Experimental and Numerical Investigation on the Bearing Behavior of Curved Continuous Twin I-Girder Composite Bridge with Precast Concrete Slab

Chuandong Shen, Yifan Song, Lei Yan, Yuan Li, Xiaowei Ma, Shuanhai He, and Xiaodong Han

1School of Highway, Chang’an University, Xi'an 710064, China
2Shanxi Expressway Construction Group Company, Xi'an 710065, China

Correspondence should be addressed to Chuandong Shen; shenchuandong1991@126.com and Yifan Song; syf@chd.edu.cn

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Curved twin I-girder composite bridge (TGCB) is becoming popular in Chinese highway bridge building. To study its ultimate bearing behavior, in this paper, one 1:5 scale intact model of a two-span curved continuous TGCB was tested to failure to evaluate its safety reserve and ductility. Afterwards, based on the experimental result, 3D FE models were developed and validated. At last, using the validated 3D FE models, the effect of construction scheme, radius of curvature, yield strength of steel, concrete compressive strength, crossbeams, and bottom lateral bracings on the ultimate bearing capacity were examined. The experimental results showed that the ultimate load (Pu) is approximate 13.6 times the service equivalent load. The cracking load and yielding load are approximately 0.12 and 0.47 Pu, respectively. The ductility coefficients are 4.06~4.40. These above may indicate that the TGCB designed according to Chinese codes has good safety reserve and ductility. From parameter analysis results, it was concluded that the TGCB with full-support construction scheme has larger yield load and ultimate load compared with the one with erecting machine construction scheme. On the other hand, the ultimate bearing capacity reduces nonlinearly with the increase of curvature. Besides, the yield strength of steel, crossbeams, and bottom lateral bracings has a significant effect on the ultimate bearing capacity of curved TGCB. And the smaller the radius of curvature, the more obvious the effect of the latter two factors is. Unfortunately, it is unwise to continuous to improve the ultimate load by increasing the grade of steel for the TGCB when steel grade exceeds Q390. Moreover, in consideration of the big difference in bearing capacity between the inner girder and outer girder of the TGCB with small radius of curvature as well as the economy, it is suggested that the inner and outer steel girders of that TGCB should be designed differently.

1. Introduction

In recent years, as a typical candidate for Accelerated Bridge Construction (ABC) techniques, twin I-girder composite bridge (TGCB), especially curved ones, is becoming a very competitive scheme in the selection of medium- and small-span bridge structure in China. In fact, it has been used widely throughout some developed country highway systems [1]. On one hand, compared with traditional multi-girder steel bridges, the TGCB is greatly simplified. On the other hand, it provides possibility of reusing materials so as to reduce the life-cycle cost of bridge compared with concrete bridges. Meanwhile, the twin I-girder composite bridges are often designed to be curved to adapt to the direction of traffic routes [2~5].

Numerous experimental and numerical studies have been carried out on steel-concrete composite beams [6~9] (Shim and Chang, 2003; Hyung-Keun Ryu, 2004; Sang Ahn, 2005; Weiwei Lin, 2014; Hamdy Afefy, 2017; Jianan Qi, 2020). For example, the experimental study of Hyung-Keun Ryu indicated that the cracked section of continuous composite box-girder bridge should include the area of
longitudinal reinforcements. In addition, Lin et al. concluded that the failure modes of curved composite girders with I-section are changing from bending failure to torsional failure with the curvature increasing. Besides, the study of Jianan Qi et al. indicated that the ultimate bearing capacity of the composite girders with I-section is not sensitive to the grade of steel beam. It still contributes to improving the bearing capacity of composite beams to a certain extent. In summary, the previous studies mainly focus on the individual steel-concrete composite girder due to its easy fabrication, small dimensions, and low cost as well as the ease of operation of loading and constraints compared with intact structure. However, individual girder is unable to consider the effect of transverse members on the mechanical behavior, which has been pointed out by Kennedy (1983) [10]. The overall mechanical behavior of twin I-girder systems is different from that of individual composite I-girder. In particular, due to influence of auxiliary components, such as crossbeams and lateral bracings, the stress of curved bridge is more complex. The overall mechanical performance and load-carrying capacity of curved bridge are not simply superposition of all individual girders or controlled by the minimum value. Therefore, it presents further practical significance to carry out the test on the intact bridge model, despite its high cost compared with individual beam.

