Scientific Research

Research on the Technical Index of Vertical Comfort for Long-Span High-Speed Maglev Bridge Tracks

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Abstract: The comfort of maglev trains is related to the vibration in the passenger area of the vehicle and is determined by the acceleration and impact in three directions as the train passes over the track. The comfort of the maglev track is achieved by limiting the maximum acceleration and impact to not exceed specified limits. Unlike small-span maglev bridges, long-span bridges experience changes in deck alignment under the effects of temperature, wind load, and train load, which directly affect the vertical curve alignment of the bridge track. Based on the existing technical standards and related calculation principles and methods for vertical and horizontal curve minimum radii in current maglev track design, the relationships among the vertical acceleration, impact, and alignment and bending angles of the bridge track at the train operating velocity are determined. By verifying the instantaneous deformation curves of the bridge deck using the representative positions of the train's head, middle (center of the car length), and tail during travel, the technical index for the comfort of the bridge track during instantaneous bridge deformation are established. This paper provides the calculation loading method and results for the comfort of long-span bridges using a specific long-span maglev bridge as an example.

Keywords: long-span bridge; high-speed maglev; bridge track beam; comfort; acceleration; impact

Citation: Xu, L.; Liu, J.; Zhang, C. Research on the Technical Index of Vertical Comfort for Long-Span High-Speed Maglev Bridge Tracks. *Prestress Technology* **2024**, *2*, 32-40. https://doi.org/10.59238/j.pt.2024.02.003

Received: 04/02/2024
Accepted: 07/06/2024
Published: 30/06/2024

1 Introduction

Passenger comfort is related to the vibration in the passenger area of the train and is determined by the acceleration and impact in three directions as the train passes over the track. The acceleration is the rate of change in the velocity, while the impact is the rate of change in the acceleration. The comfort of the maglev track is achieved by limiting the maximum values of acceleration and impact to not exceed specified limits. Even during emergency braking at 500 km/h, passengers do not feel a loss of balance. Passengers can walk freely while the train is in motion without the need for seat belts.

The ISO 2631 standard provides test results related to acceleration and comfort: an acceleration less than 0.315 m/s² is not uncomfortable; an acceleration of 0.315 m/s² ~ 0.63 m/s² is slightly uncomfortable; an acceleration of 0.5 m/s² ~ 1.0 m/s² is somewhat uncomfortable; an acceleration of 0.8 m/s² ~ 1.6 m/s² is uncomfortable; and an acceleration of 1.25 m/s² ~ 2.5 m/s² is very uncomfortable.

For route selection, it is necessary to consider not only the technical aspects of each segment and its connecting lines but also the sequence of the segments and their impact on passenger comfort. According to the "Design Standards for High-Speed Maglev Transportation" (CJJ/T 310—2021) section 5.4, the maximum acceleration value determined by the track must not exceed the limits specified in Table 1.

Similarly, the maximum acceleration change rate (impact value) must not exceed the limits specified in Table 2.

Table 1 The limit of the maximum acceleration value [1] (Unit: m/s^2)

Item	General conditions	Tough conditions
Longitudinal, a_x	1.0	1.5
Lateral (Transverse), a_y (Control point of line)	1.0 (outward) 0.5 (inward)	1.25 (outward) 0.5 (inward)
Vertical, a_z (Control point of line)	0.5 (convex curve) 1.0 (concave curve)	0.6 (convex curve) 1.2 (concave curve)

Table 2 The limit of the maximum impact value [1] (Unit: m/s^3)

Item	General conditions	Tough conditions
Longitudinal, $\dot{a}_x$	0.5	1.0
Transverse, $\dot{a}_y$ (Control point of line)	0.5	1.0
Vertical, $\dot{a}_z$ (Control point of line)	0.5	1.0

For general span bridges, the track beam elevated system, built according to the designed vertical and horizontal curves, does not undergo significant changes in alignment and elevation during operation. The elastoplastic deformation of the substructure of the elevated system is minimal and localized. However, for long-span bridges, track beams behave differently. Due to the effects of temperature and train loads, long-span bridges will experience large-scale and large-magnitude deformations (compared to elevated structures), leading to changes in deck alignment. This is particularly important because it affects key parameters related to train travel comfort, such as the curvature radius, curve length, and curve type (concave or convex) of the vertical and horizontal curves [2,3]. Based on the existing technical standards for the minimum radii of vertical and horizontal curves and their calculation principles and methods, the post-deformation deck alignment is calculated to determine whether the deformed bridge track beams meet the comfort requirements.

