Rehabilitation of the Boivre Viaduct — A Multispan Prestressed Box Girder Bridge

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This project confirms the veracity of the old adage that "necessity is the mother of invention." Indeed, during a routine inspection of the Boivre Viaduct, four types of cracks were discovered which prompted an in-depth inspection of the structure in addition to conducting a field test with an actual truck loading.

The field findings, together with a review of the original computations, led engineers to the diagnosis that the bridge was underdesigned and that strengthening through rehabilitation of the structure became an absolute necessity. This paper describes the innovative rehabilitation methods adopted. These included the:

1. Use of stainless steel bars for the transverse and vertical prestressing, requiring a novel type of anchorage.

2. Utilization (in the longitudinal direction) of galvanized strands 965 ft (294 m) long, from one end of the viaduct to the other.

3. Protection of the tendons from corrosion by means of a wax-based material with the development of the related equipment for injection in the ducts.

The bridge was totally rehabilitated and subsequent detailed inspections at

Note: An abbreviated version of this paper was presented by Serge H. Barbaux on June 17, 1985 at the 2nd Annual International Bridge Conference and Exhibition in Pittsburgh, Pennsylvania, which was sponsored by the Engineers' Society of Western Pennsylvania.

regular intervals of time indicated that the adopted corrective measures did successfully rehabilitate this bridge at a reasonable cost.

Synopsis

This paper describes the steps taken to rehabilitate a seven-span, 951 ft (290 m) long, continuous prestressed concrete box girder bridge, located about 180 miles (290 km) southwest of Paris, France. Evaluation of the distressed structure was accomplished through both instrumentation and visual inspections over a 4-year period.

The final corrective measures required the development of new materials and procedures along with a high level of quality assurance controls to ensure a successful project. The completed bridge resulted in a substantial savings of time and money.

GENERAL DESCRIPTION OF STRUCTURE

The Boivre Viaduct is a seven-span, 951 ft (290 m) long, continuous prestressed concrete bridge (Fig. 1), located about 180 miles (290 km) southwest of Paris (Fig. 2). This viaduct is one of the first bridges constructed in France where towers and provisional stays were used for the progressive placement (incremental launching) of the deck elements.

Monolithic precast prestressed (post-tensioned) box girders of constant depth and width (Fig. 3) make up the basic deck elements. The overall width of the deck, including sidewalks, is 44 ft (13.41 m). The constant depth of the box girder along the centerline of the roadway is 8 ft 2½ in. (2.50 m).

The top slab is 10 in. thick (25.4 cm), the bottom slab 8 in. (20.3 cm), and each web 18 in. (45.7 cm). The five interior span lengths are 141 ft (43 m) and the two exterior span lengths are 117 ft 1½ in. (35.80 m), both measured centerline

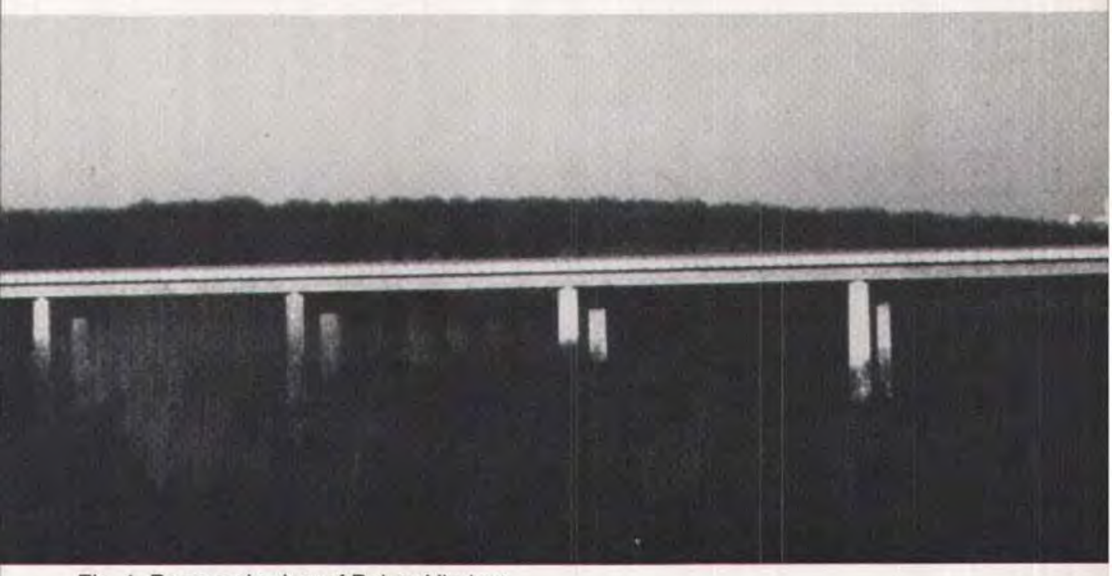

Fig. 1. Panoramic view of Boivre Viaduct.

