Construction of Underground Railways Station beneath Unsound Buildings in Densely Populated Urban Area, Istanbul Turkey

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INTRODUCTION

The Marmaray project is currently under construction in Istanbul, Turkey to provide a rail link between Europe and Asia crossing the Bosphorus Strait. The $1.03 billion project includes the design and construction of 4 railway stations and 13.6 km of double track tunnel. The design build construction contract has been undertaken by the Taisei-Gama-Nurol Joint Venture and is financed by JICA. Tunneling elements include a 1.4 km immersed tube segment, 18.5 km of TBM tunnel, 1 underground station (Sirkeci station) constructed by SEM/NATM and two stations constructed by open cut (Fig. 1).

In this paper, observational method that is applied for Sirkeci station during tunnel excavation is described. This paper also provides monitoring plan to observe buildings above this station, updating of prediction using back analysis and building risk re-assessment based on revised prediction.

Fig. 1 General plan of Marmaray Project

OUTLINE OF THE SIRKECI STATION

Structure of the station

Sirkeci station is the deep underground station [1], which excavation works are now going by NATM. Fig. 2 shows a bird's-eye view of Sirkeci underground station. This station has so complicated configuration with not only 3 horizontal tunnels with several connections and 4 large ventilation and platform tunnels connecting to ventilation shafts, but also a couple of the shallow inclined shafts and horizontal tunnels to the two entrances. Furthermore, the ground surface above the station is densely occupied by the existing unsound buildings (Fig. 3).

The structural design of Sirkeci station becomes quite complicated and actual construction is to be faced with such technical difficulties as very close excavation to each tunnel, a lot of intersections, four platform and ventilation chambers with a large cross section (excavation area : about 200 m²) and their intersections.
GEOLOGICAL CONDITION
The geology consists of 3 layers as filling, clayey gravelly sand and rock from the surface, as shown in Fig. 4. Filling layer contains man made disposal including archaeological relic. Bed rock, called Trakya formation, consists mainly of sandstone-mudstone alternation. They are, in general, highly weathered and weak, and many sheared zones exist. Furthermore diabase intrudes into these sediment rocks. Diabase itself is relatively hard, but boundaries between mudstone and diabase are highly altered and deteriorated.
CONSTRUCTION PROCEDURE

Because the countermeasure for existing unsound buildings within the influence zone of tunnel excavation had been unsolved, full-face of horizontal tunnels could not be excavated. To reduce the risk of buildings, advanced pilot tunnels were applied instead of full-face excavation as shown in Fig. 5.

Due to delay of excavation of East Ventilation Shaft by archeological survey, horizontal tunnels can be excavated only from West Ventilation Shaft. Following the pilot tunnel, it is enlarged into top heading of platform tunnel. After excavation of the top heading, both the bench and invert sections are excavated and supported respectively. For large tunnel sections, multiple bench cut method for excavation is applied.

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**Fig. 4 Longitudinal geological section**

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**Fig. 5 Plan and section of pilot tunnel**
MONITORING FOR SIRKECI STATION

During tunnel excavation, both ground movement and building deformations are monitored with settlement and convergence measurements both on the ground level inside the tunnels as daily base. About 330 number settlement bolts and 160 number 3D targets were installed on the buildings to monitor the settlement and inclination of the buildings as shown in Fig. 6.

Underground instruments are shown in Fig. 7. Tunnel convergence measurements are conducted within every 20m, special measurement instruments such as concrete stress cell, steel rib strain gage, extensometer and in-place inclinometer are applied at not only the intersections of tunnels, but also the large platform and ventilation chambers. Extensometers and inclinometers from ground surface have been installed to observe 3 dimensional ground movements due to the tunnel excavation.

The frequency of monitoring depends on the tunnel excavation progress. For each measurement item (i.e., settlement and inclination of buildings on the ground surface; convergence, crown settlement and stress inside the tunnels), three alert levels, which are based on the predicted values by back analysis, are set up. Actual measurements are checked by comparing threshold values. If they exceed threshold limits, necessary actions will be taken. The maximum ground surface settlement is close to 60mm. However, differential settlements of all buildings are not so significant. Consequently building inclinations are not so critical.

