SEISMIC DESIGN OF UNDERGROUND STRUCTURES
THE BOLU TUNNEL

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ABSTRACT: Traditionally low seismic risk is associated to tunnels and buried structures. Nonetheless collapses or heavy damages in underground structures have been reported following to Kobe and Loma Prieta earthquakes. More recently, following to Düzce earthquake, in Turkey, an almost 400 m long stretch under construction of the Bolu tunnel collapsed confirming that, in soft soils or close to important seismic sources, a seismic analysis of tunnels is advisable. Here the procedure followed for seismic upgrade of the design of the Bolu tunnel is exposed together with some main results.

1. GROUND SHAKING INDUCED LOADS FOR UNDERGROUND STRUCTURES

Main causes of seismic risk for underground structures can be liquefaction, active faults crossings, seismically induced landslides and ground shaking intensity. Damages induced at Bolu tunnels by the 12th November 1999 Düzce earthquake where mainly due to the elevated intensity of ground shaking.
Below the design procedure followed for Bolu Tunnel against ground shaking loads is summarized together with some preliminary outstanding results.
Concerning loads induced by ground shaking, some differences exist between above and below the ground structures: while surface structures are loaded mainly by inertial forces on the basis of structural amplification, in underground structures, due to the high grade of constraint, very low inertial forces are experienced. In table 1 the major conceptual differences in handling the two phenomena are resumed.
Seismic loads due to pure soil shaking in underground structures are thus induced by the relative displacements caused in the medium by the seismic wave propagation.
The possible loads induced by seismic waves on tunnels are due to:

- Seismic waves propagating in the longitudinal direction of the tunnel (see Figure 1a and b).
- Seismic waves hitting the tunnel in the plane of the cross section (see Figure 1c).

Seismic Loads to Structures due to Ground Shaking		
	Surface structures	**Underground structures**
Phenomenon	The soil surface vibrates and the structure oscillates causing inertial forces	The seismic waves propagating in the medium induce drifts in the structure.
Loads	Load introduced by seismic forces (proportional to accelerations)	Load introduced by seismic strains (proportional to velocities)
Interaction	In common problems is possible to neglect soil-structure interaction	Not possible to neglect soil-structure interaction
Analysis	In common problems linear analyses are possible	Highly non-linear approach

Table 1 – Differences in Seismic Analyses for underground and surface structures

While longitudinal loads (figures 1a and 1b) are generally not critical for overall stability (mainly radial cracks are induced and can be partly absorbed by radial joints in lining), the ovaling of the cross section is critical, and, affecting the integrity of the cross section, can induce the failure of the lining.

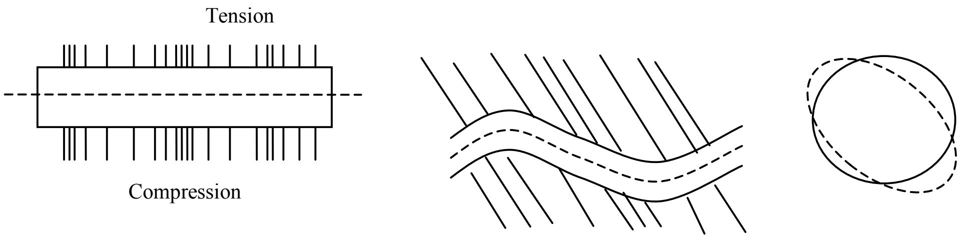


Figure a – Axial Compression Figure b – Axial Bending Figure c – X-Sect. Ovaling

Figure 1 - Seismic load on tunnel linings (Ground Shaking)

The ovaling of the cross section is induced by shear waves propagating within the soil from the vibrating bedrock toward the surface. The phenomenon can be sketched as in Figure 2.

