Numerical Simulation of Explosion of Late-excavated Tunnel Parts of Double-arch Tunnel without Middle Drift

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Abstract. Compared with the construction method of a tunnel with a middle drift, the construction method of a tunnel without a middle drift is a more advanced, novel, efficient, and energy efficient. Moreover, the blasting excavation of late-excavated tunnel parts exerts a vibration impact on the lining structure of early excavated tunnel parts. To analyze and predict the influence of late-excavated tunnel blasting on an early excavated tunnel and surrounding rocks in a triangular area, this study uses the Ansys LS-DYNA software to build a three-dimensional tunnel model simulating multihole blasting and the fluid–solid interaction method to assess the interaction of different materials. In addition, the different working conditions of surrounding rock levels and explosive equivalents are simulated. The distribution law of peak particle vibration velocity and maximum principal stress peak value of surrounding rocks and the lining along the transverse and longitudinal directions of the early excavated tunnel are generalized based on the numerical simulation results. The influence of the blasting of a late-excavated tunnel on the initial lining, secondary lining, and the middle triangular area of a double-arch tunnel is analyzed. The relationship between the surrounding rock level, explosive equivalent, and vibration velocity peak value is concluded. The field experiment-measured vibration velocities from the double-arch tunnel without a middle drift in the Chenjiachong tunnel of the Chuyao expressway in Yunnan Province, China, are in satisfactory agreement with the numerical simulation results. The results of this study are verified by the field blasting vibration data of the Chenjiachong and Changba tunnels and can provide reference and guidance for the blasting engineering practices of late-excavated double-arch tunnels without a middle drift.

1. Introduction

With the development of the economy and rapid advancements in mountain tunnels, a large number of near construction projects have emerged in the form of new structures adjacent to existing structures¹, upper and lower cross tunnels², small clear-distance parallel tunnels³, multiarch tunnels⁴, and so on. As one of the main construction techniques of the New Austrian tunneling method, drilling and blasting construction remains the main construction method for mountain tunnels around the world.
Drilling and blasting construction inevitably produces vibration hazards for surrounding structures. How to ensure the smooth progress of tunnel drilling and blasting construction and reduce impacts on existing structures are research hotspots in the blasting field. Presently, though studies exist on the blasting vibrations of small clear-distance tunnels[5] upper and lower cross connection tunnels[6], and other close connection projects[7], relatively few studies are conducted on the impact of blasting vibrations on the first tunnel in the construction of a second tunnel without a middle pilot tunnel. A multiple-arch tunnel without a middle drift is a new construction method for such tunnels. Construction without a middle drift is an advanced, novel, and energy-efficient construction method for multiarch tunnels. The construction method without a middle drift can accelerate construction progress and reduce the number of temporary supports and project costs. In addition, the left and right tunnel waterproof and drainage systems are independent of each other, thereby eliminating the water leakage problem in the middle wall. In recent years, newly developed double-arch tunnels without a middle drift have inherited the advantages of traditional double-arch tunnels and demonstrate characteristics such as simple working procedures and minimal disturbance to surrounding rocks. These traits are helpful in shortening construction periods, reducing costs, and overcoming several disadvantages of traditional double-arch tunnels. Given that no clear distance exists between two arches in a tunnel, the middle wall consists of the first support and second lining of the two tunnels, and the thickness of the middle wall is thin. Consequently, the blasting vibrations of late-excavated tunnel excavations have a considerable influence on early excavated tunnels, specifically, rock mass damage in the triangular area between adjacent multiarch tunnels, vibrations of rock mass in the middle, even in far areas, and other hazards. These effects cause deformations and damage to surrounding rocks and the lining structure of the triangular area between the arches and adjacent early excavated tunnels. How to control the influence of the blasting excavation of late-excavated tunnels on the lining of early excavated tunnels and ensure the safety of early excavated tunnels are among the difficulties and major problems encountered in the construction of multiarch tunnels without a middle drift.

