Simplified numerical method for tunnel design under seismic condition: Some examples about Istanbul Metro design, Kadikoy-Kartal Line

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ABSTRACT: Historically, underground facilities have experienced a lower rate of damage than surface structures. Nevertheless, some underground structures registered significant damages in recent large earthquakes; the resultant database which can be found in literature, together with the quite slight overburden of many stretches of the Istanbul new metro line, required a particular care on the verification of the vulnerability of the underground structures under severe seismic events.

This article describes briefly the main assessments on the approaches used by engineers in quantifying the seismic effect on an underground structure, both referring to deterministic either probabilistic methods. However, this study has the goal to propose a method that could be as more consistent as possible with the common procedures nowadays adopted to evaluate the effect of seismic motion on tunnels. In particular, as the complex geometry of most of the cross sections in Istanbul Metro underground structures is not simple enough to make proper use of the closed form solutions, a numerical method is recognized as the only one to assist the design process. However, a simple yet rational method of analysis for evaluating the ovaling effect was preferred than a complete coupled analysis, with an application of the time dependent acceleration spectrum, because the large number of different structures and geological conditions required to optimize the time cost of the procedure.

1 INTRODUCTION

Historical data usually report that underground facilities experienced a lower rate of damage than surface structures, thus leading to the assumption that tunnels, particularly the ones which lie deeper, can be affected by very little damage during a seismic shake, due to their vibration mode which is very close in terms of amplitude and frequency to that of the surrounding ground.

Nevertheless, some underground structures registered significant damages in recent large earthquake; the resultant database which can be found in literature, together with the quite slight overburden that many stretches of long underground structures can have, require a particular care on the verification of the vulnerability of the underground structures under severe seismic events.

2 ASSESSMENTS ON TUNNELS SEISMIC DESIGN

It is not common practice to consider seismic actions in tunneling project; usually only for very shallow (overburden minor than 6 m) underground structures the seismic actions are considered when performing the structural design. However, due to the high seismicity level of certain areas, as for instance the case of the densely urbanized area of Istanbul as showed in Figure 1, it is a requirement of the Municipality that all the final linings have to be designed to resist the seismic loads.

2.1 Tunnels induced effects by earthquakes

Tunnels and underground structures are usually affected by earthquakes in terms of transient ground motion and permanent ground deformation, which take place due to the shaking. As for transient
The ground strain and the curvature due to wave propagation influence the response of the tunnels. The motion of the soil particle depends on the type of waves, but can always be resolved into a longitudinal and transverse component with respect to the tunnel and immersed tube axis. The propagation velocity of the body and surface waves along the alignment (apparent velocity of propagation) and the peak ground velocity are the two important parameters that control wavelength and amplitude. The maximum ground curvature will be equal to the second derivative of the transverse displacement with respect to distance and is controlled by the peak ground acceleration, transversal to the direction of wave propagation, and by the apparent velocity of propagation.

The main assumption which is widely accepted is that the response of the buried tunnels in earthquakes follows closely the deformation imposed on by the soil media. For small to moderate ground motion the tunnel strain can be taken equal to the ground strain. However, for large ground motion the tunnel or immersed tube strain is limited by the slippage that occurs at the pipe-soil interface.

2.2 Seismic design criteria for tunnels

Several studies documented earthquake damages to underground facilities: as an instance, Sharma and Judd (1991) collected qualitative data for 192 reported observations while Power et al. (1998) provided an extensive database with 217 case histories. Thus, in recent years, the earthquake resistant design criteria have been revised to incorporate performance-based design both for building and non-building structures. NEHRP (2003) Recommended Provisions presents criteria for the design and construction of new structures subject to earthquake ground motions in order to minimize the hazard to life and to increase the expected performance of structures having a substantial public hazard due to occupancy or use. For essential facilities, damage limitation criteria are stricter in order to better provide for continued functioning of the facility.

