Bolu Tüneli Detaylı Sismik Analizleri

Detailed Seismic Analyses for Bolu Tunnel

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ÖZET Gümüşova-Gerede Otoyolunda yer alan Bolu Tüneli 1999 Düzce depreminin merkezine 20 km mesafededir. Deprem sırasında yapım çalışmaları devam eden tünelin Elmalık portal bölgesinde ön kaplama ile desteklenen kesimde yüksek şiddetli yer sarsıntısı nedeniyle göçük meydana gelmiştir. Göçük bölgesinin jeolojik yapısı yüksek plastisiteli flişler ve düşük açılı fay zonundan oluşmaktadır. Bu bölge statik yükler altında yüksek deformasyon ölçülen ve tarama gerektiren kesimleri içermektedir. Bolu Tüneli sismik dizaynı Düzce depremi verilerine göre gözden geçirilmiş ve revize edilmiştir. Proje gereklilikleri depremin iki etkisine, yer sarsıntısı ve aktif fay geçişine göre belirlenmiştir. Bu bildiri yer sarsıntısı dikkate alınarak gerçekleştirilen Bolu Tüneli sismik projelendirme aşamalarını içermektedir. İlk aşamada basitleştirilmiş yöntemlerle sismik değerlendirme yapılmış ve detaylı sismik analiz için kritik kesitler seçilmiştir. Seçenek 3 ve 4 destekleme sınıfları için sonlu elemanlarla dinamik analiz gerçekleştirilmiş ve tünel kaplaması yükleme durumu belirlenmiştir.

ABSTRACT Bolu Tunnel situated at the most important section of Gümüşova-Gerede Motorway is located 20 km east from epicenter of Düzce earthquake at 1999. Tunnel tubes driven from Elmalık portal and supported with primary lining collapsed due to the high intensity of ground shaking. Geology of the collapsed part consists of flyshoids with high plasticity and fault zone with low dip angle. Low mechanical properties of the fault zone material flyshoid resulted in excessive deformation due to static loads and reprofiling required. Seismic Design of Bolu Tunnel was reviewed and revised with consideration of new data after Düzce earthquake. Design requirements were determined according to two influences of earthquake, ground shaking and active fault crossing. This paper consists of the seismic design stages according to ground shaking. At the first stage seismic screening was performed according to the simplified methods and critical sections were chosen for detailed seismic analyses. Finite Element dynamic analyses were performed for the sections with support classes of Option 3 and 4 and state of load induced in the tunnel lining was determined.

1 INTRODUCTION

Gümüşova-Gerede section of Istanbul-Ankara Motorway is located at the influence zone of North Anatolian Fault. This section of motorway encountered damages during 1999 Düzce earthquake. Bolu Tunnel, which has a distance of 20 km to earthquake epicenter, collapsed at the parts in flyshoid with high plasticity and fault gouge. The collapsed part of the tunnel was supported with primary lining and required reprofiling due to excessive deformations under static loading.

Investigations showed that the collapsed part of the tunnel extend from Elmalık portal to the area consisting pilot tunnel. Re-excavation of the tunnel in collapsed area would require ground improvement before excavation, slow advance and high density of tunnel support. Tunnel advance had been carried out along the by pass alignment. Seismic design of the by pass section was performed according to the updated seismic data and characteristics of the ground. For the excavated parts seismic design was reviewed and revised in required sections.

The aspects of the design philosophy were determined as follows;

- Seismic screening was performed with simplified methods to determine the critical sections requiring detailed analyses.
- Site-specific seismic report was updated according to the new seismic data of Düzce earthquake.
- Thickness of the inner lining and Bernold lining was kept constant. Reinforcement in the lining was determined to carry seismic loads.
- Seismic screening and detailed analysis were performed with methods given in FHWA manual

2 SEISMIC SCREENING

The screening procedure is based on closed form solution involved approximations:

- Circular lining
- Estimation of shear strain with empirical relationship
- Uniform elastic soil profile

One of the parameters required for screening procedure is soil shear modulus. Soil shear module referring to the static values considers large shear strain and underestimates seismic shear modulus corresponding to the low deformation induced by the earthquake. Seismic shear module was estimated with an iterative procedure considering the diagram in Figure 1.

