PERFORMANCE OF A HISTORIC PRESTRESSED CONCRETE BLOCK BRIDGE

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ABSTRACT

A historic prestressed concrete block bridge in Wilson County, N. C. was recently replaced due to deterioration of its timber pile foundation. One of a few constructed by the North Carolina Department of Transportation in the early 1950s, the bridge used post-tensioned concrete block beams with cast-in-place topping to form the bridge deck. It had provided excellent service for 50 years, and at the time of replacement, the bridge was believed to be the last one of its type still in service in N. C. During replacement, the bridge deck was preserved for historical documentation. Three prestressed concrete block beam sections were tested to failure to assess their performance. This paper describes the construction details of the bridge deck and presents the results of the performance tests including the beam material properties, the effective prestress, the load-deflection characteristics, and the ultimate strength.

Keywords: Prestressed, Post-tensioned, Concrete, Masonry, Block, Bridge, Historic

INTRODUCTION – BRIDGE HISTORY

This bridge was one of only a few bridges known to have been constructed using post-tensioned concrete masonry blocks and was the last known bridge in North Carolina (and potentially the southeast) to be constructed by this method. Bridge construction of this form was used in the early 1950s, but abandoned by the late 1950s in favor of precast concrete construction that is common today. The description of a similar prestressed concrete block beam system using what was known as "Michigan block" can be found in Nasser[1].

The bridge, that was the subject of this study, was constructed in 1954 in Wilson County, NC and was known to the North Carolina Department of Transportation (NCDOT) as Bridge No. 97029. It spanned Black Creek on route SR1653, located 0.2 miles south of the junction with SR1659. It consisted of three continuous concrete spans supported by a timber substructure. The bridge had an inventory rating of HS-14, an operating rating HS-32, and was posted for a service vehicle of 36 tons[2].

A routine bridge inspection in 2000 identified the need to replace the bridge due to a deteriorating substructure. The substructure could not be repaired without removing the superstructure, which could have resulted in damage to the masonry girders. It was decided to replace the entire bridge and salvage a number of prestressed masonry block beams for further investigation, testing and historic documentation at North Carolina State University. Unfortunately the archival construction documents of the bridge are no longer available, therefore it is not possible to identify the designer, contractor or producer of the prestressed block beams. Likewise, no information on the original design criteria of the bridge can be found.

DESCRIPTION OF STRUCTURE

The three-span, two-lane concrete block bridge is shown in service in Figs. 1 and 2. Each span had a length of 5283 mm (17 ft 4 in.) and an overall deck width of 7747 mm (25 ft 5 in.) supported by 19 prestressed block beams placed side-by-side as shown in Fig. 3. Each prestressed block beam was constructed by joining 26 concrete masonry units together, each with a height of 197 mm (7 ¾ in.), using mortar with a joint thickness of 6 mm (¼ in.). The cells of the blocks used on each end were filled with concrete to form a solid block to provide an increased bearing surface for the prestressing anchors and to carry the shear force into the substructure. Fig. 4 depicts the specially shaped concrete masonry block. It should be noted that dimensions are shown in units of millimeters with US units shown in parentheses.

