

Article

Safety Assessment of the Cover-and-Cut Method Under Blasting Vibration Induced by Tunnel Excavation

Yunhao Che and Enan Chi *

School of Mining Engineering, Guizhou University, Guiyang 550025, China; yhche3000@gmail.com

* Correspondence: chi8888@sina.com

Abstract: In evaluating the construction safety of the building in the subway tunnel using the cover-and-cut method, the main objective is to analyze the diaphragm wall, the central pillar, and the roof. This article conducted a blasting vibration test based on the background of the Guiyang Metro Line 3 project and used the FLAC3D software to establish a three-dimensional numerical model. The results showed that the peak particle velocity (PPV) decreased with increasing distance from the blasting center. The PPV measured at the underground diaphragm wall was 1.424 cm/s, while at the bottom of the central pillar it was 1.482 cm/s. The predicted PPV on the roof was up to 1.537 cm/s, which met the safety standards. According to the cloud map of particle vibration velocity and the comprehensive analysis of particle vibration velocity, the degree of impact of artificial structures in the subway tunnel was the central pillar, the underground diaphragm wall, and the roof in order from high to low. After eight blasting operations per day, the vibration velocity trend at the vulnerable point of the central column increases, but it will not exceed the safety standard.

Keywords: blasting vibration; cover-and-cut method; subway tunnel construction; peak particle velocity (PPV); structural safety assessment



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1. Introduction

1.1. Challenges and Advances in Vibration Control for Urban Tunnel Construction

The cover-and-cut method has become popular for tunnel and metro station construction due to its efficiency and minimal disruption to surrounding areas. This study focuses on the Guiyang Metro Line 3, specifically the Guizhou University Station, which presents unique geological and construction challenges. The project site involves mixed artificial fill, clay, karst formations, and argillaceous dolomite, conditions rarely addressed in previous studies. By analyzing blasting vibration effects under these specific circumstances, this research contributes practical insights into urban metro construction safety [1–3]. However, when blasting is used in tunnel excavation, vibrations can propagate through nearby structures, potentially causing damage or even collapse. Ensuring structural safety in these conditions requires precise control of blasting vibrations.

Blasting-induced vibrations impact various structural elements, such as foundation pits, retaining walls, and bridge columns. Understanding the propagation and interaction of these vibrations with adjacent structures is crucial to prevent lasting damage. Research shows that both the intensity and frequency of seismic waves significantly affect structural integrity, with vibrations within a structure's natural resonance range posing a heightened risk. In urban subway projects, where adjacent buildings are common, vibration control is essential to ensure public safety [4–7].

Numerous studies have examined the effects of blasting vibrations [8–10]. Ozer et al., for instance, developed empirical models to predict vibration amplitude and frequency in subway tunnels [11–13]. Valliappan used finite element simulations to study seismic wave behavior in rock surfaces, demonstrating how deep tunnel blasts can create high-amplitude waves on the surface. Other studies focus on how underground blasting impacts infrastructure like roads, pipelines, and existing subway tunnels, emphasizing the role of wave intensity and frequency in determining vibration-induced damage [14–17].

Eslami's research employed 3D Distinct Element Code simulations to predict surface structural damage from tunnel blasting [18], linking damage severity to both wave intensity and frequency [19–22]. Yet, most studies emphasize vertical loads, pressure effects on diaphragm walls, and settlement of surrounding structures, with less attention to the specific impacts of blasting vibrations on diaphragm walls, roof plates, and central columns within foundation pits [23–25].

To address this gap, this study investigates the Guizhou University Station on Guiyang Metro Line 3, where the cover-and-cut method is used. Field tests and simulations using FLAC3D software assess how vibrations from multiple blasting cycles affect critical structures, such as diaphragm walls, central pillars, and roof plates. The three-dimensional model provides insights into vibration velocity, frequency distribution, and structural responses to repeated blasts [26,27].

The assessment of blasting vibration impacts on underground structures, especially those caused by subway tunnel excavations, has become a critical area of research in geotechnical and structural engineering. Xia developed a 3D numerical model for assessing the safety of a buried DN1200 water pipeline subjected to blasting vibrations from subway tunnel excavation. The model, validated by field blasting monitoring data, indicated that the pipeline remained safe in both empty and water-filled states under blasting-induced vibrations, using a parallel cut blasting method. In a related study, Yan explored the blasting vibration response in an existing railway cross tunnel, examining the evolution of peak particle velocity and frequency. The study introduced a dimensional analysis method for evaluating the influence of frequency on vibration response, incorporating wavelet packet analysis and structural dynamics. Yan's research highlighted the dynamic amplification factor as a key parameter for quantifying frequency effects and employed numerical simulation to overcome limitations in field monitoring data. Chen focused on the frequency characteristics of blasting vibrations, proposing a wavelet-based method to analyze non-stationary and non-periodic signals, which is more suitable than traditional Fourier transform methods. The study showed that the main frequency band, identified through wavelet packet transform, could accurately describe blasting vibration signals. The analysis also revealed how explosive charge and distance affect frequency distribution, providing insights into the relationship between these variables and vibration energy [28–30].

Based on the current research, it is found that existing studies have limitations, such as focusing primarily on instantaneous impacts without considering the effects of long-term and repeated construction activities. Additionally, there is limited analysis and protection measures for the most vulnerable areas within the tunnel space that are most susceptible to vibration impacts. Therefore, this paper will innovatively propose a "protection theory for the most vulnerable points of structures during subway construction". Using a 3D model, this research will identify the most vulnerable points, where the vibration velocity is the highest, and conduct observational analysis and protection. By combining these two aspects, this study will also consider the cumulative effects of repeated vibrations on these vulnerable points and propose protective measures to mitigate the impact.

This paper will address the following issues: 1. Identifying the most vulnerable points of tunnel structures using field measurements and numerical simulation results. 2. Considering the impact of repeated blasting vibrations on these vulnerable points. 3. Implementing methods to control and protect the most vulnerable points from blasting vibrations.

1.2. Background of the Blasting Operation

This project is located in the Guizhou University section of Guiyang Metro Line 3, which spans 40.03 km from Tongmu Ling Station in Huaxi District and passes through Jingkai, Nanming, and Yunyan Districts (see Figure 1 for the line diagram).

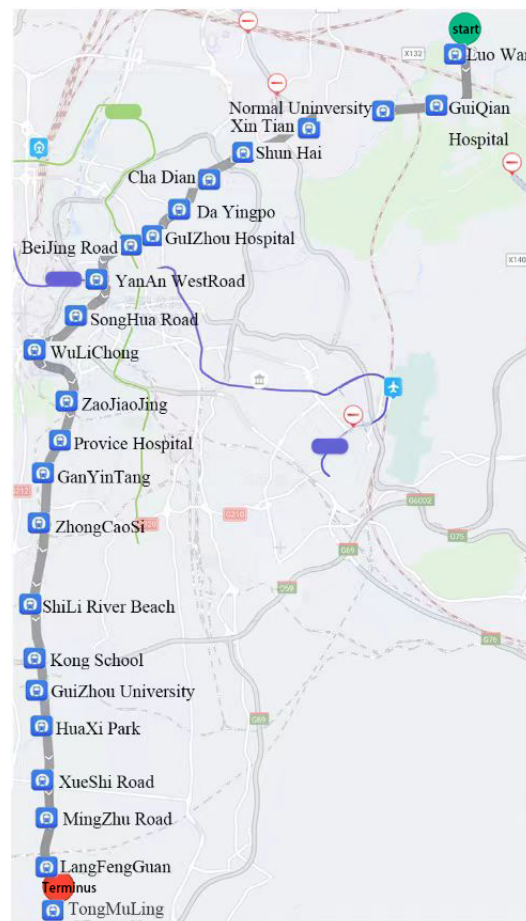


Figure 1. GuiYang subway line 3 layout.

Guizhou University Station, situated along the southern part of Huaxi Street, has a 270 m main structure with a standard segment length of 20.3 m and a total underground construction area of 15,500 m³. The station features four entrances and two ventilation pavilions. Key geological formations in this area include mixed artificial fill, clay, karst formations, and argillaceous dolomite. Figure 2 shows the subway tunnel cross-section for the cover-and-cut construction method.

Considering the local hydrological and fence quality conditions, excavation with controlled blasting was chosen. Figure 3 presents the explosion hole and detonation delay design, and Table 1 provides details on blasting parameters. Blast holes are angled, 40 mm in diameter, and designed for a one-meter cycle.

