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Load Carrying Capacity and Durability of PC Girder Using CFCC Tendon under Long-Term Exposure in Corrosive Salt Environment

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Abstract

The Shinmiya Bridge was completed in October 1988 in Ishikawa, Japan. This was the first prestressed concrete bridge in Japan, and the world, to utilize carbon-fiber-reinforced plastic (Carbon Fiber Composite Cable, CFCC) tendons in its main girders to counteract salt damage. To investigate and clarify the serviceability and durability of the main girders after a long period of service in a corrosive environment, two full-scale PC test girders were fabricated and placed next to the main girders in the same conditions at the time of the original construction from 1988. One of them was used for a destructive test in 1994. In this study, another test specimen exposed to a severe environment for 30 years was subjected to a destructive bending test and CFCCs taken out from the test girder were analyzed via mechanical and chemical tests. The suitability of the CFCC and durability of the main girder were confirmed. In addition, a numerical model was proposed to predict the behavior of a main PC girder using a finite element analysis program called DIANA. The influences of the input data from the experiment, the decrease in material properties, and the bond-slip model were studied in the simulation.

1. Introduction

The Hokuriku region, which is along the coastline of the Sea of Japan, is highly affected by seasonal-wind-based airborne salinity from the Northwest. Reinforced concrete structures are corroded by salt penetration, and thus the serviceability and durability of the structures are decreased. This problem has become a challenge for civil engineering works, especially for bridges. The old Shinmiya Bridge in Ishikawa was a reinforced concrete bridge that had been damaged by salt corrosion. It was necessary to build a new bridge after 12 years in service

(Enomoto *et al.* 2012).

However, since the 1980s in Japan, carbon-fiber-reinforced plastic (CFRP) had been studied as an alternative to steel in civil engineering to combat deterioration due to salt damage (JSCE 1998). Carbon fiber is made from either polyacrylonitrile-based (PAN-based) or pitch-based (Erki and Rizkalla 1993). Carbon Fiber Composite Cable (CFCC) was a type of CFRP and was the brand name of the Tokyo Rope Manufacturing Company, Ltd. It was shown that CFCCs have outstanding features compared to regular steel. CFCC not only has high tensile strength, high tensile modulus, light weight, and high corrosion resistance, but it also features excellent creep resistance, high fatigue resistance, low linear expansion, low relaxation, and is easily coiled (Kimura *et al.* 1990). Therefore, CFCC was selected as the tensioning material for the main girders of the new Shinmiya Bridge in October 1988. The new Shinmiya Bridge was the first prestressed concrete bridge in Japan and the world to utilize CFCC tendons against salt damage (Santoh *et al.* 1993).

Subsequently, researchers across the globe took note of this technique and it began to be applied in a variety of structures in civil engineering. In November 1993, the Beddington Trail Bridge was built in Calgary, Alberta, Canada. CFCC tendons and Leadline strands were selected to prestress two spans of this bridge (Rizkalla and Tadros 1994). In 1997, another study on the flexural behavior of concrete beams prestressed by CFRP reinforcement was conducted by Abdelrahman and Rizkalla in Canada. In this study, T beams 6.2 m in length and 0.33 m in depth were used, the CFRP bars had a diameter of 8 mm, and the tensile strength and elastic modulus were 1970 MPa and 147 GPa, respectively. While eight

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concrete beams were prestressed by CFRP bars, two beams were prestressed by regular steel strands for comparison. The observations of the different failure modes indicated deflection and cracks (Abdelrahman and Rizkalla 1998). In the United States, the Bridge Street Bridge was built in Southfield, Michigan in 2001. It was the first bridge in the USA that used a CFCC tendon in the T beam, and the deflections, concrete strains, CFRP and CFCC strains will be monitored until 2020 to record the long-term value (Grace *et al.* 2002). The first CFRP application for a prestressed girder in the road bridge in Belgium was in 2004. Carbon-fiber polymer wires with a diameter of 5 mm, tensile strength of 2450 MPa, and Young's modulus of 160 GPa were chosen for a T beam (Corte and Van Bogaert 2005). In 2012, Enomoto *et al.* summarized the results of the structural bridges using CFCCs and confirmed that CFCCs were suitable for prestressed concrete bridges (Enomoto *et al.* 2012). Furthermore, a study conducted by Grace *et al.* (2012) showed that the initial cost of CFRPs was higher than the regular steel-reinforced bridges, but the life cycle cost of the bridge was not high. The cost will become the least significant obstacle after 20–40 years of service time (Grace *et al.* 2012).

Therefore, recent studies on the usage of CFCC in a prestressed concrete structure focus on standards in design and durability in actual service. A study in 2015 analyzed a concrete beam prestressed with a CFRP tendon according to ACI 440.4R-04 and ISIS design manual No.5. The difference in the structural behavior between both codes was presented (Katarzyna and Łukasz 2015). In 2016, Sevil Yaman conducted a series of experiments for CFCC application as a corrosion-resistant material in the prestressed precast concrete bridge for marine environments. T-Beam specimens (length of 3350 mm, height of 304.80 mm, flange width of 914.40 mm and 304.80 mm in the web width) were analyzed and compared the results obtained using the ACI 318 Code prediction (Yaman 2016).

However, there were few findings to be confirmed regarding the durability of the bridge prestressed by CFRPs subjected to actual long-term salt damage. In 2017, Mark F. Green presented research on the durability of CFRPs for bridges over a long period in Canada. The strength and durability of a prestressed CFRP sheet and CFRP tendon over 13 years were confirmed (F. Green 2017). In Shinmiya Bridge, the first prestressed concrete bridge in the world using CFCC tendons in the main girders, investigations to clarify the serviceability and durability of the main girders reinforced by CFCC tendons have been conducted from the time of construction. Three test girders were manufactured with the same size as the main girders in 1988. At the same time, one girder was used to perform a flexural experiment to determine the ultimate behavior and load carrying capacity of the PC girder (Yamashita *et al.* 1989) as well the deflection and the strain behavior of the CFCC (Futakuchi *et al.* 1991). Furthermore, two other girders

(mountainside, seaside) were placed next to the main girders in the same conditions for confirming their long-term quality. Six years after the construction time (1994), a destructive test was carried out on the seaside girder, a comparison of load carrying capacity with the results at the time of construction was done, and the durability of the CFCC tendon was confirmed (Kanda *et al.* 1995).

