Numerical Simulation of Damping Performance of Elastically Supported Underground Arch Structures Subject to Penetration and Explosion

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To reduce the damage of earth penetrators on underground fortifications against blast and fragmentation penetration, a model of underground arch structure with elastic support was established for a modal analysis on the structure subject to penetration and explosion with ANSYS/LS-DYNA software for dynamic analysis. In the process, the effect of spring stiffness coefficient on the dynamic response of the structure was analyzed in terms of pressure, equivalent stress, and peak vertical displacement. The simulation results showed that the elastic support reduced the dynamic response of the structure, while the equivalent stress and pressure of each part of the underground arch structure were reduced, and the peak vertical displacement was increased as the stiffness coefficient of the elastic support was lowered, and the isolation efficiency of equivalent stress and pressure at the arch shoulder was lower than that at other parts. Therefore, reducing the stiffness coefficient of the elastic support alone cannot meet the need for vibration isolation of the arch shoulder, and the type or stiffness coefficient of elastic support should be selected reasonably according to the actual engineering requirements, so as to achieve a good vibration isolation effect.

1. Introduction

The effects of earth penetrator include a high-speed penetration of warhead and a comprehensive action of explosion under initial conditions for penetration and damage. The huge energy generated by the penetration and explosion is released in a very short time, and the stress wave generated rapidly spreads to the deep structure through the rock mass. Even if the underground structure cannot be destroyed directly, the intense vibration caused by the propagation of stress waves can cause great damage to the underground structure [1, 2]. The design of structural vibration reduction has important scientific significance and engineering application value.

In order to further understand the underground penetration explosion and structural dynamic response, a large number of scholars have carried out theoretical, experimental, and numerical simulation studies. Chai et al. [3] analyzed the propagation law of wave with theoretical analysis method. Li et al. [4] derived the dynamic response of the structure under the action of seismic waves based on the wave function expansion method and verified the effectiveness of the derivation process. Haibo et al. [5] studied the propagation of shock wave in jointed rock mass by a field explosion test. Chen et al. [6] proposed some equations related to calculating explosion pressure and dynamic response based on empirical data. Compared with the experimental test, numerical simulation is more convenient and economical [7]. Deng et al. [8] conducted numerical simulation of large-scale decoupling underground explosion test of 10 tons of TNT in Alvdalen area of Sweden by combining DEM and FEM. The numerical simulation results of the hybrid AUTODYN-UDEC method are compared with empirical estimation, pure AUTODYN simulation results, and field test data. Mobaraki and Vaghefi [9] and Xie et al. [10] used LS-DYNA to simulate the blasting load of underground tunnels, which reflected the accuracy and analytical advantages of numerical simulation by comparing...
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with the measured data. Ma et al. [11, 12] studied the underground explosion process of rock mass by using AUTODYN program based on finite element and proposed a new discrimination basis for structural damage. Xie et al. [13] conducted a systematic study on rock dynamics disturbed by underground engineering including blasting and pointed out the important influence of explosion load on the safety of underground engineering.

Considering the safety of the structure, vibration isolation devices are generally used to reduce the damage of the structure under transient or short-term strong dynamic load. Vibration isolation may alleviate the damage of transient or short-term strong dynamic loads on the structure. Cancellara and De Angelis showed that rubber bearings have good applicability for seismic isolation reinforced concrete structures [14–16]. Yang et al. [17] studied the dynamic characteristics of a bridge with elastic support. The results showed that the elastic support reduced the natural vibration frequency of the structure significantly. Kitamura et al. [18] verified the applicability of large size disc spring in vertical vibration isolation system through a relevant experimental analysis. Yamamoto et al. [19] proposed and verified an analytical model for elastic vibration isolation bearing. Ma et al. and Chen et al. [20, 21] have demonstrated through finite element analysis and demonstration that the installation of seismic isolation supports can significantly reduce the seismic response of subway structures. Chakraborty et al. [22] compared the dynamic responses of the base isolators, such as composite rubber bearing, N-Z type base isolation pure friction (PF) system, and the friction pendulum (FP) elastic friction base isolators. The results showed that the vibration of the structure was effectively reduced by a base isolation system of appropriate stiffness and type. Zhu et al. [23] designed three thick-layer lead-core rubber isolation bearings with different single-layer rubber thicknesses and conducted mechanical performance tests to study the changes in basic mechanical properties such as vertical compression stiffness with compressive stress and shear strain. Kang et al. [24] studied the influence of elastic support on dynamic property and dynamic response of arch structure under blast impact load, and the results showed that the vibration isolation of reasonable stiffness improved the anti-explosion bearing capacity of structure effectively. At present, relevant studies mainly focus on isolation of the vibration load caused by earthquakes in civil buildings, subways, and highway bridges on the ground [25–30], and various vibration isolation devices including elastic support have different effects, but all can reduce vibration damage to different degrees, while there are few studies on the vibration isolation of underground arch structures under the action of the penetration and explosion of precision-guided earth-boring projectiles [31–33]. There is an obvious difference between the effect of weapon strike and the earthquake load, and the effect of elastic support on reducing the vibration damage caused by weapon strike is unknown. Therefore, it is necessary to conduct in-depth study on the damping effect of elastic support under weapon strike.