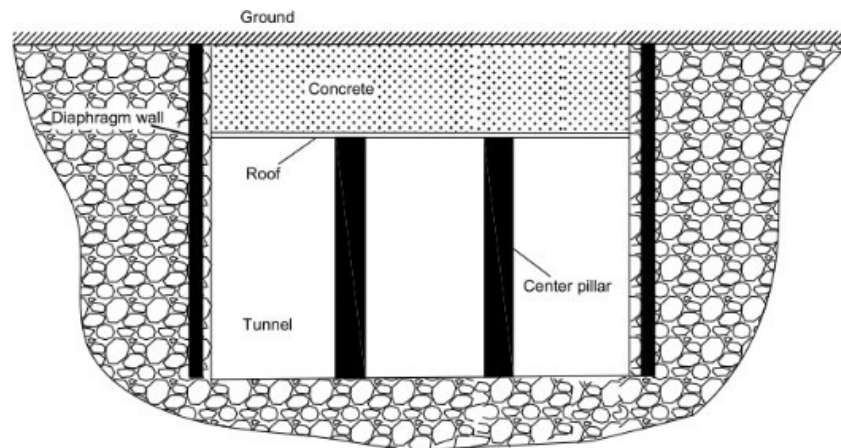


Figure 2. Section of the subway tunnel in the cover-and-cut method.

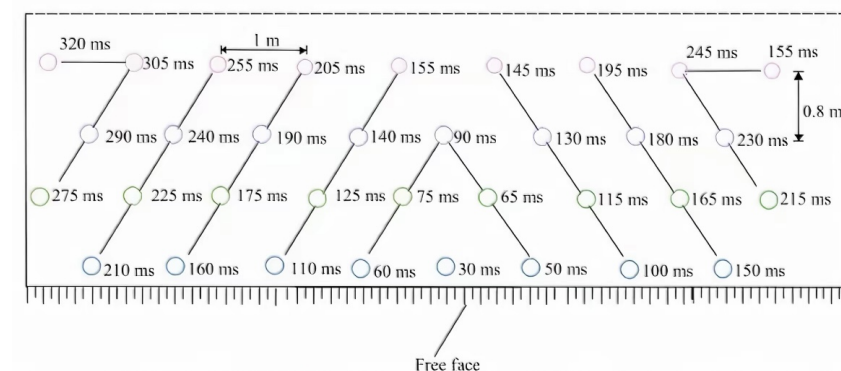


Figure 3. Blast hole and delay layout diagram.

The blasting method involves four rows of blast holes at a depth of 1.2 m, with a single-hole charge of 0.5 kg/m. The specific blasting parameters are outlined in Table 1. A precise digital electronic detonator system enables controlled delayed blasting, with explosions progressing from the center of a triangular layout toward the edges. The layout of the blast holes and delays is illustrated in Figure 3.

Table 1. Blasting scheme parameter table.

Depth	Bore	Hole	Row	Single Hole	Filling
L (m)	diameter (mm)	distance a (m)	spacing b (m)	Charge Q (kg)	length (m)
1.2	40	1.0	0.8	0.3	≥ 0.8

2. Numerical Modeling and Parameter Selection

2.1. Numerical Modeling of Dynamic Responses in Tunnel Construction

This study employed FLAC3D (Fast Lagrangian Analysis of Continua in 3 Dimensions) to simulate the dynamic soil–structure interaction during tunnel construction under blast-induced vibrations. The model incorporated key structural components, including the diaphragm wall, central pillar, and roof, as well as the surrounding rock mass. The tunnel geometry and material properties were based on field data from the Guizhou University Station site. The numerical simulation model of the subway tunnel, established by FLAC3D6.0 software, is shown in Figure 4.

This study employs a three-dimensional finite element model (FEM) to simulate the dynamic response of the underground metro station and its surrounding environment. The model is built based on specific parameters of the underground geological structure and calibrated with actual engineering cases. To ensure the accuracy of the model, the

following methods were used to determine its parameters: First, the model was calibrated using field vibration monitoring data to ensure that the model's response under various vibration source conditions matches the actual measured data. Second, numerical optimization algorithms were applied to adjust the parameters of the geological layer's elastic modulus, Poisson's ratio, and metro car vibration characteristics, thereby improving the model's precision.

Model Construction and Setup: The foundation pit structure was modeled as reinforced concrete integrated with the surrounding rock, treated as a uniform, continuous, isotropic medium. Mechanical properties of the rock mass were defined using the Mohr–Coulomb failure criterion, and the Saint-Venant principle was applied to minimize boundary effects. The model dimensions were set to 10 times the actual structure size (length, width, and depth) to reduce boundary interference.

Mesh sensitivity analysis was conducted to ensure accuracy. A coarse mesh was initially used for preliminary simulations, with subsequent refinement around high-stress areas, such as the blasting zone and critical structural elements. The final grid size of 0.5 m in these regions provided a balance between computational efficiency and precision. Additionally, a convergence analysis verified the stability of the model, with no significant variation (less than 2%) in peak vibration velocities observed upon further mesh refinement or reduction of the time step size.

Boundary Conditions and Soil-Structure Interaction: Boundary conditions were set to realistically replicate field constraints. The model's base was fixed to represent bedrock, while lateral boundaries used free-field conditions to prevent wave reflections. Contact elements were employed to simulate the interaction between the soil and structural components, ensuring accurate representation of the dynamic response.

Dynamic Loading and Frequency Analysis: Blast-induced loads were applied to simulate multiple blasting cycles. Parameters such as charge weight and the distance from the blast source were calibrated to reflect real-world conditions. The simulation revealed that vibration intensity decreased with distance from the blast, with higher frequencies dominating near the source and lower frequencies propagating further. This frequency-dependent behavior highlighted the significance of considering the natural resonance frequencies of nearby structures, as they may amplify vibrations and lead to potential damage.

Validation and Results: The simulation results were validated against field data from vibration monitoring stations. Predicted peak vibration velocities and frequency distributions closely matched the measured values, with a maximum deviation of 16%. This agreement confirmed the reliability of the model in assessing the dynamic response of tunnel structures under blasting conditions.

In summary, the numerical model provided precise insights into the propagation of vibrations and the structural responses of key components, offering a robust tool for optimizing construction strategies and mitigating risks in urban tunnel projects.

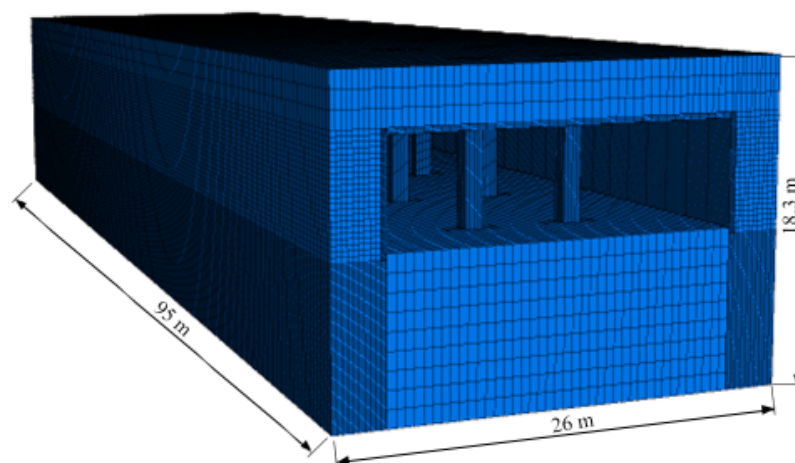
Model parameters include explosives, dolomite, and reinforced concrete. No. 2 emulsified explosive, commonly used in Guiyang subway projects, is specified, with the material and explosive parameters detailed in Tables 2 and 3.

Table 2. Explosive parameters.

Density of Explosive	Detonation Velocity of Explosive	Detonation Velocity of Explosive	Hole Diameter	Hole Spacing
$\rho/(\text{kg}/\text{m}^3)$	$D/(\text{m}/\text{s})$	$d_c/(\text{mm})$	$d_b/(\text{mm})$	$a/(\text{m})$
1100	3500	40	42	0.8

Table 3. Material parameters.

