Tunneling Excavation Induced Settlements in Granular Soils: Case of Keçiören Subway (Ankara)

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Abstract

The need for underground transportation is increasing to reduce traffic within metropolitan cities. During excavation, ground loss within the soft soils and associated ground settlements might occur. Tunneling induced ground settlement has to be within tolerable limits. Otherwise, damage to ground level and infrastructures are experienced. In this study, reasons of ground settlement occurred during the excavation of twin subway tunnels at Keçiören (Ankara) are evaluated. In 2006, the first phase excavation caused ground settlement with no considerable damage. The second phase excavation started in 2012, both within the saturated sandy and gravelly soils of Çubuk Creek’s alluvium. At the time, several collapses and associated ground settlements were reported. Due, an investigation program started including resistivity surveying, borehole drilling and in-situ testing. Since ground settlements were known, back calculation was performed to obtain the settlement trough, inflection point and the change of contraction increment. Moreover, the stress conditions and external stresses exerted by existing structures are reviewed. Collapses with volume up to 80 m$^3$ were observed nearby the buildings and free field during the second phase excavation which were determined through 2D resistivity data and boreholes. It was concluded that, the change of settlement at ground level and on the tunnel segments are related to contraction increment based on the numerical modelling of the twin tunnels. Possible reasons for collapse can be listed as TBM cutting head and shield defect, change in ground structure due to underground water level changes, and the impact of the previously excavated tunnel in 2006.

Introduction

The rapid growth of residential areas around densely populated cities brings out a need to construct underground transportation. In some cases tunnels have to be excavated beneath city centers and nearby surface structures. In order to prevent surface damage and maintain structural serviceability, determination of the surface settlement is crucial. The main issue encountered during tunnel excavation is the surface settlement related to geological material, groundwater level and inappropriate excavation method.

Prediction of the tunneling induced ground settlement prior to excavation is the key factor to outline the possible conflicts with the existing ground level facilities. This prediction must be performed considering the changes in the soil strata and groundwater level, combined with several methods. Also if ground settlement is assumed, necessary precautions should be planned accordingly. Such considerations should be applied to twin subway tunnels, which might increase the ground settlement and structural damage when compared to single tunnels.

A huge amount of research has been conducted about the settlement caused by tunneling in soft or weak soil conditions. Litwinniszyn (1956) used a stochastic ground model, composed of equal sized spheres. When one sphere is removed, the movement of the remaining spheres were displaced in the form of a Gaussian curve and the settlement trough of the model was introduced in normal probability form. Peck (1969) simplified the solution of Litwinniszyn (1956) and introduced the settlement estimations to be
expected at varying distances laterally from the centerline of a tunnel (Suwansawat 2002). Peck (1969) allowed to determine the displacement at any point along the normal probability curve, which represents the settlement curve. Cording and Hansmire (1975) stated that, the volume of settlement trough can be related to ground loss into a tunnel (Suwansawat 2002). In case the volume of ground loss is known, the displacement could be determined as offered by Peck (1969). O’Reilly and New (1982) proposed a new formulation to predict the settlement trough based on several case studies. They assumed that, plane strain deformations are associated with the radial movements in the soil strata. Attewell and Woodman (1982) took the ground settlements along longitudinal direction into account for shield tunneling. Analytical solutions were also adopted to determine the surface settlement (Verrujit and Booker 1996; Sagaseta 1987; Loganathan and Poulos 1998; Rowe and Lee 1983; Rowe et al. 1983; Gonzalez and Sagaseta 2001; Lee et al. 1992; Verrujit 1997; Pinto 1999; Mair 1996; Clough and Schmidt 1981; Taylor 1984; Cording 1991; Mair et al. 1993). Also the grounds settlements in soft soils were investigated for twin tunnel excavations (Topal and Mahmutoglu 2021; Mahmutoglu 2011; Leca and New 2007; Zhu et al. 2003; Addenbrooke and Potts 2001; Cooper and Chapman 1998; Mair et al. 1996). Suwansawat (2002) and Lee at al. (1992) extensively reviewed the above mentioned static methods.

