5D case (\( \sigma_{0-c} = 169 \) kPa), which is because of the confinement effect of EBT tunnel. The graph shows that for the same amount of stress reduction ratio (e.g. \( \sigma_{r-c}/\sigma_{0-c} \approx 0.63 \)), the 5D case experiences more volume loss and radial displacement than the 0.3D case.

5. Structural behavior of elliptical RC lining

The tunnel segments must bear all of the transitory loads, such as those for the transport, placing, and thrust of the jacks, in addition to permanent loads, including the soil and the water pressure. The amounts and distribution of the loads, the location of the segment joints, and the shape of the lining cross-section alter the distribution of the axial forces and bending moments in the lining. The beam-spring model is implemented for a jointed lining in which each segment is considered to be a curved beam and is modeled by three springs. The nodes of the beam at the location of the joints are linked together by rotational, shear, and axial springs, as illustrated in Fig. 10(b). It is a common, but costly, practice to determine the joint stiffness directly from full-scale laboratory loading tests on a joint-connected segmental lining. The joints used for the segment-to-segment connection are DUET types, as illustrated in Fig. 10(a). Full-scale shear and bending tests on DUET joint-connected segments have already been reported (Oishi et al., 2004; Nakamura et al., 2004). Figs. 11 and 12 present the setup and load-deformation readings of shear and bending tests on DUET joint-connected segments performed by Oishi et al. (2004) and Nakamura et al. (2004). According to Oishi et al. (2004), tensile tests on this type of joint were also performed by applying a load higher than the bolt tensile strength; the value of tensile stiffness shown in Table 3 is adopted from their study. In practice, joints under normal loading conditions are in elastic ranges (Guan et al., 2015). Therefore, linear elastic behavior values are considered for the rotational, shear, and axial springs. The test results from these findings on joint stiffness are used here. Table 3 lists the properties of the segmental linings and the stiffness of the joints.

5.1. Axial forces and bending moments

The structural response of the lining under the loads can be investigated by knowing the distribution of the axial forces and bending moments. The amount of stress reduction ratio, \( \sigma_{r-c}/\sigma_{0-c} \approx 0.63 \), and the volume loss, \( V_L \), are calculated and plotted in each case. Fig. 9 shows the GRC graph of the second tunnel for the two cases by changing the confinement loss factor from 0.0 to 0.4. In both cases, because of large radial displacement, the analyses do not converge for confinement loss factors of \( \alpha \) greater than 40%. For 0.3D, by relaxing the initial support pressure by 40% from \( \sigma_{0-c} = 163 \) kPa to \( \sigma_{r-c} = 102 \) kPa, the internal radial displacement reaches \( u_{r-c} = 31 \) mm at the crown, which is equivalent to a volume loss of \( V_L = 0.81\% \). For the 5D case, after a 40% reduction in the internal pressure, \( \sigma_{r-c} = 104 \) kPa, \( u_{r-c} = 37 \) mm, and \( V_L = 1.21\% \). The initial stress at the lining crown for the 0.3D case (\( \sigma_{0-c} = 163 \) kPa) is slightly smaller than that for the 5D case (\( \sigma_{0-c} = 169 \) kPa), which is because of the confinement effect of EBT tunnel. The graph shows that for the same amount of stress reduction ratio (e.g. \( \sigma_{r-c}/\sigma_{0-c} \approx 0.63 \)), the 5D case experiences more volume loss and radial displacement than the 0.3D case.
forces and bending moments in the lining. Fig. 13 presents the predicted and measured values of the axial forces and bending moments in the lining of the B line. In Fig. 13, the predicted values are obtained after the application of the grouting by considering the orientation of the joints, as illustrated in Fig. 10(b). As noted in Section 4.2, Fig. 7 also shows the predicted internal forces in the B line without considering the grouting pressure and therefore, the predicted values for the internal forces shown in Fig. 7 are somewhat smaller than the predicted results shown in Fig. 13. The measured lining forces are calculated through direct strain measurements of the outer and inner steel bars.
in the tunnel lining. Firstly, the corresponding values for the stress in the concrete and the steel bars are calculated and then the bending moments and axial forces are obtained. The measured values are read at three different times, namely, after the machine tail passes through the section, after grouting, and after finishing ring 87 in the B line (7k226m).