On the other hand, crossbeams, cross-frames, diaphragms, and lateral bracings form a bracing system. Generally, they are considered secondary members that are needed for erection stability and distributing loads during construction in straight bridge. However, due to the coupling effect of bending and torsion in curved composite bridge, the mechanical property of curved bridge is different from that of straight bridge. When used in curved bridges, they become important load-carrying and torsional resistance members both during construction and in service [11] (Md. Robiul Awall, 2013). In a great deal of previous research studies regarding the bracing system, the purpose is mainly focused on the effect of the bracing system on the construction behavior [12–15] (Winterling, 2007; Quadrato et al., 2010; Sanchez, 2011; Sharafbayani, M., 2014), mechanical performance of box composite girders [16] (Hamdy M. Afefy et al., 2016), and improving the dynamic performance [11, 17–21] (Schelling et al., 1989; Kim and Yoo, 2006; Maneetes and Linzell, 2003; Chavel and Earls, 2006a, 2006b; Awall et al., 2013). For instance, Sharafbayani, M. (2014) examined the effects of a bracing system on the construction performance of curved I-girder bridges. The study found that the setting of skewed cross-frames could make the load of the main girders more uniform. Hamdy M. Afefy (2016) studied the effect of cross bracings on the mechanical properties of curved box girder at different loading stages through loading tests of three box girders. They found that the external bracings helped reduce the deflection in inelastic stage and increase the ultimate bearing capacity of box girder by about 8.9%. Md. Robiul Awall (2013) investigated the effect of bottom lateral bracing on dynamic performance of horizontal curved twin I-girder bridges under running vehicles through the finite element method. Their study found that the bottom lateral bracings can make dynamic loads more uniform between the two main girders. The bottom lateral bracings are beneficial to form a more stable structure. However, there are few reports available about the effect of the crossbeams as well as the lateral bracings on the ultimate bearing capacity of curved TGCB.

Besides, the effect of initial stress caused by construction is not considered neither in traditional experimental studies as well as numerical studies regarding composite beams or bridge. However, Han Lin-hai has proved that construction initial stress has effect on bearing capacity of concrete-filled steel tubular (CFST) columns [22]. Similar to the CFST, the TGCB also has the characteristics of self-erecting members. Before the composite section works, there is a considerable stress in the steel girder section, which is called initial stress. Therefore, the effect of initial stress on the ultimate bearing capacity of the TGCB needs to be studied.

Shen et al’ previous study [23] presented the test results of one 1:5 scale intact model of the TGCB. However, the influence factors of ultimate bearing capacity of curved TGCB have yet to be further studied. In this paper, the most relevant test results were presented, and based on the test results, 3D finite element models (3D FE models) are developed and validated. Afterwards, using the validated 3D FE models, the effect of construction scheme, radius of curvature, yield strength of steel, concrete compressive strength, crossbeams, and bottom lateral bracings on the ultimate load-carrying capacity of the TGCB are studied. These parametric studies find out effective methods to improve the ultimate bearing capacity of the curved TGCB, and some useful conclusions are drawn.

2. Experimental Study

2.1. Model Design and Building. Figure 1 shows the prototype bridge, a two-span curved continuous TGCB, which is designed according to Chinese codes [24–28]. The bridge is located in Shanxi province, China. Its centerline arc length span is 75 m long with a radius of curvature of 460 m. The height of the composite girder is 2.2 m. Two main girders are spaced at 6.7 m, which are interconnected by end crossbeams and intermediate crossbeams at a uniform spacing of 7 m. The width of concrete slab is 12.75 m. The thickness of concrete slab is 0.22~0.4 m.

