Comparative study of seismic resilience between PC and steel-concrete bridges in a permafrost region on the Qinghai-Tibet Plateau

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Abstract

The process of the damage accumulation of bridges is studied under the extreme environment of the Qinghai-Tibet Plateau, and an evaluation scheme of the resilience of bridges is proposed, which aims for the cold region with high seismic risk. The objective of the study is to evaluate the robustness of the Prestressed Concrete (PC) bridge in service on the Qinghai-Tibet Plateau and the steel-concrete composite bridge. Based on the theoretic and statistical analysis of the existing data, the pile foundation settlement, fatigue and freeze-thaw damages during the service of concrete bridge and steel-concrete composite bridge were studied respectively. The loss function of bridge before earthquake was constructed. With implementation of bridge vulnerability, the post-earthquake recovery function of bridge system was developed. On the basis of these relationships, the variation functions of the PC and steel-concrete bridges under service are obtained. The critical maintenance and reinforcement date before earthquake are proposed, and the seismic resilience of the two types of bridges are evaluated.

1 Introduction

The Qinghai-Tibet Engineering Corridor is located in the high-altitude permafrost region, where the bridges in service are subjected to the combined influences of the frozen soil environment and seismic activities. In order to reduce the economic losses caused by earthquakes, the seismic design philosophy of bridges has been extended to include post-earthquake structural functionality restoration and rapid repair. This poses higher requirements for the seismic performance of bridges in permafrost regions with high seismic risk. To explore the post-earthquake recoverability and ease of repair of bridges, the concept of resilience has been introduced into the research related to the seismic design of bridges.

The definition of resilience by Bruneau et al. (2003), which refers to "the ability of social units to reduce disaster risks, control the impacts of disaster occurrence in terms of minimizing community disruptions and reducing future consequences while conducting recovery activities," has been widely accepted. Shinozuka et al. (2000) improved upon Bruneau's approach and developed a new method for measuring resilience. They used the cumulative exceedance probability of functional losses under different seismic motions as an indicator of resilience. Manyena (2006) defined seismic resilience as the inherent ability of a system to adapt, survive, and self-repair by changing its non-essential attributes when subjected to predetermined disturbances. Cimellaro et al. (2010) further studied seismic resilience by analyzing a hospital building, providing a new framework for quantitative research on seismic resilience and proposing various models for post-earthquake structural functionality recovery. Decò et al. (2013) assessed the probability of bridge damage through earthquake vulnerability analysis and derived a sinusoidal function with six parameters as a bridge's functional recovery function. Then they proposed an evaluation method for bridge seismic resilience, which considers the uncertainties of expected damage, recovery, and reconstruction costs. Ikpong et al. (2015) used various performance indicators to evaluate the bridge resilience under the influence of climate, and this evaluation method had been applied to many bridges in Canada. Chandrasekaran et al. (2016) studied the resilience of bridges reconstructed with column jackets of different materials under various disasters such as earthquakes and floods, and proposed a variety of different optimization schemes. Kilanitis et al. (2018) pointed out that measuring the seismic performance of bridges using a single indicator is not sufficiently accurate. They proposed resilience evaluation indicators based on various factors such as vehicle mileage, providing insights for simulating the sequence of seismic strengthening and the post-earthquake recovery process of bridges in a specific region. Vishwanath et al. (2019) took the reinforced concrete bridge deteriorated by chloride corrosion as the research object, analyzed its vulnerability through numerical simulation, and further evaluated the resilience. The results showed that the resilience of the bridge decreased with the aging of the bridge. Li et al. (2020) proposed a framework for long-term resilience and loss assessment of highway bridges under multiple independent natural hazards, providing a reference for the design of bridges under extreme conditions. Mitoulis et al. (2021) established a recovery model of bridges after flooding, and evaluated and quantified the resilience of flood-critical bridges.