This study selected elastic supports for vibration isolation, which were set at the bottom of side wall and the lower part of base plate for the underground arch structure, and utilized the ANSYS/LS-DYNA finite element analysis software for a numerical simulation on the underground arch structure with elastic support under precisely guided penetration and explosion of earth penetrator, so as to explore the influence of elastic supports of different stiffness coefficients on dynamic response of the structure and provide a reference for the construction of underground protective works.

2. Materials and Methods of Numerical Simulation

2.1. Geological Conditions. In this study, a cave depot was taken as the experimental field, for which a reinforced concrete arch structure with straight wall was built. For convenience, we made the buried depth of the dynamic load section 10 m, and the Z-axis depth was 10 m. For the simulated rock mass, Class III surrounding rock was taken as the main body, with bolt-reinforced surrounding rock of 2 m and early strength mortar bolt of 22 mm diameter. The straight wall arch with elastic support was composed of arch ring, side wall, floor, elastic support, and damping support. The arch structures were made of concrete with strength grade C35. The arch structure is made of C35 concrete with HRB400 steel bars embedded in the concrete. The double-layer mesh layout is adopted. The circular steel bars are 22 mm in diameter and 200 mm in distance, and the longitudinal steel bars are 12 mm in diameter and 200 mm in distance. A common node modeling method was used for the concrete and reinforcement. The arch ring thickness was 0.5 m, the rise of the inner contour was 4.5 m, and the clear span was 14 m. The side wall was 2 m high and 0.5 m wide. Under the side wall and base plate of the arch with elastic support, the elastic support and damping support with a height of 0.5 m were set up at the upper part of the 0.5 m thick reinforced concrete bedding, and for the damping support, the damping ratio was 0.1. The base plate for the arch with rigid support was in direct contact with the bedding.

A 3D model of surrounding rock, bolt-reinforced surrounding rock, arch with elastic support, and air was established based on the ANSYS pretreatment module, and the grid size was 150 mm. The elements near the explosion area and the covered structure were meshes encrypted, and the unit size was 100 mm. Early bolting bore all basic load, and the secondary mould cast lining serving as a safety reserve, to share a special load. Anchor reinforcement refers to the initial support, which bore the total base load, and the secondary die lining was used as a safety stock for bearing special loads jointly. The bolt-reinforced surrounding rock refers to an initial support, including system bolt, shotcrete, grid steel frame, and so on. Considering that the model was of symmetrical structure, in order to save calculation time, a 1/4 model of tunnel depth and transverse symmetry was established for numerical simulation. Symmetric constraint was adopted on the symmetric surface, and the nonreflective boundary condition was used to simulate the semi-infinite medium on the asymmetric surface, except the symmetric boundary and the upper boundary. The upper surface of
surrounding rock was a free surface, and the arch with elastic support was in the air domain. The schematic diagrams of the calculation model are shown in Figures 1 and 2.

A simplified model of the earth penetrator is shown in Figure 3. The charging radius was 0.127 m, and the length was 3.704 m. For relevant technical parameters, see Table 1.

2.2. Material Models. The MAT_JOHNSON_COOK elastoplastic model and EOS_GRUNEISEN state equation were used to describe the strength limit and failure process of metal materials under conditions of large deformation, high strain rate, and high temperature for the casing of precision-guided earth penetrator; for built-in charging, the MAT_HIGH_HOMVE_BURN constitutive model and the EOS_JWL state equation were used for description. Relevant parameters for cartridge and charging materials are given in Tables 2 and 3 [34–37].

Granite, bolt-reinforced surrounding rock, and steel bars were all regarded as elastoplastic materials, and the MAT_PLASTIC_KINEMATIC constitutive model was selected. This model is suitable for describing the isotropic hardening and dynamic hardening plasticity of materials with the effect of strain rate fully considered. The material parameters of granite, bolt-reinforced surrounding rock, and steel bars were given in relevant literature [38].

The MAT_RHT constitutive model was used for concrete of underground arch. The model fully considered the large strain, high strain rate, and high-pressure effect of concrete, as well as the material damage and damage accumulation, so it can accurately describe the dynamic response and damage-fracture process of concrete under the actions of penetration and explosion. Relevant parameters for concrete materials are given in Table 4 [39].

Air was described with the MAT_NULL constitutive model and EOS_LINEAR_POLYNOMIAL state equation as an ideal gas. Its density was 1.25 kg/m³, the initial specific internal energy was 253312.5 Pa, and the isentropic adiabatic coefficient for ideal gas was 1.4 [31].

