

Article

Field Measurement and 2.5D FE Analysis of Ground Vibrations Induced by High-Speed Train Moving on Embankment and Cutting

Junwei Bi ^{1,2,3} , Guangyun Gao ³, Zhaoyang Chen ¹, Jiyan Zhang ^{4,*}, Juan Chen ⁵ and Yuhan Li ^{6,*}

¹ Shandong Key Laboratory of Technologies and Systems for Intelligent Construction Equipment, Shandong Jiaotong University, Jinan 250357, China; bijw@alumni.tongji.edu.cn (J.B.); 25307027@stu.sdjtu.edu.cn (Z.C.)

² School of Qilu Transportation, Shandong University, Jinan 250002, China

³ Department of Geotechnical Engineering, College of Civil Engineering, Tongji University, Shanghai 200092, China; gaogy@tongji.edu.cn

⁴ School of Civil Engineering, North China University of Technology, Beijing 100144, China

⁵ College of Civil Engineering, Shandong Jianzhu University, Jinan 250101, China; 13942@sdjzu.edu.cn

⁶ School of Civil Engineering, Shandong University, Jinan 250002, China

* Correspondence: 240045@sdjtu.edu.cn (J.Z.); yuhan_li@mail.sdu.edu.cn (Y.L.)

Abstract

Field measurements of ground vibrations were conducted along the Paris–Brussels high-speed railway (HSR) to systematically analyze vibration characteristics generated by embankment and cutting sections. Utilizing the 2.5D finite element method (FEM), numerical models were developed for both earthworks to evaluate the influences of design parameters on ground vibration responses. Results demonstrate that train axle load dominates vibration amplitude in the near-track zone, while the superposition effect of adjacent wheelsets and bogies becomes predominant at larger distances. Vibration energy attenuates progressively with increasing distance from the track, with medium- and high-frequency components decaying more rapidly than low-frequency components. The dominant vibration frequency is determined by the fundamental train-loading frequency (f_1), which increases with train speed. Distinct attenuation patterns are identified between earthwork types: embankments exhibit a two-stage attenuation process, whereas cuttings undergo three stages, including a vibration rebound phenomenon at the slope crest. Furthermore, greater embankment height or cutting depth reduces ground vibrations, but beyond a critical threshold, further increases yield negligible benefits. A higher elastic modulus of the embankment material correlates with reduced vibrations, and steeper cutting slopes, while ensuring slope stability, contribute to additional mitigation.

Keywords: high-speed railway; ground vibrations; embankments; cuttings; field measurement; 2.5 FEM



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1. Introduction

The rapid proliferation of high-speed railway (HSR) has been primarily attributed to their transformative socioeconomic impacts compared to conventional transportation modes. Advances in HSR technology have enabled the development of high-speed trains (HSTs) achieving supersonic operational velocities. However, this technological advancement has introduced significant environmental repercussions, notably pronounced ground-borne vibration that threatens structural integrity of adjacent infrastructure and human comfort [1–3].

Over the past two decades, extensive research efforts have systematically investigated HST-induced ground vibrations through analytical/semi-analytical modeling, numerical simulations, and field measurements. [1]. Existing studies conducted by Connolly et al. [4] and Kouroussis et al. [5] show that ground vibrations generated by HSR are significantly more pronounced in subgrade sections than in bridge sections, and the type of subgrade configuration exerts a considerable influence on HST-induced ground vibrations.

Krylov [6] employed a theoretical model to analyze the ground vibration characteristics of HSR embankment sections, demonstrating that embankments can effectively reduce train-induced ground vibrations. Zhai et al. [7] conducted field measurements of environmental vibrations along an embankment section for the Beijing–Shanghai HSR, systematically investigating the characteristics and propagation attenuation patterns of ground vibrations under different train speeds. Connolly et al. [8] established a three-dimensional finite element model to analyze the effects of embankment material properties on ground vibration responses under HST loading. Based on the field measurements and experiential models, Federal Railroad Administration [9] reported that the railway cutting could mitigate the ground vibrations induced by HSTs' loadings. Zhang et al. [10] investigate the applicability of Bornitz model for predicting the environmental vibrations induced by HSTs moving on the subgrade of cutting. Ma et al. [11] and Zhang et al. [12] carried out in situ testing on vertical ground vibrations of the cutting section on Baoji-Lanzhou HSR, and 3D numerical models were established for analyzing the influences of different soil parameters on the vibration responses for the cutting. Among the above studies, most of the current investigations about the ground vibrations induced by HSRs across the embankment and cutting are field testing studies. The influences of the design parameters of different subgrade configurations on the HST induced surface motions remains relatively limited, with studies specifically addressing the vibration mitigation performance of cutting sections being particularly scarce.

As an efficient approach for investigating three-dimensional problems, the 2.5-dimensional finite element method (2.5D FEM) has gained increasing popularity among researchers. Based on the assumption that the geometric configuration and material properties of the subsoil and track structure remain invariant along the track direction, Yang et al. [13] pioneered the application of the 2.5D FEM to study ground vibration responses under train loads. Bian et al. [14] developed a coupled 2.5D finite element dynamic analysis model for the track-embankment-foundation system under moving loads, investigating the influences of train load characteristics and subsoil parameters on ground vibrations. Gao et al. [15] employed the 2.5D FEM to examine the effects of transverse isotropy parameters of foundation soils on ground vibration characteristics under various train speeds. Yin et al. [16] established a 2.5D finite element model to analyze the impact of anti-slide pile reinforcement on the dynamic response of the ground in loess-area high-speed railway cutting sections. Moreover, although the 2.5D FEM is an effective research approach, it still has certain limitations, including the assumptions of linear viscoelastic and homogeneous soil properties, the simplification that neglects soil anisotropy and layered heterogeneity, and so on. Kouroussis et al. [17] pointed out that whether to consider the coupling of various parts beneath the rails has a relatively minor impact on the prediction results of environmental vibrations under train loads. Therefore, 2.5D FEM is more suitable for studying ground vibrations. However, when the soil anisotropy, heterogeneity, and sophisticated vehicle–track–soil interaction mechanisms are considered, a fully 3D model or more advanced coupled dynamic model is needed.

In view of this, based on field measurements of ground vibrations along the Paris-Brussels high-speed railway, this paper aims to study the characteristics and propagation attenuation patterns of vibrations in embankment and cutting sections. The 2.5D FE models

both for the embankment and cutting are derived and established, and the reliability of which was validated by comparing with the field measurements. On this basis, considering different train speeds, the influences of embankment height, embankment stiffness, cutting depth, and cutting slope gradient on ground vibration mitigation are discussed in detail.

2. Field Experiment of Ground Vibrations

2.1. Test Overview

The embankment and cutting test sections were both selected from the Paris-Brussels HSR line, located northeast and northwest of the village of Braffe, Belgium, respectively, as shown in Figure 1. The test sections adopted a ballasted track structure consisting of UIC 60 rails ($\rho = 60.0 \text{ kg/m}$), pre-stressed concrete sleepers, a 0.3 m thick ballast layer, and a 0.2 m thick sub-ballast layer. One week prior to field testing, the track was ground to a smooth finish—mitigating track irregularity effects on HST-induced ground vibrations to negligible levels. To make the test data more comparable, both for the embankment and cutting sections, the train passages running in the same direction were selected for analyzed, and the speed of which is about 294.7 km/h. For recording the ground vibrations, the weeds on the surface were cleared, the soil surface was leveled, and the sensors were placed closely to the ground. Vibration measurements were acquired using SM-6 vibration geophones (sensitivity: $28.8 \text{ V}/(\text{m}\cdot\text{s}^{-1})$), and a data acquisition system sampling at 1000.0 Hz. The vibration geophones applied for measurement have been factory-calibrated and verified with a reference vibration source prior to installation. The raw signals were first visually inspected for anomalies, and then a band-pass filter was adopted to eliminate low-frequency drift and high-frequency noise. The velocity signals recorded by the sensors were numerically differentiated to obtain acceleration time histories. A linear detrend was applied to each record before performing spectral analysis. The peak ground vibration acceleration (PGA) was defined as the maximum absolute value within a single train passage. This standardized procedure ensures consistency and comparability of the reported vibration levels.



Figure 1. Test sections of the HSR: (a) embankment; (b) cutting.

As illustrated in Figure 2, eight monitoring points were sequentially deployed at distances ranging from 9.0 m to 35.0 m to the track for both the embankment and cutting test sections, to monitor and record the ground-borne vibrations induced by the passage of HSTs. The embankment section had a height of 5.5 m with a slope inclination of 30.0° (Figure 2a), while the cutting section had a depth of 7.2 m with a slope inclination of 25.0° (Figure 2b). Stratigraphic distribution and physico-mechanical parameters for these two sections are detailed in Tables 1 and 2, respectively.

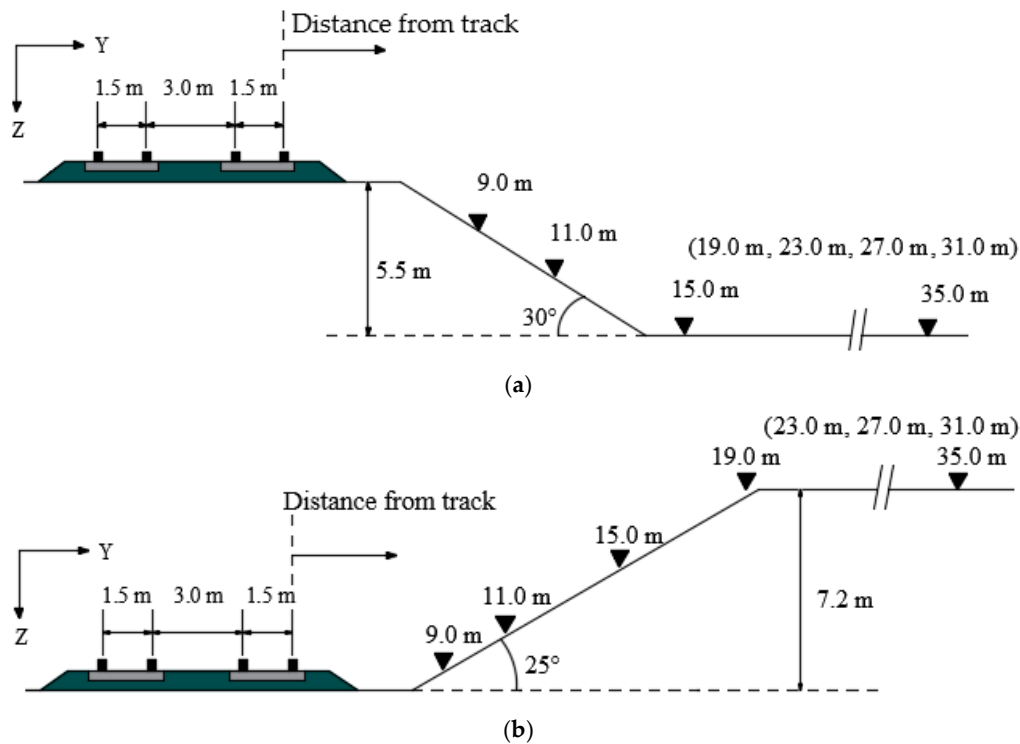


Figure 2. Measuring point arrangement for the field test sections: (a) embankment; (b) cutting.