Depending on the performance criteria, limited structural and non-structural damage can be expected as a result of the “design ground motions”. For ground motions in excess of the design levels, the intent of the Provisions is for the structure to have a very low likelihood of collapse. In order to provide for a uniform margin against collapse at the design ground motion, the ground motion hazards are defined in terms of Maximum Considered Earthquake (MCE) ground motion, described, statistically, at a probability of exceedance of 2 percent in 50 years (meaning a return period of about 2500 years). In regions where the seismic hazard
is typically controlled by large-magnitude events occurring on a limited number of well-defined fault systems (such as the Marmara Region) it is considered more appropriate to directly determine maximum considered earthquake ground motions based on the characteristic earthquakes of these defined faults and to compute the ground motion parameters through deterministic procedures.

2.3 **Applicability of current codes to underground constructions**

Although the current codes for the earthquake resistant design of transportation lifelines are not specifically applicable to the subway systems, their field of application can be extended to the evaluation of the risk basis for the assessment of design basis ground motion. In this respect, the dimensioning of the final lining of all the underground structures of Istanbul Metro project was done in conformity with the Eurocode 8 [CEN, 1994], that is often in strong agreement with the Japanese Code [JRA, 1996] and the USA Codes [AASHTO, 1996].

In all these codes, during “strong shaking”, “some” degree of damage is allowed. The definition of “some” degree of damage generally implies flexural yielding, provided that no collapse takes place. Japanese Code expects elastic performance under the design earthquake, which may have a return period in the order of 75 years. A ductility check on this design is then made assuming a much larger earthquake with a return period of about 500 years. The Eurocode stipulates that the function shall be maintained, with appropriate reliability, after the design seismic event, which corresponds to a 475-year return period earthquake for bridges of average importance. Furthermore, in AASHTO Code the definition of “strong shaking” is already taken into account for the design earthquake with a 475 year return period. Although AASHTO does not explicitly require a two-level design, the new Caltrans/Applied Technology Council Criteria [Buckle, 1996] specify design for the “Functional Evaluation Earthquake” and the “Safety Evaluation Earthquake” corresponding, respectively to 100 and 1000 year return periods. Caltrans code essentially follows the ATC-32 [Improved Seismic Design Criteria for California Bridges, ATC 1996]. Finally, AGI Guidelines for geotechnical aspects of seismic design also mention (Silvestri and Simonelli, 2005) two levels (L1 and L2) for the definition of the design earthquake ground motions, respectively for probability of exceedence of 50% and 10% of the reference lifetime.

So that a second level of ground motion was considered, identifying a return period of about 2500 years as the strongest event ever happened in the region of the Straits.

### 3 ISTANBUL METRO SEISMIC DESIGN ASSUMPTIONS

For the seismic design of the Kadikoy-Kartal Mass Transportation Route two levels of ground motion were considered for earthquake resistance design purposes and will be presented in this chapter.

#### 3.1 **Functional evaluation earthquake ground motion (S1)**

This ground motion refers to earthquakes that can reasonably affect the transportation route at any location during its lifetime. Considering the seismicity of the region and the importance of the tunnels, it will be prudent to assign a 50% probability of exceedance in 50 years. This ground motion level will also be checked with deterministic median (50 percentile) ground motions that would result from a $M_w = 7.5$ scenario earthquake occurring on the Main Marmara Fault, and the larger of the two assessments will be selected. Under exposure to this ground motion the transportation system will be fully operational (essentially linearly elastic performance).

#### 3.2 **Safety evaluation earthquake ground motion (S2)**

This ground motion can be assessed either deterministically or probabilistically. The probabilistic ground motion for the safety evaluation typically has a long return period (approximately 2500 years), which corresponds to 2% probability of exceedance during 50 years. This level of ground motion is associated with the Maximum Credible Earthquake (MCE) is defined as the largest earthquake, that is capable of occurring along an earthquake fault, based on current geologic information, which is quite similar, if not identical to the NEHRP (2003) definition. Under Safety Evaluation Earthquake only repairable damage with no danger to life is allowed.