For the elastic support, the MAT_SPRING_ELASTIC constitutive model was selected for defining the linear stiffness coefficient, and for the damping support, the MAT_DAMPER_PARAMETRIC constitutive model was used for defining viscous damping coefficient [34].

2.3. Unit and Algorithm Selection. Elastic support and damping support were simulated using display spring damper unit COMBI165, a one-dimensional two-node unit composed of the node on the upper surface of reinforced concrete bedding and the node on the bottom of the side wall or the bottom surface of the base plate. The X and Z-axis degrees of freedom of nodes were limited, so as to ensure that the elastic support and damping support can only vibrate in the Y axis. The layout of COMBI165 is shown in Figure 4. The rest of the materials were simulated using Solid 164, a three-dimensional eight-node entity unit.

In the Lagrange algorithm, relevant calculation may stop because of negative volume resulting from excessive material deformation. For this reason, an ALE algorithm was adopted.
in this study to calculate the explosion process of high energy explosive. The shell case, surrounding rock, bolt-reinforced surrounding rock, arch with elastic support, and bedding were all regarded as solid, using Lagrange grid, and the explosive and air were taken as fluid, using Euler grid.

CONTACT_ERODING_SURFACE_TO_SURFACE  was used for algorithm of surface to surface erosion contact between shell and the surrounding rock. In order to ensure the accuracy of relevant calculation, the grid of surrounding rock at the contact area between the projectile body and surrounding rock was encrypted. The interaction between bolt-reinforced surrounding rock and arch with elastic support was simulated through surface to surface contact, with 0.3 static friction coefficient and 0.2 dynamic friction coefficient. LOAD_BODY_Y and DEFINE_CURVE keywords were set to apply gravity to the overall model.

### 3. Simulation for Scheme Validation

#### 3.1. Validation of Penetration Effect

As shown in equation (1), Young’s formula [40] with high accuracy for calculation, which is widely applied, was selected to calculate the penetration depth.

\[
H = \begin{cases} 
0.0008 \text{KSN} \left( \frac{\omega}{A} \right)^{0.7} \ln \left( 1 + 2.15 \times 10^{-4} V^2 \right), & V < 60.96 \text{ m/s}, \\
0.0000178 \text{KSN} \left( \frac{\omega}{A} \right)^{0.7} \left( V - 30.5 \right), & V > 60.96 \text{ m/s},
\end{cases}
\]  (1)

where \( H \) stands for the penetration depth of projectile body; \( \omega \) is the mass of the projectile body; \( A \) is the cross-sectional area of the projectile body; \( V \) is penetration velocity of projectile body; and \( J \) is the mass ratio conversion coefficient.

Figure 5 shows the curve of displacement in \( Y \) direction for the missile body. According to Figure 5, the increase of displacement velocity in \( Y \) direction (penetration depth) of the missile body slowed down with the penetration time, and
after the velocity became zero at the end of penetration, there was a slight rebound for the displacement. When \( t = 0.022 \) s, the penetration depth reached the maximum value, i.e., 4.5 m, about 6.6% of difference as compared with \( H = 4.22 \) m, the penetration depth of the projectile body calculated by Young’s formula. The main reason why the results of numerical simulation were large was that symmetric boundary conditions were applied to the symmetry surface of the projectile body in the numerical simulation. During the calculation time, the earth penetrator was constrained and could only penetrate along the Y axis. Young’s formula was obtained through fitting based on a large number of test data, and in an actual penetration process, due to the asymmetry of the warhead’s resistance, the trajectory will deflect and the penetration depth will decrease. A relevant analysis showed that the error of the numerical simulation results was smaller than that of Young’s formula; therefore, the numerical simulation results are reliable.

3.2. Validation of Explosion Effect. The equivalent stress distribution nephogram in the process of penetration and explosion is shown in Figure 6. The empirical formula for calculating the peak stress in free field was given in the technical manual of conventional weapon protection design published by the US War Department based on a large number of test data [41], as shown in the following equation:

\[
Pm = \gamma \left( \frac{m}{r} \right)
\]

where \( P_m \) is the peak pressure at the explosion wavefront in rock mass, MPa; \( m \) is the mass of explosive, kg; \( r \) is the distance from the measuring point to the explosion center, m; and \( \gamma, \alpha \) are the test constants. For granite, \( \gamma = 32 \) and \( \alpha = 2 \) are advisable.

4. Discussion

4.1. Structural Modal Analysis. Modality is the inherent vibration characteristic of structure dependent on the overall stiffness and mass of vibration system. Each order of modality corresponds to specific frequency, damping parameter, and modal parameter, and a fixed mode of vibration is also called a modal shape. The main modal characteristics of orders for the subjects in the susceptible frequency range obtained with the modal analysis method may be used to predict the actual vibration response of the structure in this frequency range under the influence of external vibration source.

