Laboratory Static Load Tests on Five Sunshine Skyway Bridge Girders

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The Sunshine Skyway that crosses the entrance to Tampa Bay south of St. Petersburg, Florida, on a 6.5-km (4-mile) bridge consists of two separate two-lane structures constructed primarily of prestressed concrete girders on reinforced concrete pile bents. The older of the two structures was completed in 1954 and was one of the earliest of its type in the U.S. The girders were post tensioned with steel bars grouted in steel conduits, but spalling and rusting were observed before the bridge was 20 years old. Florida Department of Transportation engineers investigated the structural integrity of the bridge and transported six girders to the University of Florida in Gainesville for laboratory tests. Five of these girders were tested under static loads to determine ultimate flexural capacities, and two were found to be extensively spalled with corroded prestressing bars. These two were the only ones to exhibit a premature fracture of the bars. The tests indicated that no serious loss of strength would be likely to occur before extensive spalling of the concrete revealed badly corroded or fractured bars. Girders can therefore be periodically inspected for removal and replacement before collapse becomes imminent.

The highway structure that crosses the entrance to Tampa Bay south of St. Petersburg, Florida, is known as the Sunshine Skyway. It consists of two separate two-lane bridges extending 6.5 km (4 miles) across the bay; about 5 km (3 miles) of it are of low-level prestressed concrete girders on reinforced concrete pile bents and about 1.5 km (1 mile) of high-level steel girders and trusses that carry traffic without interruption over the main ship channel to the Port of Tampa. The first, the northbound bridge, was completed in 1954, and the second, the southbound bridge, was completed in 1969.

Periodic maintenance inspections by the Florida Department of Transportation (DOT) have revealed extensive corrosion on prestressing and reinforcing steel in girders and pile bents of the older bridge. The problem has evidently been accelerating in recent years. A corrosion survey conducted in 1973 listed 81 girders in 54 of 349 spans in poor or critical condition from spalled concrete and exposed and severely corroded prestressing steel. Florida DOT engineers, concerned about the deterioration and probable reduction in strength of the bridge girders, initiated a program of in situ tests to evaluate the strength capacity of the bridge superstructure (1).

After consultations with engineering faculty at the University of Florida, a research program was contracted to load test bridge girders in the Department of Civil Engineering Laboratory. Six girders were removed from the older bridge and transported from Tampa Bay to Gainesville.

THE OLDER BRIDGE

Bridge Construction

One of the salient features of the older bridge is the post-tensioned girders with composite reinforced concrete deck slab, one of the earliest uses of this type of construction in the United States. The girders were cast and post tensioned in casting yards near the construction site. Each girder was post tensioned by three 25.4-mm (1-in) diameter, 1105-MPa (160 000-lbf/in^2) ultimate strength Macalloy steel bars. Two bars are essentially straight; the third is draped parabolically. The bars were post tensioned through 31.8-mm (1.25-in) conduits and grouted. The Macalloy steel was imported from England, but the concrete materials were local products.

The reinforced concrete piles and caps extend about 2.5 m (8 ft) above mean high water, and depths in the bay along most of the length of the bridge average about 5 m (16 ft). The pile bents are spaced at 14.6 m (48 ft), providing 14-m (46-ft) bridge spans.

Environmental Conditions

The bridge girders are usually 2.5 m (8 ft) to 3 m (10 ft) above the water surface when the bay is calm. The mean tidal range is 0.5 m (1.5 ft), but sustained winds may raise or lower the tidal range by as much as 1.25 m (4 ft). In rare hurricanes, the tidal stage may rise 2 m (6 ft) or more. The bay is relatively open to the Gulf of Mexico, with wide passes to the west and southwest on either side of Egmont Key. The generally shallow waters of the bay become severely choppy when wind velocities rise to 24 km/h (15 mph) and higher. The salinity of the bay waters around the bridge varies with rainfall and freshwater runoff, but the mean is 32 to 33 parts per thousand, compared with 37 parts per thousand in the open Gulf waters at this latitude.

Description of Corrosion

As a result of frequent exposure to chop and northeasterly winds, the eastern sides of the girders show appreciably more cracking, spalling, and corrosion than the western sides. Most of the deterioration occurs near the ends of the spans, where waves strike the pile bents and splash up against the lower girder flanges. The end blocks are reinforced with vertical stirrups and longitudinal tie-bars. The lower tie-bars extend into the tapered region where the girder section changes from rectangular to I-shape. Here the concrete cover is as little as 6 mm (0.25 in).

