

Design-Construction of Bridge Street Bridge – First CFRP Bridge in the United States



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The design, fabrication, and erection – and the research leading up to this cutting-edge technological achievement in civil engineering – are presented, as well as the long-term monitoring program and ongoing research that will continue. The Bridge Street Bridge in Southfield, Michigan, is the first vehicular concrete bridge ever built in the United States that uses carbon fiber reinforced polymer (CFRP) material as the principal structural reinforcement. The project consists of two parallel bridges – Structures A and B – over the Rouge River in the City of Southfield. Both structures use three skewed spans, each over 62 m (204 ft) long, to carry vehicular traffic. Structure A consists of a new substructure as well as a new superstructure, and incorporates five equally spaced conventional AASHTO Type III girders in each of its three spans. Its cast-in-place concrete deck slab is placed continuously across the three spans. Structure B consists of 12 special double-tee (DT) girders (four per span) using pretensioned Leadline™ tendons and post-tensioned carbon fiber composite cable (CFCC)™ strands. This project recently won PCI's Harry H. Edwards Industry Advancement Award.

The Bridge Street Bridge consists of two separate parallel and independent bridges (Structures A and B) over the Rouge River in the City of Southfield, Michigan (see Figs. 1, 2, and 3). Both bridges comprise three spans skewed at 15 degrees over a 62 m (204 ft) length and carry traffic for an industrial subdivision. Structure A was constructed first, and it consists of a new substructure and a new superstructure that incorporates five equally spaced conventional AASHTO Type III precast concrete I-girders in each of the three spans, with a continuous cast-in-place concrete deck slab.

Structure B consists of four special precast, prestressed double-tee (DT) girders in each of the three spans configured as simply supported spans. Each DT girder is structurally reinforced using pretensioned carbon fiber reinforced polymer (CFRP) Leadline™ tendons* and post-tensioned CFRP carbon fiber composite cable (CFCC)™ strands† in both longitudinal and transverse directions. The non-prestressed reinforcement in the girders and deck structure consists of CFCC strands manufactured in bent configurations, straight CFCC reinforcing bars, CFRP NEFMAC™ grid reinforcement,‡ and stainless steel reinforcing bars for stirrups.

Ultimately, this project is expected to demonstrate that the use of CFRP material as structural reinforcement can increase the service life of highway bridges, thereby reducing construction-related safety concerns and annual maintenance costs. To optimize bridge durability even further, a quality review of materials was conducted and decisions were made to require



Fig. 1. (top photo) Bridge Street Bridge, Southfield, Michigan, looking south, with CFRP Structure B on the right. (bottom photo) Side view of Structure B.

* Leadline tendons manufactured by Mitsubishi Chemical Corporation, Japan.

† CFCC strands manufactured by Tokyo Rope Mfg. Co., Ltd., Japan.

‡ NEFMAC grid reinforcement manufactured by Autocon Composites, Inc., Ontario, Canada.

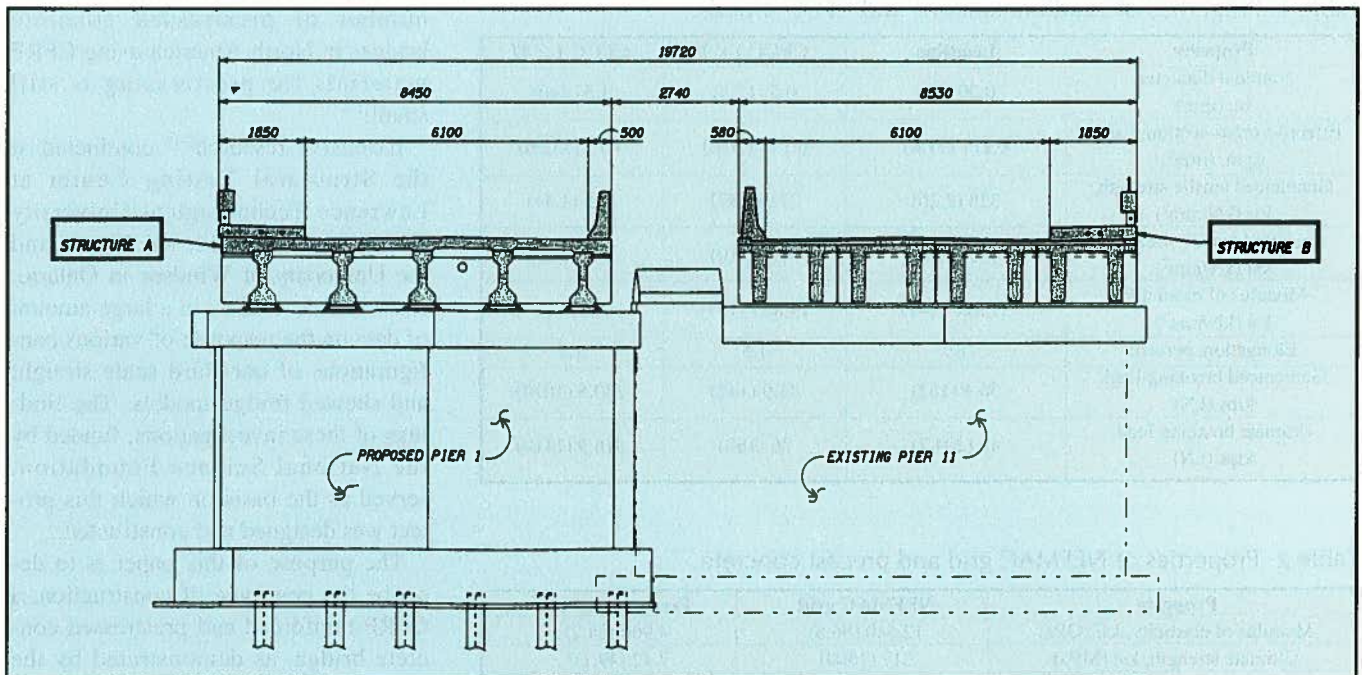


Fig. 2. Cross section of Bridge Street Bridge: Structure A has conventional AASHTO Type III girders and a new substructure. Structure B is the CFRP double tee bridge supported by an existing substructure.

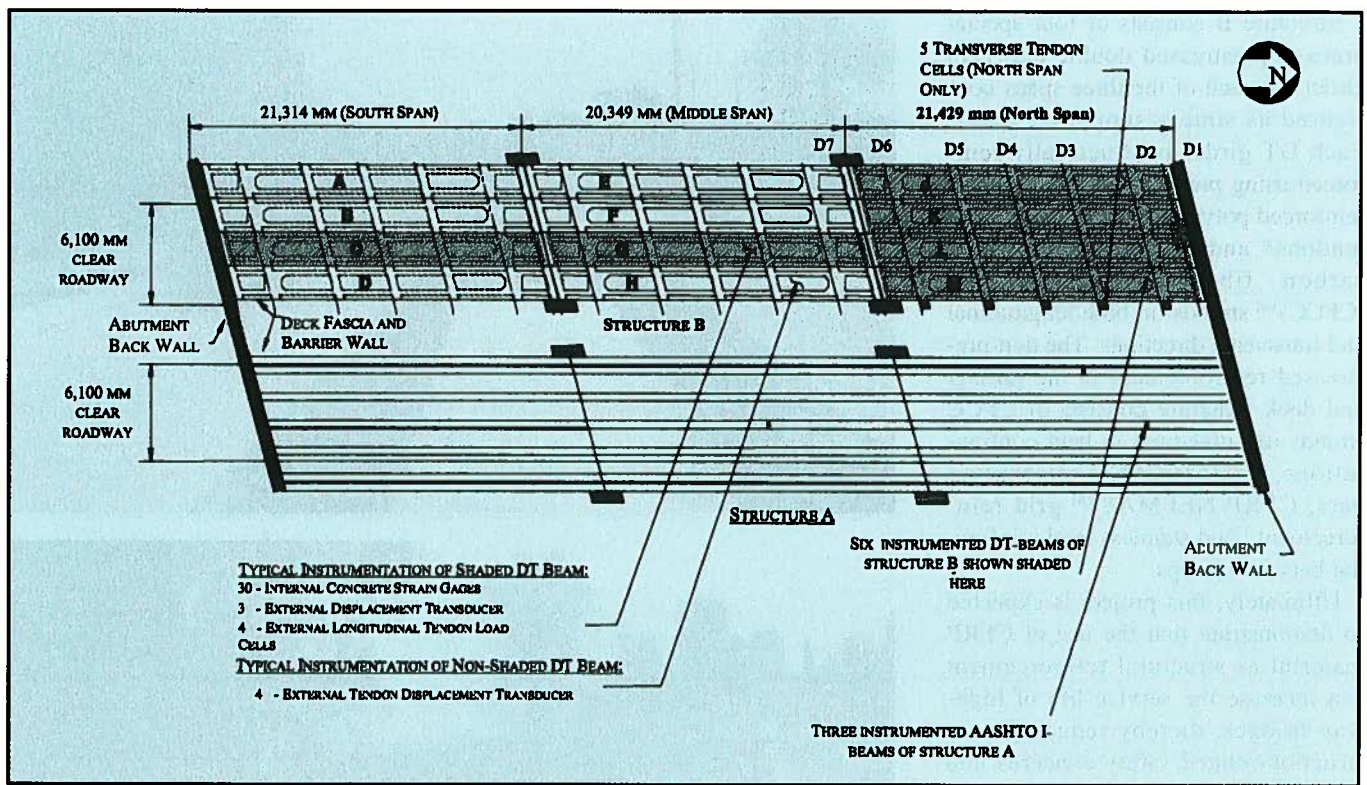


Fig. 3. Plan view of the bridge, with instrumented CFRP double tee girders and conventional AASHTO I-girders identified (dimensions in SI units).

very high quality concrete and to allow metallic reinforcement made only of stainless steel.