The Qinghai-Tibet Engineering Corridor is situated in a high-altitude permafrost zone, which is also characterized by high seismic intensity. Unlike urban bridges, the Corridor faces distinct challenges due to the unique combination of high altitude, low temperature, and intense seismicity. And now, there is still a lack of suitable seismic resilience evaluation methods for bridges in permafrost region with high seismic risk. Based on the author's previous research (Xiong et al. 2018), it is found that currently, the thaw settlement, overloading, and freezing-thawing phenomena are quite common in bridges along the Qinghai-Tibet highway. The accumulation of these unfavorable

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environmental factors makes the functional performance of bridges more susceptible to degradation. Taking the existing PC girder bridge and the proposed steel-concrete composite girder bridge in permafrost region with high seismic risk as the research object, this paper comprehensively considers the three functional damage factors of the bridge, such as freeze-thaw damage, fatigue damage and uneven settlement of pile foundation, and combined with the analysis of the seismic vulnerability of the bridge, the functional change of the bridge's life cycle is studied. Based on this, the critical maintenance and reinforcement years of bridges are proposed, and the seismic resilience of bridges during their whole life cycle are evaluated, which can provide a reference for seismic design, post-disaster maintenance and repair of bridges in permafrost regions with high seismic risk on the Qinghai-Tibet Plateau.

2 Bridge Loss Function Construction

2.1 Time-varying Effect of Freeze-thaw Damage

In cold regions, the freeze-thaw cycle can result in physical damage to concrete, affecting its strength, elastic modulus, and the anchoring performance of internal steel bars. Consequently, the durability of the structure is compromised. This study employs the variation of the concrete's relative dynamic elastic modulus under freeze-thaw conditions to represent the functional changes in the bridge system. Based on this, the loss function for freeze-thaw damage of bridges is constructed.

According to the indoor rapid freeze-thaw test (Zheng et al. 2018), the relative dynamic elastic modulus of concrete decreases with the increase of the number of freeze-thaw cycles. The relationship between the relative dynamic elastic modulus of C50 concrete and the number of freeze-thaw cycles can be obtained by fitting the data of the indoor test, as shown in Eq. 1.

$$P = \exp(-0.0034 - 1.7505\times10^{-4} N - 1.9097\times10^{-6} N^2) \quad [1]$$

In the formula, P is the relative dynamic elastic modulus of C50 concrete; N is the cumulative number of freeze-thaw cycles.

Due to the difference of indoor and outdoor environments, the freeze-thaw damage of concrete in indoor environments cannot be equated with that in outdoor environments. According to the research of Li et al. (2000), the expected value of the ratio of the number of freeze-thaw cycles between indoor and outdoor is 1:12 for the currently prescribed rapid freeze-thaw test methods. In this paper, the annual average number of freeze-thaw cycles of Xining, a representative city in Northwest China, is taken as the annual average number of freeze-thaw cycles of the research site, that is, 118 times per year. The loss function of freeze-thaw damage $Q_1(t)$ of concrete is fitted, as shown in Eq. 2.

$$Q_1(t) = \exp(-0.0034 - 1.7213\times10^{-3} t - 1.8466\times10^{-4} t^2) \quad [2]$$

In the formula, $Q_1(t)$ is the loss function of the bridge system under the freeze-thaw cycles, which is a dimensionless quantity in [0, 1]; t is the service time of the bridge.

2.2 Time-varying Effect of Fatigue Damage

In practical engineering, except for static load, bridges resist the repeated action of vehicle load, which causes irreversible fatigue damage of concrete. The fatigue failure of concrete is mainly due to the repeated action of vehicle load, and the internal damage of concrete material leads to the decline of its strength, resulting in the decline of the bearing capacity and durability of the bridge, and the loss of the function of the bridge system. The fatigue deformation modulus of concrete can reflect the degradation of concrete performance under the repeated action of load. According to the research of Xiong et al. (2022), the residual function of the bridge system under fatigue damage is quantified by the reduction coefficient of fatigue deformation modulus, and the calculation formula is shown in Eq. 3.

$$Q_2(t) = 0.93503 - 0.01456\ln(t + 0.01154) \quad [3]$$

In the formula, $Q_2(t)$ is the loss function of the bridge system under the fatigue load, which is a dimensionless quantity in [0, 1]; t is the service time of the bridge.

2.3 Time-varying Effect of Pile Settlement Damage

Under the background of global warming, the annual average air temperature of the Qinghai-Tibet Plateau has been increasing, which has caused the permafrost to thaw continuously, the permafrost table to deepen, and the permafrost mechanical parameters to change constantly. These factors make the settlement of the pile foundation of the bridge more and more serious, and affects the traffic capacity of bridge.

