Study on the safety criterion of blasting vibration in adjacent tunnels under hard rock strata based on energy principle

Junru ZHANG
   Key Laboratory of Transportation Tunnel Engineering, Ministry of Education, Southwest Jiaotong University
Zhijian YAN (yzjian@my.swjtu.edu.cn)
   Key Laboratory of Transportation Tunnel Engineering, Ministry of Education, Southwest Jiaotong University
Jiaming LIU
   Key Laboratory of Transportation Tunnel Engineering, Ministry of Education, Southwest Jiaotong University
Zhiyong WANG
   China Railway Design Corporation
Kaimeng MA
   Key Laboratory of Transportation Tunnel Engineering, Ministry of Education, Southwest Jiaotong University
Jianchi MA
   Key Laboratory of Transportation Tunnel Engineering, Ministry of Education, Southwest Jiaotong University
Bo WANG
   Key Laboratory of Transportation Tunnel Engineering, Ministry of Education, Southwest Jiaotong University

Article

Keywords: Energy Principle, Adjacent tunnel, Blasting Vibration, Numerical Simulation, Safety Criteria

Posted Date: November 17th, 2022

DOI: https://doi.org/10.21203/rs.3.rs-2238318/v1

License: This work is licensed under a Creative Commons Attribution 4.0 International License. Read Full License
Abstract

This study addresses the contradiction between the strict requirements of vibration speed control in existing operating tunnels and the difficulties of tunnel excavation in hard rock strata, as well as the lack of theoretical basis for blasting vibration speed control standards in adjacent tunnels. Based on the energy principle, this study establishes the energy balance equation in a single cycle of the low frequency band of blasting vibration wave curve, and proposes to replace the maximum elastic strain energy accumulated in concrete by twice the kinetic energy corresponding to the peak vibration speed under blasting action (the "Twice the Peak Kinetic Energy Method") according to the conversion relationship between the input energy of concrete mass unit and the elastic strain energy and kinetic energy. On the basis of this method, the formulae of safe vibration velocity of blasting in two states were derived by combining the energy criteria corresponding to the damage and destruction states of concrete. It is verified in a new tunnel under existing operating railway section in hard rock stratum in Xiamen area. The results show that the maximum principal stress and displacement reach the maximum value in the cycle when the peak vibration speed decays close to zero for the first time in the low frequency band; the elastic strain energy also reaches the maximum value in the cycle when the kinetic energy decays close to zero; affected by a small amount of dissipation energy, the kinetic energy corresponding to the peak vibration speed is slightly less than twice the maximum elastic strain energy, which verifies the reasonableness of the theoretical analysis. Finally, the safe vibration speed corresponding to different concrete grades is given based on the design value of tensile strength.

1 Introduction

With the continuous development of China's transportation industry, the number of tunnel construction is increasing, and the construction of new tunnels close to existing tunnels is becoming more and more common. At present, for hard rock conditions, especially in the eastern coastal region of China is widely distributed granite strata of tunnel excavation is still dominated by drilling and blasting method of excavation. Obviously, during tunnel construction by blasting method, the impact of blasting on the surrounding structures is unavoidable[1], and if blasting control measures are not appropriate, it is inevitable that damage or cracking of existing structures will be caused, which will endanger the operational safety of existing lines. In order to ensure the safety of existing operating lines, on the one hand, new tunnels are required to use non-blasting excavation methods or strictly limit the blasting vibration speed, but simply using mechanical excavation or strictly limiting the blasting vibration speed will cause difficulties in boring in hard rock strata; on the other hand, the control standards of blasting vibration vary greatly from country to country[2][3]. Moreover, the blasting vibration control standards in engineering practice are mainly determined by a large number of engineering examples and engineering analogies, which lack a certain theoretical basis. Therefore, it is necessary to carry out further theoretical research on the safety criterion of blasting vibration under the conditions of hard rock strata.

