Permanent deformation limits of long-span track cable-stayed bridges based on service performance analysis

Xiaogang Li¹,², Mangmang Gao³, Jianting Zhou¹, Xiaohu Chen² and Shulin Tan²

Abstract
With the continuous service time, long-span track cable-stayed bridges are inevitably affected by material time-varying and fatigue load periodic effects, and permanent down-deflection phenomenon is inevitable. The down-deflection of the main girder would lead to the irregularity of bridge deck, which would directly affect the running comfort and safety of the train. The permanent deformation control value should be set reasonably. In this paper, based on the research and analysis of domestic and foreign codes and standards, the evaluation criteria for the service performance of long-span track cable-stayed bridges was determined, aiming at the problem of deformation classification control in the service stage of the long-span track cable-stayed bridge. The limit of permanent deformation of main girder was analyzed, and the decision processing was put forward and applied to an engineering example. According to the vehicle-bridge coupling vibration analysis, the maximum vertical deformation safety control value in the service stage is L/400, and the pre-warning control value is L/500. With the non-permanent deformation value deducted, the permanent deformation safety control value is L/400-β, and the pre-warning control value is L/500-β. The rationality of the grading control limit of permanent deformation based on service performance analysis has been verified by application analysis.

Keywords
Bridge engineering, track cable-stayed bridge, permanent deformation, service performance, vehicle-bridge coupling

Introduction
With the rapid development of urban rail transit, long-span rail bridges have been applied more and more widely. Especially, long-span track cable-stayed bridges with beautiful structure, large span capacity, and strong adaptability are widely favored as the main bridge type of long-span rail bridges.¹,² However, in the process of bridge service life, permanent deformation cannot be recovered under the influence of material characteristic variation, mechanical behavior evolution, repeated load action, and other factors.³ Ballastless track is commonly used in urban rail transit bridges, and the non-restorable deformation of main girder can only be maintained through the adjustment of fasteners. However, the adjustable range of fasteners is extremely limited, and it is difficult to guarantee the smoothness of ballastless track, which seriously affects the comfort of train running and may even endanger the safety of train running.

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Therefore, scholars at home and abroad have made in-depth research on the relationship between bridge alignment change and train running performance analysis by means of vehicle-bridge coupling numerical simulation and experimental analysis. 4–7 For example, Gou et al.8,9 analyzed the dynamic performance of railway bridges and studied the influence of bridge deformation on the geometric alignment of high-speed railway tracks. Esveld10 studied the relationship between train dynamic response and track irregularity and obtained the transfer function of track irregularity to vibration acceleration. Zhai et al.11 analyzed the influence of additional structural deformation on the train-bridge interaction and explored the influence law of track irregularity on train dynamic response. Also, there are many relevant works about cable-stayed bridge dynamics.12,13 But studies on permanent deformation limits based on service performance analysis are still insufficient for long-span track cable-stayed bridges which are the throat of urban rail. Therefore, in this paper, the vehicle-bridge coupling vibration analysis technology is adopted for the service stage. Based on deducing the maximum vertical deformation control limit in the service stage, a permanent deformation hierarchical control system based on safety control and pre-warning control is proposed, which provides an important basis for guiding the effective control of bridge alignment.

**Evaluation of service performance**

Permanent deformation of main girder is an important embodiment of structural stiffness, which directly affects the service performance of bridge under train, temperature, and other loads. The service performance is evaluated by two levels: running comfort and running safety.

Running comfort is an important index to measure the service performance of a train and the main basis for evaluating passenger comfort. It reflects the influence of train vibration on passenger comfort. Running comfort is the upper limit of deformation value of a bridge in normal use. Running safety mainly involves derailment, capsizing, and other problems endangering running safety. It is a deformation limit index that the bridge structure must meet in order to ensure the safe passage of the train at the specified speed. Running comfort is characterized by index vertical acceleration, lateral acceleration, and Sperling index, while running safety is characterized by index derailment coefficient, wheel weight loss rate, and wheel lateral force.

