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Durability Assessment from the Design Phase to Execution

Prise en compte de la durabilité du projet à l'exécution

Berücksichtigung der Dauerhaftigkeit vom Entwurf bis zur Ausführung

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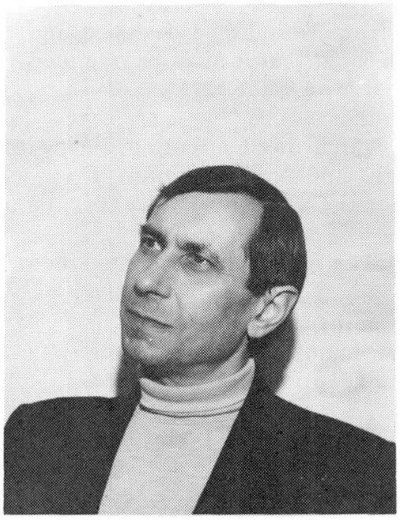

Daniel Poineau, born in 1937, studied engineering at the ENTPE. Throughout his carrier he has worked on engineering structures. For about fifteen years, Daniel Poineau has been specialised in pathology and repair of concrete bridges.

SUMMARY

Durability assessment of bridges from the design phase, constructive measures and execution is dealt with in this article under the following four aspects: assessment of factors which affect the operational life; assessment of necessity for inspection and maintenance; assessment of necessity for adaptation or replacement; effect of construction methods on durability.

RÉSUMÉ

La prise en compte de la durabilité des ponts au niveau du projet, des dispositions constructives et de l'exécution est abordée dans cet article sous les quatres angles suivants: prise en compte des facteurs qui affectent leur durée de vie; prise en compte de la nécessité de leur visite et de leur entretien; prise en compte de la nécessité de leur adaptation ou de leur remplacement; incidence des méthodes d'exécution sur leur durabilité.

ZUSAMMENFASSUNG

Die Berücksichtigung der Dauerhaftigkeit von Brücken bei Entwurf, Ausschreibung und Ausführung wird in diesem Beitrag unter vier Aspekten diskutiert: Berücksichtigung der Faktoren, die die Lebensdauer beeinflussen; Berücksichtigung von Inspektion und Unterhaltung; Berücksichtigung der Notwendigkeit ihrer Umbaues und Entfernung; Einfluss der Baumethoden auf die Dauerhaftigkeit.

1. INTRODUCTION

Throughout its operational life, a structure must fulfil its functions, with an acceptable probability, without incurring excessive investment or maintenance expenses, without its operation being interrupted except for short periods for maintenance (usual or specialised), repairs, or even strengthening or changes and lastly without suffering notable damage with regard to safety and comfort of users and third parties.

To obtain these objectives, the structure must be designed and built with an initial adequate quality. It should be emphasised that it must be operated with the same concern for quality, which unfortunately is not always the case.

I shall try to review four points which merit attention during the first two stages of the life of the structure, i.e. "design" and "construction", if we wish to obtain the abovementioned objectives meaning a "satisfactory durability" for the structure.

Three of these points should be taken into account at the structure design stage:

- factors which can affect its durability;
- the requirement for supervision, maintenance, even repair;
- the necessity of replacing it and adapting it.

The fourth point deals with both the design and construction phase as it concerns the effect of construction methods on durability.

My report will be oriented towards concrete bridges (reinforced concrete and prestressed concrete) and metal bridges. The construction principles I shall discuss are, of course, applicable "mutatis mutandis" to other engineering structures.

2 - REPORT

2.1 - Assessment at design phase of factors which may affect the structure durability

2.1.1 - General

At the design phase, all the external factors should be listed and classified which might have major unfavourable effects on the durability of the future structure during its service phase but also during construction. Afterwards, this is taken into account in the choice of structure type, certain construction specifications, calculation, quality of materials to use, various protection systems....

It is possible to separate factors having a "negative influence on durability" into two categories:

The first category lists the factors which can cause major structural disorders or even ruin the structure. Generally their actions are sudden.

The second category lists factors which downgrade the structure by attacking the component materials. Generally, the disorders appear slowly. However, when they have reached a certain magnitude, they also lead to major structural disorders and ruin the structure.