Saltwater penetrating this thin layer of concrete produces corrosion and expansion of the steel and spalling of the concrete and further exposure to corrosion. Eventually the prestressing conduits become exposed, and corrosion progresses down the length of the conduits and penetrates the prestressing bars. Concrete cover over the conduits varies from about 13 to 32 mm (0.5 to 1.5 in). In many girders several meters of conduit were disintegrated, and in a few girders the prestressing bars had reduced in section to the point where they fractured under the prestress force (these girders have been replaced). Figures 1 and 2 show typical spalling and corrosion of girders.

STRUCTURAL PROPERTIES OF BRIDGE GIRDERS

Geometric Properties

The bridge superstructure consists of six post-tensioned girders spaced on 1.83-m (6-ft) centers with a 178-mm (7-in) reinforced concrete deck 11.43 m (37.5 ft) wide. This provides for two lanes of traffic plus walkways on either side as shown in Figure 3. Composite behavior of the girders and the deck slab is provided by steel shear ties and concrete keys. The girders span between the pile bents as simple beams, and the concrete deck is provided with control joints at each bent so that flexural restraint at the supports is negligible. Precast transverse diaphragms are located at the one-third points of the span and are grouted in place and post tensioned with a single transverse 25.4-mm (1-in) Macalloy bar. The girder dimensions are shown in Figure 4.

Mechanical and Physical Properties

The properties of the girder material were determined by laboratory tests on samples from the girders that were tested in the laboratory. The compressive strengths of the deck and girder concrete were evaluated from core tests and were found to be 29.9 MPa (4336 lbf/in^2) and 43.72 MPa (6339 lbf/in^2) respectively.

The percentage composition of the Macalloy prestressing steel is approximately 97.4 iron, 0.6 carbon, 1.2 manganese, and 0.8 silicon. The Department of Materials Science tested corrosion and notch sensitivity of this steel (2).

The results of tension tests indicated that the ultimate tensile strength in air without notching was about 1070 MPa (155 150 lbf/in^2). When the specimens were notched, the strength fell 50 percent. An additional reduction in strength occurred when notched specimens were immersed in a synthetic seawater solution while tested.

In summary, the Macalloy steel used in the Sunshine Skyway Bridge is a medium-carbon, silicon, manganese steel exhibiting moderate tensile properties with low impact strength, high notch sensitivity, and an apparent adverse reaction to a corrosive environment. A stress-strain diagram for the bars is shown in Figure 5.

Figure 1. Typical concrete spalling and steel corrosion in bridge girders.

Figure 2. Exposed and corroded prestressing bar in girder lower flange; lower bar is tie bar from end block.

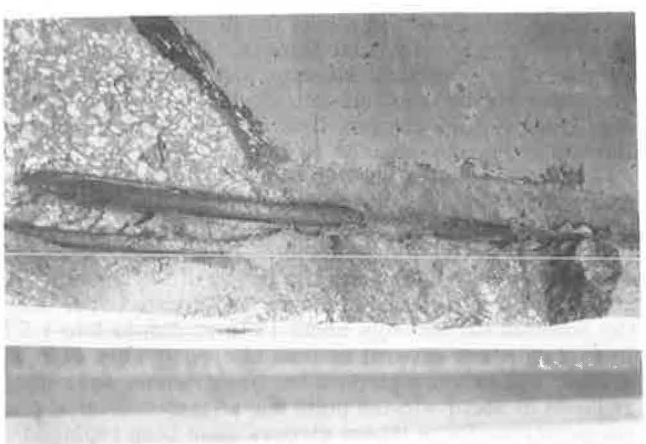
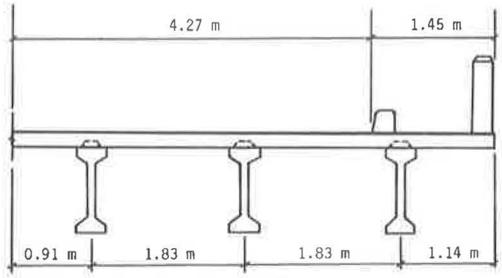


Figure 3. Bridge cross section.



Note: 1 m = 3.28 ft.

