SEISMIC DESIGN OF THE STATIONS AND THE INTER-STATION TUNNELS OF A METRO-LINE IN SOFT GROUND: A CASE STUDY

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ABSTRACT

The seismic design of underground structures is controlled by ground deformation and soil-structure interaction, whereby kinematic effects dominate over inertia effects. Available methods of numerical analysis for the 2D response in the transverse direction are applied for a case study referring to the stations and the inter-station tunnels of a metro-line in a seismic region. Two continuum approaches are considered: quasi-static and time-domain dynamic finite-element analyses. Sectional forces are determined for a typical earthquake shaking. They are compared against each other and to those obtained from the static design of the excavation support.

Keywords: underground stations; tunnels, deformation method, numerical analysis

INTRODUCTION

Underground structures, such as metro-line stations and the inter-station tunnels, are generally considered less vulnerable to seismic actions than above ground structures. In contrast to surface structures, whose seismic response is governed by inertia effects, the response of these structures is primarily kinematic, i.e. it is caused by the compatibility of deformations to the surrounding ground. Therefore, soil-structure interaction effects are of fundamental importance. The analysis is usually carried out by imposing appropriate displacement patterns (deformation method). Inertia effects are of secondary importance and can be neglected. The available analysis methods are presented by St. John & Zahrah (1987), Wang (1993), Kawashima (2006). The majority of papers refer to circular or rectangular tunnel structures. Wide or deep underground stations have seldom been considered in the literature, due to the variability in the system geometry and the soil depth-profile making it difficult to derive simple methodologies for the design.

A further point of interest during the design of underground stations refers to the comparison of the sectional forces for the static and the seismic case. For the static case, the critical situation occurs during the various construction phases, while for the seismic case the completed structure is considered. Due to the different safety factors applicable to the actions, the seismic case becomes relevant only for large seismic excitation levels.

The aspects mentioned above are demonstrated for the design of the stations and inter-station tunnels of a metro line constructed in a region of moderate to high seismicity. The stations are constructed in a dense

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populated area in a top-to-down excavation method. Due to restrictions in space, a single-shell structure was necessary without any central support of the roof-slabs. The structural design was governed by the high groundwater level and the relatively poor soil conditions as well as by the presence of vulnerable buildings in the close neighborhood. The inter-station tunnels were constructed by an earth pressure balance tunnel boring machine.

In the sequel, the results of the static analysis are first summarized in terms of sectional forces. The analysis of the station is then presented using two methods: a dynamic 2D analysis of the soil-structure interaction problem and a quasi-static analysis using imposed displacements as obtained from a seismic site response analysis. The two methods are then applied to the design of the inter-station tunnels. Conclusions are drawn for the suitability of the methods.

SOIL CONDITIONS AND SEISMIC ENVIRONMENT

The section used in the frame of the present case study consists of sedimentary deposits underlain by the neogene, sandstone-marl. They are divided into the units A1 and B. The formation A1 is further subdivided into three sub-units: soft to firm, sandy clays of low plasticity, clays and silts (A1a); medium dense clayey / silty sands gravels, with intercalations of medium dense, clayey / silty gravels with sand (A1b); firm to stiff sandy clays of low plasticity, with local intercalations of medium dense to dense clayey / silty sands with gravels (A1c). Unit B consists of stiff to very stiff calcareous sandy clays of intermediate to high plasticity, with local intercalations of dense clayey / silty sands. Groundwater was found at 5 m below ground surface.

The elasto-plastic Hardening Soil Model as implemented in the code PLAXIS (Brinkgreve et al., 2010) is adopted for the design. Bending moments are defined counter-clockwise positive, axial forces are positive for tension.

For the dynamic analyses, a continuous variation of the small strain shear modulus $G_{\text{max}}$ is selected for all layers as derived from empirical relationships:

$$G_{\text{max}} = 9500(\sigma_v')^{0.5}$$

(1)

where $\sigma_v'$ in kPa is the vertical effective stress. $G_{\text{max}}$ is given in kPa.

The unit weight is set equal to 20 kN/m$^3$ for all layers. For the shear modulus reduction $G/G_{\text{max}}$ and damping ratio $D$ increase with shear strain $\gamma$ typical curves for medium plasticity clays are selected for all layers. They are defined by the following equations:

$$\frac{G}{G_{\text{max}}} = \frac{1.03}{1 + 6.457\gamma^{0.8}} \leq 1 \quad ; \quad D = 2 + \frac{19.5}{1 + 0.204\gamma^{-0.8}}$$

(2)

with $\gamma$ and $D$ given in %.