This is the closest ring to the studied section (7k210m) with the available measured values for the axial force and bending moment data. Based on Fig. 13(a), the entire section is under compressive force with values ranging from 243 to 1545 kN. Similarly, Fig. 13(b) shows that the bending moments alternate between positive and negative values in the lining. The maximum positive and negative predicted and measured values for the bending moments occur in segments C and A (see Fig. 10(a)), respectively. The positive bending moments deform the segment inward and the maximum value is at the mid-span of segment C, which is close to the lining spring line.

5.2. Actual and permissible crack widths

Structural cracks can occur during the lining installation (e.g., machine jack forces); however, it is the final soil-water pressure following the lining installation that represents the dominant load and which normally causes cracking in a tunnel. Herein, the measurement data for the tunnel present several longitudinal cracks along the tunnel alignment. Fig. 14 shows a crack map of the segments in rings 86, 87, and 88 of the B line. The widths of the cracks were recorded manually for all the rings after finishing the B line. Most cracks were observed in the longitudinal direction and at the mid-span of segment C with widths of less than 0.2 mm. In Fig. 14, the crack width is limited to 0.1 mm. Large values for the bending moments are generated in the time period after the grouting application until the final measurement, after finishing the excavation of the entire line. At the time of the final measurement, the filling material in the gap between the soil and the lining is believed to have reached its final strength and the lining bears the final overburden load. Based on the JSCE Concrete Committee (2007) and the JSCE Tunnel Engineering Committee (2007), the permissible crack width \(w_a\) under normal environmental conditions is \(w_a = 0.005c_r\), where \(c_r\) is the concrete cover. The concrete cover of the segment in both the outer and inner steel bars is \(c_r = 65\) mm; and therefore, the crack width is limited to \(w_a = 0.325\) mm. The maximum recorded crack width (=0.2 mm) is below the allowable value. The JSCE Tunnel Engineering Committee (2007) also suggests following equations for the width and spacing of cracks mainly based on the concrete cover, the distance between the reinforcement bars,
and J2 (segment C) and at the outer reinforcement layer strain at the inner reinforcement layer between joints J1 bars of ring 87 for the B line. The values for the tensile sent the measured axial strains at the inner and outer steel l1 the diameter of the rebar, the steel tensile stress, and the shrinkage of concrete, l1 are as follows:

\[ l_1 = 1.1 \times k_1, k_2, k_3 \{4c_s + 0.7(c_s - \phi_s)\}, k_2 \]

where \( w \) is the maximum spacing of the rebar, \( \sigma_s \) is the rebar tensile stress, \( E_s \) is the steel's Young's modulus, \( e'_{\text{cad}} \) is the compressive strain due to the shrinkage and creep of concrete, \( l_1 \) is the spacing of the cracks, \( k_1 \) is a factor for considering the effect of the reinforcement surface on the crack width, \( k_2 \) is a constant that takes into account the effect of the concrete quality, \( k_3 \) is a constant defining the effect of multiple layers of tensile reinforcement, \( c_s \) is the concrete cover, \( c_s \) is the center-to-center distance of the tensile reinforcement, \( \phi_s \) is the rebar diameter, \( f'_c \) is the compressive strength of concrete, and \( n \) is the number of layers of tensile reinforcement. The values of the parameters for the examination of the crack width are as follows: \( f'_c = 60 \text{ N/mm}^2 \), \( n = 1 \), \( k_1 = 1 \), \( k_2 = 0.89 \), \( k_3 = 1 \), \( c_s = 65 \text{ mm} \), \( c_s = 150 \text{ mm} \), \( \phi_s = 19 \text{ mm} \) for D19 rebar, \( l_1 = 343 \text{ mm} \), \( E_s = 210 \text{ GPa} \), and \( e'_{\text{cad}} = 150 \times 10^{-6} \), \( l = 250 \text{ mm} \).

