Seismic Response and Damage Mechanism of the Subway Station in Rock and Soil Strata

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Abstract. Numerical simulations are carried out to study the seismic response of a subway station in rock and soil strata. In addition, during the numerical simulation, Drucker-Prager yield criterion and the constitutive model for concrete are introduced to be able to adequately describe the yield failure of surrounding rock and damage behavior of concrete. Based on the numerical results, the time histories of principal stresses, distribution of principal stresses at particular moments, the plastic deformation of surrounding ground and the damage of structure are fully studied. According to results and discussion, it can be concluded that the largest tensile stress generally occurs at corners including the spandrel, top of sidewall and the bottom of the sidewall. In addition, large shearing deformation is observed at the interface of rock and soil layer, due to the incongruous deformation. Structure damage is mainly observed at corner area in symmetrical distribution.

1. Introduction
With fast subway development in Chinese cities, in recent years a large number of subway tunnels and stations have been built. Subway stations, due to their large spans and complex structures, are given priority in ensuring construction and operation safety. Normally, they are embedded at shallow depths where the upper part of the surrounding ground is soft while the lower part is hard, as found in Beijing, Chongqing, Qingdao, Dalian and Shenzhen.

Past researches on subway stations in such "upper-soft and lower-hard" strata mostly focus on construction methods and structure stresses during construction. Xiong Tianfang et al. [1] analyzed structure stress characteristics and stability of surrounding ground at various depths for column-free, single-arch, large-span subway stations with arch cover method, represented by the Xinggong Street Station in soft-hard heterogeneous stratum in Dalian. In terms of seismic resistance study on subway station structures, Li Jidong et al. [2] investigated seismic response of subway stations with pre-constructed pipe-roof integrating support and structure under strong earthquake effect. Through analysis of Daikai subway station damaged during 1995 Osaka-Kobe Earthquake, Du Xiuli et al. [3] reviewed the effects on Daikai station from earthquake motion, field characteristics and structural factors. Additionally, the seismic response and damage mechanism of Daikai station were studied systematically based on numerical simulation and analysis [4]. Chen He et al. [5] analyzed the seismic response of subway stations co-constructed with utility tunnel by building a ground-structure...
numerical model. Fan Yi et al. [6] proposed a new effective method to analyze the damage and failure of subway station structures under strong earthquakes.

Literature review shows that researches on subway stations in soil and rock strata were focused on construction methods and structure stresses under static loading effect, and that studies on seismic performance of subway stations were mostly conducted on those in homogeneous strata. Therefore, in response to the insufficiency of existing researches this paper proposes a numerical model of a typical single-arch, large-span subway station in soil and rock strata. It also simulates the nonlinear yield behavior of soil and rock masses under earthquake effect using D-P model, and simulates the damage accumulation behavior of the subway station structure under seismic cyclic loading effects using a concrete plastic-damage model. Based on numerical simulation results, this paper discusses in detail critical stress response and distribution of the station structure, plastic yield deformation of surrounding ground and the station structure damage model.

2. Numerical Simulation

2.1. Calculation method

This numerical simulation uses the dynamic time-history analysis method which can simulate geometry of a complex structure taking into account peak value and duration of seismic motion effect as well as seismic wave spectrum characteristics. Additionally, this method can help to derive the stress and deformation behaviors of the structure at different times throughout the seismic motion effect. In this work the dynamic and time-historical analysis is conducted by ABAQUS software. A dynamic explicit FE solver based on central difference method is used to solve the problem. In explicit time integral computation, ABAQUS/Explicit derives the result of each incrementation from the equilibrium condition of previous incrementation. During computation total stiffness matrix is avoided by eliminating the need to solve complex simultaneous equations. This could significantly save solution cost and computation time. Furthermore, the operational efficiency of an explicit algorithm in simulating large deformation of structure and elasto-plastic nonlinearity of material is much higher than that of an implicit one.

\[ M\ddot{x}(t) + C\dot{x} = P^n - F^n + H^n \]

where \( M \) is total mass matrix; \( P \) is total load vector; \( F \) is a set of equivalent nodal force vectors in unit stress field; \( H \) is total hourglass viscous damping force matrix for the structure; \( H \) is damping coefficient matrix, usually taken as Rayleigh damping.