In this study, the finite element software ABAQUS was used to model the bridge pile foundation, and C3D8R elements were used for both the pile foundation and the surrounding soil. The loads considered including the dead weight of the bridge superstructure, the dead weight of the substructure, and the live load. The changes of the permafrost table and the mechanical parameters of permafrost caused by the temperature change were considered in different models, and numerical simulation was carried out on the bridge pile foundation with different service time. The changes of the permafrost limitable and the mechanical parameters of permafrost were referred to the research of Xiong et al. (2022). The finite element model is shown in Figure 1. Through the analysis, the changes of pile foundation settlement of the PC girder bridge and the steel-concrete composite girder bridge versus service time were obtained.

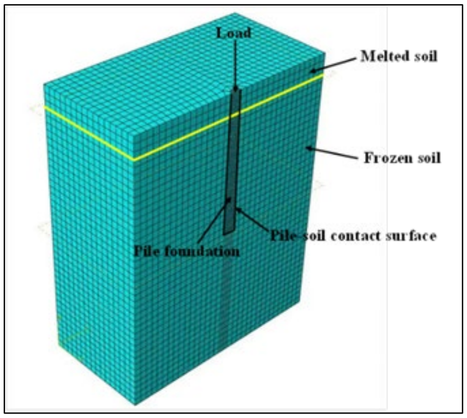

Figure 1. Finite element model.

According to the mechanical performance requirements of the bearing in "Highway Bridge Plate Rubber Bearing" (JT/T-2019), the measured rotation angle tangent value of the plate rubber bearing is tanθ⩾1/300, and the allowable rotation angle tangent value of the bearing is taken as 1/300 in this paper. Considering the most unfavorable effect, the unilateral pile foundation settlement is regarded as the uneven settlement of the bridge pile foundation, and the service range is taken as the tangent value of the bearing's rotation angle from 0 to 1/300, to quantify the loss function of pile foundation settlement by nonlinear fitting of the data. The loss function of the PC girder bridge and the steel-concrete composite bridge is shown in Eq. 4 and Eq. 5, and the fitting curve is shown in Figure 2.

$$Q_3(t) = 0.95283 - 0.01091e^{\frac{t}{44.1677}} \quad [4]$$

$$Q_3'(t) = 0.9598 - 0.0093e^{\frac{t}{44.1677}} \quad [5]$$

In the formula, $Q_3(t)$ and $Q'_3(t)$ are the loss functions of the bridge system under the pile foundation settlement, which are dimensionless quantities in [0, 1]; *t* is the service time of the bridge.

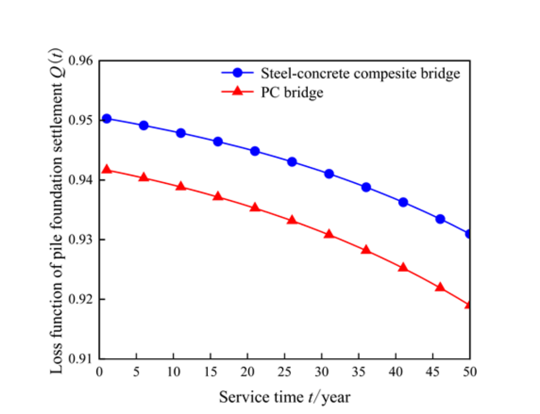

Figure 2. Loss function of pile foundation settlement of the PC bridge and the steel-concrete composite bridge.

2.4 Loss Function

2.4.1 PC Girder Bridge

For the PC girder bridge, as shown in Figure 3, the loss function can be constructed by superimposing the three damage factors of concrete freeze-thaw damage, concrete fatigue damage, and pile foundation settlement.

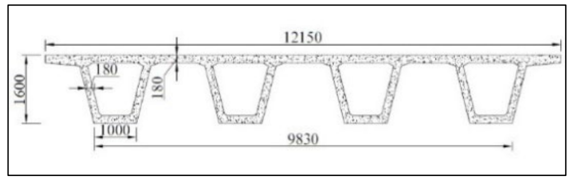

Figure 3. Schematic diagram of the PC bridge.

