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Extending the Life of the Williamsburg Bridge

Prolongation de la durée de vie du pont de Williamsburg

Verlängerung der Lebensdauer der Williamsburg-Brücke

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SUMMARY

The decision to close the Williamsburg Bridge sent shock waves through transportation in New York City and brought the Williamsburg Bridge out from the shadows of the Brooklyn Bridge. The program to determine the present condition and the projected durability of the structure for possible rehabilitation rather than replacement is the subject of this paper.

RÉSUMÉ

La décision de fermer le pont de Williamsburg a eu des répercussions importantes dans le domaine des transports de la métropole de New York. Cette décision a aussi permis au pont de Williamsburg de se rapprocher de la position prépondérante qu'occupait le pont de Brooklyn. Le sujet de cet article est un programme pour déterminer l'état actuel du pont de Williamsburg ainsi que la durée de vie restant de cette structure, si elle devait être assainie plutôt que complètement remplacée.

ZUSAMMENFASSUNG

Der Beschluss, die Williamsburg-Brücke zu sperren, schockierte alle Verkehrskreise in New York und liess dieses Bauwerk aus dem Schatten der Brooklyn Brücke hervortreten. Der folgende Artikel beschreibt ein Programm, um den gegenwärtigen Zustand der Brücke zu ermitteln und ihre Eignung für eine Sanierung anstelle eines Neubaus abzuklären.

It was on April 12, 1988 that the decision was made to close the Williamsburg Bridge. This act sent shock waves through transportation in New York City, disrupting the lives of its residents and bringing the Williamsburg Bridge out from the shadows of the Brooklyn Bridge.

Long overshadowed by its famous neighbor, the Williamsburg Bridge also spans the East River in New York City linking the Brooklyn community of Williamsburg with Manhattan's Lower East Side. When completed in 1903, it

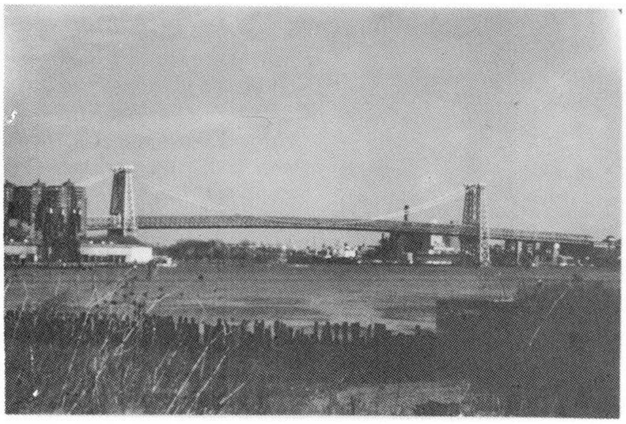

Fig. 1 Williamsburg Bridge

was the world's longest suspension bridge with a main span of 1600 feet [487.7 m] (5 feet [1.52 m] longer than the Brooklyn Bridge). The bridge has side spans of 596'-6" (181.87 m) which are not suspended but are supported by three intermediate towers. The designer's goals were to build a bridge longer, stronger, cheaper and faster than the Brooklyn Bridge, resulting in the world's first suspension bridge with steel towers and non-galvanized steel wires.

As originally constructed, the bridge had two outer roadways, four inner trolley tracks, two inner train tracks, and a bicycle path and footpath. The structure has gone through many modifications throughout the years and

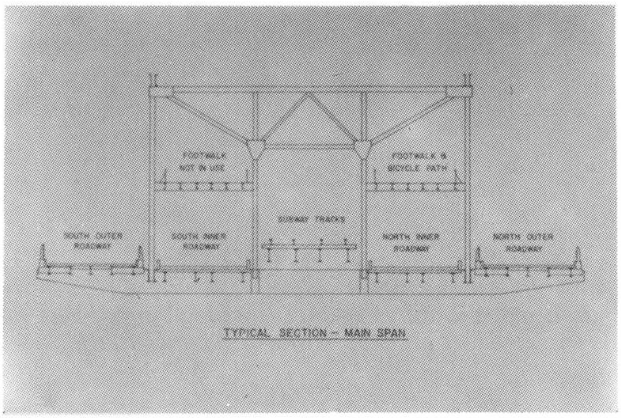

Fig. 2 Bridge Cross Section

presently has two outer roadways (2 lanes of HS20 on each roadway), two inner roadways (2 lanes of H10 on each roadway) two BMT tracks and one footwalk.