Since comfort is perceived through the passengers' bodily reactions while riding the train, only the acceleration and impact values derived from the instantaneous deck alignment within the maglev train length are meaningful. Therefore, this paper selects three representative positions under the train load—the train head, the train center (center of the train length), and the train tail—to calculate the vertical acceleration and impact, taking the most adverse envelope as the control value. Due to space limitations, this paper focuses on the calculation and analysis of the most critical vertical comfort.

2 Technical Index for the Vertical Comfort of Bridge Track Beams

2.1 Calculation and Loading Method for the Vertical Acceleration of Bridge Track Beams

For bridge track beams, if a circle is determined using the coordinates of three points on two adjacent track beams, the vertical angle between adjacent track beams can be converted into the vertical circular curvature at the midpoint. Taking a 12.384 m [4] track beam as an example, the calculation diagram is shown in Figure 1.

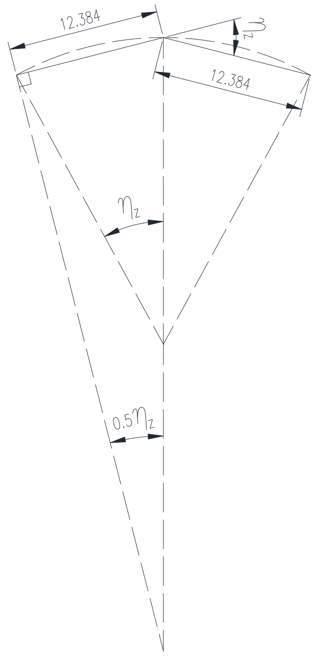


Figure 1 Fitting vertical curve of track beam (Unit: m)

$$2R_z \cdot sin(\eta_z/2) = 12.384\ m \quad (1)$$

$$R_z = \frac{1}{k_z} = \frac{12.384}{2 \cdot sin(\eta_z/2)} \quad (2)$$

Substituting centrifugal force formula $R_z = v^2/a_z$ into equation (2), it can be obtained:

$$\eta_z = 2 \cdot \arcsin\left(\frac{12.384\ a_z}{2\ v^2}\right) \quad (3)$$

Where:

R_z: Vertical curvature radius of the bridge deck caused by the bridge's vertical curve and various variable loads.

a_z: Vertical acceleration of the maglev train.

v: Velocity of the maglev train.

k_z: Vertical curvature of the bridge deck corresponding to R_z.

η_z: Vertical angle between two adjacent track beams on the bridge.

It should be noted that the angle between the track beams in the above equation is only the follow-up angle (the angle of the track beam with the deformation of the bridge) and does not include the inherent angle of the track beams.

Based on the above equation and the permissible maximum acceleration indicators specified in Table 1, the requirements for the track beam angle on the bridge deck at a specified running velocity can be obtained.

Within the length of the bridge, the maglev train load is applied sequentially every 12.384 m to calculate the vertical angles of the track beams. The maximum and minimum vertical angles at the positions of the train head, center, and tail are taken to form the vertical angle envelope diagram of the track beams, which is then converted into the vertical curvature envelope diagram of the bridge deck, thereby obtaining the vertical acceleration value at the specified train velocity.

2.2 *Calculation and Loading Method for the Vertical Impact on Bridge Track Beams*

In the operation of long-span maglev bridges, the bridge deck curve is always changing, so it is impossible to analyze the impact using a static vertical curve. Therefore, the vertical acceleration of a fixed point on the train as it passes through the endpoints of each track beam must be used for analysis.

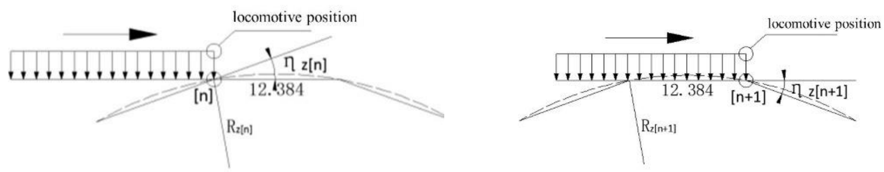
The vertical impact is the time rate of change in the vertical acceleration, which is related to the curvature, and the curvature is related to the angle between adjacent track beams. Therefore, the time rate of change of acceleration can be represented by the difference in angles between two adjacent track beams, meaning that the impact at a specific point on the train can be represented by the difference in angles as it passes through the endpoints of a track beam.