Many scholars have conducted a lot of research on the effects of blasting vibration in new tunnels close to existing buildings (structures). Japanese scholar Hisatake et al.[4] proposed that the maximum
damage stress point in tunnel blasting is related to the distance from the blast source, and obtained the conclusion that the vibration speed on the blast-face side is the maximum; Nakano et al.[5] studied the relationship between lining concrete cracking and blasting grade through field tests. Ramulu et al.[6] studied the dynamic response of tunnel structures with different cross-sectional forms under the action of blasting seismic waves. Zhao et al.[7] studied the safety and stability of existing tunnel linings under the action of blasting vibration in adjacent underpass tunnels by combining blasting vibration speed and vibration frequency. Duan et al.[8] studied the effect of tunnel blasting opening on the vibration speed and vibration frequency of existing high-voltage tower, taking the Chashan highway tunnel project as an example. Luo et al.[9] studied the distribution patterns of cracks at different locations in anchor-supported cavities under the action of blast stress waves. In terms of blasting vibration safety criteria or safety allowable criteria research, blasting vibration control criteria were studied in the United States and other countries as early as the 1920s[10]. YU et al.[11] proposed the dynamic stress ratio DSR as a safety criterion. Jiang et al.[12] used numerical simulation to analyze the vibration velocity and effective stress of tunnel lining and surrounding rock under blasting vibration based on the example of excavation and blasting near the top of railway tunnel, and determined the safety criterion of peak vibration velocity based on the maximum tensile strength theory and numerical calculation results. Xu et al.[13] established a safety criterion for blasting vibration in circular tunnel surrounds based on the Mohr-Coulomb strength criterion and the Fourier-Bessel expansion method using the response vibration speed, and proposed the maximum radial vibration speed for determining the safety criterion based on the calculation results. At present, most countries use "vibration speed-frequency" as the safety criterion for blasting vibration.

Energy has a clear physical meaning in the process of damage occurring in materials subjected to external forces until destruction, and the energy method can better reflect the vibration characteristics, especially the comprehensive effect of blasting on the damage of buildings (structures), and the transmission of blasting vibration is essentially the transmission of blasting energy[14–16]. Zhao et al.[17] analyzed the impact of tunnel blasting vibration from the perspective of vibration energy transfer and established a prediction model of blasting vibration energy by using the dimensional analysis method, which concluded that the energy of blasting vibration is mainly concentrated in the low frequency band (0 ~ 25 Hz). To address the shortcomings of the existing "vibration speed-frequency" safety criterion, Tang et al.[18] conducted an exploratory study of blasting seismic waves on existing buildings (structures) from the energy point of view and achieved certain application value. Li et al.[19] constructed a blasting seismic safety evaluation method based on the "equivalent peak energy" (EPE) by combining the energy damage mechanism of buildings. Scholars have conducted a series of studies on the effects of blasting vibration in existing tunnels, safety criteria, and the application of the energy method, such as the dynamic response law of existing structures, the maximum tensile strength theory to determine the safe vibration speed, and the blasting vibration energy evolution process to achieve certain results, which have practical guidance significance for engineering. However, the influence factors on the adjacent tunnel structure material itself are not well considered; the criterion based on the energy principle can reveal the nature of the hazard of blasting vibration to existing structures, but the energy evolution process is complicated when blasting seismic waves are input to existing structures, how to extract the
main energy that causes structural damage or destruction, and how to establish the internal connection with the commonly used mass point vibration criterion in engineering is rarely studied. By studying the peak kinetic energy and maximum elastic strain energy of the existing structural mass of blast vibration, it is possible to understand more accurately and comprehensively the deformation, force and damage of existing structures.

Based on the energy principle, this study first analyzes the energy criterion for damage and destruction by combining the stress-strain curve of concrete materials. Then the process of input and conversion of blasting energy in existing tunnel structure is analyzed, the energy balance equation in a single cycle of blasting vibration curve is established, and the relationship between peak vibration speed and material strength parameters is deduced according to the conversion relationship between elastic strain energy and kinetic energy for both cases of undamaged and undestroyed concrete materials. Finally, relying on the example of adjacent tunnel blasting projects with granite hard rock stratum conditions, numerical simulation is used to verify the energy method of safe vibration speed criterion, and analysis and discussion are carried out to propose a more applicable blasting vibration speed control standard for existing tunnel structures.

2 The Methodology

2.1 Energy analysis of concrete damage and destruction

The first law of thermodynamics reflects the conservation of different forms of energy in the process of transfer and transformation, energy transformation is the essential feature of the physical change process of materials, and material damage is a state destabilization phenomenon driven by energy[20]. For concrete lining structures, energy is easily released along the direction of minimum compressive stress or tensile stress, and the change of energy during failure is analyzed by the stress-strain curve of concrete under uniaxial tests, as shown in Fig. 1.

From the energy point of view, the damage process of concrete under external load can be divided into three stages: the first stage is the energy accumulation stage (OA, Elastic stage), this stage is mainly for the external input energy and its internal elastic strain energy conversion process; the second stage is the energy dissipation stage (AB, Elastic-Plastic stage), this stage is mainly for the external input energy and strain energy and dissipation energy conversion process; the third stage is the energy release stage (BC, Failure stage), this stage is mainly for the release of elastic strain energy and concrete fragmentation surface energy and kinetic energy conversion process, resulting in the overall destruction of concrete.