An intact bridge model with 1:5 linear scale of the prototype bridge configuration for experimental study was designed. The typical geometry of the bridge model and dimensions of all steel plate cross-section parameters are shown in Figure 2 and Table 1. The total centerline arc length span of the bridge model is 14000 mm, with a radius of curvature of 92 m. The net span arrangement is 2 × 6814 mm. The bridge model is 440 mm in depth and 2550 mm in width. It is consisted of two main steel I-girders with a space of 1340 mm. The outer girder and the inner girder are labeled G1 and G2, respectively. Figure 2(b) shows the cross-sections of the bridge model. The height of steel girder is 360 mm. The thickness of concrete slab is 44 mm~80 mm. Four shear pockets are reserved in each precast slab to connect concrete slab and steel girders with group studs, shown in Figures 2(d) and 2(e). The stud is 10 mm in diameter and 40 mm in height, respectively.
Figure 1: The prototype bridge (TGCB).

Figure 2: Continued.
Figure 2: Continued.
2.2. Materials. The aggregate size of the actual prototype concrete is 9.5 mm–19 mm. In order to ensure the operability of the construction, according to our experimental experience and similar experiments in other literature studies, C50 fine aggregate (less than 10 mm) concrete having the same strength as the actual prototype was used for concrete precast slab. Expansive concrete material is poured into the transverse wet joints to reduce the initial drying shrinkage. Nonshrinkage mortar is poured into the reserved shear pockets to make the steel girders and concrete slab work together. The steel plates used in the model are Q345b. The thickness of steel plate includes 6 mm, 8 mm, 10 mm, and 12 mm. In addition, 6 mm and 10 mm diameter reinforcing bars are used as transverse reinforcement and longitudinal reinforcements in the slab, respectively. The shear studs are made of MLA15. Tables 2 and 3 list the material properties of concrete and steel, respectively.

2.3. Test Method and Instrumentation. The test loading condition and setup is shown in Figure 5. We developed a test setup for field experiment, which was consisted of a concrete ground anchor, two reaction crossbeams, and six anchor rods, as shown in Figure 5. Each reaction crossbeam is welded by three 3.2 m long 36b I-beams. Each anchor rod’s diameter and length are 32 mm and 7500 mm. Both calculated and experimental results indicate that the deformation of reaction crossbeam is very small and can be neglected compared with that of the bridge model.

The load was applied through four hydraulic jacks over the webs at midspan nearby. In order to ensure the synchronous loading and real-time monitoring, we used one intelligent tension control system to control all jacks. At the same time, the loading force could be measured by a pressure sensor under each Jack. All the test data were displayed on a data acquisition system (TDS-303) in real time. Each concentrated load was increased at an increment of 10 kN. Then, at each loading step, the load was kept constant to record all the deflection data and strain data. The load-deflection curve of the model was drawn during the test. When the curve obviously enters the nonlinear stage, we slow down the loading to ensure the safety of the test process.

Figure 6 shows measurement contents. Figure 6(a) presents the layout of test section. The strain gauge distributions of section B, D, and E are shown in Figure 6(b).

3. Test Results and Analysis

3.1. Failure Modes. This paper only presents the most relevant test results to verify the accuracy of the 3D finite element model. More test results can be seen in a previous study [23]. Table 4 lists the main quantitative features of the test. The ultimate load was 407 kN which was approximate 13.6 times the service equivalent load. The ductility coefficient of the external girder was 4.40. The ductility coefficient

\[ \text{Ductility Coefficient} = \frac{\mu_{\text{final}}}{\mu_{\text{yield}}} \]

where \( \mu_{\text{final}} \) is the ultimate strain and \( \mu_{\text{yield}} \) is the yield strain. This demonstrates the bridge model has sufficient ductility.
Figure 3: Gravity compensation: (a) gravity compensation of steel girder and (b) gravity compensation of concrete slab.

1 Installing the first steel girder segment (40m) in place;
2 Installing precast concrete slabs in sequence;
3 Pouring concrete in the shear pocket and transverse joints;
4 Completing the deck pavement.

1 Installing the second steel girder segment (30m) in place,
2 Welding two segments together;
3 Installing precast concrete slabs in sequence;
4 Pouring concrete in the shear pocket and transverse joints;
5 Loading pavement weight sandbags.