LABORATORY LOAD TESTS

We selected six composite bridge girders on the basis of their different stages of deterioration and removed them from the Sunshine Skyway for testing. In order not to disrupt traffic, we took the girders from one lane at two spans about 0.5 km (0.3 mile) apart. This provided two undamaged exterior girders, two undamaged interior girders and two severely corroded interior girders. The deck and diaphragms were sawed free, and the girders were removed from the bridge, loaded onto flatbed trucks, and transported to Gainesville.

Description of the Test Procedure

The test arrangement is shown in Figure 6. The girders were placed on support blocks with neoprene bearing pads on a span of 14 m (46 ft), measured between center of bearings. A loading frame constructed of rolled steel beams was placed over the girder and secured to existing floor anchors. The test load was applied by two hydraulic jacks placed between the load frame and girder. The load was metered by electric load cells placed between the jacks and steel plates bearing on the girder flange. Deflections were measured in relation to a taut wire stretched over supports located at each end of the girder.

A single concentrated load to the girders was applied and increased at a relatively constant rate; it was held static at intervals to inspect for cracks and to read deflections.

The original design load specified for the bridge could not be documented, but some evidence was found to indicate that it was designed for the standard AASHO H20-44. Because there is little difference between the distribution of bending moments and shears produced by the H truck loading and by a single concentrated load, the latter was used to facilitate testing.

The theoretical ultimate flexural and shear capacities of the girders were calculated in accordance with recommendations of the American Concrete Institute Standard 318-71. The resulting equations for flexural capacity (M_u) and shear capacity (V_u) are

$$M_u = A_{ps} f_{ps} d[1 - 0.59 (A_{ps} f_{ps}/bdf'_c)] \quad (1)$$

$$V_u = 0.6\sqrt{f'_c}\, b_w d + V_d + [(V_l M_{cr})/(M_{max})] \quad (2)$$

where

A_{ps} = area of prestressed reinforcement,

Figure 4. Typical girder dimensions.

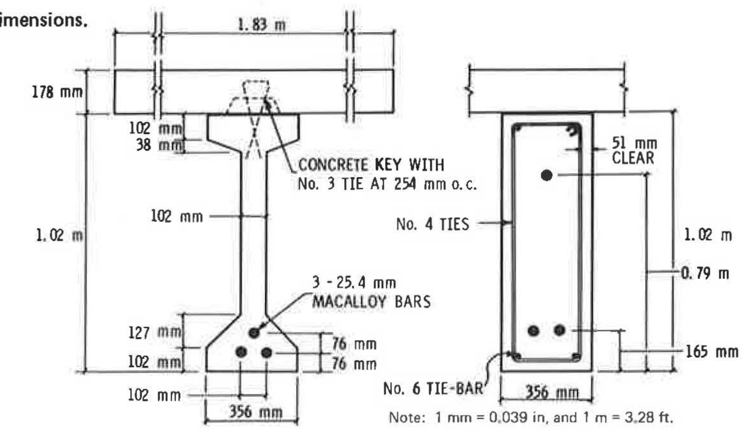


Figure 5. Tensile stress-strain test for Macalloy steel bar.

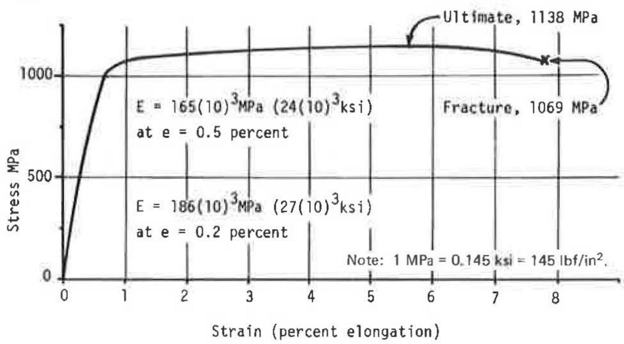


Figure 6. Alternate test load positions.

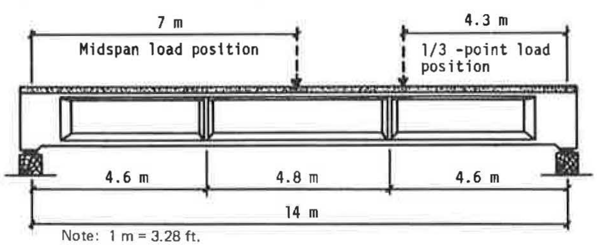


Figure 7. Theoretical ultimate flexural capacity of girders in terms of a single load P_u.