4.1.1. Influence of Stiffness Coefficient on Frequency of Structural Natural Vibration Circle. Damping factors have little effect on the natural frequency and mode of the structure, so the effect of damping may be ignored in modal analysis. The relative stiffness coefficient for elastic support is defined as \( K = KL^3/\varepsilon I \), where \( I \) is the flexural stiffness of the structure and \( L \) is the clear span of the structure. When \( k = 0 \), it is of vertical freedom; when \( k = \infty \), it is of vertical consolidation, which is regarded as the rigid support condition. Figure 7 shows the relation curves between the rigid support and the first 5 orders of natural vibration circle frequencies of elastic supported arch with different relative stiffness coefficients \( \omega_1 \sim \omega_5 \). The data in the figures are listed in Table 6. After a comprehensive analysis, the conclusion was drawn as follows:

(1) The natural vibration circle frequency increased with model order for both arch with rigid support and arch with elastic support. When the relative stiffness coefficient \( k \in (0, 3000) \), the first five orders of natural vibration circle frequencies of arch with elastic support were all smaller than those of arch with rigid support. The smaller the \( k \) value was, the greater the amplitude for reduction of natural vibration circle frequencies of elastic supported arch became.

(2) There is a positive correlation between the natural vibration circle frequency of the arch with rigid support and the modal order, and the amplitude for increase of \( \omega_2 \sim \omega_3 \) was the largest, 153.8 rad/s, and that of \( \omega_4 \sim \omega_5 \) was the least, 43.8 rad/s.

(3) When \( k = 850 \), \( \omega_3 = \omega_4 \), and \( k = 1266.3 \), \( \omega_1 = \omega_2 \). The natural vibration circle frequency curves of adjacent orders diverted when they were close to each other, and there were proportional internal resonance
relations among the natural vibration circle frequencies of orders, such as \( \omega_3 = 3\omega_1 \), \( \omega_4 = 4\omega_1 \), and \( \omega_5 = 10\omega_1 \).

(4) For the arch with elastic support, the natural vibration circle frequencies of orders increased with \( k \), the relative stiffness coefficient. The growth trend of \( \omega_1 \) curve changed obviously. Also, \( \omega_1 \) increased rapidly in a low relative stiffness coefficient range. When \( k \geq 1266.3 \), the growth rate of \( \omega_1 \) curve gradually slowed to the first-order natural vibration circle frequency for the arch with rigid support.

(5) When the relative stiffness coefficient for elastic support \( k \) increased, the higher the order of natural vibration circle frequency was, the later it approached the natural circle frequency of the same order for the arch with rigid support. Therefore, the growth rates of natural vibration circular frequencies of orders with \( k \) were as follows: \( \omega_1 > \omega_2 > \omega_3 > \omega_4 > \omega_5 \).

4.1.2. Influence of Stiffness Coefficient on Structural Shape. Figure 8 shows the modal shape diagram of arch with rigid supports \( \omega_1 \sim \omega_5 \). As compared with that of arch with elastic support, when \( k < 1266.3 \), the modal shape diagrams of arch with elastic supports \( \omega_1 \) and \( \omega_2 \) were mainly symmetric and antisymmetric, respectively, which was contrary to the arch with rigid support. When \( k \geq 1266.3 \), the modal shape diagrams of arch with elastic supports \( \omega_1 \) and \( \omega_2 \) were mainly

<table>
<thead>
<tr>
<th>Distance (m)</th>
<th>Numerical simulation results (MPa)</th>
<th>Theoretical calculation results (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>322.8</td>
<td>363.3</td>
</tr>
<tr>
<td>2.5</td>
<td>206.4</td>
<td>232.5</td>
</tr>
<tr>
<td>3.0</td>
<td>140.8</td>
<td>161</td>
</tr>
<tr>
<td>3.5</td>
<td>105.7</td>
<td>118.6</td>
</tr>
<tr>
<td>4.0</td>
<td>79.6</td>
<td>90.8</td>
</tr>
</tbody>
</table>
Table 6: Natural vibration circle frequency of each order of structure (unit: rad/s).