Steinman was engaged by the New York State Department of Transportation (NYSDOT) and the New York City Department of Transportation (NYCDOT) to perform two separate inspections of the Williamsburg Bridge; the Biennial Inspection which was a complete inspection of the entire bridge, and the Cable Inspection which included inspection, wire sampling and testing of the main cable wires in order to determine the present strength of the cable and its remaining useful life.

The Biennial Inspection was performed in accordance with NYSDOT criteria whereby all deteriorated and fracture - critical members receive a "hands on" inspection and all others a visual inspection. Due to extensive deterioration and non-redundancy of the structure, most of the 15,000 primary members had a hands-on inspection.

The work commenced in November 1987 on the main span since it was the easiest to inspect and the inspection had to continue through the winter months. The underdeck was easily accessible via the maintenance traveller and the inspection continued uneventfully throughout the winter with only four flags reported (all minor in nature). A structural flag identifies a structural condition which may be a potential threat to public safety. It does not signify imminent danger, but rather a location that needs further analysis, monitoring or repair.

In early Spring, the inspection shifted to the end spans. The underdeck of the end spans was more difficult to access since there is no maintenance traveller. A mobile underdeck inspection platform called a "Moog" with a 56' (17 m) horizontal reach was used to access the underdeck of the inner and outer roadways. The 56' (17 m) Moog is tractor trailer mounted, self propelled and occupies only one lane so that one lane of traffic could be maintained. Rigging had to be installed below the center of the end spans since the moog platform could not reach the tracks.

The first major areas of deterioration were found on the Brooklyn Bound outer roadway cantilever floorbeams on March 28, 1988. The webs of the floorbeams adjacent to the stiffening truss (at location of maximum moment and maximum shear) were severely deteriorated (many with through holes) in an "L-shaped" pattern vertically along the truss chord and horizontally along the floorbeam bottom flange angle. Upon being notified of this condition, the NYSDOT and NYCDOT decided to close the roadway to traffic. Considering the condition of the Brooklyn bound roadway, the cantilever floorbeams on the Manhattan bound roadway were then immediately inspected and similar conditions were found. This roadway was subsequently closed to traffic. In total, 46 out of 116 floorbeams required emergency repair.

The severity of the deterioration resulted from the fact that the roadway has no drainage system and water continually flows off the roadway onto the structural steel below. Modern design calls for a drainage collection system but frequently the discharge is not carried below structural members resulting in unnecessary corrosive conditions.

The Transit Authority was now concerned about the condition of the floorbeams on the approaches after seeing the end span floorbeams, so the emphasis of the inspection switched to the approach track structure in early April. The next day on April 11, the transit track floorbeam at

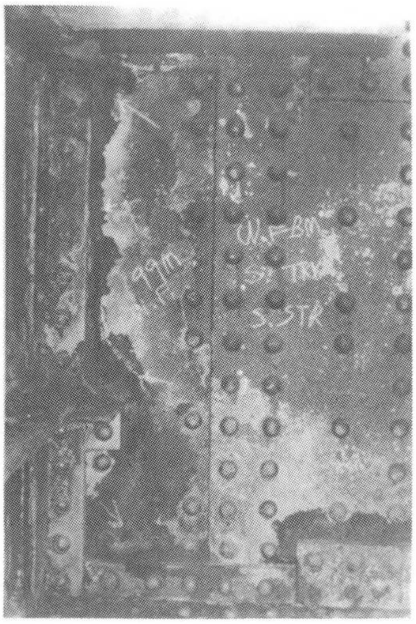

Fig. 3 Web Deterioration

PP99 was found to have its web completely deteriorated between the flange angles at the point of near maximum shear adjacent to the exterior stringer. Upon being notified of this condition the Transit Authority immediately suspended service across the Williamsburg Bridge.