Due to the lack of indicators and standards to determine the running comfort and safety of long-span track cable-stayed bridges in existing domestic and foreign codes, service performance evaluation standards were determined through comparative analysis according to relevant domestic and foreign regulations14–18 as shown in Table 1.

**Analysis and decision of permanent deformation limitation**

During the service stage of long-span track cable-stayed bridges, the comfort and safety of trains must be satisfied. Therefore, the permanent deformation limit of the structure is one of the design control conditions. In the long-term use, the bridge will be subject to the coupling effect of train, temperature, shrinkage, and creep and other factors. The existing design specifications clearly specify the strength under the action of various load combinations,14 but have no uniform limit for the stiffness under the action of load combinations. Thus, it is necessary to determine the permanent deformation limit of long-span track cable-stayed bridges based on service performance analysis. The aim is to provide the effective guidance for alignment adjustment and control of long-span track cable-stayed bridges during service life.

| Evaluating indicator                  | Limited standard          |
|---------------------------------------|---------------------------|
| Comfort                               |                           |
| Vertical acceleration (m/s²)           | 1.5                       |
| Transverse acceleration (m/s²)        | 1.8                       |
| Sperling index                        | <2.5 (Excellent)          |
|                                       | (2.5, 2.75) (Good)        |
|                                       | (2.75, 3) (Qualified)     |
| Safety                                |                           |
| Derailment coefficient                | 0.8                       |
| Wheel load reduction rate             | 0.6                       |
| Transverse force of wheelset (kN)     | 80                        |

Table 1. Evaluation criteria of service performance.
Permanent deformation of long-span track cable-stayed bridges is closely related to vertical acceleration of train. So, the minimum curvature radius can be determined by controlling limit of vertical acceleration of train, and the maximum vertical deformation control value can be put forward by superposing initial deformation and deformation under train load. The specific process is shown in Figure 1. Among them, the maximum vertical deformation control limit is divided into two levels: safety control and pre-warning control. The safety control value is the control index that the bridge must meet when the train passes safely. The pre-warning control value is the limit value of bridge deformation under normal service condition. In the service stage, if the warning control value is exceeded, the bridge should be carefully checked for any diseases, such as serious down-deflection of the main girder and excessive deformation. The abnormal incentives of the train, such as poor vehicle conditions and eccentric loading, should be investigated. Also, necessary measures should be taken to deal with them in time.

Due to the comprehensive influence of shrinkage and creep of concrete, loss of prestressing, damage of stay cables, material variation, uneven settlement of the foundation, repeated action of fatigue load, concrete cracking, and other factors, the alignment of long-span track bridges has been continuously evolving. This change of alignment has non-recoverable characteristics, that is, permanent deformation. The non-permanent deformation is the influence of time-varying effects of load such as train and temperature. Thus, the vertical deformation of the main girder is formed by the accumulation of permanent deformation and non-permanent deformation. The limit value of the non-permanent deformation is denoted as $\beta$, which can be determined according to finite element analysis and combined with the feedback of safety monitoring information.\textsuperscript{19,20} After deducting the non-permanent deformation value $\beta$, the grading control limit of permanent deformation is obtained.

**Application analysis**

**Engineering outline**

The Caijia Jialing River Bridge is a special track bridge on Chongqing Rail Transit Line 6, starting from the line pile number K38 + 153.149 and ending at K39 + 403.149. The main bridge structure is a cable-stayed concrete bridge with twin towers and double cable planes where tower-girder consolidation is adopted. The layout of the span is 60 m + 135 m + 250 m + 135 m + 60 m, and the transverse layout is 1.5 m (cable area) + 1.4 m (maintenance lane) + 4.6 m (carriageway) + 4.6 m (carriageway) + 1.4 m (maintenance lane) + 1.5 m (cable area). The shape of the tower is diamond, and the auxiliary piers are rectangular cross-sectional hollow piers. Cables adopt the steel strand and the standard strength $f_{pk} = 1860$ MPa, which are protected by a high density polyethylene tube. The single-cell box girder with a constant height is adopted as the main girder. The concrete grade is C55. The width,

![Figure 1. Analysis flow of permanent deformation control limit for long-span track cable-stayed bridges.](image-url)
height, and the length of the girder are 15, 3.5, and 8 m, respectively. The cross beam is set at the stay cable, and the anchor blocks are set at both ends of the cross beam. The layout of the main bridge is shown in Figure 2.