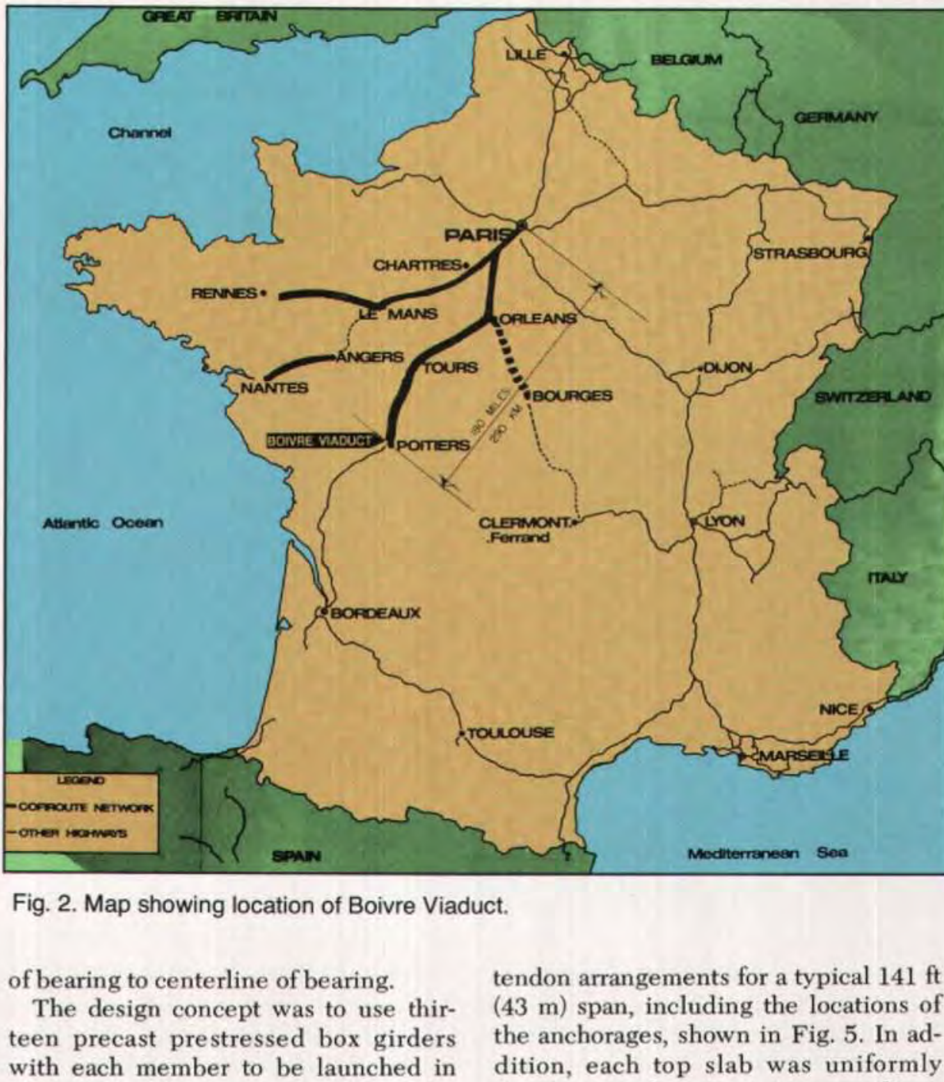

Fig. 2. Map showing location of Boivre Viaduct.

of bearing to centerline of bearing.

The design concept was to use thirteen precast prestressed box girders with each member to be launched in place (Fig. 4). One precast end box element was to be 80 ft 4 in. long (24.5 m), eleven interior elements were to be each 70 ft 6 in. long (21.5 m), and the other end element was to be 86 ft 11 in. long (26.5 m). Once in place, additional longitudinal prestressing tendons over the supports would provide the necessary forces to insure a continuous structure for the entire length of the viaduct.

The top and bottom slabs of each of the box girders were post-tensioned longitudinally with the longitudinal tendon arrangements for a typical 141 ft (43 m) span, including the locations of the anchorages, shown in Fig. 5. In addition, each top slab was uniformly post-tensioned in the transverse direction throughout its entire span length. Webs, however, were only post-tensioned vertically in the vicinity of their pier supports for a distance of 37 ft (11.2 m) away from the centerline and along each side of the piers for a total length of 74 ft (22.50 m) at each pier (Fig. 6).

The Boivre Viaduct was constructed in 1971 for the Vienne subdivision of France's Federal Department of Transportation. Its construction was under the supervision of the Direction Départe-

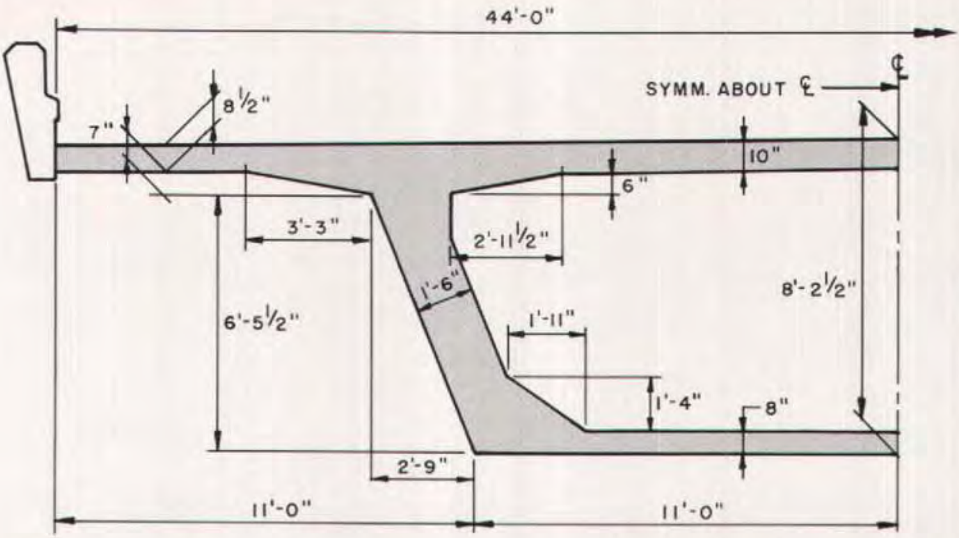

Fig. 3. Cross section of box girder.