To analyze and predict the influence of late-excavated tunnel blasting on early excavated tunnels and surrounding rocks in a triangular area, this research uses the Ansys LS-DYNA software to build a three-dimensional tunnel model simulating multiblast hole blasting and the fluid structure coupling method to assess the interaction of different materials. Moreover, this study selects four working conditions of different surrounding rock levels and varying explosive equivalents and analyzes the influence of the lining of an early excavated tunnel when the tunnel is blasted. In addition, the distribution law of peak particle vibration velocity and the maximum principal stress peak value of the lining along the transverse and longitudinal directions of an early excavated tunnel are analyzed. The influence of the blasting of a late-excavated tunnel on the initial lining, secondary lining, and the middle triangular area of a double-arch tunnel is likewise analyzed. Finally, the relationship between the surrounding rock level, the explosive equivalent, and the vibration velocity peak value is determined.

In this research, vibration characteristics and the propagation law of the explosion wave after the blasting of a double-arch tunnel without a middle drift are examined. Furthermore, the relationship between the vibration intensity and charge of a single-blasting cutting hole is discussed. It can guide blasting design onsite, reasonably adjust blasting parameters, timely and accurately predict possible accidents, and avoid accidents. The results of this research on blasting technology for multiple-arch tunnels without a middle drift can help resolve the difficult blasting construction problems encountered in the Chenjiachong tunnel and other multiple-arch tunnels without a middle drift and provide a reference for similar blasting projects in the future.

2. Engineering background
The design layout of the Chenjiachong tunnel is a double-arch tunnel, with a total length of 425 m, a section width of 11.49 m, a height of 7.1 m, and a maximum buried depth of 54.22 m. The surrounding rocks of the tunnel are mainly quaternary eluvial diluvial silty clay and J1t mudstone with a thin layer of argillaceous siltstone on the upper Jurassic system (see Table 1). The overlying strong-
to-moderately weathered mudstone with argillaceous sandstone and fissures is highly developed, the rock mass is fragmentary, and the local structure is inlaid with a fragmentary structure. The stratum of the tunnel crossing is composed mainly of mudstone and argillaceous siltstone, wherein grade V surrounding rock accounts for 30%, and grade IV rock accounts for 70%. Mudstone, argillaceous structure, medium-thick layer, moderately weathered differential weathering, extremely developed joints and fissures, gravelly, broken rock mass, argillaceous siltstone, particle structure, medium-thick layer structure, calcium argillaceous cementation, moderately weathered, fragmentary, relatively complete rock mass, massive mosaic structure or layered structure. In conclusion, the surrounding rocks belong to the typical red-bed soft rocks of Central Yunnan.

Table 1. Physical and mechanical property index of rocks

| Rocks                          | Natural density (g/cm³) | Water absorption (%) | Porosity (%) | Softening coefficient | Compressive strength (Mpa) |
|-------------------------------|------------------------|----------------------|--------------|-----------------------|----------------------------|
|                               |                        |                      |              |                       | Dry | Saturated     |
| Moderately weathered mudstone | 2.587                  | 4.06                 | 7.84         | 0.68                  | 17.7 | 10.61        |
| Moderately weathered argillaceous siltstone | 2.64 | 1.83 | 4.7 | 0.74 | 46.3 | 35.63 |

Given that a multiarch tunnel without a middle drift tunnel employs a design without a middle partition wall, two independent single tunnels are constructed. The early excavated tunnel is excavated first, and the late-excavated tunnel is excavated after the primary and secondary linings of the early excavated tunnel are completed. Table 2 presents the construction process, and Fig. 1 shows the excavation diagram of the Chenjiachong multiarch tunnel.

Table 2. Construction procedure of double-arch tunnel without middle drift

| Step | Construction part I | Construction part II | Cross operation |
|------|---------------------|----------------------|-----------------|
| 1    | Excavation and support of early excavated tunnel |                       |                 |
| 2    | Inverted pouring for early excavated tunnel | Late-excavated tunnel excavation and support | Parallel construction of each process |
| 3    | Secondary lining pouring for early excavated tunnel | Inverted pouring for late-excavated tunnel |                 |
| 4    | Secondary lining pouring for late-excavated tunnel |                 | Construction with a distance of 40 m between the late-excavated tunnel face and the secondary lining of the early excavated tunnel. |
Therefore, the pressure on the left and right sides of the double-arch tunnel without a middle drift tunnel is concentrated in the middle part. In view of the above situation, during the construction process, grouting measures for the base of the triangular area at the top of the survey center line of the double-arch tunnel and bottom of the arch foot are emphasized. This procedure is performed to strengthen the self-stability capacity of surrounding rocks and the bearing capacity of the base and reduce the pressure of the rock mass at the top of the tunnel on the tunnel. The inverted part of the tunnel is followed up over time to achieve early closure and timely looping to share the local concentrated load.