The South inner roadway floorbeam at PP99 was also found to be completely deteriorated at its connection to the girder, except for about 9" of web. Fortunately, a knee bracket initially installed to provide rigidity was able to transmit the end reaction and thus avoid failure. At this point, with the concurrence of the Mayor, it was decided to completely close the bridge to expedite inspection and repair.

On April 12, 1988, the inspection of the main bridge and 4,300 feet (1,310.6 m) of approach structure was about 45% complete and Steinman was instructed to complete the inspection in three weeks. To accomplish this, the inspection crews were increased from 5 to 17 and the work period increased to 10 hours per day, seven days per week.

As the inspection progressed, typical patterns of deterioration became evident, due largely to the open expansion joints and open curbs which allow water to run onto the superstructure below. This included deterioration of the portion of the floorbeams between the inner and outer roadways; floorbeams, stringer ends and girder ends at expansion joint locations; exterior roadway stringer flanges; and the transit floorbeam ends on the Manhattan Approach where the tracks are below roadway level.

This calls attention to another design necessity, the elimination of expansion joints or at least the minimization of them to enhance durability.

Another typical pattern was the cracking of the transit track stringer top flanges. This condition resulted from the fact that the gauge of the tracks is less than the stringer spacing which produced bending and end rotation of the ties. The repeated tie rotation was transmitted to the outstanding leg of the stringer top flange resulting in a local fatigue failure of the outstanding angle leg. A design with a more appropriate spacing could avoid this problem.

It must be mentioned that it was a herculean task to design and certify the repairs. As with the inspection, this work went on 10 and 12 hours per day and some days even 24 hours. Lanes were opened sequentially, trains were back in service after two months and service was completely restored in three months.

Let us learn that a maintenance manual must be an inherent part of a bridge design and an iron-clad funding program for maintenance must be in place when a bridge is opened.

One of the key elements in the decision to rehabilitate or replace the Williamsburg Bridge had to be a consideration of the condition of the main cables. There are four main cables each composed of 37 strands with 208 wires per strand, 0.192 inch (0.488 cm) diameter ungalvanized (bright) wires for a total of 7696 wires per cable. These main cables had been under study since early in 1980 in an attempt to determine their useful life. These studies concluded that the cables would have to be replaced. The Federal Government questioned the wisdom of replacing the cables and rehabilitating the bridge since they suspected that a new bridge providing present design standards would be cost effective. This question led to a review of the early studies in which the wire testing program used only surface wires and the models for remaining life of the cables used extremely conservative clamping lengths and corrosion rates.

It was in this climate that the Steinman Firm was called upon to do a more indepth evaluation of the cables.

The Steinman program included a visual inspection of the cable exterior throughout the entire length of the four cables; an in-depth inspection of the cable interior including a representative wire sampling program; the wire testing program to be performed in Carleton Laboratories at Columbia University in New York City; an investigation of cable "D" in the Manhattan anchorage; an inspection of cables "A" and "B" at the Manhattan Tower that were damaged by fire in 1902 while the bridge was under construction.

The four main cables were made of ungalvanized steel wires in order to provide more carrying capacity for a given unit cost. To protect the ungalvanized wires they were oiled, wrapped with canvas strips and then encased within a metal sheathing. In 1920 rusting of the sheathing was discovered and the old wrapping system was removed and the cable was wrapped with galvanized wire. The wire was painted but no protective coating of red lead paste was used under the wrapping.

Our visual inspection of the wrapping found that the paint had peeled from the wrapping throughout most of its length but especially at the bottom of the cable. The wrapping had corroded at many locations and there were damaged

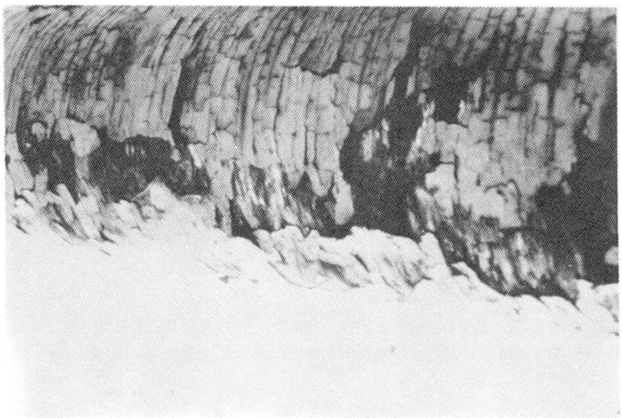

Fig. 4 Cable Paint Condition

sections of the wrapping that permitted water to penetrate to the cable. The locations of the worst situations plus a statistical approach led to the decision to pick five locations at which to unwrap cables and perform the sampling programs.