The stress in the reinforcement (\( \sigma_s \)) can be obtained from direct measurements of the rebar strain. At this site, the strains of the rebar were measured in the outer and inner layers of the reinforcements of the segment at intervals of 18° (i.e., at 0°, 18°, 36°, ..., and 342°). Fig. 15 presents the measured axial strains at the inner and outer steel bars of ring 87 for the B line. The values for the tensile strain at the inner reinforcement layer between joints J1 and J2 (segment C) and at the outer reinforcement layer between joints J3 and J4 (segment A) are noticeable. The maximum values are recorded during the final measurement. The maximum recorded tensile strain is \( \varepsilon_t = 114 \times 10^{-6} \) at segment C, which is equivalent to \( \sigma_t = 23.9 \text{ N/mm}^2 \). Using the values of the measured rebar tensile stress and the previously obtained parameters, the crack width becomes \( w = 0.066 \text{ mm} \), which is similar to the values shown on the crack map in Fig. 14.

5.3. Segment capacity based on the allowable cracks

Cracks may become wider under a larger or new combination of flexural and axial loads. A common way to investigate the segment capacity under the axial loads and bending moments (P, M) is to use an interaction diagram. The lining section reaches a yielding condition if the (P, M) point lies outside of the interaction diagram. Section failure is not a common concern in tunnels under normal loading conditions. However, crack development larger than the allowable values should be prevented, and it is safe to specify a zone of axial forces and bending moments in which cracks are not triggered or they are limited to the allowable values. Fig. 16 illustrates segment C, which has two layers of steel bars (each layer with 10-D19 and 2-D13) at the top and bottom of the section. To plot the interaction diagram for a rectangular RC segment, basic assumptions, constitutive material relationships, design strengths, and reduction factors defined according to the JSCE Concrete Committee (2007), JSCE Tunnel Engineering Committee (2007), and ACI 318 (ACI 2014) are used. It is assumed that the plane section remains plane during bending and that the strain distribution across the depth of the segment is linear. The model proposed by the JSCE Tunnel Engineering Committee (2007) is used for concrete stress-strain constitutive law in compression, and the tensile strength is ignored. Fig. 17(a) presents the stress-strain curve, while Eqs. (7) and (8) show the formulations of the model:

\[
\sigma'_{\text{cu}} = k_1 f'_{\text{cd}} \times \frac{\varepsilon'_{\text{cu}}}{0.002} \times \left( 2 - \frac{\varepsilon'_{\text{cd}}}{0.002} \right)
\]

\[
\varepsilon'_{\text{cu}} = \frac{155 - f'_{\text{ck}}}{30000}, \quad 0.0025 \leq \varepsilon'_{\text{cu}} \leq 0.0035, \quad \text{(Unit for } f'_{\text{ck}} \text{ is MPa.)}
\]

where \( k_1 \) is a strength reduction factor, defined by \( k_1 = 1 - 0.003f'_{\text{ck}} \leq 0.85, f'_{\text{ck}} \) is the compressive strength of concrete (=60 MPa), \( f'_{\text{cd}} \) is the design compressive...
Fig. 15. Measured axial strains at the (a) inner and (b) outer steel bars of ring 87 on the B line.

Fig. 16. Detailed illustration of segment C.
strength of concrete \( f'_{cd} = \frac{f_{cd}}{1.5} \), and \( e'_c \) is the ultimate compressive strain of concrete.