2.4.2 Steel-concrete Composite Girder Bridge

For the steel-concrete composite girder bridge, as shown in Figure 4, assuming that the freeze-thaw damage mechanism of concrete is similar to the fatigue damage, and both are mainly aimed at the concrete part of the superstructure of the steel-concrete composite girder bridge. Therefore, after linearly superimposing the damage amount of the two factors, it is reduced according to the mass of the concrete part of the superstructure. Considering the most unfavorable situation, the reduced freeze-thaw damage and fatigue damage are superimposed with the pile foundation settlement damage to construct the loss function *Q*(*t*) of the steel-concrete composite girder bridge, as shown in Eq. 6.

$$\begin{aligned} Q(t) &= 1 - \begin{bmatrix} (1-Q_1(t)) + (1-Q_2(t)) \times 0.87 \\ +(1-Q'_3(t)) \times 0.87 \end{bmatrix} \\ &= Q_1(t) + 0.87Q_2(t) + 0.87Q'_3(t) - 1.74 \end{aligned} \quad [6]$$

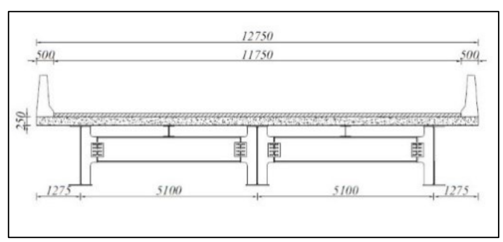

Figure 4. Schematic diagram of the steel-concrete composite bridge.

3 Seismic Resilience Assessment for Bridges

3.1 Vulnerability Analysis

In order to obtain the vulnerability of the bridge system, usually according to the first-order reliability theory, the upper and lower limits of the vulnerability of the bridge system can be obtained from the vulnerability of the vulnerable components, and the actual vulnerability of the bridge system is between the upper and lower limits. The average value of the upper limit and the lower limit can be taken as the vulnerability of the bridge system, and the upper and lower limits of the vulnerability of the bridge system in a certain damage state can be determined by Eq. 7.

$$\max_{i=1}^{m}\left[P\left(F_i\right)\right] \le P\left(F_{system}\right) \le 1-\prod_{i=1}^{m}\left[1-P\left(F_i\right)\right] \quad [7]$$

In the formula, $P(F_{\text{system}})$ is the exceeding probability of the bridge system reaching a certain damage state; $P(F_i)$ is the exceeding probability of the i-th component reaching a certain damage state; m is the number of vulnerable components.

Xiong et al. (2023) studied the component vulnerability of the PC girder bridge and the steel-concrete composite girder bridge with the same site, span, design level and other conditions in permafrost region with high seismic risk. In this paper, with reference to this study, the system vulnerability of the PC girder bridge and the steel-concrete composite girder bridge is obtained through component vulnerability, as shown in Figures 5 and 6.

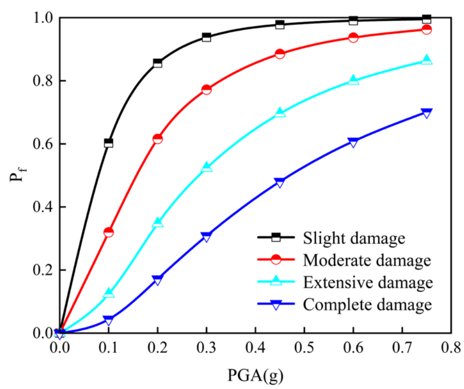

Figure 5. Vulnerability of the PC bridge.

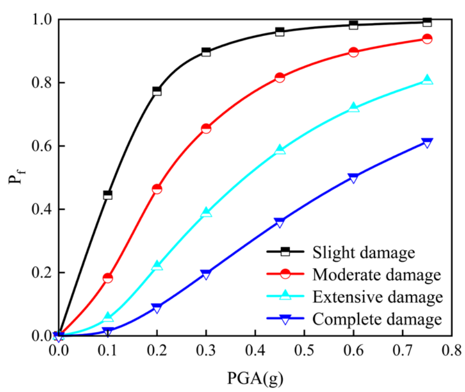

Figure 6. Vulnerability of the steel-concrete composite bridge.