2.2.1 Vertical Impact for a Single Track

Taking a 12.384 m track beam as the calculation length, the vertical acceleration is calculated every 12.384 m of train travel. The difference between the before and after accelerations, Δa_z, is calculated for the train head, center (center point of the train length), and tail.

For example, at the train head (locomotive) position, the vertical acceleration calculation before and after traveling 12.384 m is shown in Figure 2.

 https://doi.org/10.59238/j.pt.2024.02.003



a) Train reaches the [n] node position b) Train reaches the [n+1] node position

Figure 2 Impact calculation diagram for the train head (locomotive) position

In the figure, [n] represents any node position on the bridge, [n+1] represents the position 12.384 m after node [n], and $\eta_{z[n]}$ and $\eta_{z[n+1]}$ are the vertical angles at nodes [n] and [n+1], respectively. The vertical acceleration values before and after train movement are:

$$\begin{cases} a_{z[n]} = \dfrac{v^2}{R_{z[n]}} \\ a_{z[n+1]} = \dfrac{v^2}{R_{z[n+1]}} \end{cases} \tag{4}$$

From equation (2), it can be obtained that:

$$\begin{cases} R_{z[n]} = \dfrac{1}{K_{z[n]}} = \dfrac{12.384}{2 \cdot sin(\eta_{z[n]}/2)} \\ R_{z[n+1]} = \dfrac{1}{K_{z[n+1]}} = \dfrac{12.384}{2 \cdot sin(\eta_{z[n+1]}/2)} \end{cases} \tag{5}$$

The time required for the train to pass through the 12.384 m track beam at velocity v is t; then, the change in the vertical acceleration Δa_z is:

$$\Delta a_z = a_{z[n+1]} - a_{z[n]} = \frac{2 \cdot v^2 \cdot sin(\eta_{z[n+1]}/2)}{12.384} - \frac{2 \cdot v^2 \cdot sin(\eta_{z[n]}/2)}{12.384} \tag{6}$$

Assuming that the beam end angles are small, the impact can be approximated by:

$$\dot{a}_z = \frac{a_{z[n+1]} - a_{z[n]}}{t} \approx \frac{2 \cdot v^3}{12.384^2} \cdot \frac{\eta_{z[n+1]} - \eta_{z[n]}}{2} = \frac{v^3}{12.384^2} \cdot (\eta_{z[n+1]} - \eta_{z[n]}) \tag{7}$$

Here, $\eta_{z[n+1]} - \eta_{z[n]}$ is the vertical angle difference between adjacent track beams. Equation (7) establishes the relationships among the vertical impact $\dot{a}_z$, vertical angle difference $\eta_{z[n+1]} - \eta_{z[n]}$ of the track beams, and train velocity v. Based on Table 2, the limits for vertical impact are as follows:

Genera Conditions: $\dot{a} \le 0.5\ m/s^3 \rightarrow v \le \sqrt[3]{0.5 \cdot \frac{12.384^2}{|\eta_{z[n+1]} - \eta_{z[n]}|}}$

Tough Conditions: $\dot{a} \le 1.0\ m/s^3 \rightarrow v \le \sqrt[3]{1.0 \cdot \frac{12.384^2}{|\eta_{z[n+1]} - \eta_{z[n]}|}}$

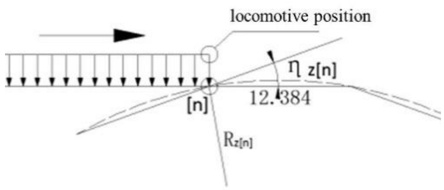
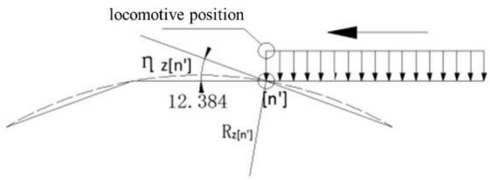
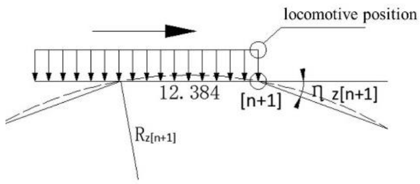
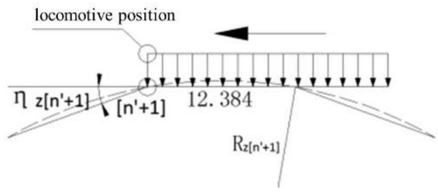
After calculating the angle differences for various train positions (train head, center, and tail), the maximum and minimum angle differences are taken to form the angle difference envelope diagram for a single track. Using the conversion formula between the angle difference and impact, the impact value $\dot{a}_z$ is calculated.