Fig. 1 NCDOT Bridge # 97029

Fig. 2 The Substructure

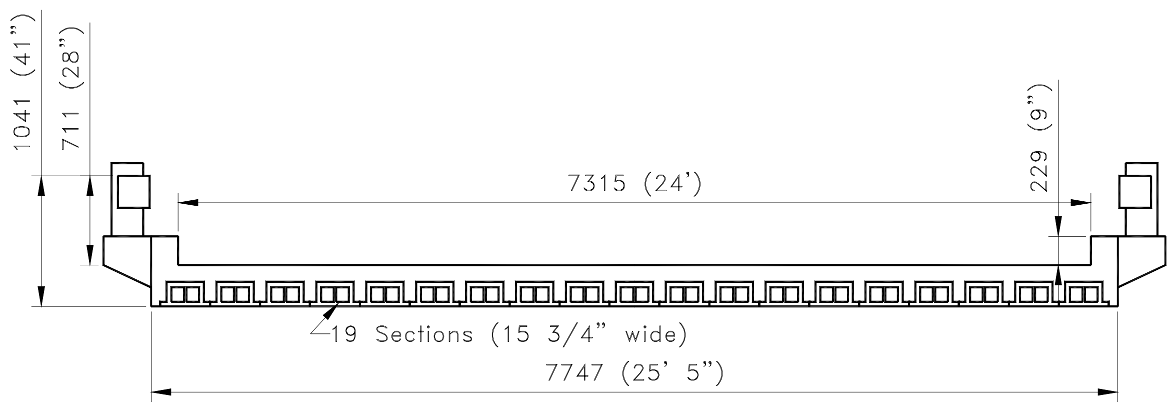


Fig. 3 Cross-section of Masonry Block Bridge

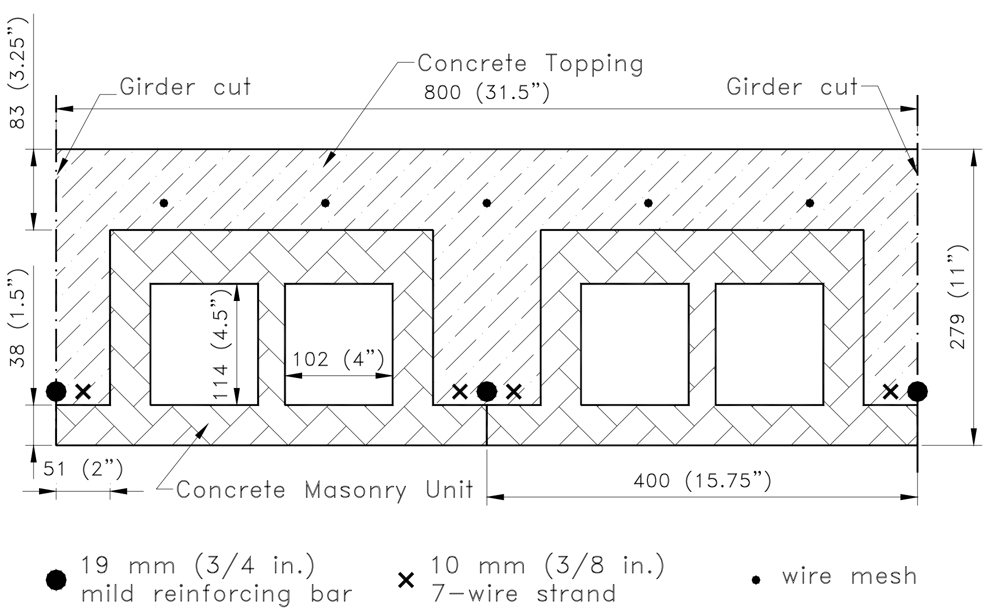


Fig. 4 Cross-section of Masonry Block Bridge Girder

Two rows of blocks were then butted together without using mortar, and a steel anchor plate 813 mm (32 in.) long placed across each end. This provided the anchorage for four 10 mm (3/8 in.) 7-wire prestressing strands that were installed with the draped profile shown in Fig. 5. The draped tendons were held in place by drilling holes in the masonry blocks and inserting smooth round steel rods as tie-downs. The strands were then stressed, resulting in a prestressed bridge girder 5283 mm (17 ft 4 in.) long by 813 mm (32 in.) wide, which could be trucked to site and installed on the substructure. Additional reinforcement, in the form of 19 mm (¾ in.) mild steel bars between the blocks, and wire mesh in the topping concrete, was installed as shown in Fig. 4. Topping concrete was then poured, which bonded the mild reinforcing and prestressing steel, and provided an 83 mm (3 ¼ in.) deep concrete road surface. The two voids in each block remained ungrouted, resulting in a partially grouted prestressed concrete masonry bridge.

