Nonlinear Behavior of Reinforced Concrete Circular Tunnel under Seismic Motions in Clayey Soil

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Abstract. In this paper, nonlinear response of reinforced concrete circular tunnels subjected to seismic loads in clayey soil is presented. A series of two dimensional finite element models were adopted to investigate the dynamic behavior of tunnel soil system. The influence of several parameters including ground motion intensity, soil type, tunnel depth and tunnel diameter on the tunnel response was computed. Firstly, 1D equivalent linear viscoelastic analysis was conducted in the frequency domain to evaluate the profile of Rayleigh damping coefficients. Hence, these profiles were used in order to simulate the 2D fully coupled finite element adopting visco-elasto-plastic effective stress models for the soil. The dynamic behaviour of tunnel was presented in terms of internal forces induced in tunnel lining. The analyses indicated that in case of the stiff clay, the results of closed form solution were close enough to the 2D analysis and the earthquake effects are considerable when the tunnel is close to the bed rock. In case of medium clay deposits, the effects vary according to the tunnel dimensions. Furthermore, Soft clay soil yielded when it exposed to earthquakes of greater than 0.5g magnitude. Earthquakes of 0.25g magnitude have a slight effect on the circular tunnels located at any depth in clayey deposit, but if another time domain are used with the same peak ground acceleration value, different effects might appear.

1. Introduction

During serves time, tunnels could be exposed to ground shaking. There are several examples of damages to sub-structures for which dynamic forces were not considered in the original design. Hashash et al. [1] conducted a study on Daikai subway station collapse in Kobe during the earthquake that hit Hyogoken-Nambu in 1995, the highway tunnels damages in Central Taiwan during the earthquake that hit Chi-Chi in 1999 and Bolu tunnel collapse in Turkey during the earthquake that hit Koceali in 1999. The complete contact between tunnel and surrounding soil as well as its significant length make the design of tunnel withstand earthquake loading distinct from the surface structures. The inertia of soil media around the tunnel is large relative to the inertia of underground structures. Thus, the seismic response of tunnel is dominated by the response of surrounding soil [2].

The performance of tunnel under seismic load has been evaluation by several approaches: empirical and analytical methods, numerical models and physical model tests. Several closed-form solutions were developed in order to determine the induced forces in tunnel due to seismic load [3-7]. Physical model tests have been conducted by many researches to obtain the response of tunnel due to seismic conditions [8-10]. Due to the high cost of physical models and their complexity, the results obtained from physical model remain limited.
Many numerical studies were carried out in order to investigate and compare the internal forces acting on tunnel lining under seismic effect. In broad sense, the numerical analysis is usually modelled using 2D analysis technique [11-14] or 3D modelling [15-17]. In numerical analysis, earthquake loads are usually simulated by quasi-static loads. The main disadvantages of the quasi-static model are that it does not take the change in structural behaviour with time into consideration. In addition, some researchers reported that the equivalent static method would yield smaller structural internal forces induced in tunnel lining than of a true seismic solution [18,19].

This study has the aim to study the response of circular tunnel under seismic conditions. A series of 2D numerical analysis using finite element method is performed in order to determine the tunnel transverse behaviour of tunnel due to ground motion in terms of bending moment and normal force. Fully dynamic analysis with taking complex interaction between tunnel and soil into consideration, which also called coupled analysis, is used to achieve this study. In addition, the current analysis dealt with the nonlinear seismic response of soil by performing one dimensional analysis to predict the profile of Rayleigh damping coefficients and soil shear modulus profile depending on shear strain level. However, the results of 1D analysis are also used to calibrate the parameters of the linear visco-elastic and visco-elasto-plastic models adopted in the coupled finite element analyses. The effect of several parameters including intensity of ground motion, soil type, tunnel depth and tunnel diameter on the dynamic behaviour of tunnel is presented.

2. Numerical model
2.1. Model geometry and materials
Ideal 60m thick soil deposits were considered rested on a bed rock. The width of the deposits is equal to eight times its thickness (480 m) the tunnel is founded at depth (H) and diameter (d) on as shown in Figure 1.