mentale de l'Equipment, which is responsible for the preparation of design contract documents, the contractual arrangements with contractors and for the supervision of construction of projects financed by the French Government.

However, at the time of the 1977 opening of the highway section between the cities of Tours and Poitiers, the viaduct was taken over by COFIROUTE,* a private organization specializing in the financing, construction and maintenance management of highways and bridges.

*COFIROUTE is a private organization formed in 1970 by the association of six of France's largest highway contractors. The association is responsible for conceptual design, financing, supervision of the construction by others and the maintenance of the highways and bridges, for which an agreement has been made with the French Federal Government. Tolls are charged by COFIROUTE for such highways and bridges, the ownership of which is transferred to the French Federal Government after 35 years. Currently, COFIROUTE's network under such agreements covers approximately 400 miles (630 km) of highways (including bridges) with 70 additional miles (112 km) still under construction.

Prior to including the viaduct into COFIROUTE's network of projects, a detailed survey was made of the bridge in order to determine its structural integrity.

SURVEYS OF VIADUCT STRUCTURE

The initial survey disclosed that repairs to the bridge had been made as early as 1976 by the Direction Départmentale de l'Equipment de la Vienne. These repairs consisted of (a) injection of epoxy resin in the larger cracks and (b) installation of bridles made up of standard prestressing steel bars similar to the bridles installed later.

The survey also determined that corrosion of the bars under stress which make up the bridles had not been adequately studied creating additional problems, because corrosion did set in.

The repairs made were unsuccessful in that further deterioration was not prevented. The seriousness of the structural

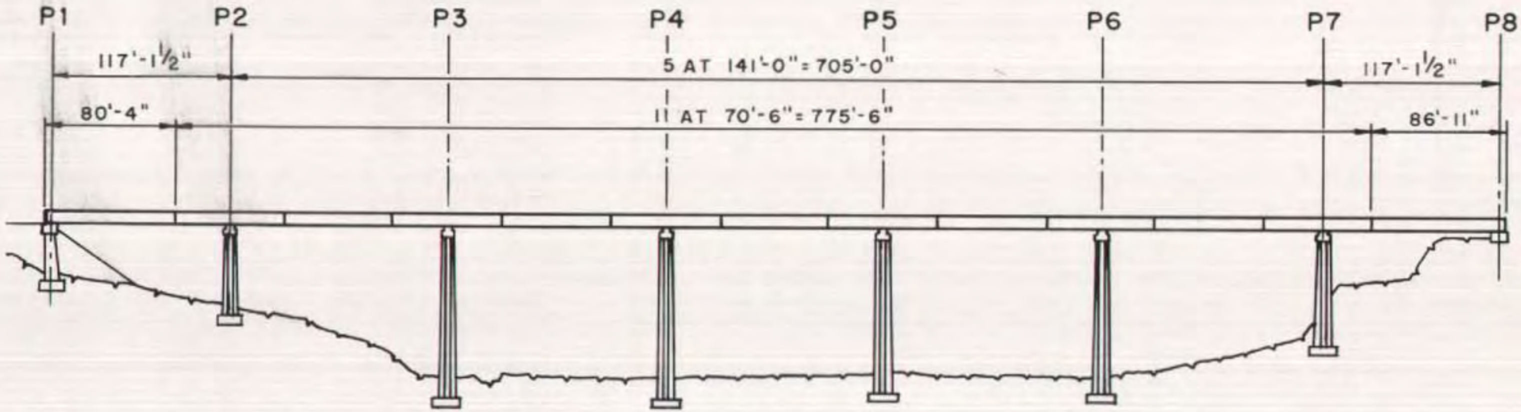

Fig. 4. Elevation of viaduct.

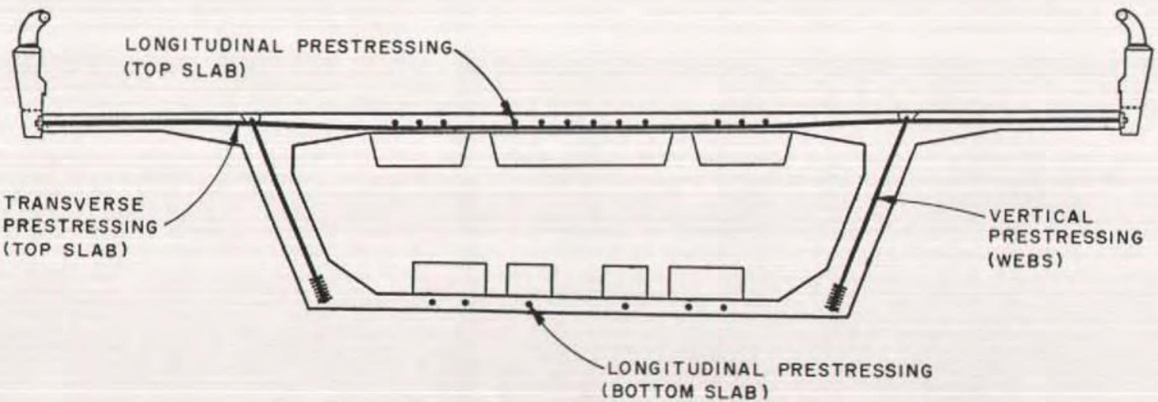

Fig. 5. Tendon locations as provided in original structure.