**Analysis model**

The vehicle-bridge coupling vibration analysis of the long-span track concrete cable-stayed bridge was carried out. The train and bridge were regarded as two subsystems of motion equation. The train and bridge analysis models were established, respectively, to conduct vibration analysis independently. The geometric displacement relation and wheel/rail interaction force at the contact point of wheel/rail were constructed. Then, the track irregularity was simulated according to the line spectrum of American class 6. The train, bogies, and wheelsets all vibrate slightly and were regarded as rigid bodies. Elastic components and damping components were connected between rigid bodies. Springs are linear, damping was calculated as viscous damping, and creep force was calculated as linear. Along the vertical direction, the vertical displacement of wheelset and rail is the same. The nodding motion, wheel-to-wheel roll, and shaking motion of the frame are ignored. At the same time, the spatial vibration model was established without considering the influence of train on bridge vibration and running speed, as was shown in Figure 3.

MIDAS/CIVIL, a professional program of bridge structure analysis, was used to establish the finite element model. The model is divided into 819 nodes, 583 beam elements, and 112 truss elements, in which the main girder, bridge tower, and pier are simulated by beam elements, and the stayed cable is simulated by truss elements. Furthermore, the boundary conditions are treated according to the actual situation. The bottom of bridge tower and pier is completely consolidated. The bridge tower, the cable, and the girder are rigidly connected at the boundary. Also, the CEB-FIP (1990) model was used for shrinkage and creep of concrete. Based on these, the finite element model of bridge is established as shown in Figure 4. Based on D’Alembert and Hamilton’s

![Figure 2. The layout of the main bridge (unit: cm).](image-url)
principle, the motion equations of the train and the bridge were established, respectively, as shown in equations (1) and (2):  

\[
M_v \ddot{u}_v + C_v \dot{u}_v + K_v u_v = F_{bv} \\
M_b \ddot{u}_b + C_b \dot{u}_b + K_b u_b = F_{bh}
\]  

where \( M \) is the system quality matrix, \( C \) is system damping matrix, \( K \) is system stiffness matrix, \( \ddot{u} \) is acceleration, \( \dot{u} \) is speed, \( u \) is displacement, the subscripts \( b \) and \( v \) mean bridge and train, respectively, and \( F_{bh} \) and \( F_{bv} \) are wheel-rail interaction force matrices.

Track irregularity models were shown in equations (3) to (5).

Vertical irregularity

\[
S_v(\Omega) = \frac{K_A \Omega^2}{\Omega^2 (\Omega_e^2 + \Omega^2)}
\]
Direction irregularity

\[
S_a(\Omega) = \frac{KA_v\Omega_c^2}{\Omega^2(\Omega^2 + \Omega_c^2)}
\]

(4)

Horizontal and gauge irregularities

\[
S_c(\Omega) = S_g(\Omega) = \frac{4KA_v\Omega_c^2}{(\Omega^2 + \Omega_c^2)(\Omega^2 + \Omega_s^2)}
\]

(5)

where \(S(\Omega)\) is the power spectral density of track irregularity, \(\Omega\) is the spatial frequency, \(\Omega_c, \Omega_s\) are cut frequencies, \(A_v, A_u\) are roughness coefficients, and \(K\) is equal to 0.25.