![Figure 1 Layout for soil modelling geometry.](image)

Three types of soil deposit will be analyzed, the first deposit is a single soft clay stratum, the second is composed of a medium clay stratum, and the third is a stiff clay one. The physical properties and mechanical parameters of these materials are reported in Table 1. In these deposits, a horizontal ground water table surface was assumed, with the ground surface level.

| Table 1. Physical and mechanical parameters of the three soil deposits analyzed in the numerical analyses. |
|--------------------------------------------------|-------------------------------------------------|-------------------------------------------------|----------------|
| Parameters                                      | Stiff clay                                      | Medium clay                                    | Soft clay       |
| Dry unit weight of volume (kN/m³)                | 17.4                                           | 16.2                                           | 15              |
| Saturated unit weight of volume (kN/m³)          | 20                                             | 18.5                                           | 17              |
| Coefficient at rest K₀                           | 0.5                                            | 0.5                                            | 0.5             |
| Poisson’s ratio ν                                | 0.35                                           | 0.35                                           | 0.35            |
| Maximum shear stiffness Gmax (kN/m²)             | 250000                                         | 140000                                         | 35000           |
The shear wave velocity \( V_s \) of soils varies with depth in proportion to \((p_m)^{1/4}\) where \( p_m \) is the effective mean confinement pressure. Hence, the profile of low strain shear stiffness \( G_0 \) with depth is computed by \( G_0 = r V_s^2 \) [20], where \( r \) denotes to the mass density of soil. For tunnel lining, in this study the tunnel lining follows the elastic behavior and modeled as a circular plate element with diameter \( D \). The tunnel lining is assumed to be concrete material with elastic modulus \((E)\) of 2.21e7 kN/m², unit weight of 25 kN/m³ and the Poisson's ratio \((\nu) = 0.20\).

In order to investigate the impact of tunnel size on the dynamic response of tunnel, three values of tunnel diameter are examined under seismic condition. Table 2 illustrated the values of tunnel diameter and the corresponding stiffness as well as lining thickness.

### Table 2. tunnel geometry and stiffness.

| Tunnel diameter (m) | Normal stiffness (kN/m) | Flexural stiffness (kN/m²/m) | Lining thickness (m) |
|---------------------|------------------------|----------------------------|--------------------|
| 5                   | 6.64e7                 | 4.98e7                     | 0.3                |
| 10                  | 1.11e7                 | 2.31e5                     | 0.5                |
| 20                  | 1.55e7                 | 6.33e5                     | 0.7                |

2.2. One dimensional analysis

The 1D ground response analyses are performed by closed form solution of the EERA code. The equivalent-linear visco-elastic soil behavior was considered. Variation of shear strain level \((\tilde{\gamma})\) with damping ratio \((D)\) and modulus reduction curve \((G/G_0)\) are defined according to Idriss and sun [21] modulus for clay upper range and damping for clay as shown in Figure 2. A total number of 41 layers were assumed to discretize the profiles of stiffness and damping ratio with depth. Soil profile is divided into 29 upper most layer of 1 m thickness, followed by 5 of 2 m, 7 of 3 m. In the iterative procedure, the ratio of effective and maximum shear strain is assumed equal to 0.5.

![Figure 2. Modulus reduction curve G/Go and variation of damping ratio D with shear strain \(\tilde{\gamma}\), after [21].](image_url)

2.3. Two dimensional analysis

The coupled numerical analyses is conducted using the finite element code PLAXIS 2D [22], a 2D plane strain and axi-symmetric finite element code which implements the coupled Biot dynamic equations [23], adopting the u–p simplification method (where \( u \) is the skeleton displacement and \( p \) the pore pressure), assuming that the fluid acceleration relative to the solid skeleton is negligible. The code in the dynamic solution allows generating frequency dependent viscous damping using the Rayleigh formulation. Below is the definition of the damping matrix:

\[
[C] = \alpha_R [M] + \beta_R [K]
\]

