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Research on the Dimension of Ballastless Track Transition Section in High-speed Railway Based on UM Simulation

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Abstract

Based on the vehicle-track coupled dynamics theory, this study establishes a dynamics analysis model for a Chinese standard electric multiple unit (EMU). Aiming to systematically analyze the dynamic performance of high-speed trains passing through subgrade-bridge transition zones from the perspective of operational safety, simulation calculations were conducted using UM software under identical track irregularity excitation conditions. These simulations modeled the dynamic responses of the vehicle-track system in ballastless track transition zones of high-speed railways under various structural dimensions and operating speeds. Key dynamic performance indicators, including wheel-rail forces and car body accelerations, were selected to evaluate train operational safety and ride comfort. The research findings provide an important theoretical basis for the optimized design and construction of high-speed railway subgrade-bridge transition zones.

Keywords

High-Speed Railway (HSR); Ballastless Track; Subgrade-Bridge Transition Zone; Dynamic Response

1. Introduction

In recent years, China's high-speed railway (HSR) has experienced rapid development. The "Four Vertical and Four Horizontal" HSR network has been completed, and the "Eight Vertical and Eight Horizontal" HSR network is being further densified. All Chinese cities with populations exceeding one million are now covered by HSR. Compared to conventional railways, HSR is more efficient, safer, and offers higher cost-effectiveness, demanding extremely stringent track precision requirements. If abrupt changes in structural stiffness occur at the ends of a railway transition zone, the smoothness of the track structure can be severely compromised, significantly threatening train operational safety [1]. Therefore, implementing reasonable track structural measures to achieve a smooth transition in track stiffness is a critical factor enabling the rapid development and widespread adoption of HSR. During the planning and construction of HSR, international researchers place particular emphasis on the design of track structures at transition zones, employing various measures to precisely control changes in track stiffness and rail surface deformation, thereby ensuring high track geometric quality. For instance, current German Federal Railway specifications such as *DS 899/5: High-Speed Railway Bridges for Speeds above 160 km/h* and DV 804: Railway Bridges and Other Structures [3] detail design and construction standards for subgrade-bridge transition zones, utilizing methods like subgrade reinforcement and installing sliding plates at the abutment end during construction. Japan employs the Japanese Railway Technical Standards [2], utilizing reinforced earth to reduce post-construction settlement of embankments.

Professor David Davis et al. [4] in the US used the finite element software Geotrack to simulate the dynamic responses generated by heavy-haul trains entering transition zones, investigating the influence of parameters such as damping and stiffness. Their results indicated that the attenuation of wheel-rail interaction forces can be controlled by adjusting track stiffness and damping. Ding Qing et al. [5] experimentally simulated the formation process of structural defects in bridge transition zones and proposed corresponding solutions based on their findings. Keer and Bathurst [6], along with Hyslip et al., suggested that the dynamic response of transition zones could be improved by reducing structural stiffness. Hyslip et al. also proposed that creating a gap in the ballast layer on the bridge could address transition zone challenges by reducing bridge stiffness.

Cai Chengbiao et al. [7] established a spatial coupled model for ballasted-ballastless track transition zones, identifying a series of parameter indicators for evaluating the dynamic performance of track transition zones. They investigated the influence of initial rail deformation, train direction, and operating speed on the dynamic performance of the wheel-rail system under the effect of differential foundation settlement [8]. Their research proposed that when determining the length of track transition zones, factors such as operating speed and differential foundation settlement should be considered, with car body acceleration and subgrade bed surface stress having significant impacts. Zhang Xing [9] obtained the longitudinal settlement pattern of subgrade-bridge transition zones through numerical simulation: longitudinal settlement increases with distance from the abutment back, stabilizing beyond a certain distance. Ma Qiuhuan [10] simplified transition zone settlement into three types: cosine, angular discontinuity, and step. She proposed grouting technical solutions for different settlement scenarios, finding that all indicators significantly decreased after repair, achieving favorable remediation outcomes.