Figure 4: Construction sequence of the bridge model and prototype bridge.

| Material name                  | Compressive strength/(MPa) | Young’s modulus/(GPa) |
|-------------------------------|---------------------------|-----------------------|
| C50 fine aggregate concrete   | 52.2                      | 36.8                  |
| Expansive concrete            | 54.0                      | 36.4                  |
| Nonshrinkage mortar           | 58.0                      | 36.8                  |

| Material name   | Thickness/(mm) | Yielding strength/(MPa) | Tensile strength/(MPa) | Young’s modulus/(GPa) |
|-----------------|----------------|-------------------------|------------------------|-----------------------|
| Steel plate     | 6              | 394                     | 563                    | 203                   |
|                 | 8              | 387                     | 568                    |                       |
|                 | 10             | 406                     | 574                    |                       |
|                 | 12             | 395                     | 556                    |                       |
| Reinforcement   | 6              | 571                     | 754                    | 200                   |
|                 | 10             | 492                     | 673                    |                       |
| Shear stud      | 40 (diameter)  | 397                     | 453                    | 190                   |
Figure 5: The test loading condition and setup.

Figure 6: Measurement contents (unit: mm): (a) layout of test section; (b) the strain gauge distributions of test section.

Table 4: The main quantitative features of the experiment.

|                | $P_{cr}$/(kN) | $P_{y}$/(kN) | $P_{u}$/(kN) | $P_{u,cal}$/(kN) | $\Delta_{y}$/(mm) | $\Delta_{u}$/(mm) | $\mu$   |
|----------------|---------------|--------------|--------------|------------------|-------------------|-------------------|--------|
| Outer girder   | 50            | 190          | 407          | 378              | 11.14             | 49.06             | 4.40   |
| Inner girder   | 232           | 407          | 7.25         | 29.52            | 7.25              | 29.52             | 4.06   |

Note: $P_{cr}$ and $P_{y}$ present cracking load and yielding load obtained from test. $P_{u}$ and $P_{u,cal}$ present the ultimate load obtained from test and calculated. $\Delta_{y}$ and $\Delta_{u}$ are the deflection corresponding to the yielding load and the ultimate load obtained from test, respectively. Define ductility coefficient $\mu = \Delta_{u}/\Delta_{y}$. 
of the internal girder was 4.06. These above may indicate that the TGCB has good safety reserve and ductility. Table 4 also gives the loading force values corresponding to ultimate bending for the tested specimens with Chinese estimated code design value [26]. The calculated ultimate load was 378 kN. The difference between the tested and calculated result was about 8%. The effect of the shear was not considered because it was insignificant compared with the bending effect.

There were four typical failure modes in the final failure of the test model bridge: ①the majority of cross-section of steel girder yielded at the middle support position; ②the steel webs presented local buckling both at loading section; ③the concrete in the negative bending moment zone was cracked in large area; ④the upper surface of the concrete slab was crushed near the loading section. Figure 7 shows the overall shape and local details of the bridge model after failure. There are obvious bending angles at the loading section and the middle bearing section. It is worth mentioning that the sandbags did not touch down ground until the test ended.

3.2 Load-Deflection Relationship. Figure 8 depicts the load-deflection relationships of the test model at the loading section. It can be observed that both G1 (outer girder) and G2 (inner girder) have good elastic-plastic deformation. There were four stages in each load-deflection relationship curve: ①from the beginning of loading to the time when the concrete slab was cracked, the relationship between load and deflection is linear. The whole bridge model worked well. This stage can be called elastic stage before cracking. ②When the load exceeded the cracking load, the concrete cracks appeared in the tensile zone, and the slope of load-deflection curve slightly decreased. However, due to the small crack width of the tensile concrete, the load-deflection curve was still approximately linear. This stage is called the elastic stage after cracking. ③Afterwards, the load-deflection relation curve presented a nonlinear behavior. The slopes of the curve decreased gradually. This represented the decreasing of stiffness. This stage is called the elastic-plastic stage. ④Finally, at a load of 407 kN (ultimate load), the steel webs appeared local buckling as shown in Figure 7. It was taken at the mid-support section. Subsequently, this region developed into plastic hinge. Soon, the structure completely lost its bearing capacity.

3.3 Crack Development and Distribution. When the load reached about 0.12 Pu, the concrete cracks first appeared on the top surface of slab over mid-support. It was along the interface between cast-in-situ concrete and precast concrete. With increasing load, the cracks quickly propagated and spread. The width and the number of cracks increased. At the load of 0.34 Pu, there are twelve concrete cracks near mid-support region. The width of major crack is 0.15 mm. At the load of 0.37 Pu, the width of major crack is 0.2 mm. Generally, the maximum crack width of reinforced concrete slab should be limited within this width [27].