The site is characterized by moderate to high seismicity. In this study we use only one seismic record. The record selected refers to the Loma Prieta 1989 Earthquake recorded at Palo Alto SLAC Lab, USGS Station 1601 with site conditions corresponding to rock. The record exhibits in the predominant horizontal
direction: peak ground acceleration (PGA) = 0.278 g, peak ground velocity (PGV) = 29.3 cm/s, and peak ground displacement (PGD) = 9.72 cm.

ANALYSIS FOR STATIC LOADS

Underground station
For the underground station, the design for permanent static loads considered the various excavation stages of the top-to-down construction including installation of diaphragm walls, construction of roof slab, lowering of the water level inside the pit, construction of the two intermediate slabs, and construction of the base slab. In order to avoid the installation of multiple struts at the lowest excavation level, a grouted strut is constructed in the soil just below the final excavation depth. The resulting system is displayed in the next sections and has the following geometry: roof slab at a depth of 3 m below ground surface, followed by two slabs at 8 and 13 m depth; base slab at 22 m depth. The tips of the diaphragm walls are located at a depth of 45.5 m. The structural elements have the following thicknesses: diaphragm walls 1.20 m, base slab 2.0 m, roof slab 1.0 m, intermediate slabs 0.80 m. The Young’s modulus of reinforced concrete is set equal to 30·10^6 kN/m^2, and the Poisson’s ratio equal to 0.15.

The maximum bending moments in the diaphragm walls are obtained at the stage just preceding the construction of the base slab and amounts to $M_{slab} = 3342$ kNm/m. After construction of the base slab and activation of the groundwater pressure the maximum bending moments are: $M_{wall} = 1360$ kNm/m in the diaphragm walls, and $M_{slab} = 1573$ kNm/m in the base slab.

Twin tunnels
The twin-tunnels of the inter-station sections were also analyzed by the code PLAXIS using an appropriate emulation of the 3D arching effects. The cross-interaction of the tunnel structures is included by considering the excavation sequence. The tunnel diameter is 5.80 m, and the center-to-center distance is 12 m. Tunnel axis is located at a depth of 19 m below ground surface. Tunnel lining has normal stiffness $E_I A_I = 10.5·10^6$ kN/m and flexural rigidity $E_I I_I = 39.38·10^6$ kNm^2/m. Poisson’s ratio of the lining is set equal to 0.15. The system is displayed in the next sections. It is assumed that the tunnel on the left-hand side is excavated first. The elasto-plastic analysis yielded the following values for the sectional forces in the lining: a) Excavation of the left-hand side tunnel: axial force $N_I = 970$ kN/m, bending moment $M_I = 49$ kNm/m; b) Both tunnels excavated: $N_I = 1080$ kN/m, $M_I = 57$ kNm/m for the left side, and $N_I = 985$ kN/m, $M_I = 45$ kNm/m for the right side tunnel.

1D SEISMIC SITE RESPONSE ANALYSIS

The code EERA (Bardet et al., 2000) was chosen for this purpose. The profile investigated has a thickness of 80 m and is discretised as follows: top layer of 0.50 m, underlain by 10 of 1 m, 10 of 3 m, 1 of 3.5 m, and 9 of 4 m. The ratio of effective to average strain is set equal to 0.5. The computed results in terms of maximum shear strain, normalized shear modulus, damping ratio, and maximum acceleration are given in Figure 1. The acceleration time history at the interface soil/bedrock is plotted in Figure 2 and exhibits a peak value of 0.178 g.
2D DYNAMIC RESPONSE ANALYSIS

The analysis is performed using PLAXIS. The model is 80 m thick, while the lateral extent is set equal to 640 m in order to minimize the influence of the boundary conditions. The domain is partitioned into 7 layers with a linear distribution of soil stiffness within each layer. The layer depths are 2 m, 7 m, 15 m, 24 m, 26 m, 54 m, and 80 m. The shear modulus depth-profile approximates the distribution of the strain-compatible modulus obtained from the 1D free-field analysis with the following values at the surface and at the layer interfaces: 20.5, 51.7, 84.7, 92.8, 109.6, 141.0, 153.7, and 188.9 MPa. The Poisson’s ratio is set equal to 0.25.