Materials	Density (g/cm ³)	Elastic Modulus/MPa	Poisson's Ratio	Tensile Stress /MPa
Dolomite	2.8	60	0.25	80,250
Clay	1.93	26	0.33	-
RC	3.0	25.5	0.13	1.5

**Figure 4.** Finite element model of the tunnel structure.

2.2. Determination of the Blast Load

The blast-induced load impacts both the tunnel wall and its supporting structures. To model the stress propagation caused by blasting, the dynamic load is simulated using FLAC3D software. For simplicity, the time-history curve of the blast load is represented as a triangular wave, serving as an approximation of the equivalent blast load. The corresponding calculation formula is provided below:

$$P_D = \frac{D^2 \rho_0}{2(\gamma + 1)} \quad (1)$$

The parameter P_D denotes the average detonation pressure of the explosive, while D corresponds to its detonation velocity. The symbol ρ represents the density of the explosive, and γ refers to its isentropic coefficient, given as 3.0. When the blast hole is partially filled with a decoupled charge, the resulting blasting gas pressure, P , can be expressed as:

$$P = A \rho^{v_0} \quad (2)$$

In this context, P represents the gas pressure of the explosive under specific conditions, A is a constant, ρ denotes the gas density in the same state, and v_0 corresponds to the isentropic index of the explosive gas. When the uncoupling coefficient of the charge is low, the conditions $P \geq P_k$ and $v_0 = \gamma = 3$. Also, when $P < P_k$, and $v_0 = v = 4$, $v_0 = v = 1.4$. Here, P_k indicates the critical blasting pressure, and the initial average pressure generated by the blast holes is given by:

$$P_0 = \frac{1}{8} \rho_0 D^2 k_d^{-6} \eta \quad (3)$$

When the charge uncoupling coefficient is high, the explosive gas expansion proceeds only through the stage where $P \geq P_k$, leading to the derivation of Formula (4).

$$P_0 = P_k^{\frac{\gamma-v}{\gamma}} \left(\frac{Q_e D^2}{2(1+\lambda)} \right)^{\frac{v}{\lambda}} k_d^{-6} \eta \quad (4)$$

The function $p(t)$ is typically regarded as an exponential time-delay function, characterized by a generally decreasing exponential trend. Calculations involving this function can be addressed using the triangle approximation. The relationship between load and time is expressed as follows: [31]:

$$P(t) = P_0 \frac{t}{t_r} (0 \leq t \leq t_r) \quad (5)$$

$$P(t) = P_0 \frac{t_r + t_b - t}{t_r} (t_r \leq t \leq t_r + t_b) \quad (6)$$

Based on the treatment approach for explosive parameters and the relevant literature, the rise time of the explosion load is established at 100 μ s, while the duration of the blasting seismic wave is designated as 0.6 s. Figure 5 illustrates the triangular shock wave load, representing the peak value of the blasting load. Here, t_r denotes the rise time, and it indicates the duration of the positive pressure action [32].

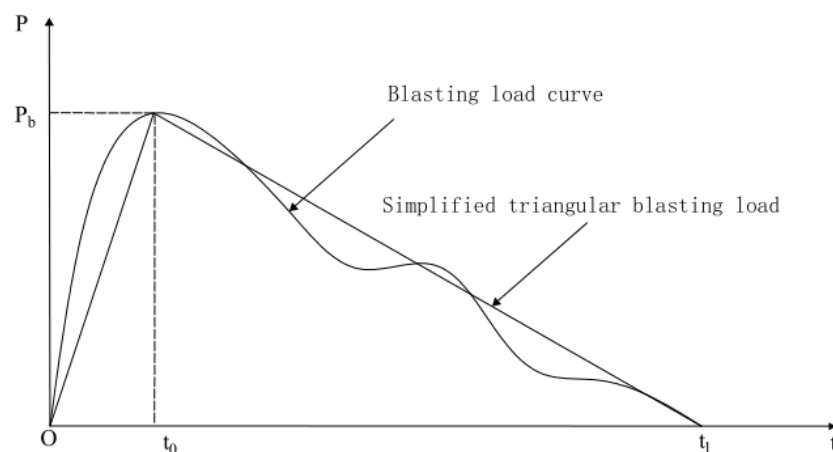


Figure 5. The diagram of a triangular shock wave load curve.

2.3. Numerical Calculation and Analysis

The primary focus of this paper is to study the dynamic response of three structural elements in the tunnel: the central pillar, the diaphragm wall, and the roof. To achieve this, vibration data were collected during blasting. For comparative analysis of tunnel blast vibrations, historical monitoring points were set on the central pillar and diaphragm wall in the model to record peak vibration velocity (PPV). Model points 1 to 5 are positioned on the diaphragm wall, while points 6 to 10 are adjacent to the central pillar, each spaced 10 m apart. The placement of these monitoring points in the numerical model is shown in Figure 6.

In vibration monitoring, vertical displacement velocity is the easiest to measure, and PPV is used here to ensure consistency across directions. FLAC3D software simulated and recorded PPV values for the vibration measurement points, with the distribution on the diaphragm wall shown in Figure 7. At a distance of 30 m from the blast source, the highest recorded PPV on the diaphragm wall is 1.656 cm/s at measuring point 1.

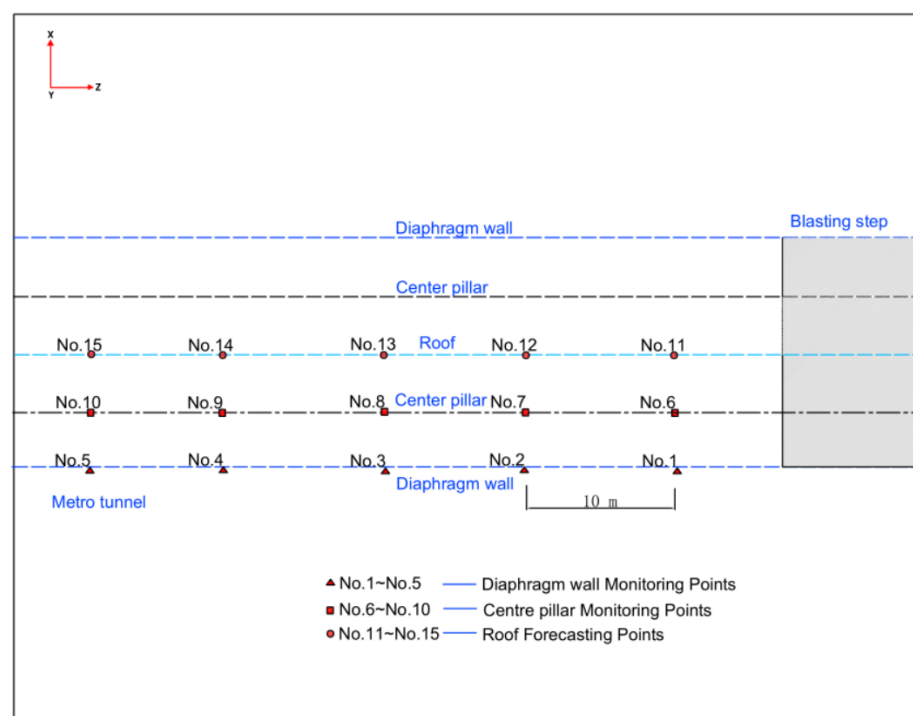


Figure 6. Layout of the monitoring points.

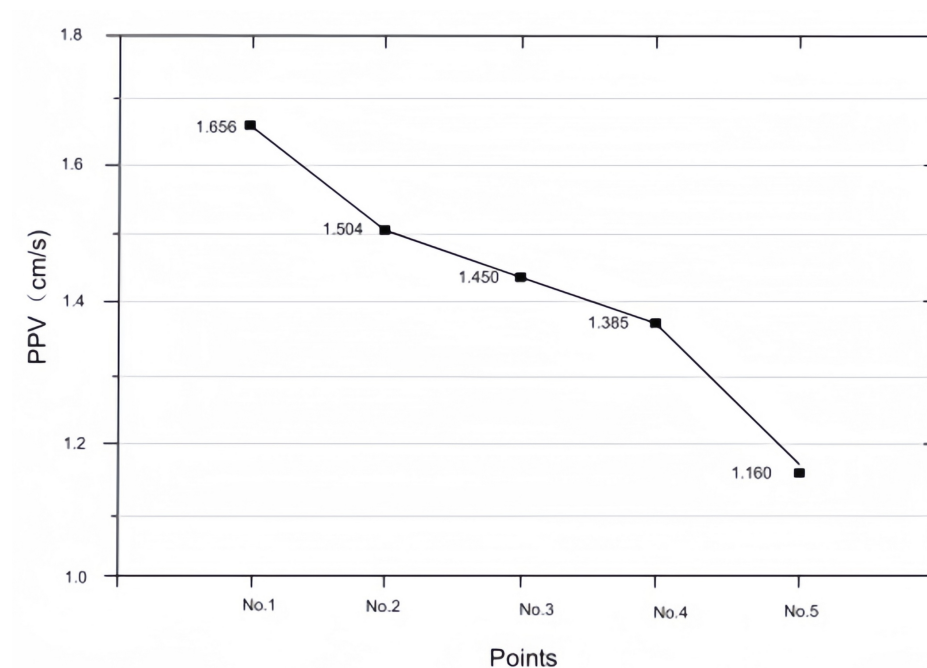


Figure 7. Simulation of the vibration speed of the diaphragm wall measurement point.