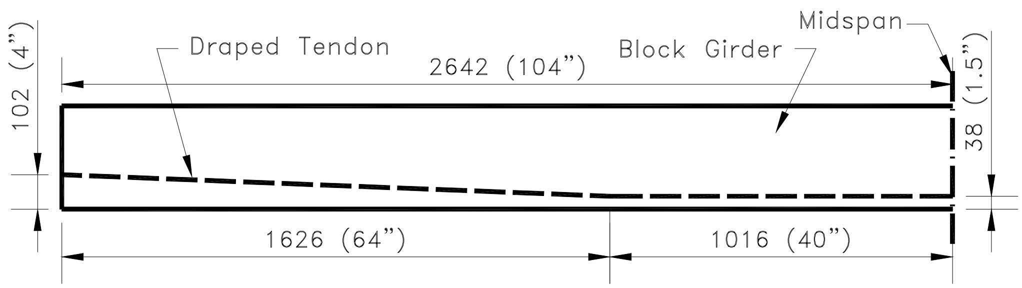


Fig. 5 Half Longitudinal Section of Girder showing Draped Tendon

MATERIAL PROPERTIES

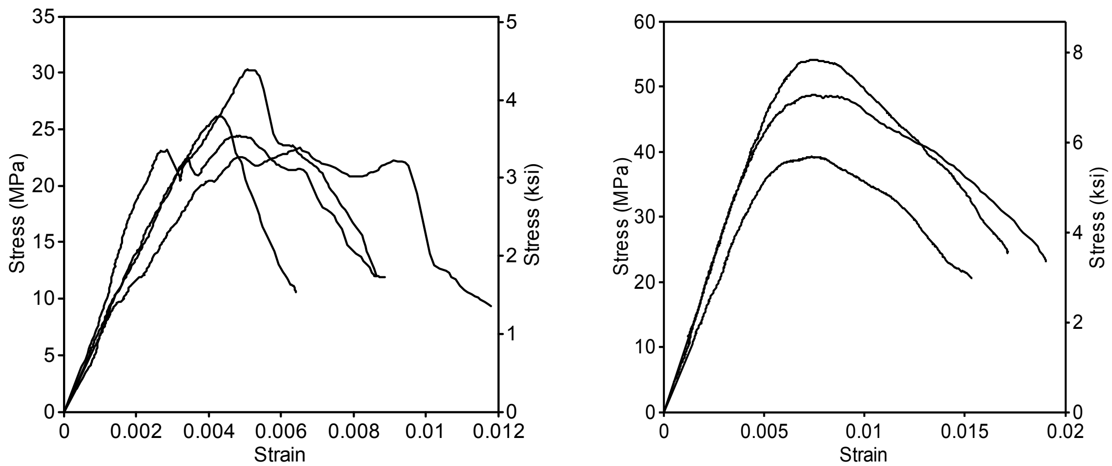
Material samples were taken from the bridge girders after testing, from locations that would ensure the samples had not been damaged during girder testing. The compressive strengths of the masonry units and concrete topping, and the tensile strengths of the mild reinforcing bars and prestressing strand were found.

CONCRETE MASONRY BLOCKS

As the masonry blocks were partially grouted in the girders, extracting full blocks for compression testing was impossible; therefore small cubes were cut from the flanges. The cubes had approximate dimensions of 32 mm (1.25 in.) square by 64 mm (2.5 in.) high, and an average density of 2125 kg/m^3 (133 lb/ft^3) was recorded. Four masonry cubes were tested in compression giving an average compressive strength of 26.0 MPa (3.8 ksi) with a standard deviation of 3.1 MPa (0.45 ksi). Fig. 6(a) shows the four stress-strain curves for the concrete masonry cubes.

TOPPING CONCRETE

Three core samples of concrete topping were taken from adjacent girder sections that were not used for structural testing. The core samples contained topping concrete, the top flange of the masonry block and varying amounts of wire mesh. The masonry was cut off and the remaining concrete sample ground to ensure a flat smooth bearing surface on the top and bottom. The three samples had an approximate diameter of 95 mm (3.7 in.) and approximate height of 86 mm (3.4 in.). Fig. 6(b) shows the three stress-strain curves obtained, indicating concrete compressive strengths of 39.2 MPa (5.7 ksi), 48.7 MPa (7.1 ksi) and 54.1 MPa (7.8 ksi). The large spread in results was attributed to varying confinement provided by differing amounts and orientation of reinforcing mesh that was contained within the samples.