#### 3.3 **Expected performance levels under design ground motions**

The performance levels expected under $S_1$ and $S_2$ ground motions are defined below:

<table>
<thead>
<tr>
<th>Earthquake level</th>
<th>Expected performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_1$</td>
<td>Continuous</td>
</tr>
<tr>
<td>$S_2$</td>
<td>Limited</td>
</tr>
</tbody>
</table>

“Continuous Functionality” performance criterion refers to the uninterrupted service of the
structure immediately after an earthquake. On the other hand the “Limited Functionality” performance criterion will guarantee only limited use a few days after an earthquake. Full functionality is aimed to be obtained in at least a few months. The “Minimal” damage criterion refers to the nearly linearly-elastic response. To meet the performance criteria required under the S1 earthquake the “Response Modification Factor” should not exceed 2 [Caltrans, 1999]. The “Repairable” Damages should be repaired with minimum effects on functionality. The “Considerable” damages should not cause total collapse and loss of life.

These S1 and S2 levels of ground motion were originally quantified in frequency domain using the standardized response spectral shape of NEHRP, 2003 or IBC, 2006 in terms of the short-period (0.2 s) and 1 s-period spectral amplitudes at NEHRP B/C site class boundary. The site dependent spectra was then generated using the NEHRP Site Class definitions and the associated spectral site amplification factors [refer to “Preliminary determination of the design basis ground motions for the Istanbul Kadikoy—Kartal railed mass transport system”, Bogazici University, Kandilli Observatory and Earthquake Research institute Department of Earthquake Engineering].

3.4 Ground deformation approach

The general procedure for seismic design of tunnel structures is based primarily on the ground deformation approach. During earthquakes, tunnel structures are assumed to move together with the surrounding soil media. The structures, therefore, are designed to accommodate the deformations imposed by the ground. However, the effects of soil structure interaction can play an important role in the seismic response of tunnel or over buried structures, particularly when the structure is surrounded by soft media (such as the case for an immersed tunnel) and therefore should be considered in the analysis. Furthermore, for tunnel structures with considerable structural discontinuities (such as joints between tunnel segments and between the tunnel and the station structure) detailed evaluations have to be given to the effects of these discontinuities on the earthquake resistance of the tunnel.

3.4.1 Spatially varying ground motion time histories

Due to the length of the structures, the effect of spatial variations of ground motions is an important factor to be considered for the proposed tunnel structures. The wave-passage effect results from different arrivals of seismic waves at different parts of the structure and can be taken into account by assuming a time lag of the ground motion time histories between any two locations along the tunnel alignment.

3.4.2 Ground failure

Stability of ground surrounding the tunnel structures shall be considered in the design. The surrounding ground includes natural and backfill earth mass located within a zone that may influence the performance of the tunnel during and after earthquakes. For the Kadikoy-Kartal Mass transportation Route, the ground failure as a result of seismic shaking may include liquefaction and slope stability (landslides occurrence was often verified with pseudo-static methods).

3.4.3 Site response analysis

In all cases the earthquake excitation can be represented by a vertically propagating horizontally polarized shear wave incident from the engineering bedrock (NEHRP B/C boundary).

An underground tunnel structure undergoes three primary modes of deformation during seismic shaking (Fig. 3):
- ovaling deformation;
- axial deformation;
- curvature deformation.

The ovaling deformation is caused primarily by seismic waves propagating perpendicular to the tunnel longitudinal axis. The axial and curvature deformations are induced by components of seismic waves that propagate along the longitudinal axis and/or by spatially varying ground motions resulting from local soil/site effects. Wave propagation strain tend to be most pronounced at the junctions of dissimilar buried structures (such as tunnel connecting with a building) or at the interfaces of different geological materials (such as passing from rock to soft soil).

The approach used to examine the effect of the seismic actions on the tunnel stability is the free-field

Figure 3. Primary deformation modes of tunnels due to seismic shaking [Owen and Scholl, 1981].
shear deformation method [Wang, 1993, Power et al. 1998; Hashash et al. 2001], which assumes that the deformation of the structure should conform to the deformation of the soil in the free-field under the design earthquakes (S1 and S2).

3.5 Application of the free-field shear deformation method

The methodology of the seismic design has to incorporate the additional loading imposed by ground shaking and deformation. In general seismic design, loads for tunnel are characterized in terms of the deformations and strain imposed on the structure by the surrounding ground based on their interaction.