(1)

Where \( M \) and \( K \) are the mass and the stiffness matrix of the system respectively. The values of the coefficients \( \alpha_R \) & \( \beta_R \) are generated from the following relationship with the damping ratio \( D \) [22]:

\[
\alpha_R + \beta_R \omega_i^2 = 2 \omega_i x D_i
\]

(2)

Where \( \omega_i \) refers to the angular frequencies which the viscous damping is equal to or lower than \( D_i \). The standard boundary conditions were adopted for the static stages of the analyses: nodes at the bottom of the mesh were fixed in both horizontal and vertical directions, while those along the lateral sides were fixed in the horizontal direction only. The bottom of the mesh...
was assumed in the dynamic analyses to be rigid and the lateral sides were characterized by the viscous boundaries which Lysmer and Kuhlmeyer suggested in their study [24]. A linear viscoelastic constitutive model for the soil was first selected in the dynamic stage of the analyses to perform a thorough analysis using the EERA results, thus coupling the Rayleigh viscous formulation and a linear isotropic elastic model. Plasticity was then added which leaded to a non-associated constitutive viscoelasto-plastic assumption characterized by a Linear Elastic yield criterion. The mesh employed in the current numerical is presented in Figure 3 by means of 15-node plane strain triangular elements. The domain was partitioned into 41 horizontal layers to simulate the damping and stiffness parameters with depth similar as obtained from 1D ground response analyses EERA. All analyses carried out were performed using a set of initial stages, followed by the dynamic stage. Furthermore, the seismic signal is applied at the mesh bottom and the analysis is carried out under undrained conditions. The seismic signal is applied with a time step of 0.02s, equivalent to the time step used in the seismic signal input data. The elastic soil shear stiffness modulus G which is assumed in the initial stages of the analyses were selected by scaling down the equivalent initial values G₀ according to the normalized modulus reduction curves.

Figure 3. Mesh employed in the FE analyses.

2.4. Modelling of ground motions
The response of circular tunnel is examined under three different earthquake waves with different magnitudes. Figure 4 to Figure 6 show the acceleration time history and the scaled acceleration for the three motions as well as the equivalent bed rock motion for the case of medium clay (as example). Moreover, it well known that the seismic waves are measured at the ground surface. Therefore, the equivalent bed rock motions are determined by performing an equivalent linear deconvolution with the closed form solution by EERA code. Hence, the equivalent bed rock wave is applied at the bottom of two dimensional model.

Figure 4. Acceleration time history of upland Rancho Cucamonga (ground motion 1).

Figure 5. Acceleration time history of Mendocino (ground motion 2).
2.5. Parametric Study

In the current study the impact of several parameters on the dynamic response of circular tunnel is investigated. The depth of bed rock is kept constant at 60m from ground surface. The depths of tunnel (H) are assumed to be as 5, 10, 20, 30m and its diameter (d) is considered as 5, 7.5, 10m. Moreover, in order to present dimensionless study, the tunnel depth is normalized to tunnel diameter. The studied cases through this paper are reported in table 2.

| Peak Ground Acceleration | Stiff | Medium | Soft | Stiff | Medium | soft | Stiff | Medium |
|--------------------------|------|--------|------|-------|--------|-----|-------|--------|
| Soil type                |      |        |      |       |        |     |       |        |
| H/d                      |      |        |      |       |        |     |       |        |
| 0.25 g                   | 1    | 1      | 1    | 1     | 1      | 1   | 1     | 1      |
| 0.5 g                    | 2    | 2      | 2    | 2     | 2      | 2   | 2     | 2      |
| 0.85 g                   | 4    | 4      | 4    | 4     | 4      | 4   | 4     | 4      |
| H/d                      | 6    | 6      | 6    | 6     | 6      | 6   | 6     | 6      |
| 1.33 g                   | 0.67 | 0.67   | 0.67 | 0.67  | 0.67   | 0.67| 0.67  | 0.67   |
| 2.67 g                   | 1.33 | 1.33   | 1.33 | 1.33  | 1.33   | 1.33| 1.33  | 1.33   |
| H/d                      | 4    | 4      | 4    | 4     | 4      | 4   | 4     | 4      |
| 0.5 g                    | 2.67 | 2.67   | 2.67 | 2.67  | 2.67   | 2.67| 2.67  | 2.67   |
| 0.85 g                   | 4    | 4      | 4    | 4     | 4      | 4   | 4     | 4      |
| H/d                      | 0.5  | 0.5    | 0.5  | 0.5   | 0.5    | 0.5| 0.5   | 0.5    |
| 1.33 g                   | 1    | 1      | 1    | 1     | 1      | 1   | 1     | 1      |
| 2.67 g                   | 2    | 2      | 2    | 2     | 2      | 2   | 2     | 2      |
| 4.05 g                   | 3    | 3      | 3    | 3     | 3      | 3   | 3     | 3      |