2.2.2 Vertical Impact for Dual-Track Trains

When considering the case of dual-track maglev trains passing each other on the bridge, the influence of a train on the other track must be considered on the basis of a single-track train. Assuming that trains on both tracks travel at the same velocity v and taking a track beam length of 12.384 m, the vertical acceleration is calculated every 12.384 m of train travel. The difference between the before and after accelerations, Δa_z, is computed. Typically, calculations are performed for three representative positions: the train head, center (center point of the train length), and tail.

For example, at the left-side train head (locomotive) position, the vertical acceleration calculation before and after traveling 12.384 m is shown in Table 3.

Table 3 Vertical impact calculation method for dual-track train heads

Time	Left-side train	Right-side train
T_0		
$T_0 + t$ Where, $t = 12.384/v$		

At time T_0, the left-side train head reaches node [n], and the right-side train reaches any position node [n′] (with trains traveling in opposite directions). At this moment, the vertical angle at node [n] for the left-side train head position is $\eta_{z[n]}$. As the trains continue to travel and the left-side train head reaches node [n+1] and the right-side train reaches node [n′+1] at time $T_0 + t$, the vertical angle at node [n+1] for the left-side train head position is $\eta_{z[n+1]}$. Thus, the vertical angle difference at the left-side train head position from time T_0 to $T_0 + t$ is $\eta_{z[n+1]} - \eta_{z[n]}$.

Due to the random relative positions of dual-track trains passing each other on the bridge, the right-side train could be at any position on the bridge when the left-side train head reaches node [n]. Therefore, all possible scenarios need to be traversed to calculate the vertical angle at the left-side train head position. After the left-side train travels 12.384 m each, the vertical angle at the left-side train head position is recalculated. The difference in angles before and after travel is then calculated, and the maximum and minimum angle differences are taken as the most adverse dual-track vertical angle differences for the left-side train head moving from node [n] to node [n+1].

The calculation method for the vertical angle difference at the train center and tail positions is the same. Thus, the vertical angle differences for the train head, center, and tail moving from node [n] to node [n+1] are obtained. The maximum and minimum values among these three positions are taken to determine the most adverse dual-track vertical angle difference between nodes [n] and [n+1]. This process is repeated to obtain the dual-track vertical angle differences for any node position across the entire bridge.

3 Example of a Bridge Scheme and Comfort Index

The technical scheme for the double-tower cable-stayed bridge has a span layout of 210.528 + 458.208 + 222.912 = 891.648 m. Three auxiliary piers and one end pier are set on each side span, with the left-side pier distances from left to right being (3×49.536 + 61.902) m and the right-side pier distances from left to right being (2×61.920 + 2×49.536) m. The bridge towers are made of concrete, arranged in a herringbone layout, with a height of 155 m above the foundation base and 109 m above the bridge deck. Each tower has 2×18 stayed cables on each side, with a cable spacing of 12.384 m on the girder and 3 m on the tower. The girders are special steel trusses, with the lower chord being an integral steel–concrete composite box girder [5]. The track beam length modulus on the bridge deck is 12.384 m, and the track beams are arranged in a simply supported manner on the bridge deck. A total of 72 track beams are used for the single-line track on one side of the bridge, numbered from 1# to 72# from left to right. The overall layout of the bridge is shown in Figure 3.