To describe the procedure used to compute deformations and forces induced by this interaction, two design approaches have been introduced:

- Free-field deformation approach [Wang 1996; Power et al. 1998; Hashash et al. 2001]
- Soil–structure interaction approach

In free-field deformation approach, the ground deformation caused by seismic waves is assumed to occur in the absence of structure or excavation. These deformations ignore the interaction between the underground structure and the surrounding ground, but can provide a first-order estimate of the anticipated deformation of the structure. The closed form elastic solution results in combined axial and curvature deformations, assuming the tunnel as an elastic beam and maximum strain at critical incidence angle. The advantages and disadvantages of this method have been reported by Wang (1993).

The presence of an underground structure modifies the free-field ground deformations; so a method based on soil-structure interaction is required. This solution uses the beam-on-elastic foundation approach that is used to model (quasi-static) soil-structure interaction effects. Under seismic loading, the cross-section of a tunnel experiences axial bending and shear-strain due to free-field axial, curvature.

The closed form solutions for estimating ground-structure interaction for tunnels are generally based on the assumptions that:

- The shape of the tunnel is circular,
- The ground is an infinite, elastic, homogeneous, isotropic medium,
- The circular lining is generally an elastic, thin walled tube under plane strain conditions,
- Full-slip or no-slip conditions exist along the interface between the ground and the lining,
- Loading conditions are to be simulated as external loading (overpressure loading) or excavation loading.

3.6 Evaluation of the maximum shear deformation in free field conditions

As could be evaluated in previous paragraph, there are some limitations in the application of the free-field method, related to the complex geometry of tunnel sections. This section is devoted to describe the approach adopted for the simulation of the seismic effects taking the advantage of using numerical analysis in order to overcome the listed limitations.

The applied methodology foresees the application of a deformation to the ground so as to deform the underground structures and to obtain the stresses acting in the final lining in case of a seismic event.

As for the aforementioned approach, from the value of the short period spectral acceleration $S_S$ (according to the mentioned Seismic Report of Bogazici University) it is possible to get the Pick Ground Acceleration (PGA, here $a_{pR}$) by the equation (1):

$$a_{pR} = 40\% \cdot S_S$$

which assumes that the deformation of the structure should conform to the deformation of the soil in the free-field under the design earthquakes (S1 and S2).

The design basis ground motion parameters to be used in the earthquake resistant design of the tunnels will be stipulated as follow:

- The $S_S$ values used into the analyses for the S1 (Functional Evaluation) Level earthquake were obtained from “$S_S$ deterministic Median” values for each borehole or station location.
- The $S_S$ values used into the analyses for the S2 (Safety Evaluation) Level earthquake were obtained from “$S_S$—2475 years” values for each borehole or station location.

The site-specific Pick-Ground-Acceleration ($a_{max,i}$) is given by equation (2), where S is the soil factor, defined in terms of the ground type [EC 8]:

$$a_{max,i} = S \cdot a_{pR}$$

The value of $S$, suggested in Eurocode 8, is based on the types of elastic response spectra and its values are recorded in specific depending on the earthquake moment magnitude scale: $M_p < 5.5$ or $M_p > 5.5$.

In order to consider the depth of the tunnels, a simplified procedure [Hashash et al. 2001] was considered to define the pick acceleration at the depth of the tunnel $a_{z,max}$. It consists in the determination of a reduction coefficient $C$ for the peak acceleration on the surface depending on the depth of the tunnel (Table 1) as for equation (3):

$$a_{z,max} = C \cdot a_{pR,max,Z}$$

where $a_{z,max}$ is the pick acceleration at the depth of the tunnel.
The value of $a_{z,\text{max}}$ is used to determine the $\gamma_{\text{max}}$ (maximum shear deformation in free-field condition) from the peak ground velocity $V_s$, which is a function of earthquake magnitude and distance from the seismic source, as shown in equations (4) and (5):

\[
\gamma_{\text{max}} = \frac{V_s}{C_s} \quad (4)
\]
\[
V_s = k \cdot a_{z,\text{max}} \quad (5)
\]

where $k$ is the ratio of peak ground velocity to peak ground acceleration; $C_s$ is the apparent propagation velocity of S-wave and it was assumed (along the entire alignment) equal to 2 km/s, according with the prescription given by Prof. Dr. Mustafa Erdik.