Based on the above three stages, the energy corresponding to the different stages is calculated from the area of the zone enclosed by the stress-strain curve of concrete. The maximum energy that can be absorbed in the first stage is recorded as $U_{OA}$, such as the area of the green area in Fig. 1, corresponding to the maximum stress $\sigma_A$; the maximum energy that can be absorbed in the second stage is recorded as
$U_{AB}$, the area of the yellow area in Fig. 1, corresponding to the maximum stress that is the peak stress $\sigma_B$.

The energy calculation formula corresponding to the two stages is as follows (1) and (2).

\[
U_{OA} = \frac{1}{2E_0}\sigma_A^2 \quad 0 < \varepsilon \leq \varepsilon_A
\]

1

\[
U_{AB} = \int \sigma d\varepsilon \quad \varepsilon_A < \varepsilon \leq \varepsilon_B
\]

2

In the formula: $E_0$ is the modulus of elasticity of concrete; $\varepsilon_A$ is the maximum strain of the elastic phase; $\varepsilon_B$ is the strain corresponding to the peak stress.

The maximum energy that can be absorbed without failure of the concrete is the sum of the energy accumulated in the first stage and the energy accumulated in the second stage, as in Eq. (3).

\[
U_{OB} = U_{OA} + U_{AB}
\]

3

In the equation: $U_{OB}$ is the maximum energy that can be absorbed without damage. When the input energy is less than $U_{OA}$, the concrete material is considered to be in the elastic phase and no damage occurs; when the input energy is greater than $U_{OA}$ and less than $U_{OB}$ when the concrete material is considered to be steadily expanding internal micro-cracks leading to damage, accompanied by irrecoverable plastic deformation, causing loss of strength; when the input energy is greater than $U_{OB}$ when the concrete occurs as a whole failure.

For the case of concrete without failure, this study uses the releasable elastic strain energy of concrete to characterize the maximum amount of energy that can be absorbed by concrete without failure. Figure 2 shows the relationship between the dissipated energy and the releasable elastic strain energy in the unit, the gray area is the energy dissipated by the damage and plastic deformation of the unit, and the blue area is the maximum releasable elastic strain energy stored in the unit. When the elastic strain energy is less than the maximum releasable elastic strain energy stored in the unit, the mass point unit is not destroyed as a whole under the action of external forces.

2.2 Blasting vibration safety criteria

When the tunnel is blasted, most of the energy generated by tunnel blasting is consumed in the crushing of the rock at the palm face, and a small portion of the energy is propagated outward along the geotechnical body in the form of stress waves, which are gradually converted into elastic strain energy, damping energy and kinetic energy of the geotechnical body in the process of propagation. Stress waves
cause vibration of geotechnical particles and existing tunnel structure, which may cause damage or
destruction of the surrounding rock and existing structure, and explore the nature of the process of energy
input, conversion and release. Using the vibration velocity time curve of concrete mass as shown in Fig. 3,
the energy conversion process of forced vibration of mass unit is divided into three stages.

**The first stage is the energy input stage**

the stress wave generated by the blast is the energy input source, under the action of the stress wave, the
mass unit is stressed to produce vibration acceleration, for a single cycle, in $\Delta t_1$ time, with the increase in
time the vibration speed gradually becomes larger (kinetic energy gradually increases), the resulting strain
also gradually increases (elastic strain energy increases), until it reaches the peak vibration speed $v_{\text{max}}$
(at this time the kinetic energy reaches the maximum, elastic strain energy is not the maximum), at this
time the stress wave input energy reaches the maximum, the input energy conversion as shown in
formula (4).

\[ U = U_m + U_e^1 + U_d \]

4

In the equation: $U$ is the maximum energy of the stress wave input; $U_m$ is the maximum kinetic energy at
the peak vibration speed; $U_e^1$ is the elastic strain energy accumulated in the moment of $\Delta t_1$; $U_d$ is the
energy loss generated by the vibration of the mass unit.