Using explicit central difference method this equation is solved as follows:

\[ \ddot{x}(t_{n+1/2}) = \ddot{x}(t_{n+1/2}) + (\Delta t_{n+1/2} + \Delta t_{n+1/2})\ddot{x}(t_n)/2 \]  

\[ x(t_{n+1/2}) = x(t_n) + \Delta t_{n+1/2}\ddot{x}(t_{n+1/2}) \]

where \( \ddot{x}(t_{n+1/2}) \) and \( x(t_{n+1}) \) are nodal velocity vector at \( t_{n+1/2} \) and nodal coordinate vector at \( t_{n+1} \) respectively.

2.2. Mechanical damping and boundary conditions

Using Rayleigh damping the relationship of damping matrix \( C \) in the dynamic equation with stiffness matrix \( K \) and mass matrix \( M \) can be expressed as:

\[ C = \alpha M + \beta K \]

where \( \alpha \), \( \beta \) are mass proportional damping coefficient and stiffness proportional damping coefficient respectively.

In numerical simulation study on underground structure response to seismic motion, simulation of infinite field using a finite area must take account of the effect of FE lateral boundary on seismic wave propagation effect to avoid any effect on dynamic response of internal FE structure from seismic wave reflection on the boundary. In this work non-reflective boundary is simulated by setting boundary viscous dampers separately attached in normal and tangential directions of the boundary.
2.3. **Input seismic motion**

Input seismic motion is greatly affected by site type and propagation path. Propagation of seismic waves at different frequencies varies widely with site. Therefore, in analyzing seismic response of a site-specific structure, selection of appropriate input motion is required to accurately reflect the dynamic response characteristics of the structure. Considerations for seismic wave selection are: seismic dynamic intensity, seismic wave spectrum characteristics and duration. As the world's first earthquake wave with fully recorded data, EL-Centro wave is widely used in structure seismic study; meanwhile, its spectrum characteristics also meet the dynamic characteristics of project site in question, such as characteristic site period. According to the geological safety assessment report, peak acceleration of the acceleration time history curve under a rare earthquake is 0.31g and the return period is 2450 years. In addition, in accordance with the Code for Seismic Design of Buildings, peak acceleration shall be considered as 1:0.65, i.e. 0.31g and 0.20g if horizontal and vertical seismic motions are input simultaneously.

2.4. **Numerical model and material parameters**

Build a 2D FE model using ABAQUS based on characteristics of soil-rock strata and large-span subway station. Additionally, research on underground structure response to seismic motion shows the effect of boundary on FE simulation can be ignored if lateral boundary width of the model is more than 5 times the span width of the underground structure. Therefore, transverse width on both sides of the soil mass model is 120m (approximately 6 times the station span width); the distance from station bottom to the model bottom is 50m, approximately 3 times the station height. Both the station structure and its surrounding ground are simulated using 2D solid elements. In numerical computation the initial stress and displacement field of the station structure under self-weight and surrounding ground gravity are simulated through static analysis before applying seismic dynamic load to the model bottom for dynamic time-history analysis. Review on damage causes and disaster mechanism of Daikai station shows a subway station is likely to experience severe structural damage under seismic effects. However, the conventional elastic constitutive model for concrete cannot truly reflect damage accumulation and failure of concrete structure under seismic dynamic loading effects. Therefore, this paper uses dynamic damage constitutive model for concrete to simulate concrete damage behavior under compression and tension. Based on actual reinforcement ratio of the station structure, the calculated disperse steel bar model in ABAQUS is used to equivalently simulate the structural effect of steel bars.

According to characteristics of the soil-rock strata in which the station is located, the soil mass model is divided into two layers: soft soil in the upper part and rock in the lower part. In addition, in order to truly reflect the yield failure behavior of soil and rock masses under strong earthquakes, the elasto-plastic model based on D-P yield criterion is used to simulate nonlinearity of surrounding ground from elastic deformation to plastic yield. Basic physical parameters of materials for FE model calculation in this study are taken in accordance with concrete design codes and geotechnical survey reports, as shown in Table 1.

| Items          | Density $\rho$ (kg/m$^3$) | Elasticity modulus $E$ (GPa) | Poisson's ratio $\nu$ | Internal friction angle $\phi(\circ)$ | Cohesion $c$ (MPa) |
|----------------|---------------------------|----------------------------|----------------------|--------------------------------------|------------------|
| Rock stratum   | 2450                      | 11.5                       | 0.27                 | 50                                   | 1.3              |
| Soft soil      | 1850                      | 1.20                       | 0.35                 | 33                                   | 0.5              |
| Concrete       | 2500                      | 30                         | 0.2                  | —                                    | —                |