As the distance from the blast source to the diaphragm wall increases, the PPV gradually decreases, reaching a minimum of 1.160 cm/s at point 5. Figure 8 illustrates the simulated peak vibration velocities at the base of the central pillar, where point 6 recorded a PPV of 1.763 cm/s, and point 10, 1.181 cm/s. The vibration velocity on the central pillar is generally higher than on the diaphragm wall, indicating a stronger impact on the pillar compared to the wall.

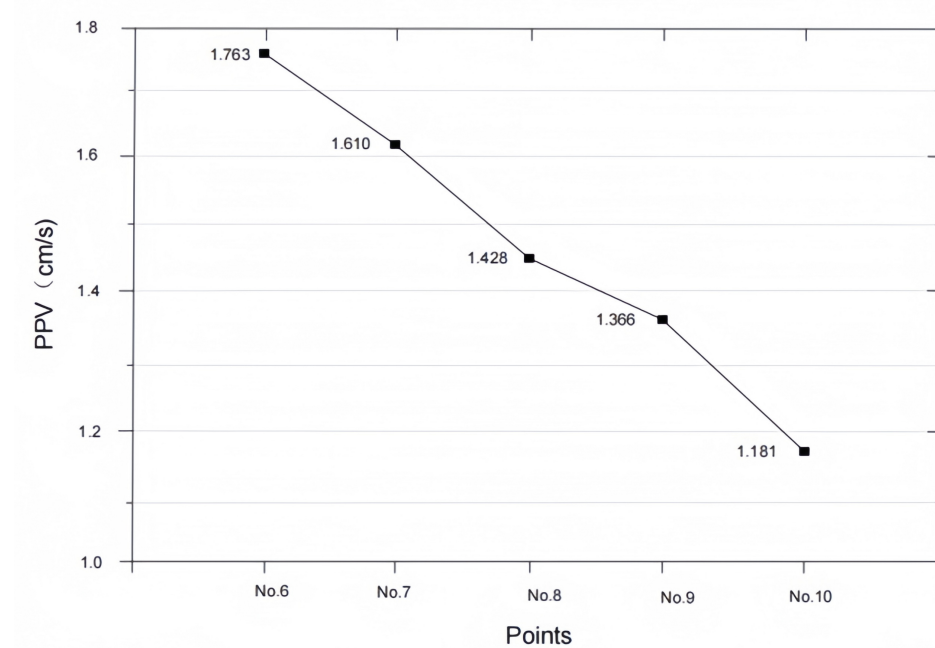


Figure 8. Vibration speed peak of the bottom measuring point at the bottom of the neutral column.

3. Field Blasting Vibration Monitoring and Analysis

To analyze the in situ blasting vibration results, monitoring points were established on the central pillar and diaphragm wall to measure peak vibration velocity (PPV). A vibrometer was placed on both the central pillar and diaphragm wall, positioned 30 m from the blast site. Measurement points 1 to 5 were suspended along the diaphragm wall, while points 6 to 10 were set near the central pillar, each spaced 10 m apart. These monitoring locations align with those in the simulation model, as shown in Figure 6. Due to challenges in directly measuring PPV on the roof, a model was used to simulate it. Figure 9 illustrates the field setup for monitoring on the diaphragm wall.

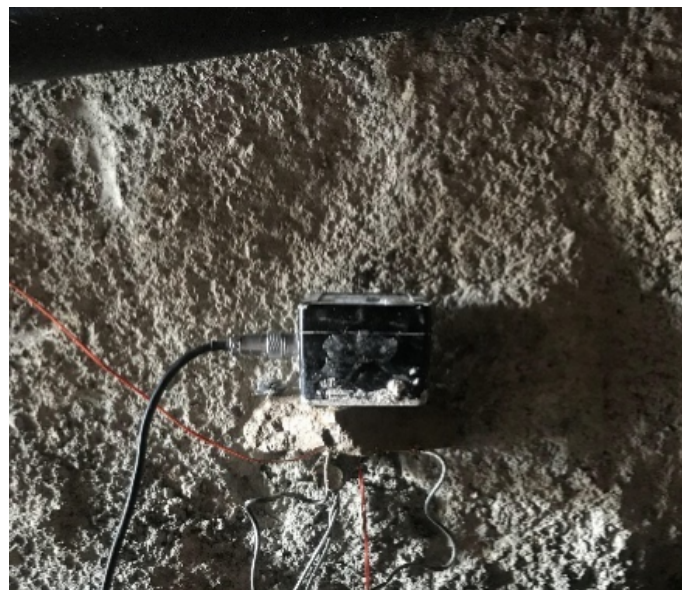


Figure 9. Diaphragm wall measurement diagram.

The NUBOX-8016 vibrometer from Sichuan Topu Measurement and Control Co., Ltd., equipped with three channels, recorded data in the horizontal (x, y) and vertical (z) directions, capturing particle velocity, frequency, and duration in each direction. In this

study, point 6, which experiences higher vibration levels, was selected for detailed analysis. A comparison of field and simulated PPV values (Figure 10) reveals that the simulation closely matches the trend of field vibration velocity, reflecting the impact on the structure.

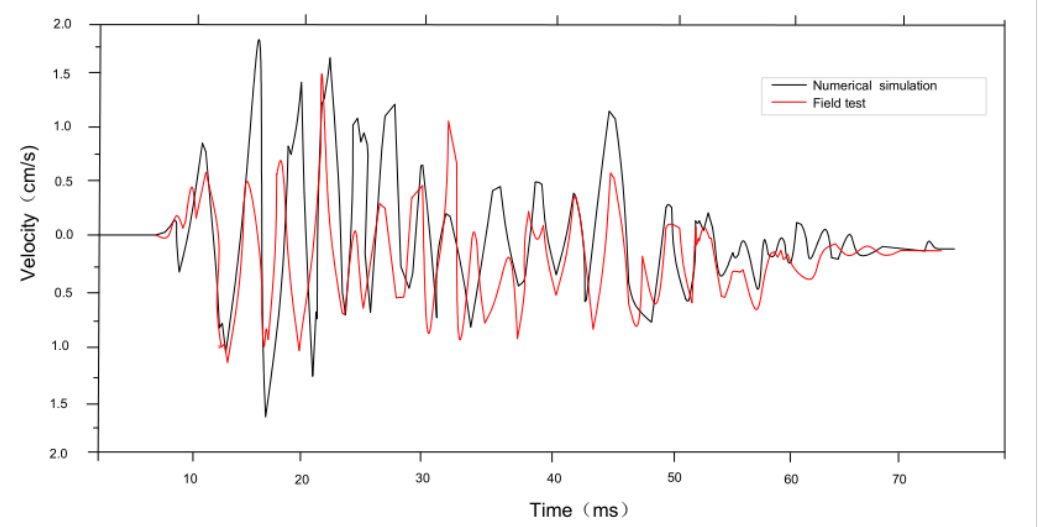


Figure 10. Comparison of PPV curves between numerically simulation and field test on the No. 6 point.

The installation of sensors plays a critical role in ensuring the accuracy of vibration measurements and the overall reliability of the data. Proper installation is essential to correctly assess the impact of blast-induced vibrations on the tunnel structure. In Figure 9, while the sensor setup is illustrated, the details regarding the sensor installation procedure are not fully described. To address this, the following description outlines the key steps involved in installing the geophones and related equipment.