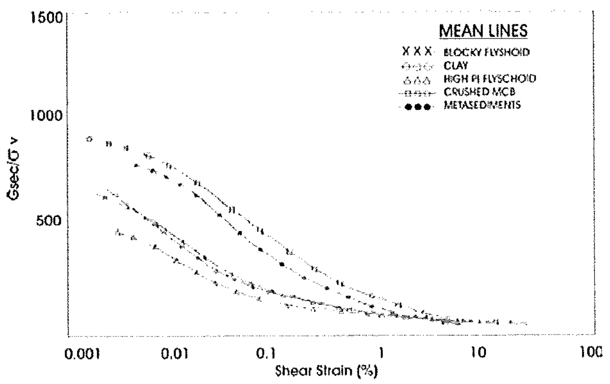

Figure 1. Diagram for determination of the shear modulus (Yüksel-Rendel Report, 2000)

The design peak ground acceleration (PGA) of 0.81 was scaled to account for the tunnel depth. Peak ground velocity (PGV) corresponding to the PGA was determined. An induced shear strain for the design earthquake was estimated and corresponding shear modulus was determined from the diagram in Figure 1. The corresponding estimated shear strain was checked with the one determined from the formula 1. Iteration was performed until estimated shear strain was verified with the calculated one and corresponding shear modulus was determined.

$$\gamma s = PGV / \sqrt{Gs / \rho} \qquad (1)$$

where PGV= peak ground velocity
Gs= shear modulus
ρ=density of the medium

The maximum seismic loads were determined with the formulations given in Table 1.

In the first phase three linings, shotcrete, Bernold and inner lining were considered as resistant. In the second phase if yielding is induced in shotcrete and Bernold lining, they don't contribute to the bending stiffness of the crossection.

Bending moments were reevaluated to get the final loads expected to act on inner lining that will remain elastic under the seismic loads. The ultimate resistance (without safety factor) was considered for shotcrete and Bernold lining, while at inner lining the resistance was given by considering factor of safety 1.5 for concrete and 1.15 for reinforcement.

The static axial loads to the linings were considered from static design calculations. Finally seismic loads were superposed to static axial loads to check the adequacy of the linings. Moment-Axial load envelopes (M-N envelope) were drawn for each lining. Those envelopes show that bernold lining yield and load on the inner lining should be evaluated for Option 4 (Figure 2).

Table 1. Formulations for the determination of maximum seismic loads (Penzien and Wu 1998)

Maximum cross section loads		
Max seismic axial load	$N=2*G_m*D*K*\gamma_{smax}$	$K=(1-\nu_m)/(F+3-4*\nu_m)$
Max seismic shear load	$S=2*G_m*D*K*\gamma_{smax}$	$F=G_m*(1-\nu_l^2)*D^3/(24*E_l*I_l)$
Max seismic bending moment	$M=1/2*G_m*D^2*K*\gamma_{smax}$	
ν_m:soil poissons ratio ν_l: lining poissons ratio G_m : soil shear modulus	D:cross section diameter E_l: elastic modulus of lining I_l: inertia of lining cross section	

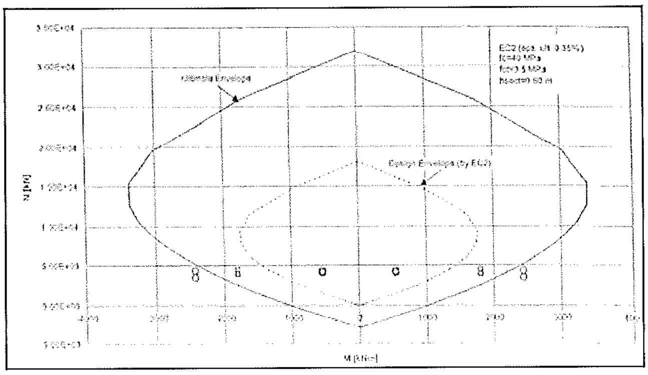

Figure 2. M-N envelope for bernold lining in Option 4 (Lombardi 2002)

While bernold lining yielded in the calculations for the first phase, the inertia of the cross section was reduced to the one given by shotcrete and final lining. The bending moments was carried by shotcrete and inner lining, while the axial loads were still carried by the three linings.

