Nonlinear Response of a Subsea Tunnel Constructed by Two Tunnelling Methods subjected to Strong Seismic Shaking

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Abstract. In this paper, a three-dimensional numerical model is built to study the seismic response of a subsea tunnel constructed by two tunnelling methods, including the TBM tunneling and drill-and-blast tunneling. In the numerical model, the plastic-damage model is used to simulate the elastic and plastic behaviour of concrete under dynamic loadings. To illustrate the seismic response of the tunnel and yet limit the amount of data included in the paper, the work presented is limited to the deformation of tunnel cross sections and dynamic stress responses. Based on the results, the deformation and stressed of different tunnel sections are discussed. Results show that the amplitude of maximal principal stress occurs at sidewall basement of the inner liner. The spandrel and the sidewall basement are the most precarious area.

1. Introduction
In China, underground construction has seen an increasing development in recent years, especially, the subsea tunnelling. For instance, the projects such as the Qingdao Kiaochow Bay subsea tunnel, the Xiang’an tunnel, the Zhanjiang bay tunnel, etc. have been completed in last few years. The strong earthquakes happened in recent years, i.e. Hyogoken-Nanbu (Kobe) Earthquake (Mw=6.9), Chi-Chi earthquake (Mw=7.6), Niigataken-Chuetsu Earthquake (Mw=6.8) and the Wenchuan earthquake (Mw=8.3), etc. show that seismic damage to underground structures may be significant and may result in a catastrophic collapse of the tunnel with unacceptable consequences to life and property. Numerical simulations and shaking table tests have been carried out to study the seismic response of underground structures \cite{1-3}. Model tests were also conducted on a variety of cases, such as the response of shallow tunnels \cite{4}, stiff tunnels embedded in soft clay \cite{5}, tunnel displacements due to ground liquefaction \cite{6}.

To study the seismic response of the subsea tunnel constructed by different tunnelling methods and take the above issues into account, in this paper, a full three-dimensional (3D) numerical model is proposed. In addition, from the results, damage mechanisms of the tunnel are estimated.

2. The Xiamen Line-3 subsea tunnel
The subway tunnel, Xiamen Line-3 tunnel, is located at the northeast of the Xiamen Island, connecting the Huli district and the Xiang’an district. The total length of the tunnel is 5,052 m, with a cross section composed of two parallel twin tunnels, one for each direction, separated 25 m between axes. In particular, due to the complex geologic condition, a hybrid tunnelling solution is proposed in this project, which combines a central drill-and-blast tunnel in the bedrock and the shield tunnels at two coastal area. At the
west side, a working shaft is constructed to connect the shield tunnel and drill-and-blast tunnel. However, due to the deep water and adverse geologic condition, it is hard to construct a working shaft at the east side; instead, the tunnel sections are connected directly undersea.

Figure 1 shows the longitudinal vertical section of the transition area; meanwhile, the three typical sections (D1-D3) are the drill-and-blast tunnel section, the expanded crossing section and the shield tunnel section, respectively.

The drill-and-blast tunnel was designed following the New Austrian Tunneling Method, which consists of a primary and a secondary support. Note that the inner support extends into the shield tunnel for only 10 m. At last, the transitional section is composed of five layers, i.e. primary lining (30 cm), secondary lining (30 cm), filling layer (30 cm), segment (35 cm) and inner layer (25 cm). The expanded cavity is excavated by drilling and blasting.

The properties of the concrete used for the supporting layers are included in Table 1.

| Items                  | Density (kg/m³) | Elastic Modulus (GPa) | Poisson’s Ratio |
|------------------------|-----------------|-----------------------|-----------------|
| C25-grade shotcrete    | 2250            | 21                    | 0.2             |
| C50-grade concrete     | 2500            | 33.5                  | 0.2             |
| C25-grade gravel concrete | 2200          | 15                    | 0.25            |
| Tunnel segment         | 2550            | 35.5                  | 0.2             |

3. Numerical model

Generally, pseudo-static analyses are carried out to develop close form solutions and used to evaluate the magnitude of seismic-induced strains in underground structures. This approach is attractive because of its simplicity, requires that the tunnel is located far from the epicenter. However, the cyclic loading effect from the strong earthquake may not be captured well by the pseudo-static approach and the dynamic behavior of concrete cannot be fully depicted. Therefore, full dynamic simulation is required, which take time history motions, nonlinear soil-structure interaction and elastoplastic constitutive models into consideration.