First, a base plate was used as the foundation for mounting the geophone sensors. This base plate provides a stable and rigid surface for the sensor, ensuring that measurements are not affected by any movement or instability during the installation. The geophone was carefully aligned on the base plate, ensuring that its orientation was correct to measure vibrations in the desired direction.

To ensure precise positioning and to account for any surface irregularities, leveling screws were incorporated into the installation. These leveling screws were adjustable and used to fine-tune the alignment of the geophone, ensuring that it was perfectly level and aligned with the direction of vibration measurement. The screws allowed for minor adjustments, which is crucial in obtaining accurate readings, especially when measuring low-frequency vibrations.

Once the geophone was positioned and leveled, it was securely fixed to the base plate using a set of mounting screws. These screws were tightened to ensure that the geophone remained in place throughout the entire measurement process. This installation procedure ensures that the geophone is firmly attached to the base plate and maintains its stability during the duration of the measurements, preventing any potential movement that could distort the data.

This detailed installation procedure was carried out at all monitoring points, ensuring that each sensor was positioned optimally for accurate measurement of vibration velocities. The precise alignment and secure mounting of the geophones, as described, are essential for obtaining reliable data that can be used to assess the safety of the tunnel structure under blast-induced vibrations.

Table 4 lists the PPV values from both field measurements and simulations for all monitoring points. The analysis reveals that simulated PPV values are generally higher than field measurements, with a maximum deviation of 16%. This discrepancy may be due to unmodeled factors, such as rock joints and weak surfaces, which could attenuate vibration waves.

Table 4. On-site measurement.

Monitoring Points	PPV of Numerical Simulation (cm/s)	PPV of Field Monitoring (cm/s)	Error Rate (%)
No. 1	1.656	1.424	14
No. 2	1.504	1.375	8.5
No. 3	1.450	1.213	11.8
No. 4	1.385	1.236	10.7
No. 5	1.160	1.087	6
No. 6	1.763	1.482	16
No. 7	1.610	1.362	15.4
No. 8	1.428	1.271	14
No. 9	1.336	1.158	13
No. 10	1.181	1.074	9

To enhance the theoretical understanding of blasting vibrations, this study includes a detailed analysis of vibration frequencies and waveform propagation. Specifically, we examined the potential resonance risks between the blasting-induced vibration frequencies (40–80 Hz) and the natural frequencies of structural components (50–200 Hz). If there is overlap between the blasting vibration frequencies and the natural resonance frequencies of the tunnel structure, resonance may occur. Resonance could be a concern, particularly if the lower end of the blasting vibration range (40–50 Hz) coincides with the natural frequencies of the structure.

The natural frequency range of concrete tunnel structures typically spans from 50 Hz to 200 Hz, depending on various factors such as the structure's design and materials. Therefore, while the majority of the recorded blasting vibrations (40–80 Hz) fall within a safe range, any potential overlap with the structure's resonance frequencies should be carefully considered.

Additionally, vibrations above 100 Hz, which are often seismoacoustic in nature, require proper filtering to isolate the relevant frequencies that could affect structural integrity. In contrast, vehicle-induced vibrations typically range from 0.5 to 25 Hz, which are lower than the frequencies generated by blasting, making resonance with these vibrations unlikely.

The placement of multiple monitoring points along the diaphragm wall and central pillar provides a unique dataset to analyze vibration attenuation with increasing distance from the blast source. This study reveals a clear diminishing trend in PPV values, confirming the inverse relationship between vibration impact and distance. Additionally, a comparative analysis of structural responses demonstrates significant differences in how the diaphragm wall, central pillar, and roof respond to identical vibration sources. This finding fills a gap in the existing literature, offering new insights into the spatial dynamics of blast-induced vibrations.

A statistical analysis of the vibration frequencies recorded by the vibrometer (Figure 11) indicates that around 95% of the data fall within the 40–80 Hz range. While these frequencies generally fall within a safe range for typical tunnel structures, resonance risks should be evaluated based on each structural component's individual resonance frequencies.

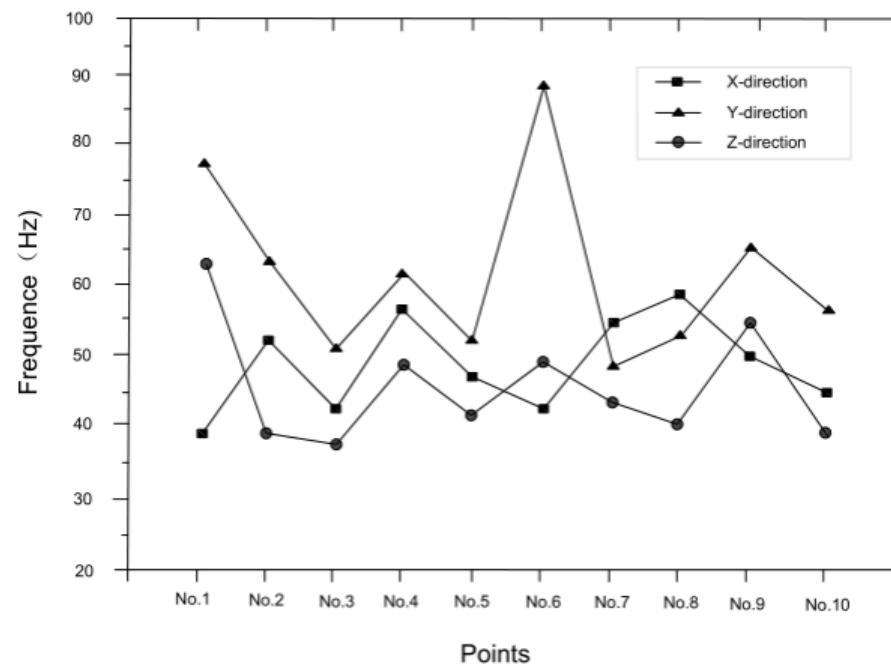


Figure 11. Main vibration frequency of the monitoring points.

4. Vibration Analysis of Subway Tunnel Construction

4.1. Vibration Speed Analysis of the Subway Tunnel Model from Blast Vibration

Figure 12 presents a stress cloud diagram that highlights vibration velocity across three key areas: the roof, center pillar, and diaphragm wall, post-blasting. The middle section of the pillar appears yellow–green, indicating higher particle velocity, with color fading on either side, showing reduced speeds. The roof is light green, while the diaphragm wall appears in shades of yellow–green, suggesting a vibration speed higher than the roof but lower than the pillar. From these results, the FLAC3D model reveals that blasting has the greatest impact on the center pillar, followed by the diaphragm wall, and least on the roof.

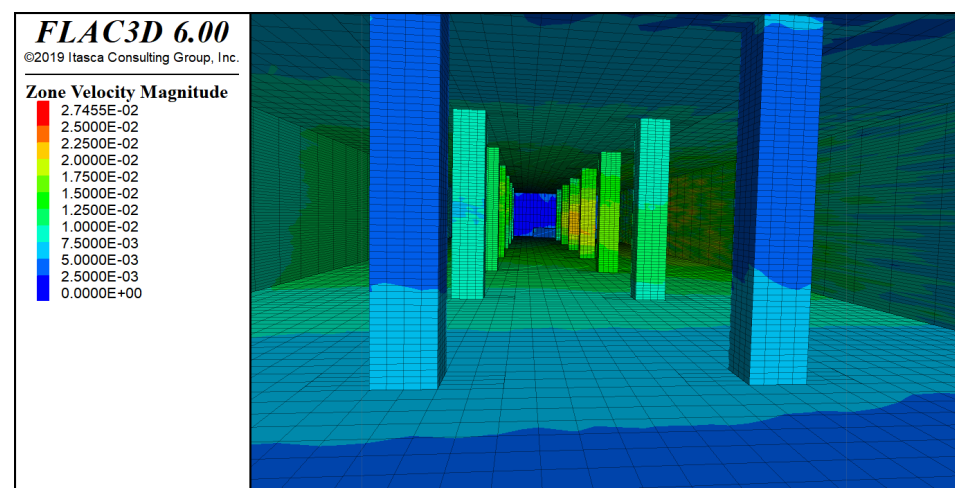


Figure 12. Metro tunnel vibration speed cloud map.