<table>
<thead>
<tr>
<th></th>
<th>Rigid support</th>
<th>0</th>
<th>500 k</th>
<th>1000 k</th>
<th>1266.3 k</th>
<th>1500 k</th>
<th>2000 k</th>
<th>2500 k</th>
<th>3000 k</th>
</tr>
</thead>
<tbody>
<tr>
<td>ω1</td>
<td>91.0</td>
<td>3.7</td>
<td>57.6</td>
<td>80.8</td>
<td>90.4</td>
<td>90.6</td>
<td>90.7</td>
<td>90.8</td>
<td>90.8</td>
</tr>
<tr>
<td>ω2</td>
<td>182.0</td>
<td>15.1</td>
<td>70.9</td>
<td>89.7</td>
<td>90.4</td>
<td>98.0</td>
<td>111.8</td>
<td>123.5</td>
<td>133.4</td>
</tr>
<tr>
<td>ω3</td>
<td>335.8</td>
<td>91.3</td>
<td>91.7</td>
<td>101.1</td>
<td>112.4</td>
<td>121.7</td>
<td>139.5</td>
<td>155.2</td>
<td>169.2</td>
</tr>
<tr>
<td>ω4</td>
<td>432.4</td>
<td>188.5</td>
<td>189.2</td>
<td>189.9</td>
<td>190.4</td>
<td>190.8</td>
<td>191.9</td>
<td>193.3</td>
<td>194.9</td>
</tr>
<tr>
<td>ω5</td>
<td>476.2</td>
<td>337.3</td>
<td>337.3</td>
<td>337.4</td>
<td>337.4</td>
<td>337.4</td>
<td>337.5</td>
<td>337.6</td>
<td>337.7</td>
</tr>
</tbody>
</table>

Figure 7: Change of natural vibration circle frequency for arch with elastic support with K.

Figure 8: The first five orders of modal shapes for the arch with rigid support. (a) ω1. (b) ω2. (c) ω3. (d) ω4. (e) ω5.
antisymmetric and symmetric, respectively, which were the same as the arch with rigid support. Therefore, \( k = 1266.3 \) can be defined as the cutoff for dynamic characteristic change of underground arch structure with elastic support.

One of the basic principles for vibration isolation is to reduce the natural frequency of vibration system. Generally, for the arch with elastic support with small relative stiffness coefficient \( k \), as compared with arch with rigid support, the natural frequency of the structure was reduced, and the natural vibration period extended. Consequently, external loads reduced the probability of resonance phenomenon in low-order modal excitation. Elastic support played a positive role in reducing the dynamic response of the structure, but the specific vibration isolation efficiency is still to be further studied.

### 4.2. Analysis on the Influence of Stiffness Coefficient for Elastic Support on Dynamic Response

In this study, the dynamic responses of arch with elastic support and arch with rigid support \((k \longrightarrow \infty)\) with relative stiffness coefficient \( k \) ranging from 100 to 20000 (stiffness coefficient \( k = 9.49 \times 10^7 \text{N/m} \sim 1.90 \times 10^{10} \text{N/m} \)) were simulated, with the damping coefficient for damping support \( c = 2.56 \times 10^5 \text{N/s/m} \).

#### 4.2.1. Influence of Stiffness Coefficient on Structural Pressure

Figure 9 shows the pressure time history curves of the arch. From the figure, it can be seen that the wave impedance of surrounding rock was different from that of concrete, and the compression wave generated by the penetration and explosion constantly reflected and transmitted between the two media, for which the overlying structure was subjected to high-frequency oscillation, repeated action of stress and tension. The pressure at a study point under explosion was much greater than that under penetration, and the curve always peaked at the end of the penetration (after \( t = 0.022 \text{s} \)) because the explosion occurred within a few microseconds usually, and the action time was very short, but the shock wave overpressure was much higher than the pressure generated in the process of penetration. After the vertical elastic support was set, the times for curves of arch foot and wall foot to reach peaks were prolonged obviously. Take Figure 9(c) as an example; for the arch with elastic support, if \( k = 100 \), the peak pressure appeared when \( t = 0.06 \text{s}, 0.011 \text{s} \) later than that for the arch with rigid support, and the smaller the \( k \) value was, the later the peak pressure appeared. The curves were similar for the arch foot and the wall foot. When the pressure reached its peak, it decayed rapidly and tended to reach a plateau, but the curves of vault and spandrel fluctuated largely. When \( k = 1000 \), for the spandrel curve, the seventh peak may be greater than the first one. The reason for this was that the vault and spandrel were located on the blasting face and were subjected to the action of compression wave more intensely and for a longer time.

Table 7 lists the peak pressure values in the pressure time history curves of the study points for the arch with elastic support and the arch with rigid support at different \( k \) values. The values in the brackets are the vibration isolation efficiencies of the peak pressures for the vault, spandrel, arch foot, and wall foot. The vibration isolation efficiency is defined as the ratio of the vibration response amplitude difference after vibration isolation (attenuation) and before vibration isolation (attenuation) to the amplitude of vibration response before isolation (attenuation).