3. Parameter selection and establishment of numerical model

3.1. Unit type and material of numerical model
A three-dimensional tunnel model is established using the Ansys LS-DYNA software and SOLID164 element form. The model consists of four parts, namely, rocks, explosives, reinforced concrete, and air. The explosives are the LS-DYNA software’s high-performance explosive materials, with a keyword of *MAT_HIGH_EXPLOSIVE_Burn. The Johnson Holmquist model is selected for the rocks and concrete materials. The model is suitable for large strains, high strain rates, and high pressure conditions. The equivalent strength of the rocks is related to pressure, strain rate, and damage. The keyword used is *MAT_JOHNSON_HOLMQUIST_CONCRETE. By adding to the keyword *MAT_ADD_EROSION from the K file, rock material is used to define the compressive strength and failure principal stress of rocks as the failure criteria such that when the compressive strength of the rocks or the unit principal stress reaches the set value, the rock will fail, thereby simulating the failure of rock blasting.

3.2. Parameter selection of numerical model
The Johnson Holmquist model can be used for concrete subjected to large strains, high strain rates, and high pressures. The equivalent strength is expressed as a function of pressure, strain rate, and damage. Pressure is expressed as a function of volumetric strain and includes the effect of...
permanent crushing. Damage is accumulated as a function of plastic volumetric strain, equivalent plastic strain, and pressure. The normalized equivalent stress is defined as follows:

\[ \sigma^* = A(1 - D) + B P^{\gamma N} \left[ 1 + C \ln(\dot{\varepsilon}^*) \right], \]

where \( D \) is the damage parameter, \( P^* = P/f^*_c \) is the normalized pressure, \( f^*_c \) is the quasistatic uniaxial compressive strength, and \( \dot{\varepsilon}^* = \varepsilon/\dot{\varepsilon}_0 \) is the dimensionless strain rate. The model incrementally accumulates damage \( D \) from equivalent plastic strain and plastic volumetric strain and is expressed as

\[ D = \sum \frac{\Delta \varepsilon_p \Delta \mu_p}{D_1(P^* + T^*)^2} \]

where \( \Delta \varepsilon_p \) and \( \Delta \mu_p \) are the equivalent plastic strain and plastic volumetric strain, \( D_1 \) and \( D_2 \) are material constants, and \( T^* = T/f^*_c \) is the normalized maximum tensile hydrostatic pressure. The main selected mechanical parameters of the concrete and surrounding rocks in the numerical model are shown in Table 3.

| Material       | Density (kg/m³) | Dynamic elasticity modulus (Gpa) | Poisson’s ratio | Compressive strength (Mpa) | Dynamic shear modulus (Gpa) | Cohesion (Mpa) | Friction angle (Å) |
|----------------|----------------|---------------------------------|----------------|-----------------------------|-----------------------------|----------------|--------------------|
| Grade V surrounding rock | 2374 | 2.03 | 0.24 | 17.73 | 0.164 | 0.2 | 23 |
| Grade IV surrounding rock | 2587 | 8.635 | 0.28 | 46.25 | 0.762 | 0.5 | 34 |
| C25 | 2380 | 28 | 0.3 | 25 | 6 |
| C30 | 2385 | 35 | 0.3 | 30 | 8 |
| Air | 1.164 | 0 | 0 | 0 |

The detonation pressure is calculated by the JWL equation of state, as follows:

\[ P = A \left( 1 - \frac{\omega}{R_1 V} \right) e^{-R_1 V} + B \left( 1 - \frac{\omega}{R_2 V} \right) e^{-R_2 V} + \frac{\omega E}{V} \]

where \( P \) is the detonation pressure, \( E \) is the internal energy of the detonation product, \( V \) is the relative volume of the detonation product, and \( A, B, R_1, R_2, \) and \( \omega \) are the property constants of the selected explosive.