(a) Concrete Masonry Block (b) Concrete Topping

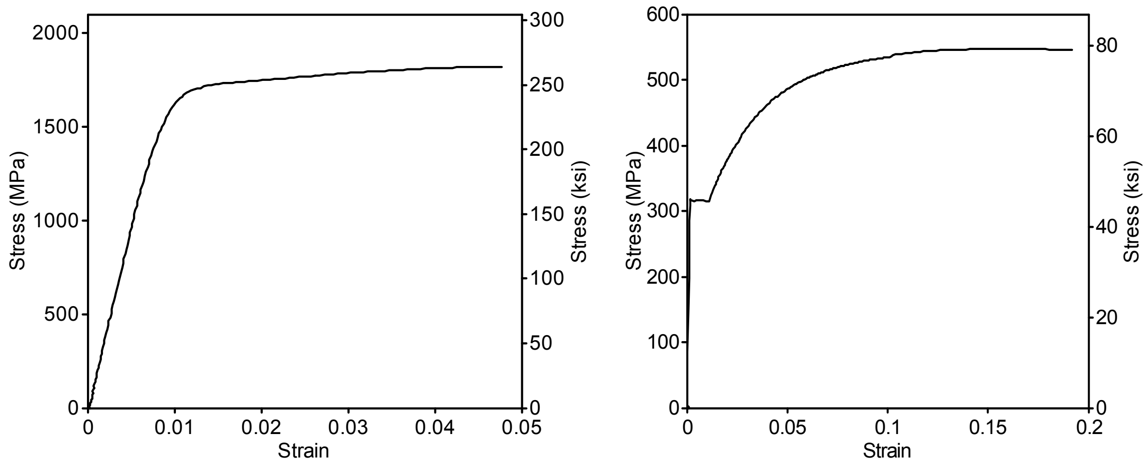
Fig. 6 Concrete Stress-Strain Curves

POST-TENSIONING STRAND

The 9.5 mm (3/8 in.) seven wire prestressing stand sample was removed from girder 2 after flexural testing. The strand sample had been grouted, showing no signs of corrosion, and was located towards the anchorage, therefore would not have yielded during testing. 3/8 in. strand wedge anchors were installed on each end of the sample and subsequently gripped in a tensile testing machine. The strand was tested as per ASTM 370 (Test methods and definitions for mechanical testing of steel products) and Fig. 7(a) shows the resulting stress-strain curve, indicating a 0.2% offset yield stress of 1650 MPa (239 ksi) and an ultimate stress of 1821 MPa (264 ksi).

MILD REINFORCING STEEL

A sample of 19 mm (¾ in.) mild reinforcing steel was extracted from a similar location as the prestressing strand in the same girder. The steel was in excellent condition, showing minimal signs of corrosion. The bar was tested as per ASTM 370 (Test methods and definitions for mechanical testing of steel products), resulting in the stress-strain curve depicted in Fig. 7(b), having a yield and ultimate stress of 315 MPa (46 ksi) and 547 MPa (79 ksi) respectively.



(a) Prestressing Strand (b) Mild Reinforcing Bar

Fig. 7 Steel Stress-Strain Curves

TEST SETUP

The bridge girders were cut into sections roughly 813 mm (32 in.) (2 masonry blocks) wide, as shown in Fig. 4, and transported to the Constructed Facilities Laboratory at NC State University for testing. A typical test setup is shown in Fig. 8, consisting of two steel supports secured to the strong floor to resist sliding and overturning. Neoprene pads were provided between the girder and supports to allow for rotation, and hydrostone was used between the strong floor and supports and between the supports and girder to provide proper seating. The first four tests involved the use of a 1270 mm (50 in.) spreader beam, connected to the 220 kip actuator, subjecting the girders to two-point bending at 1118 mm (44 in.) apart. The spreader beam was used to ensure a region of constant moment for the girders tested in flexure. The spreader beam was removed for the final shear test, providing a single load point at mid-span. Neoprene was installed between the spreader beam and girder surface to allow for girder rotation.

Instrumentation was installed to measure displacements along the length of the girders and at the supports. The load was taken directly from the actuator load cell. Data was recorded on a data acquisition system using a scan rate of 2 scans per second.

The first three girders tested were initially loaded to first cracking, which was defined as the point at which the first visual observation of concrete grout cracking was noted. The load was then released at the same rate until the girder was unloaded. Loading then began again until failure occurred. In the case of the two shear tests, the girders were tested monotonically until failure. In all cases an initial loading rate of 0.05 mm/sec (0.002 in./sec) was used, which was doubled for the first three tests once the girders had reached yield.

Fig. 8 Typical Test Setup (Girder 3 shown)

GIRDER TESTING AND RESULTS

GIRDER TEST 1 - FLEXURE

Girder 1 was tested with a span of 4880 mm (16 ft.) and failed in flexure, with the force-displacement history shown in Fig. 9. The numbers on the graph refer to the following events that occurred during testing.

1) First cracking of the mortar joints at mid-span was recorded at a load of 37.8 kN (8.5 kips).
2) At a load of 111 kN (25.0 kips) the first cracking of face shells was recorded.
3) At a mid-span displacement of 92 mm (3.62 in.) with a corresponding load of 134 kN (30.1 kips), one wire in a 7-wire prestressing strand ruptured resulting in a small drop in load.
4) A further 2 wires ruptured at a displacement of 118 mm (4.65 in.).
5) An additional wire ruptured at a displacement of 135 mm (5.31 in.) resulting in some loss of face-shell. Crushing of compression concrete was first recorded.

6, 7, 8) Further wire rupturing and loss of masonry face shells was observed at displacements of 164 mm (6.46 in.), 167 mm (6.57 in.), 178 mm (7.01 in.) and 185 mm (7.28 in.).

Testing ceased because of the significant drop in girder strength due to tendon rupturing. Fig. 10(a, b) shows a section of the girder at mid-span, where wires ruptured and masonry face shells were lost.