Table 1. The mechanics parameters for the field test section of railway embankment.

Layers of Cutting	Thickness (m)	Young's Modulus E (MPa)	Poisson Ratio ν	Damping ζ	Density ρ ($\text{kg}\cdot\text{m}^{-3}$)	Shear Wave Velocity v_s ($\text{m}\cdot\text{s}^{-1}$)
Ballast layer	0.30	150.00	0.30	0.030	1700.0	184.21
Sub-ballast layer	0.20	250.00	0.30	0.030	1900.0	224.96
Subgrade	5.50	200.00	0.30	0.030	2000.0	196.11
Silt ①1	1.30	87.11	0.35	0.074	1600.0	142.00
Silt ①2	1.30	111.69	0.33	0.070	1600.0	162.00
Silt ①3	1.20	104.91	0.33	0.070	1600.0	157.00
Sand ②1	2.85	407.68	0.30	0.050	2000.0	280.00
Sand ②2	2.85	566.28	0.30	0.034	2000.0	330.00

Table 2. The mechanics parameters for the field test section of railway cutting.

Layers of Cutting	Thickness (m)	Young's Modulus E (MPa)	Poisson Ratio ν	Damping ζ	Density ρ ($\text{kg}\cdot\text{m}^{-3}$)	Shear Wave Velocity v_s ($\text{m}\cdot\text{s}^{-1}$)
Ballast layer	0.30	150.00	0.30	0.030	1700.0	184.21
Sub-ballast layer	0.20	250.00	0.30	0.030	1900.0	224.96
Silt ①1	1.35	108.95	0.33	0.077	1600.0	160.00
Silt ①2	1.35	124.45	0.33	0.070	1600.0	171.00
Sand ②1	3.10	256.60	0.29	0.031	2000.0	223.00
Sand ②2	3.10	348.82	0.29	0.050	2000.0	260.00

Note: The thickness of Silt ①1 is calculated from the bottom surface for the subgrade, and the properties of 7.2 m thick soil layer above the subgrade bottom are identical to Silt ①1.

The HST load was provided by the TGV trainset configured with eight vehicles: two power cars (Y230 A) at the leading and trailing ends, and six passenger cars (Y230 B) in the middle. The geometric dimensions and physical parameters of TGV are detailed in

Figure 3 and Table 3, respectively. During testing, the TGV train traversed the embankment and cutting sections at a speed of 294.7 km/h.

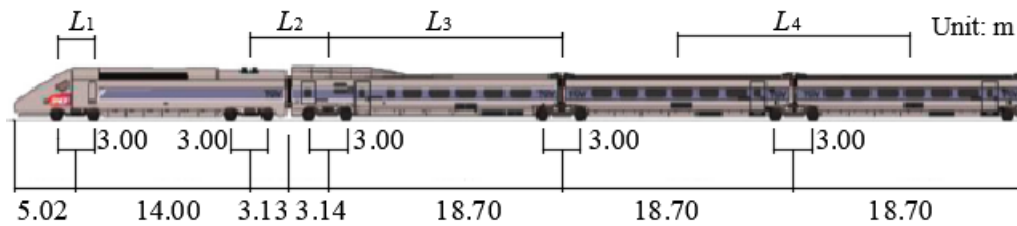


Figure 3. The configuration for TGV high-speed train.

Table 3. The parameters of TGV high-speed train.

Properties	Driving Cars	Passenger Cars
Car body mass (kg)	55,790.00	24,000.00
Bogie mass (kg)	2380.00	3040.00
Wheelset mass (kg)	2048.00	2003.00
Primary suspension stiffness ($\text{MN}\cdot\text{m}^{-1}$)	2.45	1.40
Primary suspension damping ($\text{kN}\cdot\text{s}\cdot\text{m}^{-1}$)	20.00	120.00
Secondary suspension stiffness ($\text{MN}\cdot\text{m}^{-1}$)	2.45	0.45
Secondary suspension damping ($\text{kN}\cdot\text{s}\cdot\text{m}^{-1}$)	40.00	40.00

2.2. Time–Frequency Characteristics of Ground Vibrations for the Embankment and Cutting

The time–history curves of vertical ground vibration acceleration at different distances from the track for the embankment test section are presented in Figure 4. Analysis reveals that at locations near to the track, the acceleration time-history curves exhibit clearly distinguishable periodic peaks induced by the successive passage of wheelsets. At 9.0 m from the track, the vibration amplitudes generated by the leading and trailing vehicles are marginally greater than those induced by the intermediate vehicles, which is attributed to the higher mass of the power cars (leading/trailing) compared to the passenger cars. With the increasing distance from the track, the amplitude of the vertical ground vibration acceleration progressively attenuates. Beyond 27.0 m from the track, the periodic excitation pattern becomes progressively obscured. Furthermore, at far from the track, the vibration amplitude contributed by the intermediate vehicles slightly exceeds that of the leading and trailing vehicles. This phenomenon is likely due to the superposition effect of vibrations generated by the closely spaced axles of adjacent bogies.

Fourier transform was applied to process the time–history data of ground vibration for the embankment test section, yielding the frequency spectrum of vertical vibration acceleration as shown in Figure 5. The spectrum reveals that the ground vibration in the near track zone contains relatively rich frequency components, predominantly distributed within the 20.0 Hz–80.0 Hz range. As the distance from the track increases, the frequency components above 40.0 Hz attenuate rapidly. In contrast, vibrations below 40.0 Hz decay at a slower rate. Within the distance range of 27.0 m–35.0 m from the track, the frequency components of ground vibration gradually concentrate between 20.0 Hz and 60.0 Hz. Furthermore, spectral analysis indicates that the first-order dominant frequency of ground vibration in the embankment section remains around 26.5 Hz as the measurement point moves away from the track. Based on existing research [7], this frequency is close to the fundamental excitation frequency f_1 generated by the train at the current operating speed (294.7 km/h), which is calculated as $f_1 = v/L_1 = 27.3$ Hz (where $L_1 = 3.0$ m, representing the characteristic length related to the bogie spacing, as illustrated in Figure 3).

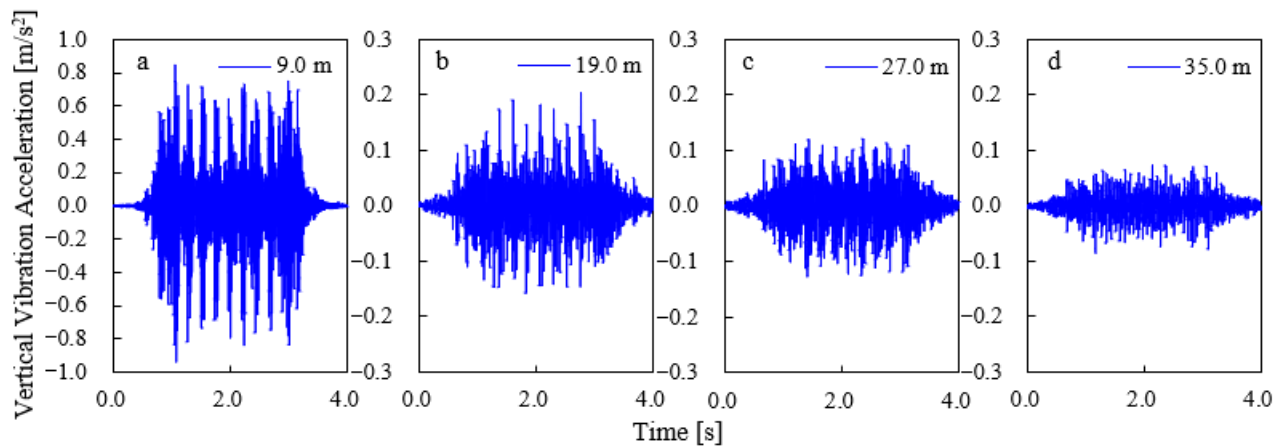


Figure 4. Time histories of vertical ground vibration accelerations of the measuring points at various distance from the embankment track: (a) 9.0 m; (b) 19.0 m; (c) 27.0 m; (d) 35.0 m.

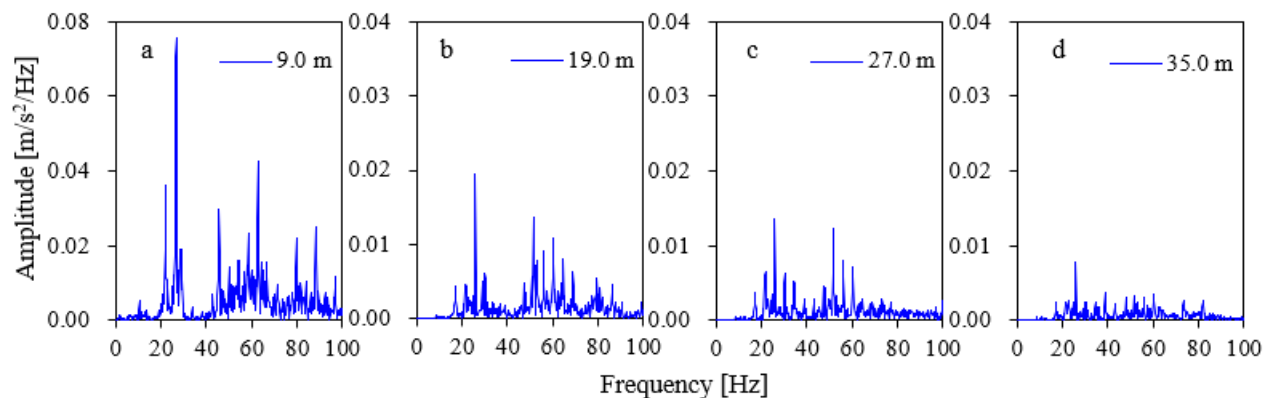


Figure 5. Frequency spectra of vertical ground vibration accelerations of the measuring points at various distance from the embankment track: (a) 9.0 m; (b) 19.0 m; (c) 27.0 m; (d) 35.0 m.