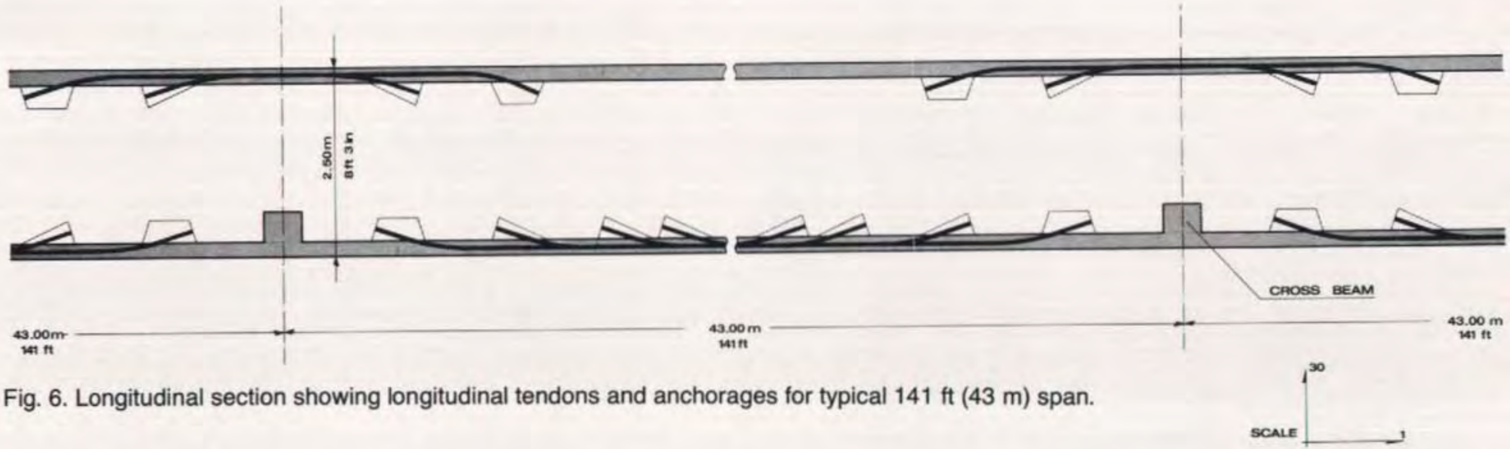

Fig. 6. Longitudinal section showing longitudinal tendons and anchorages for typical 141 ft (43 m) span.

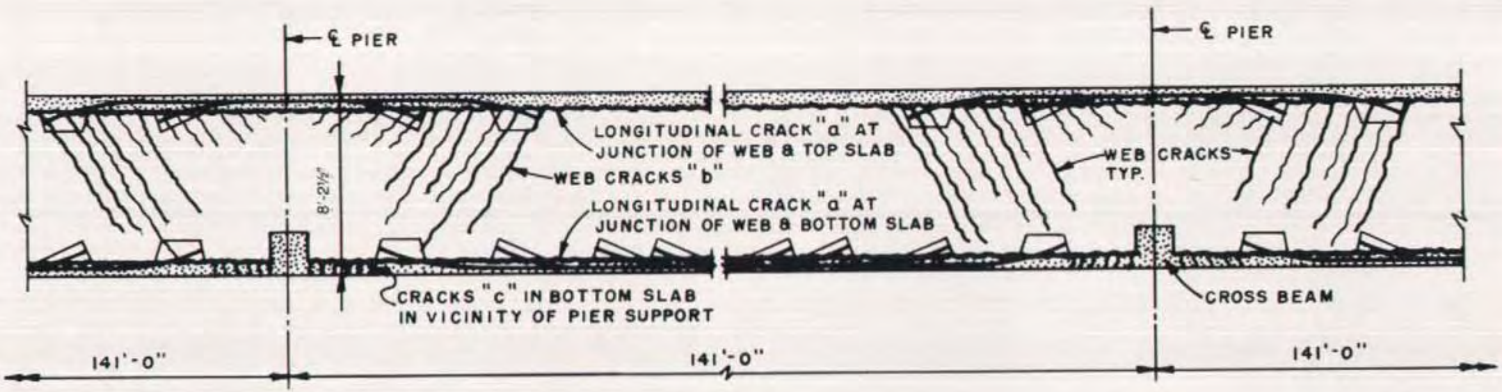

Fig. 7. Longitudinal section through box girder showing the types of cracks together with their location and direction [typical 141 ft (43 m) span].