Fig. 6 Monitoring plan on ground level

Fig. 7 Monitoring plan in tunnels
OBSERVATIONAL METHOD AND BACK ANALYSIS IN TUNNEL PRACTICE

The geological and geotechnical characteristics of the ground must be estimated as correctly as possible at the design stage. However, it is hard to investigate all those ground conditions including the existing fracture and shear zones precisely, because of technical and economical reasons. Therefore, the actual ground conditions which can be seen during the excavation often different from those estimated at the design stage. If the actual ground conditions were almost same as those estimated at the design stage, the initially designed support measures would be evaluated as a suitable pattern. At the Sirkeci station the actual ground conditions have gradually been revealed and, roughly speaking, they are approximately the same as the initially estimated conditions. This implies that the initial support pattern can be evaluated as suitable. It should be noted, however, that if we take a close look at the ground conditions, they differ from place to place, and slightly different from the initially predicted conditions. Therefore, we should take into account this difference in excavation practice by modifying the initially designed support structures on the basis of the field measurement results. This modification procedure is the basic idea of NATM, and is called as "Observational Method".

Basic idea of assessing both the ground settlements and the stability of the tunnel is based on back analysis of field measurements. The back analysis method used here was originally developed by Sakurai (1999), in which the “anisotropic damage parameter” plays a major role to obtain a good agreement between the measured and calculated values of both displacements around the tunnel and the ground surface settlements. It should be emphasize that if we use the anisotropic damage parameter, there is no need to assume any numerical model. Any type of mechanical behaviors of the ground such as anisotropic and elasto-plastic behaviors including perfect plasticity, strain hardening, and even strain softening behaviors can be back-analyzed by the use of the anisotropic damage parameter. This must be a great advantage in back analysis, because of that numerical model of the ground should not be assumed, but it should be identified by back analysis considering the real behavior of the ground.

Therefore, when we determine the anisotropic damage parameter by back analysis so as to get a good agreement between the measured and the computed displacements and surface settlements, we can immediately evaluate what is occurring mechanically in the underground, for instance, whether strain localization called “shear band” tends to occur or not. The shear band occurring around the tunnel is extremely important for assessing the adequacy of tunnel support structures.

PREDICTION OF GROUND SURFACE SETTLEMENT BASED ON BACK ANALYSIS

In the Sirkeci station, the ground surface settlements due to the excavation of the vertical shaft as well as tunnels became larger than the predicted values. It is quite common in tunnel practice to have such a difference between the real behaviour and the predicted one, even though the actual ground classification is more or less the same as the predicted one. It is obvious that this difference is mainly due to the various uncertainties involved in ground, such as existing fracture and shear zones, underground water conditions, interaction between support structures and surrounding ground. This is the reason why we need the observational method in tunnel practice where field measurements play an important role, particularly in NATM. The procedure for the analytical prediction is described as follows.

- The two dimensional finite element method (FEM) shall be used.
- With respect to relationship between stress and strain in the soft ground, elasto-plastic element conforming to the Mohr-Coulomb criterion (elasto-plastic analysis) is applied [2,3].
- With respect to modeling of rock mass, anisotropic damage parameter is introduced as described below.

Considering the anisotropic damages, a stress-strain relationship can be expressed as follows.

\[
\{\sigma\} = [D']\{\epsilon\} \quad \text{(Eq.1)}
\]

where

\[
[D'] = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} (1-\nu) & \nu & 0 \\ \nu & (1-\nu) & 0 \\ 0 & 0 & m(1+\nu)(1-2\nu) \end{bmatrix} \quad \text{(Eq.2)}
\]

Shear stiffness decreases from original value as a function of the anisotropic damage parameter d.
The anisotropic damage parameter $d$ is defined as a function of maximum shear strains. The schematic relationship between $d$ and $\gamma$ is shown in Fig. 8.

$$d = m_0 \left(1 - \exp(-100\alpha(\gamma - \gamma_0))\right)$$  \hspace{1cm} (Eq. 4)

where
- $m_0$: converged value of anisotropic damage parameter ($=1/2(1+\nu)$)
- $\alpha$: material constant (degree of changing ratio)
- $\gamma_0$: critical shear strain ($d=0$ for $\gamma < \gamma_0$)
- $\gamma$: maximum shear strain

![Fig.8 Relationships between Damage parameter and Maximum shear strain](image)

**Analysis Model**

The boundary between the bedrock and soft ground was determined from making reference to the results of boring inspections in the vicinity. A model of the strata has been produced using the results of these borings. The cross section for back analysis at the representative platform section is shown in Fig. 9.