Figure 9 shows the cracking distribution at the end of the test. The cracks developed through the entire concrete slab at transverse. The cracks are uniformly distributed in the two precast concrete slabs and cast-in-situ concrete joints on both sides of the mid-support section. The cracks are uniformly distributed in the concrete slabs within about 80 cm from the mid-support section. The maximum crack width is 4.5 mm. The wide of other visible cracks is basically more than 0.1 mm. Therefore, cracked section of the TGCB is located within about 0.11 times the span from the mid-support section.

3.4 Cross-Section Strain Distribution. Figure 10 shows the normal strain increment distribution across the depth at section B at different loading stages. At the linear elastic stage ($\varepsilon < \varepsilon_y$), the distribution of strain increment points was according with the assumption of plane section. When the strain $\varepsilon > \varepsilon_y$, the model entering the inelastic stage, strain distribution exhibited some nonlinear characters. The assumption of plane section gradually fails. Besides, as the bottom flange and steel web enter the yield stage successively, the position of the composite section neutral axis tends to move up.

4. Finite Element Analysis

ABAQUS 6.14 is powerful general finite element software [30]. It was used to further study the ultimate bearing behavior of the TGCB. Figure 11 shows the 3D FE model of the tested bridge model above. The concrete and steel loading blocks are simulated by C3D8R solid elements. The steel plate girders, cross beams, and stiffeners are simulated by S4R shell elements. The reinforcement and bottom lateral bracings are simulated by T3D2 truss elements [31, 32]. The 3D FE model in Figure 11 contains 58654 elements and 73905 nodes.

The slip between steel girders and concrete slabs is not considered, and the concrete slabs are connected to the steel girders with "tie" constraint in the 3D FE model [31].

Bearing steel backing plates were also established. On one hand, the top surface of bearing steel backing plates was tied with girders. On the other hand, the bottom surface was coupled with a reference point below. Boundary constraint is applied to the reference point as follows, respectively. The reference points of mid-support are constrained by the vertical and longitudinal translation displacement; the reference points of end support are constrained by the vertical translation displacement.

In this paper, based on the modeling approach of INP files, the simulation of the construction process of the test composite bridge model was implemented by using "birth-and-death elements." The construction process of the test composite bridge model is presented in Figure 4.
Figure 7: Failure pattern of the bridge model (view of internal girder).

Figure 8: Load-deflection relationships at the loading section: (a) outer girder G1; (b) inner girder G2.

Figure 9: Concrete slab cracking distribution at the end of the test.
The concrete damaged plasticity (CDP) model is used to simulate the constitutive relation of concrete. Coefficients in CDP for concrete are shown in Table 5. Code for Design of Concrete Structures (GB50010-2012) gives the stress-strain relationship of concrete [33]. The concrete material mechanical properties under compression could be described as follows:

\[
\sigma = (1 - d_c)E_c \varepsilon_c, \quad (1)
\]

\[
d_c = \begin{cases} 
1 - \frac{\rho_c n}{n - 1 + X_c}, & X_c \leq 1, \\
1 - \frac{\rho_c}{\alpha_c (X - 1)^2 + X_c}, & X_c > 1,
\end{cases} \quad (2)
\]

\[
\rho_c = \frac{f_{ck}}{E_c \varepsilon_{c,r}}, \quad (3)
\]

where \(\sigma\) is the stress of concrete; \(\varepsilon\) is the strain of concrete; \(E_c\) is Young’s modulus of concrete; \(\alpha_c\) is the coefficient effecting the shape of the stress-strain relation curve; \(f_{ck}\) is the characteristic strength of concrete, which could be calculated by \(f_{ck} = 0.67 f_{cu}\); \(f_{cu}\) is the cube strength of concrete; \(\varepsilon_{c,r}\) is the strain of concrete corresponding to the peak of the stress-strain curve; \(d_c\) is the defined damage coefficient; and \(X_c\) is the normalized compressive strain calculated by \(\varepsilon / \varepsilon_{c,r}\). Poisson’s ratio is 0.2. Table 6 lists the value of \(\varepsilon_{c,r}\) and \(\alpha_c\).