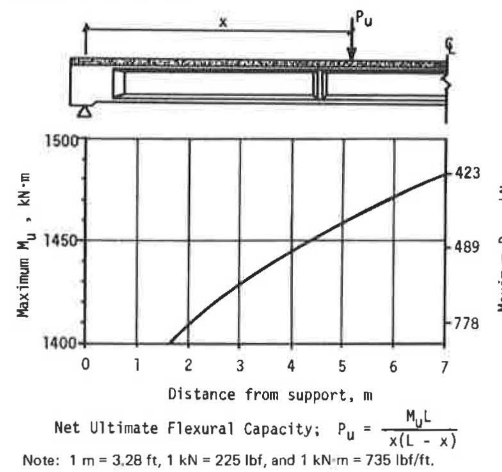
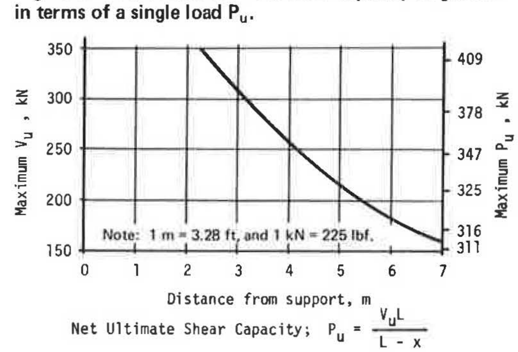


Figure 8. Theoretical ultimate shear capacity of girders in terms of a single load P_u.



b = effective flange width,
b_w = web width,
d = effective depth,
f'_c = compressive strength,
f_{ps} = stress in prestressed reinforcement,
M_{cr} = cracking moment,
M_{max} = maximum applied load moment,
V_d = dead load shear force, and
V_l = applied load shear force.

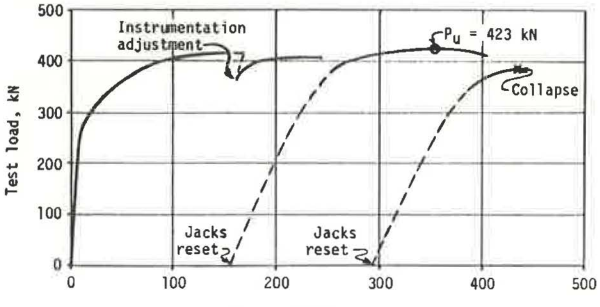
Net ultimate capacities for use in estimating maximum applied test loads were obtained by subtracting bending moments and shears produced by the girder dead weight. The results, plotted in Figures 7 and 8, show that a midspan load is the most critical for both flexure and shear and that shear is more critical than flexure. The maximum capacity for a midspan test load is 420.8 kN (94 600 lbf) for a flexural failure and 314 kN (70 600 lbf) for a shear (diagonal tension) failure.

Test of Girders 139-S1 and 139-S2

Both girders were loaded in the same manner, and the structural behavior under loading was essentially the same for both. The midspan test load was applied through a matched pair of hydraulic jacks spaced about 15 cm (5.9 in) on each side of the girder centerline. Heavy steel plates were used to provide lateral distribution of the load over the width of the slab. The test results are plotted in Figures 9 and 10.

The test ultimate capacity of both girders was 422.6 kN (95 000 lbf) compared to the predicted flexural strength of 420.8 kN (94 600 lbf). It is also interesting to note here that the test load developed an ultimate bending moment at midspan exactly equal to the ultimate moment that would be produced by a standard AASHTO

Figure 9. Test of girder 139-S1, midspan load.



Note: 1 mm = 0.039 in, and 1 kN = 225 lbf.

Figure 10. Test of girder 139-S2, midspan load.

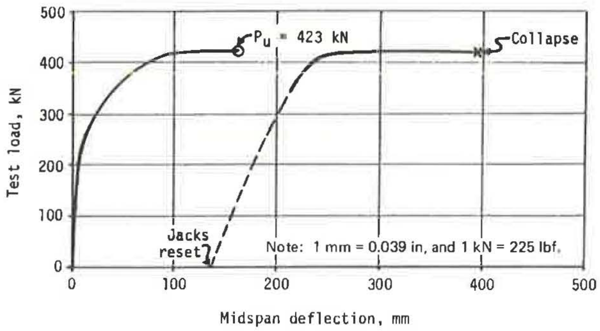
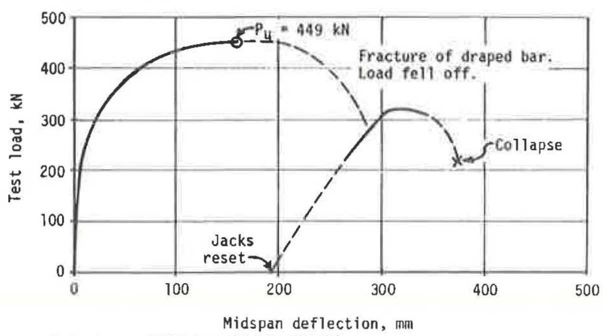