Figure 6 presents the time–history curves of vertical ground vibration acceleration within the distance range of 9.0 m to 35.0 m from the track for the cutting test section. Similarly to the observations of embankment section, the periodic peaks induced by the individual wheelsets of the train are distinctly identifiable in the acceleration time-history recorded at locations proximal to the track in the cutting section. And the amplitude of the ground vibration gradually attenuates with the increasing distance from the track. When the measurement point is 19.0 m or more away from the track, the distinct periodic pattern caused by the wheelsets becomes blurred, and is no longer clearly distinguishable. A comprehensive analysis of Figures 4 and 6 indicates that, the amplitude of ground vibration in the near track zone both for the cutting and embankment sections is predominantly influenced by the axle weight of the train. In contrast, at larger distances from the track, the vibration responses are primarily governed by the superposition effect of vibrations generated by the closely spaced axles of adjacent bogies of the intermediate vehicles.

The frequency spectrum of the vertical ground vibration acceleration for the cutting test section is shown in Figure 7. The figure shows that the ground vibration induced by the HSTs exhibits relatively rich frequency components at locations close to the track. Within the distance range of 9.0 m to 27.0 m from the track, the ground vibration of cutting section is primarily concentrated within the 20.0 Hz~90.0 Hz range. As the distance from the track increases, vibrations above 40.0 Hz attenuate rapidly. When the measurement point is 35.0 m or farther from the track, the ground vibration energy becomes predominantly concentrated in the 20.0 Hz~40.0 Hz range. Furthermore, the first-order dominant frequency

of the ground vibration remains approximately 26.8 Hz regardless of the increasing distance from the track. This frequency is close to the fundamental excitation frequency f_1 of the TGV train traveling at 294.7 km/h, calculated as $f_1 = v/L_1 = 27.3$ Hz.

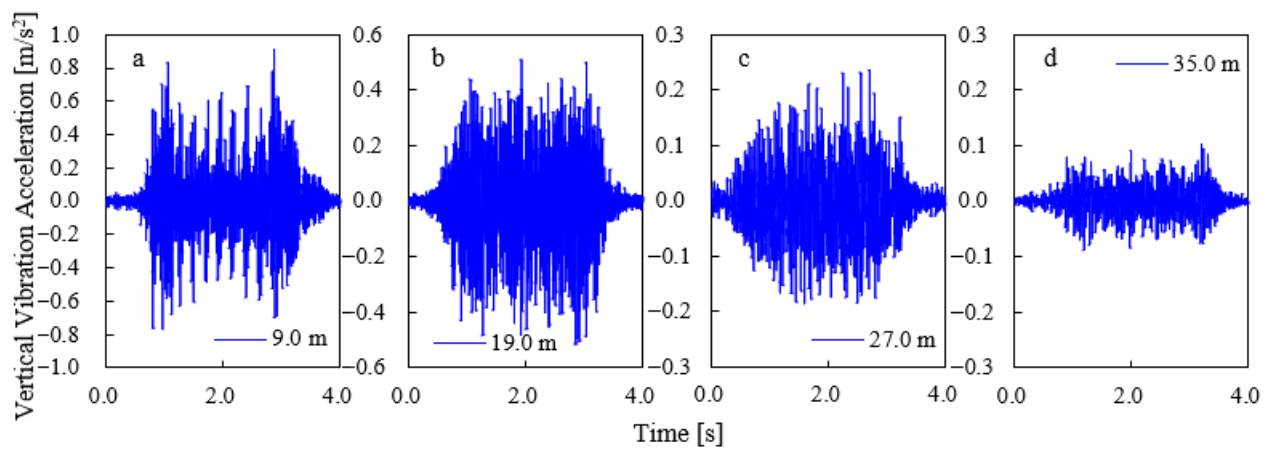


Figure 6. Time histories of vertical ground vibration accelerations of the measuring points at various distances from the cutting track: (a) 9.0 m; (b) 19.0 m; (c) 27.0 m; (d) 35.0 m.

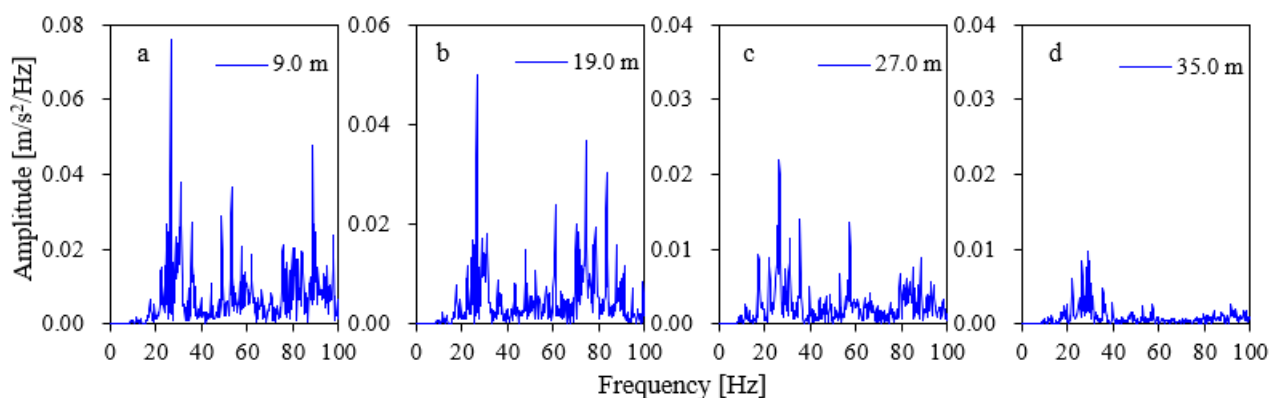


Figure 7. Frequency spectra of vertical ground vibration accelerations of the measuring points at various distances from the cutting track: (a) 9.0 m; (b) 19.0 m; (c) 27.0 m; (d) 35.0 m.

2.3. Attenuation Characteristics of Ground Vibrations for the Embankment and Cutting Sections

The attenuation curve of the peak vertical ground vibration acceleration (PGA) with the increasing distance from track in the embankment test section under HST loading is shown in Figure 8. Analysis of the figure indicates that the attenuation process of ground vibration in the embankment section can be broadly divided into two distinct stages. Within the distance range of 9.0 m to 19.0 m from the track, the PGA attenuates rapidly. Specifically, at 19.0 m from track, the vibration peak has decreased by 77.4% compared to its value at 9.0 m. This phase is thus identified as the “Rapid attenuation stage”. When the distance from the track is 19.0 m or greater, the rate of vibration attenuation gradually slows down, characterizing the “Slow attenuation stage”. It is noteworthy that a minor local rebound amplification of ground vibration is observed at 23.0 m to the track. This phenomenon is primarily attributed to the superposition of vibration waves reflected from internal soil interfaces within the foundation with those propagating along the ground surface.

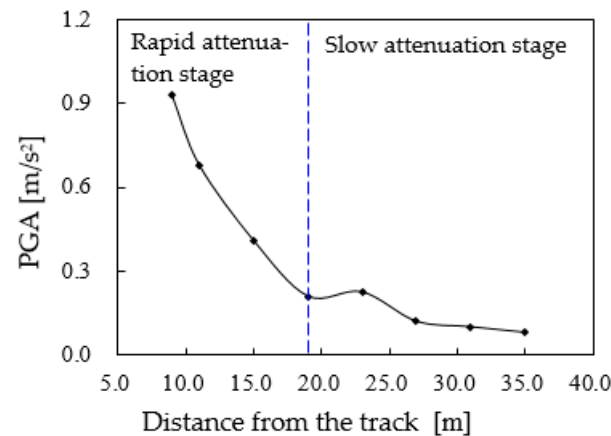


Figure 8. Attenuation curves for the peak ground accelerations with the distance away from the embankment track.

Figure 9 shows the variation curve of the PGA with increasing distance from the track in the cutting test section under HST loading. Unlike the embankment section, the attenuation process of ground vibration in the cutting section can be divided into three distinct stages. Within the distance range of 9.0 m to 15.0 m from the track, the PGA attenuates rapidly. Specifically, the vibration amplitude at 15.0 m is reduced by 50.2% compared to that at 9.0 m. This phase is thus identified as the “Rapid attenuation stage”. Within the distance range of 15.0 m to 27.0 m, the attenuation rate of the peak acceleration slows down compared to the first stage, characterizing the “Relatively fast attenuation stage”. Beyond 27.0 m, the attenuation rate further decreases, entering the “Slow attenuation stage”. Furthermore, a significant vibration amplification phenomenon is observed at the top of the cutting slope (approximately 19.0 m from the track). This phenomenon is primarily attributed to the diffraction of Rayleigh waves at the slope crest [18,19].

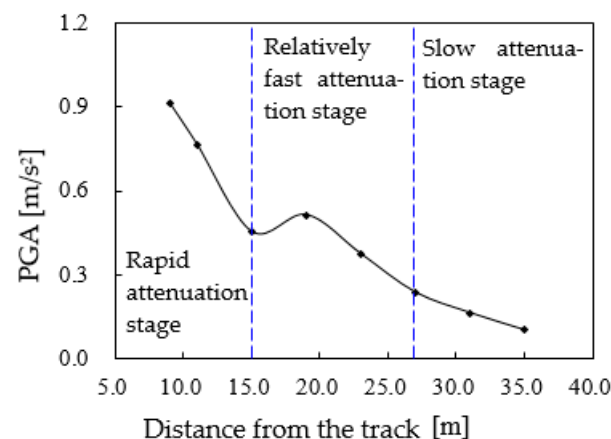


Figure 9. Attenuation curves for the peak ground accelerations with the distance away from the cutting track.

3. Modeling Approach and Validation

The 2.5D FEM addresses the 3D transient dynamic problem by applying a wavenumber expansion along the direction of train movement (x -direction) and a Fourier transform with respect to time t . This transforms the problem into the frequency-wavenumber domain for solution. The final response in the time-space domain is then obtained by performing an inverse Fourier transform on the results. Figure 10 shows the 2.5D FE models established in this study for both the embankment and cutting sections. The x -direction is defined as

the direction of train travel, the z -direction is vertical, and the y -direction is horizontal and perpendicular to the track. The double Fourier transform with respect to x coordinate and time t is defined to simplify the 3D dynamic issue, which is described as:

$$\tilde{u}(\xi_x, y, z, \omega) = \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} u(x, y, z, t) e^{i\xi_x x} e^{-i\omega t} dx dt \quad (1)$$

The corresponding inverse transform with respect to ξ_x and ω is expressed as:

$$u(x, y, z, t) = \frac{1}{4\pi^2} \int_{-\infty}^{+\infty} \int_{-\infty}^{+\infty} \tilde{u}(\xi_x, y, z, \omega) e^{-i\xi_x x} e^{i\omega t} d\xi_x d\omega \quad (2)$$

where ω and ξ_x represent circular frequency and the wavenumber corresponding to x -direction, respectively.