The five locations of statistical sampling were the lowest panels of the main span cables "A" and "D", an adjacent panel of cable "D", two panels near the quarter point of the mainspan on cable "D", and the lowest area near the Manhattan anchorage of cable "B".

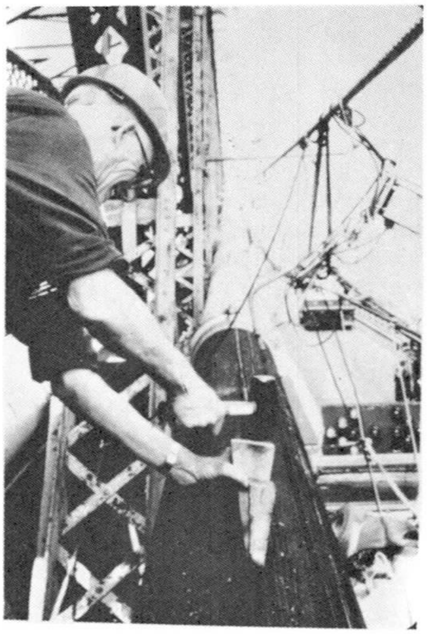

Fig. 5 Wedging the Cable

The cable was divided into eight segments and samples at depths of surface, 2 1/4 inches (5.72 cm), 4 inches (10.16 cm), and 7 inches (17.78 cm) were taken for a total of 32 samples at each of the five locations.

The procedure was to open a groove in the cable using wooden wedges and hydraulic jacks, visually inspect and take macro lens photographs, extract, coil and tag the sample for shipment to the laboratory. As the wire was cut the retraction of the wire was measured. The groove opening was then cleaned using a soft wire wheel brush or a cup wire brush. Samples of the corrosion and existing lubricant were then collected and sent to the laboratory for analysis. After cleaning photographs of the wire were taken at five foot intervals. One coat of protective oil (Vitalife 400) was then applied in the groove opening. After this procedure was completed at the eight groove openings, additional protective oil was applied liberally into the cable at that work location. The cable was then wrapped with Herculite and the inspection proceeded to the next work location.

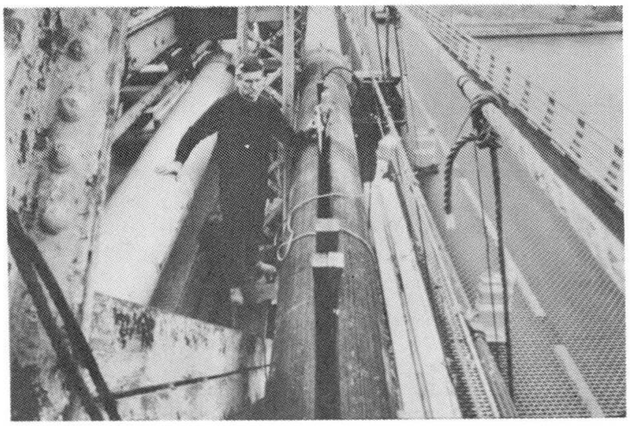

Fig. 6 Hydraulic Jack

In addition, if any in-situ broken wires were encountered, these were also sampled and sent to the laboratory for testing.

The testing of the samples was done in accordance with a testing program established specifically for 0.192 inch (0.488 cm) diameter uncoated stress relieved wire. The program was developed with assistance from a team consisting of a metallurgist, a corrosion expert and a statistician all from Columbia University.

1. TESTING PROGRAM

1.1 Initial Examination

The sample was photographed and cut into 18 inch (45.72 cm) sections. The nature of the surface coating was recorded and the section rated as to its degree of corrosion. The corrosion grades had been established from 0, (least corrosion) to 5 (worst corrosion). Measurements of the wire diameter were made.