In this study, a destructive bending experiment was conducted on another mountainside specimen that was exposed to an actual salt environment for nearly 30 years. The ultimate load and crack initiation load were compared with those values at the time of construction and six years after the time of construction. From the mechanical and chemical tests of the CFCCs removed from the main girder, its rupture load and durability were clarified. In addition, the commercial finite element software, DIANA, was used to surmise the behavior of a PC girder using the CFCC tendon, which was compared with the results of the experiment. Moreover, parametric studies were conducted with the following objectives: to investigate the influence of the input data from the results of the CFCC tensile test and concrete compressive test on the results and accuracy of the simulation, to determine the change in structural behavior with the modification of the material properties, and finally to study the model with or without the bond-slip model for contact between the CFCC and concrete.

2. Description of Shinmiya Bridge

The Shinmiya Bridge was built at the Shika-machi, Hakui-gun, Ishikawa Prefecture, Japan in October 1988. Originally, the bridge was a reinforced-concrete-slab bridge, but under the corrosive environment of the building site, the steel material significantly corroded, and the concrete surface was broken within 12 years after the original construction. Therefore, CFCCs were selected to reinforce the main girders in the new bridge in October 1988. **Figures 1(a)** and **1(b)**, respectively, show the side view and cross-section of the Shinmiya Bridge. The bridge length was 6100 mm and the effective width was 7000 mm. In addition, **Fig. 2** shows the I-shaped cross-section (JIS A 5313, S106-325) of the main girders. Regarding countermeasures against salt damage to the main girders, D6 epoxy-coated rebar and eight CFCCs of $\Phi 12.5$ mm with seven strands were introduced, as shown in **Figs. 2** and **3**. The surface of the CFCCs was twisted and roughened to enhance the adhesion with concrete.

As shown in **Fig. 1(b)**, the main bridge has a structure in which 24 main girders with an I-shaped cross-section, as shown in **Fig. 2**. On both sides of the bridge, two full-scale test girders (mountainside, seaside) were fabricated and transversely tightened together with the main girders of the bridge in the same environment (see **Fig. 1(b)**) to confirm their long-term robustness. When six years passed (1994) after construction, the test girder on the seaside was removed and subjected to a destruc-

tive bending experiment. In addition, the transmission length test, salinity measurement, tensile test and chemical composition test of the CFCC tendons removed were performed. In this research, other test girder on the mountainside was removed from the main bridge and subjected to the same tests as mentioned above.

3. Experimental methods

The experimental process followed in this research is shown in **Fig. 4**. First, after removing the test girder from the bridge site, it was brought into the laboratory; the effective stress was estimated using the stress release technique with a core incision, and the obtained value was compared with the design value. Next, a flexural experiment by applying a bending load was carried out, and the difference from the ultimate load obtained in the same test conducted six years after construction was considered. The transmission length of the prestressed girder, obtained by cutting the upper and lower flanges, was examined by using half of the girder after the flexural experiment. Furthermore, the cores were collected, and salt content was measured. Finally, the mechanical and chemical characteristics of the CFCC were determined.

4. Estimation of effective stress

4.1 Test method

Regarding the use of CFCC, it was considered that the effective stress might decrease owing to material fatigue

caused by repeated live loading and relaxation caused by material deterioration under exposure to an actual service environment over a long period. Therefore, to determine whether the effective stress of the PC girder using CFCC tendons was still in the design stress state after nearly 30 years, the effective stress estimation method based on the stress release technique (Niitani *et al.* 2009) was used. The effective stress was estimated by the release of the working stress and measuring the strain.

This test girder was a pre-tensioned beam, and a transmission length existed at the end of the girder. The stress was stabilized in the cross-section at the center of the span, and the bottom flange with a large stress was selected as the measurement position. Although no noticeable deterioration was observed in the appearance of the test specimen, the stress might vary along the width of the bottom flange. Hence, the positions deviated from the center of the cross-section to the left and right at 160 mm from the center and 60 mm from the edge, and the distance between the two positions was 1000 mm. **Figure 6** shows the three positions considered in this experiment.

In term of the stress decreased by core incision, the core incision was made to a depth of 18 mm using a drilled core of Φ 50 mm, and the stability was confirmed at intervals of 150 s after incision and the measurement was completed.

4.2 Estimation results

By applying the effective stress estimation method based on the stress release technique, the effective stress

Fig. 1 Drawing of Shinmiya Bridge.

Fig.2 Cross-section of main girder (unit: mm).(O: CFCC)

Fig. 3 CFCC tendon.

Table 1 Comparison of effective stress of design value and estimated value. (did not consider stress due to dead load)

No.	Design value (N/mm ²)	Estimate value (N/mm ²)	Ratio	Difference
1	11.08	10.76	97%	-0.32
2	11.08	11.72	106%	0.64
3	11.08	12.05	109%	0.97
Average	11.08	11.51	104%	0.43

on the main girders was estimated and compared to the design value of the effective stress to evaluate the durability of the CFCC tendons. **Table 1** compares the design value of the effective stress with the estimated values (these values did not consider stress due to dead load). The estimated values at each measurement sample almost approximated to the design value. In comparison with sample 2 (No.2), sample 3 (No. 3) was slightly higher by 0.3 N/mm² and sample 1 (No. 1) was lower by about 1 N/mm². This result was consistent with the deformation behavior of the warp due to the increase and decrease in the tensile force. The average of the three positions (11.51 N/mm²) was higher than the design value by 0.43 N/mm², and the ratio between

the average value and the design value was 104%. It appeared that there was no significant difference between the effective stress in this study and the design value.

5. Flexural experiment

5.1 Test method

The flexural experiment was conducted using the same method that was used in 1988 and six years after the construction (1994). The loading method is shown in **Fig. 5**, and the arrangement of measurement points is shown in **Fig. 6**. In the destructive bending experiment, the static bending load was applied at two points in the middle of the span with a distance of 1000 mm. Measurement items included the load cell, displacement at the support positions, the center of the main girder, embedded strain gauge, and concrete surface strain gauge.