The stress-strain relationship of steel and reinforcement can be described as

\[
\sigma = \begin{cases} 
E_c \varepsilon_c, & \varepsilon \leq \varepsilon_y, \\
f_y, & \varepsilon > \varepsilon_y,
\end{cases} \quad (5)
\]
4.2. Parameter Analysis. In order to further investigate the effect of importance parameters on the ultimate load-carrying capacity and stiffness of curved continuous TGCB, more numerical results are shown in Figures 13–19. The geometries of finite element models in the parametric study are the same as the test bridge model above. The considered parameters in the paper included the construction scheme, radius of curvature, yield strength of steel, compressive strength of concrete, crossbeams, and bottom lateral bracings.

4.2.1. Effect of Construction Scheme. According to the traffic demand and topographic condition under bridge during construction, the construction schemes can be divided into support construction and nonsupport construction. The nonsupport construction mainly includes incremental launching [34] and bridge erecting machine construction which was adopted by the prototype bridge and test bridge model in this paper. This kind of scheme is not restricted by the terrain and traffic under the bridge. However, the full-support construction scheme is usually adopted by bridges without traffic requirements under it. This kind of scheme does not need a large bridge erecting machine, and the construction support can be removed after the TGCB is formed. Figure 13 shows the load-deflection curves of the curved TGCB models with two construction schemes under the action of transverse symmetrical loads. In the elastic stage, the mechanical response of the curved TGCB models formed by the two construction schemes is the same. However, there are deviations in ability to carry external load. Taking the outer girder (G1) as an example, the yield load and ultimate load of the TGCB model with full-support construction scheme are 280 kN and 468 kN, respectively, while that of the TGCB model with erecting machine construction scheme is 225 kN and 417 kN, respectively. It indicates that the composite bridge constructed by latter scheme has smaller yield load and ultimate load. The reason is that different construction schemes lead to different construction initial strain (ε0) or stress (σ0). For the construction of bridge erecting machine, both the self-weight of steel girder and bridge deck are borne by the cross-section of steel girder before composite section formed, and the construction initial stress of steel girder is relatively large, so the capacity of remaining resistance to external load is small. However, when the full-support construction scheme is adopted, the gravity of steel girder and concrete is borne by the composite section, and the construction initial stress of steel girder is small. Therefore, it is very necessary to consider construction process for ultimate load-carrying capacity analysis.

4.2.2. Effect of Radius of Curvature (R). To investigate the effect of radius of curvature (R) on the behavior of the TGCB, load-deflection curves are obtained from the straight
TGCB model ($R = \infty$) and curved TGCB models with three different radii of curvatures ($R = 90 \text{m}, 50 \text{m}, \text{and} 10 \text{m}$), as shown in Figure 14. In the straight model, it can be seen that the developed deflection of outer and inner girders is the same due to the absence of the torsional effect under the action of transverse symmetrical load.

In contrast to straight bridge, the curved bridge has a larger deflection even under the same load. And the greater the curvature, the greater the deflection. On the other hand, the curvature reduces the ultimate load of the TGCB model. And the smaller the radius of curvature, the greater the influence. For the 50 m and 10 m radius of curvature, the calculated ultimate loading capacity was reduced by 2% and 22%, compared with that of the straight bridge model.

In addition, the outer steel girder always yields and fails earlier than the inner steel girder for curved TGCB. And the greater the curvature, the more obvious the difference. For the 10 m radius of curvature, the yielding strain

![Figure 12: Strain analysis results at the mid-support section E: (a) load-strain relation of steel main girders; (b) load-strain relation of longitudinal reinforcements.](attachment:image12)

![Figure 13: Load-deflection curves of curved TGCB models with different construction schemes: (a) outer girder; (b) inner girder.](attachment:image13)
load and ultimate load of outer steel girder are 76 kN and 322 kN, respectively. The yielding load and ultimate load of inner steel girder are 130 kN and 375 kN, respectively. That big difference between outer and inner girders is due to torsional effects in the curved TGCB. And torsional effects become larger with the decreasing radius of curvature. Therefore, it is suggested that the inner and outer steel girders of the curved bridge with smaller radius of curvature should be designed differently to increase the stiffness of the outer steel girder. Besides, Figure 15 shows the relationship between ultimate load and radius of curvature. The ultimate load increases nonlinearly with the increase of the radius of curvature. When the radius of curvature is small, the change is fast, and when the radius of curvature is large, the change is slow.