3. Ground response analyses, parameters calibration of visco-elastic and free-field 2D ground

In this context, the solution depends on the assumed profile of the damping and stiffness parameters with depth. Thus, the parameter calibration might be not trivial when adopting linear visco-elastic assumptions, due to the well-known dependency of both damping and stiffness on the strain level. This paper use the same approach reported by Zidan [25] to calibrate the process of the viscoelastic parameters to be assumed in dynamic analyses using finite element method. Shear modulus and damping ratio profiles are set in such way to match the corresponding profiles resulting from the EERA code as a closed form solution. Therefore, the numerical model is subdivided in a large number of layers to obtain the closest possible correspondence. For each layer, a single value of shear modulus and damping ratio are identified. For instance, Fig. 7 shows the results of 1D analysis obtained by EERA in terms of shear modulus and damping profiles for stiff clay deposit subjected to Ground motion 2. As shown in Fig. 7c, the Fourier spectra are constructed at different depths to identify the frequency interval where the highest energy content is observed. It shows that the energy concentrates between 1 and 4.5 Hz. This frequency interval is then used to define the corresponding Rayleigh coefficients αR and βR according to Eq. (2). After then, the profiles of Rayleigh coefficients αR and βR are determined as shown in Fig. 7b.

To check the consistency between the closed form solution and finite element analysis a preliminary comparison between the EERA and PLAXIS predictions at any depth in the deposit is provided for each them. Figure 8 shows an example of comparison graph, in term maximum acceleration time history of upland
acceleration against depth, between 2D visco-elastic PLAXIS analysis and 1D free-field analysis performed with EERA for all soils subjected to Ground motion 2.

It shows a good agreement between 1D and 2D analysis. The results indicated that in case of soft clay deposit, a slightly less satisfactory agreement is observed, while in other cases, i.e. the medium and stiff clay deposits, a good agreement is obtained, demonstrating the effectiveness of the proposed calibration strategy. The same procedures are conducted for all types of soils.

(a) G and D profiles assumed in the FE analyses on the basis of EERA results
(b) αR and βR profiles assumed in the FE analyses on the basis of EERA results
(c) Fourier spectra computed by EERA at different depths

**Figure 7.** Shear modulus (G), Damping ratio (D), Rayleigh damping coefficients (αR and βR) and Fourier spectra obtained from 1D analysis for case of stiff clay deposit subjected to scaled earthquake wave PGA equal to 0.5g

**Figure 8.** Comparison between the variation of maximum acceleration with depth obtained from the FE analyses and EERA.
4. Dynamic internal forces induced in tunnel lining

In this section, the dynamic internal forces induced in tunnel lining related to the static condition are discussed. The maximum induced internal forces \( f_d \) during and after the earthquake in terms of bending moment \( M \), normal force \( N \) are normalized to static internal forces \( f_s \) in which the internal forces with the subscribe "d" refers to the dynamic condition while those with the subscribe "s" refer to static condition. The parametric studies are conducted in order to determine the influence of these parameters on the dynamic behavior of reinforced concrete circular tunnel. Figure 9 to Figure 14 illustrated the results of 108 analysis cases to investigate the influence of tunnel dimensions, tunnel depth and soil type on the dynamic performance of tunnel subjected to the three seismic waves are presented. The depth of tunnel \( H \) is normalized to the diameter of tunnel \( d \) \((d_r=H/d)\). In addition, the dynamic internal forces is normalized to the static ones whereas, normalized bending moment \((M_r) = M_d/M_s\), and normalized normal force \((N_r) = N_d/N_s\).