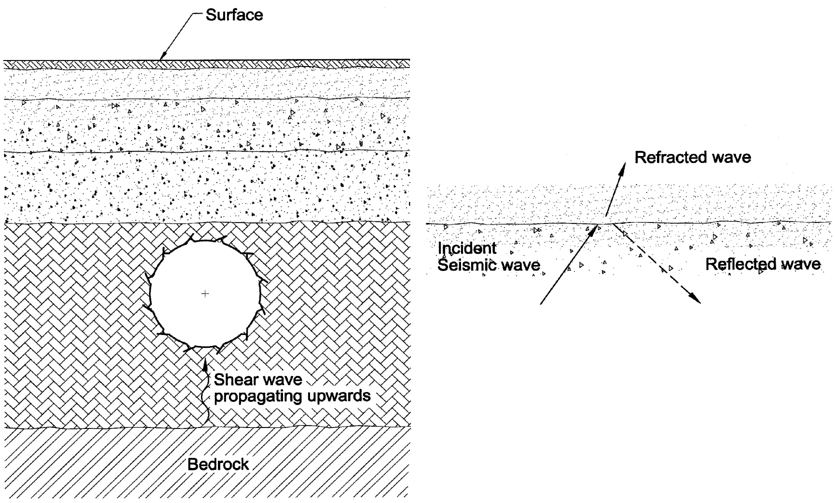


Figure 2 – Drifts induced by seismic shear.

The induced shear strain at the tunnel level depends on a huge amount of factors (i.e.: layering of the medium, shear modulus and density of the layers, damping ratio, inclination of layers boundaries, etc…). In particular, in a layered medium reflection and refraction of seismic waves can induce concentration of shear strains at certain levels. Therefore the evaluation of the shear strain induced by an earthquake at a certain depth cannot neglect the layering of the soil stratigraphy.

2. PROJECT DESCRIPTION

A portion of the Trans European Motorway (TEM) network, the Turkish motorway linking Ankara to Istanbul is currently under construction. The crossing of the Bolu Mountain is included in the 120 km section between Gümüşova and Gerede. The last 25 km stretch, the most challenging, runs parallel to the North Anatolian Fault. This stretch, under construction, includes several viaducts and the Bolu tunnel.
The Bolu tunnel is a 3360 m long twin bored three lanes tunnel.
The average excavated radius is 8 m, corresponding to an x-section area between 190 and 260 m^2. The width of the ground pillar between tubes varies from about 28 m at portals to almost 48 m. Depths are up to 250 m; on 86% of the total length, overburdens are higher than 100 m and on 48% higher than 150 m.

In Figure 3 the Bolu tunnel longitudinal profile is sketched and the lithology encountered is indicated. A wide range of soils is represented, mainly highly tectonised series of mudstones, siltstones and limestones, fault gouge clays. The consistence of the soil varies from competent rock requiring blasting to very weak clayey zones where heavy advancement problems were encountered.