Hollowcore Incorporated (HI), Windsor, Ontario, Canada, fabricated all the girders for both structures,

while Construction Technology Laboratories, Inc. (CTL), Skokie, Illinois, installed the instrumentation for long-term monitoring.

The current state of the art in research¹ underscores an increasing in-

terest in advancing the technology of civil engineering infrastructure projects, including bridge girders and slabs that use advanced carbon fiber reinforced polymer (CFRP) materials. Despite the large-scale worldwide research to find the suitable fiber reinforced polymeric (FRP) materials, the number of prestressed concrete bridges in North America using CFRP materials for prestressing is still small.¹⁻⁴

Extensive research⁵⁻¹⁶ conducted at the Structural Testing Center at Lawrence Technological University (LTU) in Southfield, Michigan, and the University of Windsor in Ontario, Canada, has resulted in a large amount of data on the response of various configurations of one-third scale straight and skewed bridge models. The findings of these investigations, funded by the National Science Foundation, served as the basis on which this project was designed and constructed.

The purpose of this paper is to describe the new type of construction, a CFRP reinforced and prestressed concrete bridge, as demonstrated by the Bridge Street Bridge (Structure B), during various phases of its construc-

Table 1. Properties of Leadline tendons and CFCC strands.

| Property | Leadline | CFCC 1 × 7 | CFCC 1 × 37 |
|---|--------------|--------------|--------------|
| Nominal diameter, in. (mm) | 0.39 (10) | 0.5 (12.5) | 1.57 (40) |
| Effective cross-sectional area, sq in. (mm ²) | 0.111 (71.6) | 0.118 (76.0) | 1.17 (752.6) |
| Guaranteed tensile strength, ksi (kN/mm ²) | 328 (2.26) | 271 (1.87) | 205 (1.41) |
| Specified tensile strength, ksi (kN/mm ²) | 415 (2.86) | 305 (2.10) | 271 (1.87) |
| Modulus of elasticity, ksi (kN/mm ²) | 21,320 (147) | 19,865 (137) | 18,419 (127) |
| Elongation, percent | 1.9 | 1.5 | 1.5 |
| Guaranteed breaking load, kips (kN) | 36.4 (162) | 31.9 (142) | 240.5 (1070) |
| Ultimate breaking load, kips (kN) | 46 (204.7) | 36 (160) | 316.9 (1410) |

Table 2. Properties of NEFMAC grid and precast concrete.

| Property | NEFMAC grid | Precast concrete |
|----------------------------------|---------------|------------------|
| Modulus of elasticity, ksi (GPa) | 12,540 (86.5) | 4,960 (34.2) |
| Ultimate strength, ksi (MPa) | 217 (1500) | 7.12 (49.1)* |
| Ultimate strain, percent | 1.8 | 0.25 |

* Refers to average 28-day strength of concrete.

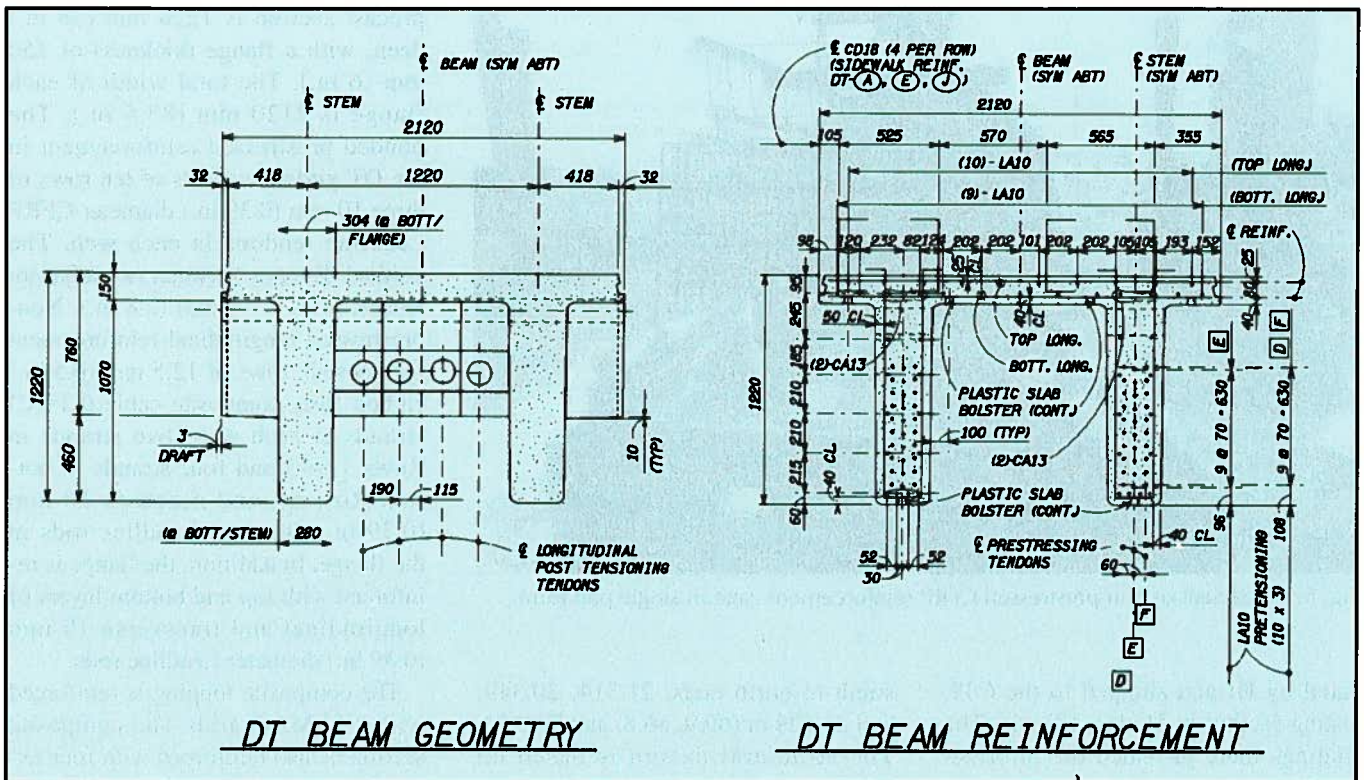


Fig. 4. Cross-sectional details of Structure B (dimensions in SI units).

tion. In addition, the values of various parameters such as pretensioning forces in Leadline tendons, concrete strains, early-age deflections, and post-tensioning forces in external unbonded CFCC strands (longitudinal and transverse), measured during the various construction stages, are presented and discussed.

FULL-SCALE DT GIRDER TEST

Because of the lack of design guidelines for CFRP prestressed concrete bridges in the United States and elsewhere, several design assumptions^{5,10} had to be made during the design stage and throughout the preparation of the construction documents. To ensure the

success of the project, a multitask program was assembled for modeling, testing, monitoring, and acquiring reinforcement and special services.