The loading procedure was the same as that used in the flexural experiment, which was conducted in 1988 and 1994. First, the load was applied up to 35.4 kN (design load 35.3 kN) and returned to 0 kN. Second, the sample was loaded to crack initiation load and returned to 0 kN again. Finally, the sample was loaded to the ultimate load as the final stage.

Fig. 4 The flow chart of research.

Fig. 5 The setup of loading test.

Table 2 Comparison of crack initiation load and ultimate load.

		In 1988	In 1994	In 2017
	Design value	At the time of construction	After 6 years	After 29 years
Crack initiation load (kN)	68.3	70.6	98.3	82.8
Ultimate load (kN)	131.2	132.3	167.1	157

5.2 Load-displacement relationship

Figures 7 and 8 show the images of the girder before the test and at failure, respectively. The load-displacement relationship and crack pattern obtained by the flexural experiment are shown in **Figs. 9 and 20**, respectively. In addition to the results of this flexural experiment, the crack initiation load, load carrying capacity at the time of construction and six years after construction, and design values are summarized in **Table 2**.

The failure mode imposed in this flexural experiment was the crushing of concrete in the compression area after the bending crack propagation. This failure mode occurred because the CFCC had no yield phenomenon, and the girder was designed to be a destructive form, in which the upper concrete collapsed before the CFCC

ruptured. This destruction was similar to that observed on test girders in the experiments at the time of construction and six years after construction.

Figure 10 shows the condition of the PC girder surface after the concrete collapse with detailed CFCC observations. It was found that the CFCC in the upper side was twisted and broken after crushing of the concrete. The situation of CFCC in the upper side was shown on the yellow dash-line areas on **Fig.10**. However, failure due to tension could not be confirmed at the CFCC on the lower side.

The design crack initiation load (68.3 kN) and design bending fracture load (131.2 kN) are mentioned in **Fig. 9**. From the load-displacement relationship, when the load was removed after loading up to 35.4 kN and the crack initiation load, almost no residual displacement could be confirmed. Although the results of the flexural experiment of the mountainside girder at this time and of the seaside girder at six years after construction time were almost overlapped, the ultimate load was reduced by around 6% as compared with the result obtained at six years after construction. However, as observed in **Table 2**, this value exceeded the value at the time of construction and design values. Therefore, it was judged that there was no problem in terms of the load carrying capacity.

Fig. 6 Measurement positions (unit: mm).

Fig. 7 Girder specimen before test.

Fig. 8 Girder specimen at failure.

6. Transmission length test

6.1 Test method

In pre-tensioning concrete members, the steel strands have an initial stress following the required force, then concrete is cast around strands. When the concrete reaches a sufficient compressive strength, the prestress force is transferred to the concrete due to the bond between the strands and the surrounding concrete through the release of prestress. After this release, there is a variability in the prestressed force from zero in the end of structure to the constant maximum value (effective prestressing force) in the center zone. The length required to achieve the effective prestressing force is defined as transmission length or transfer length (Martí-Vargas *et al.* 2013).

After long-term service of the girder, to investigate whether the bond between the CFCCs and concrete was decreased or not and the transmission length changed or

not, a transmission length test was conducted.

In this test, the technique of concrete surface strain, which recorded a relationship between strains on the concrete girder surface versus the distance to the end of the girder, was used to determine the transmission length. After the flexural test, the main girder was cut at the center of the span and one of two half girders was used. First, 40 strain gauges were attached on the neutral axis in an interval of 2000 mm from the end of the girder. **Figure 11** shows an arrangement of strain gauges following the zigzag because the size of the strain gauge (WFLM-60-11) for this test was large. Next, the web where tendons were not present was removed from the main girder by concrete cutting machine. The blue line in **Fig. 11** indicates the boundary among the web, lower flange, and upper flange. The strains were measured at the time of cutting and the transmission length was determined by observing the variation of strains.

6.2 Result

Figure 12 shows the measurement values of strain in three cases. The blue circles are used to show the measured strains after the lower flange was completely cut; the red triangles indicate the results measured immediately after both of the upper and lower flanges were cut off; and the green squares serve to illustrate values of strain on the concrete surface measured after 16 hours since after both of the upper and lower flanges were cut off.

The transmission length according to the road bridge specification is defined as 65ϕ (Japanese Road Association 2002) hence it was 812.5 mm in the case of this main girder. However, from the results of this test shown in **Fig. 12**, the transmission length was considered 500 mm, which was smaller than 65ϕ . The same result was obtained in the test conducted six years after construction (Kanda *et al.* 1995), and it became clear

Fig. 9 Load-displacement relationship at the center of the girder.

Fig. 10 Status of CFCC in compression and tension zone at failure.

Fig. 11 Setup of transmission length test.

that both of the transmission length and the bond between the CFCC and concrete had not changed.

7. Measurement of salt content

7.1 Test method

After the flexural test, four drilled cores were collected on the web side and bottom surface of the girder. Three cores were on the web side and one core was on the bottom surface of the girder. Then, the salt content was measured on these samples.

Fig. 12 Changes in transmission length in web.

Fig. 13 Chloride ion concentration on web of main girder.

Fig. 14 Chloride ion concentration on the lower flange of main girder.

On the web side, three concrete cores with a diameter Φ 40 mm and length of $L = 80$ mm were collected at positions of 1080 mm, 1150 mm and 1280 mm from the end of the girder. A core with a length of $L = 30$ mm from the concrete surface to the stirrup position at the position 920 mm from the end of the girder was collected on the bottom surface of the girder. The concrete cores were sliced to a 10-mm-thick layer, and the total chloride ion concentration in each slice was measured according to JIS A 1154 (Japanese Industrial Standards 2012). From the measurement results of the chloride ion concentration, the total chloride ion concentration at each depth was calculated using the Fick's diffusion coefficient of chloride ions (D_c) and surface chloride ion concentration (C_0).