The time-domain algorithm uses frequency-dependent Rayleigh damping for the soil. The Rayleigh damping coefficients $\alpha_R$ and $\beta_R$ to simulate the strain-compatible damping ratio $D$ of the 1D frequency domain analysis are obtained by adopting the calibration procedure suggested by Amorosi & Boldini (2009). Fourier
spectra computed from the EERA analysis at different depths are used to define the frequency interval for the
determination of $\alpha_R$ and $\beta_R$. Here an interval between 0.8 Hz and 2.6 Hz is selected, yielding $\alpha_R = 7.65 \cdot 10^{-2} \cdot D$, and $\beta_R = 9.5 \cdot 10^{-4} \cdot D$, with $D$ given in %. In the seven soil layers $D$ has the following values in %: 4.0/5.2/7.1/8.2/7.8/7.8/8.0.

The base excitation is uniformly distributed along the 400 m wide central region of the model, and tapered in
the outer regions with vanishing values at the side boundaries that are fixed horizontally. This configuration
with viscous boundaries is shown to minimize spurious effects of reflection at the lateral sides, Visone et al.
(2009). In the region around the structure the mesh is refined accordingly. The grouted strut underneath the
level of the base slab is not taken into account and is replaced by the soil at that level. The model is shown in
Figure 3.

![Figure 3. Model for the 2D dynamic analysis of the station with shear modulus profile](image)

The results of the time-domain dynamic analysis with PLAXIS without any structure installed show a good
agreement with the EERA results. For example, peak values of the acceleration at depths of 2.5/7.5/13.5 m in
the PLAXIS analysis are 0.339/0.295/0.213 g compared to the EERA values 0.343/0.303/0.22 g.

In the first stage of the 2D dynamic analysis the structural system with finite mass is installed. In the second
stage the seismic base excitation is applied. The results for the sectional forces obtained in the dynamic
calculation stage inevitably include the sectional forces due to the structure installation in the model. The net
maximum/minimum values due to the seismic actions are derived from the envelope values as obtained from
all calculation steps after subtraction of the forces due to the structure installation.

**Results for the station**

We consider the joints between the diaphragm walls and the base and roof slab, respectively. The peak
values of the bending moments due to the seismic base excitation are:

Diaphragm walls at the level of the base slab: left wall: $M_{\text{wall}} = -2229 \text{ kNm/m}$; right wall: $M_{\text{wall}} = 2966 \text{ kNm/m}$.

Diaphragm walls at the level of the roof slab: left wall: $M_{\text{wall}} = -918 \text{ kNm/m}$; right wall: $M_{\text{wall}} = 627 \text{ kNm/m}$. 
The peak values of the bending moments in the base slab are: left side: \( M_{\text{slab}} = -3156 \text{ kNm/m} \); right side: \( M_{\text{slab}} = -4108 \text{ kNm/m} \).

The peak bending moment at the level of the base slab occurs in the right diaphragm wall at time \( t = 10.2 \text{ s} \). The peak bending moment at the level of the roof slab occurs in the left wall also at this point of time.

The envelope from all calculation steps including the static values due to the installation of the structure is shown for the two diaphragm walls in Figure 4. The distribution at time \( t = 10.2 \text{ s} \) is shown in Figure 5.

**Results for the twin tunnels**

The axial forces induced by the seismic base excitation exhibit the following peak values: left tunnel \( N_\ell = -390 \text{ kN/m} \); right tunnel: \( N_\ell = -402 \text{ kN/m} \). The corresponding values for the bending moments are: \( M_\ell = -51.7/52.9 \text{ kNm/m} \) for the left tunnel, and \( M_\ell = -52.8/52.2 \text{ kNm/m} \) for the right tunnel. The system and the distribution of the envelopes of the sectional forces including the static values due to the tunnel installation are plotted in Figure 6.