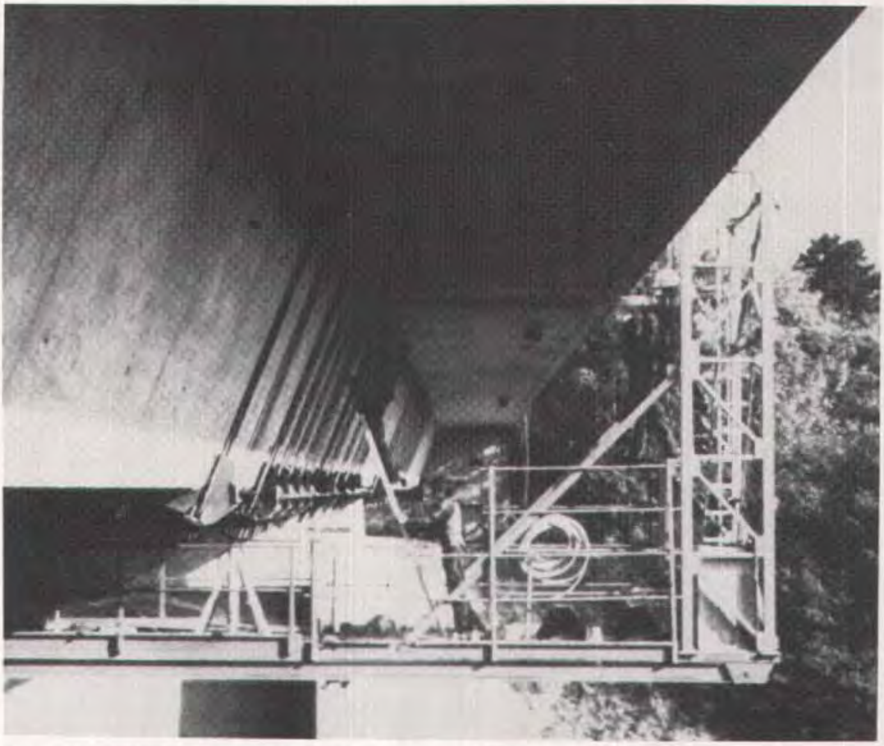

Fig. 8. Inspection of exterior faces of box girders from snoopers.

distress affecting the viaduct structure, however, did not become apparent until a routine visit was made by COFIROUTE in 1977.

This visit was part of the normal bridge survey program COFIROUTE had instituted for the bridges on its network. This survey program calls for an annual routine visit to all bridges and a detailed inspection of the structures every 5 years.

Annual routine visits consist of a visual inspection of the structure, the results of which are compared with previously made visual inspections. It includes measurements of each span made at the supports and at midspan to determine deck edge elevations which then are plotted and compared. The significance of the findings is then evaluated.

The 5-year inspection program is an in-depth examination of all the elements making up the structure. For large structures such as the Boivre Viaduct the use of mobile walkways and snoopers is required (see Fig. 8). Also included in the in-depth inspection are measurements of bridge bearing deformations and joint movements due to temperature variations. Elevations along the bridge spans are also determined.

Field findings are plotted on large drawings with the data supplemented by detailed photographs.

Widths of cracks located in the structure are measured with sensitive instruments having an accuracy of 0.00039 in. ($\frac{1}{100}$ mm) and any variations occurring over a period of time are noted.

The in-depth inspection of a bridge such as the Boivre Viaduct requires the services of three technicians for about 5 days.

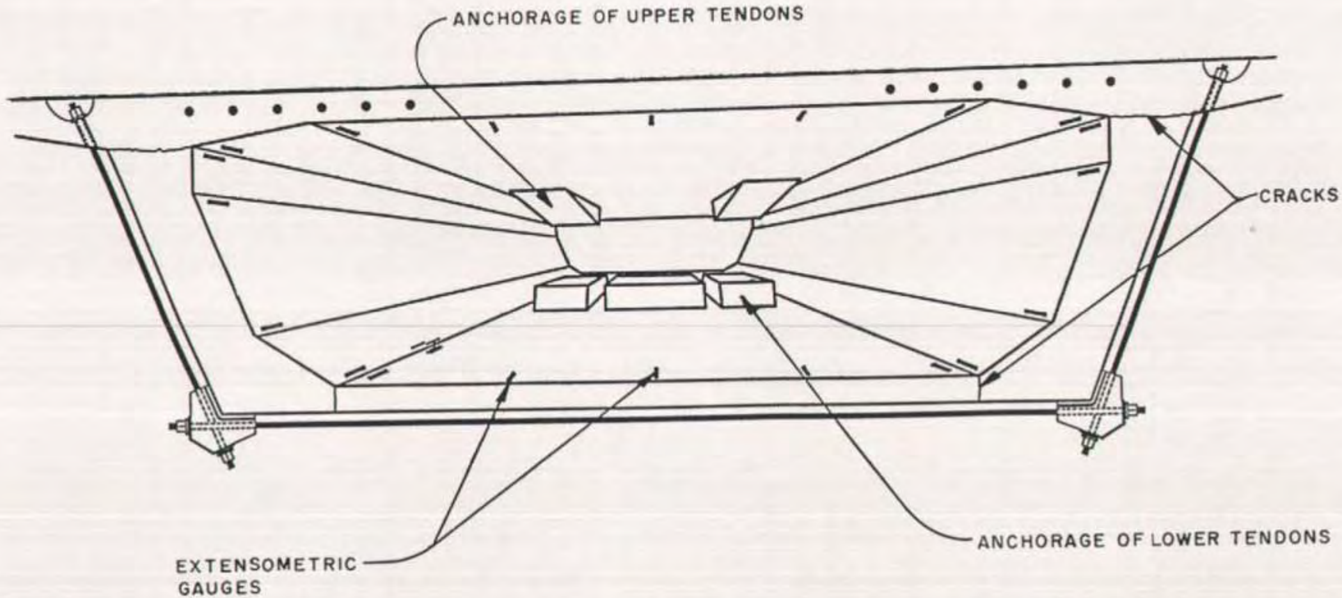

Fig. 9. Location of gauges in one of five box girder sections to measure strains in longitudinal direction.