1.2 Tension Test

A statistical determination was made of the number of 18 inch (45.72 cm) segments of the specimen to be tested in accordance with ASTM A370-IV specifications. The stresses were calculated based on the nominal wire diameter of .192 inches (0.488 cm). A reduction in area measurement was taken at the fracture point and the permanent elongation in 10 inches (25.4 cm) was measured.

1.3 Stress - Strain Tests

Stress-strain curves were plotted for selected specimens of various corrosion grades.

1.4 Fatigue Test

Fatigue tests were made of selected specimens based on carbon content and corrosion degree.

1.5 Chemical Analysis

Twelve selected specimens were tested in the Materials Bureau of the New York State Department of Transportation to determine the chemical composition of the wire. All wires were tested for carbon content.

1.6 Corrosion and Fractographic Studies

Specimens of each corrosion category were examined under a Scanning Electron Microscope (SEM) to determine loss of area and the type of corrosion development. The tips of fracture surfaces were examined in an SEM to classify them by recurring fractographic patterns based on the mechanics of the fracture.

2. TEST RESULTS

2.1 Tension Test

The overall average tensile strength was about 218 ksi (15 N/mm^2). The overall average of the minimum tensile strength for each wire was about 212 ksi (14.62 N/mm^2). Permanent elongation of wires averaged 2.6%, per ASTM A-370. Reduction in area varied somewhat between samples, but averaged about 20% for all tests at the four independent locations sampled. Deducting breaks outside the gage length did not significantly alter the average UTS or reduction of area.

2.2 Fatigue Test

The fatigue tests provided enough information to indicate fairly consistent fatigue characteristics. The corroded specimens tended to have fatigue properties almost as good as the grade "0" specimens.

2.3 Chemical Analysis

The significant finding was that the carbon content, determined by the total combustion method was found to vary from 0.59% to 1.06%. The tensile strength and other mechanical properties were found to be more affected by the carbon content than by corrosion grade. The relationship between carbon content and the minimum breaking strength of 18 inch (45.72 cm) specimens was found to agree very closely with values expected by comparing the data with standard Ultimate Tensile Strengths (UTS) vs. Carbon (C) charts.

2.4 Fractography and Corrosion

Fewer than 10% of the tensile specimens shew fractures originating at the surface. The others initiate fracture predominantly near the center, by coalescence of the microvoids which develop during plastic elongation. This implied that the wires are "ductile".

For a 10-ft. (3.05 m) section of cable at mid-span, the average cross section is reduced to 96% of the uncorroded state. The most corroded wire of the samples from the wrapped portion of the cable retained 79% of its area. Using the most probable date for insipient significant corrosion as 1930, the linear model corrosion rate is .189 mil (0.0048 mm) per year.

The worst condition of the cable in the anchorages was found at cable "D" in the Manhattan anchorage. Several dozen broken wires were protruding from a few strands on the south side of the splayed cable. Many wires had been spliced with short sections of galvanized wire between ferrules. Some of these wires were under load while others had slacked off.

Several wires were heavily corroded near the splay casting, with significant loss of cross sectional area. Corrosion exists at all exposed wires along the top surface of the cable where it emerged from the original splay casting, especially in the first half meter or so from the casting, where large amounts of straw, bird feathers, and other material were found packed between the outer and inner shrouds that closed off the space between the cable and the front stone wall of the anchorage. The heaviest concentration of corroded wires is at the south side of the casting, where groups of wires have corroded to 3/32" (2.38 mm) or less dia and several have broken. At strand 19 and/or 20, several wires have corroded to less than half their original areas, without failing. Close examination revealed apparent plastic elongation, within the corroded areas, that extended over a few centimeters of length of the wires. This apparently resulted in increase of the unstressed length of the wires over the length between strand shoe and splay casting so that the stresses in these wires were kept below the ultimate, even at the smaller cross-sectional areas.

There was considerable concern, by NYCDOT about the condition of the cable under the splay casting. Consequently, the casting was shifted to a new location by leap-frogging clamping bands to the new location of the splay casting. The subsequent inspection found that there were no additional broken wires at the location of the splay casting.