![Figure 4. 2D dynamic analysis. Distribution of bending moment in the diaphragm walls: wireframe distribution is for the envelope from all calculation steps, solid line for the distribution due to the structure installation](image-url)
max M = 6411 kNm/m ; min M = -2399 kNm/m

Figure 5. 2D dynamic analysis for the station. Bending moments in the walls at $t = 10.20$ s

min/max N = -1459/0 kN/m

min/max M = -54.1/51.7 kNm/m

Figure 6. 2D dynamic analysis for the twin-tunnels. Partial model view; axial forces and bending moments. The wireframe distribution is for the envelope from all calculation steps, the solid lines for the values due to the structure installation.
2D QUASI-STATIC RESPONSE ANALYSIS

The approximate 2D quasi-static analysis is carried out using PLAXIS by imposing an appropriate displacement pattern at both vertical boundaries of the model and along the free surface. This displacement at the vertical boundaries is calculated from the depth profile of the maximum shear strain obtained from the 1D EERA analysis (Figure 1, center), yielding at the soil surface the maximum value of 0.0693 m. This displacement is imposed also along the top boundary of the model. The lateral extent of the model is set here equal to 80 m. The subdivision into layers and the shear modulus in the individual layers with a linear distribution are the same as in the dynamic analysis described above. The model is shown in Figure 7.

Due to the variability of the shear strain along the vertical boundaries and of the shear modulus in the domain, the above configuration does not reproduce under free-field conditions (no structure installed) the 1D displacement profile along the vertical cut through the model at the location of the station walls. However, the differential displacement between the depths of the roof slab and the base slab is comparable to that from the 1D analysis, thus justifying the use of this approximation.

Results for the station
The distribution of the bending moments in the underground station is shown in Figure 8. The largest values occur at the joint of the base slab with the diaphragm walls. At this location the maximum value in the diaphragm wall is $M_{wall} = 1953 \text{kNm/m}$, and in the base slab $M_{slab} = 2794 \text{kNm/m}$. In the roof slab the maximum bending moment equals $M_{slab} = 510 \text{kNm/m}$.

Results for the twin-tunnels
The results of the analysis are shown in Figure 8. The maximum sectional forces are: axial force $N_{f} = 322 \text{kN/m}$, and bending moment $M_{f} = 39.8 \text{kNm/m}$. 
COMPARISON AND CONCLUSIONS

First, the sectional forces obtained from the design of the excavation and those arising from the seismic actions are compared. The code adopted for the design requires the application of a safety factor of 1.35 for permanent loads, 1.2 for temporary actions, and 1.0 for seismic actions. For simplicity, loads due to moving loads such as equipment, are neglected. The design value for the bending moment in the diaphragm walls at the level of the base slab is $M_{wall, d} = 1.2 \cdot 3342 = 4010$ kNm/m for the static case compared to the peak value of the dynamic calculation $1.0 \cdot 2966$ kNm/m or to the value of $1.0 \cdot 1953$ kNm/m from the quasi-static approach with imposed displacements. The analysis for static loads by far dominates the design. In addition, the results of the dynamic analysis are peak values that have to be multiplied by a reduction factor in order to obtain effective values. This factor will depend on the particular time history, and on the definition of the effective value.

For the twin-runnels too, static loading governs the design: Due to permanent loads we have $N_{t, d} = 1.35 \cdot 1080 = 1458$ kN/m, whereas the seismic actions yield peak values of $1.0 \cdot 402$ kN/m in the dynamic analysis or $1.0 \cdot 322$ kN/m in the quasi-static approach. For the bending moments, design for permanent loads yields $M_{t, d} = 1.35 \cdot 57 = 77$ kNm/m compared to $52.9$ kNm/m from the dynamic analysis.

The comparison of the dynamic time domain FEM analysis and the quasi-static FEM analysis is addressed next. We first consider the diaphragm walls of the underground station. The peak value of the bending moment at the level of the base slab is $2966/1953 = 1.5$ times higher than the value from the quasi-static analysis. Assuming approximately a reduction factor of $2/3$ for transforming to effective values, the
difference vanishes. However, the displacement imposed in the quasi-static analysis is derived from the EERA analysis as a peak value. So, for compatibility it has also to be reduced in some extent in order to represent an effective value. Unless, the methodology followed in the quasi-static approach explicitly requires that the maximum values of strain have to be applied with no further reduction.

The situation for the tunnels is different. The dynamic analysis yields for the peak axial force 402 kN/m, which is 25% higher than the corresponding value of the quasi-static analysis. Here, the application of a reduction factor of 2/3 would make the design value derived from the dynamic analysis lower than the quasi-static one.

The reasons for this discrepancy may be found in the different geometry of the structures (deep and narrow underground station versus twin-tunnels), and in the suitability of the quasi-static approach adopted herein when applied to soils with variable depth profiles of the shear modulus and of the seismically induced shear strains.

REFERENCES