The apparent propagation velocity of S-wave ($C_s$), in fact, is not necessarily equal to the real propagation velocity; in fact, several authors [O’Rourke & Liu, 1999; Power et al. 1996; Paolucci & Pitilakis, 2007] have suggested values between 1 and 5 km/s.

The value of $\gamma_{\text{max}}$ corresponds to the maximum horizontal displacement imposed in the numerical model, calculated as per equation (6).

\[
\Delta x_{\text{max}} = \gamma_{\text{max}} \left( \frac{h_{\text{mod}}}{2} \right) \quad (6)
\]

The horizontal displacement $\Delta x_{\text{max}}$ is obtained applying to the sides of the model punctual forces in order to generate a pure shear deformation of the entire model (Fig. 4) and consequently the ovaling effect of the excavation boundary, as shown in Figure 5. Results will be deeply discussed in following paragraphs, but it is already possible to appreciate how the deformed shape is similar to the theoretical approach shown in Figure 7.

Through this methodology, it was possible to obtain the stresses, due to the ovaling deformations, acting in the underground final structures for the cases of the design earthquakes. In order to verify also axial and curvature deformation (usually these effects are secondary in respect of the ovaling ones), it was accepted to use the analytical closed-form solution [Barla et al. 2008]. The estimated deformations allowed to define the thickness of the movement joints with the specific function to absorb the seismic deformations (Fig. 6).

### 3.7 FEM analysis of the ovaling phenomenon

A series of computational analyses using finite element code (Phase2—v6, RocScience) were performed in order to verify the proposed procedure in the previous section and to properly calibrate the model, so to get same results for a circular shaped tunnel as from simplified methods with closed form solutions [Barla et al. 1986; Wang, 1993; Bobet, 2003; Corigliano et al. 2008].
The assumptions made for these analyses include the following:

- Plane strain model with no gravity loading was performed.
- No water pressure and no flow boundary conditions were assumed in the model.
- Seismic shear wave loading is simulated by pure shear conditions, through a “trial & error procedure”, by applying horizontal line-forces to the upper and lower external boundaries of the model, thus checking whether the obtained horizontal displacement $\Delta x_{\text{max}}$ was the desired one or further analyses are required to achieve it.
- The Authors verified that the direct application of the displacement field to the mesh rather than a force distribution could not allow a proper evaluation of the real effects in terms of stresses on the tunnel lining.
- In order to make possible the rigid distortion of the model and to create a pure shear condition, high strength liners were modeled at the vertical external borders and a hinge was created on both sides at an height of $h/2$. Hinges were introduced in order to create a proper restraint that can avoid numerical errors due to eventual fictitious horizontal translations.
- As the geometry of the cross sections (particularly of the switches, exemplified in this article) is not regular, no advantage of the anti-symmetric loading conditions could be taken, thus the entire lining/ground system was analyzed.
- Lining was modeled by a series of continuous flexural beam elements of linear elasticity through the “Equivalent Axis Beam Method”. The hypothesis of an elastic dominion for the structure even in the case of the S2 seismic condition is a conservative way to take into account the difficult evaluation of the activation and evolution of plastic hinges.
- Due to its high stiffness, subsoil layers were modeled as a linear elastic homogeneous and isotropic material.
- No-slip condition along the lining-ground interface is assumed as it is recognized the most suitable for rock formations and for a proper simulation of waterproofing.
- Mechanical parameters implemented in the model were derived directly from the information given by field tests and laboratory studies on the specimens taken from the borehole: unfortunately, specific values from dynamic tests could be obtained along the whole alignment, thus a number of correlations found in literature were considered and compared: as for the elastic modulus, Stacy correlation was adopted, a ratio $E_{\text{dyn}}/E_{\text{static}} = 2$ was used, with: $E_{\text{dyn}}$ = dynamic modulus of the equivalent material used for numerical analyses and $E_{\text{static}}$ = elastic Young modulus measured with site testing (pressure-meter) on the rock mass.

3.8 Dimensioning and verifications of the final lining

The numerical analyses have been chosen to dimension the final lining in static and seismic conditions with and without water pressure.