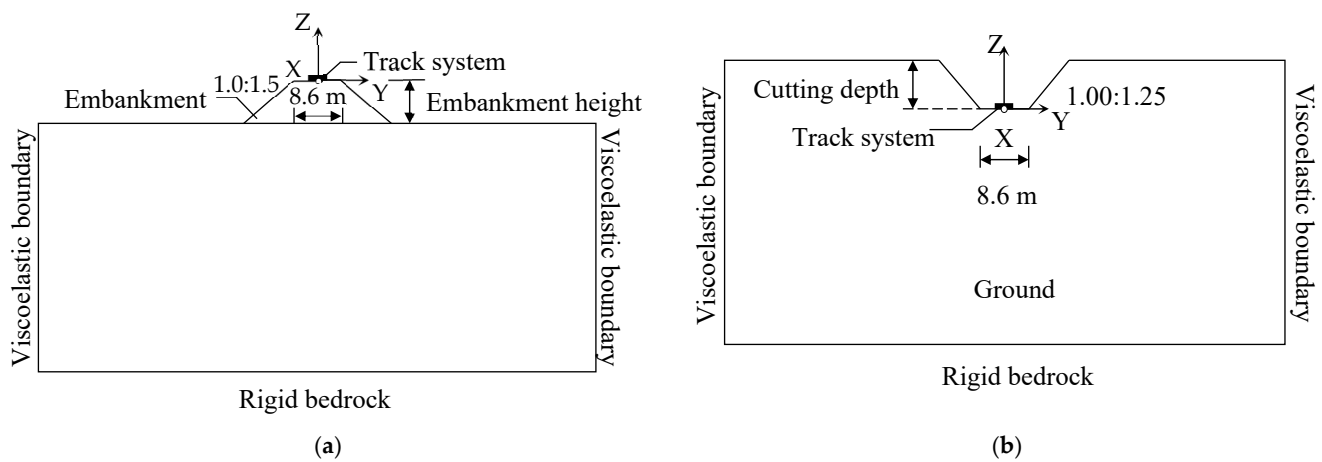


Figure 10. Schematic of the 2.5 FE models: (a) embankment; (b) cutting.

3.1. 2.5 FE Models for Embankment and Cutting

As highlighted in Ref. [12], the dynamic strain of ground soil caused by the rail traffic is generally 10^{-5} or less, thus the model was considered as viscoelastic media, and the constitutive equation is as follows:

$$\sigma_{ij} = 2\mu_d \varepsilon_{ij} + \lambda_d \delta_{ij} \theta \quad (3)$$

in which σ_{ij} is the stress tensor component; $\varepsilon_{ij} = (u_{i,j} + u_{j,i})/2$ and θ are the strain and dilatation of the medium; λ_d and μ_d are denote the Lamé constants with material damping defined by $\lambda_d = \lambda(1 + 2\beta_i)$ and $\mu_d = \mu(1 + 2\beta_i)$, where $\lambda = E\nu/[(1 + \nu)(1 - 2\nu)]$, $\mu = E/2(1 + \nu)$, E , ν , and μ are the Young's modulus, Poisson's ratio and shear modulus, respectively, β represents damping ratio.

Substituting Equation (3) into the equilibrium equation of $\sigma_{ij,j} + F_i = \rho \ddot{u}_i$, the dynamic motion equations of the ground in frequency domain are described as:

$$\mu_d u_{i,jj} + (\lambda_d + \mu_d) u_{j,ji} = \rho \ddot{u}_i \quad (4)$$

in which ρ is the material density; the superimposed dot $\ddot{\cdot}$ denotes the spatial and time derivations, respectively.

Introducing Equations (1)–(4), and meshing the calculation area with 8 nodes iso-parametric elements, the governing equation in matrix form in the frequency and wavenumber domain is:

$$\left(\tilde{\mathbf{K}} - \omega^2 \tilde{\mathbf{M}} \right) \tilde{\mathbf{U}} = \tilde{\mathbf{F}} \quad (5)$$

where $\tilde{\mathbf{U}}$ is the displacement vector in frequency and wavenumber domain; $\tilde{\mathbf{K}}$, $\tilde{\mathbf{M}}$, $\tilde{\mathbf{F}}$ denote the stiffness matrix, mass matrix, and equivalent nodal force vector, respectively.

To eliminate the influence of reflected waves from the truncated model boundaries on the calculation results, an viscoelastic boundary was adopted as the boundary condition for the model [15]. In studies focusing on the characteristics of ground-borne vibrations induced by HSRs, it is common practice to consider only the effect of a single train passage. Unlike the cyclic loading generated by repeated train operations, the consolidation settlement of the soil induced by a single train load is negligible. Moreover, the stress waves generated within the soil mass under such short-term loading are considered elastic waves [20,21]. Consequently, in the 2.5D FE model of the embankment and cutting sections established in this study, all components are connected using a shared-node approach.

3.2. Modeling the Track and Train Load

The whole track system is treated as a Euler beam with only vertical movement, and the corresponding parameters are integrated mass per unit length m_r and bending stiffness $E_r I_r$. The governing equation of the vertical motion u_r for the track in frequency-wavenumber domain is:

$$(E_r I_r \xi_x^4 - m_r \omega^2) \tilde{u}_r = \tilde{f}_T(\xi_x, \omega) + \tilde{P}(\xi_x, \omega) \quad (6)$$

where $\tilde{f}_T$ is the interaction force between the embankment and track; $\tilde{P}$ is the force excited by trains.

A typical train model is depicted in Figure 11. The train comprises N carriages, each with one body, two bogies, and four wheelsets. For a train running at speed c with self-oscillation frequency ω_0 , the successive axle load in frequency-wavenumber domain is [13]:

$$\tilde{P}(\xi_x, \omega) = \frac{2\pi}{c} \delta(\xi_x - \frac{\omega - \omega_0}{c}) \chi(\xi_x) \quad (7)$$

where $\chi(\xi_x) = \sum_{n=1}^N [p_n ((1 + e^{-i a_n \xi_x} + e^{-i(a_n + b_n) \xi_x} + e^{-i(2a_n + b_n) \xi_x})) e^{-i \sum_{m=0}^{n-1} L_m \xi_x}]$, in which the train comprise N carriages with the length of l_m (subscript $m = 1, 2, 3, \dots$, denotes the m th carriage). Each carriage includes two bogies with a spacing of b_n ; each bogie includes two wheelsets separated by distance a_n ; each wheelset contains two wheels; and each wheel load is P_n .

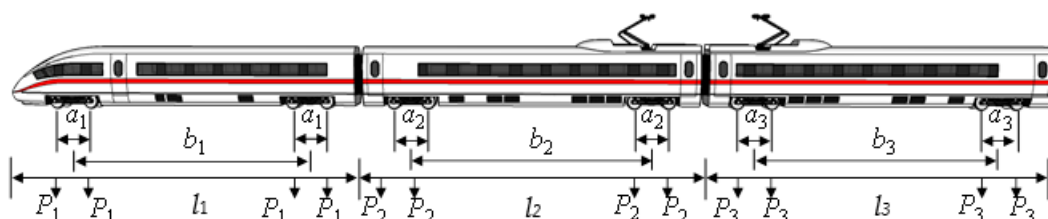


Figure 11. Geometry characteristics of high-speed train.

3.3. Validation with the Field Measurements

To validate the correctness of the 2.5D FE model developed in this study, a computational framework was implemented in Fortran. In this model, the track was simulated as an Euler-Bernoulli beam with bending stiffness $EI = 80.0 \text{ MN}\cdot\text{m}^2$ and a distributed mass $m = 10.8 \text{ t/m}$. Utilizing the parameters of the TGV HST (Table 3, Figure 3), the ballast track and the embankment test section (Figure 2a, Table 1) detailed in the preceding sections, a 2.5D FE model of the embankment section under HST loading was established. Considering a train speed of 294.7 km/h, a comparison between the measured and calculated time-history and frequency spectrum of the vertical ground vibration acceleration at 9.0 m from the track in the embankment section is presented in Figure 12. The figure shows that the calculated time-history of ground vibration acceleration agrees well with the field measurements. The periodic peaks induced by the passage of each wheelset are distinctly identifiable in the computed time-history curve. Furthermore, the computational results effectively capture the frequency-domain characteristics of the ground vibration under HST loading and accurately predict the dominant frequency of the vibration. Comparative analysis indicates that the measured data contain richer high-frequency components than the numerical predictions. This discrepancy is primarily attributed to the complexity of field conditions and the multitude of influencing factors present in real-world measurements, which are challenging to fully replicate in a numerical model.

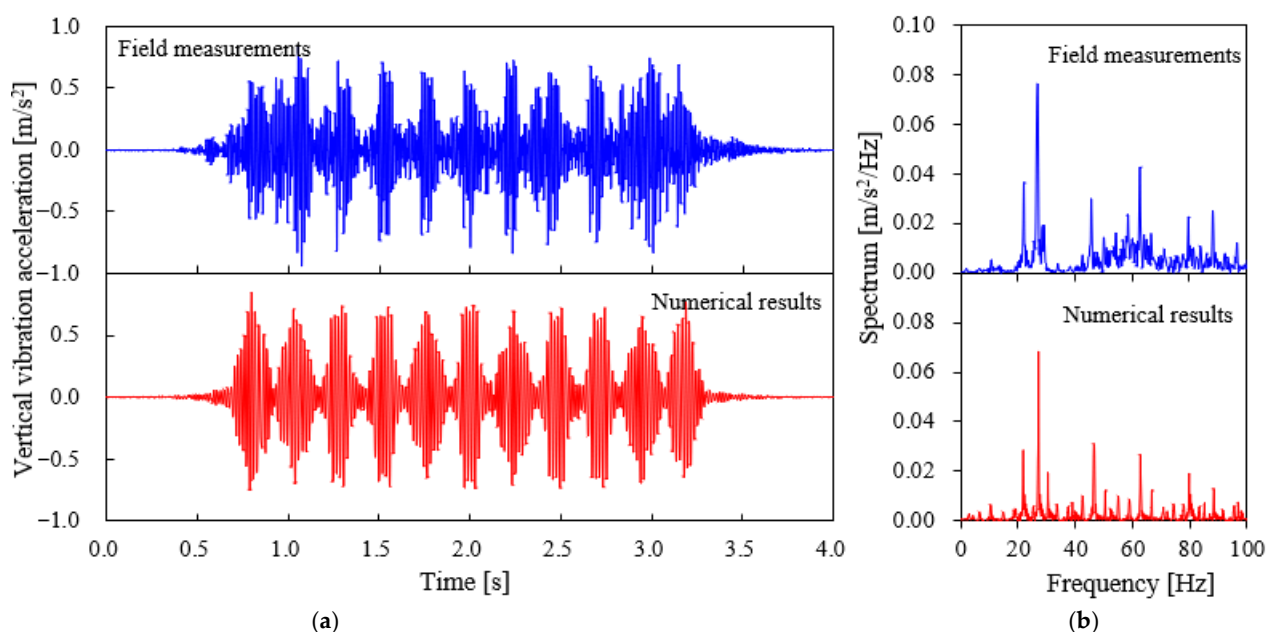


Figure 12. The simulation and measuring results at 9.0 m away from the embankment track: (a) time histories; (b) frequency spectra.