**The second stage is the energy conversion stage**

in $\Delta t_2$ time, the peak vibration speed is reached and then the vibration speed gradually decreases (kinetic
energy gradually decreases), but the strain generated at this time will still increase (elastic strain energy
continues to increase), until the kinetic energy in the cycle reaches the minimum value (the elastic strain
energy reaches the maximum), the energy conversion process in this stage is the conversion of kinetic
energy into elastic strain energy and dissipation energy, as shown in Eq. (5). Therefore, at the end of this
phase, the elastic strain energy of the mass unit reaches its maximum value, as shown in Eq. (6).

\[ U_m = U_e^2 + U_d \]

5

\[ U_e^{\text{max}} = U_e^1 + U_e^2 \]

6

In the equation: $U_e^{\text{max}}$ is the sum of the elastic strain energy accumulated in the moments $\Delta t_1$ and $\Delta t_2$;
$U_e^2$ is the elastic strain energy accumulated in the moment $\Delta t_2$.

**The third stage is the release of energy**
the accumulated elastic strain energy is gradually released and converted into kinetic energy and dissipative energy. From the whole vibration process, the total energy input of stress wave is finally converted into dissipative energy and residual elastic strain energy.

Based on the energy principle of the concrete failure criterion, it is known that the failure of concrete is due to the accumulation of elastic strain energy reaching the limit value, as long as the calculation of the elastic strain energy when the decay of the peak vibration speed is the minimum value, the safety criterion of concrete failure under blasting vibration can be derived. Specific analysis: the slope of the rising and falling sections of the peak vibration speed is the acceleration of the mass vibration, and the difference between the duration of the rising and falling sections of the peak vibration speed is not large ($\Delta t_1 \approx \Delta t_2$), that is, the acceleration of the mass in the rising and falling sections of the peak vibration speed is the same size, according to Newton's second law of motion, the displacement produced by the rising and falling sections of the peak vibration speed is the same. According to Hooke's law, the elastic strain energy of the rising and falling sections of the peak vibration speed is the same, as in Eq. (7).

$$U_e^1 = U_e^2$$

7

The elastic strain generated by the mass in the falling section can be converted to kinetic energy at the peak vibration speed, as in Eq. (8).

$$U_e^2 = U_m = 0.5 \Delta m v_{\text{max}}^2$$

8

Therefore, the maximum elastic strain energy of the mass unit is twice the kinetic energy corresponding to the peak vibration speed, as shown in Eq. (9).

$$U_e^{\text{max}} = U_e^1 + U_e^2 = 2 \times \frac{1}{2} \Delta m v_{\text{max}}^2 = \Delta m v_{\text{max}}^2$$

9

Then, according to the energy principle described above, the existing tunnels are divided into the following two methods of calculating the safety criterion for blast vibration:

For existing tunnel lining structures, a certain amount of elastic strain energy has been accumulated after the end of construction, and the elastic strain energy that has been accumulated in the static state can be calculated according to the stress state of the structural unit, in the principal stress space, the calculation formula is as in Eq. (10):

$$U_e^0 = \frac{1}{2E_0} [\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\nu(\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_1\sigma_3)]$$
In the equation: \( \sigma_1, \sigma_2, \sigma_3 \) are the three principal stresses corresponding to the maximum strain energy of the rock unit; \( \nu \) is the Poisson's ratio.

The existing tunnel structure does not produce damage to meet (11):

\[
U_{e}^{max} \leq U_{OA} - U_{e}^{0}
\]

Substitution of equations (1), (9) and (10) into Eq. (11) yields the vibration speed criterion for the existing tunnel structure without damage to any unit volume:

\[
\nu_{max} \leq \sqrt{\frac{1}{2 \rho E_0} \sigma_A^2 - \frac{1}{2 \rho E_0} \left[ \sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2 \nu (\sigma_1 \sigma_2 + \sigma_2 \sigma_3 + \sigma_1 \sigma_3) \right]}
\]

In the inequality: \( \rho \) is the density. In the design of hard rock tunnels, the secondary lining is often used as a safety reserve, which is subjected to smaller forces, so the elastic strain energy already stored in the tunnel secondary lining under static force is less, and the elastic strain energy already accumulated in the secondary lining is ignored in the calculation. The Eq. (12) is simplified as:

\[
\nu_{max} \leq \frac{\sigma_A^2}{\sqrt{2 \rho E_0}}
\]

The existing tunnel structure does not fail to meet (14):

\[
U_{e}^{max} \leq U^e - U_{e}^{0}
\]

In the inequality equation: \( U^e \) is the critical value of releasable elastic strain energy in tension. The releasable elastic strain energy of the tunnel lining concrete in compression is much greater than that in tension, and blasting damage in existing tunnels is often manifested in the form of cracks and tensile failure. The critical value of releasable elastic strain energy in tension is calculated as in Eq. (15).