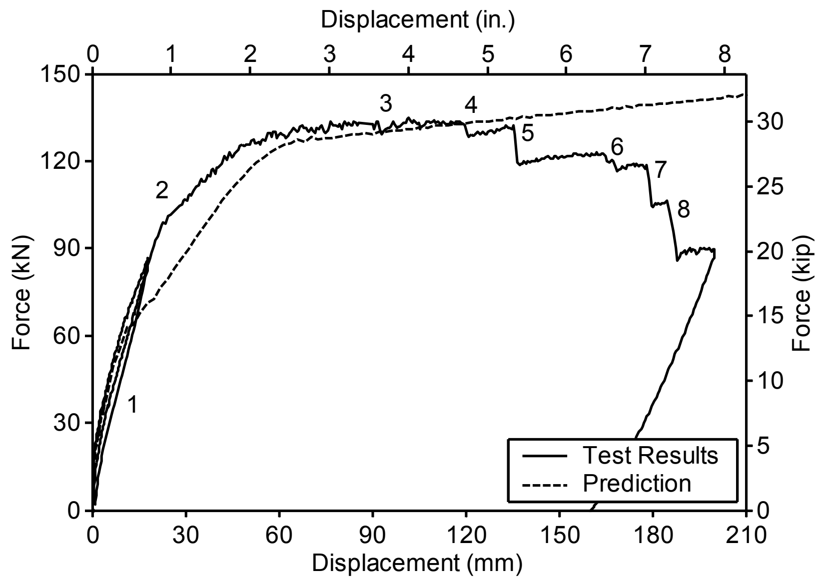


Fig. 9 Force-Displacement History Girder 1 – Flexure

(a) Damage at mid-span (b) Ruptured tendon and face shell lost

Fig. 10 End of Testing Girder 1 in Flexure

GIRDER TEST 2 - FLEXURE

The second girder was tested in flexure using the same method as for the previous girder. Fig. 11 shows that the girder was initially taken to the previously discussed level of first cracking and then unloaded. At a load level of 125 kN (28.1 kips) and a displacement of 56 mm (2.20 in.), a prestress anchor failed resulting in a significant drop in strength. When the girder had been cut on site and separated from the remaining bridge, a section of grout had been lost from around the tendon. This resulted in an unbonded tendon over a 2845 mm (112 in.) length and an increased force on the anchorage when the girder was loaded. Testing continued until shear cracking occurred when the girder reached a displacement level of 74 mm (2.91 in.) with a corresponding load of 110 kN (24.7 kips). The girder was deemed to have failed and testing was ended. Fig. 12(a) shows the anchorage after it had failed and Fig. 12(b) shows the cracked girder at the end of testing.

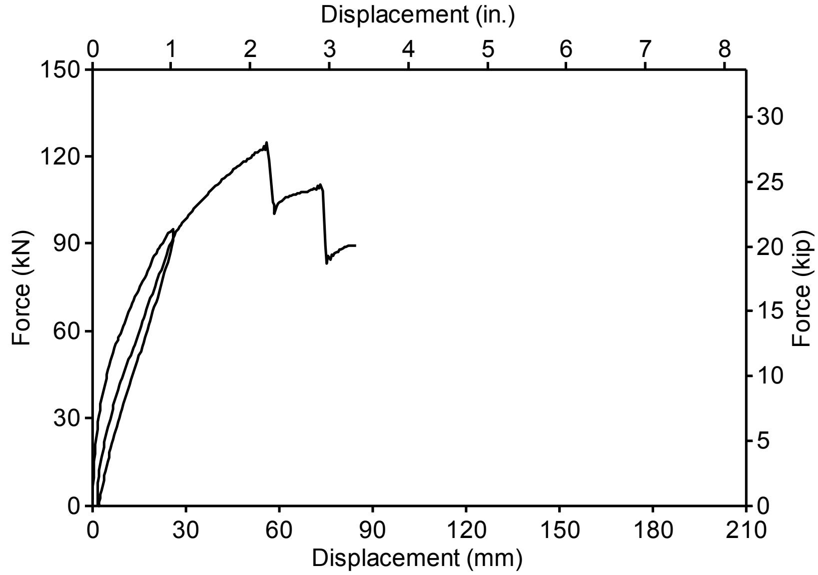


Fig. 11 Force-Displacement History Girder 2 – Flexure-shear

(a) Failed anchorage (b) Shear cracked girder

Fig. 12 End of Testing Girder 2 in Flexure

GIRDER TEST 3 - FLEXURE

The third girder was setup and tested consistent with the previous two tests, and a flexure-shear failure was observed. The first sign of cracking was recorded at a load of 40 kN (9.0 kips) and significant shear cracking observed at a maximum load of 127 kN (28.6 kips) and displacement of 73 mm (2.87 in.). Loading continued with a further drop in load due to shear cracking at a displacement of 94 mm (3.70 in.) and partial anchor failure and compression concrete crushing at a displacement of 107 mm (4.21 in.). Testing ceased and the remaining load was removed as can be seen in Fig. 13.