The perfect elastic plastic model is also employed for steel using the yield strain \( (e_y = f_{ys}/E_y) \) in compression and tension \( (f_{ys} = 410 \text{ MPa} \text{ and } E_y = 210 \text{ GPa}) \). In the case of the ultimate capacity, except when the entire cross-section is in compression, the compressive stress distribution of concrete may be assumed as the so-called equivalent stress block (JSCE Tunnel Engineering Committee, 2007). Fig. 17(b) presents the assumed strain distribution and stress block according to the JSCE Concrete Committee (2007). The height of the stress block is \( a = \beta c \), where \( c \) is the depth of the neutral axis, and \( \beta = 0.52 + 80e_y \). Different modes of failure including pure compression, compression-controlled, tension-controlled, pure bending capacity, and pure axial-tension capacity are considered. The formulation details of these modes are given in Appendix A. Fig. 18 represents the interaction diagram of \( P_n - M_n \) and \( \phi P - \phi M \) with all the modes of failure. The diagram also covers the tensile axial capacity of the section. Strength reduction factor \( \phi \) is also used to plot the \( \phi P - \phi M \) diagram as follows:

\[
\begin{align*}
\phi &= 0.65, \\
\phi &= 0.65 + 0.25 \frac{e_y}{e_y} < 0.005, \\
\phi &= 0.90, \\
\phi &> 0.005
\end{align*}
\]  

where \( e_y \) and \( e_y \) are the steel tensile and yield strains, respectively. The intersection of the dashed line with the diagram represents the zero strain mode in the intrados steel bar (Fig. 17(b)). In this mode, ignoring the concrete tensile strength, a crack has already developed on the intrados side of the section.

The axial strain of concrete, related to the allowable opening of the first crack in tunnels under a normal environmental condition \( (w_a = 0.325 \text{ mm}) \), is \( \varepsilon'_c = w_a / L = 87.2 \mu \), in which \( L \) is the arc length of the segment \( (L = 3726 \text{ mm} \text{ according to Fig. 16}) \). Fig. 19 represents the calculated \( \phi P - \phi M \) diagram. The status corresponding to the onset of the first cracks with allowable widths and the values of the measured bending moments and axial forces after the application of the grouting in all segments of ring 87 for the B line (Fig. 13) are plotted in Fig. 19. In practice, the occurrence of multiple cracks with openings not larger than the permissible crack width along a single segment might also be allowed. The spacing between these cracks can be calculated using the recommendation of construction codes such as CEB-FIP (1978) and Euro code 2 (2004). The CEB-FIP Code (1978) proposes the average crack spacing, \( S_m \) (mm), as follows:

\[
S_m = 2(c_r + 0.1c_s) + k_1k_2\phi_s/\rho_{s,eff}
\]  

where \( c_r \) is the concrete cover, \( c_s \) is the center-to-center distance of the tensile reinforcement, \( k_1 \) is a coefficient which takes account of the bond properties of the reinforcement (0.4 for deformed bars and 0.8 for plain bars), \( k_2 \) is a coefficient which takes account of the strain gradient \( (k_2 = 0.25) \), \( \phi_s \) is the bar diameter, and \( \rho_{s,eff} \) is the effective reinforcement ratio, \( \rho_{s,eff} = A_f/A_{s,eff} \). The \( A_{s,eff} \) is the area of concrete around the reinforcing bar at the distance of 7.5 bar diameter. The values of the parameters are \( c_r = 65 \text{ mm} \) \( c_s = 150 \text{ mm} \) \( k_1 = 0.4 \) \( k_2 = 0.25 \) \( \phi_s = 19 \text{ mm} \) and \( \rho_{s,eff} = 0.0179 \). Using Eq. (10), the average crack spacing becomes \( S_m = 266 \text{ mm} \) and the number of cracks along the length of the segment, \( n \), is obtained as \( n = L/S_m = 14.0 \).
in which \( L \) is the arc length of the segment \((L = 3726\, \text{mm})\). The axial strain of concrete on the inner side of the segment, corresponding to the allowable opening \( w_a \) of the multiple cracks \((n = 14.0)\), is \( \varepsilon_c = (n \times w_a)/L = 1221 \, \mu \). The line AB and the enclosed gray zone in Fig. 19 show the segment mode and the safe zone, respectively, corresponding to the onset of several cracks with allowable widths and average crack spacing. In the field, there might be other cases of segments with larger crack numbers and smaller crack widths than the ones shown here. By using the procedure given here, the axial strain of concrete and the related safe zone can be predicted before the construction. According to Fig. 19, the measured data, including the data for segment C, lie closely on the AB line. The methodology presented here was obtained by assuming identical crack widths and an equal spacing between the cracks; however, the spacing and the widths of the cracks are usually not uniform in the field and this might be the reason for the slight deviation of the measurement data from the shown zone.