As part of this program, a full-scale DT girder (the same design as that of the 12 DT girders used on this project) was constructed, instrumented, and tested to failure. This girder was fabri-

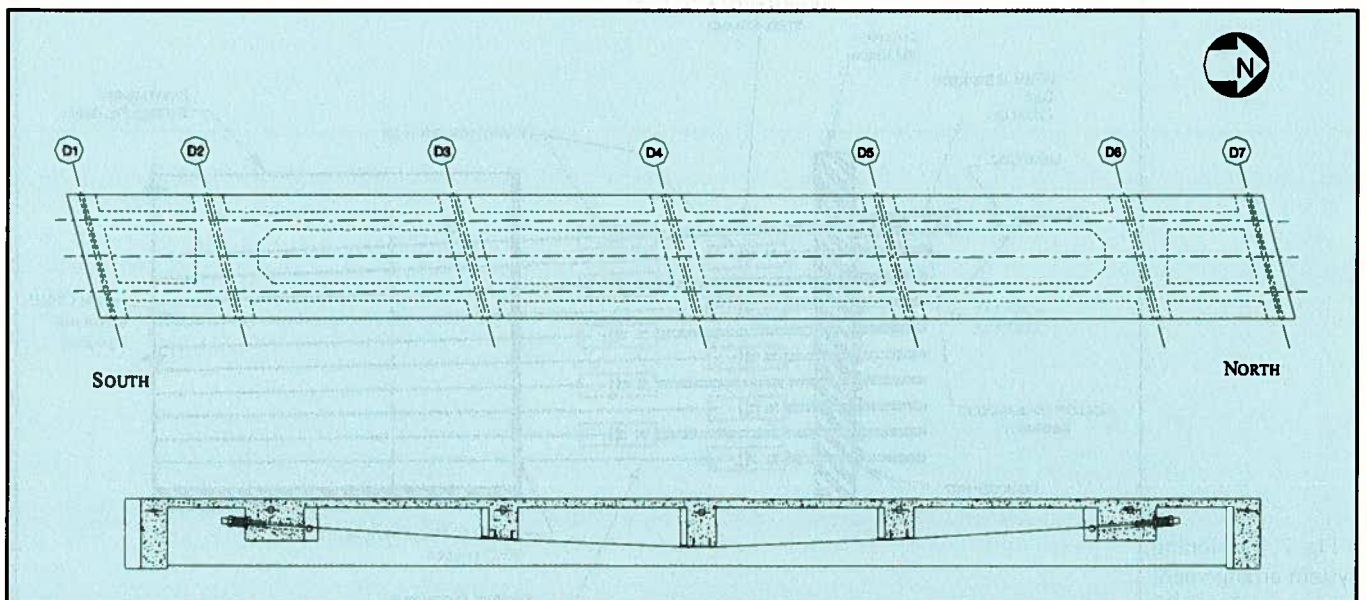


Fig. 5. Plan and elevation of Structure B, with transverse diaphragms identified.



Fig. 6. Placement of non-prestressed CFRP reinforcement cage in single pan form.

cated by HI and shipped to the CTL testing facility in Skokie, Illinois. The findings there provided the information needed to confirm the design assumptions and aided the design team in refining the fabrication details of the DT girders to be used for the project. The results of this testing program are not presented in this paper but can be found elsewhere.¹⁵⁻¹⁶

BRIDGE DESIGN

The CFRP bridge, Structure B, comprises spans with lengths of, from

south to north ends, 21.314, 20.349, and 21.429 m (69.9, 66.8, and 70.3 ft). The structural design is based on Michigan MS-23 truck loading (approximately 1.25 times HS-20). The 28-day design strength is 52 MPa (7500 psi) for the precast concrete girders and 38 MPa (5500 psi) for the composite concrete topping. The cross-sectional details of Structure B are shown in Fig. 4.

The bridge cross section consists of four precast DT sections and a minimum 75 mm (3 in.) thick non-continuous deck slab. As shown in Fig. 4, the

precast section is 1220 mm (48 in.) deep, with a flange thickness of 150 mm (6 in.). The total width of each flange is 2120 mm (83.5 in.). The bonded prestressed reinforcement in the DT girders consists of ten rows of three 10 mm (0.39 in.) diameter CFRP Leadline tendons in each web. The vertical distance between two adjacent tendon rows is 70 mm (2.8 in.). Non-prestressed longitudinal reinforcement includes six rows of 12.5 mm (0.5 in.) carbon fiber composite cable (CFCC) strands in each web (two strands in Rows 1 to 5 and four strands in bottom Row 6), and nineteen 10 mm (0.39 in.) diameter Leadline rods in the flange. In addition, the flange is reinforced with top and bottom layers of longitudinal and transverse 10 mm (0.39 in.) diameter Leadline rods.

The composite topping is reinforced with NEFMAC grids. The composite section is also reinforced with four externally draped 40 mm (1.57 in.) diameter unbonded CFCC post-tensioning strands. The tendons are draped longitudinally between Diaphragms D2 and D6 (see Fig. 5) and bear against the bottoms of Diaphragms D3, D4, and D5. At midspan (Diaphragm D4), the depth of these tendons from the top surface of the composite concrete is 1118 mm (44 in.).

Transverse unbonded CFCC post-tensioning strands were installed

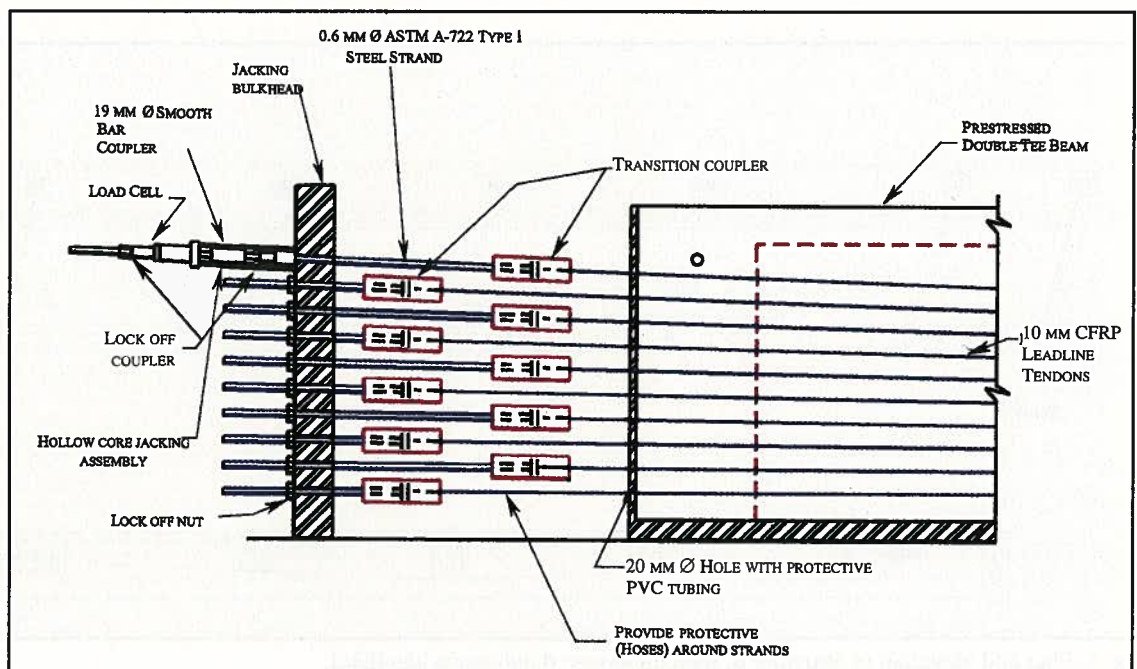


Fig. 7. Tensioning system arrangement for the CFRP Leadline tendons.

through each of the seven diaphragms in each span; these strands were 21.8 mm (0.86 in.) in diameter at end Diaphragms D1 and D7, and 40 mm (1.57 in.) in diameter at interior Diaphragms D2 through D6. These cables were inserted along the 15-degree skew through the four DT girders of each span.

For both transverse and longitudinal post-tensioning, the unbonded strands were stressed in stages to account for different loading conditions. At the precast plant, for example, longitudinal post-tensioning was partially applied to help prevent cracking during handling, shipping, and erection.

The properties of the CFRP Leadline tendons and CFCC strands are given in Table 1, while those of the NEFMAC grids and the precast concrete are presented in Table 2. The fabrication and instrumentation details of the bridge are presented in the following sections.