4.2.3. Effect of Yield Strength of Steel (f_y). Given the failure mode of the continuous TGCB model is mainly the yielding of steel girder. In order to compare the influence of yield strength of different steels on ultimate load-carrying capacity, load-deflection curves are obtained from curved TGCB models having R = 90 m and straight TGCB models with four different yield strengths of steel (f_y = 235 MPa, 345 MPa, 390 MPa, and 420 MPa), as shown in Figure 16. With the increase of yield strength of steel girder, the bridge model shows higher bearing capacity. The ultimate loads of curved TGCB models with 345 MPa, 390 MPa, and 420 MPa are 49%, 70%, and 74% higher than that with f_y = 235 MPa, respectively. And the same is true for the straight TGCB model. It indicates that when the yield strength of steel girder is less than 390 MPa, the ultimate load-carrying capacity of composite bridge can be effectively improved by increasing the strength grade of steel girder. However, when the yield strength exceeds 390 MPa, it is uneconomical and undesirable to continue to improve the bearing capacity by increasing the yield strength of steel. In addition, the initial stiffness of each bridge model is basically unchanged in the elastic stage.

4.2.4. Effect of Concrete Compressive Strength (f_c). To compare the influence of concrete compressive strength of slab on ultimate load-carrying capacity, Figure 17 shows the load-deflection curves of curved TGCB models having R = 90 m and straight TGCB models with different concrete compressive strength (f_c = 40 MPa, 50 MPa, and 60 MPa). No matter for straight TGCB models or curved bridge models, the discrepancy can hardly be seen by analyzing the load-deflection curves. The reason is that the cracking load of the composite bridge is low, and the concrete does not contribute too much in the flexural stiffness of the bridge after concrete cracks appear. Therefore, the concrete compressive strength has little effect on the stiffness and crack load and ultimate load-carrying capacity of the TGCB. So, it is unreasonable and uneconomical to expect to improve the bearing capacity of the TGCB by increasing the concrete compressive strength.

4.2.5. Effect of Crossbeams. In general, crossbeams, crossframes, and diaphragms play a role of secondary members because they make contribution to stabilize the compression zones of noncomposite girders during construction [13]. Therefore, the previous research on the role of crossbeams mainly concentrated in the construction stage or elastic stage. Keller (1994) and Davidson et al. (1996) studied the effects of cross-frame and lateral bracing configurations on the performance of curved bridges during construction. Afefy (2017) examined the effect of cross-frame configuration and number on the free-vibration response of curved bridges. However, the research of crossbeams on ultimate load-carrying capacity of