7.2 Result

From the measured and calculated results, the total chloride ion concentration on the web side (three samples) and lower flange are shown in **Figs. 13** and **14**, respectively. In **Fig. 13**, the red circles, blue triangles and green squares show measurement results of cores at positions of 1080 mm, 1150 mm and 1280 mm on the web, respectively. These values were measured with 10 mm-thick layers of samples, hence they are plotted in the center of each slice. In addition, the red line, blue line and green line illustrate the calculated results of the total chloride ion concentration for each depth of cores corresponding to three positions (1080 mm, 1150 mm and 1280 mm). In the same way, the red circles are used to demonstrate measured results of the core in the bottom surface while the red line shows the calculated results of salt content at each depth of this core in **Fig. 14**. Moreover, blue triangles and green squares, which indicate the results of the chloride concentration survey in 2012 by the drill method at positions located on the bottom surface of this girder on two directions (abutment A1 and abutment A2, as shown in **Fig. 1(a)**), were added on **Fig. 14** for comparison.

First, in terms of comparing the results in **Figs. 13** and **14** of two measurement sites, the bottom surface has a higher chloride ion concentration than the side surface. It was considered that this phenomenon was caused by rinsing of the salt on the concrete surface on the side by rainwater.

Second, considering the results for the web of the girder in **Fig. 13**, there is a decrease of chloride ion concentration in the width of the web from the mountain-side to the adjacent girder side. Salinity penetration from the mountainside was remarkable. In addition, the results chloride ion concentration on the adjacent girder side (the side that was face-to-face with the other girder) was quite lower. The average value of the surface chloride ion concentration (C_0) was 3.58 kg/m^3 , and the average value of the diffusion coefficient of chloride ions (D_c) was $0.04 \text{ cm}^2/\text{year}$.

However, from the results for the bottom surface of the girder, the surface chloride ion concentration (C_0)

Table 3 Results of compressive strength test of concrete.

	Compressive strength (N/mm ²)	Elastic modulus (kN/mm ²)	Poisson's ratio
No. 1	79.8	39.8	0.2
No. 2	70.4	34.6	0.2
Average	75.1	37.2	0.2
In 1988	59.8	32.1	0.2

Table 4 Results of CFCC tensile test.

	Rupture load			Elastic modulus (kN/mm ²)
	Average (kN)	Maximum (kN)	Minimum (kN)	
At the time of construction (in 1988)	156.8	165.1	148.0	139.2
After 6 years (in 1994)	155.8	165.4	148.8	142.1
After 29 years (in 2017)	156.8	161.0	148.6	132.5
Quality standard value		137.2		129.4~144.1

was 8.42 kg/m³, and the diffusion coefficient of chloride ions (D_c) was 0.83 cm²/year. Compared to the chloride ion concentration on bottom surface measured in 2012, there was no significant increase in chloride ion concentration for the depths from 0.0 to 2.0 cm from the concrete surface in **Fig. 14**, but the chloride ion concentration at a depth from 2.0 to 3.0 cm from the concrete surface has significant change. It was confirmed that the concentration increased, and it was estimated that the chloride ion concentration was nearly 6.0 kg/m³ at the position of the stirrup rebar (reinforcement cover was 27.5 mm). It largely exceeded the corrosion occurrence limit of steel, 1.2 kg/m³; the rebar could have been considered corroded if general steel materials were used.

8. Compressive strength test of concrete

After destructive test using a bending load, two cores with a diameter of approximately Φ 40 mm was taken from the upper flange of the girder at 740 mm and 870 mm from the end of the span, and the compressive strength test was carried out. The compressive strength, elastic modulus, and Poisson's ratio are summarized in **Table 3**. It seems that the average value of the compressive strength (75.1 N/mm²) is higher than the value recorded (59.8 N/mm²) in 1988, which is also given in **Table 3**.

9. CFCC tensile test

A concrete cutting machine removed CFCCs in the upper and lower flanges of the girder that were separated in the transmission length test. CFCCs, which located at positions of 1, 4, 5, and 8, shown in **Fig. 2**, were collected as samples for this test with the length of 2.2 m. Subsequently, they were subjected to preparation for the

tensile test and the ultimate tensile load and elastic modulus were determined. In addition to the ultimate tensile load and elastic modulus obtained in this study, the results at the time of the construction, the results at six years after construction, and the standard values are summarized in **Table 4**.

The elastic modulus decreased by approximately 7% from the value obtained six years after construction, but this value was still within the allowable variation range. In addition, the ultimate tensile load was comparable to that of past values and residual tension load capacity was confirmed.

10. Chemical analysis of CFCC tendons

10.1 FE-SEM observation

CFCCs were exposed to an alkaline environment in concrete and were affected by chloride ions and repeated loading due to the live load. Here, surface observation was carried out for CFCCs at positions 3, 4, and 8, shown in **Fig. 2**. The CFCC (polyester, wrapping) surface was observed with an optical microscope; the surface of the carbon fiber removed from the CFCC surface coating and the epoxy resin were observed in detail by FE-SEM (field emission scanning electron microscopy). **Figure 15** shows the CFCC surface observed with an optical microscope (20 times) in the left side and the surface of the carbon fiber observed with FE-SEM (2000 times) in the right side.

Although some scratches were caused when CFCCs

Fig. 15 CFCC surface and carbon fiber surface.

Fig. 16 Analysis result by FT-IR.

Table 5 Concrete properties for FE analysis.

	Young's modulus (kN/mm ²)	Poisson's ratio	Compressive strength (N/mm ²)	Tensile strength (N/mm ²)	Maximum size of coarse aggregate (mm)	Fracture energy (N/mm)
Concrete	37.2	0.2	75.1	4.09	20	0.1145

were collected from concrete, there was no position where the coatings and carbon fibers themselves seemed to have deteriorated owing to the salt environment and live load. Therefore, it was confirmed that there was no problem in the CFCC surface and carbon fiber surface.