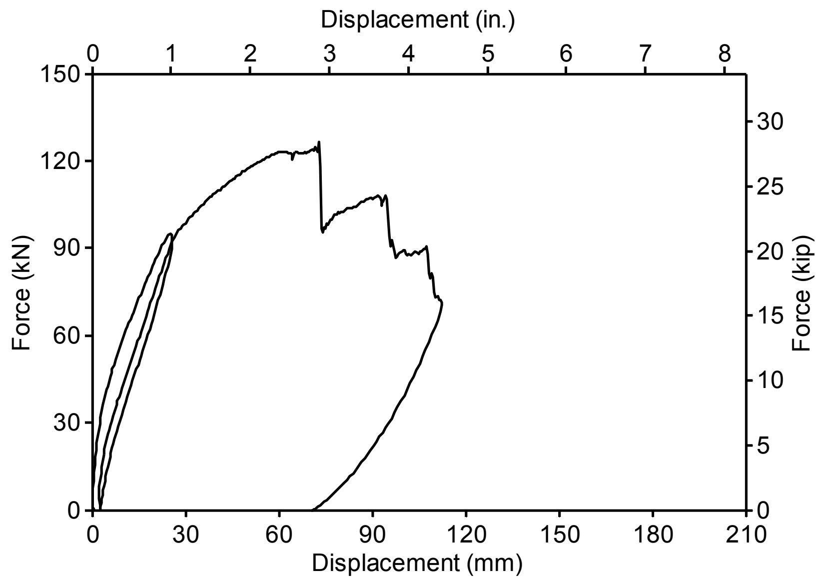


Fig. 13 Force-Displacement History Girder 3 – Flexure-shear

GIRDER TEST 4 - SHEAR

One end of girder 3 was retested with a shear span of 2930 mm (9 ft 7 in.) with the load points offset 200 mm (8 in.) from mid-span. The retesting of girder 3 was known as girder test 4 and failed in shear, with the shear crack initiating from a location of flexural cracking that had occurred during the previous test. As Fig. 14 shows, the load reached a maximum of 222 kN (49.9 kips) before the onset of shear cracking.

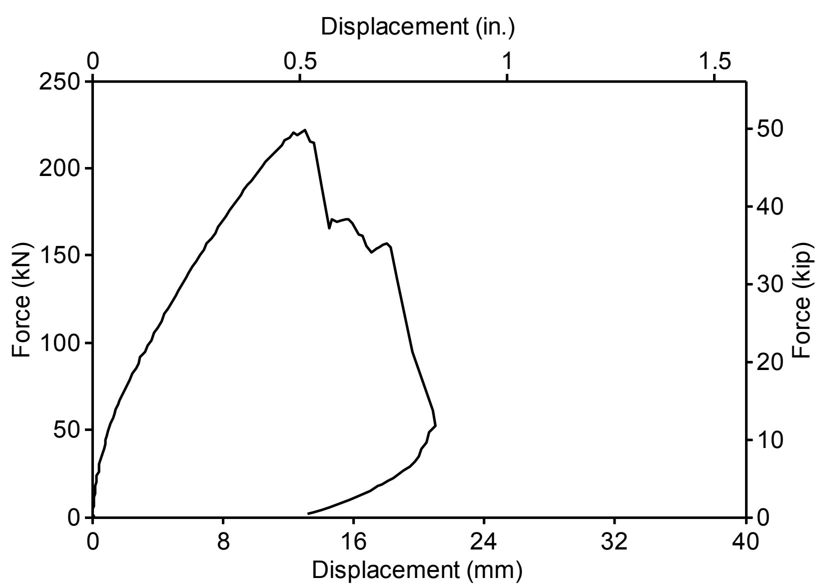


Fig. 14 Force-Displacement History Girder 3 – Shear

GIRDER TEST 5 - SHEAR

This test involved retesting one end of girder 1 with a span of 1020 mm (3 ft 4 in.) and without the spreader beam, as shown in Fig. 15, resulting in a shear span of 435 mm (17.1 in.). Fig. 16 shows the force-displacement history for the test, indicating that the girder resisted a maximum force of 130 kN (29.2 kips) at a displacement of 1.6 mm (0.06 in.) before the onset of shear failure.