Meanwhile, a comparative analysis of measured and simulated peak vertical ground vibration accelerations (PGAs) with increasing distance from the track in the embankment section is presented in Figure 13. It can be observed from the figure that the 2.5D FE model established in this study effectively captures the attenuation trend of ground vibration with increasing distance from the track. Otherwise, from the error bars in the figure, there are some differences between the field measurements and numerical results. The error in the near track zone is relatively large, and decreases gradually with the increasing distance from the track, which is within the acceptable level of accuracy for the investigation of the HST induced ground vibration responses.

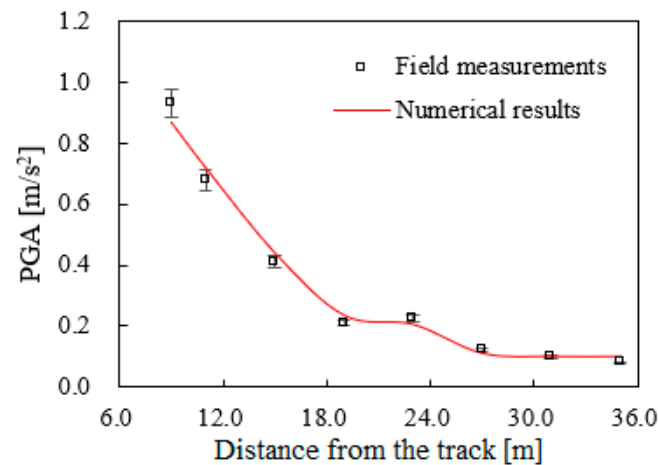


Figure 13. Attenuation curves for the PGA with distance away from the embankment track.

To further validate the reliability of the 2.5D FEM, a model of the cutting section under HST loading was established, and the diagram and parameters of which are shown in Figure 2b and Table 2. Figures 14 and 15 show the comparative analysis of measured and simulated ground vibrations with increasing distance from the track in the cutting section. Even some slight differences between the numerical results and measurements are observed, which is mainly due to the complexity of on-site testing situation, and the simplification and assumptions for the establishment of 2.5D FE model. Therefore, the close agreement between the numerical predictions and the field measurement data confirms the correctness and reliability of the proposed model.

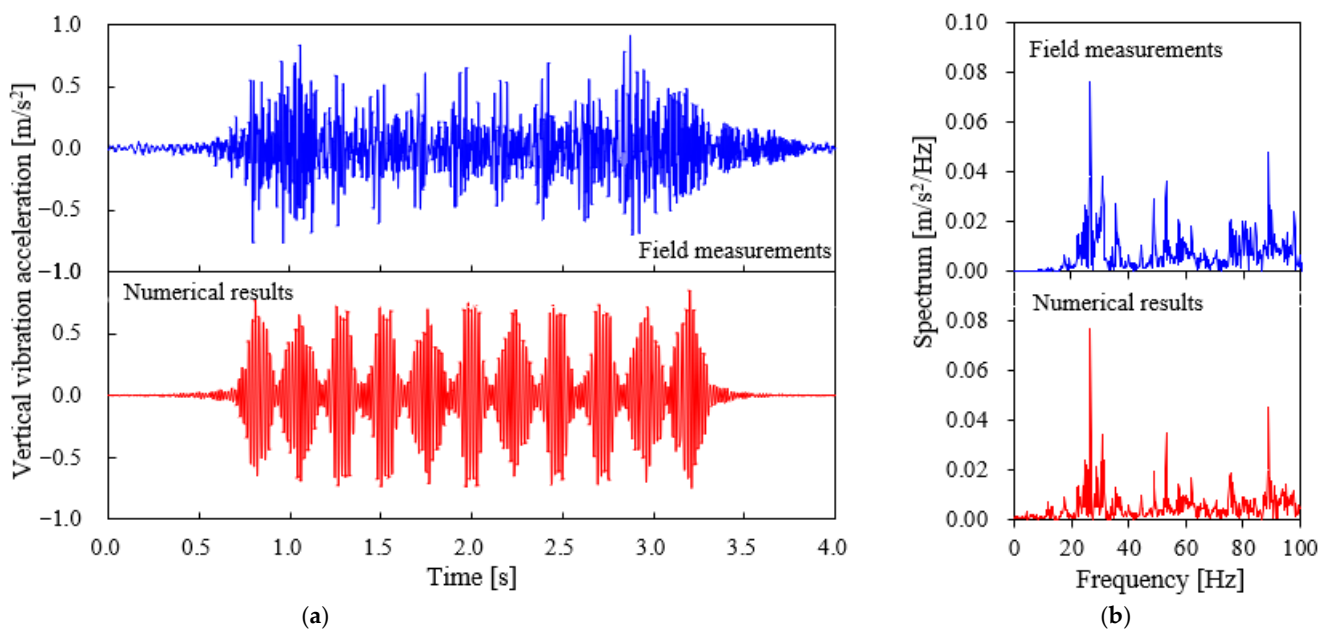


Figure 14. The simulation and measuring results at 9.0 m away from the cutting track: (a) time histories; (b) frequency spectra.

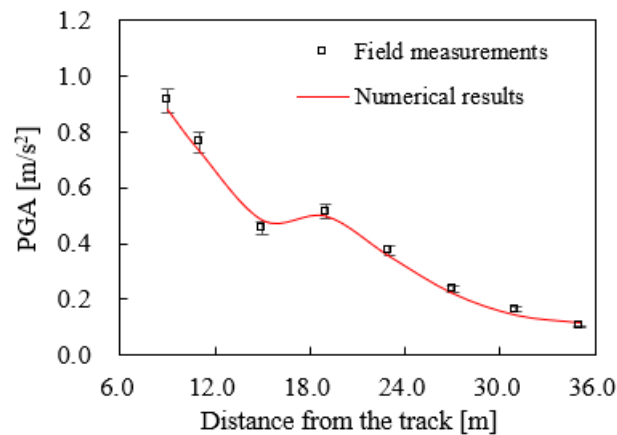


Figure 15. Attenuation curves for the PGA with distance away from the cutting track.

4. Effects of Train Speed and Earthwork Parameters on Ground Vibrations

2.5D FE models for the embankment and cutting sections with a single-track line were established, as illustrated in Figure 10. The model has a width of 100.0 m and a depth of 35.0 m. The width of the embankment top surface and the cutting bottom surface are both 8.6 m, with an embankment slope ratio of 1.0:1.5. The CRTS II slab track was chosen for the parametric analyses because it represents the standard and dominant configuration in Chinese high-speed railways. It should be noted that the track type in this section is different from the validation model applied in Section 3.3. Although the track types do really have a certain influence on the train induced dynamic responses, according to the Refs. [22,23], the effects of track types on the characteristics and propagation patterns of ground vibrations, which this study aims to, are relatively small. Thus, the parametric analysis and discussion on ground vibrations regarding the key influencing factors could be considered robust within the scope of this study.

The CRTS II track system primarily consists of rails, fasteners, CRTS II slabs, and base slabs. The components above the base slab were simplified as a Euler–Bernoulli beam with a bending stiffness EI of 65.28 MN·m² and a distributed mass m of 8.02 t/m. The base slab has a thickness of 0.3 m, with an elastic modulus E of 3.25×10^{10} Pa, a Poisson's ratio ν of 0.25, a damping ratio ζ of 0.38, and a density ρ of 2290.0 kg/m³. Viscoelastic boundary was applied on both lateral boundaries of the model to simulate the radiation damping of elastic waves in the semi-infinite space, while the bottom boundary was set as a fixed boundary. The detailed calculation parameters are summarized in Table 4.

Table 4. The calculating parameters of 2.5D FE model.

	Thickness (m)	Young's Modulus E (MPa)	Poisson Ratio ν	Damping ζ	Density ρ (kg·m ⁻³)	Rayleigh Wave Velocity v_R (km·h ⁻¹)
Embankment	—	200.0	0.30	0.05	2100.0	638.12
Ground	35.0	64.0	0.38	0.05	1850.0	382.68

Note: The soil parameters are the same for both the embankment and the cutting models.

Existing research indicates that when the train load consists of four or more carriages, the influence of the number of carriages on the ground vibration response becomes negligible [24]. Balancing computational efficiency and accuracy, a six-carriage CRH380 AL electric multiple unit formation was selected as the train load to investigate the influence of

train speed [7], as well as embankment and cutting design parameters, on the characteristics of ground vibrations induced by HSR traffic.

4.1. Ground Vibrations of Embankment and Cutting Sections Under Different Train Speeds

The Rayleigh wave velocity v_R of the ground exerts a significant influence on ground vibrations induced by HST operations [15]. Based on the Rayleigh wave velocities of the foundation soils listed in Table 4, and considering the current operational speeds and future development plans of China's HSR system, train speeds of 350.0 km/h, 380.0 km/h, and 410.0 km/h were selected for analysis. The embankment height was set at 3.0 m, the cutting depth at 4.0 m, and the cutting slope gradient at 1.00:1.25. Figures 16–19 present the time-history curves and corresponding frequency spectra of the vertical ground vibration acceleration at distances of 1.5 m and 30.0 m from the track centerline for both embankment and cutting sections under these different train speeds.

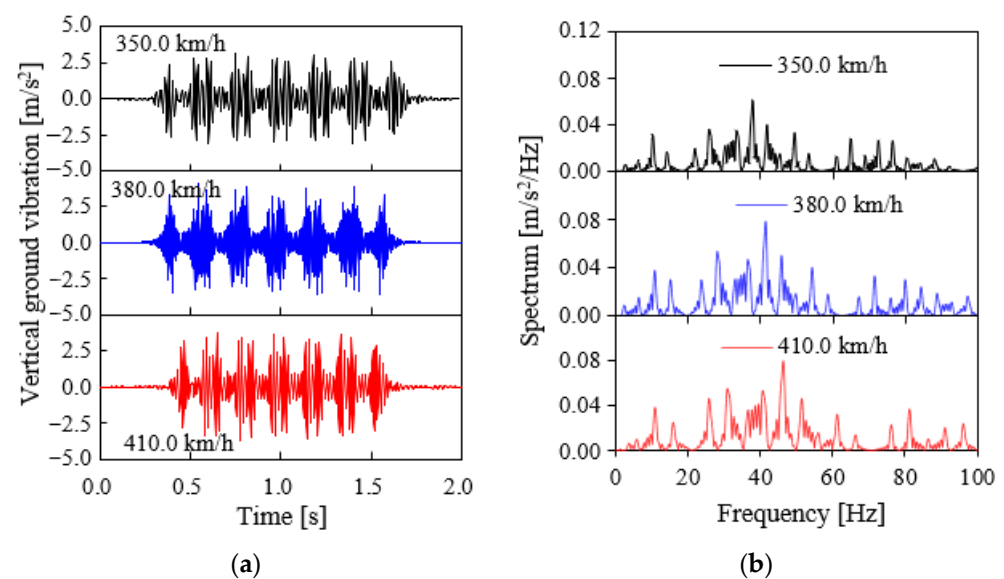


Figure 16. The ground vibrations at 1.5 m away from the embankment track center with different train speeds: (a) time histories; (b) frequency spectra.