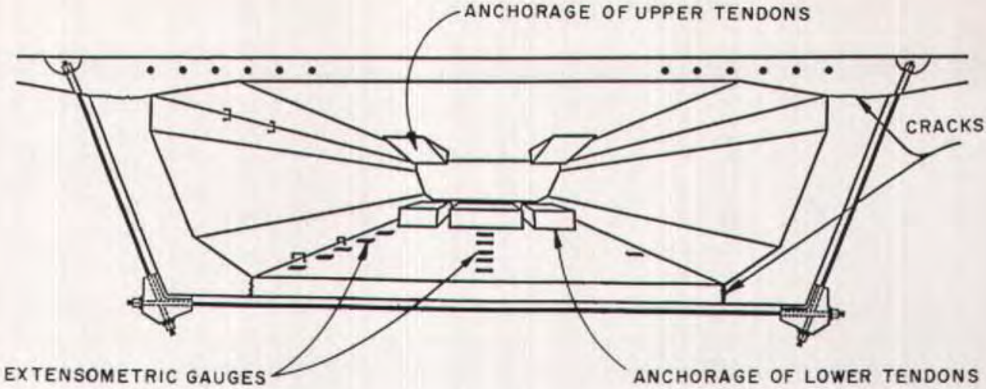

Fig. 10. Location of gauges in box girder to measure strains in transverse direction.

Findings of Annual Routine Survey

The cracking patterns uncovered during the routine inspection of the structure are diagramatically shown in Fig. 7 and can be categorized as follows:

Longitudinal Cracks — These cracks cross the concrete width at the juncture of the webs and the bottom slab, as well as at the juncture of the webs and top slab, almost completely separating the slabs from the webs (Cracks "a").

Inclined Web Cracks — In the webs, inclined cracks were found between the bottom and the top longitudinal tendons in the vicinity of the anchorages (Cracks "b").

Transverse Cracks — Transverse cracks in the bottom slab were found behind the anchorages of the longitudinal tendons (Cracks "c").

Vertical Cracks — Localized, midspan vertical cracks occurred in the webs of the interior spans, beginning at the bottom of the webs for variable heights [30 in. (76.2 cm) maximum height], and could only be detected when thermal response parameters such as solar radiation, ambient temperatures, or wind speed fluctuations were acting on the structure.

The seriousness of discovering such a large number of cracks, some reaching widths up to ¼ in. (6.35 mm), left the engineers no choice but to develop an additional, comprehensive survey program for this specific viaduct. The purpose of this new survey was to obtain a better understanding of the behavior of the structure so that adequate corrective measures could be developed and applied.

Additional Comprehensive Survey Program for Boivre Viaduct

The additional survey program included the following tasks:

1. Detailed Monthly Site Inspections — These inspections included examining the box girders carefully each month and measuring the cracks found. By progressively plotting the cracks, it was possible to determine crack width variations with elapsed time.

2. Detailed Inspection of Exterior Faces of Box Girders — Detailed inspections of the exterior faces of the box girders were made at 6-month intervals from moving scaffolds and snoopers (see Fig. 8).

3. Topographic and Planimetric Control System — A topographic and planimetric control system using 162 points inside the structure and distributed over 27 transverse sections (or six points per section) was developed.

4. Extensometric Gauges — As shown in Figs. 9 and 10, a large number of extensometric gauges were installed in the box girders to measure strains in both the longitudinal and transverse directions. Altogether, a total of 110 longitudinal extensometric gauges were installed in five transverse sections, i.e., 22 gauges per section, as shown in Fig. 9 in order to measure strains in the longitudinal direction. In addition, 77 gauges were installed to measure the transverse strains in seven sections (see Fig. 10).

5. Loading Tests — To determine the actual behavior of the structure under live load, full scale loading tests were conducted. These tests involved placing trucks on the bridge decks.

Laser flexigraphs and inclinometers positioned at the pier tops measured the resulting deformations. By means of strain gauges, the stress distribution between webs and slabs was determined fairly accurately.

Inspection and Loading Tests of Additional Survey Program

Both monthly and 6-monthly bridge inspections described above were made during a 4-year period in order to study and evaluate the long-time behavior of the structure (see Fig. 11).

The major observations and findings were as follows:

1. Inclined Cracks — The size and width of the inclined cracks in the webs. which had remained unchanged until 1979 began to increase. Furthermore, new cracks appeared.