In summary, researchers worldwide have conducted relevant studies on various factors concerning ballastless track transition zones, including structural design, track stiffness, geometric smoothness, and connection strength, yielding a series of achievements. However, there is no unified, universally applicable theoretical basis for determining the optimal length of ballastless track transition zones. Construction often relies on engineering experience. The latest Code for Design of High Speed Railway only stipulates that the transition zone length should be no less than 20m. With the continuous development of

HSR and increasing operational speeds of modern trains, whether ballastless track transition zone design will become a weak link in HSR requires further investigation.
Therefore, this paper conducts an analysis based on vehicle-track coupled dynamics principles. A vehicle-track coupled model is established using UM software to study the impact of excitations such as track irregularities on the dynamic performance of the vehicle-track system. The goal is to explore appropriate structural dimension requirements for transition zones, providing a crucial foundation for the design and construction of high-speed railway transition zones.

1. Introduction to UM Software

Universal Mechanism (UM) software, developed by Bryansk State Technical University in Russia, is a computational multibody system dynamics software package. It is used to simulate the mechanical performance of vehicle systems, including peak loads, and excels at conducting rigid-flexible coupling analysis. UM finds wide application across multiple domains such as railways, highways, monorails, maglev systems, and tracked vehicles.
Utilizing the Computer Aided Engineering (CAE) simulation capabilities provided by UM software, engineers can guide physical testing and conduct parametric simulation studies during the design phase. This enables the filtering out of interference factors, identification of optimal solutions, and significant cost savings. The software supports modeling of both single vehicles and multi-vehicle consists, as well as calculations for both variable and constant speeds.UM offers specialized tools for railway alignment design (both horizontal and vertical planes) and supports the importation of measured track geometry data. It also provides dedicated tools for generating track irregularities and assessing critical vehicle performance metrics such as critical speed, safety indices, and ride stability (running smoothness).
Considering the characteristics and primary influencing factors of the vehicle, track, and subgrade structure, establishing a reliable vehicle-track-subgrade spatial coupling model integrated with actual field data holds significant theoretical significance and engineering practice value for the simulation analysis and design research concerning the dynamic characteristics of ballastless track subgrade-bridge transition zones in high-speed railways.

2. Vehicle-Track Dynamics Model

2.1 Vehicle Model

Based on the actual engineering parameters of the Shanghai-Nanjing Intercity Railway, this study categorizes the subgrade-bridge transition zones according to their dimensional characteristics (as shown in Figure 1). A CRH3 vehicle model was developed using the multibody dynamics software UM, while the substructure model of the ballastless track was established using the finite element software ABAQUS. To comprehensively capture the dynamic response data of the vehicle under accelerating and variable-speed conditions, the track model length was set to 200 meters, employing the standard Chinese CHN60 rail profile for the rails.

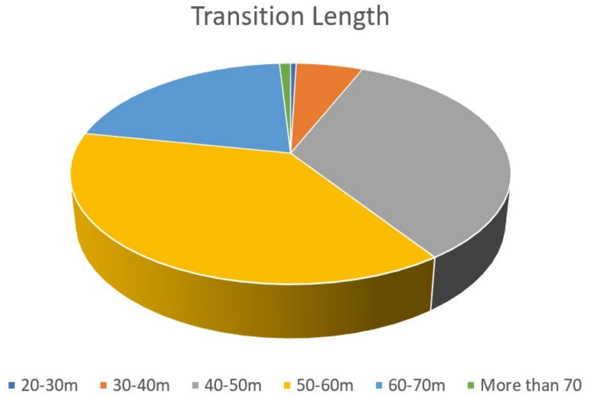


Figure 1. Dimension Distribution of the Transition Zone

The vehicle model is a multi-rigid-body system, comprising one car body, two bogies, and four wheelsets. In the modeling process, it is assumed that no elastic deformation occurs in any component during vibration. The car body is connected to the bogies via the secondary suspension system, while the bogies are connected to the wheelsets via the primary suspension system. The suspension systems primarily consist of spring and damping components.The spatial vehicle system possesses a total of 35 degrees of freedom (DOF).