![Figure 14: Load-deflection curves of curved TGCB models with different radius of curvature: (a) outer girder; (b) inner girder.](image-url)
Curved bridges are rarely reported. In order to research the effect of crossbeams on ultimate load-carrying capacity of straight and curved TGCB, this paper examined the effect of crossbeam removal on the load-deflection curves of straight and curved TGCB models, respectively, as shown in Figure 18. Six FE models were established. Model MS-C1 is the straight bridge without crossbeam systems between the supporting lines. Model MS-C1’ is straight bridge model but with four crossbeams between the supporting lines. Curved TGCB models having $R \leq 50$ m and $R \leq 10$ m without crossbeam systems between the supporting lines are labeled MC-C1 and MC-C2, respectively. Curved TGCB models having $R \leq 50$ m and $R \leq 10$ m with four crossbeam systems between the supporting lines are labeled MC-C1’ and MC-C2’, respectively. Comparison of MC-C1 and MC-C1’ presented that the presence of crossbeams increased the ultimate bearing capacity of the curved TGCB model by approximately 5%. However, comparison of MC-C2 and MC-C2’ presented that the presence of crossbeams increased the ultimate bearing capacity of the curved TGCB model by approximately 42%. It indicated that the effect of crossbeams is prominent in bridge with large curvature compared to bridge with small curvature. On the other hand, the crossbeams have no effect on the deflection in elastic stage, but when the TGCB model enters into plastic stage, the deflection of steel girders will be reduced significantly due to crossbeams at the same load. In addition, in comparison of MS-C1 and MS-C1’, no significant change was observed for load-deflection curves. It indicated that the presence of crossbeams does not contribute too much in the ultimate bearing capacity of the straight TGCB model.
4.2.6. Effect of Bottom Lateral Bracings. Linzell (2003) and Md. Robiul Awall (2013) investigated the improvement effect of bottom lateral bracings on dynamic performance of horizontal curved TGCB. Bottom lateral bracings are also important load-carrying and torsional resistance members in horizontally curved bridges both in construction stage. However, the influence of bottom lateral bracings on ultimate load-carrying capacity has not been studied. To examine the effect of bottom lateral bracings, four FE models were established. One straight TGCB model without bottom lateral bracings is labeled MS-L1. Another straight TGCB model with bottom lateral bracings is labeled MS-L1’. One curved TGCB model having $R = 10 \text{ m}$ without bottom lateral bracings is labeled MC-L1. Another curved TGCB model having $R = 10 \text{ m}$ with bottom lateral bracings is labeled MC-L1’. None of the four models above have crossbeams between the supporting lines. Besides, a curved TGCB model having $R = 10 \text{ m}$ with crossbeams and without bottom lateral bracings was also established, which is labeled MC-CB1. Load-deflection curvatures of five bridge models are drawn in Figure 19. By comparing the curvatures of MC-L1 and MC-L1’, we found that bottom lateral bracings can

Figure 17: Load-deflection curves of the TGCB models with different concrete compressive strength: (a) outer girder G1 in curved TGCB models, (b) inner girder G2 in curved TGCB models, and (c) straight TGCB models.

Figure 18: Load-deflection curves of the TGCB models with and without crossbeams: (a) outer girder G1; (b) inner girder G2.
enhance the ultimate load-carrying capacity of the curved TGCB model by 35%. On the other hand, at elastic stage, the slope of load-displacement curvatures of MC-L1' is slightly larger than that of MC-L1. This indicates that bottom lateral bracings can also improve the torsional stiffness of the model bridge in the elastic stage, because the setting of bottom lateral bracings makes the original open section become a quasiclosed section. However, by comparing the curvatures of MS-L1 and MS-L1', no significant change was observed for load-displacement curves. It indicated that bottom lateral bracings have no effect on the bearing capacity of the straight bridge.

5. Conclusions

In this paper, a cautious static test on one 1/5-scale intact bridge model of curved continuous twin I-girder composite bridge (TGCB) was carried out. To further investigate the ultimate load-carrying capacity of curved continuous TGCB, a series of finite element models were established and analyzed. From the experimental and the 3D FE model results above, the following conclusions could be drawn:

1. From the experimental results, The TGCB designed according to Chinese codes has good safety reserve and ductility. The composite section within 0.11 times the span from the mid-support section became cracked section in finally failure.

2. Based on the 3D FE models, it was concluded that the TGCB with full-support construction scheme has larger yield load and ultimate load compared with the one with erecting machine construction scheme due to the influence of construction initial stress ($\sigma_0$). On the other hand, the ultimate bearing capacity increases nonlinearly with the increase of the radius of curvature.

3. The yield strength of steel, crossbeams, and bottom lateral bracings has a significant effect on the ultimate load-carrying capacity of curved TGCB. And the smaller the radius of curvature, the more obvious the effect of crossbeams and bottom lateral bracings on the ultimate load-carrying capacity is. Unfortunately, concrete compressive strength has insignificant effect on the ultimate loading capacity.

4. There are also some design suggestions: it is recommended to use UHPC in the $\pm 11\%$ L areas to improve the crack resistance of the negative bending moment region. Although the strength grade of steel beam has a significant effect on the bearing capacity, it is unwise to continuous to improve the ultimate load by increasing the grade of steel for the TGCB when steel grade exceeds Q390. In consideration of the big difference in bearing capacity between the inner girder and outer girder of the TGCB with small radius of curvature as well as the economy, it is suggested that the inner and outer steel girders of that TGCB should be designed differently.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.
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