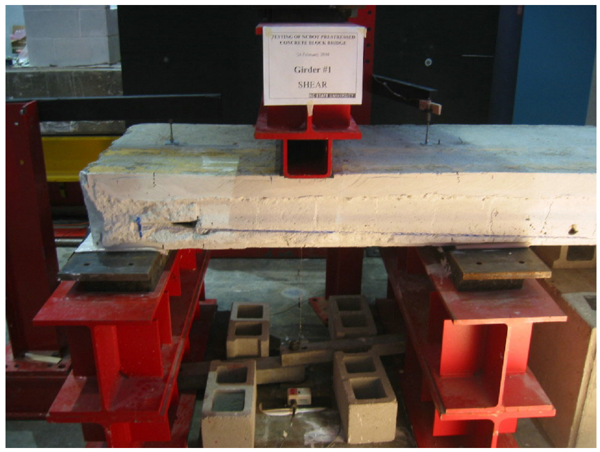


Fig. 15 Girder test 5 setup

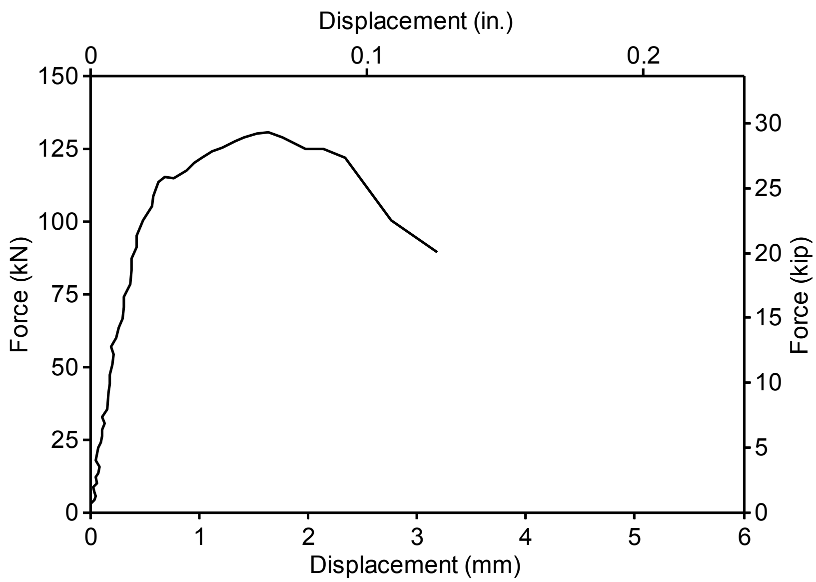


Fig. 16 Force-Displacement History Girder 1 – Shear

DISCUSSION OF RESULTS

MATERIAL PROPERTIES

The average compressive strength f_{cm}' of 26.0 MPa (3.8 ksi) from testing the four masonry cubes is what one would expect from the masonry unit manufactured in 1954. Based on this value, the modulus of elasticity of the masonry is calculated as $E_m = 900\sqrt{f_{cm}'} = 900\sqrt{26} = 4{,}590$ MPa (666 ksi).

The compressive strength of the topping concrete was determined by testing three cores, which gave an average compressive strength of 47 MPa (6.87 ksi). It should be noted, however, that due to practical limitations, the height of the core was 86 mm (3.4 in.) and its diameter was 95 mm (3.7 in.). To account for this size effect, a correction factor of 0.8 is applied to the average value. Therefore, the actual compressive strength f_c' of the cover concrete is taken as $0.8 \times 47 = 38$ MPa (5.5 ksi). Based on the 1954 industry standard, this concrete would be considered as high strength concrete. It is likely that when the bridge was constructed in 1954, the specified concrete strength could be somewhere between 28 MPa (4 ksi) and 35 MPa (5 ksi). Using the compressive strength of 38 MPa (5.5 ksi), the modulus of elasticity E_c of the concrete is computed as 29 GPa (4,230 ksi) according to Section 8.5.1 of the ACI Code[3]. The high values of compression strain recorded are most likely a result of the small specimen size relative to the test machine, or the confinement provided by the steel contained within the cores.

The stress-strain curve of the 3/8 in. prestressing strand shown in Fig. 7(a) confirms that the strand used in the bridge was of Grade 250. This was the industry standard in 1954.

The stress-strain curve of the mild reinforcing steel shown in Fig. 7(b) also confirms that the steel was of Grade 40, one that was commonly used in the 1950's.

LOSS OF PRESTRESS

As shown in Fig. 4, each block beam was post-tensioned by two 3/8" 7-wire strands of Grade 250. The initial prestress in the strand can be taken as 1379 MPa ($0.8 \times 250 = 200$ ksi) as is generally specified. Using the sectional properties of the masonry block, the initial compression in the bottom fiber of the block due to prestress can be computed as 7.1 MPa (1,030 psi).