FABRICATION OF DOUBLE TEES

A single pan form was used to fabricate the 12 DT girders. Fig. 6 shows the placement of a non-prestressed CFRP reinforcement cage being placed into the single pan form at the precast plant. The pan form accommodated the two stems, top flange, and seven transverse integral diaphragms. The compressive cylinder strengths of concrete for the 12 girders varied slightly with an average 28-day cylinder strength of 55.8 MPa (8100 psi). The sequence of the fabrication procedure of the 12 girders is given below:

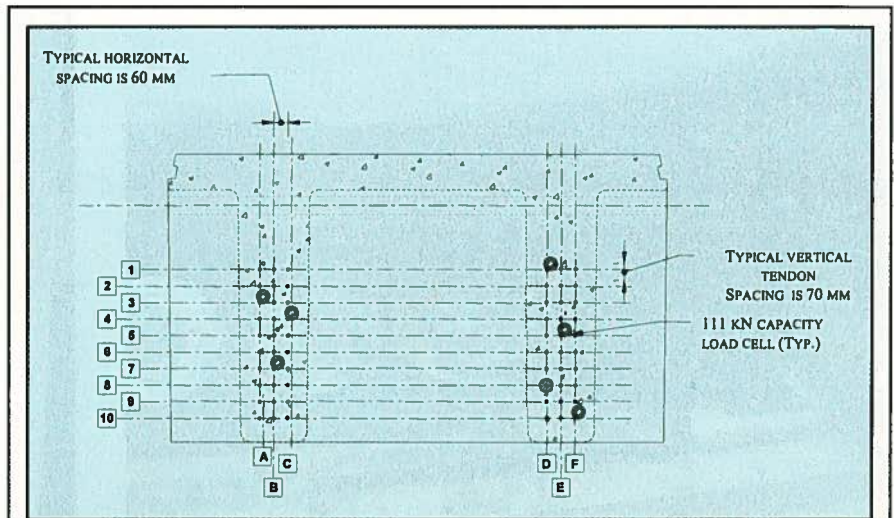
1. Installing the non-prestressed CFRP reinforcement, epoxy-coated stainless steel stirrups, prestressing Leadline tendons, and other embedded items (see Fig. 6).

2. Prestressing the Leadline tendons from the live end (see Fig. 7) and monitoring the prestressing forces at the dead end (see Fig. 8).

3. Installing the vibrating wire strain gauges on the Leadline tendons (see Fig. 9).

4. Placing and curing the concrete.

5. Releasing the Leadline tendons after the concrete achieved the desired strength.



(a) Dead end section with row numbers of tendons



(b) Prestressing force measurements at dead end

Fig. 8. Monitoring of prestressing forces of Leadline tendons at dead end.

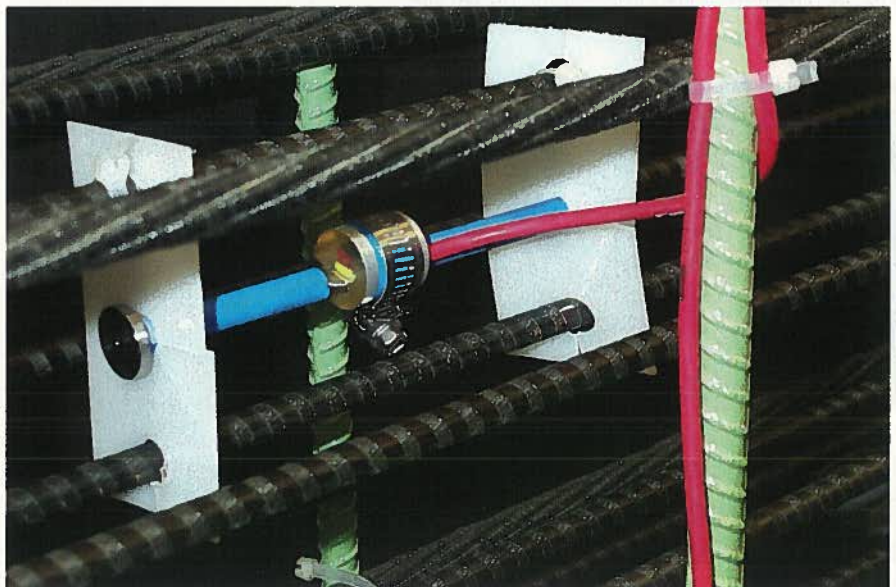


Fig. 9. Vibrating wire strain gauge on Leadline tendons.



Fig. 10. Post-tensioning at the live end (left) and load cells installed at the dead end (right) for the longitudinal CFCC strands.



Fig. 11. Erection of DT girders at the bridge site.



Fig. 12. Installed DT girders with external CFCC post-tensioning strands in place.

6. Removing the prestressed girder from the form.

7. Installing the longitudinal CFCC post-tensioning strands and applying 60 percent of the total post-tensioning force just before shipping (see Fig. 10).

Before stressing each Leadline prestressing tendon, an aluminum sleeve was pressed on to both ends with a tapered steel wedge installed around it. A threaded coupling sleeve was crimped on the ends and inserted into steel guide sleeves that supported the tendons at the proper elevation and restrained their rotation.

The ends of each tendon were inserted into threaded anchor heads, which had a matching tapered interior surface. The dead end anchor head was then positioned into the curved support brackets and connected to the stressing bulkhead. The live end anchor head was positioned within a slotted plate assembly that accommodated the elongation of the Leadline tendons and also restrained tendon rotation (see Fig. 7).

Once supported, a short length of conventional seven-wire steel prestressing strand was threaded through the anchorage bulkhead and inserted into the transition coupler. The free end of the steel strand was anchored at the outside surface of the bulkhead using a standard strand chuck.

The 60 straight Leadline tendons



Fig. 13. Structure B completed and ready for traffic.

were stressed individually. The stressing sequence began with Row 10 (see Fig. 8a) and continued in the order of Rows 9, 1, 2, 5, 6, 3, 8, 4, and 7. Prior to prestressing, load cells were installed at the dead end of the selected tendons to monitor the applied prestress force. Fig. 8b shows the arrangement of the load cells at the dead end.

The prestressing force in each Leadline tendon was about 91 kN (20.5 kips) after seating losses. After the

concrete achieved adequate strength, the Leadline tendons were released in a specific sequence. The time of tendon release from girder to girder varied from 3 to 5 days. On average, the concrete strength at the time of release was 44.1 MPa (6400 psi).

The four externally draped post-tensioning CFCC strands were installed in each DT girder at the precaster's yard prior to shipping. Fig. 10 shows the installed CFCC strands (view from below the bridge). The draping of

these strands was achieved using a specially designed tendon slide plate and a tendon alignment shoe placed beneath Diaphragms D3, D4, and D5. Both the tendon slide plate and tendon alignment shoe are made of stainless steel.

The 13 mm (0.5 in.) thick steel tendon slide plate was embedded in the corresponding diaphragm using 12 x 150 mm (0.5 x 6 in.) headed studs and a guide pin. The strand slide plate and strand alignment shoe were connected

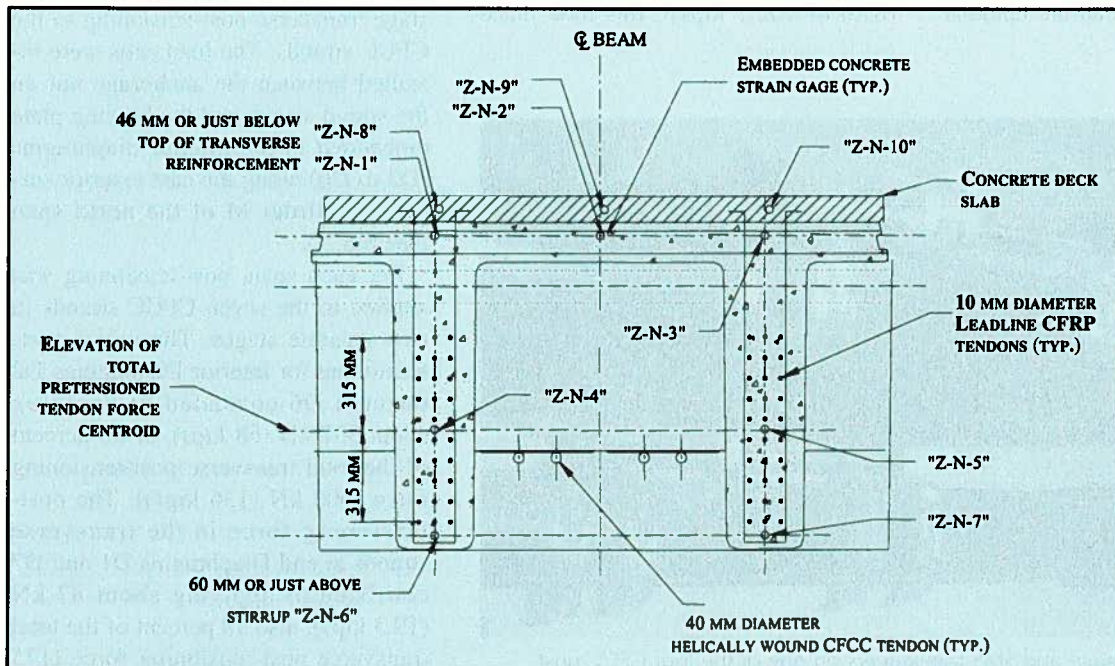


Fig. 14. Section through instrumented DT girder at north end quarter span (looking north).