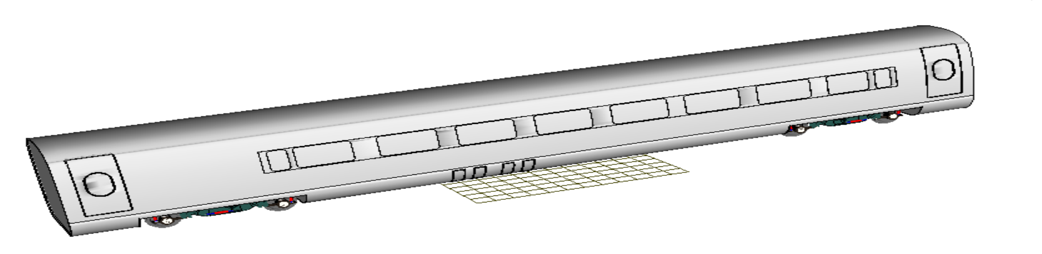


Figure 2. Vehicle Model

2.2 Track Model

The CRTS Type I ballastless track structure consists of the following layers from top to bottom: rail, fastening system, track slab, CA mortar layer, and supporting layer (or base slab). The rail, track slab, CA mortar layer, and supporting layer were all modeled using solid elements. The fastening system was modeled using connector elements. The subgrade section comprises the subgrade surface layer, subgrade subbase layer, and subgrade embankment body. All three subgrade layers were simulated using solid elements. Tie constraints were applied between the track slab, CA mortar layer, supporting layer, the various subgrade layers, and the foundation.

2.3 Bridge and Transition Zone Model

The bridge section employs a 32-m simply supported beam constructed of C50 concrete. The bridge model was established using solid elements. Bridge bearings were simulated using connector elements. Two bearings were arranged on each side of the bridge: one fixed bearing and one expansion (movable) bearing.

1. The subgrade-bridge transition zone lengths investigated in this paper are 20 m, 40 m, and 60 m. Within the transition zone section: A certain amount of cement was added to the graded crushed stone within the subgrade surface layer.
2. The lower portion of the subgrade surface layer was filled with layered graded crushed stone.
3. The connection segments adjacent to the graded crushed stone were filled with Group A and Group B fills, respectively.
4. The transition zone structure was constructed using C30 concrete.

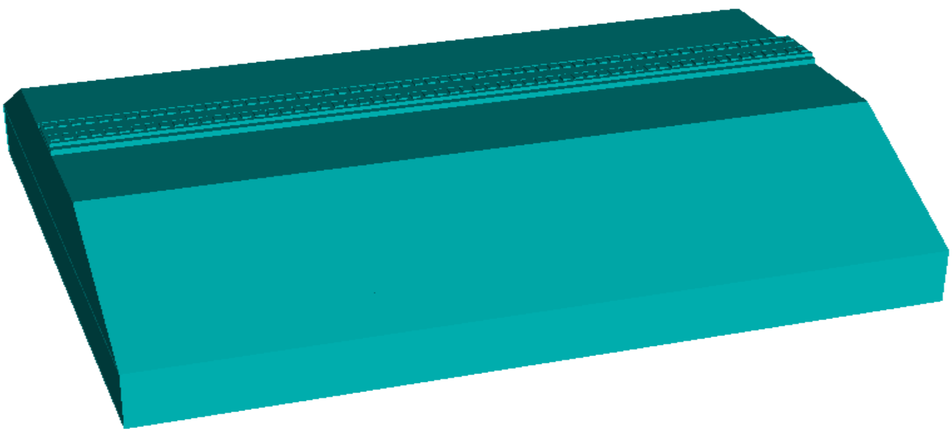


Figure 3. Track Model

2.4 Track Irregularity

This study primarily considers two types of track irregularities: longitudinal level (vertical profile) irregularity and alignment (lateral profile) irregularity. To enhance the authenticity and accuracy of the model, measured track irregularity samples obtained from the Shanghai-Nanjing Intercity Passenger Dedicated Line were employed as input parameters for the simulation model.

Within the model, the aforementioned measured track irregularity excitation was applied by modifying the coordinate values of the surface nodes belonging to the rail elements. This implementation was specifically achieved by generating corresponding way files compatible with the simulation software.