As shown in Table 7, the smaller the \( k \) value, the smaller the peak pressure of vault, spandrel, arch foot, and wall foot and the higher the vibration isolation efficiency, indicating that the vertical elastic support can reduce the impact load on the structure. When the \( k \) value was different, the peak arch foot pressure was the largest in the study points because the surrounding rock had a strong constraining force, which inhibited the deformation trend of the arch foot to both sides. The arch foot was subjected to the reverse effect of pressure from surrounding rock, so the arch foot was always the most stressed as compared with other study points. Therefore, the arch foot is the key position to be protected in the structure. With the decrease of \( k \) value, in the study points, the highest vibration isolation efficiency at peak pressure changed from the vault to the wall foot. When \( k = 100 \), the vibration isolation efficiency at peak pressure of the wall foot was the highest and could reach 86.77%. The reason for this was that the vertical elastic support changed the original rigid contact, causing the vertical displacement of side wall for energy consumption and reducing the load at the wall foot. For the same \( k \) value, the vibration isolation efficiency at peak pressure was the lowest for spandrel. For example, when \( k = 100 \), the vibration isolation efficiency at peak pressure was 46.97% for vault, 67.89% for arch foot, and 86.77% for wall foot, while at spandrel, the peak pressure was 6.02 MPa, 2.18 MPa lower than that of the arch with rigid support, 8.20 MPa, while the vibration isolation efficiency was only 26.59%. The reason for this was that in structural vibration, spandrel collided with surrounding rock, increasing the interaction between the spandrel and surrounding rock, so that the effect of vertical elastic support for reducing the stress on the spandrel was limited.

#### 4.2.2. Influence of Stiffness Coefficient on von Mises Equivalent Stress

von Mises equivalent stress follows the fourth strength theory of material mechanics, i.e., the von Mises equivalent stress at a point within the material fails when it reaches a certain limit value, and its value is calculated as follows:

\[
\sigma = \sqrt{\frac{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}{2}},
\]

where \( \sigma_1, \sigma_2, \) and \( \sigma_3 \) refer to the first, second, and third principal stresses, respectively.

von Mises equivalent stress can be used to describe the distribution of stress inside the structure and reflect the overall stress in the structure clearly. Figure 10 shows the von Mises equivalent stress time history curves of the arch, spandrel, arch foot, and wall foot for the arch with elastic support and the arch with rigid support under the action of explosion.
### Table 7: Peak pressure values at study points.

<table>
<thead>
<tr>
<th>$K$</th>
<th>Vault (MPa)</th>
<th>Spandrel (MPa)</th>
<th>Arch foot (MPa)</th>
<th>Wall foot (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>7.62 (46.97%)</td>
<td>6.02 (26.59%)</td>
<td>7.68 (67.89%)</td>
<td>1.06 (86.77%)</td>
</tr>
<tr>
<td>1000</td>
<td>7.58 (47.25%)</td>
<td>7.60 (7.32%)</td>
<td>9.84 (58.74%)</td>
<td>2.74 (65.79%)</td>
</tr>
<tr>
<td>3000</td>
<td>7.09 (50.66%)</td>
<td>7.98 (2.68%)</td>
<td>12.65 (47.12%)</td>
<td>4.27 (46.69%)</td>
</tr>
<tr>
<td>5000</td>
<td>7.40 (48.50%)</td>
<td>8.38 (~2.20%)</td>
<td>14.73 (38.42%)</td>
<td>4.86 (39.33%)</td>
</tr>
<tr>
<td>10000</td>
<td>8.51 (40.78%)</td>
<td>7.90 (3.66%)</td>
<td>16.62 (30.52%)</td>
<td>5.72 (28.59%)</td>
</tr>
<tr>
<td>15000</td>
<td>9.90 (31.11%)</td>
<td>8.74 (~6.59%)</td>
<td>17.43 (27.13%)</td>
<td>6.13 (23.47%)</td>
</tr>
<tr>
<td>20000</td>
<td>10.32 (28.18%)</td>
<td>8.15 (0.61%)</td>
<td>18.46 (22.83%)</td>
<td>6.43 (19.73%)</td>
</tr>
<tr>
<td>Rigid support</td>
<td>14.37 (0)</td>
<td>8.20 (0)</td>
<td>23.92 (0)</td>
<td>8.01 (0)</td>
</tr>
</tbody>
</table>

**Figure 9:** Time history curve of study point pressure. (a) Vault. (b) Spandrel. (c) Arch foot. (d) Wall foot.
Similar to the pressure time history curve, the von Mises equivalent stress time history curve showed an oscillatory attenuation after reaching the peak value. The curves of arch foot and wall foot gradually reached a plateau over time. The time for study points to reach von Mises equivalent stress peak values was different. The vault peaked first, while the wall foot was the last to peak. The times from the top to the bottom of the structure were prolonged successively. The von Mises equivalent stress at the explosion stage ($t = 0.022 \sim 0.16$ s) was much higher than that at the penetration stage ($t = 0 \sim 0.022$ s), and the effect of explosive after explosion on the dynamic response of the structure was even greater.

Table 8 lists the peak values of von Mises equivalent stress time history curves at study points for the arch with elastic support and the arch with rigid support. The values in brackets are the peak von Mises effective stress vibration isolation efficiencies of vault, spandrel, arch foot, and wall foot.