The final lining had to be designed elastically withstand the maximum load expected during the earthquake. The reinforcement in inner lining was determined according to the total loads. To design tunnel lining detailed dynamic analysis were performed with numerical analyses.

3 DETAILED DYNAMIC ANALYSES

3.1 Input ground motion

Input ground motion is one the most important parameter in detailed dynamic analysis. To match seismic activity of the tunnel site a large number of accelerograms were evaluated. They were corrected to account directivity effects and depth of bedrock. Three records were selected for the analyses:

- Bolu station record of the Düzce earthquake
- A synthetic earthquake
- A Loma Prieta earthquake

Loma Prieta earthquake record is shown in Figure 3.

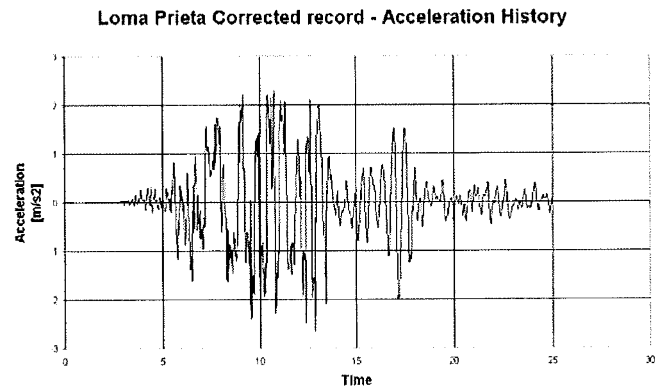

Figure 3. Loma Prieta corrected acceleration record (Lombardi 2001)

3.2 Soil profile

The variation of the soil profile from ground surface to the bedrock was considered in the numerical simulations. The soil profile for chainage 63+550 is shown in Figure 4. For sandstone layers Coulomb model, for soil layers such as clay and flyshoid with high plasticity strain softening model were used. The bedrock modeled elastically. The parameters for each layer are indicated at soil profile in Figure 4.

3.3 Numerical Model

The boundaries of the numerical model for the analyses were considered in two phases. Firstly to initialize the system initially horizontal displacements were fixed at side boundaries and vertical displacements were fixed at bottom boundary. At the second phase for dynamic analyses the boundaries were modeled as free field, which means that the grid elements at the boundary were attached by a spring dashpot system to a fixed surface. The external boundaries absorbed any incident wave and did not reflect it into the grid. The bottom boundary was fixed vertically

The lateral side pressure coefficient was set to 1 and vertical acceleration to 10 m/s^2. The soil layer below the groundwater table at –60 m was considered as saturated with a permeability of 10^{-10} m/s and porosity of 0.5. The groundwater flow was allowed only after completing the excavation.