Addition of new case studies and tunnel excavation technology, 2D and 3D finite element analyses, numerical methods are also integrated to tunneling work with respect to determine settlement (Shi et al. 2015; Coffman et al. 2014; Garner and Coffman 2013; Ercelebi et al., 2011; Suwansawat and Einstein 2006; Bloodworth 2002; Augarde and Burd 2001; Augarde 1997; Addenbrooke 1996; El Nahhas et al. 1992; Lee and Rowe 1990; Finno and Clough 1985). Selby (1999) compared the in-situ and numerically determined settlements. He concluded that, the trough as obtained by numerical studies is close to the in-situ trough. 3D finite element analysis of an EPB shield tunnel was performed in order to determine the ground disturbance caused by shield tunneling (Akagi and Komiya 1996; Suwansawat 2002). Eberhardt (2001) also adopted a numerical work to simulate the tunneling advance and come up with the ground disturbance. Moreover, centrifuge models (Atkinson and Potts 1977) and 2D models made up of Perspex window and clay were used to simulate the ground settlements (Kimura and Mair 1981). A scaled miniature shield tunneling machine was also developed to model the shield tunneling and occurrence of surface settlement (Nomoto et al. 1999).

The twin subway tunnels of Keçiören metro were excavated in saturated sandy and gravelly alluvial soils by a single shield earth pressure balanced (EPB) type tunnel boring machine (TBM). Surface settlements up to 33 mm were recorded during the first phase excavation in 2006. In 2012, the second phase started and after the first collapse was reported nearby the Nail Oraman building, a shaft was excavated to save the TBM cutting head. The excavation was suspended during the maintenance of the cutting head and rapid drawdown of groundwater took place resulting with change of stress conditions. Meanwhile, soil collapse related surface settlements were observed nearby the buildings. Also settlement induced cracks and dilations happened within the buildings. This study aims to investigate the possible reasons of tunneling induced settlements and damage to infrastructures by numerical analyses. The maximum surface settlement providing contraction increment (CI) is evaluated taking the geotechnical parameters and surface loading into account. The CI is maximum 1.5% for ordinary tunneling applications, however
CI is related to ground loss and reaches up to 16% which corresponds to monitored excessive surface settlements.

The Project Area

The area covers the section of Tandoğan-Keçiören subway between Km: 3 + 280 to Km: 3 + 580 (Fig. 1). The subway is 10.58 km long with 11 stations. First tunnel of this section located within the Ankara University, Faculty of Agriculture. Surface settlements and damage were observed within the infrastructures. The whole alignment is excavated by an earth pressure balance (EPB) type tunnel boring machine (TBM) which aims to counterbalance the surcharge and hydrostatic pressure during the excavation. New Austrian Tunneling Method (NATM) and related support systems were applied to whole tunnel alignment. The section was excavated within the Quaternary aged alluvial soils of Çubuk Creek, mainly composed of silty sand and gravel (Kılıç et al. 2013).

The project aims to link the town of Keçiören to central Ankara and reduce the traffic. The depth of the tunnel was 13.5 m with an average slope of 3.5 %. Projected tunnel outer diameter was 6.50 m and shield outer diameter was 6.45 m. The TBM has an average advance of 15 m/day with an expected maximum of 33.6 m/day. The technical details of the TBM are represented in Table 1 (Sojoudi 2015).

| Segment length | 1.400 mm |
|----------------|---------|
| Segment number | 5 + 1   |
| Maximum working pressure | 3 bar |
| Complete length | 70 m |
| Maximum advance rate | 80 mm/min |
| Total shield length | 7.460 mm |
| Shield outer diameter | 5.950 mm |
| Cutting wheel nominal diameter | 5.980 mm |
| Cutting head power | 30 kW |
| Erector type | Hydraulic, centre free |
| Screw conveyor capacity | 275 m³/h |
| Center shield outer diameter | 5.94 mm |
| Front shield outer diameter | 5.95 mm |
Field Investigations

After the surface settlements and damage to buildings took place, a field investigation program was planned in order to identify the possible reasons. The settlement induced cracks and dilations took place within the Nail Oraman and Food Engineering buildings of Ankara University. The Soil Science building did experience minor settlements within permissible limits (25 mm). The investigations included multi-channel analyses of surface waves (MASW), resistivity surveying, borehole drillings, in-situ tests and laboratory testing of the samples. A total of 10 boreholes were drilled along the section where settlements occurred with depths ranging between 10 m to 13 m. A single borehole was drilled away from the tunnel alignment to depict the soil stratum with a depth of 25 m (Fig. 2). The P and S-wave velocities of the soils were determined through MASW and also the collapses within the soil strata were detected by resistivity surveying.