The selected explosive materials and state equation parameters of the numerical model are listed in Table 4.

| Density (g/cm³) | Detonation velocity (cm/us) | Explosion pressure (Gpa) | A (Gpa) | B (Gpa) | R₁ | R₂ | ω | E (Gpa) |
|----------------|-----------------------------|--------------------------|---------|---------|-----|-----|----|---------|
| 1.26           | 0.55                        | 3.43                     | 321.9   | 0.182   | 4.2 | 0.8 | 0.15 | 3.51    |

### 3.3. Size selection of numerical model

According to the design section and geological investigation report of the Chenjiachong multiarch tunnel of the Chuyao Expressway in Yunnan Province, a 20 m-long section with a completed early excavated tunnel secondary lining and an unexcavated late-excavated tunnel is selected for the dynamic analysis. It is used mainly to analyze the influence of the early excavated tunnel on lining vibrations when blasting excavation is conducted for the late-excavated tunnel. To eliminate the influence of the transverse boundary, the single tunnel in the Chenjiachong multiarch tunnel is 11.49
m wide and 7.1 m high. Therefore, the calculation size selected in this model is 60 m × 45 m × 20 m, and the buried depth of the tunnel vault is 21.45 m. The primary and secondary linings of the early excavated tunnel are completed, 10 m of the late-excavated tunnel has been excavated, and the primary lining is completed. Given that the explosives used in the construction site are no. 2 rock emulsion explosives, the blast hole diameter is set to 42 mm, and the explosive diameter is set to 32 mm in the three-dimensional numerical model.

3.4. Meshing of numerical model
In the finite-element three-dimensional numerical model, the grid connection of the explosives, air, rocks, and concrete adopts the common node. The element grid in the explosive area is very dense to simulate the explosive element effectively. The behavior of a solid under an impact load is similar to that of a fluid. A tetrahedron element is too rigid, whereas a hexahedron element demonstrates high precision and satisfactory quality. Therefore, an eight-node hexahedron isoparametric solid element, such as SOLID164, is used in the model. In the numerical model, the influence of blasting vibrations on the surrounding rocks and lining of the early excavated tunnel is analyzed. The side length of the hexahedron grid in the explosive area is approximately 2.6 mm; the grid size of the transition section from the explosive area to the excavation area is approximately 25.3 mm; the side length of the hexahedron grid in the excavation section rocks, intermediate rocks, primary lining, and secondary lining is approximately 170 mm; the side length of the hexahedron grid in the peripheral rocks is 200 mm, 400 mm, 600 mm, and 1200 mm; the length–width ratio of the hexahedron elements is not more than 3; and the quality of the elements is strictly controlled. The total number of model elements is 2,472,400, and the total number of model nodes is 2,521,670. The three-dimensional numerical grid model is shown in Fig. 2 (a), and the grid of the primary and secondary linings is shown in Fig. 2 (b).

3.5. Boundary condition of numerical model
A boundary condition without reflection is applied to the boundary surface[12], and the *BOUNDARY_NON_REFLECTION keyword is used. Except for the top of the numerical model, all the other surfaces restrict normal movement.