2.1.2 - List of factors in the first category

We refer here to accidental occurences or else factors whose effect can be equivalent to those of accidental occurences. We can mention:

- earthquakes;
- tropical cyclones;
- impacts due to road vehicles - Figure 1 or rail traffic, boats or even aircraft;
- fires and explosions;
- broken piping;
- slippage;
- landslides and avalanches;
- mining subsidence;
- local, general or regressive under mining;
- floods;
- human error....

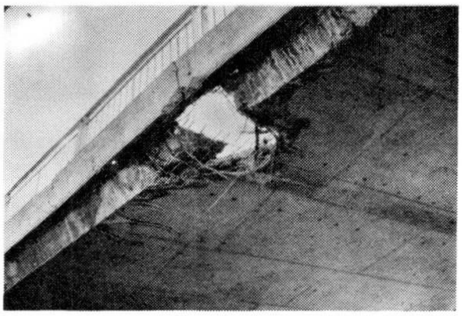

Fig. 1

To illustrate my report here are three typical examples:

. The first example is that of the new cable stayed bridge in TJORN (Sweden) whose pylons were installed on the banks necessitating a central span of 366 m. This structure is designed to protect the deck and pylons from boat impacts. This structure replaces a metal arch bridge with 278 m span destroyed on 18 January 1980 by a ship which collided with the arch near one of the abutments outside the shipping lane where theoretically it should have been sailing.

. The second example is that of a bow-string metal bridge in BELGIUM which was destroyed on 14 August 1985 in about ten minutes due to a fire which broke out after an explosion of a gas pipe attached underneath the deck.

. The third example concerns the tipping over, during the construction phase, of a bridge balance beam built with successive cantilevers using prefabricated segments. The balance beam was resting on two wedges forming simple supports and safety was provided by the unsymmetrical counterweights and the order of installing the segments. A human error led to reversing the fitting process which caused the static balance to be broken. This accident has led the French government to impose stability regulations for balance beams enabling the static balance to be verified and secondly resistance of all structural elements (deck section, provisional support equipment, pile shaft, foundation...).

2.1.3 - List of factors in the second category

This refers to factors which might attack the different materials used in building bridges and their accessories (screed, bearing fittings, pavement joints...). These different factors may be classed according to their origins under the following headings:

- actions exerted by the environment
  . sea air;
  . industrial atmospheres;
  . either natural or industrial water, soft or hard;
  . certain soils.

- climatic influences developed by:
  . sun;
  . frost and thaw;
  . rain often containing aggressive agents due to pollution;
  . wind....

- influences exerted by the different living organisms:

  . man (pollution, vandalism...);
  . animals (defecations, underground passageways...);
  . vegetal (effects produced by growth of plants);
  . micro-organisms such as certain bacteria....

- influences exerted by traffic:

  . earthwork machines;
  . HGVs;
  . exceptional convoys.....

- influences exerted by materials or their components on one another:

  . certain cements reacting with certain aggregates;
  . ordinary steels corroding after contact with stainless steels.

The different factors mentioned above cause damage to materials by the following main processes:

- chemical and physico-chemical reactions. For instance, carbon dioxide causes carbonation of concrete coating the reinforcement which leads to depassivation of steels and corrosion).

- electrochemical reactions (for instance in the presence of water containing salts which form an electrolyte, the richer sections of a steel part will take on the role of a soluble anode compared to the more heterogeneous parts which take on the function of the cathode. A corrosion will result - Figure 2).

- physical reactions (for instance at low temperature metal materials have a reduced deformability which can lead to brittle phenomena in the case of impact or presence of nicks - Figure 3).

- mechanical reactions (for instance, repeated loads on a structure can lead to fatigue phenomena).