10.2 Fourier Transform Infrared Spectroscopy (FT-IR) Analysis

FT-IR (Fourier Transform Infrared Spectroscopy) analysis was conducted using CFCCs from positions 3, 4, and 8, shown in **Fig. 2**. A total of nine core wires and two lateral wires of each cable were used as samples to investigate the chemical structural changes that accompany deterioration. As an example of the analysis results obtained using FT-IR, **Fig. 16** shows the comparison between the result of the core strand of the CFCC in position 4 and that obtained six years after construction. There was no major change in the result of this test compared with the FT-IR measurement result at six years after construction, but there was a position where a fine peak appeared. This may be because of the excellent accuracy of the current measuring machine. Therefore, it can be said that although several differences were observed, no deterioration attributed to chemical structural changes occurred because large changes, such as the disappearance of the main peak, could not be confirmed.

11. Numerical analyses

11.1 Overview of analysis model

To obtain a deeper understanding of the variations in the structural behavior of the girder, numerical simulations were performed using the finite element (FE) method. In the numerical approach, the models were first validated by comparing the elicited results with those obtained in real-scale tests. The emphasis here is on the methodology, i.e., on the identification of ways to apply FE analysis to the CFCC girder that possesses new characteristics. Because the destructive test itself encompasses nonlinear structural features and is associated with significant deformation, modeling the behavior

of the specimen until the final state requires the consideration of both high-order geometrical and nonlinear mechanical parameters, particularly for the selected nonlinear models. The developed model considered the nonlinear mechanical properties of the materials. As shown in **Fig. 17**, a three-dimensional (3D) numerical model is produced in accordance with the specifications of the actual specimen using FX+ in DIANA, which is a commercially available program used for nonlinear FE analyses. Regarding the element types used in the analyses, the concrete was modeled as a solid element, while the embedded steel reinforcement elements reproduced the stirrups and CFCC. The bonding between the concrete, steel, and CFCC was assumed to be complete. The energy-controlled convergence norm and regular Newton-Raphson were selected as the iterative methods. The analysis was carried out by applying incremental load factors with specified sizes. To facilitate simulation, several simplifying assumptions for the mechanical properties of the materials and supports were adopted.

11.2 Material properties of concrete

The rotating total strain crack model for concrete (DIANA 2017) was employed in this study, which includes the JSCE tension softening model for tensile behavior, and the multilinear model for compressive behavior. The parameters of concrete for the rotating strain crack model are presented in **Table 5**. Specifically, the Young's modulus and compressive strength of concrete were obtained from compression tests on concrete cores collected from the test specimens. The multilinear model was utilized to reproduce the compression stress-strain relationship of the concrete, as shown in **Fig. 18(a)**. Moreover, the critical parameters governing the JSCE tension softening model for tensile behavior (see **Fig. 18(b)**) were the fracture energy and tensile strength of concrete. The fracture energy of concrete is the energy consumed to form cracks per unit area and is calculated based on Eq. (1) (JSCE 2012).

$$G_f = 10(d_{\max})^{1/3} f_{ck}'^{1/3} \quad (1)$$

Fig. 17 Three-dimensional (3D) analysis model for the CFCC girder.

where G_f , $d_{\max}$, and f'_{ck} , are related to the fracture energy (N/mm), maximum aggregate size (mm), and compressive strength of concrete (N/mm²), respectively. In the present study, the value of $d_{\max}$ is assumed to be equal to 20 mm. Additionally, owing to the absence of experimental data, the estimation of the tensile strength of concrete f_{tk} can be performed using Eq. (2), based on the characteristic compressive strength f'_{ck} (JSCE 2012).

$$f_{tk} = 0.23 f'_{ck}{}^{2/3} \quad (2)$$

where the unit of strength is N/mm².

11.3 Steel materials

The stirrups were assumed to have the same Young's modulus as the linear elastic material model, $E_s = 207000$ N/mm², and the same Poisson's ratio, $\nu = 0.3$. The Von Mises plasticity was employed to simulate the behaviors of CFCCs. Specifically, the CFCC was defined by a property with a Young's modulus of $E_s = 132500$ N/mm² and Poisson's ratio of $\nu = 0.3$. The yield stress was defined as 2063 N/mm² based on results of the tensile test. Regarding the prestressing force, a value of 931 N/mm² was assigned to the CFCC. In addition, the prestress was applied to the numerical model as the initial stress at the execute

start step.

11.4 Comparison between analytical and experimental results

The evaluation of the accuracy of the simulation result was made by comparison with the experimental result, including the ultimate load, middle span deflection, and the failure mode, crack pattern.

The relationships between the applied load and displacement of the girder predicted from the numerical simulation was compared with the results obtained from the experiment in Fig. 19. There was a reasonable consistency between the experimental and simulation results. In the elastic stage, the load values increase linearly with the deflection. The predicted result of the FE model was close to the experimental result in terms of both the load and deflection. In the plastic stage, the load-displacement curve in the FE analysis showed a similar trend to that obtained in the experiment, which provided a slightly higher inclination angle compared to the numerical result. The experimental value reached the final load of 157 kN, and the value of the displacement in the center of girder was 84 mm. Regarding the numerical results, the applied load was estimated to be 160 kN at the corresponding the maximum displacement of 110 mm.

The failure mode in the simulation result was the compressive failure which was similar to the experiment result. In addition, Fig. 20 shows the crack patterns of the girder from the DIANA analysis and bending test. The ranges of the appearance of bending cracks and shear cracks in the actual test were similar to results by the analysis. The first cracks were seen in the flexure area after new flexure and shear cracks occurred when the load increased. These cracks tended to expand the compression zone.

The numerical results agreed well with those obtained from the experiment. Therefore, the proposed model could provide a reasonable outcome. This work is a fundamental first step in the parametric analyses presented in the following section.

Fig. 19 Load-displacement relationship at the middle of span.

Fig. 18 Stress-strain curve for concrete.

Table 6 Cases to parametric study.