In order to verify the final lining under static and seismic load conditions (with and without water pressure) different loading combinations have been investigated through FEM and structural numerical simulations, with the hyperstatic reaction method.

Three typologies of numerical analyses have been performed, as shown in Table 2. It must be

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Description</th>
<th>Induced stresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>FEM analysis in static condition without water pressure</td>
<td>$S(g)$</td>
</tr>
<tr>
<td>B</td>
<td>FEM analysis in static condition with water pressure</td>
<td>$S(g) + S(w)$</td>
</tr>
<tr>
<td>C</td>
<td>Pseudo-static FEM Analysis (free field method—ovaling effect) without water pressure</td>
<td>$\Delta S(g)$</td>
</tr>
</tbody>
</table>

$S(g)$: Stress function of the field stress; $S(w)$: Stress function of the water pressure; $\Delta S(g)$: Increase of stress function of the field stress.
underlined that analysis C has been carried out with two different seismic conditions (called S1 and S2).

Table 3 shows the six different loading combinations based on the previously mentioned analysis. In the seismic conditions, the internal forces obtained from A (or B) simulations have been summed to the internal forces obtained from C + S1 (or C + S2) analysis, as for the common known method of superposition of the effects of the actions.

The structural verifications are performed according to EN 1992 [Eurocode 2] for Ultimate Limit State (U.L.S.) and Serviceability Limit State (S.L.S.), except for the seismic case in which this last one is not required. The structural analyses were performed in order to demonstrate the final lining adequateness in terms of geometry, thickness and reinforcement.

3.8.1 Structural verification for ultimate limit state (U.L.S.)
As for the U.L.S. verifications, for the final lining verification $\alpha$, that is a factor taking into account for the effects of long duration loads, is considered equal to 0.85 [EC2, paragraph 3.1.6].

The magnitudes of the bending moment, axial force, and the shear force were determined by numerical analysis after applying the pounded water load in order to simulate the long term conditions. The axial force, bending moment and shear force values, which were obtained directly from the numerical analyses, were multiplied by a factor $\gamma_G$, as shown in equations (7):

$$N_d = \gamma_G \cdot N_k \quad M_d = \gamma_G \cdot M_k \quad V_d = \gamma_G \cdot V_k$$

(7)

For the bending moment—axial force verification, the resistance envelope for the final lining is obtained according with formulations (8) and (9):

$$f_{cd} = \frac{\alpha}{\gamma_c} R_{ck}$$

(8)

$$f_{yd} = \frac{f_{ck}}{\gamma_s}$$

(9)

where:

- $\gamma_c$ is a resistance reduction factor for the concrete
- $\gamma_s$ is a resistance reduction factor for the steel

The factors for actions and resistance depending on the condition considered: Static, Seismic S1 or Seismic S2.

Refer to Table 4 in order to review a summary of the considered input data for the S.L.U. verifications and all the corresponding adopted coefficients.

3.8.2 Structural verification for serviceability limit state (S.L.S.)
For the S.L.S., the structural analyses consist in verifying that the width of the crack $w$ is less than 0.3 mm. In the same way the maximum stress...
must be always lower than the allowable stress for concrete and steel.

In detail, the maximum compressive stress for the concrete and for the steel is obtained as:

\[ f_{c,SLS} = 0.6 \cdot f_{ck} \quad [EC2, \text{paragraph 7.2}] \]

\[ f_{y,SLS} = 0.8 \cdot f_{yk} \quad [EC2, \text{paragraph 7.2}] \]

For the seismic condition, the S.L.S. verification is not required, so that it will not be shown in this article, but it was necessary to verify the Static conditions of the final lining.

4 APPLICATION CASE: SHAFT S12

This chapter deals with the case study of Shaft S12, one of the biggest shafts of the entire line Kadıköy-Kartal of Istanbul Metro. Its function is double, both for temporary access for workers and equipment in the tunnels and for definitive holding of the lifts and ventilation to access Kucukyali Station. Because of this reason, a proper evaluation and dimensioning of the final lining was necessary to be delivered to the Engineer, including seismic verifications as for the aforementioned concern to this load condition by the Municipal Authority.