Standard penetration tests (ASTM D1586 1999) were performed in order to determine the stiffness of the soils, to outline relative density and obtain samples. The soils were classified as silty sand (SM) with some well graded sands (SW) and well-graded silty gravel (GW-GM) based on ASTM D2487-00 (2000) (Table 2). The SM group soils are dense and very dense (ASTM D1586 1999), however the SPTN values indicate very loose-loose soils at some levels which are considered to be close to the collapses. Menard pressuremeter (PMT) tests (ASTM D4719, 1995) were conducted following the SPT data as to obtain the elastic modulus (E) of the soils around the tunnel. The coefficient of permeability of the soil samples were determined by constant head permeameter (ASTM D2434-68 2000). The cohesion of the silty sand is 1 kPa and the internal friction angle is extracted from SPT blow count and previous geotechnical data. Moreover, the Poisson’s ratio of the sands was calculated using the ratio of in-situ P-wave and S-wave velocities (Kılıç et al. 2013). Table 2 introduces the geotechnical parameters of the soils around twin tunnels.

| Table 2 Input geotechnical parameters for numerical modelling of Nail Oraman building |
|---------------------------------------------|-------------------------------------|----------------|----------------|-------|-------|
| Depth, m | Soil | $\gamma_0$ (kN/m$^3$) | $\gamma_{sat}$ (kN/m$^3$) | $k$ (cm/s) | $E$ (MPa) | $v$ | $\phi(\degree)$ |
| 0-4       | SM   | 17                          | 19.5            | 4.05x10$^4$  | 11     | 0.35 | 30         |
| 4-6.5     | SW   | 16                          | 18              | 4.97x10$^4$  | 13.2   | 0.30 | 32         |
| 6.5-10    | SM   | 16                          | 18.5            | 5.56x10$^4$  | 18.1   | 0.30 | 31         |
| 10-16     | SW   | 18                          | 20.5            | 1.15x10$^4$  | 27     | 0.35 | 35         |
| 16-21     | SW   | 19                          | 21              | 1.15x10$^4$  | 28     | 0.35 | 35         |

In order to evaluate pore water pressure and stress conditions along the tunnel alignment, periodic groundwater level observations were implemented (Fig. 3). Before the drillings, the mean groundwater level was reported as 4.5 m. During the measurement period, the excavation of the tunnel was suspended and the groundwater level reduced about 2 m, especially in the boreholes where settlement took place.

During the progress and the second tunnel excavation, groundwater level has drawn down to 6.5 m and last record was 8.5 m when the settlements were observed. The effective stress has changed from 164
kPa to 218 kPa during the course, taking the vertical downward seepage into account. The EPB could counterbalance the pore water pressure until the excavation had to be suspended.

**Settlement Induced Damage**

Surface settlements were measured during the excavation of the first tunnel at the Nail Oraman building. The maximum settlement was reported as 33 mm (Gülermak-Kolin 2012). As the second tunnel excavation proceeded, the progress was aborted due to the wear of TBM cutting head. A shaft with 8 m x 2 m dimensions and 16.5 m depth was excavated to maintain the cutting head. Following this, a collapse on the surface level took place at the northwest of Nail Oraman (NO) building due to the rapid drawdown of groundwater level since the EPB was not performing. The shaft was filled with cement and after the replacement of the cutting head (Fig. 4), the excavation resumed within 2 months towards Food Engineering building (FE).

The Food Engineering building is located at 25 m north of Nail Oraman (NO) building. Also surface settlements were observed with significant structural damage within the Food Engineering building (Fig. 5). Also, minor heave nearby the building was observed due to settlement.

**Numerical Modelling**

Numerical modelling of the tunnel excavations leads to evaluate the settlements and verify the real time monitoring data. The use of finite element method (FEM) is the most popular one in order to adopt the site specific factors which affect the stability and performance of the tunnels. A large amount of studies exist using the 2D and 3D FEM (Ercelebi et al. 2011; Mahmutoğlu 2011; Garner and Coffman 2013; Wang et al. 2019; Shong-Loong and Po-Chia 2018; Topal and Mahmutoğlu 2021). The numerical modelling also allows to back calculate the subsurface and surface damage during the underground excavations.

In order to depict the surface settlements, a numerical modelling was performed using the Plaxis 2D software. It is possible to choose the β-method or contraction of tunnel lining to perform the analysis to simulate the ground volume loss due to the excavation. The contraction increment (CI%) is an input of the software, which is defined as the ratio of the area reduction and the original outer tunnel cross-section area (Plaxis 2002). Such parameter is suggested to be between 0.3 % to 0.6 %, with a maximum value of 1% (Gatti and Cassani 2007). Coffman et al. (2014) extensively discussed the relation between the CI and ground loss (g%) using data from several projects. They introduced an exponential relation between g and CI parameters, stating that CI might exceed the offered values depending on local site conditions.