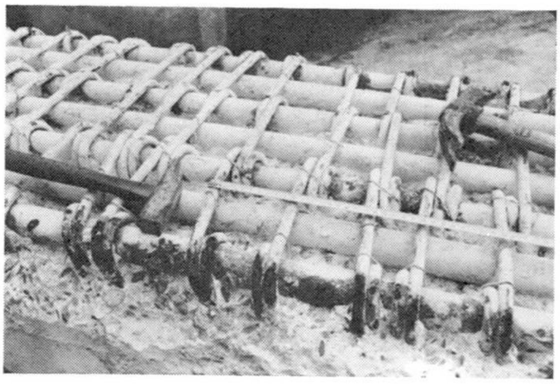

Fig.2

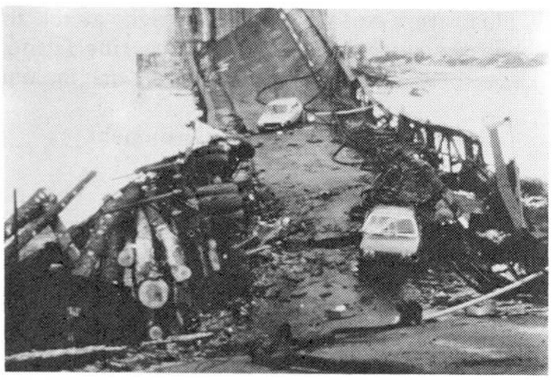

Fig. 3

To illustrate what I am saying, here are two typical examples which show the measures and precautions to take in the fight against certain factors which cause damage to materials.

. first example concerns the fight against water which is the main enemy of bridges especially as its effects are aggravated by thawing salt and the frost-thaw process. The designer must conceive the structure in order to:

- evacuate water by giving a bridge a longitudinal slope of at least 0.5% or better 1% (attention should be paid to top and low points where the water can stagnate) and a transversal slope of 2% or better 2.5%;
- avoid the water stagnating on the different bridge elements (mouldings, head rail of bridge railing, supports or piles and abutments) by providing these with slopes and arrangements such as flaps, water bars, gutters to guide the water outside;

- prevent the water from penetrating in the deck by protecting the whole surface with a sealing course. The deck itself can be self-protected by wide cantilevers;

- design solid, simple shapes which provide a minimum area open to aggression and which enable correct installation of concrete;

- provide the use of strong sheets for a metal deck and make sure that the beam ends are protected against drains and condensation;

- use air entrainers to improve the resistance of concrete to frost-thaw (the characteristics of the bubble network should generally be as follows: the spacing coefficient $\leq$ 200 um and the volume $\geq$ 25 mm^{-1}).

The second example concerns the choice of protection procedures against corrosion and warranty periods which apply within the scope of French public works contracts.

With regard to painting, there are different warranty periods concerning anti-corrosion and secondly the appearance (separation, pealing and blistering - changing colour - change in surface film) according to the following three parameters:

- firstly: classification of structures into three categories according to minimum thickness of elements ($\geq$ 8 mm, $\geq$ 4 mm, < 4 mm);
- secondly: surface preparation conditions (degree of stripping or scraping and brushing);
- thirdly: the protection system used in the three categories A, B and C. (Systems A benefit from an approval with control).

Furthermore, the warranties take into account exposure conditions. They are applicable to the following structures:

- overhead structures in country, urban or industrial and coastal atmosphere;
- structures submerged in soft water, sea water and aggressive water.

Certain structures are excluded which give rise to special research:

- structures located inside or immediately leeward of industrial or chemical complexes;
- structures in tropical atmosphere;
- structures submerged and protected by hot galvanising with electrolytic zinc plating.

2.2 - Assessment at design stage for the necessity of monitoring, service or even repair of structures

2.2.1 - General

During its existence, a structure must undergo maintenance work or even repairs. To carry out these maintenance operations in good time the structure must be monitored.

Therefore, a note must be made at the design phase of all sections of the structure which must be monitored in order to make them accessible for inspection in total safety, and secondly the sections which must be maintained or even repaired in order to make these operations easy to execute.
Furthermore, when the structure is completed it must be officially handed over to the manager by the chief consultant accompanied either by a visit and maintenance file if the structure is complex or large, or otherwise, a simple manual.

These two documents describe the monitoring actions and their frequency, specifying in what conditions and with what precautions maintenance or repair work should be carried out.

Normal bridges which cross over a road or are located at a low height above the ground are generally accessible by simple methods (ladder, scaffolding, platform on vehicle, inspection platform). The necessity of the inspection and service therefore has an influence on the design. However, access should be provided to the tops of piles and springers of abutments, bearing fittings and drainage and water discharge devices.