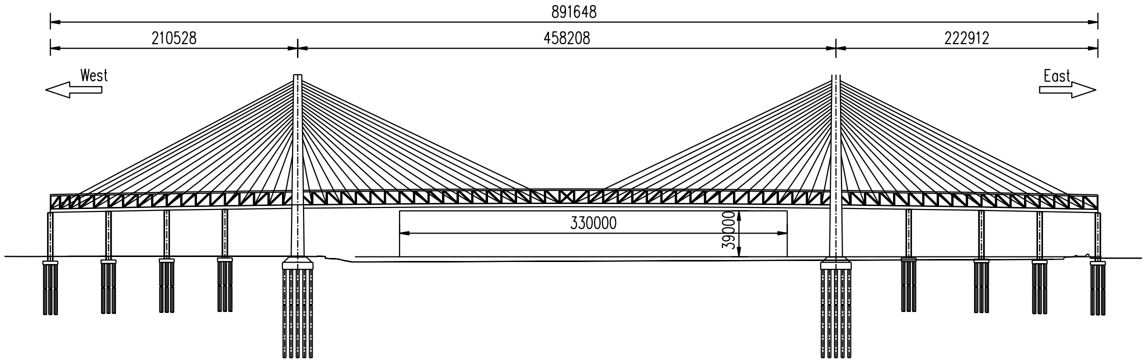


Figure 3 Elevation layout of the 458 m main span double-tower cable-stayed bridge (Unit: mm)

3.1 Acceleration Index

When the track beams on the bridge deck are in operation with no vertical curves (the vertical curve during bridge completion is not considered, calculated as a flat bridge), the envelope of the vertical curvature under the standard load combination is shown in the following figure.

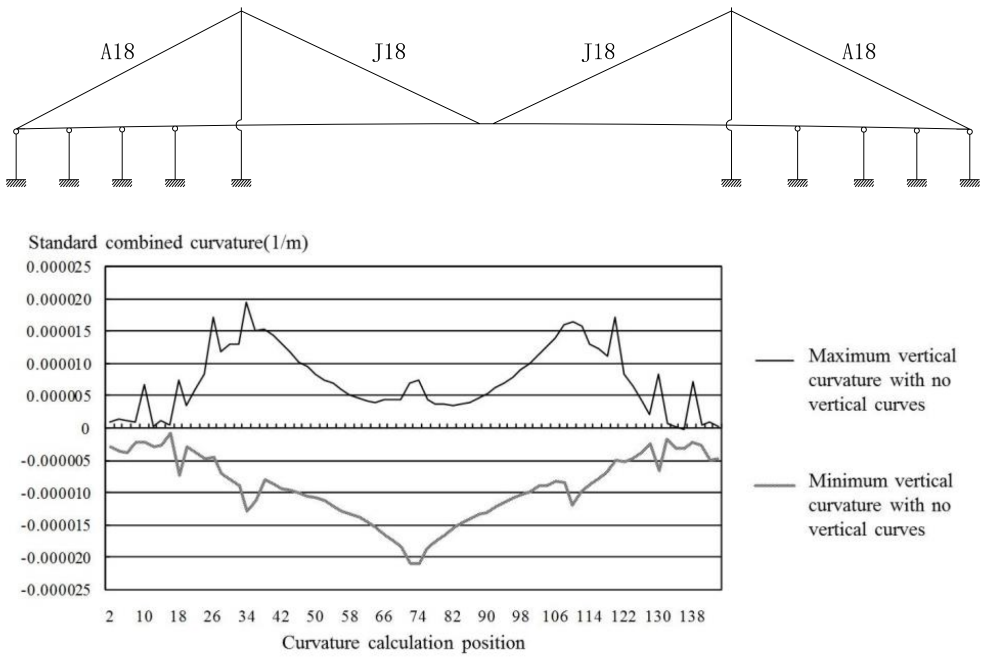


Figure 4 Envelope of the vertical curvature of track beams under standard combinations (dual-track) (Unit: m^{-1})

When different vertical curves are set for the bridge, the curvature can be calculated by superimposing it with the envelope of curvature from the standard load combination. Then, the vertical acceleration at a specific velocity can be calculated. The specific calculation results are as follows.

Table 4 Conversion table of the curvature and acceleration under different vertical curves

Item	Curvature under standard combination (m^{-1})	Curvature after bridge construction (m^{-1})	Curvature radius after bridge construction (m)	Superposition curvature (m^{-1})	Superposition curvature radius (m)	Normal acceleration (m/s^2)	
						350 km/h	400 km/h
MAX	1.95E-05	5.00E-05	20,000	6.95E-05	14,397	0.66	0.86
		3.33E-05	30,000	5.28E-05	18,954	0.5	0.65
		2.50E-05	40,000	4.45E-05	22,492	0.42	0.55
		2.00E-05	50,000	3.95E-05	25,342	0.37	0.49
		0.00E+00	∞	1.95E-05	51,386	0.18	0.24
MIN	-2.10E-05	5.00E-05	20,000	2.90E-05	34,477	0.27	0.36
		3.33E-05	30,000	1.23E-05	81,267	0.12	0.15
		2.50E-05	40,000	4.01E-06	249,677	0.04	0.05
		2.00E-05	50,000	-9.95E-07	-1,005,195	-0.01	-0.01
		0.00E+00	∞	-2.10E-05	-47,631	-0.2	-0.26