Table 1. Properties of the surrounding rock and the tunnel liner
3. Result Analysis

3.1. Acceleration response

Under seismic dynamic effects, acceleration response demonstrates the structure’s vibration. Meanwhile, spectrum characteristics demonstrate seismic motion component of dynamic response. Figure 1 shows the acceleration time histories at station crown in horizontal and vertical directions. Peak accelerations in both directions are 0.41g and 0.23g respectively. The amplification effect in horizontal direction is bigger than in vertical direction; therefore, the dynamic response in vertical direction is also smaller. In addition, Figure 2 shows spectrum curves corresponding to acceleration. To ease comparison Figure 2 also presents the spectrum curve of input motion. It can be seen that the amplification effect at 3 Hz ~ 10 Hz in horizontal direction is noticeable while low-pass filtering is observed at frequencies of lower than 3 Hz. In vertical direction neither amplification nor filtering effect of seismic motion is noticeable.

Figure 1 Acceleration time histories at arch crown.

Figure 2 Acceleration spectrum curve at arch crown.

In addition, acceleration at critical points in the bottom center of the station is analyzed. The acceleration time histories and spectrum curves are shown in Figure 3 and 4 respectively. From these figures it can be seen that acceleration response at the bottom is smaller than at the top of the station; peak acceleration is 0.37g and 0.22g. Meanwhile, the spectrum curve shows the acceleration amplification effect of the structure is not noticeable.

Figure 3. Acceleration time histories at station bottom.

Figure 4. Acceleration spectrum curve at station bottom.
3.2. Stress distribution analysis
To work out stress distribution in the structure under seismic effects at particular times, stress distribution diagrams at one peak time for input motion are selected for analysis. Figure 5 shows the distribution of maximum and minimum principal stresses in the station structure at 2.16s under seismic motion effects. It can be observed that maximum tensile stresses in the structure occur at the left side of arch bottom, peaking at 2.17 MPa. Meanwhile, maximum compressive stresses occur at the right spandrel, peaking at 8.97 MPa.

![Figure 5 Distribution of principal stresses at 2.16s](image)

3.3. Plastic failure mode of surrounding ground
Under seismic effects, stress deformation of the underground structure is primarily a result of squeezing by surrounding ground. Large cyclic seismic loading will lead to yield failure of surrounding ground. Figure 6 shows the distribution of plastic deformation of soil mass at 4.56s. Further increase in plastic deformation of the surrounding ground can be observed. Figure 7 shows the distribution of plastic deformation of soil mass at 5.04s. A sharp increase in peak values of plastic deformation of the surrounding ground to 8.97E-4 can be observed. It is to be noted that the plastic deformation at this time is consistent with plastic deformation distribution and strain peak at 35s when seismic motion ends. This suggests seismic motion that comes after the peak does not increase plastic deformation of surrounding ground.

![Figure 6. Plastic deformation of surrounding ground at 4.56s.](image)

![Figure 7. Plastic deformation of surrounding ground at the end.](image)

3.4. Structure damage and failure mechanism
In this section the structure's damage and failure mechanism are analyzed at the same critical times of seismic motion. Given tensile strength of concrete is much lower than its compressive strength, concrete mostly experiences tensile failure under seismic motion effects. Therefore, only tensile damage of the structure is addressed. At 4.56s the structure experiences damage at multiple locations such as left and right spandrels as well as top and toe of sidewall, as shown in Figure 8. It can also be observed that most damages occur at corners and in symmetrical distribution. Figure 9 shows the distribution of tensile damage of the structure at 5.04s when the damage factor at spandrels rises up to 0.2 without any increase in the number of damage locations. In line with plastic deformation of surrounding ground, this damage behavior is consistent with failure behavior when seismic motion ends.
4. Conclusions
From the above numerical simulation analyses of seismic response characteristics and damage mechanism of a subway station in soil and rock strata, the following conclusions can be drawn: (1) the structure experiences high tensile stresses at spandrel, and the top and toe of sidewall under the effect of shear deformation in soil and rock strata. Therefore, the structure damage and failure are primarily tensile. (2) under dynamic loading effects the surrounding ground undergoes severe plastic deformation at the interface between soil and rock and at the crown, especially yield failure of soil cover in large areas due to incongruous deformation. (3) Damage and failure of the structure are in symmetrical distribution and mostly observed at spandrel, top and bottom of sidewall and other corner areas.

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