For the other structures, i.e. normal bridges with difficult access (over water, inaccessible overhanging ground, railway) and exceptional structures (suspension bridges or cable stayed bridges, box plate girder bridges, arch bridges). The necessity of inspections and maintenance influences their design and vice versa as the rest of my report shows.

2.2.2 - Effect on the deck design

. It is generally preferable to provide separate decks to support motorway pavements or assimilated in order to be able to execute maintenance operations wihtout traffic.

. The access to the outside of the deck can be provided by means of specialised commercially available machines such as electric foot-bridges if the following main measures are taken:

- the central space between two paired decks must allow clearance of the suspension column of the foot-bridge if the horizontal penetration of the foot-bridge is less than the deck width;

- the deck height must be adapted to the vertical penetration of the foot-bridge;

- the pavements, if they are wide, must allow the foot-bridge carrier vehicle to circulate (absence of high curves and fragile slabs);

- the lateral obstacles such as lamp posts and overhead traffic signs are to be avoided.

The presence of anti-noise shields requires the use of certain specific models of electric foot-bridges.

In the cases where usual inspection and maintenance machines cannot be used a special foot-bridge should be provided under or over the structure or else longitudinal fixed foot-bridges.

Special equipment is used for the large cable stayed bridges.

. Access inside the deck (for box girders) requires the presence of openings of sufficient dimension to enable personnel and maintenance equipment to pass through for installation or replacement of structures of utility companies (water pipes, electrical cables).

These openings must not be accessible to the public. Therefore, they are blocked off with doors, grids or trap doors which lock with a key. In addition, there must be no danger to the maintenance personnel and they should not adversely affect the structure durability. Consequently, manholes under pavements are formally advised against due to risk of accident and their doubtful watertightness. Furthermore, openings made in the piles and deck should be proscribed due to risks of falls from a great height.

Inside the deck, as far as possible, a sufficient space should be reserved to allow personnel to pass through. The passageways should be free of all obstacles or else these should be indicated by white paint and/or a local reinforcement of lighting.

In line with the spaces, or bridge parts there should be ladders or stairways plus openings for the passage. Figure 4.

It is desirable to light up the inside of the deck (20 to 25 lux). This signal lighting can be completed by headlamps provided the necessary sockets are reserved. The power supply can be either from the mains or a power plant.

The different sections of the structure must be indexed and oriented (numbering of segments and bearings, differentiation between upstream and downstream...).

2.2.3 - Effects on the design of bearings (piles and abutments)

Most of the recommendations on access, circulation and lighting in the decks apply to the bearings.

. Access to the outside of the piles can be provided by means of a suspended platform with a foot-bridge or more simply using a scaffolding provided the top of the pile has a small wall where the scaffolding suspension brackets can be secured.

. Access inside the hollow piles is from fire ladders intersected by stairs and if necessary a hanging scaffolding.

. The bearing supports must be designed to allow adjustment and changing of bearing fittings. It is desirable to have a minimum free height of 0.15 m to be able to install jacks and safety wedges where necessary.

With regard to large size supports and all those located at a considerable height, the free height must be increased (by 0.40 m to 0.80 m or even more) to allow the workers to operate in the space located between the top of the deck and the bearing supports - Figure 5.