According to Table 8, the smaller the $k$ value, the smaller the peak von Mises equivalent stress values of vault, spandrel, arch foot, and wall foot and the higher the vibration isolation efficiency. The internal interaction within the structure was significantly reduced by the vertical elastic support. With the decrease of the $k$ value, for the increased amplitude of peak von Mises equivalent stress isolation efficiency, a retardation phenomenon occurred, i.e., the amplitude was high at first...
and then became low. For example, when the $k$ value decreased from 3000 to 100, the peak value of von Mises equivalent stress at the wall foot decreased from 9.91 MPa to 1.33 MPa, and the isolation efficiency suddenly increased from 47.48% to 92.95%, i.e., increased by 45.47%, which was the highest among all of the study points. For the arch with rigid support, the peak value of von Mises equivalent stress at the arch foot was the highest, so the arch foot was the most dangerous position in the structure. After the vertical elastic support of $k = 100$ was set up, the peak value of von Mises equivalent stress at spandrel was 20.81 MPa, 2.02 MPa lower than the arch with rigid support, and the vibration isolation efficiency was only 8.85%. At this point, the spandrel had the highest peak equivalent stress value in the arch as the most dangerous position. The reason for this was that, on the one hand, in structural vibration, the spandrel collided with surrounding rock, enlarging the internal force at the spandrel, and on the other hand, the spandrel was located at the connection of the vault and arch foot. Because the vertical displacement of the vault was relatively large and the displacement of the arch foot was restricted by surrounding rock, the spandrel had a stress concentration to coordinate the internal deformation. As a result, the effect of vertical elastic support on the equivalent stress of spandrel was not so obvious.

### 4.2.3. Influence of Stiffness Coefficient on Vertical Displacement of Structure

Figure 11 shows the time history curve of displacement for the vault, spandrel, arch foot, and the bottom of the side wall of the underground arch structure under the action of explosion. The positive value of displacement indicated that the structure moved forward along the $Y$ axis.

As shown in Figure 11, the vertical displacement values of vault, spandrel, arch foot, and wall foot were all negative, indicating that the structure moved along the negative direction of $Y$ axis. Because of the compression deformation of the vertical elastic support, the peak vertical displacement value of the arch with elastic support was greater than that of the arch with rigid support. With the disappearance of the impact effect, the vertical displacement gradually decreased after the peak and tended to reach a plateau but did not completely return to the initial state, which indicated that the concrete at the study point had exceeded the reciprocating vibration stage of elastic deformation and had undergone different degrees of plastic deformation. Different from the pressure time history curve, the vertical displacement time history curve was relatively flat, with only two peaks at most. There was no large fluctuation, and no latter peak exceeded the former one, indicating that the displacement of the structure changed stably even if a high-frequency oscillation pressure acted on the structure. The curves of study points approximately coincided with each other in the penetration stage and the initial stage of explosion, and then the slope of the curve with a high $k$ value increased and peaked, while the slope of the curve with a low $k$ value almost remained unchanged. The vertical displacement continued to increase, and the peak appeared later, with a larger value.

Table 9 lists the peak displacement values in time history curves for the arch with elastic support and the arch with rigid support at different $k$ values. As shown in Table 9, the peak vertical displacement at vault, spandrel, arch foot, and wall foot increased with the decrease of $k$ value. For the same $k$ value, the peak vertical displacement values from large to small were as follows: vault > spandrel > arch foot > wall foot, i.e., with large displacement at the upper parts and small displacement at the lower parts. For the arch with rigid support, the relative displacements of vault relative to the spandrel, arch foot, and wall foot were $-9.87$ mm, $-15.66$ mm, and $-17.38$ mm, respectively, larger than arch with elastic support when $k = 100$, i.e., $-5.45$ mm, $-5.77$ mm, and $-5.93$ mm, respectively, indicating that the vertical elastic support reduced the relative displacements between the study points, resulting in an overall displacement trend of the structure, and the degree of plastic deformation decreased accordingly. When the $k$ value decreased from 1000 to 100, the vertical displacements of study points increased by about 40 mm, and the peak value of vertical displacement increased abruptly. At this point, the vertical elastic support should not bear any load any more for excessive deformation. Therefore, as the stiffness coefficient of the vertical elastic support is designed, the requirements of the structure for anti-explosion bearing capacity and limit displacement should be considered comprehensively.