On November 10, 1902 a fire broke out on the top of the Manhattan Tower after the main cables had been erected. The cable saddles were being filled with cable compound to protect the cables and the compound caught fire at saddles "A" and "B". Many wires that were directly in the fire became red hot and some became annealed, losing much of their strength. About 400 wires were spliced in at that time to compensate for this damage.

The 1988 inspection of the area consisted of using wedges to open the cable at the six and twelve o'clock positions. Wires were removed at two, four and six inches (5.08, 10.16, 15.24 cm) deep at each groove. The corrosion grade of the wires ranged from 1 to 3. Tensile-strengths ranged from 197 to 223 ksi (13.58 to 15.38 N/mm^2) which is very close to the overall average. A fatigue test indicated that the wires have not suffered significant degradation in this area.

Using the data found in the wire testing program, factors of safety for the present cable were developed as well as projections for the future life of the cable.

The factors of safety for the cable had to be developed separately for the unwrapped portion of the cable, the anchorage section and the tower top. The wrapped portion of the cable is more difficult to repair as compared to the anchorage where the strands can be replaced with relative ease. Furthermore, various models were developed to express the factor of safety.

For one model, contours of the breaking loads of tested wires at each inspection location were plotted and all wires below 5500 lbs. (24.46 kN) and any broken wires found at that location were discarded from consideration. The capacity of the remaining wires divided by the maximum load gave a minimum factor of safety for the wrapped portion of the cable of 4.0 at mid-span.

The staff at Columbia University also developed a brittle wire model, a ductile wire model and a ductile-brittle wire model to further develop factors of safety. However, after finding that the wires in strands at the anchorages behaved elastically the brittle models could be safely discarded. The ductile model, while much more conservative than the contour model, still provided a factor of safety of 3.98 at mid-span.

In the anchorage, safety factors were computed continuously during geometry change work and had been updated as variations in eyebar tensions were reported. The most recent calculation shows a factor of safety of 3.48, locally at the Cable D Manhattan Anchorage, due to the large number of broken wires. The ongoing repair work will increase this factor of safety to near 4.0.

The testing of the sample wires from the fire damaged area at the Manhattan tower has shown that there is no degradation in strength at this location, from the fire damage or subsequent corrosion. There has been no significant loss of wire area and the computed factor of safety is 3.89 at this location.

The developed data was used to establish a corrosion rate and clamping length and various maintenance models were developed to forecast a cable life for the wrapped portion of the cable. A minimum maintenance model and a historic maintenance model indicated a safe minimum life of over fifty years. However, with rehabilitation and special maintenance there should be no further deterioration of the cable.

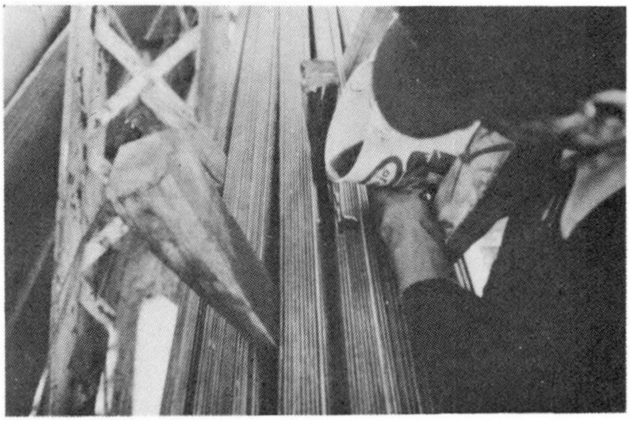

Fig. 7 Applying Protective Oil

With the results of this in-depth investigation at hand, the decision to proceed with the rehabilitation rather than replacement of the structure could be made. An opportunity now exists to rehabilitate the structure eliminating those causes of serious corrosion and establishing a rehabilitation design and maintenance program developed to optimize the life-cycle costs of a durable structure.

Acknowledgement of the great effort put forth by the staffs at Steinman, NYCDOT, NYSDOT, and Carleton labs in accomplishing this work in minimum time should be recognized. Thanks go to the Technical Advisory Committee for their input and particular thanks go to Kenneth P. Serzan, P.E., Project Manager, for the Bridge Biennial Inspection and to Terry L. Koglin, P.E., Project Manager for the Cable Investigation.

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