Comparing Table 1 and Table 4, it is evident that when the vertical curve radius during bridge completion is 30,000 m or more, it can meet the requirements for vertical acceleration under general conditions for a travel velocity of 350 km/h. However, if the travel velocity is 400 km/h and aims to meet the requirements for vertical acceleration under general conditions, the vertical curve radius during bridge completion needs to be greater than 50,000 m. Therefore, in terms of the comfort index, the vertical acceleration requirements for the design train travel velocity can be met by adjusting the vertical curve radius during bridge completion.

3.2 Impact Index

When the track beams on the bridge deck are in operation with no vertical curves (the vertical curve during bridge completion is not considered, calculated as a flat bridge), and operating with dual-track (single-track operation examples are omitted for brevity), the envelope of the track beam's vertical angle difference under the standard load combination (pseudo long-term load + maglev train load) is shown in Figure 5.

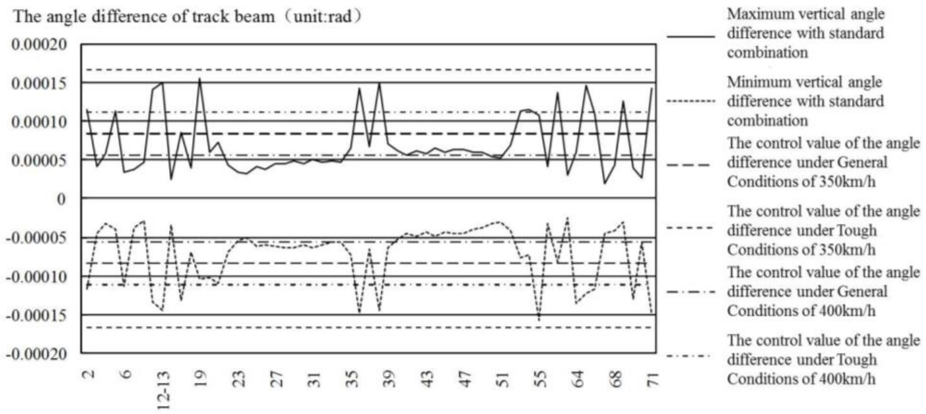


Figure 5 Envelope of the vertical angle difference of track beams under the standard combinations (dual-track) (Unit: rad)

According to the relationship between the angle difference and impact described in Equation (7), the corresponding impact values can be obtained, as shown in Table 5.

Table 5 Vertical impact of track beams under standard combinations (dual-track) (Unit: m/s^3)

Velocity	350 km/h	400 km/h
Vertical Impact	0.9	1.345

Note: The maximum angle difference is 0.00015 at the auxiliary piers

From the table above, it can be observed that at a travel velocity of 350 km/h, the maximum impact when the train operates on double tracks is 0.9 m/s^3, which meets the allowable impact value under tough conditions.

4 Conclusions

According to the design technical standards for high-speed maglev lines, a technical index for track beams on long-span bridges has been introduced, mainly including allowable values for acceleration and impact. Based on the calculation theory and methods for acceleration and impact on high-speed maglev lines and after studying the mechanism of deformation in track beams and bridge deck alignments on long-span bridges, formulas for calculating the angle, angle difference, vertical acceleration, and vertical impact caused by the maglev line at different train velocities were developed. This study proposes a method for calculating the comfort index envelope values for the train head (locomotive), middle, and tail by using the track beam length as the loading step and verifies the comfort of the entire bridge range. Furthermore, this paper presents practical methods for calculating the comfort of high-speed maglev long-span bridges considering the different scenarios of single-track and dual-track train operations. Finally, by considering a 458 m main span high-speed maglev long-span cable-stayed bridge under the most adverse load combination, the vertical acceleration and impact values of the track beams on the bridge deck are calculated, contributing to beneficial exploration in the design of long-span high-speed maglev bridges.

Conflict of Interest: All authors disclosed no relevant relationships.

Date Availability Statement: The data that support the findings of this study are available from the corresponding author, Zhang, upon reasonable request.

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https://doi.org/10.59238/j.pt.2024.02.003