Input geotechnical parameters (Table 2) and the parameters related to the segments and external load (Table 3) were used to construct the input model (Fig. 6) for Nail Oraman building.
Table 4 Input geotechnical parameters for numerical modelling of FE building

| Depth (m) | Soil | \(\gamma_n\) (kN/m\(^3\)) | \(\gamma_{sat}\) (kN/m\(^3\)) | k (cm/s) | E (MPa) | \(\nu\) | \(\phi^\circ\) |
|----------|------|--------------------------|--------------------------|---------|--------|------|----------|
| 0-4      | SM   | 17                       | 19.5                     | 1.15x10^-4 | 11     | 0.30 | 30       |
| 4-13     | SW   | 18                       | 20.5                     | 5.05x10^-4 | 27     | 0.30 | 32       |
| 13-25    | GW-GM| 20                       | 22                       | 6.3x10^-3  | 30     | 0.25 | 40       |

Several formulations have been developed for the prediction of settlement due to single and twin tunnel excavations, mostly based on Peck (1969)'s state of the art work. O'Reilly and New (1982) have proposed Eq. (1) for the prediction settlement based on twin tunnel excavation.

\[
S = S_{maxA} \exp \left[ \frac{-(x+i)^2}{2i^2} \right] + S_{maxB} \exp \left[ \frac{-(x-i)^2}{2i^2} \right] \tag{1}
\]

Where; \(S_{maxA,B}\) is the maximum settlement for each tunnel, \(L\) is the distance between two tunnels, \(x\) is horizontal distance from tunnel center and \(i\) is the inflection point of the settlement curve (O'Reilly and New, 1982) which depends on the tunnel depth (\(Z\)) and dimensionless parameter (\(K\)) (Eq. 2).

\[
i = K.Z \tag{2}
\]

The \(K\) parameter has been investigated by several researchers and extensively outlined by Topal and Mahmutoğlu (2021) based on varying soil conditions. Eq. (3) is adopted for this study as offered by O'Reilly and New (1982) for cohesionless soils.

\[
i = 0.28Z - 0.12 \text{ (i is in meters)} \tag{3}
\]
The settlement trough of the twin tunnels below Nail Oraman and Food Engineering buildings were constructed separately using the monitoring data. There is 15 m horizontal distance between the first and second tunnels both at the same elevation. The settlement monitoring data indicated a maximum of 33 mm for the Nail Oraman building (Fig. 8) during the first phase excavation. However, collapses nearby the building indicate larger settlement values on the second tunnel in 2012.

The contraction increment is related to the ground loss and might reach up to 16% which correspond to a ground loss of 4.82% (Coffman et al. 2014). Based on this, the FEM model was run to evaluate the effect of CI on settlement values. When CI = 1%, the surface settlement is 27.35 mm and the settlement on the segment’s outer diameter is 23.19 mm (Fig. 9), which indicates a 1.87% of ground loss (Eq. 4).

\[ g\% = \frac{4 \cdot (2\pi)^{0.5} \cdot S_{\text{max}}^{1.100\%}}{D^2\pi} \tag{4} \]

Where, \( S_{\text{max}} \) is the maximum settlement, \( i \) is the inflection point, \( D \) is the tunnel diameter (Coffman et al. 2014; Peck 1969).

Following the first collapse (Fig. 5) the shaft excavation and the suspended excavation, the measured surface settlements reached up to 225 mm nearby the Nail Oraman building. The FEM model was calibrated based on the CI to achieve the measured settlement values. The surface settlement is 273.85 mm and the settlement on the segment’s outer diameter is 336.232 mm which corresponds to 2.58% ground loss. (Fig. 10).

When compared to Fig. 9, as the CI is increased, the shape of the settlement trough is different. It was observed that, the settlement is a fixed value even the CI is increased. Since the effective stress above the segments are higher due to the groundwater drawdown, the soil skeleton also experiences a compression. The form of settlement trough is reversed because of the ground loss should be balanced from the segments towards the soil skeleton. The settlement trough before the excessive settlements and damage to surface structures took place is represented (Fig. 11) for the twin tunnels below Food Engineering building.

The surface settlement is 51.02 mm and the settlement on the segment’s outer diameter is 36.83 mm when CI = 1% which indicates a 1.98% of ground loss (Fig. 12) as extracted from the FEM model output.

The groundwater level is about 2 m deeper when compared to Nail Oraman building and more permeable gravelly soils around the tunnel segments might cause the groundwater flow toward the tunnel more rapid. Also, the external load is much higher for Food Engineering building. In order to attain the actual settlements after the damage was observed, the CI is increased up to 16%, which corresponds to surface settlement of 361.21 mm and 418.08 mm above the segments (Fig. 13).