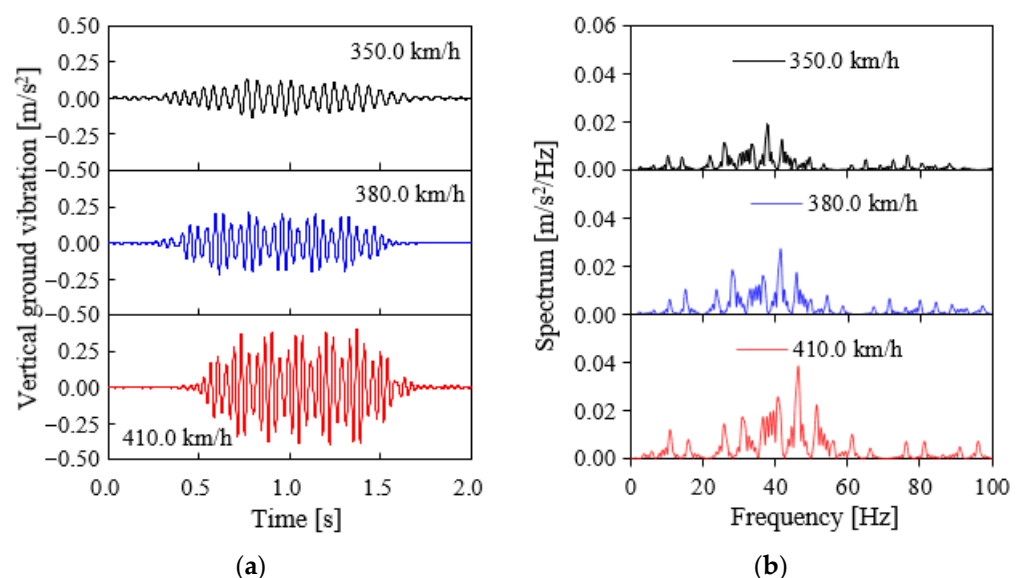


Figure 17. The ground vibrations at 30.0 m away from the embankment track center with different train speeds: (a) time histories; (b) frequency spectra.

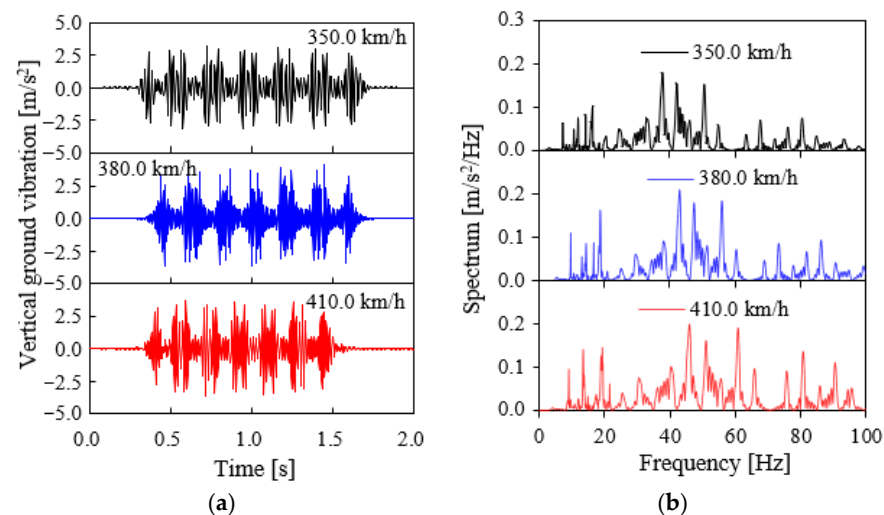


Figure 18. The ground vibrations at 1.5 m away from the cutting track center with different train speeds: (a) time histories; (b) frequency spectra.

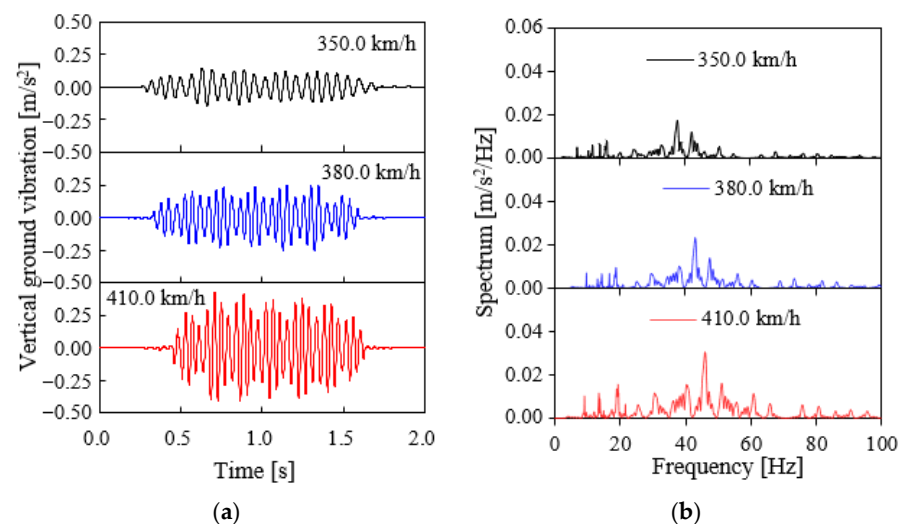


Figure 19. The ground vibrations at 30.0 m away from the cutting track center with different train speeds: (a) time histories; (b) frequency spectra.

At 1.5 m from the track center, as shown in Figures 16a and 18a, a series of periodic acceleration peaks induced by the successive passage of train wheelsets are clearly identifiable in the ground vibration time-history curves for both the embankment and cutting sections. As the distance from the track centerline increases (Figures 17a and 19a), the amplitude of the ground vibration acceleration gradually attenuates, and the distinct periodic pattern associated with individual wheelsets becomes progressively obscured. Concurrently, the vibration amplitude exhibits a general trend of increase with higher train speeds. It is noteworthy that at a speed of 380.0 km/h, the ground vibration amplitude recorded 1.5 m from the track centerline exceeds that observed at 410.0 km/h for both sections (Figures 16a and 18a). This phenomenon is primarily attributed to the train speed approaching the Rayleigh wave velocity v_R of the ground (382.68 km/h), which triggers a resonance-like effect. Furthermore, differing from the field measurement results presented earlier, the vibration amplitudes induced by the intermediate cars consistently exceed those generated by the leading and trailing cars at all monitored distances in both the embankment and cutting sections. This is due to the fact that the mass of the leading/trailing power cars of the CRH380 AL HST is less than that of the intermediate passenger cars. Thus, under these

conditions, the ground vibrations are predominantly influenced by the superposition effect of vibrations caused by the closely spaced axles of adjacent bogies in the intermediate cars.

As shown in Figures 16b and 18b, the frequency content of ground vibrations at 1.5 m from the track is relatively rich for both the embankment and cutting sections. With the increasing distance from the track center (Figures 17b and 19b), the amplitude corresponding to each frequency component gradually attenuates. Notably, the attenuation rate of vibrations above 60.0 Hz is significantly higher than that in other frequency bands. Furthermore, the first-order dominant frequency of ground vibrations in both embankment and cutting sections remains essentially constant with increasing distance from the track centerline. However, it shifts towards higher as the train speed increases. Specifically, for the embankment section, the dominant frequencies are 37.9 Hz (350.0 km/h), 41.5 Hz (380.0 km/h), and 46.3 Hz (410.0 km/h). For the cutting section, they are 37.7 Hz (350.0 km/h), 43.1 Hz (380.0 km/h), and 46.1 Hz (410.0 km/h). These observed dominant frequencies closely match the fundamental excitation frequency f_1 of the CRH380 AL HST at the corresponding speeds, calculated as 38.9 Hz (350.0 km/h), 42.2 Hz (380.0 km/h), and 45.6 Hz (410.0 km/h).

Figure 20 presents the attenuation curves of the PGA with increasing distance from the track centerline for both embankment and cutting sections under different train speeds. Analysis of the figure reveals that the attenuation curves are relatively smooth at a speed of 350.0 km/h. However, when the train speed approaches or exceeds the Rayleigh wave velocity v_R of the ground (382.68 km/h), the curves exhibit significant fluctuations. In the near track zone, the peak ground vibration amplitude at 380.0 km/h is observed to be higher than that at 410.0 km/h for both sections. As discussed previously, this phenomenon is attributed to a resonance-like effect induced as the train speed approaches the Rayleigh wave velocity of the soil. This observation also indicates that ground vibrations near the track are co-dominated by the soil resonance condition and the train speed. At greater distances from the track center, the PGA for both embankment and cutting sections generally increases with train speed. Notably, when the speed (410.0 km/h) surpasses the soil's v_R , the ground vibration amplitude is significantly amplified. This underscores the effectiveness of speed limit control as a measure to mitigate ground-borne environmental vibrations along HSR lines. Furthermore, as shown in Figure 20b, a local amplification of ground vibration occurs near the top of the cutting slope (approximately 9.3 m from the track centerline) across all tested speed conditions. As mentioned above, this amplification is primarily caused by the diffraction of Rayleigh waves at the slope crest [18,19].

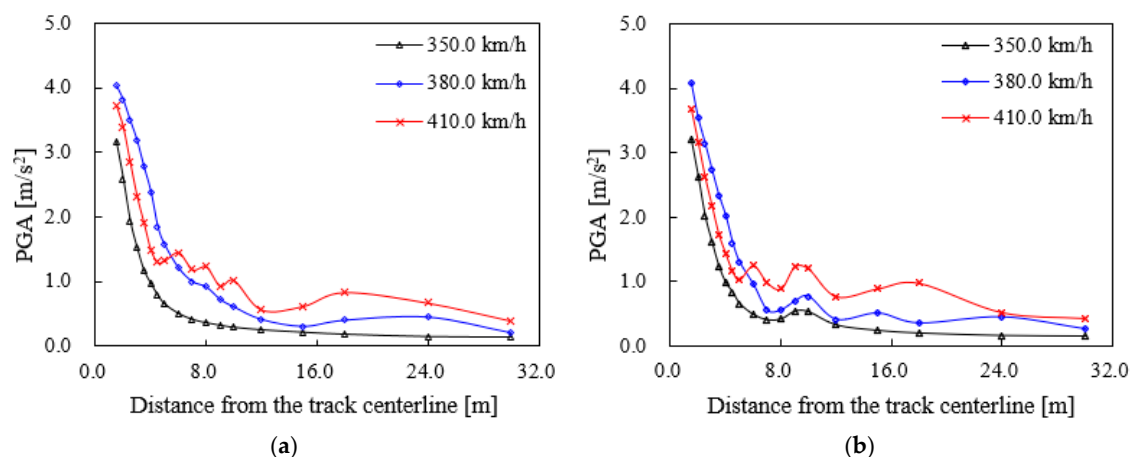


Figure 20. Attenuation curves for the peak ground acceleration with distance away from the track center with different train speeds: (a) embankment; (b) cutting.