4.2. Vibration Speed Analysis of the Top Plate of the Subway Tunnel

During subway tunnel excavation, vibration velocities at the diaphragm wall and center pillar are measurable, but direct monitoring on the roof is not feasible. Therefore, FLAC3D was used to simulate and predict roof particle velocities. Prediction points are shown in Figure 13, with the maximum roof vibration speed predicted at 1.537 cm/s.

Comparing roof measurements to center pillar data, it is evident that roof velocities are generally higher, while both roof and pillar velocities exceed those of the diaphragm wall.

These findings align with the stress cloud results: tunnel blasting produces a maximum predicted vibration velocity of 1.537 cm/s at the roof, followed by 1.482 cm/s at the base of the center pillar, and 1.424 cm/s at the diaphragm wall. According to China's Blasting Safety Regulations, allowable vibration for newly poured concrete in foundation pits is 2.0–2.5 cm/s, confirming structural safety within these limits.

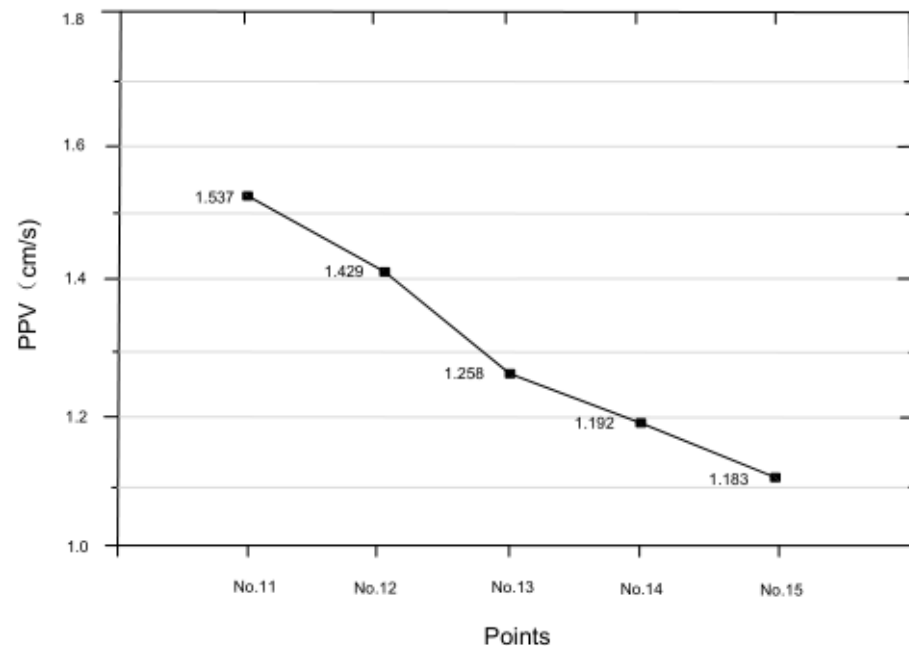


Figure 13. Peak values of particle vibration velocity at different measurement.

4.3. Summary and Analysis of Results

4.3.1. Overview of Field Vibration Monitoring Results

This study conducted a detailed analysis of field blasting vibrations by setting up monitoring points on the central pillar and diaphragm wall. The NUBOX-8016 vibrometer was used to measure particle velocity, frequency, and duration in both horizontal (x, y) and vertical (z) directions. Monitoring point 6, which experienced higher vibration levels, was selected for detailed analysis. A comparison of field and simulated peak particle velocity (PPV) values indicated that the simulated values closely followed the trend of the field data, reflecting the impact of blasting on the structure (Figure 10).

In Figure 10, the time-domain signal suggests a lower frequency event, which appears to contradict the previously stated 40–80 Hz frequency range. This discrepancy may arise due to the scaling of the time-domain signal, which corresponds to a short-duration event of approximately 60 ms. Such a short event duration can lead to an apparent broadening of the frequency content in the time-domain representation, as it may introduce lower frequency components that are not representative of the true dominant frequency range. Specifically, the limited duration of the event in the time-domain could widen the signal's spectral components, making it appear as though lower frequencies dominate the event. To resolve this issue, the frequency-domain analysis provides a clearer picture of the true frequency content of the vibrations.

To further clarify the frequency content, we conducted an analysis of vibration energy distribution across different frequency bands. The frequency-domain analysis, using a Fourier transform, decomposes the time-domain signal into its frequency components, allowing us to assess how the energy is distributed across different frequencies. This

analysis was performed over key frequency ranges, including 40–80 Hz, to evaluate whether the peak energy aligns with the natural resonance frequencies of the structure.

The results of the energy distribution analysis (shown in Figure 11) indicate that the peak energy is predominantly concentrated in the 40–80 Hz range, which corresponds to the expected dominant frequencies observed in the vibration signal. However, it is important to note that while there is some energy content in this frequency range, the peak energy does not significantly overlap with the structural resonance range of the tunnel, which typically lies between 50 and 200 Hz. As a result, the energy levels in the 40–80 Hz range are not sufficient to induce resonance in the tunnel structure, confirming that resonance is unlikely to occur.

By incorporating the vibration energy distribution analysis across these frequency bands, we resolve the previous discrepancy noted in Remark 4 regarding the apparent lower-frequency event in the time-domain signal. The frequency-domain analysis provides a more accurate and detailed characterization of the vibration and demonstrates that the observed vibrations fall within the safe zone with respect to the structural resonance frequencies. Therefore, this analysis ensures that the measured blast-induced vibrations do not pose a resonance risk to the tunnel structure.

4.3.2. Comparison of Field and Simulated Data

Table 4 lists the PPV values for all monitoring points from both field measurements and simulations. The analysis reveals that the simulated PPV values are generally higher than the field measurements, with a maximum deviation of 16%. This discrepancy could be attributed to factors not accounted for in the model, such as rock joints and weak surfaces, which may cause attenuation of vibration waves. This result indicates that while the simulation provides idealized predictions, further consideration of environmental factors affecting vibration propagation is necessary in practical applications.

Additionally, the simulation results provide a theoretical framework for engineering applications, especially during the design phase, helping to predict vibration impacts at various distances and structural responses. The data from the simulations offer valuable support for optimizing design and construction plans for tunnel vibration management.

4.3.3. Vibration Frequency and Resonance Risk Analysis

The study further analyzed the relationship between blasting-induced vibration frequencies (40–80 Hz) and the natural frequencies of structural components (50–200 Hz). The results show that the majority of the blasting vibrations fall within the safe frequency range for structures and do not pose a resonance risk with the natural frequencies of the concrete tunnel structure. This finding has significant engineering implications, suggesting that under normal construction and blasting conditions, resonance due to frequency matching is unlikely.

In comparison, vehicle-induced low-frequency vibrations (0.5 to 25 Hz) are much lower than the frequency of the blasting vibrations, reducing the likelihood of resonance. The statistical analysis in Figure 11 shows that approximately 95% of the vibration frequency data fall within the 40–80 Hz range, aligning with the safety threshold for tunnel structures and further confirming that the blasting vibrations remain within a safe range, avoiding potential resonance issues.

4.3.4. Vibration Attenuation Trend and Spatial Dynamics

This study gathered extensive vibration attenuation data by setting multiple monitoring points along the diaphragm wall and central pillar. The results show a clear attenuation trend in PPV values as the distance from the blast source increases, which confirms the inverse relationship between vibration impact and distance. Comparative analysis of

structural responses reveals that the central pillar experiences the highest vibration impact, followed by the diaphragm wall, and the roof experiences the least. This finding fills a gap in the existing literature by offering new insights into the spatial dynamics of blast-induced vibrations.

Moreover, using FLAC3D simulations, the maximum predicted vibration velocity at the tunnel roof was 1.537 cm/s, which is higher than the 1.482 cm/s observed at the central pillar and the 1.424 cm/s at the diaphragm wall. These results, consistent with the stress cloud simulation in the figures, further verify the spatial distribution of tunnel vibrations.

4.3.5. Stress Cloud and Vibration Speed Analysis

Figure 12 displays the stress cloud diagram showing vibration velocity in the tunnel model after blasting, highlighting the distribution of vibration velocity at the roof, central pillar, and diaphragm wall. The diagram shows that the central pillar area experiences the highest vibration speed, represented in yellow–green, while the diaphragm wall experiences lower vibration velocity compared to the central pillar but higher than the roof. This result aligns with the analysis from the FLAC3D model, indicating that blasting has the most significant impact on the central pillar, followed by the diaphragm wall, and the roof experiences the least impact.