3.2 Earthquake Loss Function

The loss of the bridge system under earthquake action includes two types: direct loss and indirect loss. This paper only considers the impact of direct loss. For a bridge, the direct loss caused by the event E can be expressed by the dimensionless quantity L_D, which is expressed as the ratio of the repair cost C_{rE} to the replacement cost C, which reflects the expected value of the loss. The calculation formula is shown in Eq. 8 (Venkittaraman et al. 2014).

$$L_D = \frac{C_{rE}}{C} = \sum_k P_E\left(F_{\text{j}}\middle|IM\right)\times r_k \quad [8]$$

where C_{rE} represents the repair cost; C represents the replacement cost; k represents the damage state of the bridge; $P_E(F_{\text{j}}|IM)$ is the probability of the bridge being damaged at the damage state k in the earthquake event E, which can be obtained from the fragility curves at all levels of damage states; IM is the earthquake intensity index; r_k is the damage ratio corresponding to the damage state k.

3.3 Post-earthquake Recovery Function

According to HAZUS (Zhou et al. 2010), the average and standard deviations of the bridge system recovery time under different damage states are obtained. The expected value of recovery time under different intensities of earthquakes is defined as the sum of the average value of the recovery time under different damage states and the product of the probability of occurrence of the corresponding damage state, which reflects the expected value of the recovery time. The calculation formula is shown in Eq. 9.

$$T = \sum_k P\left(F_{\text{j}}\middle|IM\right)\times \mu_k \quad [9]$$

T is the expected value of the recovery time; k represents the damage state of the bridge system; $P(F_{\text{j}}|IM)$ is the probability of the bridge system occurring at the damage state j; μ_k is the average recovery time corresponding to the damage state j.

Fitting the loss L_D under different intensities of earthquakes with the expected value of recovery time T, from which *PC* girder bridge's loss function can be obtained.

3.4 Critical Repair and Reinforcement Time Analysis

The critical time when the functionality of the bridge is lower than the critical state is defined as the critical maintenance and reinforcement period. For the bridge system, maintenance and reinforcement must be carried out at this time to ensure the continued opening to traffic.

In this paper, a number of different peak ground accelerations (PGAs) were selected for research, and the schematic diagram of the critical maintenance and reinforcement period and the state of temporary maintenance of traffic of the PC girder bridge and the steel-concrete composite girder bridge is shown in Figure 7 and Figure 8.

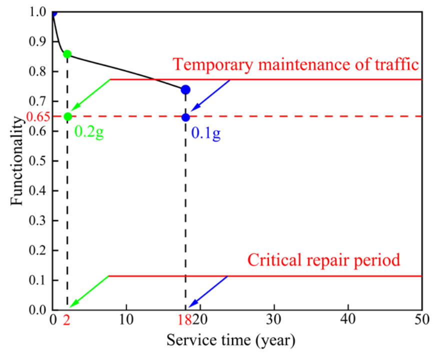

Figure 7. Critical maintenance and reinforcement period of the PC Bridge under different PGA.

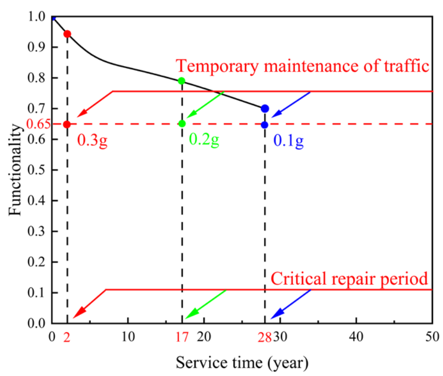

Figure 8. Critical maintenance and reinforcement period of the steel-concrete Bridge under different PGA.

As shown in Figure 7, for the PC girder Bridge, when the PGA is 0.1 g and 0.2 g, the critical maintenance and reinforcement period of the bridge system are 18 years and 2 years, respectively. When the PGA is greater than 0.3 g, the vibration reduction and isolation device must be considered in the bridge design scheme.

As shown in Figure 8, for the steel-concrete composite girder bridge, when the PGA is 0.1 g, 0.2 g and 0.3 g, the critical maintenance and reinforcement period of the bridge systems are 28 years, 17 years and 2 years, respectively.