2. New Cracks — New cracks of a different nature began to appear at the junction of the webs and the top slabs.

3. Deflections — The permanent deflection at midspan reached 1½ in. (38 mm), as read from surveying instruments.

4. Truck Loadings — The truck loading tests produced deflections which were 25 percent greater than the computations indicated. However, rotations above the piers were in accordance with the predicted design computations.

5. Measurements — The extensometric measurements at the bottom of the webs showed stress discontinuities which reached 50 percent under the test loadings: For example, tensile stresses were 285 psi (20 kg/cm²) at the bottom of the webs, but were only 142 psi (10 kg/cm²) in the bottom slab, which meant that under load the slabs were sliding with respect to the webs, thus causing horizontal cracking.

Under these conditions the need for corrective measures became imperative. To assist in determining which measures should be taken, COFIROUTE engaged the services of SOGELERG, an experienced and distinguished consulting engineering organization specializing in the preparation of design and construction documents for large civil engineering projects such as bridges, tunnels, nuclear plants and other significant structures.

CAUSE OF CRACKING

Evaluation of the truck load test results and the type of cracks observed led to the obvious conclusion that the longitudinal prestressing originally provided was inefficient, inadequate and/or ineffective. A thorough review of the original design computations, developed by the contractor's engineers* prior to the commencement of construction, confirmed these conclusions:

None of the installed tendons induced in the webs the compressive stresses which were necessary to overcome the tensile stresses produced by the exterior

*European practice is for the contractor's engineer to develop all the design calculations.

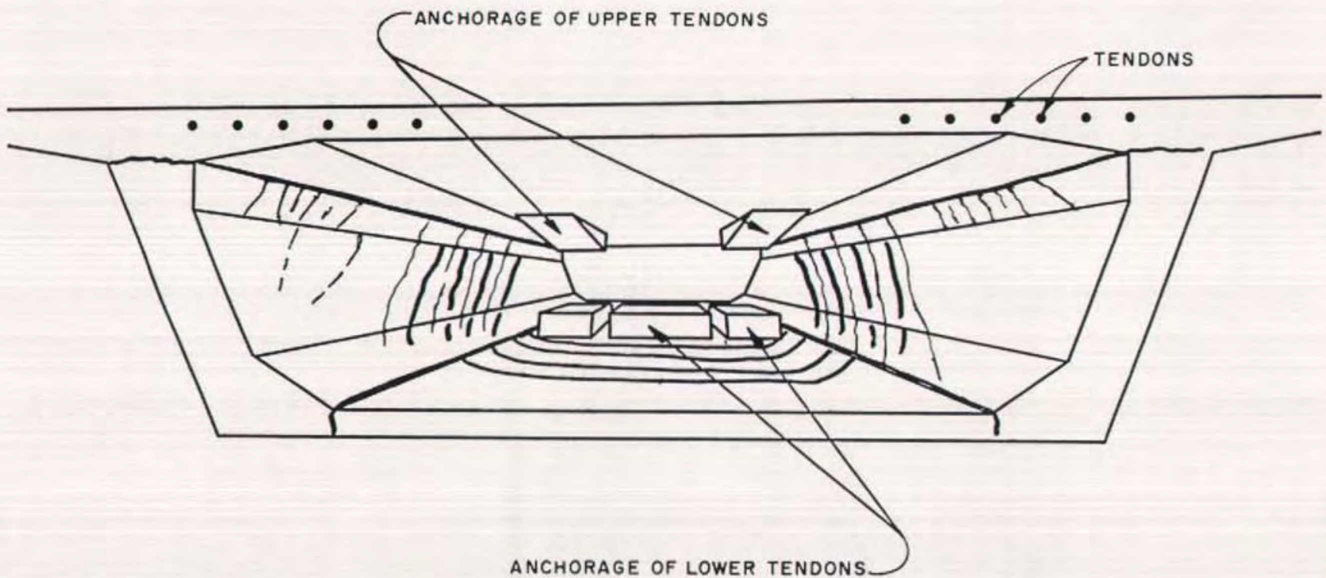

Fig. 11. Evolution of cracks in box girders from 1977 to 1981.