For both cases, as the CI is increased the shape of the trough indicates that the settlement values are due to the excavation of second tunnel as a result of the cavitation already occurred during the first tunnel excavation combined with the change of stress conditions. The settlement on the segments and ground
level are not the same since the soil type, groundwater level and compressibility of the surrounding soils vary. An attempt to depict the relation between the settlement on the segments $S_s$ and ground level $S_g$ based on changing CI is represented (Fig. 14a, b). As the CI increases, the ratio of $S_s/S_g$ decreases exponentially. Based on the stress level, the settlement on the segments are relatively smaller and after reaching a specific CI value, the maximum settlements on the segments and ground level are obtained. One could predict the settlement on the segments or ground level in case one of each is known.

**Conclusions**

Tunnelling induced settlements in soft ground should be evaluated taking many aspects into account. Most of the empirical relations and parameters are site and tunnel conditions dependent. Therefore, the in-situ geological and geotechnical conditions should be considered thoroughly.

Groundwater level during the excavation should be monitored especially to counter balance the overburden effective stress for EPB type TBM.

Three collapses occurred nearby Nail Oraman and Food Engineering buildings with volumes ranging between 60 m$^3$ to 80 m$^3$ as the contractor reported since they are filled with cement immediately. Following the first collapse, the TBM cutting head had to be dismantled for maintenance and a shaft was excavated for the evacuation. During the process, a drawdown of groundwater was determined resulting with change of pore water pressure and effective stress.

The maximum settlement was recorded as 33 mm near Nail Oraman and 42 mm for the Food Engineering buildings before the collapses took place. The settlement troughs for both reveal that, excavation of the second tunnel has caused excess settlements. In order to depict this, the numerical modelling was constructed. The surface settlement is 27.35 mm when CI is 1% for Nail Oraman building, which is close to the measured value, while settlement is 51.02 mm for the Food Engineering building (CI = 1%).

The CI value is suggested to be between 0.5% and 1.5%. However, CI exponentially increases with the ground loss (g%). In order to determine the CI which corresponds to the settlements recorded after the collapses, an attempt to investigate the relation between settlement and CI was performed using the input models. It was observed that, CI increases to 13% and 16% for the Nail Oraman and Food Engineering building sections. In case the CI increases, the settlement reaches fixed values.

The settlement troughs extracted from numerical analyses are asymmetrical since the second tunnel is affected by the cavity of the first tunnel and when the CI exceeds suggested values, the shape of the trough seems to be reverse, since CI is related to the settlement at the segment’s outer diameter.

The main reasons for the settlements during the twin subway tunnel excavation are thought to be the defects on the TBM cutting head which caused a gap during the advance. Once this happened, the saturated granular soils were over excavated and resulted with the change of stress conditions over the
tunnels. During the maintenance of the cutting head, following settlements were believed to be triggered. Therefore, in case twin tunnels are planned within granular soil, possible shortcomings should be considered to prevent settlement and damage to structures and environment.

**Declarations**

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Figures
Figure 1

Location of the Tandoğan-Keçiören tunnel alignment between Km: 3+280 000 and Km:3+580 000

Figure 2

Plan of boreholes along twin tunnels
Figure 3

Monitoring of groundwater level in the boreholes
Figure 4

Collapse due to the settlement beneath Nail Oraman building and filling with cement afterwards
Figure 5

Settlement induced cracks inside and outside the Food Engineering building
Figure 6

Input geometry for the twin tunnels below Nail Oraman building (1 and 2 represent tunnel numbers)

Figure 7

Input geometry for tunnels below Food Engineering building (1 and 2 represent tunnel numbers)
Figure 8

Settlement curve for the Nail Oraman building before the collapses

Figure 9

Settlements for the twin tunnels below Nail Oraman building (CI=1%)
Figure 10

Settlements for the twin tunnels below Nail Oraman building (CI=13%)

Distance from tunnel center line, m

First tunnel axis

Second tunnel axis

$S_{max} = 0.020 \text{ m}$

$S_{max} = 0.042 \text{ m}$

Figure 11
Settlement curve for the Food Engineering building before the collapses

Figure 12

Settlements for the twin tunnels beneath Food Engineering building (CI=1%)

Figure 13

Settlements for the twin tunnels beneath Food Engineering building (CI=16%)
Figure 14

Ss/Sg versus Cl for NO building (a) and FE building (b)