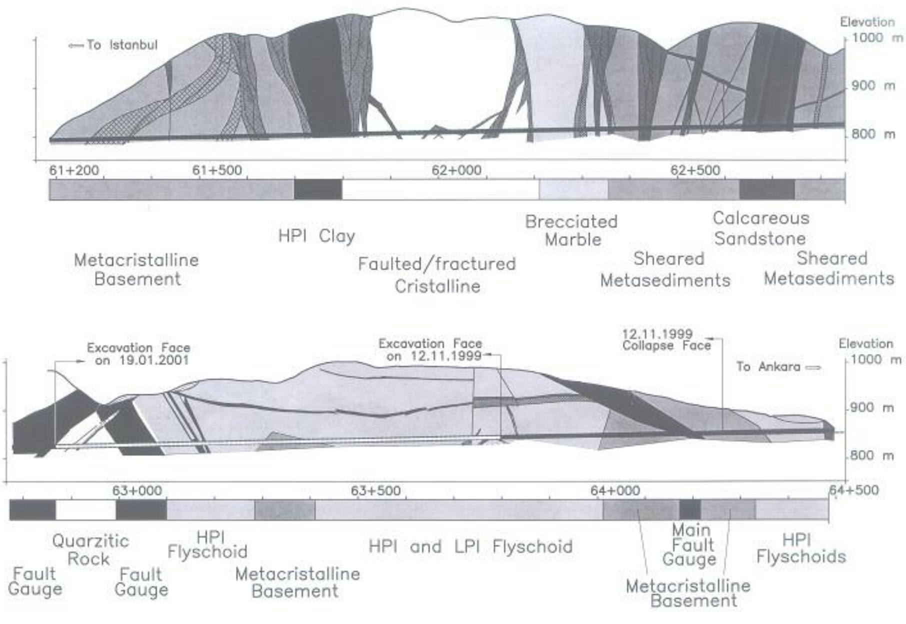


Figure 3 – Bolu Tunnel - longitudinal profile

The original tunnel design followed the NATM principles, proceeding with shotcrete, rock bolts and light steel ribs. While excavating, following to major tunneling difficulties, new heavier cross sections needed to be added to the original design.
The excavation was proceeding from both portals, western (Asarsuyu) and eastern (Elmalik). On 12.11.1999, stretches of about 1670 m and 730 m were already excavated respectively from Asarsuyu and Elmalik portals. A stretch of about 960 m was still to excavate and a reconnaissance gallery was excavated on the right tube (Elmalik face) whose face was only about 380 m from the opposite face.

3. DAMAGES DESCRIPTION

The area suffered the two major Turkish earthquakes of 1999 (see figure 4 for a seismic map of the area). Kocaeli (Izmit) Earthquake struck on 17.08.1999, with an M_w 7.4. The epicenter was located about 150 km west of the site. Close to tunnel site a PGA of 0.25÷0.35 g was recorded. No major damages were suffered by the tunnel.

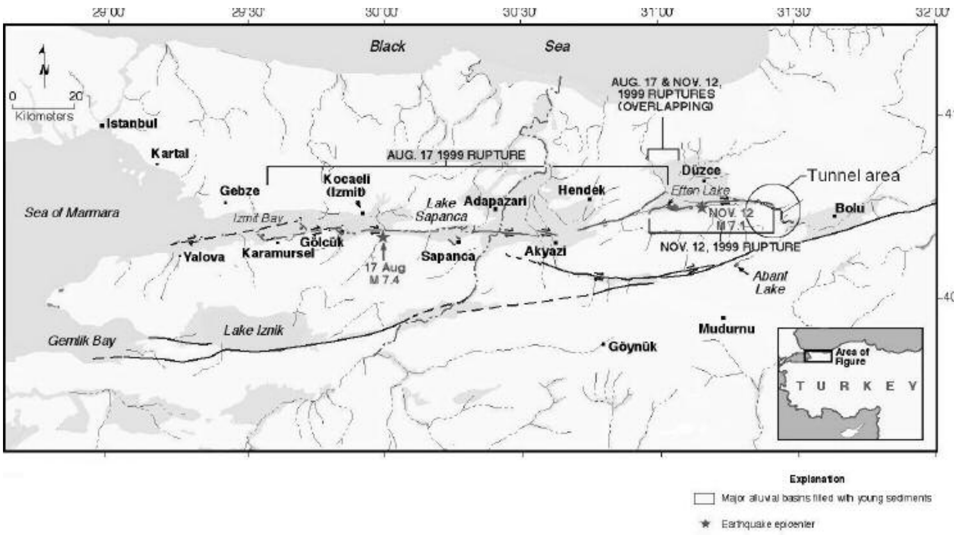


Figure 4 – Seismic map of Turkey close to Bolu area

Düzce Earthquake struck on 12.11.1999, the magnitude (M_w 7.2) was lower than Izmit one, but the epicenter was only 20 km west of the site. PGA and PGV of 0.81 g and 0.66 m/s were experienced at site; the surface rupture extended to 3 km from western portal. The tunnel suffered major damages and collapsed in a stretch of almost 400 m (figure 3).
The heavier damages have been recorded at Elmalik (Ankara) side where the above mentioned total collapse occurred in both tubes.
Also at Asarsuyu side damages to lining and invert uplift (in non reinforced stretches) were observed. The excavation, where a major fault gauge was being crossed, was proceeding by advancing two 5 m diameter pilot tunnels at benches (OPTION 4). These last were badly affected by the earthquake: the ribs buckled and invert heaved, requiring heavy re-profiling and reparations.

Other main consequences evidenced by the damage survey where:
1. Permanent increment of axial loads in lining that has been observed at monitored sections following to the Düzce event.
2. Important damages at shotcrete and temporary support were observed in soft ground stretches, where poor mechanic characteristics soils were encountered.
3. Among similar ground stretches, cracks at invert were recorded where invert was not reinforced.