\[
U^e = \frac{\sigma_t^2}{2 E_0}
\]
In the equation: $\sigma_t$ is the tensile strength of the concrete, which can be obtained by referring to the current code or by testing the value of this parameter. Substitution of equations (9), (10) and (15) into Eq. (14) yields the vibration speed criterion for existing tunnel structures without tensile failure to any unit volume.

\[
v_{\text{max}} \leq \sqrt[2]{\frac{\sigma_t^2}{2\rho E_0} - \frac{1}{2\rho E_0} \left[ \sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\nu (\sigma_1 \sigma_2 + \sigma_2 \sigma_3 + \sigma_1 \sigma_3) \right]}
\]

The calculation ignores the elastic strain energy $U_t^0$ that has accumulated in the secondary lining, and simplifies the Eq. (16) as follows:

\[
v_{\text{max}} \leq \frac{\sigma_t}{\sqrt{2\rho E_0}}
\]

### 3 Engineering Application

#### 3.1 Project Overview

New Damaoshan Tunnel 1 is located on the upstream connecting line from Xiamen North Station into the island of Fuzhou Xiamen Passenger Dedicated Line in Xiamen, Fujian Province, China. The surrounding environment and location of the project is shown in Fig. 4, the southwest direction is Xiamen North Station, from the figure can be seen that the interior of Damaoshan is a complex tunnel group consisting of several existing tunnels and new tunnels, the existence of parallel or up and down overlapping tunnels, tunnel lines are complex. The New Damaoshan tunnel 1 is a single line tunnel, 1280m long, with a design speed of 120km/h. The tunnel inlet mileage is XLSDK1 + 725, the exit mileage is XLSDK3 + 005, the maximum depth of 115 m. The New Damaoshan tunnel 1 at mileage XLSDK2 + 090 crosses the existing line 2 New Liutang tunnel 1, the length of the cross section is 455 m, the minimum net distance between the two tunnels is 8.392 m, the intersection angle between the tunnels is about 30°. New Liutang tunnel secondary lining concrete grade for C35, arch wall thickness of 30cm, back arch thickness of 40cm. intersection section of the cave surrounding rock is granite, weak weathering, rock hard.

The difficulties of the project: two tunnels net distance is small. existing line train operation is busy, the new tunnel construction shall not affect the safe operation of existing tunnels. high rock strength, conventional mechanical excavation is difficult to ensure smooth construction, so the new tunnel using blasting method excavation control requirements are very high.

New Damaoshan Tunnel 1 is excavated by controlled blasting using the bench cut method. The upper bench uses two-stage compound wedge-shaped trenching, the trenching holes are located in the middle of the section, 6 primary trenching holes, single hole charge 0.6kg, spacing 125cm, row spacing 35cm.
Secondary trenching hole 6, single hole loading 0.6kg, spacing 210cm, row spacing 35cm. There are 67 auxiliary holes with a single charge of 0.4kg, a spacing of 55cm and a row distance of 70cm; 52 peripheral holes with a single charge of 0.2kg and a hole distance of 30cm. The arrangement of the blasting holes is shown in Fig. 5.

### 3.2 Rock mechanics test

The rock mechanics test was carried out using MTS-815 electro-hydraulic servo rock triaxial testing machine. The rock samples used were taken from the granite at the tunnel face of tunnel XLSDK1 + 890 in New Damaoshan 1, and were made into standard rock samples for the test by drilling core sampling, cutting and grinding, as shown in Fig. 6(a), with a diameter of 50mm and a height of 100mm, for a total of 8 samples, and the test conditions are shown in Table 1; then the indoor rock mechanics triaxial test was conducted, as shown in Fig. 6(b).

<table>
<thead>
<tr>
<th>Samples number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Confining pressure (MPa)</td>
<td>0</td>
<td>1</td>
<td>3</td>
<td>5</td>
<td>10</td>
<td>15</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The test results are shown in Fig. 7. Figure 7(a) shows the uniaxial compression stress-strain curves of samples 1 ~ 3, and the average compressive strength is 147.90MPa, which indicates the high uniaxial compressive strength of granite in the field; Fig. 7(b) shows the triaxial compression stress-strain curves of samples 4 ~ 8, and the compressive strength increases as the surrounding pressure increases. Uniaxial compression and triaxial compression indicate that the granite rocks present brittle failure.