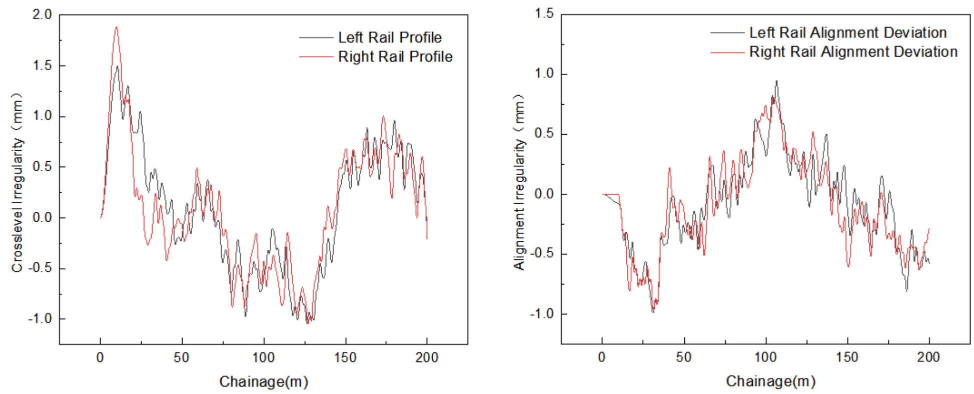
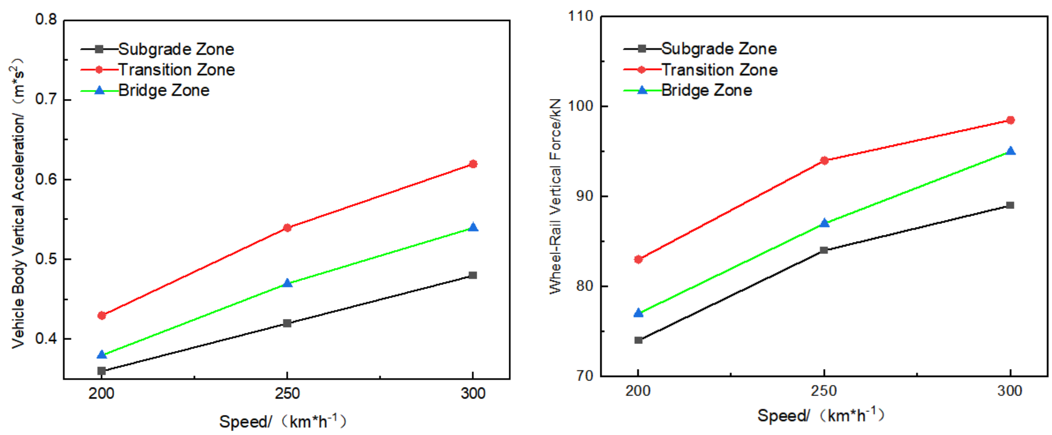


Figure 4. Track Irregularity Sample

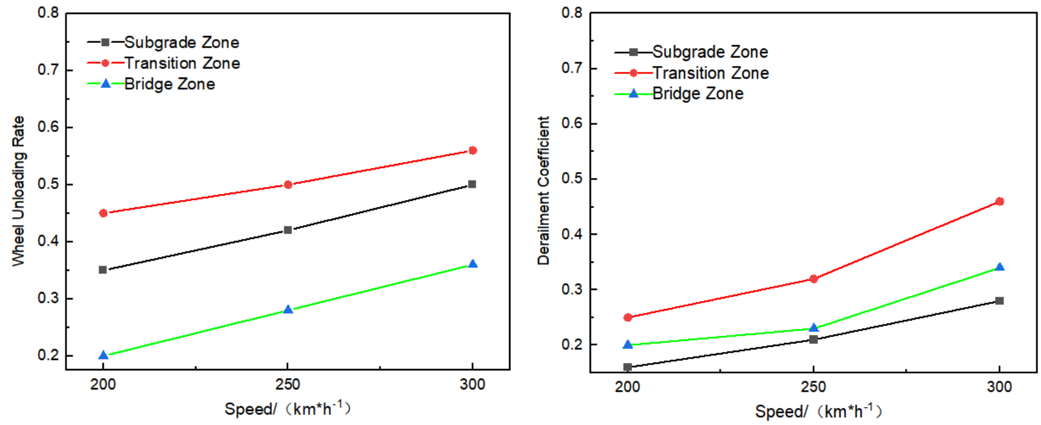
3. Analysis of Dynamic Response in the Transition Zone