Figure 11. Test of girder 171-S3, one-third point load, showing one lower bar extensively corroded.



Note: 1 mm = 0.039 in, and 1 kN = 225 lbf.

HS20 truck load. Note that the predicted shear capacity was exceeded, even though the girder web was only 101.6 mm (4 in) and no web reinforcing was provided. This was the result of not considering the shear capacity of the flange, the redistribution of stress in the web after cracking, and dowel strength of the reinforcing in the flange when capacity was calculated.

Test of Girder 171-S3

This girder exhibited evidence of extreme corrosion of the prestressing conduits and bars at one end of the span. One side of the lower flange was spalled from the end block for a distance of about 2.2 m (7 ft), exposing badly rusted conduit and about a meter of rusted prestressing bar. The rusted surface of the bar was irregular, and the rust penetrated about 3 mm (0.12 in). The girder web was severely cracked at the level of the upper inclined prestressing bar. A single crack through the web followed the bar from the end block to the diaphragm block. After the test, when we removed the concrete to expose the steel, we found extensive rusting of the conduit and the bar.

We loaded the girder by placing a single concentrated load just outside of the diaphragm 4.3 m (14 ft) from the support. This produced the maximum bending moment in the damaged section of the girder and also produced high shear and diagonal tension in this region.

The test data are plotted in Figure 11. The first crack occurred under a load of 205 kN (46 000 lbf). The lower flange cracked vertically at the load point, and the crack progressed diagonally upward through the web as the test continued. As the load approached 445 kN (100 000 lbf), diagonal tension and flexure cracked the web extensively throughout the end region. At 449 kN (101 000 lbf), we heard the upper bar fracture, and the load quickly fell off to 294 kN (66 000 lbf) with a rapid increase in deflection. Later inspection revealed a brittle fracture of the upper bar at the load point and at a point on the bar where the surface had eroded to a depth of about 2 mm (0.08 in). After releasing the load, resetting the jacks, and reloading, a second bar failed at about 311 kN (70 000 lbf), and the girder collapsed. The second bar to fail was the one that had been initially exposed by spalling of concrete. The break occurred in the region of exposure near the end block.

A bond failure between the bar and the conduit permitted the bar to slide several millimeters as the beam "opened up" at the load point. This bar also failed by brittle fracture in a region of reduced cross section because of corrosion. The third bar was not corroded or rusted, and it failed ductily with extensive necking.

As in the previous tests, the maximum test load was essentially equivalent to the AASHTO HS20 truck loading for bending moment and shear in the region of the one-third point of the span (1 kN•m = 735 lbf/ft and 1 kN = 225 lbf).

Capacity	Test Load	AASHTO HS20 Load
M_u	1599 kN·m	1582 kN·m
V_u	347.4 kN	331.8 kN

Test of Girder 139-S3

Of the six girders removed from the Skyway bridge, 139-S3 was most extensively corroded. One side of the lower flange was spalled from the end block for a distance of about 3 m (10 ft), exposing badly rusted conduit, rusted prestressing bar, and rusted end block reinforcement. One prestressing bar had completely fractured as a result of extensive rusting. There was a 2-m (6.6-ft) long, barely visible crack running parallel to the lower prestressing bars on the side opposite the spalled side. After the load test, the steel was exposed and revealed extensive rusting of the conduits of both lower bars for an additional meter beyond the point exposed by the pretest spalling.

This girder was loaded in the same way as girder 171-S3, by placing a single concentrated load just outside of the diaphragm and 4.3 m (14 ft) from the support, and producing maximum stresses in the damaged section of the girder.