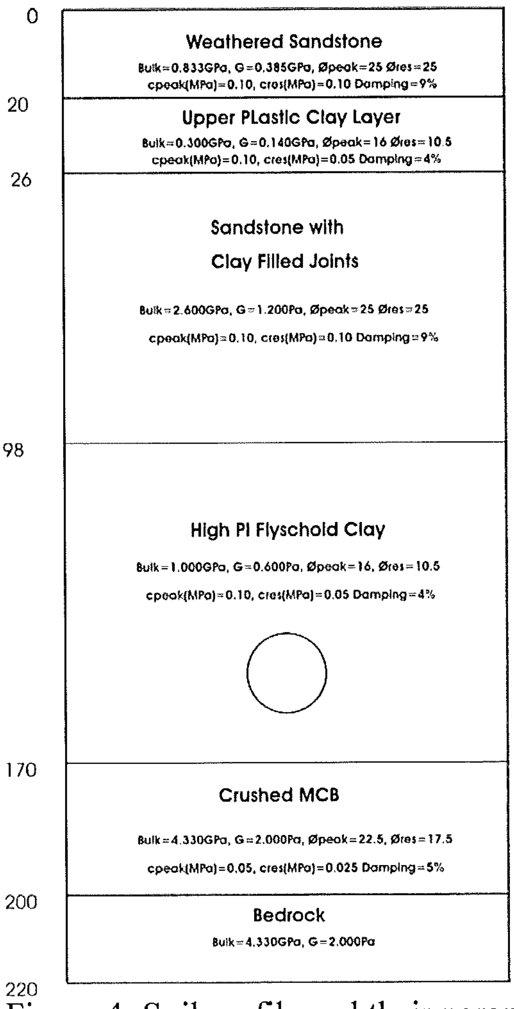

Figure 4. Soil profile and their parameters at chainage 63+550

3.4 Monodimensional Analyses

The design parameters in each layer were another important point in the seismic analyses. In addition to specifying the appropriate undisturbed shear modulus Gmax, and shear wave velocity Vs, it is also important to account for degradation of shear modulus and increase in damping that accompanies cyclic shear strain. The undisturbed shear modulus for each layer was determined according to stiffness-strain relationships obtained from the pressuremeter tests or according to the shear wave velocities and effective stresses.

The ground responses resulting from the application of ground motions were obtained by computer program SHAKE91. Although Shake91 was a 1-D program and it could not account for the presence of the tunnel, results provides valuable insight pertaining to the ground response. As a result of SHAKE91 analyses, it was seen that, significant degradation of the shear modulus may occur within the flyschoid clay layer. Monodimensional analyses were performed on the soil column with the Finite Difference program FLAC to asses the compatibility of the FLAC model with SHAKE analysis and to define damping ratio and Young modulus by confronting the shear stress and strain attained in the model. In this analysis the mechanical parameters, stresses and groundwater conditions were initialized and earthquake for three different cases was applied as acceleration record at the base of the model. The analysis gave the shear strain histories, shear stress at different depths and plastic zones after the earthquake. One of the results of monodimensional analyses was the plots of the degraded G/Gmax profiles with depth for three strong ground motions (Figure 5). Analyses indicated that Loma Prieta earthquake gave the most severe in terms of attained shear strains.

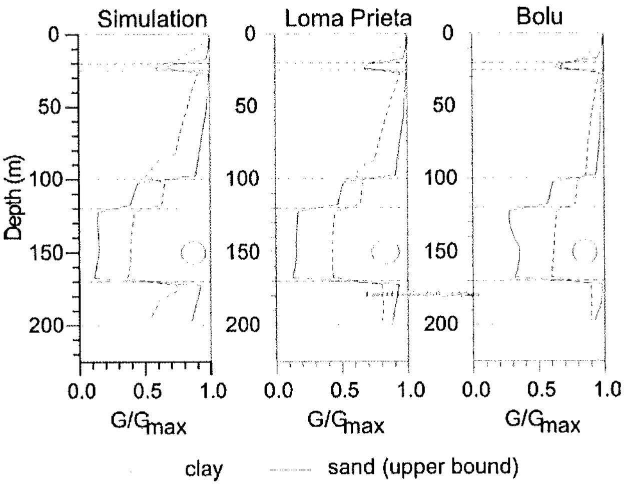

Figure 5. Variation of G/Gmax with depth for flyshoid clay layer (Lombardi 2001)

3.5 2 Dimensional FLAC Analyses

With seismic screening, sections at Km: 63+400 and 63+680 were assessed as critical and finite element analyses were performed with FLAC for both support classes of Option 3 and 4.

Layering and parameters of soil, groundwater conditions and boundaries were same as defined in the former parts of the paper. The design philosophy was same that outer linings might reach to yielding threshold while inner lining will be able to withstand earthquake by remaining elastic.

Static stress state for K=1 and groundwater situation were initialized before excavation. Tunnel was excavated in nondrained condition (no drainage). For long term equilibrium of tunnel groundwater pressure was allowed. The disturbed soil around the tunnel was modeled by reducing the compressibility modulus to 80 % of the original. Ground motion was applied to the base of the mesh as velocity.

3.5.1 Support Class Option 3

Figure 6 shows the cross section of Option 3. The calculation steps for support class Option 3 and explanations are given in Table 2.

At dynamic analysis the most critical results were obtained from Loma Prieta earthquake. Higher bending moments were concentrated and the bottom of the crown at the invert junction. The histories for loads at the linings and invert were determined and reinforcement was designed accordingly. The reinforcement was concentrated in this zone at the inner part of the section, while at the top of the crown a lighter reinforcement was adopted. To increase the ductility of bernold and inner lining a light steel fiber dosage was suggested.