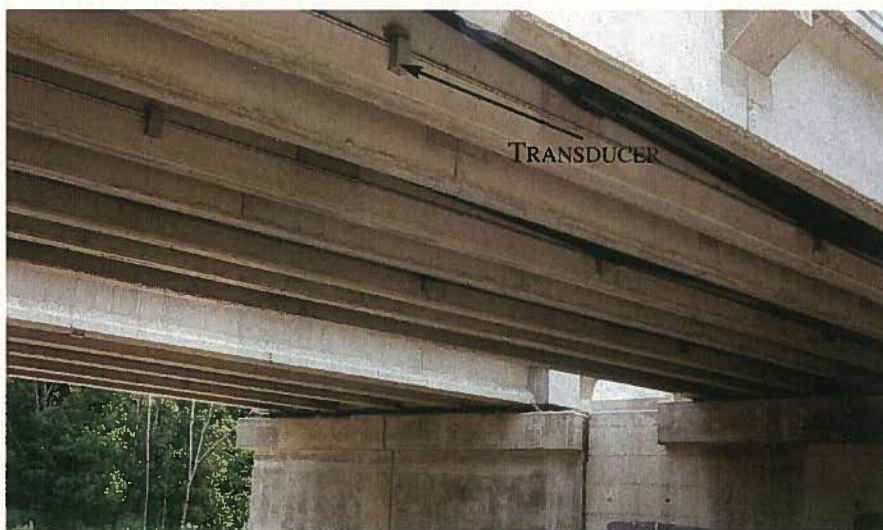


Fig. 15. Automated displacement transducer installed on Structure B.

to each other using the same guide pin connected to the diaphragm through the slide plate. The CFCC strands were raised from below and positioned within the strand alignment shoe. A neoprene elastomeric protective pad was placed between the alignment shoe and the supported length of the strand to prevent surface damage to the strand.

As part of the monitoring program, load cells were permanently installed to measure and monitor the force levels in the CFCC strands of six DT girders (the shaded girders identified in Fig. 3). In addition, strain gauges and displacement transducers were used to monitor the Leadline tendons

and the longitudinal CFCC strands in the remaining six DT girders (unshaded girders in Fig. 3).

Prior to post-tensioning each girder, all four 40 mm (1.58 in.) diameter CFCC strands were configured with load cells at one end. The load cells were installed between the anchorage nut on the tendon sleeve and the bearing plate embedded in Diaphragm D6 (see Fig. 10).

Post-tensioning was applied to the four longitudinal CFCC strands in two distinct stages. The initial post-tensioning consisted of applying approximately 60 percent of the total post-tensioning force value [274 of 457 kN (61.6 of 102.7 kips)]; this took place

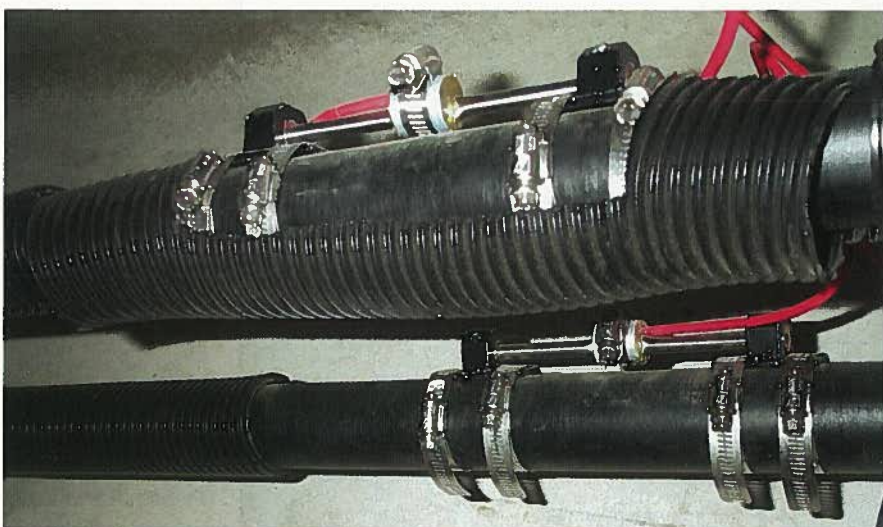


Fig. 16. Typical strain gauge and SMS transducers on one of the four CFCC post-tensioning strands.

just before the girders were to be shipped to the bridge site. The final 40 percent of post-tensioning was achieved at the bridge site after the composite deck slab was placed.

BRIDGE CONSTRUCTION

The unique construction method was based on a process developed and tested previously at LTU.⁵⁻⁸ The transportation of the 12 girders from the precast plant in Windsor, Ontario, to the bridge site in Southfield, Michigan, required a special barging arrangement. The girders were erected using two large capacity cranes at opposite ends of the bridge.

The four DT girders of the middle span were erected first using both cranes. The double tees of the north- and south-end spans, however, were handled independently by the crane stationed closer to that particular span.

Fig. 11 shows the erection of a DT girder for the south span, and Fig. 12 shows the installed girders with external post-tensioning strands in place. It should be noted that all the transverse post-tensioning strands were pulled through their sleeves immediately after erecting the girders and just prior to grouting the girder joints. Provisions were made to prevent grout from leaking into the sleeves.

Construction of the bridge deck continued with the application of first-stage transverse post-tensioning to the CFCC strands. The load cells were installed between the anchorage nut on the strand sleeve and the bearing plate embedded in the interior diaphragms (D2 to D6) along the east exterior surface of Girder M of the north span (see Fig. 3).

For each span, post-tensioning was applied to the seven CFCC strands in two separate stages. The initial post-tensioning for interior Diaphragms D2 through D6 consisted of applying about 307 kN (68 kips), or 50 percent of the total transverse post-tensioning force [605 kN (136 kips)]. The post-tensioning force in the transverse strands at end Diaphragms D1 and D7 consisted of applying about 87 kN (19.5 kips), also 50 percent of the total transverse post-tensioning force [175 kN (39 kips)].

The final transverse post-tensioning was applied after the composite deck slab of the bridge was placed.

After the completion of all post-tensioning operations, a sidewalk on one side of the bridge and a barrier railing on the other side were constructed. Next, a latex-modified surfacing mixture was installed over the top of the deck. After completing the railing on the sidewalk and performing a series of bridge load tests, Structure B was opened to traffic (see Fig. 13).

INSTRUMENTATION AND MONITORING

Because a bridge of this type had never been constructed before, the need for instrumentation and remote monitoring of critical parameters were identified early on. Monitoring the girders would occur from fabrication through erection and continue for five years thereafter. This program will ultimately provide relevant information on the serviceability of bridge structures that use CFRP materials. Only Structure B instrumentation is discussed in this paper. This section describes the various aspects of the DT girders responses during the various construction stages.

All 12 DT girders were instrumented and monitored during fabrication to measure and document forces and stress levels during prestressing operations. In addition, six girders were instrumented with both internal and external sensors for long-term monitoring. These are identified in Fig. 3. Most of the instrumentation was installed during the fabrication of the girders at the precast facility. The instrumentation has the following objectives for the five-year program.

1. Measure and monitor the prestressing load applied to the CFRP Leadline tendons.

2. Measure concrete strain distribution in the girder cross section and topping.

3. Measure girder camber and deflections during girder fabrication and bridge construction sequence.

4. Measure forces in the post-tensioning CFCC strands during construction.

5. Monitor the strain status of exter-



Fig. 17. NEFMAC grid placed over the flange of DT girders prior to placement of bridge deck slab.

nal transverse and longitudinal strands.

Measurement of Prestensioning Forces

Prestensioning forces were measured by a load cell installed between the fabricator's stressing jack and the anchorage (chuck) at the live end. In addition, load cells were installed at the dead end of selected tendons to monitor the applied prestensioning forces. Seven load cells were installed between the stressing bulkhead and the dead end anchorage chuck.

Measured prestensioning forces in Leadline tendons of the six instrumented girders (Girders C, G, J, K, L, and M) were recorded at 15-minute intervals from the time that prestressing was completed until the time of tendon release, which typically spanned a three-day period. Three of the six non-instrumented girders (Girders E, F, and H) were monitored until all of their tendons were tensioned, which typically took three to four hours. The remaining three non-instrumented girders (Girders A, B, and D) were monitored until the time of concrete placement.



Fig. 18. CFCC reinforcement cage for top concrete barrier (under handrail).