The test data are plotted in Figure 12. The first crack occurred at a load of 178 kN (40 000 lbf). The lower flange cracked vertically at the load point, and then the crack progressed diagonally upward through the web as the test continued exhibiting the typical diagonal tension crack. At a load of 320 kN (72 000 lbf), the cracks were so large we could see the prestressing bars in the flange and web. At 329 kN (74 000 lbf), the remaining lower prestressing bar fractured and the load

Figure 12. Test of girder 139-S3, one-third point load, showing one lower bar fractured before test.

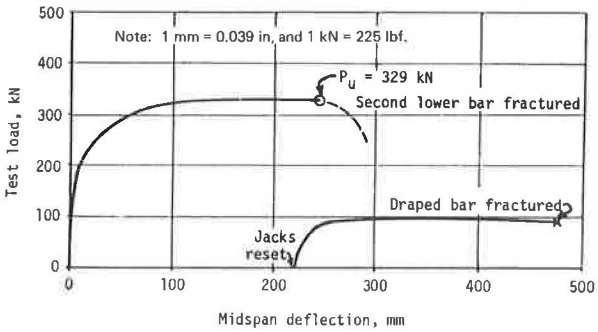


Figure 13. Test of girder 171-S2, one-third point load.

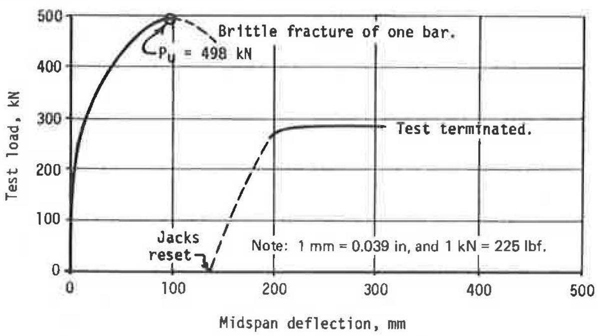


Figure 14. Typical brittle fracture of Macalloy bar.

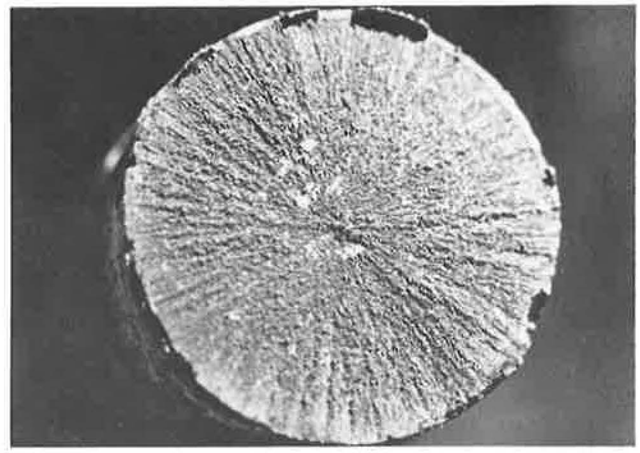
fell off. At this point the load was released, the jacks reset, and the test continued. At a load of 98 kN (22 000 lbf) the draped bar fractured.

As was expected, the draped bar, which showed no evidence of corrosion, failed ductily with extensive necking. The lower bar failed by brittle fracture in a region where it was pitted by corrosion.

The maximum test load is compared below (1 kN•m = 735 lbf/ft and 1 kN = 225 lbf) with the standard AASHO H20 and AASHTO HS20 truck loadings for ultimate bending moment at 4.3 m (14 ft) from the support.

Capacity	Test Load	AASHO H20 Load	AASHTO HS20 Load
M_u	1242 kN·m	1246 kN·m	1582 kN·m
V_u	263.8 kN	252.2 kN	331.8 kN

Figure 15. Typical ductile failure of Macalloy bar.



Although this girder did not develop the HS20 capacity that all the others tested did, it did essentially develop the H20 capacity, even though one prestressed bar was ineffective as a result of corrosion-induced fracture before the test.

Test of Girder 171-S2

This girder appeared to be in excellent condition. No spalling or cracking was present.

In order to show the percentage of strength loss from corrosion in previously tested girders 171-S3 and 139-S3, this girder was loaded in the same manner, by placing a single concentrated load just outside of the diaphragm and 4.3 m (14 ft) from the support.

The test data are plotted in Figure 13. The cracking load was 245 kN (55 000 lbf). This girder showed more uniform and evenly distributed cracking than previously tested girders, where damaged areas occurred near the load point. At a load of 498 kN (112 000 lbf), the load fell off. An autopsy following the test revealed that the draped bar failed by brittle fracture at the load position in the span. At this point the conduit and bar were located in the girder web with about 25 mm (1 in) of concrete cover. No spalling was noted, but the conduit was extensively rusted on one side, and the bar was slightly pitted.