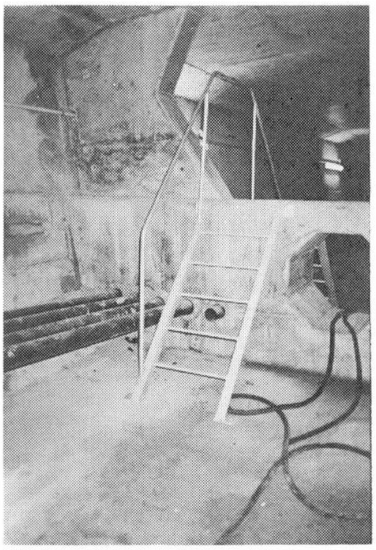

Fig.4

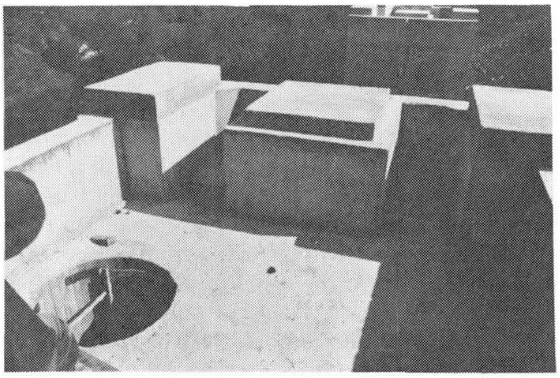

Fig.5

. With regard to the abutments, it is advisable to provide brackets at the end of the deck and on the parapet to provide a sufficient space between the two elements. This facilitates access for cleaning and maintenance (unblocking discharge pipes, renewing paint) and secondly ventilation of this damp zone as the expansion joints only provide a perfect seal in rare cases.

. Drainage and discharge of water from the bearings must be designed to be easily maintained. The water down pipes must have the most direct route possible and be accessible. For instance, with regard to abutments, it is preferable to discharge water by prefabricated ducts installed on the embankments rather than embed the pipes in the ground. In fact, pipes break under the consolidation of embankments with all the damaging consequences on the behaviour of the latter and the pavement.

2.3 - Assessment at the design stage of the necessity for adaptation and replacement of a structure or part of a structure

2.3.1 - General

In the case of bridges, these problems are initially raised for all equipment accessories (bearing fittings, pavement joints...) whose durability is considerably less than that of a bridge. The replacement or adaptation of these different elements is part of the specialised maintenance which has just been mentioned in the second point of my report. Therefore I will not bring up these problems again.

Apart from the equipment accessories, the problems of adaptation or replacement of a structure or one of its parts occurs for instance:

- for "strategic" structures for which the interruption of traffic flow is difficult to envisage;
- for structures whereby certain structural sections have a life which is probably quite less than that of the remaining structure or for structures whose life is reduced;
- for structures intended to be duplicated or enlarge or submitted to higher loads;
- for structures subjected to relatively unknown rheological phenomena.

2.3.2 - Some examples of structure adaptation

First example: It is rare to meet owners who, prior to start-up of a project research on a bridge, have precise ideas on the future of the structure. However, this does exist and was the case for the MECHELLE bridge project in Nancy which perfectly illustrates my arguments.

This structure, completed in December 1987, currently crosses the lake of the MECHELLE and the MEURTHE. It was designed to be used as an interchange for a future ring road which would use the current bed of the MEURTHE which would be transferred to the lake and made navigable.

This future adaptation led to three projects being studied:

- the project of the current bridge which has two lanes over part of its length and then three lanes;
- the project of an enlarged bridge to three and four lanes respectively;
- the project of the interchange bridge with creation of a crossroad on pile number two - Figure 6.

Furthermore, two piles had to be designed to the right of the lake capable of taking the impacts from boats.

Example two: In France for the last fifteen years, major structures in prestressed concrete have been designed to receive a complementary prestressing and an additional prestressing. The complementary prestressing is provided to be used during the construction phase. For this purpose, empty ducts are reserved to house the prestressing reinforcements if checks reveal insufficient prestressing (excessive friction, breakage of wires, wiring errors). If the empty ducts are not used, they are injected with a grouting compound when construction is completed.

Additional prestressing is planned to be used during the operational phase. Reserved spaces are made in line with the spaces to allow passage and fixture of external prestressing reinforcements to the concrete. This addition prestressing is installed if monitoring proves this necessary.

2.3.3 - Some examples where structure or structural sections are replaced

First example: Experience has shown that certain types of structures can have shorter lives than other equivalent structures when maintenance is neglected, and when aggressive agents (ice clearing salts, industrial waters) are present in the zone affecting the structure.

This may be the case for metal tubes too, the recommendations and state of the art developed by the Central Laboratory for Bridges and Highways (L.C.P.C.) and the Roads and Highways Engineering Department (S.E.T.R.A.) advise providing an extra template of about 15 cm in order to allow repair by the sheathing technique or shotcrete.

Second example: In France over the last few years external prestressing of concrete has been used in certain major structures usually in combination with internal prestressing of concrete.

Currently, this external prestressing can be removed and replaced. For this purpose, the prestressing reinforcements are arranged under a double tubing in line with deflectors and fixtures - Figure 7.