Despite the irregular geometry, showed in Figure 8 in an axonometric view corresponding to the interception with the running tunnel (section A2.2), it was chosen as typological by the Authors, because it is really representative of the typical condition of the underground structures in Istanbul, where the highly urbanized context, summed to geological and hydro-geological difficulties, produces necessarily the need for an integrated design, able to answer to every problem and to modify itself elastically.

4.1 Construction sequences for final lining

The main construction sequences of the shaft final lining are summarized as:

- intersection tunnel—shaft: invert water proofing;
- intersection between running tunnel and shaft: invert reinforced concrete cast in place;
- intersection tunnel—shaft: sidewalls and crown water proofing;
- intersection tunnel—shaft: sidewalls and crown reinforced concrete cast in place;
- shaft: water proofing;
- shaft: final lining cast in place.

4.2 Static analyses

The static study of the shafts was achieved with 2D elastic-plastic FEM analyses. In particular, the maximum loaded horizontal section was analyzed. The 2D, elastic-plastic, plane strain FEM analyses were performed with the following stages:

1. Initial homogeneous and isotropic field stress.
2. Simulation of the face excavation through a reduction of the Young modulus according to the “Stiffness reduction method”;
3. Installation of the primary support (shotcrete and rock bolts);
4. Removal of the primary rock support system and simulation of the final lining;
5. Reduction of the ground cohesion and stiffness by as much as 50% (in order to consider the long term geomechanical characteristics reduction).

In the circular shaft an additional load equal to the 10% of the pressure on the final lining has been added in order to simulate an anisotropic load condition.

4.3 Pseudo-static analyses

The passage of earthquake waves through the soil/rock causes four distinct deformations patterns of the shaft opening. These are: (1) longitudinal or axial strain which are uniform over the sections, (2) shear strain acting on horizontal sections of the shaft, (3) flexural strain resulting from bending of the shaft and (4) hoop strain arising either from the influence of axial strain as described by Poisson’s ratio or from ovaling deformations (see Fig. 9).

Two main assumptions have been considered in the seismic design of shafts:

1. the shaft lining deforms in accordance with the surrounding ground;
2. compression (P) and shear \( (S_H \text{ and } S_V) \) waves are assumed to propagate in a direction parallel to the axis of the shaft.

Based on these considerations, on the geomechanical parameters and on the specific construction procedures, the most impacting effects on the final lining of the shaft are due to the free-field shear strain acting on the horizontal sections creating ovaling deformations on the final lining.
4.4 Seismic analyses model

For Shaft S12 final structures a seismic resistant design was performed, giving to the structures the requested safety requirements. Referring to the methodology illustrated in Section 3, the data characterizing the seismic design are reported in following table:

<table>
<thead>
<tr>
<th>Earthquake level</th>
<th>S1</th>
<th>S2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_s [g]$</td>
<td>0.607</td>
<td>1.357</td>
</tr>
<tr>
<td>$a_{gr}[g]$</td>
<td>0.243</td>
<td>0.543</td>
</tr>
<tr>
<td>$a_{max,s}[g]$</td>
<td>0.243</td>
<td>0.543</td>
</tr>
<tr>
<td>$a_z[g]$</td>
<td>0.194</td>
<td>0.434</td>
</tr>
<tr>
<td>$V_s [m/s]$</td>
<td>0.188</td>
<td>0.421</td>
</tr>
<tr>
<td>$C_s [m/s]$</td>
<td>2000</td>
<td>2000</td>
</tr>
<tr>
<td>$\gamma_{max}[-]$</td>
<td>0.0000942</td>
<td>0.000211</td>
</tr>
<tr>
<td>$\Delta_{z_{max}}[m]$</td>
<td>0.0066</td>
<td>0.0147</td>
</tr>
</tbody>
</table>

Many numerical analyses were carried out in order to verify and compare the stresses, normal forces and bending moments acting in the shaft final lining according with the seismic design. Three main models were selected in order to check the opportunity of adopting the aforementioned method and all of them had the same exact dimensions and mesh refinement as the FEM static model showed in Figures 10 & 11.