A full three-dimensional analysis of the connection part has been carried out in an attempt to investigate seismic response of tunnel structure and the damage mechanism of the joint. The mesh, which includes the tunnel and support, the surrounding rock and soil layer, has been constructed according to the actual tunnel length and geologic data following the Structural design report of the Tunnel. Figure 2 shows the entire mesh. The tunnel support and the surrounding ground are discretized with eight-node hexahedral solid elements. The model has dimensions 250 m long, 200 m wide and 70 m thick and follows an idealized topography of the site. Figure 3 shows a detail of the tunnel structure of transition area. Although not shown in this figure, the dimensions are the same as those of the actual cross section.
4. Results and analyses

To illustrate the seismic response of the tunnel and yet limit the amount of data included in the paper, the work presented is limited to the deformation and the dynamic stress response. In addition, due to the symmetry of the two twin tunnels, a similar response is obtained for the north tunnel. More specifically, results will be shown for three vertical cross sections (D1 – D3), as shown in Figure 5, representing the tunnel crossing sections at different parts. With regard to distortion response, ovaling deformations develop when waves propagate perpendicular to the tunnel axis. Hence, the ovaling deformation is generally taken as a critical indicator to evaluate the cross section deformation of tunnel structure. Meanwhile, both of the maximal and minimal principal stresses along the circumferential direction for different tunnel support layers are adopted as assessment index for stress condition analysis.

4.1. The drilling and blasting tunnel section D1

Ovaling deformation provides a direct understanding of the tunnel structure’s distortion. Figure 4 shows the time history the ovaling deformation at the crossing section D1. Note that the positive value represents the increase of the distance between B and F, vice versa. Judged from the numerical value, the maximal ovaling deformation occurs at 10.88 s, with the value of 1.79 mm. At the end of the earthquake excitation, the deformation is about 0.6 mm, indicating a permanent deformation at the tunnel crossing section. It is important to note that the plastic deformation mainly occurs between 6.5 s to 11 s.

In order to provide a direct view of stress distribution for the tunnel liners and surrounding rock along the circumferential direction when the crossing section suffers the maximal distortion, the maximum principal stress and minimum principal stress are laid out with the increase of the angle $\theta$, as shown in Figure 5 (a) and (b), where control point A is defined as the origination. In addition, the maximum tensile strength of the support is included as a dashed, horizontal line, to indicate the location where
damage is likely to occur. Figure 5(a) shows that the maximum principal stress of secondary liner is mainly larger than the primary liner and surrounding rock along the path. Note that the peak values of the liners and rock both occur at the spandrel area, with the $\theta$ between 135° to 155°. In addition, the amplitude of maximal principal stress occurs at sidewall basement of the secondary liner, with the peak value of 2.05 MPa. Therefore, the spandrel and the sidewall basement are the most precarious area. In regard to the minimum principal stress, as shown in Figure 5(b), the secondary support shows larger values than the primary liner and the surrounding rock. Note that the peak values are approximately 3 MPa, which is so small, compared to the compressive strength of the concrete.

4.2. The shield tunnel section D2

In terms of distortion of shield tunnel section, figure 6 shows the time history the ovaling deformation at the crossing section D2. Note that the positive value represents the increase of the distance between B and F, vice versa. Judged from the numerical value, the maximal ovaling deformation occurs at 10.88 s, with the value of 2.18 mm. At the end of the earthquake excitation, the deformation is about 1.1 mm, indicating a permanent deformation at the tunnel crossing section.

Regarding the section D2, Figure 7(a) shows that the maximum principal stress of inner support is mainly larger than the segment and surrounding rock along the path. Note that the peak values of the liners and rock both occur at the abutment area, with the $\theta$ about 225°. In addition, the amplitude of maximal principal stress occurs at sidewall basement of the secondary liner, with the peak value of 1.74 MPa. Therefore, the spandrel and the sidewall basement are the most precarious area. In regard to the minimum principal stress, as shown in Figure 7(b), the inner support shows larger values than the
segment and the surrounding rock. Note that the peak values are approximately 3.6 MPa, which is so small, compared to the compressive strength of the concrete.

![Figure 7. Maximum and minimum principal stresses along the circumferential direction.](image)

4.3. The transition section D3

In terms of distortion of the transition section, figure 8 shows the time history the ovaling deformation at the crossing section D3. Judged from the numerical value, the maximal ovaling deformation occurs at 10.88 s, with the value of 2.27 mm, which is close to the value of cross section D2. At the end of the earthquake excitation, the deformation is about 1.21 mm, indicating a permanent deformation at the tunnel crossing section.

![Figure 8. Time history of the ovaling deformation](image)

Regarding the section D3, Figure 9(a) shows that the maximum principal stress of inner support is mainly larger than other supports along the path. Note that the peak values of the liners and rock both occur at the abutment area, with the $\theta$ about 80° and 225°. In addition, the amplitude of maximal principal stress occurs at sidewall basement of the inner liner, with the peak value of 2.51 MPa. Therefore, the spandrel and the sidewall basement are the most precarious area. In regard to the minimum principal stress, as shown in Figure 9(b), the inner support shows larger values than the segment and the surrounding rock. Note that the peak values are approximately 4.2 MPa, which is small, compared to the compressive strength of the concrete.
5. Conclusions

Three-dimensional numerical model is built to study the seismic response of the subsea tunnel constructed by different tunnelling methods. Based on the results, the deformation of different tunnel sections is discussed. In addition, the maximum and minimum principal stresses along the circumferential direction are studied. Results show that the amplitude of maximal principal stress occurs at sidewall basement of the inner liner. The spandrel and the sidewall basement are the most precarious area.

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