Case study	Material properties					Bond-slip behavior	
	Concrete		CFCC				
	Compressive strength (N/mm ²)	Elastic modulus (kN/mm ²)	Tensile strength (N/mm ²)	Fracture energy (N/mm)	Breaking load (kN)	Elastic modulus (kN/mm ²)	
Reference case: Concrete average + CFCC average	75.1	37.2	4.1	0.1145	156.8	132.5	
Case 1: Concrete average + CFCC max	75.1	37.2	4.1	0.1145	161	132.6	-
Case 2: Concrete average + CFCC min	75.1	37.2	4.1	0.1145	148.6	130	-
Case 3: Concrete max + CFCC average	79.8	39.7	4.3	0.1168	156.8	132.5	-
Case 4: Concrete min + CFCC average	70.4	34.6	3.9	0.1121	156.8	132.5	-
Case 5: Concrete average + CFCC,E=0.8	75.1	37.2	4.1	0.1145	156.8	106	-
Case 6: Concrete E=0.8 + CFCC average	75.1	29.8	4.1	0.1145	156.8	132.5	-
Case 7: Concrete average + CFCC,T=0.8	75.1	37.2	4.1	0.1145	125.4	132.5	-
Case 8: Concrete C=0.8 + CFCC average	62	37.2	3.6	0.1074	156.8	132.5	-
Case 9: Concrete average + CFCC average, Bond-slip model 1	75.1	37.2	4.1	0.1145	156.8	132.5	DSSY=2975220 N/mm ³ >DSSX= 648.11 N/mm ³
Case 10: Concrete average + CFCC average, Bond-slip model 2	75.1	37.2	4.1	0.1145	156.8	132.5	DSSY=DSSX= 2975220 N/mm ³

12. Parametric studies

12.1 Overview

This section describes the factors influencing the results of the numerical simulation as well as the behavior of the structure. First, we analyzed the model results when different input data were used. Second, we examined the behavior of the structure as the parameters of the main material were changed. Finally, we examined the difference in the simulation results with or without the bond-slip model for the CFCC tendon.

In this part, ten models were simulated as shown in Table 6. In Sections 8 and 9, to investigate the compres-

sive strength of concrete, the samples were taken at two different positions on the girder while four samples were tested to determine the tensile strength of the CFCC, and each sample had a different data set. Thus, model cases 1, 2, 3, and 4 were used to study the change in the simulation results when different data sets were selected as inputs. Case 1 used the average result for concrete with the largest data in the CFCC parameters. Case 2 was the combination of the average data set of concrete and the smallest CFCC data set. The largest and smallest data sets in the results for concrete were combined with the CFCC average data set in cases 3 and 4, respectively.

Fig. 20 Crack pattern due to loading test and DIANA predicted.

For clarifying the behavior of the girder, cases 5, 6, 7, and 8 were conducted with a modification of the strengths and moduli of the materials. Specifically, in cases 5 and 6, the simulation model used the input data with a 20% modulus reduction in the CFCC and concrete, respectively. In contrast, when the modulus was unchanged, the tensile strength of the CFCC was only 80% of the average tensile strength in case 7. Regarding the concrete in case 8, the modulus was maintained, the compressive strength decreased by 20%, and the tensile strength and fracture energy decreased.

Another highlight of this study is the implementation of a bond-slip reinforcement instead of the regular embedded model. Cases 9 and 10 were added to study the bond-slip model in DIANA. The difference in the ultimate behavior in the cases where the CFCC tendons were produced with and without bond-slip properties was investigated. Bond is the expression used to express the interaction and transfer of force between reinforcement and concrete. An excellent bond no longer exists between the concrete and reinforcement bars. Therefore, the bond-slip reinforcement models are valid when there is a relative displacement between the concrete and reinforcement nodes. Bond slip was introduced to the numerical model by defining a bond-slip material. The bond-slip reinforcement elements reproduced the stirrups and CFCCs (DIANA 2017). Regarding the bond-slip interface failure model, this study applied the cubic function according to Dörr (1980) to describe the relations between shear traction and slip. In particular, Dörr (1980) proposed a polynomial relationship between shear traction and slip, which shows a limit if the slip is larger than a specific value, Δu_t^0 . This bond-slip law is given by a cubic function:

$$t_i = \begin{cases} c \left(5 \left(\frac{\Delta u_t}{\Delta u_t^0} \right) - 4.5 \left(\frac{\Delta u_t}{\Delta u_t^0} \right)^2 + 1.4 \left(\frac{\Delta u_t}{\Delta u_t^0} \right)^3 \right) & \text{if } 0 \leq \Delta u_t < \Delta u_t^0 \\ 1.9c & \text{if } 0 \leq \Delta u_t < \Delta u_t^0 \end{cases} \quad (3)$$

where t_i denotes the bond stress, value c is the parameter c , and value Δu_t^0 is the shear slip Δu_t^0 at which the curve reaches a plateau. DIANA recommends using

Fig. 21 Bond stress-slip relationship.

$c = f_{tk}$; thereby, the maximum value for the shear traction t_i equals $1.9f_{tk}$, in which f_t is the tensile strength of the concrete (N/mm^2). Moreover, the recommended value for Δu_t^0 is 0.06 mm. **Figure 21** shows the cubic bond-slip curve employed for the bond-slip interface failure model in this study.

In addition, interface elements in bond-slip models require the input of the linear stiffness parameters, which are the shear stiffness, DSSX, and the normal stiffness, DSNY. In particular, DSSX sets the relation between the shear traction and the relative shear displacement in the x-direction of the reinforcement (DIANA FEA BV 2017). The relationship between the normal traction and normal relative movement in the y-direction reinforcement was described by DSNY (DIANA FEA BV 2017). The dimension of the stiffness moduli is force per area per length, i.e., stress per length, e.g., N/mm^3 . Regarding the cubic bond-slip formulation, the shear stiffness is calculated at the plateau of bond-slip curve, which corresponds to a slip of 0.06 mm. The formula for DSSX calculation for the cubic bond-slip formulation was proposed by DIANA (2017) as the following.

$$\text{DSSX} = 5 \frac{1.9 \times f_{tk}}{\Delta u_t^0} \quad (4)$$

where DSSX, f_{tk} , and Δu_t^0 stand for the shear stiffness (N/mm^3), tensile strength (N/mm^2), and shear slip at which the curve reaches a plateau (mm), respectively.