The adopted methods only differ for the way the deformation defined in Section 3 is applied to the model. They can be summarized as following:

1. $\Delta_{z_{max}}$ direct application on the top and bottom of the model to the extreme nodes (Fig. 12);
2. pure shear deformation of the model obtained through vertices displacement imposition (Fig. 13);
3. iterative process to determine the external forces to be applied to the external boundaries in order to obtain the prescribed horizontal displacement.

This last one will be demonstrated in following chapter as the most appropriate one from an engineering point of view.

4.5 Seismic analyses results

In subsequent figures the results of numerical analysis are shown, in order to compare the three methods: as previously anticipated, the third method is the most suitable, as the displacements are not excessively affected by secondary motions which are not proper of a pure shear (as shown in Fig. 14).

Furthermore, respect to the second method, the lower values of normal forces registered and contemporaneous similar results in terms of bending moments, lead to the conclusion that the verification would generally be on the side of safety in the third method, so that this one was adopted for dimensioning steel bars. In details, Figure 16 represents axial force (N)—positive values correspond to compressive forces-, bending moment (M) and shear forces (V) in the final lining for the static case (without water—as instances—A), while Figures 17–19 for both seismic case S1 and S2, and comparing the three methods for the three defined parameters. Those last figures show the results of the analyses combined as superposition of effect of the seismic with the static stresses (always with no presence of water). It can be noticed how the effects of an earthquake with a larger time of occurrence can be almost the double in terms of forces and moments.

4.6 Final lining dimensioning

All the load combinations (static and seismic, with and without water load) were verified and the reinforced concrete was dimensioned according with

Figure 14. Results in terms of horizontal displacements in deformed shape with method 1.

Figure 15. Results in terms of horizontal displacements in deformed shape with method 2.

Figure 16. Static results as for Normal Forces, Bending Moments and Shear Forces for case A in shaft S12.

Figure 17. Comparison between static and seismic results as for Normal Forces obtained with the three applied methodologies.
the prescription given by Eurocodes and Turkish Standards. Figures 20–22 finally, only present examples of the ULS verification of the final lining of the shaft, joining together all the different beams in which the lining was modeled. In those cases water was taken into account.

In conclusion, according to the contents of the article, in the last presented figures there is only the “axial-force/bending moment” verifications carried out for the load combination with water pressure, in both the seismic cases S1 and S2. As for the “shear verification”, instead, the maximum shear force between the static and seismic conditions had been verified in each section of the final lining. From the result of structural analyses it is possible to define the quantity of reinforcing steel for the concrete sections of the final lining.

5 FUTURE DEVELOPMENTS

One possible development of the proposed method is to dispose three-dimensional modeling in order to take into account the effect of geological discontinuities along the tunnel longitudinal direction which can affect the stress and deformation in the lining.

In fact, just a simple difference in the value of the deformation modulus of the ground could sensibly change the seismic wave velocity in different stretches of the tunnel alignment. This would cause, in reference with the previously reported...
formulas, a differential shear deformation expected in consecutive blocks of the lining, thus causing an increase of stress on the concrete related to differential movements of successive portions of the lining.

Usually this effect is avoided through the design of appropriate structural and movement joints, very often filled with very deformable but impermeable material (such as fiberglass wool or similar), that allow a massive reduction of the stresses if they are properly conceived and installed at the correct distance one from the others.

Nevertheless, it is important to verify that each block is not affected by those differential shear and torsional deformations, but this would require an accurate knowledge of the geology at each meter of the alignment in order to make effective the three-dimensional analyses.

6 CONCLUSIONS

Istanbul Metro has been a successful experience of design-construction of large-scale infrastructure projects in a complex and dense urban context. The challenge of a seismic-resistant design was obliged by the specific critical location of the city, located in proximity of active faults.

In the past, seismic design of tunnel structures received considerably less attention than that of surface structures, perhaps because of the conception about the safety of most underground structures.

Despite this understanding, in the last few years, increasing interest is given to the seismic effect of underground structures, but a significant disparity exists among engineers in design philosophy, loading criteria and methods of analysis. The hope is that of coming as soon as possible to a codification of this subject as well as it have been happening in the field of the seismic effects on over surface structures.

REFERENCES