3.6. Calculation method of numerical model
As mentioned previously, SOLID164 solid elements are used for the rocks, concrete, explosives, and air. Given that the rocks adopt a constant-stress solid element, the element belongs to a pure Lagrange algorithm. The element mesh is attached to the material, and element mesh deformation occurs with the flow of the material. However, a large structural deformation may cause serious finite-element mesh distortions and numerical calculation difficulties and even terminate program operations. Thus, the algorithm is not suitable for the air and explosives. The explosion time is very short, and a strong shock wave is generated instantly, thereby damaging surrounding objects. During the explosion process, the air and explosives deform substantially; thus, the Arbitrary Lagrangian Eulerian (ALE) algorithm is adopted for the explosives and air[11]. The ALE algorithm can overcome the numerical calculation difficulties caused by serious distortions and realize the dynamic analysis of fluid structure coupling. This algorithm is an ALE multimaterial element with a single-point integration at the center, that is, the element can contain multiple materials. The ALE algorithm first performs one or several Lagrange time steps, in which the cell grid deforms with the material flow and performs the following ALE time steps. First, retaining the boundary conditions of the deformed object, the internal elements are remeshed, and the topological relationship of the mesh remains unchanged, thereby becoming a smooth step. Second, the element variables (e.g., density, stress tensor, energy, and so on) and node velocity vectors in the deformed mesh are transported to a new mesh, thereby becoming the expectation step. In addition, by selecting the start and end times of the ALE time step and its frequency, ALE algorithm can address the problem of large deformations, such as in the explosives and air, and resolve the mesh distortion. The keyword *ALE_MULTI_MATERIAL_GROUP binds
the air and explosive materials in an element algorithm, and the interaction between the rocks, explosives, and air adopts the fluid structure coupling method. Two fluid structure coupling methods exist, that is, the common node and the keyword *CONSTRAINED_LAGRANGE_IN_SOLID. The first method is used in the model calculation.

The explosives in the model are arranged in the middle of the back-row tunnel face. To simulate the charging mode of the cut hole in the field blasting, eight blast hole coupling charges are set in the model, as shown in Fig. 2 (c). The depth of the explosive unit is 1.2 m, the end of the explosive unit is set as the initiation point, the physical calculation time of the model is 40 ms, and the scaling factor of the time step is 0.67.

(a) Overall drawing of the three-dimensional numerical grid model

(b) Grid of primary and secondary linings
4. Analysis of numerical simulation results

4.1. Comparison between numerical simulation results and field-measured data

To verify the accuracy of the numerical model, the typical blasting field measurement results are selected, and the field vibration monitoring is shown in Fig. 3. During the typical blasting operation, the mileage of the upper step face of the late-excavated tunnel is ZK10+139.4. The lithology of the face of the late-excavated tunnel is moderately weathered mudstone mixed with argillaceous sandstone, and the fissures are highly developed. The grade V rock mass is fragmentary and partially a mosaic fragmentary structure. The actual charge quantity of the cut hole in the field blasting is 8.4 kg, which is close to 7.35 kg in the numerical simulation of the grade V surrounding rock. The node at the position similar to that in the blasting monitoring is selected, that is, the blasting point of the late-excavated tunnel face corresponds to the mileage of the early excavated tunnel. In the numerical model, node N4219308 is selected. The resultant velocity time-history curve of node N4219308 and the resultant velocity time-history curve from the field are shown in Fig. 4. The time-history curve of the node X-direction vibration speed and the time-history curve of the field-measured X-direction speed are shown in Fig. 5.
Figure 3. Onsite blasting vibration monitoring of Chenjiachong multiarch tunnel

(a) Resultant velocity of typical node in the numerical model

(b) Measured resultant velocity waveform in the field

Figure 4. Resultant velocity comparison between the numerical simulation and field measurements
For the 7.35 kg explosive equivalent of the face blasting of the cut hole, Figs. 6 and 7 show that the maximum resultant velocity value of the typical node is 24.3 cm/s, corresponding to the maximum resultant velocity value of the field-measured point, which is 22.18 cm/s. The X-direction peak particle velocity (PPV) of the typical node is 22.4 cm/s, and the X-direction PPV of the field measurement is 21.13 cm/s. Consequently, the maximum vibration velocity and waveform shape are basically the same. However, the measured data wave curve has a vibration response delay of 50–100 ms, and the typical node wave curve of the numerical simulation has a vibration response delay of only 10–15 ms. This finding shows that the face blasting numerical simulation conforms to the onsite reality in terms of the PPV. However, the vibration response time fails to reflect the actual surrounding rock and lining conditions well. Moreover, this numerical model fails to simulate joint fissures, groundwater, and other special geological conditions in the actual surrounding rocks. Furthermore, it is related to increases in ethylene-vinyl acetate foam plastics and other seismic measures.
4.2. Analysis of PPV in different distances

To analyze the law of peak particle vibration velocity at different distances, the secondary lining right arch waist of the early excavated tunnel is selected for the numerical model analysis. The relative positions of each node are shown in Fig. 6 (a). The PPV distribution along the secondary lining right arch waist of the early excavated tunnel, when the explosive equivalent is 14.3 kg under the fourth-level surrounding rock, is shown in Fig. 6 (b). In the numerical model, the PPV distribution along the central axis of the late-excavated tunnel is shown in Fig. 7. Figure 8 presents the field-measured data and the Sadovsky fitting curve of the secondary lining right arch waist along the early excavated tunnel corresponding to Fig. 6 (a).