3.1 Influence of Train Speed on System Dynamic Response

To investigate the dynamic response characteristics of the subgrade-bridge transition zone, a dynamic simulation model was established in this study. Under the excitation of track irregularities, the dynamic responses of the ballastless track system were simulated and analyzed for three operational speed scenarios: 200 km/h, 250 km/h, and 300 km/h. The direction of travel was set from the subgrade section towards the bridge section. The maximum values of the dynamic responses under each scenario were extracted as analysis results; the relevant calculation results are illustrated in the corresponding figures.



(a) Car body vertical acceleration (b) Wheel-rail vertical force



(c) Wheel load reduction rate (d) Derailment coefficient

Figure 5. Train dynamic responses

As shown in Figure 5, compared to the subgrade section and the bridge section, all train dynamic indicators exhibit an increasing trend within the 20-m long subgrade-bridge transition zone. Furthermore, this amplification effect intensifies with increasing train speed. Despite this, the measured values of all indicators remain below the corresponding safety limits, indicating that train operation still satisfies safety conditions. Taking the 300 km/h operating speed as an example: The derailment coefficient in the transition zone is 0.46; The wheel load reduction rate is 0.56; Both values are below the safety limit of 0.8. The vertical acceleration of the car body is 0.62 m/s². Notably, this acceleration value is 29.1% and 14.8% higher than that in the subgrade section and bridge section, respectively, representing a relatively significant increase.

The aforementioned results demonstrate that, although meeting safety operational standards, the subgrade-bridge transition zone is more prone to generating adverse dynamic responses and carries a relatively higher potential risk compared to the adjacent subgrade and bridge sections. Therefore, in high-speed railway engineering practice, significant emphasis must be placed on the optimized design and construction quality of transition zones to prevent them from becoming relatively weak sections within the high-speed railway line.

3.2 Influence of Structural Dimension on System Dynamic Response

To investigate the dynamic response characteristics of subgrade-bridge transition zones with different structural dimensions, the system dynamic responses were calculated for transition zone lengths of 20 m, 40 m, and 60 m. The specific scenario settings are detailed in the table below.

Parameter settings for simulation scenarios

Operating Condition	Speed（km/h）	Length（m）
1	200	20
2	250	20
3	300	20
4	200	30

5	250	30
6	300	30
7	200	60
8	250	60
9	300	60

Under identical structural dimensions, the vertical acceleration of the car body, derailment coefficient, and wheel load reduction rate all increase with rising operating speed. Notably, this deterioration effect in dynamic response caused by speed increase is most pronounced when the structural dimension is at the minimum specified by the code (20 m). Under this condition, all dynamic response values are higher than those observed in the other dimension scenarios:

The maximum vertical acceleration reaches 0.62 m/s², which is 25.5% and 14.8% higher than the values for the 40 m and 60 m scenarios, respectively.

- a) The maximum derailment coefficient is 0.46, 18.5% and 43.7% higher than the values for the 40 m and 60 m scenarios, respectively.
- b) The wheel load reduction rate is 0.56, 11% and 15% higher than the values for the 40 m and 60 m scenarios, respectively.

However, even in this minimum code dimension scenario, all safety indicator values remain well below the safety limits specified by the code, indicating that train operational safety remains high. When the speed reaches 300 km/h, even with a larger dimension (60 m) subgrade-bridge transition zone, its safety indicators deteriorate significantly. Its dynamic response level even approaches that observed in the 40 m length scenario at the same speed.At the same operating speed, increasing the transition zone dimension from 20 m to 40 m significantly reduces all safety indicators:

- a) Car body vertical acceleration decreases to 0.42 m/s² (a reduction of 32.2%).
- b) Derailment coefficient decreases to 0.38 (a reduction of 17.4%).
- c) Wheel load reduction rate decreases to 0.45 (a reduction of 19.6%).