Based on the test results of samples 4 ~ 8, the best relationship curve was calculated based on the least squares method with b as the horizontal coordinate and \((\sigma_1 - \sigma_3)/2\) as the vertical coordinate. Five points (A, B, C, D, E) were selected on the best relationship curve, as shown in Fig. 8(a). Then the corresponding Mohr stress circle is drawn with the \((\sigma_1 + \sigma_3)/2\) as the center and the vertical coordinate \((\sigma_1 - \sigma_3)/2\) as the radius, and the strength envelope of the Mohr stress circle is made, as shown in Fig. 8(b). From the Mohr envelope, it is known that the internal friction angle \(\varphi\) of the granite specimen is 59° and the cohesive force is 23.13MPa.

The physical and mechanical parameters of the rocks were obtained from the uniaxial compression test as well as the triaxial compression test, and the physical and mechanical parameters of the rocks at the site were obtained by the Hoek-Brown criterion correction[21][22]. The physical and mechanical parameters of the surrounding rocks were finally determined as shown in Table 2, combined with TB 10003 – 2016: Code for Design on Tunnel of Railway[23].
### 3.3 Numerical calculation

#### 3.3.1 Numerical model

The numerical calculation is carried out in the engineering background of the New Damaoshan Tunnel 1 under the New Liutang Tunnel 1. The model size is 100m×100m×10m, and the calculation model is shown in Fig. 9(a). The surrounding rock is elastic-plastic material, and the Mohr-Coulomb constitutive model is used, and its calculation parameters are shown in Table 2; the secondary lining structure of the existing tunnel is simulated using the elastic constitutive model, and its parameters are selected according to the GB 50010 – 2010: Code for design of concrete structures[24], as shown in Table 3. The inverted arch of the existing tunnel is on the blast side, and the monitoring point is set as the center point of the inverted arch of the existing tunnel, as shown in Fig. 9(b). The boundary conditions used for the calculation are viscous boundary. The damping of the calculation process is selected from Rayleigh damping, which is determined by the critical damping ratio and the minimum center frequency, and the critical damping ratio of the geotechnical body is taken to be 5% in this study; the self-vibration frequency of the geotechnical body is used instead of the minimum center frequency, which can be achieved by the free vibration of the geotechnical body when the ground stress is in equilibrium, and the model self-vibration curve is shown in Fig. 9(c), and the minimum center frequency is calculated to be 4.2Hz.

### Table 2

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight $\gamma$ (kN/m$^3$)</th>
<th>Elastic modulus $E$ (GPa)</th>
<th>Poisson's ratio $\nu$</th>
<th>Friction angle $\varphi$ (°)</th>
<th>Cohesion $c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surrounding rock</td>
<td>25.94</td>
<td>6.5</td>
<td>0.1</td>
<td>44.55</td>
<td>6.43</td>
</tr>
</tbody>
</table>

### Table 3

<table>
<thead>
<tr>
<th>Concrete Grades</th>
<th>Weight $\gamma$ (kN/m$^3$)</th>
<th>Elastic modulus $E$ (GPa)</th>
<th>Poisson's ratio $\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C35</td>
<td>25</td>
<td>31.5</td>
<td>0.2</td>
</tr>
</tbody>
</table>
The blasting load is applied by equivalent load, and the blasting vibration generated by the trenching hole has the greatest impact, and the equivalent load of the trenching hole is applied to the tunnel excavation contour line. The blasting load was simplified to a triangular load consisting of ascending and descending sections, and the ascending time of the blasting load was taken as 10ms, the unloading time was taken as 90ms, and the total action time was 100ms. The peak load $P_{\text{max}}$ was calculated according to empirical equations \([25]\) (18) and (19).

\[
P_{\text{max}} = \frac{139.97}{Z} + \frac{844.41}{Z^2} + \frac{2154}{Z^3} - 0.8034
\]

18

\[
Z = \frac{R}{Q^{1/3}}
\]

19

In the equation: $Z$ is the proportional distance; $R$ is the distance from the trenching hole to the load surface in m; $Q$ is the charge of the blasting hole in kg. Relying on the actual project, $R$ is taken as 2.5m, and the maximum charge of blasting 12 trenching holes together is 7.2kg, and the blasting load time curve and the applied schematic in the model are calculated as shown in Fig. 10.

### 3.3.2 Analysis of results

Figure 11 shows the speed propagation cloud during the dynamic calculation. From the cloud, it can be seen that the peak speed of the whole stratum and the center of the inverted arch of the existing tunnel reaches 10ms after the blasting load is applied, and with the increase of time, the peak speed propagates gradually from the bottom to the top of the elevation arch of the existing tunnel, which also shows that the center of the inverted arch of the existing tunnel is indeed the most dangerous area on the blast side, and is also the area where cracks are most likely to occur.