4.2. Effects of the Embankment Design Parameters

4.2.1. Embankment Height

To investigate the influences of the embankment (or cutting) on ground vibrations induced by HST loads, considering the train speed of 380.0 km/h, an analysis was conducted with the embankment elastic modulus of 200.0 MPa. The variation curves of the peak vertical ground vibration acceleration with the distance from the track centerline for different embankment heights (1.5 m, 3.0 m, 4.5 m, and 6.0 m) are presented in Figure 21. The case without embankment or cutting (0.0 m) was established for comparison. Moreover, the changing rate of ground vibrations R_V is applied herein to characterize the vibration reduction capability of the embankment and cutting, which is defined as Equation (8). $R_V > 0$ represents positive performance in vibration mitigation.

$$R_V = \frac{\text{PGA for the case of 0.0 m} - \text{PGA generated by embankment or cutting}}{\text{PGA for the case of 0.0 m}} \quad (8)$$

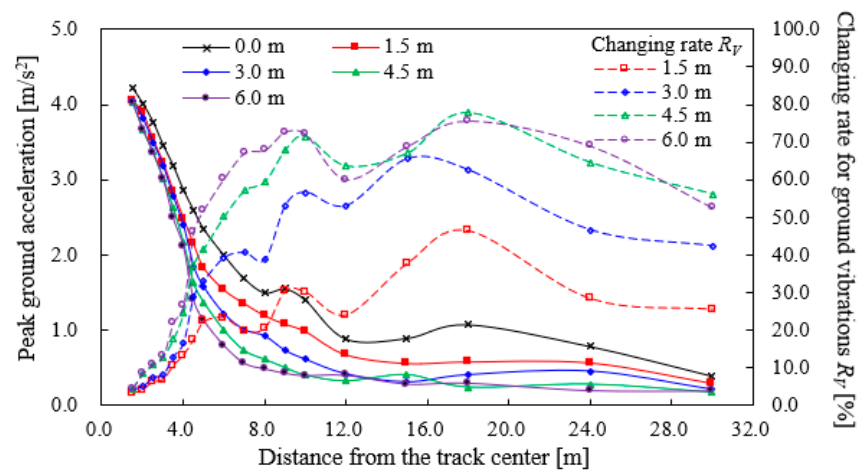


Figure 21. Variation in ground vibration and its changing rate with different embankment heights.

As shown in Figure 21, comparing to the no-embankment case (0.0 m), the presence of an embankment significantly reduces ground vibration. This reduction is attributed to the fact that the vibration waves induced by HST operations undergo multiple reflections and transmissions within the embankment structure, leading to continuous energy dissipation and, consequently, a decrease in ground vibration [5]. Furthermore, existing research indicates that the attenuation of ground vibration near the track is predominantly governed by geometric damping [25]. The embankment enhances this geometric damping effect in the near track zone, thereby further mitigating the vibrations. Analysis based on the vibration change rate derived from Figure 21 indicates that the vibration reduction effect of the embankment intensifies with increasing distance from the track centerline. The most pronounced reduction is observed within the range of 9.0 m to 24.0 m to the track center. Beyond this range, the mitigation effect gradually diminishes as the distance increases. Additionally, the PGA under HST loading decreases with increasing embankment height. However, when the embankment height reaches or exceeds a critical value of 4.5 m, further increasing the height yields negligible additional reduction in ground vibration.

4.2.2. Embankment Stiffness

Figure 22 illustrates the attenuation curves of the PGA with increasing distance from the track centerline under different embankment stiffness conditions, with a fixed embankment height of 4.5 m and a train speed of 380.0 km/h. The figure reveals that the ground vibration induced by the HST progressively decreases with the increasing elastic modulus of embankment. At 1.5 m to the track center, the PGAs corresponding to embankment elastic moduli of 150.0 MPa, 200.0 MPa, 250.0 MPa, and 300.0 MPa are 4.42 m/s², 4.03 m/s², 3.64 m/s², and 3.37 m/s², respectively. Compared to the case of 150.0 MPa, the PGAs are reduced by 8.82% (200.0 MPa), 17.42% (250.0 MPa), and 23.76% (300.0 MPa). At 30.0 m from the track centerline, the corresponding peak accelerations are 0.28 m/s² (150.0 MPa), 0.17 m/s² (200.0 MPa), 0.13 m/s² (250.0 MPa), and 0.09 m/s² (300.0 MPa), representing reductions of 39.28%, 53.57%, and 67.86% compared to the case of 150.0 MPa. These results demonstrate that the vibration reduction effect achieved by increasing the embankment's stiffness becomes more pronounced with increasing distance from the track. However, when the embankment elastic modulus reaches or exceeds a threshold of approximately 250.0 MPa, further increases in stiffness yield diminishing returns in terms of additional vibration reduction.

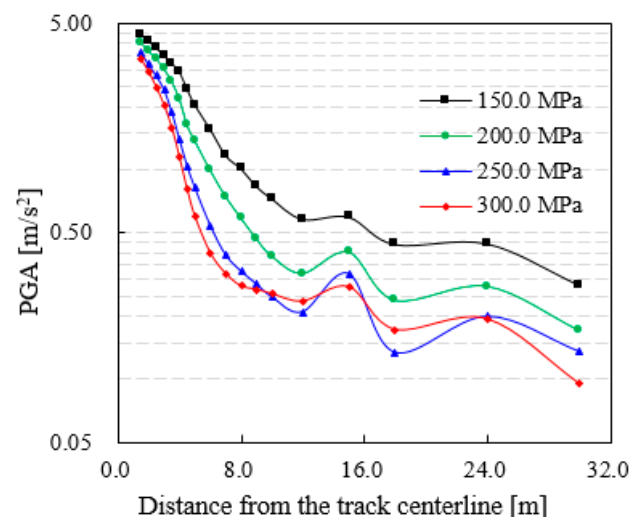


Figure 22. Variation in ground vibration rate with different embankment elastic modulus.

A comparative analysis of Figures 21 and 22 reveals that increasing the embankment height has a more pronounced effect on vibration reduction than enhancing the embankment stiffness. Consequently, in practical engineering, it is recommended to prioritize increasing the embankment height as the primary measure for mitigating ground vibrations generated by HSTs.

4.3. Effects of the Cutting Design Parameters

4.3.1. Cutting Depth

Figure 23 presents the variation curves of the PGA and the vibration change rate with increasing distance from the track center for different cutting depths (2.0 m, 4.0 m, 6.0 m, and 8.0 m), under the conditions of a train speed of 380.0 km/h and a cutting slope gradient of 1.00:1.25. It is shown in the figure that the presence of cutting significantly reduces the PGA induced by the HST compared to the no-cutting case (0.0 m depth). This reduction is attributed to the enhanced geometric damping near the track facilitated by the cutting excavation [25], and the vibration mitigation effect becomes more pronounced with increasing cutting depth. However, when the cutting depth reaches or exceeds 6.0 m, further increasing the depth yields negligible additional reduction in ground vibrations.

Furthermore, due to the diffraction of Rayleigh waves at the slope crest [18,19], a distinct local amplification of ground vibration is observed near the top of the cutting slope across all depth cases. The specific locations of this amplification are approximately 6.8 m, 9.3 m, 11.8 m, and 14.3 m from the track centerline for cutting depths of 2.0 m, 4.0 m, 6.0 m, and 8.0 m, respectively.

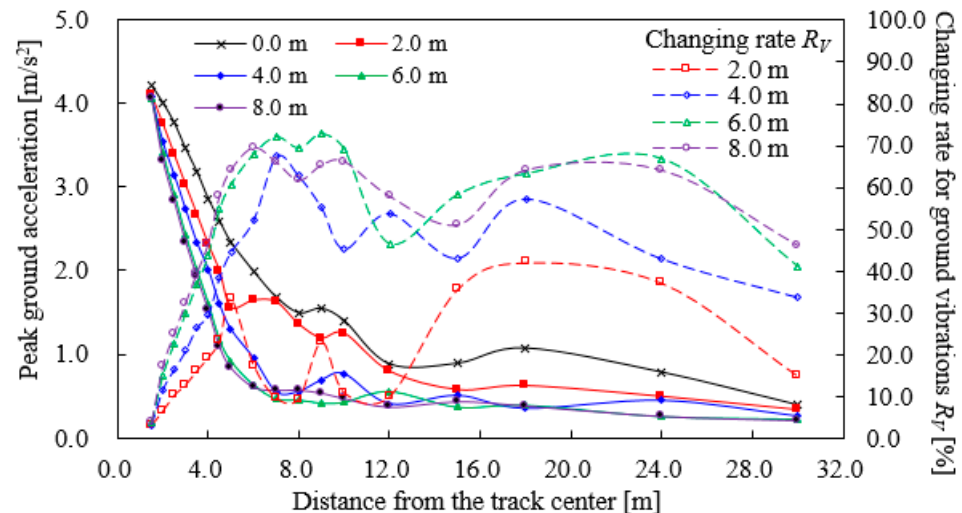


Figure 23. Variation in ground vibration and its changing rate with different cutting depths.

Additionally, in this work, it should be noted that the above critical values both for the embankment and cutting are proposed based on the analysis for the soils considered in this research, while further validation needs to be conducted for its application for other type of soil ground conditions.

4.3.2. Cutting Slope Gradient

The design slope gradient for cutting sections in Chinese HSRs typically ranges from 1.00:1.75 to 1.00:0.75 [26]. Figure 24 presents the attenuation curves of the PGAs with increasing distance from the track centerline under different cutting slope gradients (1.00:1.75, 1.00:1.50, 1.00:1.25, 1.00:1.00, and 1.00:0.75), for a cutting depth of 6.0 m and a train speed of 380.0 km/h. When the cutting slope gradient is within the range of 1.00:1.25 to 1.00:0.75 (equivalent to slope angles of 38.7° to 53.1° approximately), the ground vibration induced by HST loading decreases as the slope gradient increases (i.e., as the slope becomes steeper). This phenomenon is primarily associated with the extent of diffraction of Rayleigh waves at the crest of the cutting. Existing research indicates that within the slope angle range of 40.0° to 60.0°, the diffraction of Rayleigh waves at the slope crest diminishes as the slope angle increases [18,19], thereby reducing the ground vibration level in the cutting section. Furthermore, when the slope gradient is relatively gentle (within the range of 1.00:1.75 to 1.00:1.25), further reducing the gradient has a negligible influence on the peak ground vibration acceleration. Consequently, in practical engineering, under the prerequisite of ensuring the safety and stability of the cutting slope, adopting a steeper slope is more conducive to mitigating ground vibrations generated by the cutting sections.