This indicates that appropriately increasing the transition zone length can effectively enhance operational safety and reduce accident risk.

Simultaneously, for all speed levels, when the transition zone dimension is further increased from 40 m to 60 m, a minor increase is observed in all three safety indicators. Despite this, their values are still superior to the response values observed under the 20 m length scenario. This suggests that the improvement in safety performance gained by extending the length beyond 40 m to 60 m is relatively limited. Therefore, considering engineering economy, the application of longer transition zone dimensions can be reasonably controlled.

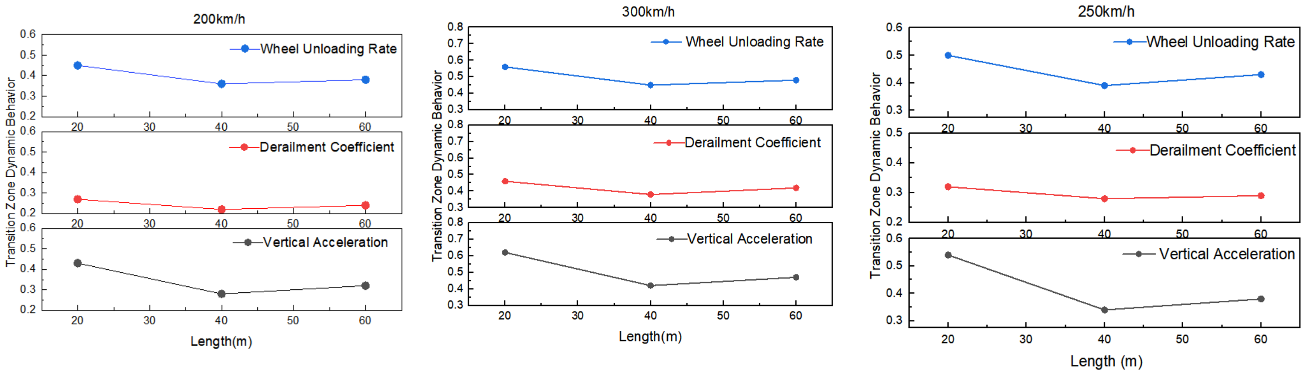


Figure 6. Dynamic response of the transition zone

4. Conclusions

The research findings demonstrate that as the train operating speed increases (from 200 km/h to 300 km/h), all dynamic response indicators within the subgrade-bridge transition zone, bridge section, and subgrade section exhibit an increasing trend. Although within this speed range, the train safety and ride comfort indicators still satisfy the relevant code requirements, ensuring safe operation, compared to the adjacent subgrade and bridge sections, the transition zone is more prone to adverse dynamic response amplification phenomena, carrying a relatively higher potential risk. Therefore, in high-speed railway engineering practice, significant emphasis should be placed on enhancing the optimized design and construction quality control of transition zones to prevent them from becoming relatively weak links within the line.

Analysis revealed that increasing the structural dimension of the transition zone can effectively enhance the safety of high-speed train operation and reduce potential risks; conversely, increasing the operating speed significantly diminishes the safety margin of the track structure, an effect particularly pronounced over transition zones with smaller dimensions. For small-dimension transition zones, appropriately controlling the operating speed is an effective strategy to prolong the service life of the track structure.

Specifically, when the length of the subgrade-bridge transition zone is increased from 20 m to 40 m, the values of all safety indicators decrease significantly. This indicates that appropriately extending the transition zone length can effectively enhance operational safety. However, when the length is further increased beyond 40 m to 60 m, the improvement in safety performance is relatively limited. Considering the combined effect of safety performance enhancement and engineering economy, the application of excessively long transition zones can be reasonably controlled. Based on the results of this study and practical engineering considerations, it is recommended to set the length of subgrade-bridge transition zones at approximately 40 meters.

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