Is worth here noting that the attained earthquake greatly exceeded the design earthquake in terms of PGA (0.81g against 0.4g). Also, following to the Izmit and Düzce events the motorway owner (Turkish General Directorate for Highways) required a revision of the design of the entire motorway to deal with a design seismic event of PGA of 0.81 g.

4. ANALYSES PERFORMED FOR BOLU TUNNEL

A two phases analysis has been performed for the Bolu tunnel seismic upgrade:
1. A “seismic screening” basing on simplified assumptions and elastic model,
2. A detailed 2-D analysis by Finite Difference Method considering the real behavior of the soil and shape of the structure.

4.1 Seismic screening

A “seismic screening” has been performed to evaluate critical sections in which more detailed analyses needed to be performed. The procedure, detailed in ref. 3, consists in applying an estimated seismic shear strain[1] to the soil surrounding the tunnel, and to deduce, moving from closed form solutions (i.e. hole in elastic half space) the thrusts, bending moments and shear forces induced in the lining. This method aims to estimate the maximum seismic load undergone by the tunnel lining.

Approximations driven by such an approach are mainly:
1. Shear strain in the subsoil only estimated, here the PGV has been estimated to 0.8 m/s by the statistical correlations for stiff soils (Shear wave velocity between 200 and 750 m/s).
2. The soil is considered elastic, (i.e. eventual rupture effects cannot be evaluated)
3. Layering of the soil is not fully taken into account
4. Tunnel shape is not modeled exactly (the tunnel cross section is assumed circular to adopt closed forms solutions).

[1] A wide literature is available about the methodology to evaluate the seismic shear induced in the subsoil by a seismic event. A rough estimation is the ratio between shear wave velocity C_s and peak particle velocity at tunnel depth:

$\gamma = \frac{V_s}{C_s}$ where $C_s = \sqrt{\frac{G}{\rho}}$ V_s is the peak ground velocity at tunnel depth

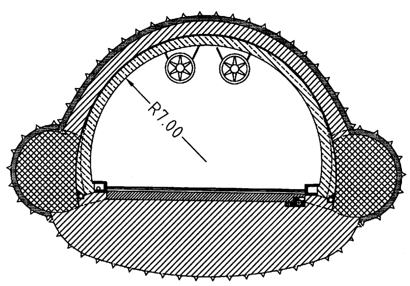
ρ is the density of the soil
G is the shear modulus of the soil

A better evaluation of the induced maximum shear strains is attained by performing a linear equivalent analysis on a resonant column modeling the soil stratigraphy.

The simplified method estimates the loads from closed form solutions depending mainly from Young's modulus of soil and lining, diameter of the tunnel, and inertia of the lining cross section.
From the seismic screening of the Bolu tunnel the main result has been that, in soils where a static shear modulus (G) of less than 350 MPa was encountered, loads induced by the design seismic event require some fiber reinforcement in final lining. Also it has been decided to perform further detailed analyses in stretches where the static G modulus was lower than 230 MPa.

4.2 Detailed FE analyses

At critical sections detailed fully non-linear analysis have been performed. The stretches considered critical were the ones where significantly high loads have been evaluated on the base of the seismic screening (Gst ≤ 350 MPa).
In these critical stretches, the cross sections adopted are OPTION 3 and/or OPTION 4 (see Figure 5). These two cross sections are characterized by a huge monolithic invert and three crown layers (shotcrete, intermediary and final). In OPTION 4 backfilled bench pilot tunnels have been adopted.
To perform detailed 2-D FE analysis the following items were defined:



Option 3 Option 4

Figure 5 – OPTION 3 and OPTION 4 Support measures

4.2.1 Accelerograms suitable in the area.

A complete seismic hazard study was not available for Bolu area by the time the analysis begun, therefore, the definition of the earthquake record to be used was made by selecting 2 real accelerograms and 1 synthetic accelerogram from the 11 records proposed by the consultant geologists for the seismic upgrade of the viaducts in the area close to the tunnel. The selected records are:

1. The Bolu station record of Düzce event.
2. The Loma Prieta record.
3. One simulated earthquake, appropriately corrected to account for site response.

The Loma Prieta accelerogram was selected on the basis of the similitude its site with the Bolu site. All the accelerograms have been scaled and corrected to account for design PGA, directivity problems, depth and frequency contents.
The main seismic source of the area has been estimated to be the North Anatolian Fault; directivity and near field effects have been accounted for on this basis.