Figure 6. Variation law of PPV along the secondary lining haunch of the early excavated tunnel in the numerical model.
Figures 6 (b) and 7 demonstrate that as distance decreases, peak particle vibration velocity increases gradually. The closer to the explosion source, the faster is the increase of the peak particle vibration velocity. In addition, Fig. 6 (b) shows that the peak particle vibration speed in the unexcavated direction (negative direction) of the late-excavated tunnel is obviously greater than that in the excavated direction (positive direction) of the late-excavated tunnel, which takes the position of the secondary lining arch waist of the early excavated tunnel corresponding to the late-excavated tunnel.
face as the zero point. This finding is observed, because the rock mass in front of the late-excavated tunnel face has yet to be excavated, and the shock wave resistance generated during blasting propagation in the surrounding rock is small. Therefore, when monitoring the vibrations of the secondary lining of the early excavated tunnel onsite, the corresponding secondary lining in front of the early excavated tunnel face should be monitored. In addition, Fig. 7 illustrates that the relationship between vibration velocity and distance in the numerical simulation results basically conforms to Sadovsky’s law. Meanwhile, the Sadovsky formula fitting curve law obtained from the measured data in the construction site is basically consistent with the law of the PPV change curve obtained from the numerical simulation results along the central axis of the late-excavated tunnel. Consequently, the reliability of the numerical simulation is verified. Thus, it can be concluded from the peak particle vibration velocity that the secondary lining of the early excavated tunnel should maintain a distance of at least 7 m in front of the late-excavated tunnel face. In such a situation, vibration velocity will decrease to approximately 30 cm/s. Thus, the construction of the secondary lining of the early excavated tunnel should be at least 7 m or more away from the front of the late-excavated tunnel face, which will have less impact on the secondary lining. Given that the construction site is located in the red-bed soft rock of Central Yunnan, geological conditions are poor, and the safety factor of the primary lining of the early excavated tunnel is low. In view of the safety situation, the secondary lining of the early excavated tunnel can be constructed 20 m away from the front of the late-excavated tunnel face.

4.3. Analysis of PPV at different positions of the section

Given that the peak particle vibration velocity is too large in the section where the blasting center of the late-excavated tunnel is located, nodes 1–7 in Fig. 9 (a) represent the peak vibration velocity of each point obtained by selecting the section where the blasting center is located when the explosive quantity is 7.35 kg and 5.88 kg under the fifth-level surrounding rocks. Nodes 8–14 are the sections where the early excavated tunnel corresponds to the late-excavated tunnel face when the explosive equivalent is 14.3 kg and 11.03 kg under the fourth-level surrounding rocks, and the peak vibration velocity distribution is shown in Fig. 9 (b).
Figure 9 shows that the peak particle vibration velocity response of nodes 1–7 on the cross section of the late-excavated tunnel exhibits a symmetrical distribution, which verifies the accuracy of the numerical simulation. In addition, the PPV of node 9 at the right arch waist nodes 8–14 of the early excavated tunnel section is the largest and much larger than that of the right arch foot, the right arch shoulder, and other parts. Figures 4 and 5 illustrate that the PPV in the X direction of the arch waist is the largest, accounting for 90%–95% of the resultant velocity. Thus, onsite vibration sensors should aim for vibrations in the X direction, which is the most sensitive direction. At the same time, vibration sensors should be set in the arch waist of the early excavated tunnel for the blasting of the upper step face of the late-excavated tunnel. The left side of the early excavated tunnel is far from the late-excavated tunnel; thus, the vibration response is very small. Thus, it can be inferred that vibration sensors need not be set. In addition, Fig. 9 (b) obviously demonstrates that as explosive charge increases, the peak particle vibration velocity of the nodes likewise increases, regardless of whether it is in the grade IV or V surrounding rocks.