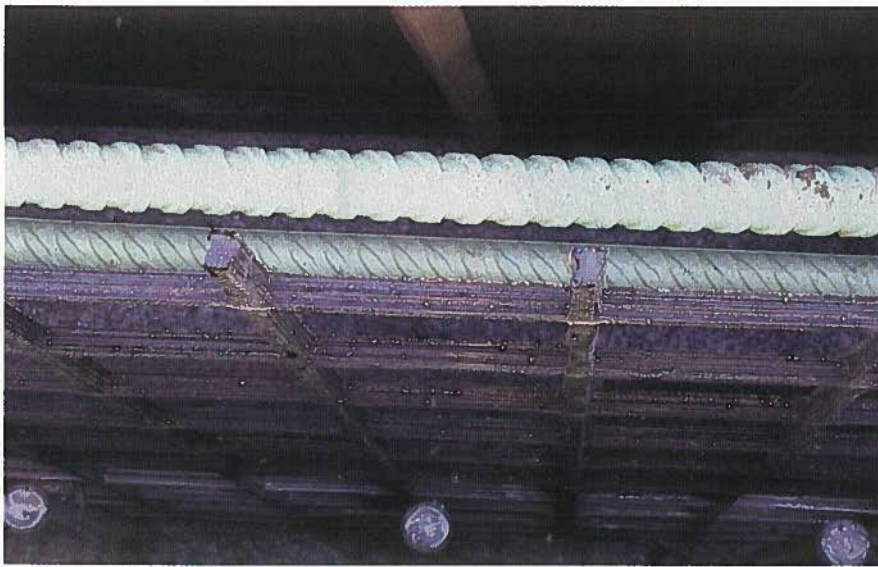


Fig. 19. NEFMAC reinforcement for the barrier wall (looking down into the formwork from the top).

Measurement of Concrete Strain

Embedded vibrating wire strain gauges with an effective gauge length of 152 mm (6 in.) were installed to measure the strain distributions along the depth of the girder cross sections. These gauges were positioned at the midspan and quarter points of each of the six instrumented DT girders. A total of 30 strain gauges per girder were installed. Of these, 21 were installed in the precast girder section at the precast plant, while the remaining nine gauges were installed in the cast-in-place deck slab at the bridge site.

Locations of these gauges are de-

scribed in Fig. 14. In the figure, “Z” denotes the DT girder designation, and “N” denotes the north end quarter point. Each vibrating wire strain gauge included a thermistor for measuring the concrete temperature associated with each strain measurement.

Measurement of Early Age Girder Deflection

To obtain a complete history of early age deflection in the six instrumented girders, deflections were measured at selected intervals from the instant of tendon release to the completion of construction. A total of



Fig. 20. CFRP Leadline and CFCC reinforcements for sidewalks.

30 reference points were embedded in the concrete surface along the top flange of the girders at midspan, the two quarter points, and near the ends.

The deflection at a particular cross section was determined by measuring the elevation of the reference points using a precision level and surveyor’s rod with high-resolution scale. Prior to releasing the tendons, elevations for each point were measured and recorded and used as the zero reference elevation for all subsequent measurements.

Midspan deflections were determined by subtracting the measured elevation at midspan from the average elevation of the two end reference points. Similarly, quarter span deflections were determined by subtracting the measured elevation at the quarter-span from the average elevation of the two end reference points and the corresponding end elevation. After final construction of the bridge, the embedded reference points were transferred to the top deck. These stainless steel points were set just below the driving surface and covered with a protective cap for future deflection measurements.

Automated Measurement of Deflection

For automated deflection measurement of DT girders, automated deflection transducers were installed on the six instrumented girders after erection (see Fig. 15). A taut high-strength stainless steel wire was strung between the two fixed anchorage points established near the ends of the girder to serve as a reference. Displacement transducers were then installed along one girder stem at the north quarter point, midspan, and south quarter point.

Measurement of Post-tensioning Forces

For the six instrumented girders, all four CFCC strands were instrumented with load cells at one end. These were installed between the anchorage nut on the tendon sleeve and the bearing plate embedded in Diaphragm D6, near the north end of the girder (see Fig. 10).

As mentioned previously, post-tensioning was applied in two distinct stages: 60 percent of the total desired post-tensioning force applied at the precast plant just prior to shipping and the remaining 40 percent applied at the bridge site after placing the composite deck slab. The measured average post-tensioning force in CFCC strands after final post-tensioning was 454 kN (102 kips).

It should be noted that the initial and final post-tensioning were each applied in two operations. First, approximately 50 percent of the desired post-tensioning force was applied by pulling the strands from the south end (see Fig. 10). The setup then moved to the north end for the remaining 50 percent.

Monitoring the Status and Integrity of External CFCC Strands

To monitor the structural behavior and long-term status/integrity of external CFCC post-tensioning strands of the six non-instrumented girders, strain gauges and displacement transducers were installed on the four longitudinal tendons of each girder. The purpose of these sensors is to determine potential failure or loss of anchorage in the external CFCC strands. Fig. 16 shows a typical transducer on one of the four CFCC strands. A total of 21 strain gauges and three hybrid displacement sensors (SMS™)* were installed on the 24 strands.

Placing the Bridge Deck Slab

After the 12 girders were erected and grouted along the longitudinal joints, the initial transverse post-tensioning was applied, and construction of the deck slab began. For the deck, CFRP NEFMAC reinforcement was installed over the top flange of the girders (see Fig. 17). The nine remaining embedded concrete strain gauges required at each instrumented girder, for the deck slab concrete strain measurements at midspan and the two quarter point locations were installed. All the instrumentation cables were routed through the sidewalk.

* SMS is the trademark name of SMS034 hybrid displacement sensor.

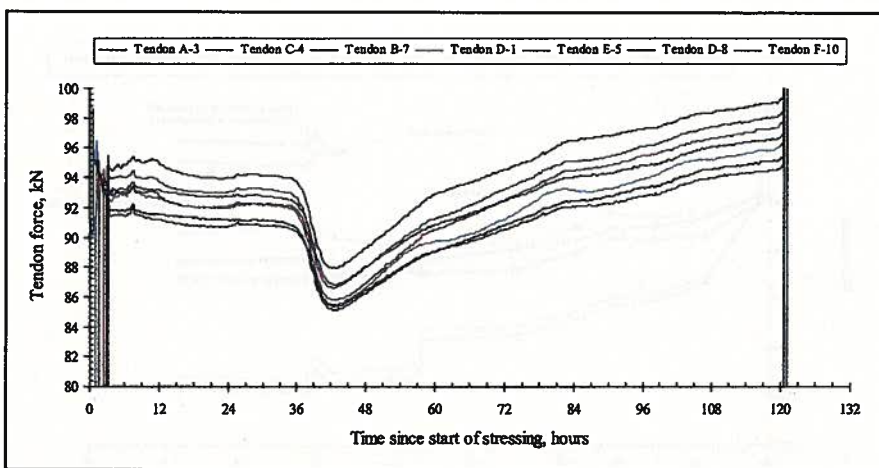


Fig. 21. Variation of pretensioning force in CFRP Leadline tendons of Girder L.

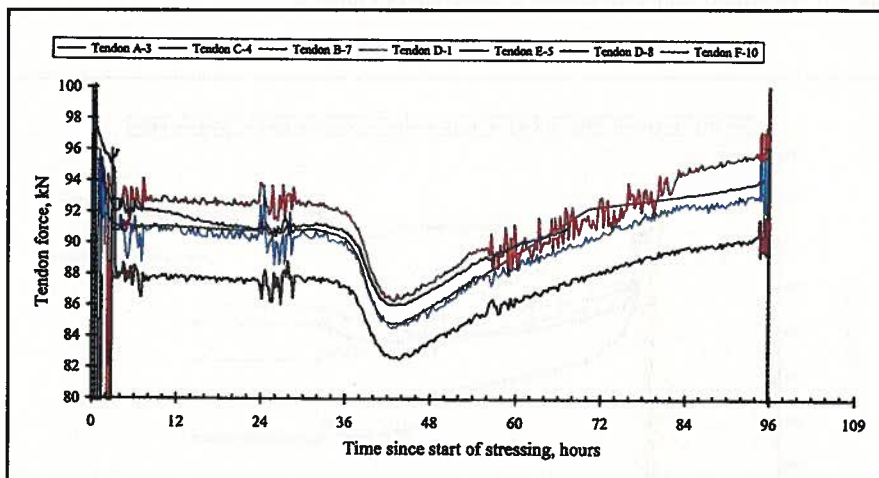


Fig. 22. Variation of pretensioning force in CFRP Leadline tendons of Girder G.

The NEFMAC reinforcement was supplied in the form of grid sheets, precut to required shapes by the manufacturer. Each grid sheet incorporated longitudinal reinforcement elements spaced at 300 mm (11.8 in.) on center and transverse elements spaced at 100 mm (4 in.).

The sheets were supported on 25 mm (1.0 in.) tall plastic slab bolsters, which were tied to both the slab bolster and girder using plastic ties, to prevent the sheets from floating during concrete placement. The deck slab was cast using a minimum 75 mm (3 in.) thick concrete topping with a minimum 28-day compressive strength of 38 MPa (5500 psi).

After the bridge deck slab, other essential components of the bridge – sidewalk, barrier railing, latex topping, and open parapet railing – were constructed. The latex-modified concrete

topping wearing surface was of 38 mm (1.5 in.) nominal thickness having 28 day compressive strength of 52.2 MPa (7560 psi).