6. Conclusions

In this study, the ground response around an existing tunnel and the structural behavior of the segmental lining during the excavation of two tunnels with elliptical-shaped cross-sections in coarse-grain soils were examined using measurement data and a numerical investigation. The effect of nearby structures and the influence distance for nearby excavations were discussed in terms of the changes in volume loss around the tunnel, the horizontal soil deformation, and the distribution of internal forces. The structural response of the lining was investigated by plotting the capacity diagram of the RC section based on the combined axial loads and bending moments and considering the crack development in the segments with elliptical shapes. Lastly, a safe zone with the combined loads of axial forces and bending moments, corresponding to the onset of several cracks with allowable widths and average crack spacing, was recommended. The main findings of the study are as follows:

1- The shape of a tunnel's cross-section affects the magnitude and distribution of the induced ground settlement trough, stress-strain regime of the soil, and influence distance of the tunneling. The existence of structures in close proximity to new excavations confines ground movement and prevents the symmetrical formation of a ground trough above the tunnels. By changing the maximum settlement eccentricity and volume loss with the pillar width, between the new and existing tunnels, it was demonstrated that the influence of close tunneling in the case of an elliptical cross-section is approximately \( 4D_{\text{min}} \), where \( D_{\text{min}} \) is the tunnel's minor diameter. By examining the effect of the pillar distance between two tunnels (by considering two cases of \( 0.3D \) and \( 5D \), where \( D \) is the minor diameter) on the internal forces of the lining, it was found that the axial forces increase slightly in the part of the lining close to the existing structure by increasing the pillar width. The changes in the values of the bending moments were small when increasing the pillar width. The ground reaction curves (GRCs) for the above-mentioned two cases demonstrated that for the same amount of stress reduction ratio, the tunnel case without any interaction (\( 5D \) case) experienced more volume loss and radial displacement in comparison with the \( 0.3D \) case. More detailed ground instrumentation of closely spaced tunnels with elliptical-shaped cross-sections are needed to capture the ground response in both longitudinal and transverse directions.
2- The measured and predicted values for the axial force and bending moment distributions in the elliptical lining helped to demonstrate that a segment under construction loads is in compressive axial force with the maximum bending moment at the mid-span of the largest segment. The measured crack map for elliptical segments demonstrated that the majority of cracks occur in the mid-span of the segment, near the tunnel spring line. The measured values of strain in the inner and outer layers of the steel bar helped to calculate the crack width. The calculated crack widths were close to the measured values of the crack map in the segment. It was found that crack generation in an important issue in an elliptical section lining; and therefore, a safe zone corresponding to the onset of several cracks with allowable widths and average crack spacing was recommended. By using the procedure in this study, the axial strain of concrete and a corresponding safe zone, in terms of cracks, can be predicted before the construction. The methodology presented here was obtained by assuming identical crack widths and equal spacing between the cracks; however, the spacing and the widths of cracks are usually not uniform in the field and this should also be considered during the design stage.