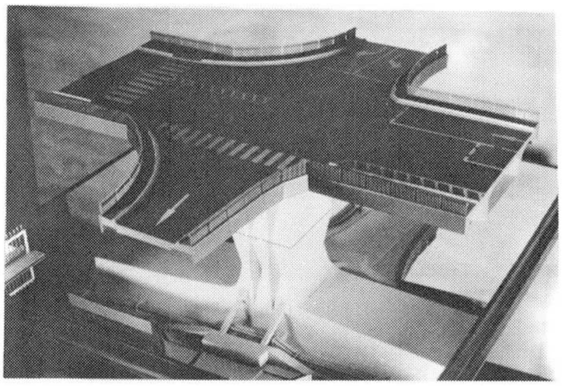

Fig.6

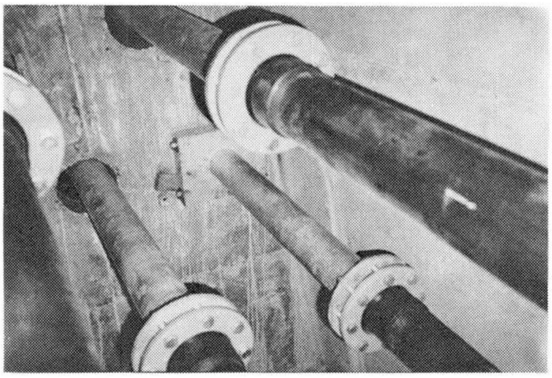

Fig.7

2.4 - **Effect of construction methods on structural durability**

2.4.1 - General

The construction method used for a structure often dictates the structure type and special construction arrangements although certain types of structures are less sturdy than others and certain building measures make the structures more sensitive to aggressive actions. Therefore, it is possible to say that construction methods have a certain influence on the durability of structures.

However, this statement should be moderated as the durability of a construction is also strongly influenced by the quality of the design and by the quality of execution. Consequently, the method for awarding study contracts and assigning works have also a certain influence on the durability of structures.

Therefore one can conclude that certain construction methods may have a negative influence on the durability of a structure, especially if certain precautions and certain constructive arrangements are not taken at the design phase and the execution phase or even management of the latter.

To illustrate my arguments I am going to deal with a few methods for building concrete bridges which make use of concreting in situ and prefabricated concrete.

2.4.2 - Construction on scaffolding and simulated techniques

First example: Structures poured in situ generally have a good behaviour in time except when the execution process leads to "slicing" the structure due to multiple repeated concreting in the longitudinal direction and in the transversal direction.

In such a case, shrinkage and differential creep appear in the form of cracks and in addition, the restart surfaces are weak points through which water and aggressive agents can penetrate.

Second example: construction by successive cantilevers with segments poured in situ on mobile rigs is a delicate technique for the following reasons:

- the multiplying of concreting restarts especially when the segment is concreted in several stages;
- deformation of rigs during concreting can lead to concrete cracking which starts to set especially at the level of the joints between segments;
- tensioning of prestressing reinforcements on fresh concrete can initiate diffusion cracks;
- the effects of redistributing stresses by deferred deformations (effects of creeping) are important due to the young age of the concrete....

2.4.3 - Prefabrication and associated techniques

First example: we refer to reinforced concrete bridges composed of prefabricated contiguous beams linked by spacers poured in situ. The current state of decks, about forty years old, in relation to the very satisfactory state of bearings shows what can be achieved without top slab and waterproofing layer - Figure 8.