The test was terminated before the two lower bars fractured, but later inspection revealed that they had undergone extensive ductile yielding.

The test load of 498 kN (112 000 lbf) is 10 percent higher than a concentrated load equivalent to an HS20 truck loading.

SUMMARY AND CONCLUSION

Of the five girders tested, only two were extensively damaged by the corrosion evidenced by spalled concrete and rusted steel. These same two girders were the only ones to exhibit a loss of strength when the failure load was less than the theoretical ultimate capacity.

Girder 171-S3 failed under a test load 8 percent below the theoretical capacity. A prestressed bar began fracturing at a point of corrosive pitting on the bar surface. The failure surface, magnified about five times, is shown in Figure 14, which clearly shows the propagation of the brittle fracture from the surface pit. The bars that were not pitted by corrosion failed in a ductile manner with necking and the associated loss of section area before breaking. Figure 15 shows the typical ductile

Table 1. Summary of test results.

Girder	Pre-Test Condition	Load Position	Predicted Ultimate Capacity (kN)	Test Collapse Load (kN)	Equivalent AASHTO Load	Apparent Loss of Strength (%)
139-S1	No apparent damage	Midspan	423	423	HS20	None
139-S2	No apparent damage	Midspan	423	423	HS20	None
171-S2	No apparent damage	⅓ point	489	498	HS20 plus	None
171-S3	Extensive corrosion	⅓ point	489	449	HS20	8
139-S3	Extensive corrosion; one bar fractured	⅓ point	311	329	HS20	33

Note: 1 kN = 224.8 lbf.

failure surface, when fracture propagates from the center of the section.

In girder 139-S3, one of the prestressed bars fractured before the girder was removed from the bridge. As expected, the girder suffered a 33 percent loss of strength as a result of the missing bar. Of the remaining two bars, one was corroded and failed by brittle fracture at near ultimate strength, and the other failed ductily with no evidence of corrosion.

Another brittle fracture occurred in girder 171-S2, although we saw no spalled or cracked concrete before the test. Here, again, the fracture was induced by corrosive pitting at the bar surface. However, the bar appeared to have reached the theoretical ultimate strength before failure.

The test results are tabulated in Table 1. The predicted ultimate capacity is the theoretical maximum concentrated load capacity for an uncorroded girder. The tabulated predicted capacity is based on flexural not shear strength, although the collapse mechanism was a combination or interaction of flexure and diagonal tension. When there was no obvious initial damage, the test capacity equaled or exceeded the predicted capacity. The equivalent AASHTO load includes consideration of load factors ($M_u = 1.5\ M_{dl} + 2.5\ M_{ll}$), distribution factor (1.09), and impact factor (1.30), in accordance with the AASHTO specifications. The test load for girder 171-S2 was 10 percent above the equivalent HS20 ultimate load. All other test loads were essentially the same as the equivalent AASHTO loads.

These tests indicate that on the Skyway bridge there is no immediate danger of girder collapse from corrosion. Even the 33 percent strength loss of one prestressing bar does not reduce the girder strength below that required for the AASHO design load. Furthermore, the load-deflection curves show that the girders are versatile enough ductily to redistribute the load to adjacent girders should one completely collapse.

Periodic visual inspection will reveal extensively corroded girders for selective removal and replacement. This process will provide time during which additional studies can be undertaken to evaluate remedial measures.

It is of interest to note that the corrosion occurs primarily in the end regions of the girders, where they are exposed to salt spray from waves splashing against the piers. Very little corrosion has been observed in the midspan regions even though the concrete is apparently less than 25 mm (1 in).

ACKNOWLEDGMENT

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Basic Evaluation of the Structural Adequacy of Existing Timber Bridges

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The properties of wood as a structural material in highway bridges are discussed. These properties give a timber bridge a very different load-carrying capability than that of a steel or a concrete bridge. Wood has the advantage of being able to sustain overloads for short time periods but is subject to decay and deterioration. By recognizing these characteristics, the engineer can determine the safe loading for the structure and can recommend procedures for prolonging its life. Guidelines are offered to assist the engineer in his work, and suggestions for presenting the information to the owner and recording it are made.

Wood is structurally very different from concrete or steel and therefore performs differently as the various