**Natural vibration characteristics analysis**

Natural vibration characteristics one of the important dynamic response features of cable-stayed bridges. Based on the finite element analysis model of Caijia Jialing River Bridge, the natural vibration characteristics are analyzed. The first and second-order vibration mode shapes are shown in Figure 5. The natural vibration characteristic parameters of the bridge were obtained according to the field ambient vibration test, and the comparison between measured and calculated is shown in Table 2. It can be seen that the modal parameters measured by the ambient vibration test are in good agreement with those calculated by finite element method. The finite element model could serve as the baseline model for a further dynamic response analysis of the bridge.

![Figure 5](image)

Figure 5. The first and second-order vibration mode shapes obtained from finite element analysis.
Permanent deformation analysis

1. Determination of train vertical acceleration control limit \( a \), vertical acceleration \( a_2 \) corresponding to bridge deck initial deformation, and minimum curvature radius \( r \)

According to the service performance evaluation standard, the control limit of vertical acceleration of the train is set as \( a = 1.5 \text{ m/s}^2 \). When there is no initial deformation of the bridge deck, the coupling vibration analysis of the vehicle bridge was carried out to obtain the maximum value of the running comfort and running safety indexes of the train, which is compared with the control limit. The results are shown in Table 3.

According to Table 3, the running safety index, vertical acceleration index, and transverse acceleration index are all within the control limit. And the Sperling index is in an “excellent” and “good” state, thus meeting the service performance requirements. Therefore, when \( a_1 = 1.10 \text{ m/s}^2 \), the design speed value is 100 km/h, and the minimum radius of curvature value is 1929 m.

2. The deformation curve of bridge deck is analyzed to determine the most unfavorable loading condition.

First, the vertical deformation of the bridge deck was calculated under two working conditions of temperature increasing 30°C and decreasing 25°C as shown in Figure 6. Under the heating condition, the maximum vertical deformation of the bridge deck is 35.816 mm, and the minimum curvature radius is 18,761.1 m. Under the cooling condition, the maximum vertical deformation of the bridge deck is 29.846 mm, and the minimum curvature radius is 22,987.9 m; all of which are located at the position of auxiliary pier. However, the vertical deformation of the mid-span generated by the six grouped railway trains of a single line is 81.822 mm, which verifies that it is feasible to consider the deformation under temperature load as the initial deformation.

Then, according to the uniform load of the whole bridge, the mid-span, and both the mid-span and the sub-side span, the vertical deformations of the bridge deck were calculated with the load of 100 kN/m. The results are shown in Figures 7 and 8.

Thus, by calculating the deflection curves of the bridge deck under the uniform load of the whole bridge, the mid-span, and both the mid-span and the sub-side span, it can be analyzed which condition has the minimum curvature radius under the same ratio of deflection to span. The results show that when the ratio of deflection to span is \( L/2000 \), the condition of mid-span uniform load is the most disadvantageous. The minimum

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**Table 2.** The comparison between measured and calculated of natural vibration characteristic parameters.

| Mode number | Measured (Hz) | Calculated (Hz) | Mode description                                      |
|-------------|---------------|-----------------|-------------------------------------------------------|
| 1           | 0.482         | 0.456           | Transverse vibration of bridge towers, transverse bending of the main girder |
| 2           | 0.728         | 0.667           | Antisymmetric lateral vibration of bridge towers, transverse bending of the main girder |
| 3           | 0.800         | 0.704           | Longitudinal drift of the main girder                 |
| 4           | 1.071         | 0.923           | First-order symmetric vertical bending                 |
| 5           | 1.214         | 1.012           | First-order symmetric transverse bending               |

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**Table 3.** Train response calculation maximum value and analysis evaluation.