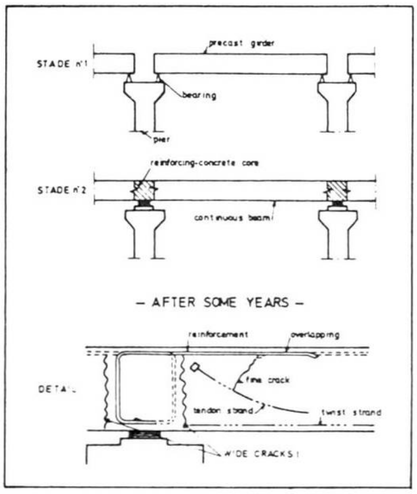

Fig.8

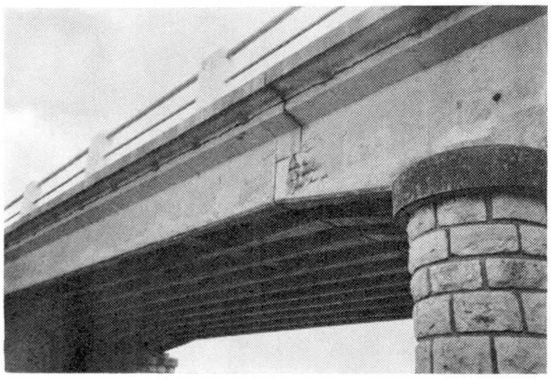

Fig.9

Second example: the assembly of prefabricated jointed elements interspersed with prestressed is a technique well developed which provides satisfaction. If the bonding film is eliminated at the joints, this causes the concrete to crack in line with the hard points through which the stresses pass preferentially. If the waterproofing layer is also eliminated, water and aggressive agents infiltrate by the cracks and cause disaggregation of the material by corrosion of reinforced concrete strengthenings.

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Third example: a certain number of small continuous structures have been built using prefabricated beams in prestressed concreted linked by a reinforced concrete core poured in situ in line with intermediate bearings. Effects of hindered delayed deformations (creepage and shrinkage) were not taken into account in design.

The current operating mode for these bridges leaves much to the imagination - Figure 9.

3 - CONCLUSION

Assessment of durability problems for a specific structure from the design phase through to execution should avoid both the premature appearance of major disorders or even its total damage, and secondly the transformation of the structure into a permanent repair site as is the case for instance for certain cathedrals whereby renewal is a never ending job much to the dismay of the owners.

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

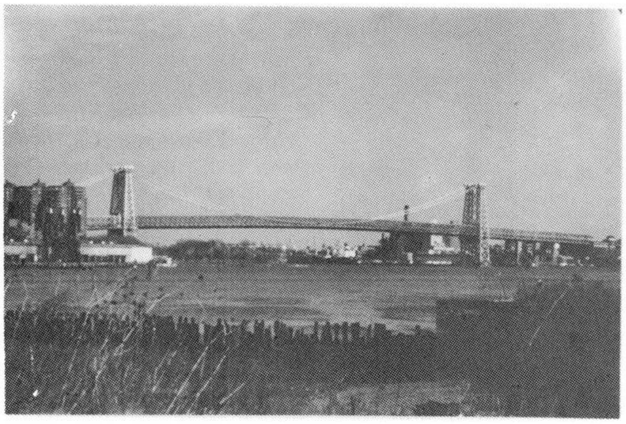

Fig. 1 Williamsburg Bridge

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

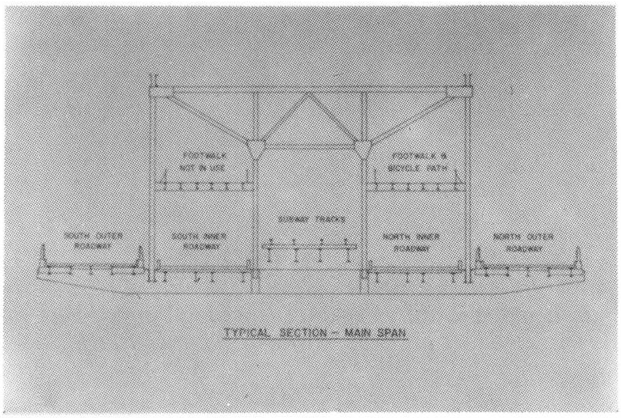

Fig. 2 Bridge Cross Section

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

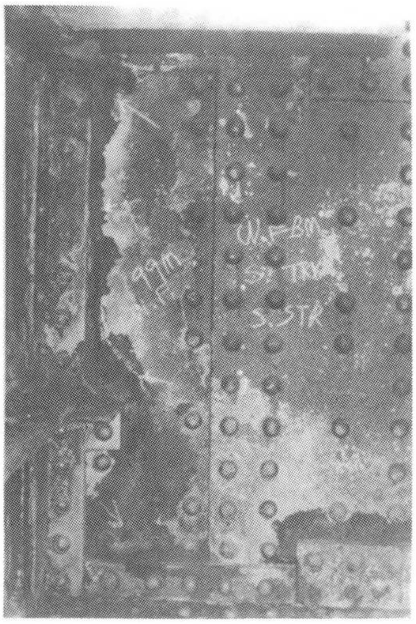

Fig. 3 Web Deterioration

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 in-depth 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