During the first flexure test, it was observed that the initial cracking in the block joint at midspan occurred when the applied load on the composite beam was 37.8 kN (8.5 kips). Based on the transformed section of the composite beam, the flexural tension in the bottom fiber under the initial cracking load was found to be 1.73 MPa (251 psi), neglecting any tensile strength of the mortar in the block joint. Therefore, it can be said that the residual compression in the block beam was 1.73 MPa (251 psi), indicating that the loss of prestress was $7.1 - 1.73 = 5.37$ MPa (779 psi) or 76%. It is apparent that due to the effects of creep and shrinkage in the past 50 years, a very large portion of the original prestress was lost in the masonry block beam. Alternatively, the beam may have been prestressed initially to a lower level for construction purposes. In the flexure test, as soon as the applied load exceeded the initial cracking load, the beam behaved more like a normally reinforced concrete beam rather than a prestressed concrete member. It is worth noting, however, that the residual compression of 251 psi (1.73 MPa) in the block beam was sufficient to keep the composite beam crack free, thus providing excellent protection against corrosion for the prestressing strands and reinforcing bars in the beam.

BEAM TESTS

A total of five tests were conducted. The first three were designed as flexure tests, but only the first test developed the true flexural failure. As seen in Fig. 9, the load-displacement curve from the test compares very well with the theoretical prediction before extensive cracking developed near the ultimate load. At the stage of progressive cracking, the experimental curve lies slightly above the theoretical curve, indicating the stiffening effect of the concrete in tension. Using the appropriate material properties of concrete and steel determined in this investigation, and neglecting the effect of small residual prestress, the theoretical ultimate load capacity of the beam was found to be 131 kN (29.4 kips) as compared to 134 kN (30.1 kips) obtained from testing, indicating an excellent prediction.

The second test failed prematurely at 125 kN (28.1 kips) due to an anchorage failure, leading ultimately to a flexural shear failure at a reduced load level of 110 kN (24.7 kips). The third test also failed prematurely at a maximum load of 127 kN (28.6 kips) when major flexural shear cracking occurred, leading to bond failure along the reinforcement. It should be noted that the initial portion of the load-displacement curve for these two tests is very similar to that of the first test except for a very slight reduction in stiffness.

The fourth and fifth tests were designed as shear tests, using a remaining portion of girder 3 and girder 1 respectively. The shear span-depth ratio was 4.5 for the fourth test and 1.55 for the fifth test. As would be expected, the fourth test resulted in shear compression failure with the major shear crack developing from a pre-existing flexural crack, and the fifth test resulted in diagonal shear failure. The maximum load carried was 222 kN (49.9 kips) for the fourth test and 130 kN (29.2 kips) for the fifth test. These load capacities were considerably higher than the load capacity developed in the first flexure test. However, the failure mode of the fourth and fifth tests was more sudden and violent with much less displacement, representing typical brittle shear failure as evidenced by the load-displacement curves of these two tests.

The shear force, of 65 kN (14.6 kips), carried by the beam in the fifth test, equates to a shear stress of 0.59 MPa. This value was found using a shear area of 109,370 mm^2 (170 $in.^2$), accounting only for the area that was solid concrete or masonry the depth of the beam. Since the test beam did not contain any web reinforcement, and the effect of prestress was negligible, the shear capacity of the test beam can be predicted by using ACI Code Eq. (11-3) or Eq. (11-5). Using Eq. (11-5), the predicted shear capacity of the test beam is 49.5 kN (11.12 kips) as compared to the shear of 65 kN (14.6 kips) carried by the beam in the fifth test. The test result exceeded the prediction by 31 percent.

CONCLUSIONS

(1) The results of this investigation indicate that the performance of the prestressed concrete masonry block beams used in the bridge was excellent after 50 years of service. There was no evidence of deterioration of prestressing steel, reinforcing steel, or concrete before the bridge had to be replaced due to deterioration of its timber substructure.

(2) The 50-year old cast-in-place concrete used in the bridge deck developed a compressive strength of 38 MPa (5.5 ksi). Considering the concrete technology in 1954, the concrete used in the bridge would be regarded as high strength during that period.

(3) It was confirmed that the prestressing steel used in the block beam was of Grade 250, the reinforcing steel was of Grade 40, and the compressive strength of the masonry was 26 MPa (3.8 ksi).

(4) The loss of prestress in the post-tensioned block beams was found to be 76%. This large amount of prestress loss was due to the long-term effects of creep and shrinkage. Alternatively, the girder may have been prestressed to a lower level originally for construction purposes.

(5) The predictions of the behavior and load-carrying capacity of the composite prestressed block beam in flexure compared closely with the test results.

(6) The shear capacity of the composite prestressed block beam obtained from the test exceeded the prediction based on the ACI shear provision by 31%.

ACKNOWLEDGEMENTS

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