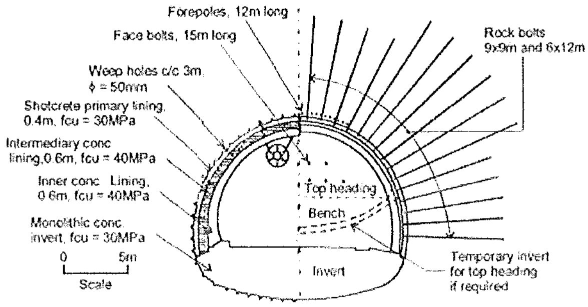

Figure 6. The cross section of support class Option 3. (Tokgözoğlu, F. & Işık, S., 2002)

Table 2. Formulations for the determination of maximum seismic loads

STEPS OF ANALYSIS FOR OPTION 3		
Step	**Type of Analysis**	**Explanation for the analysis**
1	Static undrained	Initialization of hydrostatic stress state, groundwater table
2		Excavation of top heading
3		Installation of shotcrete lining with 50 cm thickness and temporary invert
4		Installation of intermediary lining with 60 cm thickness
5		Installation of the final lining with 60 cm thickness
6		Drainage, long term static stress state after excavation
7	Static drained	Reduction of the compressibility in plastic zone to model the disturbance with excavation
8	Dynamic undrained	Application of Bolu record from step 11
9		Application of simulated record from step 11
10		Application of Loma Prieta from step 11

3.5.2 Support Class Option 4

Figure 7 shows the cross section of Option 3.

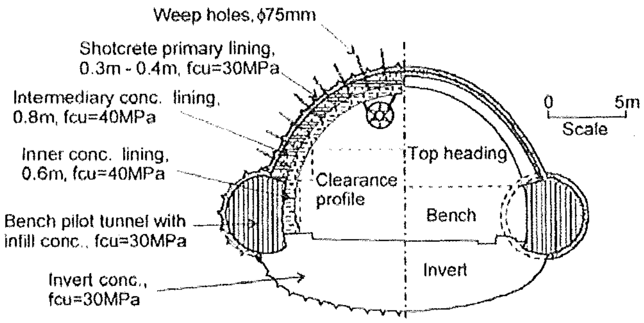

Figure 7. The cross section of support class Option 4. (Tokgözoğlu, F. & Işık, S., 2002)

The calculation steps for support class Option 4 and explanations are given in Table 3.

As the analysis for Option 3 the most critical results were obtained from Loma Prieta earthquake. It induced an offset of 15 cm. Results showed that permanent loads arise in inner lining due to the earthquake and reinforcement in inner lining shall allow withstanding those loads. The reinforcement at the invert for static loading was also adequate for dynamic loading. To increase the ductility of bernold and inner lining a light steel fiber dosage was suggested.

Table 3. Formulations for the determination of maximum seismic loads

STEPS OF ANALYSIS FOR OPTION 4		
Step	**Type of Analysis**	**Explanation for the analysis**
1	Static undrained	Initialization of hydrostatic stress state, groundwater table
2		Excavation of bench pilot tunnel
3		Backfilling of bench pilot tunnel with concrete
4		Excavation of top heading and installation of temporary invert

Table 3. Continue

5	Static undrained	Installation of shotcrete lining with 30 cm thickness
6		Installation of intermediary lining with 80 cm thickness
7		Excavation of the invert
8		Concreting of the monolithic invert
9		Installation of the final lining
10	Static drained	Drainage, long term static stress state after excavation
11		Reduction of the compressibility in plastic zone to model the disturbance with excavation
12	Dynamic undrained	Application of Bolu record from step 11
13		Application of simulated record from step 11
14		Application of Loma Prieta from step 11

4 CONCLUSION

Ground motions with high intensity induce permanent loads to tunnel lining in soft rock/soil conditions. Simplified methods and detailed analyses enable to perform the first evaluations and seismic design respectively. In Bolu Tunnel loading to the tunnel lining due to the ground motion was carried by reinforcement so that the thickness of the lining was kept constant through the tunnel. Results of the dynamic analyses showed that, the reinforcement in inner lining for option 4 was adequate for dynamic loading. On the other hand, new reinforcement design was made for inner lining for option 3, especially taking into consideration the concentrated moments at the bottom crown at the invert junction. Additionally a light steel fiber dosage was used in bernold and inner lining for both support systems in order to increase ductility.

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