| Conditions | Speed (km/h) | Derailment coefficient | Wheel load reduction rate | Transverse force of wheelset (kN) | Vertical acceleration (m/s²) | Transverse acceleration (m/s²) | Sperling index | Vertical | Transverse |
|------------|--------------|------------------------|--------------------------|----------------------------------|-----------------------------|-------------------------------|----------------|----------|-----------|
| No initial deformation | 60 | 0.19 | Yes | 0.26 | Yes | 26.68 | Yes | 0.75 | Yes | 0.68 | Yes | 2.16 | Excellent | 1.97 | Excellent |
| 80 | 0.21 | Yes | 0.28 | Yes | 29.16 | Yes | 0.93 | Yes | 0.93 | Yes | 2.35 | Excellent | 2.32 | Excellent |
| 100 | 0.24 | Yes | 0.30 | Yes | 29.73 | Yes | 1.10 | Yes | 1.18 | Yes | 2.51 | Good | 2.52 | Good |

C: calculation; E: evaluation.
The curvature radius is only 367 m, and the minimum curvature radius is 609 m under the condition of full-bridge uniform load. And the minimum curvature radius is 560 m under the condition of mid-span and sub-span uniform load.

3. The initial deformation curve of bridge deck is determined, and the maximum safety control value of vertical deformation is initially determined.

The vertical deformation curve of the bridge deck corresponding to the load condition with uniform distribution of the mid-span is taken as the reference. The amplitude is adjusted and superimposed with the bridge deck deformed curve under the cooling condition to form the initial deformation curve of the bridge deck. When the minimum curvature radius of the curve reaches the limit, the bridge deck deformation limit is preliminarily determined. Taking the train load into account, the vehicle-bridge coupling vibration analysis was conducted. Based on the principle that the vertical acceleration of the train reached or approached the control limit, the maximum vertical deformation safety control value was preliminarily determined to be 400/L.

4. The rationality of the maximum vertical deformation safety control value is verified, and the maximum vertical deformation safety control value and pre-warning control value are determined.

Under the initial deformation and train load, when the maximum vertical deformation safety control value is L/400, the vehicle-bridge coupling vibration analysis is conducted according to the working condition in Table 4. The dynamic response index results of train running are shown in Table 5.

According to the analysis results of vehicular and bridge coupling vibration, under the three working conditions, the maximum derailment coefficient is 0.624. The maximum load reduction rate of the wheel is 0.547. The maximum lateral force of the wheelset is 34.540 kN. And the maximum lateral acceleration is 1.179 m/s², which are all within the control limit and meet the requirements. The maximum vertical acceleration is 1.475 m/s², close to the control limit of 1.5 m/s². And the Sperling index has reached 2.911 (vertical), which is about to reach the...
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**Figure 8.** The position schematic diagram of minimum curvature radius of vertical deformation under different load conditions.

**Table 4.** Calculate service conditions and instructions.

| Conditions | Load combination |
|------------|------------------|
| Condition 1 | The load is uniformly distributed across the mid-span, and the temperature decrease by 25°C |
| Condition 2 | The load is uniformly distributed throughout the bridge, and the temperature decrease by 25°C |
| Condition 3 | The load is uniformly distributed across the mid-span and the side span, and the temperature decrease by 25°C |
Table 5. Maximum value of train dynamic response calculation and evaluation when the maximum vertical deformation safety control value is 400/L.

| Conditions | Speed (km/h) | Derailment coefficient | Wheel load reduction rate | Transverse force of wheelset (kN) | Vertical acceleration (m/s²) | Transverse acceleration (m/s²) | Sperling index |
|------------|--------------|-------------------------|---------------------------|----------------------------------|----------------------------|-------------------------------|---------------|
|            | C  E         | C  E                    | C  E                      | C  E                             | C  E                       | C  E                           | C  E          |
| Condition 1| 60           | 0.54                    | Yes                       | 0.18                             | Yes                        | Yes                            | 34.54         |
|            | 80           | 0.47                    | Yes                       | 0.19                             | Yes                        | Yes                            | 30.00         |
|            | 100          | 0.62                    | Yes                       | 0.21                             | Yes                        | Yes                            | 32.58         |
| Condition 2| 60           | 0.50                    | Yes                       | 0.44                             | Yes                        | Yes                            | 25.29         |
|            | 80           | 0.44                    | Yes                       | 0.41                             | Yes                        | Yes                            | 28.67         |
|            | 100          | 0.59                    | Yes                       | 0.55                             | Yes                        | Yes                            | 30.00         |
| Condition 3| 60           | 0.46                    | Yes                       | 0.41                             | Yes                        | Yes                            | 25.30         |
|            | 80           | 0.44                    | Yes                       | 0.39                             | Yes                        | Yes                            | 28.68         |
|            | 100          | 0.53                    | Yes                       | 0.51                             | Yes                        | Yes                            | 30.01         |