Geotechnical design parameters for rock mass are set as shown in Table 1. The anisotropic damage constants $\alpha$ which are described above is calculated so as to match the results of measurement. In order to achieve more precise prediction of ground surface settlement, it is of great importance to simulate vertical deformations around the tunnel in the back analysis. Both of settlements at tunnel crown and spring line are mainly used in this study to match observed values respectively. From the back analysis, the anisotropic damage material constants $\alpha$ and the lateral pressure ratio $K$ have been determined. Observed values are shown below.

- Crown settlement: $-3.3$ to $-12.0$ mm
- Spring line settlement: $-5.5$ to $-16.5$ mm
- Ratio of both settlements: $0.26$ to $0.91$

**Prediction of ground surface settlement**

After doing back analysis in the Sirkeci station, we found that the large surface settlements are mainly caused by the contraction of the filling material existing near the ground surface. This kind of contraction was not expected at the design stage before the excavation. Besides, at the design stage it is assumed that the anisotropic damage parameter does not mobilize so that the ground material is always in an isotropic state. However, back analysis results revealed that the anisotropic damage parameter mobilized during the excavation. This means that strain localization tends to occur around the tunnel, although it is not serious. Considering the back-analyzed anisotropic damage parameter, the ground surface settlements were re-evaluated. Since we have been getting many data of displacements around the tunnels as well as the ground surface settlements, back analysis can provide with a better assessment for
the tunnel behavior. As a result, a good accuracy can be achieved for predicting the ground surface settlements and for the evaluation of influences to the existing buildings and their re-assessments.

By determining parameters based on back analysis, ground surface settlements are calculated by the forward analysis. The anisotropic material damage constant $\alpha$ is assigned to 2.5 (maximum) and 1.5 (minimum). The lateral pressure ratio $K$ is 1.1. The analysis method, model and physical properties are explained in previous section. The procedure for tunnel excavation analysis is set up to reflect the actual construction. After excavation of pilot tunnel, PF tunnel excavation is undertaken sequentially in three stages as top heading, bench and invert. Fig. 10 and Fig. 11 show the distribution of surface settlements both in case of $\alpha = 1.5$ (Case1) and 2.5 (Case2) respectively. The settlement in case2 is about two times as large as that in case1. The star marks in Fig. 10 and Fig. 11 show the actual settlements due to the advanced pilot tunnel excavation and the following PF tunnel enlargements respectively. The actual settlements that correspond with analysis procedure of PF (brown line) is almost fit to the result in Case1. On the other hand, the actual settlements due to Pilot (South) (blue line) is close to the result in Case2. As a result, a good accuracy has been achieved for predicting the ground surface settlements.

![Fig. 9 Cross Section for back analysis model](image)

<table>
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<tr>
<th>Ground class</th>
<th>Deformation modulus (kN/m$^2$)</th>
<th>Poisson’s ratio</th>
<th>Unit weight (kN/m$^3$)</th>
<th>Cohesion (kN/m$^2$)</th>
<th>Internal friction angle (Degree)</th>
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</tbody>
</table>

COUNTERMEASURE FOR EXISTING BUILDING ABOVE THE TUNNELS
The exiting buildings above Sirkeci station are shown in Fig. 3. There are 174 numbers of buildings within the affected zone due to the tunnel excavations in Sirkeci area. It is important to evaluate the influences and the damages to the existing building during tunnel excavation.

In preparation stages prior to tunnel excavation, all the buildings have been inspected and evaluated using the initial prediction of ground surface settlements and the associated inclinations. Next the assessments of buildings have been carried out to be categorized as low risky, moderately risky and highly risky by experts. The precautional measures have been taken for each building and the final countermeasures are decided according to the zoning low in Turkey. According to the revised prediction based on the back analysis, the risk assessment for the building should be updated in tunnelling progress.

CONCLUSION
The construction procedure of complicated underground structures under severe conditions, with the monitoring plan to reduce the damage risk against the existing unsound buildings, is reported. During design and actual excavation stages, prediction and update on tunneling-induced ground movements and their influences on buildings have been intensively studied by back-analysis, based on the anisotropic damage parameters for the surrounding rock masses. In this report, both the observational method and the associated back analysis are a very powerful tool for tunneling works and should be continued on step by step basis to estimate more accurate behavior of ground. Progress of the project would provide not only an opportunity to learn of the useful findings to other similar international projects, but also a key that led to make the worthwhile and enjoyable decision to precede the underground developments under sensitive buildings in densely populated urban area.
Fig. 10 Distribution of Ground Surface Settlement ($\alpha=1.5$: Case 1)

Fig. 11 Distribution of Ground Surface Settlement ($\alpha=2.5$: Case 2)

Reference