4.2.2 Model, mechanic and dynamic properties of the soil layers.

As known, a decay of G modulus is shown against shear strain. The G modulus decay law has been specified on the basis of a pressiometer test campaign held at site. Here G_{max} has been reduced by a rate according to the expected induced shear strain.
The steady state after the excavation has been reproduced, the disturb induced by the excavation has been accounted for in terms of higher compressibility in the plastic zone.
The mechanical parameters of the different soils in the stratigraphy have been evaluated on the basis of testing campaigns previously held in similar soils crossed by the tunnel.
In fully non-linear model the damping of the soil is accounted for histeretically. A Rayleigh damping percentage has been specified (to about 5÷9%) by referring to relationships damping-shear strains proposed in literature.
Boundaries have been modeled as "absorbing surfaces" not to reflect into the model incident waves. Model dimensions have been adjusted on the basis of sensitivity checks. The dimensions of mesh elements have been adjusted on the base of the predominant frequencies of the problem.
Ductility of the section has been accounted for specifying limiting plastic moments and plastic hinges pattern.

4.2.3 Analysis

For grid dimension and layering refer to Figure 6 where also plastic zones after excavation and after the earthquake are indicated.

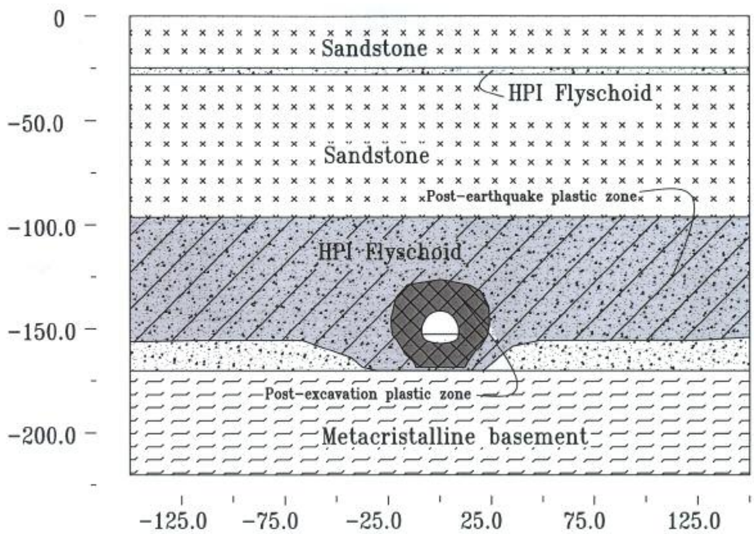


Figure 6 – Model grid, layering and extension of the plastic zones, after excavation and after the earthquake.

Once initialized the static state after the excavation, the disturb due to excavation has been modeled by reducing the compressibility parameters in the plastic zone to 80% of the undisturbed.

The corrected accelerogram has been applied at the bedrock level (i.e. base of the grid). The design earthquakes have induced maximum shear strains at tunnel level up to 0.6÷0.7%.

5. PRELIMINARY RESULTS

These analyses are still under development, but, from preliminary results, some major considerations can be made.
The analyses results match site observations, and show that the induced shear strain peak (up to 0.6÷0.8%) would cause the compressible soil layer, where the tunnel is excavated, to undergo plastic deformations during the earthquake.
This causes degradation of the mechanical properties of the soil, and was not accounted for by the simplified screening procedure, where loads after the earthquake are supposed to get to the pre-earthquake level. This seems to be the one major cause of error in the procedure adopted for the screening.
As can be seen in Figure 7, a significant increment in axial loads and bending moments acting on lining is shown when the soil failure is attained. If the soil remains elastic and no failure is induced (for less extreme earthquakes) the maximum load level seems to be roughly well estimated by the screening procedure.
Is worth noting that a check made with ground reaction curves method allowed to justify qualitatively this increment. In figure 8 are shown the two ground reaction curves of the soil with the strain-softening model (pre-earthquake situation), and of the soil with the residual parameters (post-earthquake). The possible equilibrium zone reduces, and the equilibrium point moves from pre-earthquake mechanic characteristics (i.e. point A) to post-earthquake ones (i.e. point B) on almost constant convergence level. This induces a significant increment of axial loads similar to the one recorded in the FEM detailed analysis.