“qualified” limit. The service performance is about to be in the “unqualified” state. Therefore, the maximum vertical deformation safety control value is reasonable, including the safety factor 1.25, and the maximum vertical deformation pre-warning control value is L/500.

5. The permanent deformation safety control value and pre-warning control value are determined.

For the background project, it is assumed that the actual running train reaches the design load, and the actual temperature is decreased by 20°C. Since the water pressure and wind load mainly affect the transverse deformation of the main girder, the vertical deformation analysis will not consider such factors. According to the safety monitoring and regular check, the structure in the service stage does not appear serious events, such as broken cable, significant foundation settlement, etc. According to the finite element analysis, the vertical deformation of the mid-span of the structure is 159 mm under the static and active load of the train. The deformation of the main span is 22 mm when the temperature is decreased by 20°C. Therefore, when the temperature effect is included, the permanent deformation pre-warning control value is 319 mm (L/500 – β = 500 mm – (159 + 22) mm = 319 mm), and the permanent deformation safety control value is 444 mm (L/400 – β = 625 mm – (159 + 22) mm = 444 mm).

In conclusion, the rationality of the hierarchical control limit of permanent deformation for long-span track concrete cable-stayed bridges has been verified through application analysis. The safety control and pre-warning control based on permanent deformation can effectively guide the alignment adjustment and control of the structure in the service stage.

Conclusion and suggestion

In this paper, the lack of indicators and standards to determine the running comfort and running safety of train of the long-span track cable-stayed bridge in existing domestic and foreign codes is found. The deformation limits of the design specification cannot be directly applied to the performance analysis of the long-span track cable-stayed bridge in service stage, and there is no basis for effective control of bridge alignment. To address this deficiency, the running comfort and safety evaluation indexes of long-span track cable-stayed bridges were clarified. The analysis flow of permanent deformation control limit for long-span track cable-stayed bridge based on the vehicle-bridge coupling vibration analysis was presented and applied to engineering examples. Some concluding remarks and suggestions can be summarized as follows.

1. Service performance index control limits of long-span track cable-stayed bridge could be set as “vertical acceleration 1.5 m/s², lateral acceleration 1.8 m/s², Sperling index includes excellent, good and qualified characterization, derailment coefficient 0.8, wheel weight/load reduction rate 0.6, lateral force 80kN.”

2. In service stage, the permanent deformation control limits of the long-span track cable-stayed bridge could be divided into two levels: safety control and pre-warning control. The maximum vertical deformation safety control value is L/400, and the pre-warning control value is L/500. After deducting the non-permanent
deformation value $\beta$, the permanent deformation safety control value is $L/400-\beta$, and the pre-warning control value is $L/500-\beta$.

3. Alignment control should be carried out throughout full-life cycle of the bridge. In design stage, the down-deflection of the girder body should be fully considered to optimize the parameter design. In the construction stage, the dynamic effect should be considered to set reasonable pre-camber, and effective monitoring and control should be implemented. In service stage, safety monitoring should be strengthened, and reasonable limits should be set. Also, the classification evaluation of service performance should be implemented, and corresponding decisions should be made.

**Declaration of conflicting interests**

The author(s) declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

**Funding**

This work was supported by major research and development projects on topics of artificial intelligence technology innovation in Chongqing (cstc2017rgzn-zdyfX0029), special social and livelihood key research and development projects on technology innovation and application demonstration in Chongqing (cstc2018jscx-mszdX0084).

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