Fig. 18 shows the CFCC reinforcement cage used for the open parapet railing, and Fig. 19 shows the NEFMAC reinforcement used for the concrete barrier railing, constructed on the opposite side of the deck from the sidewalk. The sidewalk reinforcement consists of a combination of straight CFCC and CFRP Leadline rods and bent CFCC strands (see Fig. 20). Concrete compressive strengths for these elements generally exceeded 51.9 MPa (7520 psi).

MEASURED RESULTS DURING CONSTRUCTION

In this section, measured tendon forces, concrete strains, deflections,

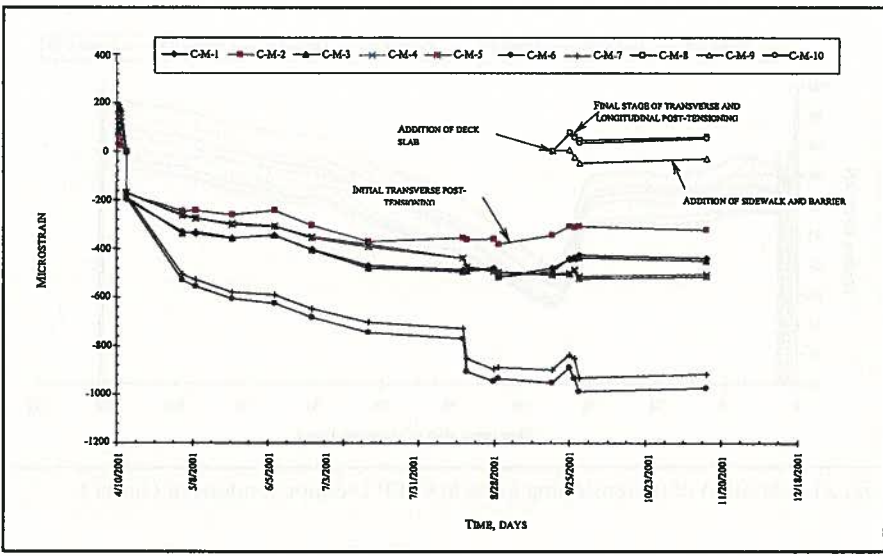


Fig. 23. Measured concrete strains at midspan of Girder C.

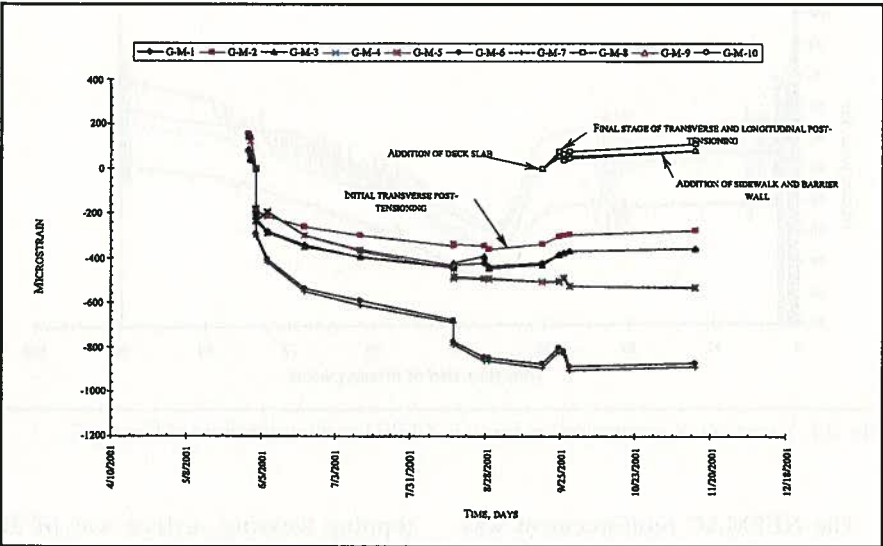


Fig. 24. Measured concrete strains at midspan for Girder G.

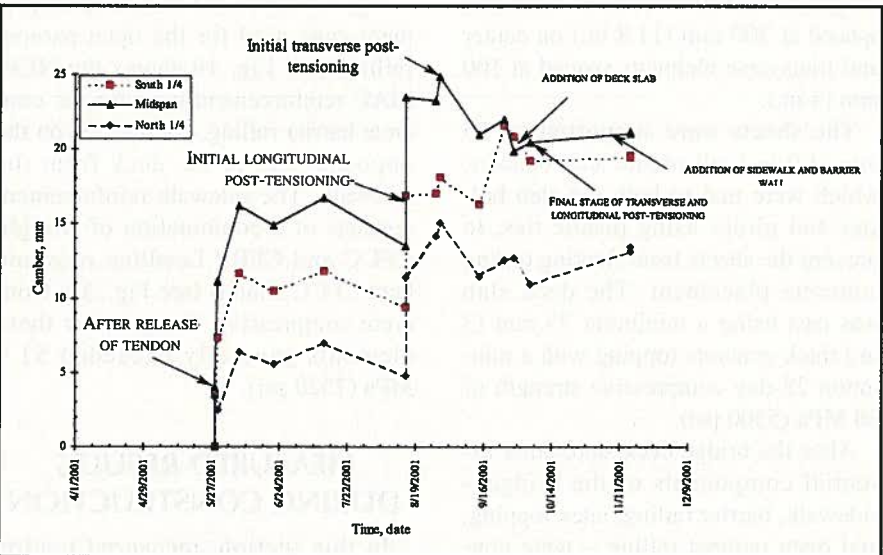


Fig. 25. Measured deflections for Girder K.

and post-tensioning forces during the construction of the bridge are presented.

Pretensioning Tendon Forces

Figs. 21 and 22 show the measured forces in selected Leadline tendons of Girders L and G, respectively, from the start of prestressing to the instant of release of prestressing forces. Girder L is located in the north span of the bridge, while Girder G is located in the middle span. Note that the loss of prestressing forces is not significant immediately after the prestressing; however, significant loss in the forces occurs after about 40 hours of prestressing. This phenomenon could be attributed to introduction of heat resulting from hydration of the concrete.

As shown in the figures, the loss in prestressing forces is restored as the curing proceeds and prior to the release of the prestressing forces. It is further observed that the measured prestressing forces, as expected, drop to zero upon release of the prestressing forces at the load cell location (see Fig. 9). The prestressed force release occurred after 120 and 96 hours from start of prestressing in Girders L and G, respectively.

Concrete Strains

The measured concrete strains at midspan of Girders C and G are presented in Figs. 23 and 24, respectively. In these figures, letters “C” and “G” denote Girders C and G, respectively, while letter “M” refers to the midspan section of the girders. The strain values are shown from the instant of release of prestressing forces, through initial post-tensioning (August 15 to 17, 2001), shipping, erection (August 21 and 22), final post-tensioning of CFCC longitudinal strands (September 28), and addition of sidewalk and concrete barrier railing. Strain values were taken as zero prior to release of pretensioning forces.

As expected, compressive strains were developed in the DT girders, while low tensile strains were developed in the deck slab of the girders after final post-tensioning. These strains are desirable as the prestress-induced compressive strains in the ten-

sion zone and tensile strains in the compression zone will counteract the strains due to service loads.

Deflections

Fig. 25 shows the early-age deflections at midspan and quarter points of Girder K. Early-age deflection refers to camber developed in the girder from the instant of release of pretensioning forces to the completion of construction of the bridge. From the figure, note that the release of pretensioning forces and application of post-tensioning forces result in camber of about 20 mm (0.79 in.). The maximum camber occurred after the initial longitudinal post-tensioning, followed by a slight camber decrease after placing the deck slab.

Post-tensioning Forces

Fig. 26 shows the forces in the four external CFCC post-tensioning strands in Girder J (one of the four north-end span girders of the bridge). The initial and final post-tensioning cause the increase in the forces in the external post-tensioning strands in a stair-step manner. It is worth noting that the post-tensioning forces remained almost unaffected during construction stages. A similar variation of the forces in transverse post-tensioning strands at Diaphragms D2, D3, D4, D5, and D6 after initial and final transverse post-tensioning is illustrated in Fig. 27.

CONCLUDING REMARKS

The Bridge Street Bridge Deployment Project has served as an extraordinarily successful example of technology transfer from research and development to serviceable structure. The bridge exhibits innovation not only in the material itself, but also in the variety of prestressing methods implemented – pretensioning and post-tensioning, internal and external.