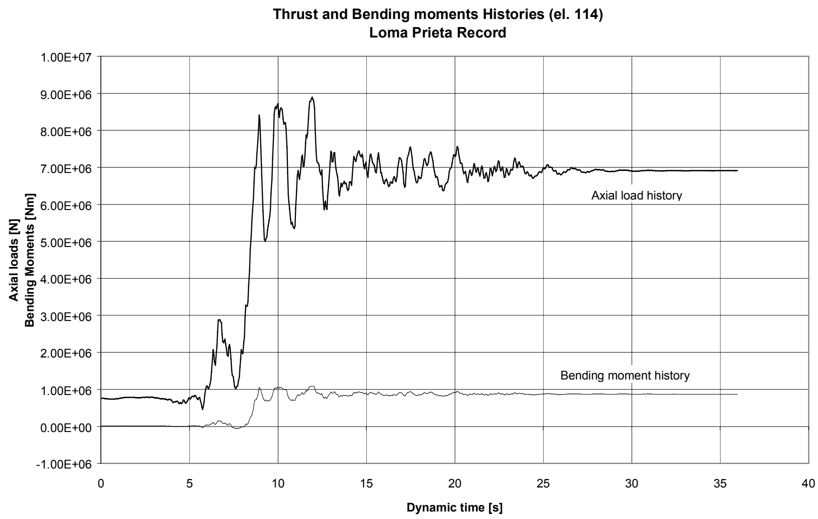


Figure 7– Typical thrust and bending moment history in inner lining

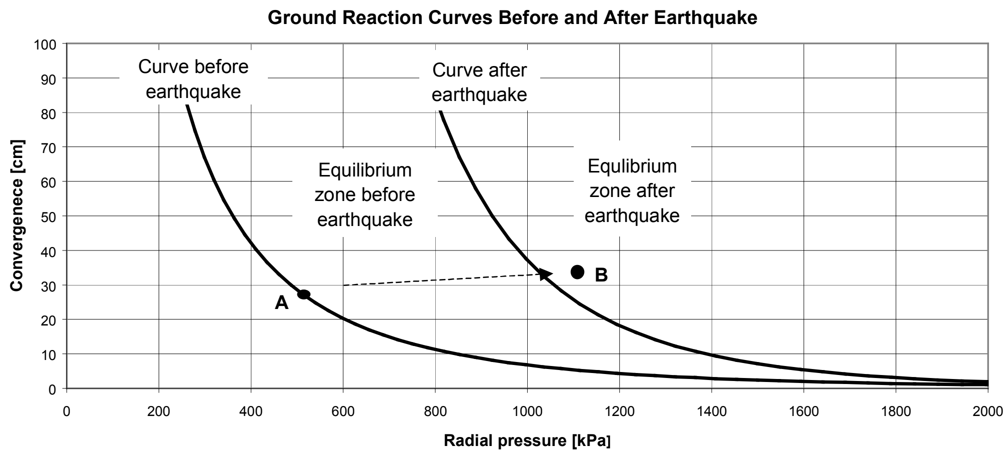


Figure 8 – Ground reaction curves before and after the design earthquake

6. CONCLUSIONS

The post earthquake site observations have been confirmed by the preliminary results of the 2-D FEM analyses. A significant permanent increment of loads has been observed after a major earthquake. This is due to the soil being sheared by the earthquake significantly beyond the elastic peak inducing a reduction of the resistance parameters. This increment was not evidenced by the simplified screening procedure, where the soil was modeled as remaining elastic, but can be at least estimated on the basis of simplified elastic-plastic methods (i.e. confronting the ground reaction curves and possible equilibriums in virgin and sheared soil).

By the seismic analysis of the Bolu tunnel has been derived that a certain grade of reinforcement is needed in the inner lining and at the invert to avoid failure following to the design seismic event.

The simplified screening procedure appears to be satisfactory for a wide series of cases where shear strains levels do not induce reaching the peak resistance of the soil. If significant shear strains are expected (i.e. close to the seismic source, or for deep tunnel in particularly poor grounds…) is advisable to perform more detailed analyses accounting for the effective constitutive model of the soil.

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