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Appendix A. Axial force-bending moment capacity of the section

According to Fig. 17(b), the strain distribution of the section is defined by the setting values, \( \varepsilon_{cu} \) and \( \varepsilon_{st} = Z \varepsilon_{sy} \), where \( Z \) is an arbitrary chosen value and \( \varepsilon_{sy} \) is the steel yield strain. The diagram is drawn by controlling the values of the neutral axis depth of \( c \) as follows:

\[
\varepsilon_{cu} = \frac{155 - f'_{ck}}{30000}, \quad 0.0025 \leq \varepsilon_{cu} \leq 0.0035, \quad (\text{Unit for } f'_{ck} \text{ is MPa})
\]

\[
c = \left( \frac{\varepsilon_{cu} - Z \varepsilon_{sy}}{\varepsilon_{cu}} \right) d_i
\]

\[
\varepsilon_{st} = \left( \frac{c - d_i}{c} \right) \varepsilon_{cu}
\]

where \( \varepsilon_{cu} \) is the ultimate compressive strain of concrete, \( f'_{ck} \) is the compressive strength of concrete (=60 MPa), and \( \varepsilon_{sy} \) and \( d_i \) are the strain in the \( i \)th layer of steel and the depth of that layer, respectively. The stress in the steel bar is as follows:

\[
\sigma_{si} = \varepsilon_{si} E_s, \quad f_{sy} \leq \sigma_{si} \leq f_{sy}
\]

where \( f_{sy} \) is the steel yield strength. The axial force in the steel bar is as follows:

\[
F_{si} = \sigma_{si} A_{si}, \quad \text{if } a < d_i
\]

\[
F_{si} = (\sigma_{si} - 0.85f'_{ck}) A_{si}, \quad \text{if } a > d_i
\]

where \( a \) is the height of the stress block, as shown in Fig. 17 (b). The nominal axial load capacity, \( P_n \), for a strain distribution is obtained by the summation of the axial forces and the nominal moment capacity, and \( M_n \), is calculated by the summation of the moment of all of the forces at the mid-height of the section as follows:

\[
P_n = C_c + \sum_{i=1}^{n} F_{si}
\]

\[
M_n = C_c (\frac{h}{2} - \frac{d}{2}) + \sum_{i=1}^{n} F_{si} (\frac{h}{2} - d_i)
\]

\[
C_c = (k_j f'_{cd})(ab)
\]

where \( C_c \) is the compressive force, \( k_j \) is a strength reduction factor, defined by \( k_j = 1 - 0.003f'_{ck} \leq 0.85, f'_{cd} \) is the design compressive strength of concrete \( (f'_{cd} = f'_{ck} = 60 \text{ MPa}) \), and \( ab \) represents the area of concrete under compression. The capacity of the section under different modes of failure is obtained as follows:

**Pure axial load:**

The nominal axial load, design axial load, and maximum allowable axial load are obtained as follows (ACI 318, 2014):

Nominal axial load:

\[
P_{no} = 0.85f'_{ck}(A_g - A_{st}) + f_{sy}A_{st}
\]

where \( A_g \) is the gross area \( (b \times h) \) and \( A_{st} \) is the cross-section area of the longitudinal steel bar.

Design axial load:

\[
\Phi P_{no} = \Phi \left[ 0.85f'_{ck}(A_g - A_{st}) + f_{sy}A_{st} \right], \Phi = 0.65
\]

Maximum allowable axial load:

\[
\Phi P_{n,\text{max}} = 0.8\Phi \left[ 0.85f'_{ck}(A_g - A_{st}) + f_{sy}A_{st} \right]
\]

where 0.8 represents the effect of an accidental moment. Zero tension in the steel bar:

\[
\varepsilon_{st} = 0.0, (Z = 0.0)
\]

**Balanced failure condition:**

\[
\varepsilon_{st} = -\varepsilon_{sy}, (Z = -1)
\]

**Pure bending capacity:** (ACI 318, 2014)

The maximum allowable bending moment is as follows:

\[
\Phi M_{n,\text{max}} = \Phi A_{st} (d - a_s/2), \Phi = 0.9
\]

\[
a_s = A_{st} (f_{sy}/0.85f'_{ck})(\text{mm})
\]
Pure axial-tension capacity:
The design axial-tension load is as follows:

$$\Phi P_{nt} = \Phi \sum_{i=1}^{n} -f_{sy} A_{nt}, \Phi = 0.9$$

References