Figure 12(a) shows the time history of the synthesis vibration speed at the monitoring point in the center of the inverted arch of the existing tunnel, the maximum vibration speed is 4.4cm/s. The kinetic energy time history of the monitoring point is calculated and plotted according to the vibration speed, as shown in Fig. 12(b), the maximum kinetic energy is 2.47J. Because the kinetic energy is positively correlated with the vibration speed, the kinetic energy reaches its maximum value when the vibration speed reaches its maximum value, and the change law of the kinetic energy time history curve is consistent with the change law of the synthesis vibration speed time history curve. 10ms when the blast load reaches its peak value, and the peak vibration speed at the monitoring point in the center of the inverted arch is about 13ms, which means that it takes about 3ms for the peak load to propagate to the existing tunnel. Figure 12(c) shows the time course curve of the maximum principal stress at the monitoring point in the center of the inverted arch, and the maximum principal stress is 0.64MPa. The elastic strain energy of the monitoring point is calculated and the time history curve of the elastic strain energy is drawn, and the maximum elastic strain energy is 6.5J as shown in Fig. 12(d).
In order to analyze the relationship between the maximum principal stress and the vibration speed, the Z-directional vibration speed time history curve and the maximum principal stress time history curve of the center monitoring point of the inverted arch are plotted under the same coordinate system, as shown in Fig. 13. The maximum principal stress is not the maximum value when the vibration speed of the mass point reaches the maximum value, but the maximum principal stress reaches the maximum value only when the peak vibration speed decays basically for the first time.

In order to analyze the relationship between vibration speed and displacement, the Z-directional vibration speed time course curve and displacement time history curve of the monitoring point in the center of the inverted arch are plotted to the same coordinate system, as shown in Fig. 14. The displacement of the monitoring point reaches the maximum value at this time when the vibration speed reaches the peak and decays to zero for the first time, and then the vibration speed corresponds to the maximum value of the displacement in the decay process when each decay is zero, respectively, in that cycle.

In order to analyze the energy conversion relationship, the kinetic energy as well as the elastic strain energy time curves of the center monitoring point of the inverted arch are plotted in the same coordinate system, and only the 100ms curve is intercepted, as shown in Fig. 15. It can be seen from the figure that, as the result of the theoretical analysis, the kinetic energy as well as the elastic strain energy of the mass point unit increases when the vibration speed increases toward the peak vibration speed (within \( \Delta t_1 \) time), but the kinetic energy at the peak vibration speed is not all the elastic strain energy acting on the mass point unit. When the vibration speed reaches the peak speed and starts to decay (within \( \Delta t_2 \) time), the elastic strain energy still increases until the peak speed decays to a small enough size, when the elastic strain energy of the mass point unit also reaches its maximum value.

## 4 Discussion

From the numerical simulation results, it can be obtained that the energy of new tunnel blasting on the existing tunnel lining structure is mainly concentrated in the low frequency band, which is consistent with the results of the literature[17], indicating that the energy curve obtained from the numerical simulation is reasonable. The elastic strain energy accumulated in \( \Delta t_1 \) time is 1.8J, and the elastic strain energy accumulated in \( \Delta t_2 \) time is 1.7J. The elastic strain energy accumulated in \( \Delta t_2 \) time is smaller than that in \( \Delta t_1 \) time due to partial energy dissipation. The elastic strain energy in \( \Delta t_2 \) time is converted from the kinetic energy at the peak vibration speed, which is 2.47J. The reason why the two are not identical is that there is a dissipation of kinetic energy in the process of conversion to elastic strain energy, but ignoring the dissipation of energy is more conservative for the calculation results. In other words, it is more conservative to use the kinetic energy at twice the peak vibration speed to characterize the maximum elastic strain energy, so the maximum elastic strain energy can be replaced by twice the peak kinetic energy, which verifies the rationality of the "Twice the Peak Kinetic Energy Method" from the perspective of numerical calculation.
Combining the theoretical research results, the design values of concrete tensile strength and modulus of elasticity according to the GB 50010 – 2010: Code for design of concrete structures[24] can be obtained for each grade of concrete vibration speed control threshold, as shown in Table 4. If the existing tunnel has been in operation for a long time and the lining shows some deterioration, the values of the material parameters required for Eq. (15) can be obtained by means of on-site coring followed by an indoor test. The maximum vibration speed obtained from the numerical simulation is less than the control threshold and the existing tunnel structure is in a safe condition, indicating that the controlled blasting design adopted for the new tunnel is reasonable.