By combining these stress cloud diagrams with the actual measurement data, the study not only validates the spatial distribution trend of vibration velocity but also provides a more detailed analysis of structural responses, offering new data to support the safety evaluation of tunnel vibration.

4.3.6. Comparison with China's Blasting Safety Regulations

During the excavation of the subway tunnel, vibration velocities at the diaphragm wall and central pillar were directly measurable; however, monitoring at the tunnel roof was not feasible. To address this limitation, a three-dimensional numerical simulation was conducted using FLAC3D to predict the particle velocities at the roof. The predicted vibration velocities at various points are shown in Figure 13, with the maximum vibration velocity at the roof predicted to be 1.537 cm/s. Comparative analysis of the measured roof velocities and center pillar data revealed that the roof experienced higher vibration velocities, with both the roof and central pillar velocities surpassing those of the diaphragm wall.

These findings are consistent with the stress cloud analysis results, which indicate that the tunnel blasting generates a maximum predicted vibration velocity of 1.537 cm/s at the roof, followed by 1.482 cm/s at the central pillar and 1.424 cm/s at the diaphragm wall.

Assessment of Safety Criteria: In order to assess the safety of the measured vibration velocities, it is essential to evaluate these values against established safety standards. The Chinese Blasting Safety Regulations provide guidelines for controlling vibration during construction activities, including permissible peak particle velocity (PPV) limits for blasting-induced vibrations. According to these regulations, the allowable PPV for newly poured concrete in foundation pits is between 2.0–2.5 cm/s, depending on the type of structure and its proximity to the blast. This threshold aligns with international standards such as those set by the U.S. Bureau of Mines (USBM), German Industrial Standards (DIN), and British Standards (BS).

The Chinese regulations also recognize the relationship between vibration frequency and structural safety. Specifically, if the vibration frequency falls within the 40–80 Hz range, the allowable PPV is typically set at 2.0 cm/s, which is consistent with international standards. However, for higher frequencies (e.g., above 100 Hz), the regulations adopt a more conservative approach, requiring further attenuation of vibrations to prevent potential structural damage.

The results of this study, which show maximum predicted PPV values of 1.537 cm/s at the roof, 1.482 cm/s at the central pillar, and 1.424 cm/s at the diaphragm wall, all fall well within the safety limits set by China's Blasting Safety Regulations. These values are also well below the limits recommended by international standards such as the USBM, DIN, and BS for newly poured concrete structures.

Additionally, the vibration frequencies observed in this study (ranging from 40 to 80 Hz) are within the safe range according to both Chinese and international guidelines. This ensures that the measured vibrations do not exceed the permissible PPV thresholds for the observed frequency range.

Thus, based on the comparison with both Chinese and international safety regulations, the blasting-induced vibrations measured in this study fall within the safe zone, confirming that they are unlikely to cause any structural damage. The results demonstrate that the structural safety of the subway tunnel, as measured in this study, remains within acceptable limits according to both national and international standards.

4.4. Trend and Regression Analysis of Repeated Blasting Vibrations at Vulnerable Points

Based on the cloud map and vibration measurement results, it can be observed that the location most affected by the blasting vibrations is near measuring point 11 on the central column. This is identified as the vulnerable point of the structure, with the highest vibration velocity recorded at 1.537 cm/s. Due to the central part being subjected to both roof pressure and vibration impacts, the pressure and vibration effects are significant. Repeated vibrations may cause the supporting structure of this vulnerable point to exceed its safety limits. To ensure safety, further research will be conducted to assess how many blasting vibrations might exceed the structural capacity of the central column. The specific trend is shown in Figure 14.

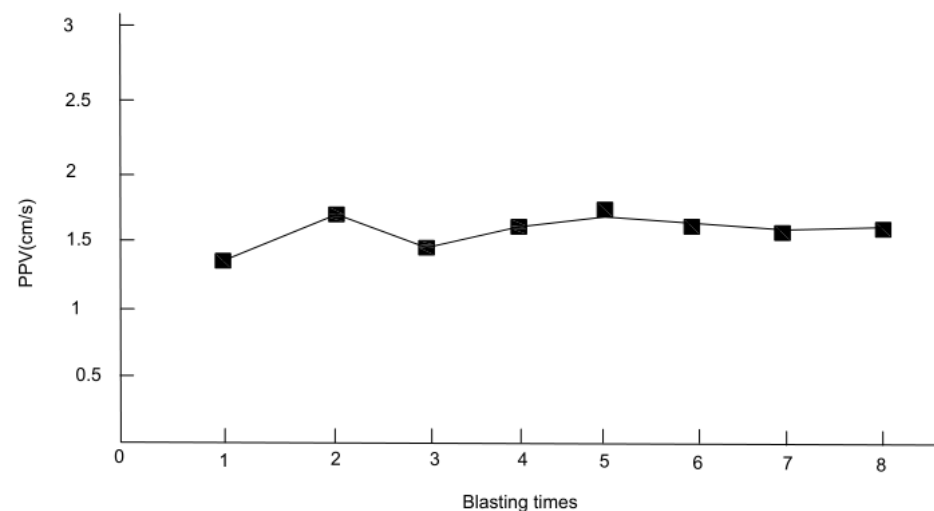


Figure 14. Regression relationship between peak particle velocity (PPV) at point 11 and blasting frequency.

After six blasting events, irregular changes in the peak vibration velocity at the monitoring points were observed. However, from the fourth blast onward, a sustained gradual increase was observed, followed by a smooth trend, all remaining below 1.537 cm/s. Therefore, the peak vibration velocities caused by cyclic blasting remained below the tunnel construction safety standard of 2.5 cm/s.

The regression equation for the blasting vibration speed at the same monitoring point is as follows:

$$F(x) = 0.0025 \times x^5 - 0.05 \times x^4 + 0.36 \times x^3 - 1.24 \times x^2 + 1.9 \times x + 0.43 \quad (7)$$

Regression analysis of the blasting vibration changes at the same monitoring point shows that a relationship between tunnel structure and blasting frequency can be established, which clarifies the overall trend of vibration velocities at the monitoring points. The polynomial regression equation between unit blasting vibration velocity and blasting frequency is given by Equation (7), with a confidence level exceeding 0.95. The results indicate that the relationship between tunnel structure damage and blasting frequency follows a cubic polynomial pattern. After eight blasts, although no damage occurred, the peak total blasting vibration velocity increased. Therefore, optimizing the existing blasting plan is imperative.

5. Determination of Safe Vibration Thresholds and Development of Mitigation Strategies

5.1. Determination of Safe Vibration Thresholds

To ensure structural safety during tunnel construction, this study integrates field measurements and numerical simulations based on the Chinese Blasting Safety Regulations (which stipulate peak particle velocity (PPV) thresholds of 2.0–2.5 cm/s). It establishes safe vibration thresholds for the central pillar, diaphragm wall, and roof.

5.1.1. Validation Through Field Measurements and Numerical Simulations

During the field measurement phase, vibration monitoring points were installed on the central pillar and diaphragm wall to record the PPVs at different locations during blasting. FLAC3D software was used to construct a three-dimensional tunnel structure model to simulate the effects of blasting vibrations on key structural components. A comparison of simulation results with field data is as follows: The simulation results indicate that the PPVs are 1.482 cm/s at the bottom of the central pillar, 1.424 cm/s for the diaphragm wall, and 1.537 cm/s for the roof. The maximum deviation between simulated and measured data is 16%, confirming the reliability of the model.

5.1.2. Establishment of Safe Thresholds

By comparing field and simulation data with relevant safety standards, the following conclusions were drawn.

The impact of blasting vibrations on tunnel structures follows the order central pillar > diaphragm wall > roof. All PPVs are below the stipulated safety threshold of 2.0–2.5 cm/s, ensuring structural safety.

Through this analysis, safe vibration thresholds suitable for this project have been established, providing valuable data and reference for similar future projects.

5.2. Development and Application of Mitigation Strategies

Effective vibration mitigation strategies are essential for ensuring the safety of surrounding structures and construction operations in tunnel projects. This study proposes targeted mitigation measures based on numerical simulations and field data analysis.

Adjustments to various blasting parameters revealed the following measures significantly reduce vibration impacts.