Extensive instrumentation and continuous remote monitoring have provided valuable technical information during various construction stages and will continue to do so over the five-year monitoring period. The findings of the full-scale girder test at CTL and the

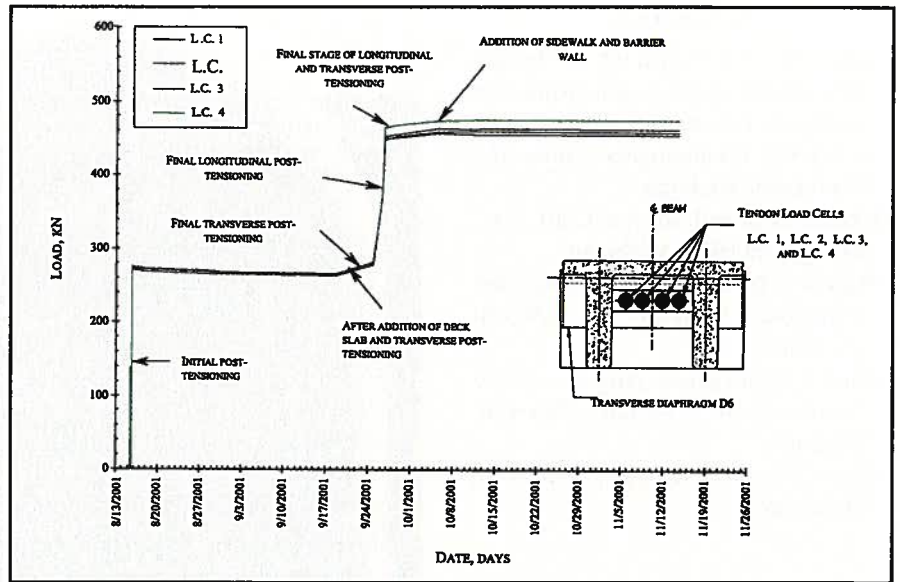


Fig. 26. Typical measured forces in longitudinal CFCC strands (Girder J).

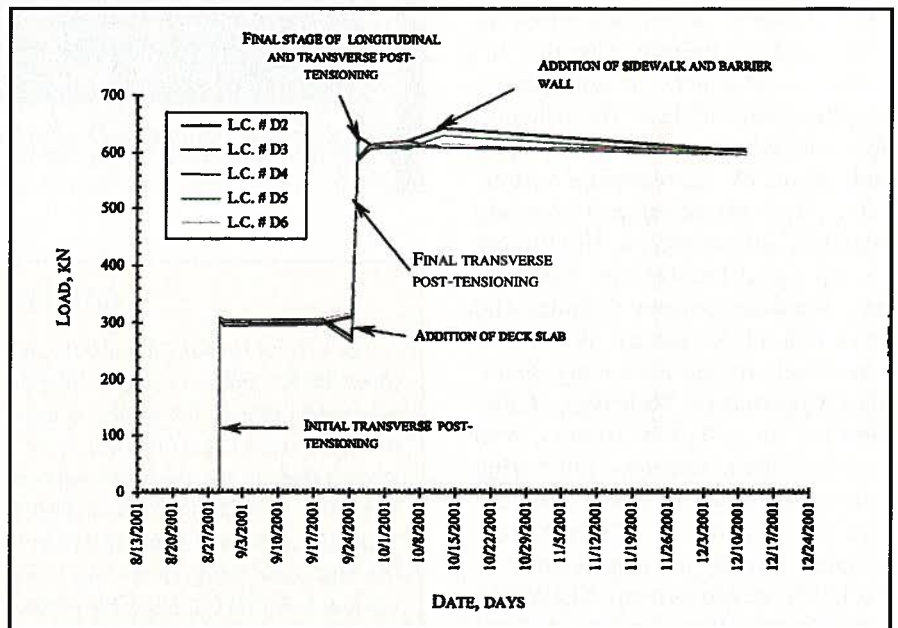


Fig. 27. Typical measured forces in transverse CFCC strands (north span).

continuous monitoring of this prototype bridge have confirmed the design assumptions made during the development of the construction documents.

This project won the Harry H. Edwards Industry Advancement Award in the recent PCI Design Awards Program. The jury citation was as follows:

“The use of CFRP tendons in precast concrete bridges opens new potential for bridge designers to solve design problems more effectively and with faster construction. The careful and detailed work undertaken by this

team of researchers, designers, and contractors holds great promise for future construction using CFRP. This project takes existing components and materials and expands on their abilities in new ways that will benefit the industry overall. These attributes define a Harry H. Edwards award winner.”

A large monument and plaque adorns the bridge on the west side of the north end approach (see Fig. 28). Fig. 29 is a group picture showing some of the officials and participants at the dedication ceremony.

CREDITS

Research, Development & Design Consultant: Civil Engineering Department, College of Engineering, Lawrence Technological University, Southfield, Michigan

Designer: Hubbell, Roth & Clark, Inc., Bloomfield Hills, Michigan

Precaster: Hollowcore Incorporated/Prestressed Systems, Inc., Detroit, Michigan

General Contractor: Angelo Iafrate Construction Company, Warren, Michigan

Owner: City of Southfield, Southfield, Michigan

ACKNOWLEDGMENTS

The success of this project is due to the energy and talent of many people, each of whom played a significant role. These include various researchers, designers, manufacturers, suppliers, and builders. The innumerable contributions made by individuals and the people representing various companies are recognized here and gratefully acknowledged: Hollowcore Incorporated/Prestressed Systems, Inc., Windsor, Ontario, Canada, who fabricated all the precast girders and performed all post-tensioning operations; Construction Technology Laboratories, Inc., Skokie, Illinois, who provided instrumentation and testing of the full-scale test model and both bridges; Autocon Composites, Toronto, Canada, the manufacturers of the CFRP reinforcement NEFMAC; ABM Corporation, New York City, and Sumitomo of America at their San Francisco office, who served as the trading company; Mitsubishi Chemical Functional Products, Inc., and Vantec Co., Ltd. (Komatsu Industries), the supplier and manufacturer, respectively, of the Leadline tendons; Mitsui & Co. (USA), Inc., Cleveland, Ohio, who served as the trading company, and Tokyo Rope Mfg. Co., Ltd., the supplier and manufacturer of the special cable reinforcement CFCC; Dr. George Abdel-Sayed, Professor Emeritus from the University of Windsor, who served as a special consultant to Hubbell, Roth & Clark; the research team (undergraduate, gradu-



Fig. 28. Monument with cast bronze plaque unveiled at the dedication.

OWNER EPILOGUE

The City of Southfield's new Bridge Street Bridge replaces a structurally obsolete steel bridge. The project consists of two parallel reinforced concrete bridges that are located at the only access point to the Bridge Street Industrial Park, an important commerce center in the city. One bridge was built using standard AASHTO Type III precast concrete I-girders with conventional steel reinforcement and a new substructure. The second was constructed using the existing piers and abutments and 12 precast concrete double tee (DT) girders reinforced with carbon fiber reinforced polymer (CFRP) material. This structure is the first known multi-span concrete highway bridge in the world to incorporate this material as its principal structural reinforcement.

This pioneering achievement was made possible only through the combined talents of many, including those from the Japanese composite material manufacturing industry and domestic academic research. The design and con-

struction was brought to fruition without the benefit of any established national or international standards for a structure of this type, which required an extraordinary effort and innovation by the City and project partners. The completed work represents the culmination of more than ten years of research and development, during which time many potential solutions to this nation's serious transportation problem were considered and discarded before this CFRP bridge solution became a reality.

The City of Southfield is proud of the Bridge Street Bridge project, both for its creative and innovative use of state-of-the-art CFRP construction technology and for the prestige that the project has brought to the city. It is also proud of the dedication, courage, and hard work demonstrated by the project team in developing and building this major municipal infrastructure project.

— Wayne Bonus
City of Southfield

ate, and post-doctorate students) of the civil engineering department at Lawrence Technological University in Southfield, Michigan; Hubbell, Roth & Clark, who performed the design and construction engineering, inspection and materials testing, and support groups; and the National Science Foundation, who funded the original research upon which the project design concept was based.

Site construction was funded in the 1998 fiscal year through FHWA as one of the TEA-21 High Priority Projects. Instrumentation and monitoring funding was provided under the Innovative Bridge Research and Construction Program of TEA-21. The Bridge Street Subdivision property owners granted approval of a Special Assessment District. The Tax Increment Finance District was created by the City.

The Michigan Economic Development Corporation awarded a grant. Congressional support came from Michigan Representatives Joseph Knollenberg and Sander Levin. Fi-



Fig. 29. Nabil F. Grace (fourth from left) with Southfield Mayor Brenda Lawrence (third from right), Shigeto Tanaka (second from right), President of Tokyo Rope Mfg. Co., Ltd., and representatives of Mitsui & Co. at the Bridge Street Bridge Dedication.

nally, the Mayor of the City of Southfield, the City Council, and the administration are commended for their vi-

sion of the future and their courage to venture into this unconventional construction arena.

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