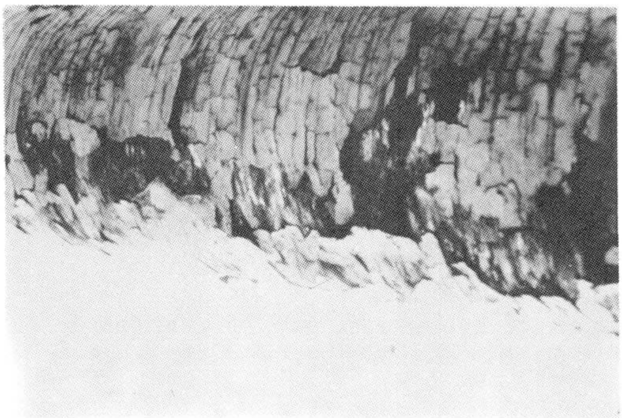

Fig. 4 Cable Paint Condition

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".

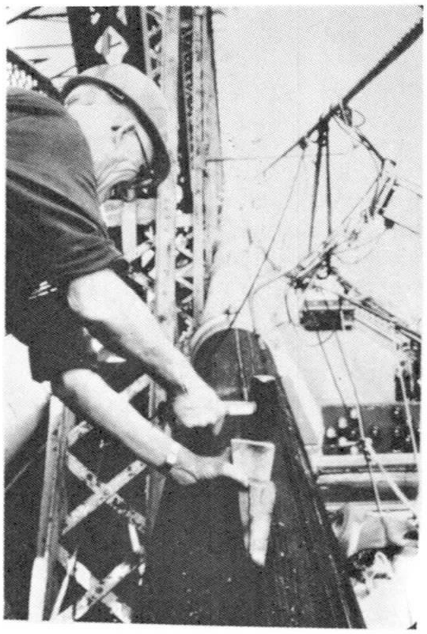

Fig. 5 Wedging the Cable

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.

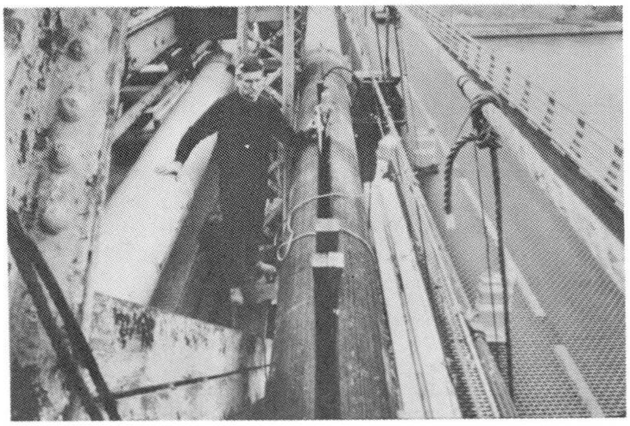

Fig. 6 Hydraulic Jack

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 shcw 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.

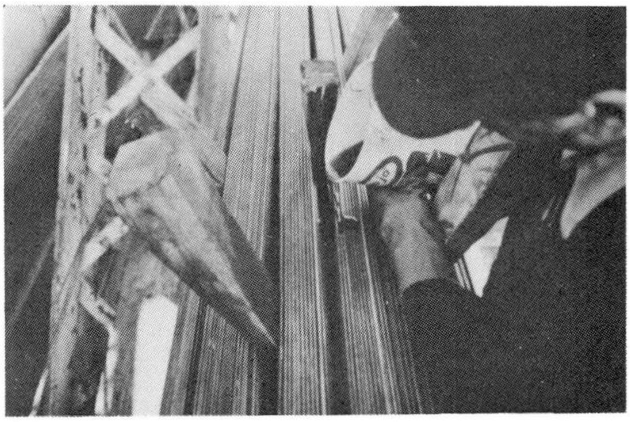

Fig. 7 Applying Protective Oil

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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