Delayed Detonation: Using digital electronic detonators for staged delayed detonation effectively disperses vibration waves and reduces peak vibration levels.

Hole Spacing Design: Optimizing blast hole spacing to 0.8 m and hole depth to 1.2 m, combined with controlled charge per hole, minimizes vibration amplitudes.

Detonation Sequence Optimization: Initiating detonation from the center outward reduces wave superposition effects.

FLAC3D software was utilized to predict vibration velocities in areas not directly measurable, such as the roof, providing a basis for optimizing blast designs.

The model predicted a maximum vibration velocity of 1.537 cm/s at the roof, higher than that of the diaphragm wall and central pillar but still within the safety range. Numerical simulation results offered scientific guidance for refining subsequent construction plans.

Based on the analysis of vibration impacts, the central pillar is identified as the most affected structural component. The following measures are recommended: Strengthen real-time monitoring of the central pillar during construction by installing high-precision vibration geophones to ensure it remains within safe limits. Implement additional reinforcement measures, such as temporary supports or vibration isolation devices, when necessary to further mitigate risks.

The optimized blasting plan was implemented on-site, and results showed that all measured vibration velocities were below 2.0 cm/s, demonstrating the effectiveness of the mitigation strategies. The proposed measures are applicable to similar urban metro projects and significantly reduce vibration risks.

5.3. Safe Vibration Thresholds and Mitigation Strategies

Establishing Safe Vibration Thresholds. Safe vibration thresholds were established by integrating simulation results, field measurements, and regulatory standards. Peak particle velocity (PPV) values for critical components—central pillar (1.763 cm/s), diaphragm wall (1.656 cm/s), and roof (1.537 cm/s)—were obtained through FLAC3D simulations and field validation. These values were compared with the Chinese Blasting Safety Regulations (2.0–2.5 cm/s). Conservative thresholds of 1.8 cm/s (central pillar), 2.0 cm/s (diaphragm wall), and 2.2 cm/s (roof) were defined, ensuring structural safety with an adequate margin. Statistical analysis and stress–displacement correlation confirmed these thresholds minimized damage risks.

Mitigation Strategies. Mitigation strategies were developed to manage vibration impacts effectively.

Blasting Optimization: Adjusting blast hole layouts and charge weights, combined with delayed detonation techniques, reduced peak PPV by up to 15%. **Structural Reinforcement:** Strengthening high-risk components with additional concrete layers or steel bracing enhanced structural resilience. **Geotechnical Stabilization:** Grouting techniques improved rock mass stability, reducing vibration transmission. **Real-Time Monitoring:** A network of geophones provided continuous PPV monitoring, enabling dynamic adjustments to blasting parameters.

These strategies, validated through controlled trials, demonstrated significant reductions in vibration impacts, ensuring compliance with safety standards and providing a robust framework for future projects.

6. Discussion

This study assessed the safety of the cover-and-cut method during metro construction through a combination of field measurements and numerical simulations. The results not only verified the effectiveness of the blasting technique but also offered insights into the dynamic response of critical structural components under vibration. This section discusses key findings, practical implications, study limitations, and recommendations for future research.

6.1. Interpretation of Key Results

The study demonstrates that all measured and predicted PPV values, including 1.424 cm/s for the diaphragm wall, 1.482 cm/s for the central pillar, and 1.537 cm/s for the roof, remain below the safety threshold of 2.0–2.5 cm/s. The agreement between simulation and field results indicates that the FLAC3D model is reliable for assessing structural responses in metro projects. These findings validate the controlled blasting strategy employed and confirm that the construction is safe for nearby structures.

A significant observation is that the central pillar experienced the highest PPV among all elements, suggesting that load-bearing components in the core structure are more susceptible to blast-induced stress. This aligns with the theory that central load paths typically absorb greater dynamic loads, making them critical targets for structural monitoring.

6.2. Comparison with Existing Literature

The findings complement previous research on blast-induced vibrations by confirming that vibration impact decreases with distance from the blast source. The vibration frequencies observed (40–80 Hz) are consistent with studies on concrete structures, which typically vibrate within a similar range. Notably, the absence of overlap with vehicle vibration frequencies (0.5–25 Hz) eliminates the risk of resonance, supporting the feasibility of combining metro operations with ongoing construction.

Unlike other studies focusing on vertical settlement or lateral displacement, this research emphasizes dynamic responses across multiple structural components. By integrating field data with numerical predictions, the study offers a more comprehensive assessment of both vibration speed and structural resilience, adding depth to the existing body of knowledge on underground construction safety.

6.3. Practical Implications for Metro Construction

The findings provide actionable insights for engineers and project managers involved in similar underground projects:

- (1) **Central Pillar Monitoring:** Given the pillar's higher vibration impact, priority should be given to monitoring and reinforcing core load-bearing components during blasting operations.
- (2) **Optimization of Blasting Parameters:** The study confirms that delayed detonation and proper spacing of blast holes can effectively control vibration levels. Adopting such techniques ensures that vibrations remain within safe limits, minimizing risks to both structures and operations.
- (3) **Model Validation and Predictive Planning:** The use of FLAC3D not only provided accurate predictions for unmeasurable areas, such as the roof, but also demonstrated the value of numerical simulations in early risk assessments. Future projects can use similar models to identify potential hazards and refine blasting designs before construction begins.

6.4. Limitations and Recommendations for Future Research

Despite the study's success in validating structural safety, several limitations must be acknowledged:

- (1) **Simplified Model Assumptions:** The numerical model did not fully account for complex factors such as rock joints, weak layers, and heterogeneous materials, which could influence vibration behavior. Future models could incorporate these variables for greater accuracy.
- (2) **Lack of Long-Term Monitoring:** While the study focused on short-term vibration effects, the long-term impacts of repeated vibrations on structural integrity remain

unexplored. Extending the monitoring period could reveal whether cumulative stress affects the durability of concrete elements.

- (3) Exploration of Alternative Construction Techniques: Although the cover-and-cut method proved effective in this case, other construction methods, such as tunnel boring machines (TBM) or NATM (New Austrian Tunneling Method), could be compared to identify optimal techniques for different geological conditions.
- (4) While this study focuses on the immediate impacts of blasting vibrations, we recognize the importance of examining the long-term cumulative effects of repeated vibrations on structural integrity. Over time, these vibrations may induce micro-cracks or fatigue in structural materials, potentially compromising durability. Future research should incorporate extended monitoring periods and explore advanced predictive models to assess long-term risks. Implementing real-time vibration monitoring systems in construction projects can further enhance safety by identifying and mitigating potential issues before they escalate.

7. Conclusions

This study focused on the Guizhou University Station project, using on-site testing and numerical simulations to systematically assess the impact of blasting vibrations on tunnel structures during cover-and-cut construction. The main conclusions are as follows:

Key Findings

- (1) The measured and predicted peak particle velocities (PPVs) of the diaphragm wall, central pillar, and roof were 1.424 cm/s, 1.482 cm/s, and 1.537 cm/s, respectively, all within the safety limit of 2.0–2.5 cm/s as defined by China's Blasting Safety Regulations.
- (2) Numerical simulation results indicated that the degree of impact on the structures ranked as follows: central pillar > diaphragm wall > roof. This order aligns with the stress cloud map analysis, confirming that the central pillar experiences the highest vibration impact.
- (3) The vibration frequencies were in the range of 40–80 Hz, well above the 0.5–25 Hz range of running vehicles, ensuring that no resonance occurs between the vibrations and vehicle operations.
- (4) The peak vibration velocity caused by blasting remains within the safe range. The regression analysis indicates that eight blasting cycles per day do not affect the safety of the vulnerable point on the central column.

This study demonstrates a high consistency between numerical simulations and field monitoring results, with a maximum error of 16%. The use of FLAC3D simulations provided reliable predictions, overcoming the difficulty of direct measurement on the roof. The methodology presented here can be applied to similar metro construction projects, offering valuable data and modeling references for risk assessments.

It is recommended to continuously monitor the vibration response of the central pillar during future construction and adjust blasting parameters based on the findings to ensure structural safety. For other metro projects with complex geological conditions, building numerical models in advance can help optimize blasting parameters and reduce risks. While this study focused primarily on the evaluation of vibration velocities, future work could explore the impact of additional factors, such as temperature variations or different material properties, on structural stability. Additionally, further research could optimize blasting sequences and delays to minimize vibration effects.

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