<table>
<thead>
<tr>
<th>Concrete Grades</th>
<th>C25</th>
<th>C30</th>
<th>C35</th>
<th>C40</th>
<th>C45</th>
<th>C50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design value of tensile strength (MPa)</td>
<td>1.27</td>
<td>1.43</td>
<td>1.57</td>
<td>1.71</td>
<td>1.80</td>
<td>1.89</td>
</tr>
<tr>
<td>Vibration speed threshold (cm/s)</td>
<td>10.73</td>
<td>11.68</td>
<td>12.51</td>
<td>13.41</td>
<td>13.91</td>
<td>14.39</td>
</tr>
</tbody>
</table>

5 Conclusion

(1) Based on Newton's second law of motion and Hooke's law, a method is proposed to calculate the maximum elastic strain energy of a mass unit as twice the kinetic energy corresponding to the peak vibration velocity. And combined with the critical value of energy corresponding to concrete damage and failure, the safe vibration speed criterion is deduced under the condition of no damage and no destruction of existing tunnel structure concrete during blasting vibration under hard rock conditions.

(2) Using numerical calculation methods to simulate the blasting excavation of a new tunnel under granite stratigraphic conditions through an existing operating railroad tunnel project, the maximum vibration speed is less than the control threshold, and the controlled blasting design of the new tunnel is reasonable to ensure that the existing tunnel structure is in a safe state. The energy evolution process of the existing tunnel monitoring points is consistent with the theoretical analysis. Except for the partial energy dissipation, the elastic strain energy accumulated in the same cycle for two periods is slightly less than twice the kinetic energy corresponding to the peak vibration speed, which verifies the correctness of the "Twice the Peak Kinetic Energy Method".

(3) Based on the safety criterion of "Twice the Peak Kinetic Energy Method", the blasting speed control thresholds for different strengths of concrete are calculated by the design value of concrete tensile strength. The results of the study can provide theoretical reference for the control of blasting vibration speed in similar proximity construction projects.

Declarations

Acknowledgements
The authors gratefully acknowledge the financial support from the High Speed Rail Joint Fund Project, National Natural Science Foundation of China (U2034205).

Contributions

Junru ZHANG provided research ideas and guided the framework of the article. Zhijian YAN wrote the main manuscript text and assembled the manuscript. Jiaming LIU and Zhiyong WANG wrote part of the content. Kaimeng MA involved in writing and revising manuscripts, as well as adjusting Figures. Jianchi MA prepared Figs. 4, 5, 6, 7, 8. Bo WANG provided modification suggestions and fund support.

Data availability statement

The datasets used and/or analysed during the current study available from the corresponding author on reasonable request.

References

7. Hua-bing Zhao,Yuan Long,Xing-hua Li,Liang Lu. Experimental and numerical investigation of the effect of blast-induced vibration from adjacent tunnel on existing tunnel. KSCE Journal of Civil Engineering. 20(1),(2016)
9. Luo Yi,Pei Chenhao,Qu Dengxing,Li Xinpeng,MA Ruiqiu,Gong Hangli. Distribution of cracks in an anchored cavern under blast load based on cohesive elements. Scientific reports. 12(1),(2022)

Figures
Figure 1

Stress-strain curve of concrete under uniaxial test
Figure 2

Dissipated energy and releasable elastic strain energy in unit volume
Figure 3

The process of energy input, conversion and release of the mass unit
Figure 4

Project area and location relationship

Figure 5

Layout of tunnel blasting holes ((a) Layout of Cross Section Blasting Holes; (b) Layout plan of cutting hole)
Figure 6

Rock mechanics experiment ((a) Sample preparation; (b) Laboratory test)

Figure 7

Stress-strain curves of different specimens ((a) Sample 1~3 (b) Sample 4~8)
Figure 8

Analysis of test data ((a) Optimal Relationship Curve (b) Mohr’s stress circle)
Figure 9

Numerical model
(a) The whole model
(b) Location of monitoring point
(c) Model natural vibration curve

Figure 10

Equivalent blasting load
Figure 11

Vibration speed nephogram during calculation
Figure 12

Analysis curve of inverted arch center monitoring point
Figure 13

Time history of z-directional vibration speed and maximum principal stress in the center of the inverted arch
Figure 14

Time history of z-directional vibration speed and displacement in the center of the inverted arch
Figure 15

Time history curves of kinetic energy and elastic strain energy of inverted arch