4 Discussion

In the assessment of the seismic resilience of bridges, in order to quantitatively evaluate the seismic resilience, the integral formula including bridge functionality $Q(t)$ is usually used to calculate the seismic resilience (Soleimani 2022), as shown in Eq. 10.

$$R = \frac{1}{\Delta t}\int_{t_0}^{t_0+\Delta t} Q(t)\mathrm{d}t \qquad [10]$$

When the PC girder bridge has been in operation for 20 years and an earthquake occurs, the recovery curves of the bridge under different PGAs are shown in Figure 9. The recovery time and resilience are shown in Figure 10. It can be seen from Figure 9 that when the PGA is 0.2 g and 0.4 g, the bridge needs to be repaired and the recovery time of the bridge system is 31 and 98 days, respectively. It can be seen from Figure 10 that the recovery time of the bridge system increases with the increase of PGA, and the resilience decreases with the increase of PGA; when the PGA is 0.2 g, 0.3 g, and 0.4 g, the resilience is 0.593, 0.553, and 0.511 respectively.

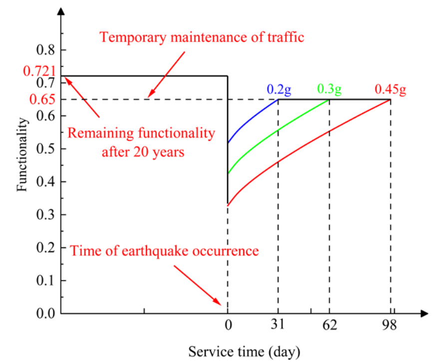

Figure 9. Function curves of the PC bridge under different PGAs.

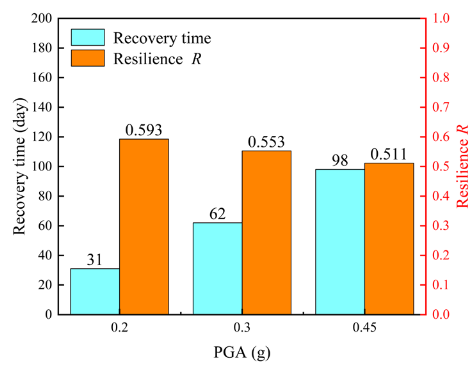

Figure 10. Recovery time and resilience of the PC bridge under different PGAs.

Similarly, when the steel-concrete composite girder bridge has been in operation for 20 years and an earthquake occurs, the bridge needs to be repaired and reinforced to achieve the state of temporary maintenance of traffic. When the PGA is 0.2 g and 0.4 g, the recovery time of the bridge system is 3 and 57 days respectively, and the corresponding resilience is 0.641 and 0.559 respectively.

As shown in Figure 11, the recovery time of the two bridges with a service time of 20 years are compared. It can be seen that under different PGAs, the recovery time of the steel-concrete composite girder bridge is significantly shorter than the PC girder bridge.

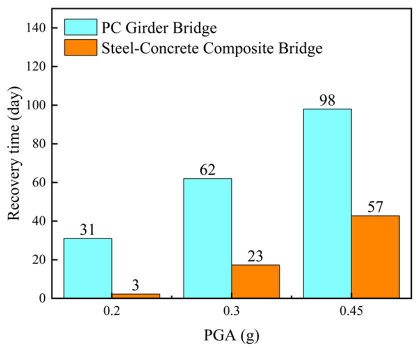

Figure 11. Comparison of recovery time between the PC bridge and the steel-concrete composite bridge.

5 Conclusions

(1) The loss function of the active PC girder bridge and the proposed steel-concrete composite girder bridge in the Qinghai-Tibet Plateau area is studied. The result shows that the pile settlement damage of the steel-concrete composite bridge is smaller.

(2) The critical repair and reinforcement period before the earthquake of the PC girder bridge and the steel-concrete composite girder bridge in the Qinghai-Tibet Plateau are studied. When the PGA is 0.1 g, 0.2 g and 0.3 g, the critical maintenance and reinforcement period of the steel-concrete composite bridge are 28 years, 17 years and 2 years, and the critical maintenance and reinforcement period of the PC bridge are 18 years and 2 years, and when the PGA is greater than 0.3 g, the vibration reduction and isolation device must be considered in the PC bridge.

(3) A seismic resilience evaluation method for bridges in permafrost region with high seismic risk is proposed, and the seismic resilience comparative evaluation was carried out between the PC girder bridge and the steel-concrete composite girder bridge in the Qinghai-Tibet Plateau. The results show that under the same intensity earthquake, compared with the PC girder bridge, the seismic resilience of the steel-concrete composite girder bridge is higher, and the repair time is shorter.

6 Acknowledgements

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