4.4. Analysis of maximum principal stress of the section
Figure 10 (a) presents the location of elements 1–14 in Figs. 10 (b) and (c). The maximum principal stress of elements 1–11 in Fig. 10 (b) is obtained by selecting the section where the blasting center is located when the explosive quantity is 5.88 kg and 11.03 kg under the fifth-level surrounding rocks. Elements 12–14 in Fig. 10 (c) are in the section where the early excavated tunnel corresponds to the late-excavated tunnel face when the explosive equivalent is 14.3 kg under the fourth-level surrounding rocks. When this section of the late-excavated tunnel face is set to zero, the curve of the maximum principal stress of the different elements varies with distance, as shown in Fig. 10 (c).
(a) Schematic diagram of cross-section monitoring points in the numerical model

(b) Maximum principal stress of different elements
Figure 10 (b) exhibits that the maximum principal stress of elements 1–11 is approximately symmetrical with respect to the tunnel central axis. Nevertheless, the maximum principal stress of elements 7–11 is slightly greater than the maximum principal stress of elements 1–5. This finding is observed because the right side of the late-excavated tunnel is near the surrounding rock, whereas the left side only includes the lining of the early excavated tunnel. Consequently, the vibrations generated by blasting are not spread out, thereby increasing the maximum principal stress on the left side of the late-excavated tunnel. According to Fig. 10 (c), the closer to the explosion source, the greater the maximum principal stress. The maximum principal stress at the arch waist is far greater than that at the arch foot. Thus, reinforcement measures should be considered in the design scheme. In addition, the maximum principal stress in the unexcavated direction (negative direction) of the late-excavated tunnel is obviously greater than that in the excavated direction (positive direction). The distribution of the maximum principal stress is completely consistent with the distribution of the peak particle vibration velocity obtained from Fig. 6 (b). Furthermore, monitoring of the lining of the early excavated tunnel in front of the late-excavated tunnel face should be strengthened, as this location is likely to generate cracks.

5. Conclusions
Through the establishment of a three-dimensional tunnel model and its comparison with field-measured data, this study verifies that the numerical model is in line with reality. The maximum seismic velocity of the characteristic nodes and maximum stress of the characteristic elements under the conditions of different explosive equivalents and surrounding rock levels are calculated and analyzed, and the following basic conclusions are drawn:
1) For the grade IV surrounding rocks, considering vibration speed and maximum principal stress, the construction of the secondary lining of the first tunnel should be at least 20 m from the face of the second tunnel. Moreover, the blasting construction of the second tunnel has little impact on the secondary lining of the first tunnel and ensures the safety and stability of the first tunnel primary lining.
2) For different working conditions, considering vibration speed and maximum principal stress, the blasting construction of the rear tunnel has considerable influence on the arch waist of the first tunnel and on the side near the explosion source. The maximum vibration speed is mainly in the X direction, and monitoring should be strengthened at the arch waist of the side near the explosion source at the site.
3) The continuous response time of the numerical simulation vibration wave is shorter than that of the field-measured vibration wave, which demonstrates that the numerical simulation is not very close to the actual situation. According to the field-measured vibration response time of the first tunnel lining, when millisecond blasting construction is adopted, the interval between the two blasting periods should be $\geq 100$ ms to avoid the overlapping of the two blasting vibration periods. To reduce disturbance in the surrounding rocks and vibrations in the tunnel linings, the explosive equivalent of each blasting construction should be strictly controlled.

4) The numerical simulation results of the blasting vibrations are used to guide the blasting construction of the back tunnel of the Chenjiachong multiarch tunnel. Compared with the original blasting scheme, the amount of explosives is reduced by 30%–50%, the cost is substantially reduced, and the blasting excavation effect is satisfactory. Finally, the effect of blasting on the lining vibrations of the first tunnel is minimized.

Acknowledgement
This research was supported by the Science Foundation of the Transport Department of Yunnan Province (Grant No. [2016] 160-[4]).

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