### 4.3. Basic Theory for Vibration Isolation

According to the theory of explosion mechanics, after precision-guided penetration and explosion of earth penetrators in the semi-infinite rock and soil medium, there will be a strong stress wave and seismic wave impact loading far from the center of the explosion. The stress wave may be divided into forward
wave (longitudinal wave) and tangential wave (transverse wave), and the forward wave (longitudinal wave) is also called compressive wave and tensile wave in the rock and soil. The forward wave is mainly transmitted in a form of longitudinal compressive stress or tensile stress, faster than the tangential wave. It acts on underground protection

![Figure 11: Time history curve of vertical displacement at study points. (a) Vault. (b) Spandrel. (c) Arch foot. (d) Wall foot.](image-url)

<table>
<thead>
<tr>
<th>K</th>
<th>Vault (mm)</th>
<th>Spandrel (mm)</th>
<th>Arch foot (mm)</th>
<th>Wall foot (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>−81.60</td>
<td>−76.15</td>
<td>−75.83</td>
<td>−75.67</td>
</tr>
<tr>
<td>1000</td>
<td>−44.31</td>
<td>−34.70</td>
<td>−33.27</td>
<td>−33.14</td>
</tr>
<tr>
<td>3000</td>
<td>−31.33</td>
<td>−21.41</td>
<td>−18.87</td>
<td>−18.44</td>
</tr>
<tr>
<td>5000</td>
<td>−27.70</td>
<td>−16.80</td>
<td>−13.50</td>
<td>−12.88</td>
</tr>
<tr>
<td>10000</td>
<td>−25.47</td>
<td>−13.21</td>
<td>−8.62</td>
<td>−7.80</td>
</tr>
<tr>
<td>15000</td>
<td>−24.42</td>
<td>−12.05</td>
<td>−6.72</td>
<td>−5.77</td>
</tr>
<tr>
<td>20000</td>
<td>−25.11</td>
<td>−11.90</td>
<td>−5.64</td>
<td>−4.67</td>
</tr>
<tr>
<td>Rigid support</td>
<td>−17.80</td>
<td>−7.93</td>
<td>−2.14</td>
<td>−0.42</td>
</tr>
</tbody>
</table>
works earlier and causes a change in unit volume of medium, thus resulting in vertical vibration of the structure. According to the characteristics of concrete, compared with the shear stress generated by tangential wave, the tensile stress and compressive stress generated by forward wave are the main factors causing the damage of concrete structure. The horizontal displacement of underground arch structure may reduce the capacity of the structure bearing dynamic load; therefore, during construction, the lining should be closely fitted with surrounding rock, which may constrain the horizontal displacement of the arch structure, so as to minimize the lateral displacement of the lining under external load. At this point, it can be assumed that there is only vertical displacement for the lining.

The transient powerful impact energy produced by penetration and explosion of earth penetrators may be stored in the vibration isolation device in the form of potential energy, which makes the device deform, release energy, and reconvert in a short time, thus reducing the vibration response of the structure, controlling indexes of the structure under dynamic disturbance within an allowable range for energy consumption [11]. The vibration isolation device for energy dissipation is installed at the foundation of underground arch structure for bearing the dynamic load together with the structure, thus effectively reducing the damage by precisely guided penetration and explosion of earth penetrator and alleviating or inhibiting the dynamic response of the structure.

5. Conclusion

This study selected a cave depot as the subject, with the vertical elastic supports provided at the bottom of the side wall and base plate as the vibration isolation means. With fluid-structure coupling algorithm, the finite element analysis software was utilized for a numerical simulation of the penetration and explosion effect of earth penetrators. The dynamic responses of vertical elastic support to vault, spandrel, arch foot, wall foot, and base plate of underground arch structure were analyzed in the study. The conclusions were as follows:

(1) The correctness of the numerical simulation results was verified. The calculated numerical simulation results of penetration depth and peak pressure of explosion wave front of earth penetrator were in good agreement with those worked out with relevant empirical formula, which verified the correctness of the numerical simulation method, constitutive model, and material parameters.

(2) After the vertical elastic support was set, with the decrease of relative stiffness coefficient $k$, the natural vibration circle frequency of the structure was reduced, the natural vibration period was extended, and the probability of resonance phenomenon in low-order modal excitation due to the influence of external load was decreased. The peak values of pressure and von Mises equivalent stress of vault, spandrel, arch foot, and wall foot were reduced, the vibration isolation efficiency was improved, the anti-explosion bearing capacity was enhanced, the peak value of vertical displacement increased, and the degree of plastic deformation decreased.

(3) When $k \leq 100$, because the deformation was too large, the vertical elastic support should not bear any load any more. When vertical elastic supports were used as vibration isolation means for underground fortifications, the stiffness of vertical elastic supports should be designed according to the requirements for space, anti-explosion bearing capacity, and limit displacement.

(4) The vertical elastic support had a poor vibration isolation effect at the peak pressure and stress of the spandrel. Only the stiffness of the vertical elastic support was reduced. Therefore, maybe the requirements for vibration attenuation or isolation of spandrel cannot be met.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Authors’ Contributions

All authors have read and agreed to the published version of the manuscript. Huixiang Sun and Yingjie Yuan were responsible for conceptualization and supervision. Yingjie Yuan was responsible for methodology. Zhuo Chen was responsible for formal analysis. Yingjie Yuan and Erlei Bai were responsible for investigation. Yingjie Yuan and Zhuo Chen were responsible for resources and data curation. Huiying Sun and Yingjie Yuan were responsible for original draft preparation.

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