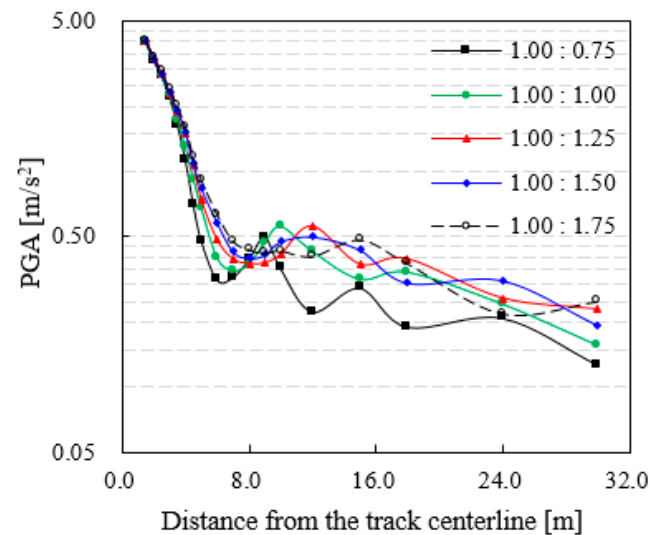


Figure 24. Variation in ground vibration rate with different cutting slope gradient.

5. Conclusions

Based on the field measurements and numerical simulations of ground vibrations induced by HSTs on embankment and cutting sections, the key findings are summarized as follows:

- (1) In the near-track zone, vibration amplitude is primarily governed by train axle load. With the increasing distance from the track, the amplitude attenuates, and the superposition effect of adjacent wheelsets and bogies becomes the dominant influencing factor.
- (2) Ground vibrations near the track contain rich frequency components. Medium- and high-frequency vibrations attenuate more rapidly with distance than low-frequency components. The dominant vibration frequency is determined by the fundamental train-loading frequency ($f_1 = c/L_1$), and shifts towards greater with increasing train speed.
- (3) The attenuation characteristics are significantly influenced by the subgrade type. The embankment exhibits a two-stage attenuation process, while the cutting section shows a three-stage process, including a distinct vibration rebound at the slope crest.
- (4) Ground vibrations in the near-track zone are co-dominated by a resonance-like condition of the ground and the train speed. At larger distances, train speed becomes the dominant factor. Significant fluctuation in vibration attenuation occurs when the train speed approaches or exceeds the Rayleigh wave velocity of the ground.
- (5) Ground vibrations decrease with greater embankment height or cutting depth. However, beyond critical values (embankment height: 4.5 m; cutting depth: 6.0 m), further increases in dimensions yield negligible reduction benefits.
- (6) A higher elastic modulus of the embankment fill correlates with reduced ground vibrations. Nevertheless, increasing the embankment height is a more effective mitigation strategy than enhancing the fill stiffness.
- (7) For the cutting section, steeper slopes (with gradients from 1.00:1.25 to 1.00:0.75) contribute to greater vibration reduction, provided the slope stability is assured.

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Data Availability Statement: Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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Conflicts of Interest: The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Abbreviations

The following abbreviations and notations are used in this manuscript:

Abbreviations

HST	high-speed train
HSR	high-speed railway
2.5D FEM	2.5 dimensional finite element method
PGA	peak vertical ground vibration acceleration

Notations

ω	the circular frequency
ξ_x	the wavenumber corresponding to x-direction
σ_{ij}	the stress tensor component
ε_{ij}	the medium strain
θ	the medium dilatation
λ_d, μ_d	the Lamé constants
E	the Young's modulus
ν	the Poisson's ratio
μ	the shear modulus
ρ	the material density
m_r	the mass per unit length
$E_r I_r$	the bending stiffness
u_r	the vertical motion
$\tilde{f}_T$	the interaction force between the embankment and track
$\tilde{P}$	the force excited by trains
P_n	each wheel load
l_m	the length of carriage
b_n	the spacing between two bogies in one carriage
a_n	the spacing between two wheelsets in one bogie

References

1. Thompson, D.J.; Kouroussis, G.; Ntotsios, E. Modelling, simulation and evaluation of ground vibration caused by rail vehicles. *Veh. Syst. Dyn.* **2019**, *57*, 936–983. [\[CrossRef\]](#)
2. Zhong, K.M.; Li, X.J.; Zhou, Z.H. High-Speed Train-Induced Vibration of Bridge–Soft Soil Systems: Observation and MTF-Based ANSYS Simulation. *Buildings* **2024**, *14*, 2575. [\[CrossRef\]](#)
3. Xing, M.T.; Zhu, J.X.; Chen, D.Q. Assessing train-induced building vibrations in a subway transfer station and potential control strategies. *Buildings* **2025**, *15*, 1024. [\[CrossRef\]](#)
4. Connolly, D.P.; Kouroussis, G.; Woodward, P.; Costa, P.A.; Verlinden, O.; Forde, M. Field testing and analysis of high speed rail vibrations. *Soil Dyn. Earthq. Eng.* **2014**, *67*, 102–118. [\[CrossRef\]](#)
5. Kouroussis, G.; Connolly, D.P.; Olivier, B.; Laghrouche, O.; Costa, P.A. Railway cuttings and embankments: Experimental and numerical studies of ground vibration. *Sci. Total Environ.* **2016**, *557*, 110–122. [\[CrossRef\]](#)
6. Krylov, V.V. Effects of the embankment topography and track curvature on ground vibration boom from high-speed trains. In Proceedings of the International Conference on Structural Dynamics, Munich, Germany, 2–5 September 2002.

7. Zhai, W.M.; Wei, K.; Song, X.; Shao, M. Experimental investigation into ground vibrations induced by very high speed trains on an on-ballasted track. *Soil Dyn. Earthq. Eng.* **2015**, *72*, 24–36. [CrossRef]
8. Connolly, D.; Giannopoulos, A.; Forde, M.C. Numerical Model. of ground borne vibrations from high speed rail lines on embankments. *Soil Dyn. Earthq. Eng.* **2013**, *46*, 13–19. [CrossRef]
9. Federal Railroad Administration. *High-Speed Ground Transportation Noise and Vibration Impact Assessment*; U.S. Department of Transportation: Washington, DC, USA, 2012. Available online: <https://railroads.dot.gov/elibrary/high-speed-ground-transportation-noise-and-vibration-impact-assessment> (accessed on 1 September 2012).
10. Zhang, G.M.; He, Y.L.; Yang, L.Z. Applicability analysis of Bornitz model for predicting environmental vibration induced by train moving on subgrade cutting. *Noise Vib. Control* **2013**, *33*, 167–170. [CrossRef]
11. Ma, K.K.; Li, B.; Wang, D. Experimental study and numerical analysis of ground vibration in cutting section of high-speed railway. *Railw. Stand. Des.* **2019**, *63*, 61–65. [CrossRef]
12. Zhang, Y.; Li, B.; Bai, G.; Luo, G.X.; Fan, X.H.; Liu, H.; Zhang, W.Q. Field test and comparative analysis of high speed railway environmental vibration of railway subgrade in loess area. *Railw. Stand. Des.* **2022**, *66*, 172–177. [CrossRef]
13. Yang, Y.B.; Hung, H.H. A 2.5D finite/infinite element approach for modelling visco-elastic bodies subjected to moving loads. *Int. J. Numer. Methods Eng.* **2001**, *51*, 1317–1336. [CrossRef]
14. Bian, X.C.; Cheng, C.; Jiang, J.; Chen, R.; Chen, Y. Numerical analysis of soil vibrations due to trains moving at critical speed. *Acts Geotech.* **2016**, *11*, 281–294. [CrossRef]
15. Gao, G.Y.; Bi, J.W.; Chen, J. Vibration mitigation performance of embankments and cuttings in transversely isotropic ground under high-speed train loading. *Soil Dyn. Earthq. Eng.* **2021**, *141*, 106478. [CrossRef]
16. Yin, K.; Wang, R.; Hu, Z.P. Dynamic response law of high-speed railway cutting induced by train operation in loess region. *Railw. Stand. Des.* **2023**, *67*, 42. [CrossRef]
17. Kouroussis, G.; Verlinden, O. Prediction of railway ground vibrations: Accuracy of a coupled lumped mass model for representing the track/soil interaction. *Soil Dyn. Earthq. Eng.* **2015**, *69*, 220–226. [CrossRef]
18. Hudson, J.A.; Knopoff, L. Transmission and reflection of surface waves at a corner 2. Rayleigh waves (Theoretical). *J. Geophys. Res.* **1964**, *69*, 281–289. [CrossRef]
19. Pilant, W.L.; Knopoff, L.; Schwab, F. Transmission and reflection of surface waves at a corner 3. Rayleigh waves (Experimental). *J. Geophys. Res.* **1964**, *69*, 291–297. [CrossRef]
20. Xia, H.; Cao, Y.M.; De Roeck, G. Theoretical modeling and characteristic analysis of moving-train induced ground vibrations. *J. Sound Vib.* **2010**, *329*, 819–832. [CrossRef]
21. Zhang, J.Y.; Gao, G.Y.; Bi, J.W. Analysis of vibration mitigation for CFG pile-supported subgrade of high-speed railway. *Soil Dyn. Earthq. Eng.* **2023**, *164*, 107612. [CrossRef]
22. Wu, Y.; Fu, H.R.; Bian, X.C. Comparative study on dynamic responses of ballasted and ballastless tracks at critical velocity. *Transp. Geotech.* **2024**, *48*, 101354. [CrossRef]
23. Feng, Q.S.; Cheng, G.; Lei, X.Y.; Lian, S.L. Influences of ballasted track and slab ballastless track structures on ground vibration of high speed railway. *J. Vib. Shock* **2015**, *34*, 153–159. [CrossRef]
24. Yang, Y.; Hung, H.; Chang, D. Train-induced wave propagation in layered soils using finite/infinite element simulation. *Soil Dyn. Earthq. Eng.* **2003**, *23*, 263–278. [CrossRef]
25. Gao, G.Y.; Song, J.; Yang, J. Identifying boundary between near field and far field in ground vibration caused by surface loading. *J. Cent. South Univ.* **2014**, *21*, 3284. [CrossRef]
26. TB 10001-2016; Code for Design on Roadbed of Railway. China Railway Publishing House: Beijing, China, 2017.

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