Regarding the normal stiffness calculation, this study employs an approach proposed by Eriksen and Kolstad (Eriksen and Kolstad 2016), which assumed that DSNY is the concrete resistance of the reinforcement penetrating and crushing the concrete. DSNY was obtained for each CFCC tendons using Eq. (5).

$$\text{DSNY} = \frac{E_c}{2R} \times 10^3 \quad (5)$$

where DSNY, E_c , and R are related to the normal stiffness (N/mm^3), Young's modulus (N/mm^2), and the radius of each tendon (mm), respectively. In the case 9, DSNY was higher than DSSX while DSSX and DSSY were almost equal in case 10.

To compare the different simulation scenarios, the results of the model described in Section 11 were employed as the reference values, and the results obtained when the concrete reaches its extreme concrete compression fiber strain of 0.0035 were the comparison values (Japanese Road Association 2002).

12.2 Result and discussion

Compressive failure mode was observed in ten cases of simulation and all of them were similar to the result obtained from the experiment. The following paragraphs discuss the ultimate load and displacement between simulation cases and the reference case.

Figure 22(a) serves to compare the simulation results

Fig. 22 FE-predicted the load-displacement relationships.

when the input data of CFCCs were changed (which include the ultimate load and elastic modulus) corresponding to the tensile specimens of CFCCs for obtain-

ing the maximum and minimum breaking force values. That there was no change in the simulation result among the three cases. The result of case 1 was similar to the

results of case 2 and close to the reference case. The ultimate load of the girder was 160 kN in case 1 and 159 kN in case 2. These results may be due to the small difference between the parameters of the three studied cases.

Regarding cases 3 and 4, the input data was based on the results of the concrete compression test after the bending test. For each sample, the compressive strength, elastic modulus, and Poisson's ratio were used as the properties of concrete in DIANA, and the results are shown in **Fig. 22(b)**. It can be seen that these parameters almost affected the simulation scenarios, but this difference was small at the inclination angle of the elastic stage and the ultimate load of the girder compared to the reference model. The ultimate loads in case 3 and case 4 were 161 kN and 155 kN, respectively.

From the above analysis, it can be concluded that the position of the samples and sample experiments, which provide input data to the model, as well as the accuracy of the numerical simulation method, play an important role in generating the results.

Figures 22(c) and 22(d) show the simulated results when the elastic moduli of the CFCC and concrete decreased by 20% in case 5 and case 6, respectively. In addition, **Figs. 22(e) and 22(f)** show the simulation results of cases 7 and 8, in which the breaking force of the CFCC decreased by 20% and the concrete compressive strength reduced by 20%. In **Fig. 22(c)**, no significant change was observed in the elastic stage after changing the CFCC parameters. However, in the plastic stage, there was a significant difference between the load-displacement relationships. The inclination angle was lower than that in the reference case. In contrast, in **Fig. 22(d)**, when the modulus of concrete decreased, there was a significant change in the load-displacement relationship in both stages. As the modulus of the concrete decreased, the initial stiffness of concrete reduced, thereby the inclination angle in the elastic stage of the load-displacement curve decreased. The final load of the girder in this case at 150 kN was also lower than the value in the reference case. In case 7, the decrease in the CFCC breaking force caused the load carrying capacity to decrease to 152 kN, as shown in **Fig. 22(e)**. However, the displacement in this case (98 mm) was lower compared to case 5. **Figure 22(f)** also shows the change in the relationship between the load and displacement of the girder in case 8. The decrease in the concrete compressive strength caused a corresponding decrease in the tensile strength of the concrete and fracture energy. Therefore, the ultimate load of the girder was predicted to be 145 kN; this number was 9% lower than that in the reference model. However, the change in the shape of the load-displacement curve occurred significantly in the plastic stage. Based on the above discussion, the change in the modulus and strength of the concrete and CFCC have a significant impact on structural behavior.

Figures 22(g) and 22(h) depict the load-displacement relationship of the reference case and bond-slip models

1 and 2, respectively. There was a slight difference in case 9 when the ultimate load was lower than that in the reference case. However, when shear stiffness increased highly, equal to the value of normal stiffness, this difference became quite small. The result in case 10 with bond-slip of was similar to the numerical result with the embedded model. The initial crack occurred in the load step from 80–85 kN in both cases while the load value in the experiment was 82.8 kN. Therefore, the model, which was selected for simulation namely bond-slip of Dörr model or embedded model, also have affected to the results of numerical simulation.

13. Conclusions

In this study, the series of experiments was conducted on the PC girder using a CFCC tendon, which was exposed to an actual corrosive environment for nearly 30 years. In addition to the estimation of the effective stress, the transmission length test, compressive strength test of concrete, the destructive bending experiment was carried out on the main girder and the mechanical and chemical properties of CFCCs removed from the main girder after the flexural experiment were examined. From the results and discussions, the load carrying capacity and the durability of the girder were confirmed. Furthermore, the numerical analysis by DIANA was performed for a deeper understanding of the variations in the structural behavior of the girder and the factors impacted on the accuracy of the numerical results.

The remarkable conclusions in this research are as follows.

- (1) From the load-displacement relationship obtained from the destructive bending experiment, the results of this test and the results at six years after the construction were almost overlapped, but the ultimate load were approximately 6% lower than that at six years after construction. However, the results exceeded the design value at the time of construction and it was judged that there was no problem with the load carrying capacity of the girder.
- (2) The transmission length of the specimen was determined to be 500 mm, which was smaller than 65ϕ . The same result was obtained in the test conducted six years after the original construction, and it became clear that the transmission length and the bond between CFCCs and concrete did not change.
- (3) From the analysis results of the salt content in the core collected from the lower surface of the main girder, the chloride ion concentration was estimated to be approximately 6.0 kg/m^3 at the position of the stirrup rebar (reinforcement cover was 27.5 mm).
- (4) It was confirmed that the rupture load obtained from the tensile test of the CFCCs, which was taken out after the flexural experiment, was comparable to the previous results, and residual tension load capacity was observed.
- (5) There was no position where the coatings and car-

bon fibers themselves seemed to be deteriorated, according to the FE-SEM observations of the CFCC tendons. It was confirmed that there was no problem in the CFCC surface and carbon fiber surface. In addition, from the FT-IR analysis results, no significant change, such as the disappearance of the main peak or appearance of a new peak, could be confirmed. Therefore, it was considered that deterioration caused by chemical structural change did not occur.