For soft clay deposit, through the variation of tunnel diameter and tunnel location, it is found that the dynamic bending moment ranges between about 6.5 to 1.42 times that of static case for peak ground acceleration of 0.5g as shown in Figure 9. As shown in figure 10 the normalized normal force \((N_r)\) due to peak ground acceleration of 0.5g varied between 3.24 to 1.86 through the variation of parameters. On the other hand, insignificant change in bending moment and normal force due to peak ground acceleration of 0.25g. In addition, the tunnel location is critical when the tunnel is close to the surface and the effect of earthquake on both bending moment and normal force decreases gradually with the increase of the tunnel depth. It worth mentioning, that for an earthquake with a maximum peak ground acceleration of 0.85g, the soft soil collapsed due to failure of shear strength and no results can be obtained.

In case of medium clay, Figure 11 and 12, it is found that the tunnel location is critical when the tunnel located in medium depth and exposed to an earthquake with a maximum peak ground acceleration of 0.5g. In case of the dynamic state, the bending moment and normal forces value are 1.35 and 1.9 times of the values of static state respectively, and the effect decreases gradually if it is located close to the surface or the bed rock, while if the tunnel is exposed to an earthquake load with a maximum peak ground acceleration of 0.25g, the effect can be neglected if the tunnel is located at any depth. For an earthquake with a maximum peak ground acceleration of 0.85g, the effect increases gradually with depth. For case of stiff clay, Figure 13 and 14, the maximum induced internal forces is observing at medium depth when the tunnel exposed to an earthquake with a maximum peak ground acceleration of 0.5g or 0.85g, where the bending moment value is 1.4 or 1.25 times that of the static state respectively. Besides, the effect decreases gradually if it is located close to the bed rock. On the other side, the induced normal force increases as tunnel depth increases as shown in Figure 14. As seen, if the tunnel is exposed to an earthquake load with a maximum peak ground acceleration of 0.25g, the effect can be neglected if the tunnel is located at any depth. It can be noted that some cases do not follow specific rule, for instance, tunnels which have diameter of 10m exposed to maximum ground acceleration of 0.85g.
Figure 10. Normal force ratio ($N_r$) versus depth to diameter ratio ($d_r$) for case of soft clay

Figure 11. Bending moment ratio ($M_r$) versus depth to diameter ratio ($d_r$) for case of medium clay

Figure 12. Normal force ratio ($N_r$) versus depth to diameter ratio ($d_r$) for case of medium clay

Figure 13. Bending moment ratio ($M_r$) versus depth to diameter ratio ($d_r$) for case of stiff clay
5. Conclusions
From the abovementioned results and charts, the following results can be drawn:
In case of the stiff clay, the results obtained from the 1D analysis or closed form solution are close enough to the 2D analysis. While in case of the medium clay, the 1D analysis results are somewhat similar to that of the 2D analysis. In case of soft clay, the 1D analysis is used to obtain rough results. Earthquakes of 0.25g magnitude have a slight effect on the circular tunnels located at any depth in clayey deposit, but if another time domain are used with the same beak ground acceleration value, different effects might appear. In case of soft clay, soil reinforcement solutions are needed to decrease the effects of earthquakes on any circular tunnel located at any depth, especially in case of shallow tunnels exposed to earthquake with PGA >0.25g. Soft clay soil yielded when exposed to earthquakes of >0.5g magnitude. In case of stiff clay deposits, the earthquake effects are considerable when the tunnel is close to the bed rock, while in case of medium clay deposits, the effects vary according to the tunnel dimensions.

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