- (6) The proposed modeling method using DIANA was effectively employed to predict the structural behavior of the girder. The result of the FE model was close to the experimental result and this mode was the basis for the parametric analyses.
- (7) From the results of the parametric studies, the position of the samples and tensile and compressive experiments impacted the results and the accuracy of the numerical simulation because these factors provided the input data for the numerical simulation. When the modulus, the strength of concrete or CFCC decreased, the final load of the girder also decreased, and the relationship between the load and displacement changes. The models in these cases are an effective basis to predict the robustness of the main girders in the future.

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Reference

- Abdelrahman, A. A. and Rizkalla, S. H., (1998). "Serviceability of concrete beams prestressed by carbon fiber-reinforced-plastic bars." *ACI Structural Journal*, 94, 447-454, ISBN 0889-3241.
- Corte, W. D. and Van Bogaert, P., (2005). "Evaluation of an experimental CFRP pre-stressed beam and slab road bridge." *Composites, Part B: Engineering*, 36(2), 91-98. doi:10.1016/j.compositesb.2004.07.001
- DIANA, (2017, April). "User's manual of FX+ for DIANA." version 9.6, Retrieved from <https://dianafea.com/manuals/d101/Diana.html>
- DIANA FEA BV, (2017). "User's Manual." Release 10.2.
- Dörr, K. (1980). "Ein Beitrag zur Berechnung von Stahlbetonscheiben unter besonderer Berücksichtigung des Verbundverhaltens." Thesis (PhD). University of Darmstadt.
- Enomoto, T., Grace, N. F. and Harada, T., (2012). "Life extension of prestressed concrete bridges using CFCC tendons and reinforcements." *Proceedings of Conference on FRP Composites in Civil Engineering, Rome Italy*.
- Eriksen, M. and Kolstad, M., (2016). "Investigation of cracking behavior in reinforced concrete panels with bond-slip reinforcement." Thesis (MS). Norwegian University of Science and Technology.
- Erki, M. A. and Rizkalla, S., (1993). "FRP reinforcement for concrete structures." *ACI Concrete International*, 15(6), 48-53.
- F. Green, M., (2017). "Long-term applications of CFRP prestressing in Canada." *Fourth Conference on Smart Monitoring, Assessment and Rehabilitation of Civil Structures*.
- Futakuchi, T., Komada, S., Kiuchi, T. and Matsumoto, K., (1991). "Report on loading test of an actual girder of a PC bridge using new material." *Proceedings of 2nd Symposium on Japan Prestressed Concrete Institute*, 307-310. (in Japanese)
- Grace, N. F., Jensen, E., Eamon, C., Enomoto, T. and Shi, X., (2012). "Life cycle cost analysis of prestressed concrete bridges using CFRP reinforcement." *Proceedings of CICE 2012-6th International Conference on FRP composites in Civil Engineering*, 1-8. [http://www.iifc-hq.org/proceedings/CICE_2012/09_Prestressing with FRP/09_033 Grace, Jensen LIFE CYCLE COST ANALYSIS OF PRESTRESSED CONCRETE.pdf](http://www.iifc-hq.org/proceedings/CICE_2012/09_Prestressing%20with%20FRP/09_033%20Grace,%20Jensen%20LIFE%20CYCLE%20COST%20ANALYSIS%20OF%20PRESTRESSED%20CONCRETE.pdf).
- Grace, N. F., Navarre, F. C., Nacey, R. B., Bonus, W. and Collavino, L., (2002). "Design - construction of Bridge Street Bridge - First CFRP bridge in the United States." *PCI Journal*, 47 (2002) 5, 20-35.
- Japanese Road Association, (2002). "Road-bridge standard specifications I Common bridge III Concrete bridge." (in Japanese)
- JIS A 1154, (2012). "Methods of test for chloride ion content in hardened concrete." Tokyo: Japanese Industrial Standards Committee.
- JSCE, (1998). "Recommendation for design and construction of concrete structures using continuous fiber reinforcing materials." Japan Society of Civil Engineers.
- JSCE, (2012). "Standard specifications for concrete structures: design code." Concrete committee: Japan Society of Civil Engineers.
- Kanda, H., Kiuchi, T. and Matsumoto, K., (1995). "Report on loading test of an actual girder of a PC bridge using new material." *Proceedings of 5th Symposium on Japan Prestressed Concrete Institute*, 529-532. (in Japanese)
- Katarzyna, R. and Łukasz, Z., (2015). "Prestressing concrete structures with CFRP composite tendons." *Emerging Technology Series Prestressing*, 1-35.
- Kimura, H., Shiratori, N. and Takagi, H., (1990). "Characteristics of carbon fiber composite cable for concrete reinforcement." *Proceedings of the Japan Concrete Institute*, 12-1, 1015-1018. (in Japanese)
- Marti-Vargas, J. R., Caro, L. A. and Serna, P., (2013). "Experimental technique for measuring the long-term

- transfer length in prestressed concrete.” *Strain*, 49(2), 125-134. doi:10.1111/str.12019
- Niitani, K., Watase, H., Sakata, K. and Ayano, T., (2009). “Study on effective stress estimation method for concrete members.” *Concrete Research and Technology*, 20-2, 27-37. (in Japanese)
- Rizkalla, S. H. and Tadros, G., (1994). “A smart highway bridge in Canada.” *Concrete International*, 6(4), 37-42.
- Santoh, N., Kimura, H., Enomoto, T., Kiuchi, T. and Kuzuba, Y., (1993). “Report on the use of CFCC in prestressed concrete bridge in Japan.” *Fiber-Reinforced-Plastic (FRP) Reinforcement for Concrete Structures, Proceedings of the International Symposium (FRPRCS)*, ACI, SP-138, 895-911.
- Yaman, T. S., (2016). “Behaviour of precast concrete beams prestressed with CFRP strands.” *Journal of the Croatian Association of Civil Engineers*, 68(10), 775-786. doi:JSE.1624.2016
- Yamashita, T., Kiuchi, T., Inukai, H. and Iwasaki, T., (1989). “PC bridge with new material - construction of Shinmiya Bridge.” *Prestressed Concrete*, 31(2), 71-78. (in Japanese)