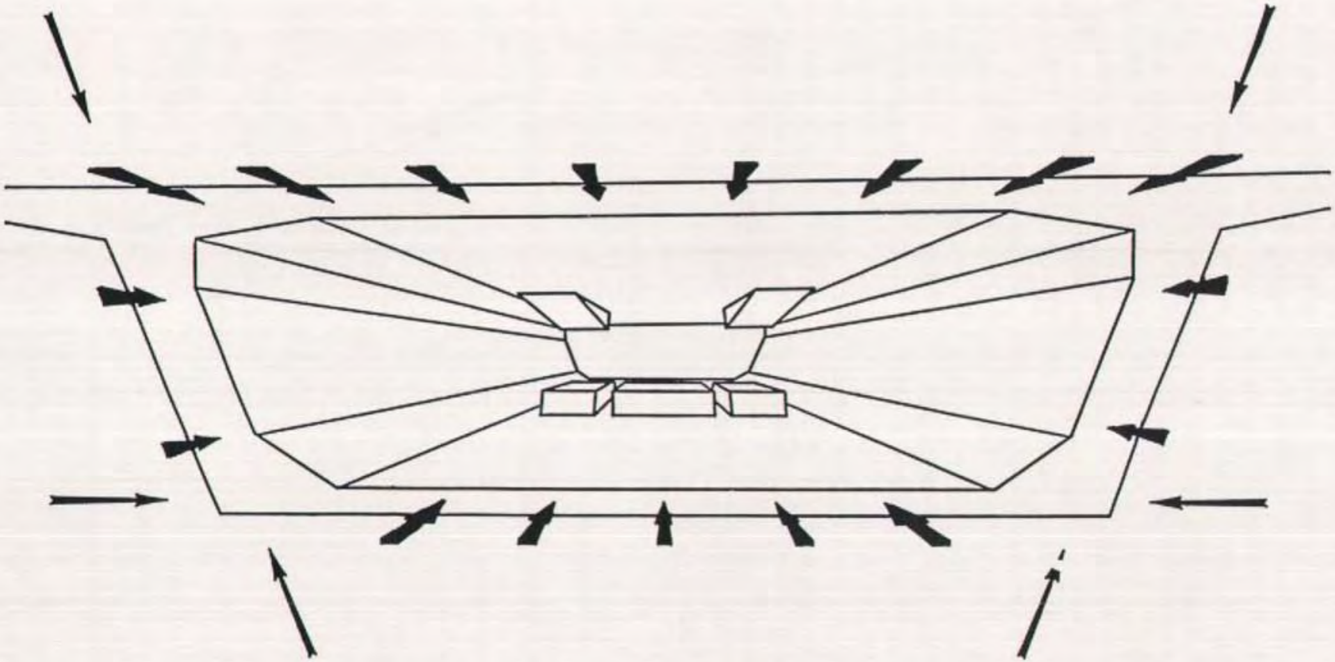

Fig. 12. Basic principles used in strengthening box girders.

loads. This was because all of the 142 ft (43 m) long tendons were straight and had been placed in the slabs. Regrettably, no longitudinal draped tendons of any kind had been called for in the design/analysis. Therefore, the transverse and inclined tendons which were installed in the structure did not provide the required compressive stresses in the webs.

In addition, the longitudinal tendons were anchored in clusters, producing high localized stress concentrations. The total forces produced were 3500 kips (1760 short tons). The design as originally developed, combined with the lack of adequate mild reinforcing steel, made the necessary stress distribution an impossibility.

It also became evident that the cracks had dramatically changed the behavior of the original monolithic structure and that a new design/analysis had to be developed for what was now a "cracked" concrete structure.

Before taking into account the cracking of the concrete, computations showed that the tensile stresses at the bottom of the webs under normal loads and conditions reached 990 psi (70 kg/cm^2). The shearing forces produced stresses of 700 psi (50 kg/cm^2). The determination of the stresses of a structure designed to be monolithic, but which is altered by the presence of numerous cracks, is a very complex problem.

To overcome the difficulties, a computing procedure was established in which a finite element program for stress distribution in the vicinity of the anchorages was used. Taking into account the severe cracking of the structure, new calculations showed that the stressing operations of the tendons should have produced differential movements between the webs and the bottom slab of approximately ⅛ to ¼ in. (3.2 to 6.4 mm).

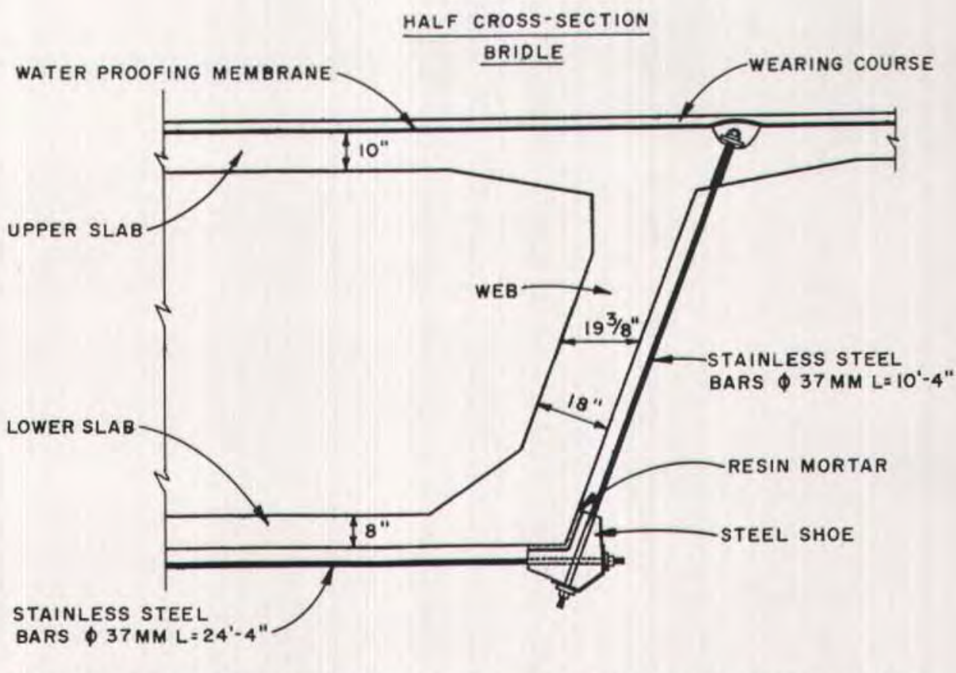

Fig. 13. Vertical and horizontal tendons employed to re-establish integrity of juncture of web and slabs.

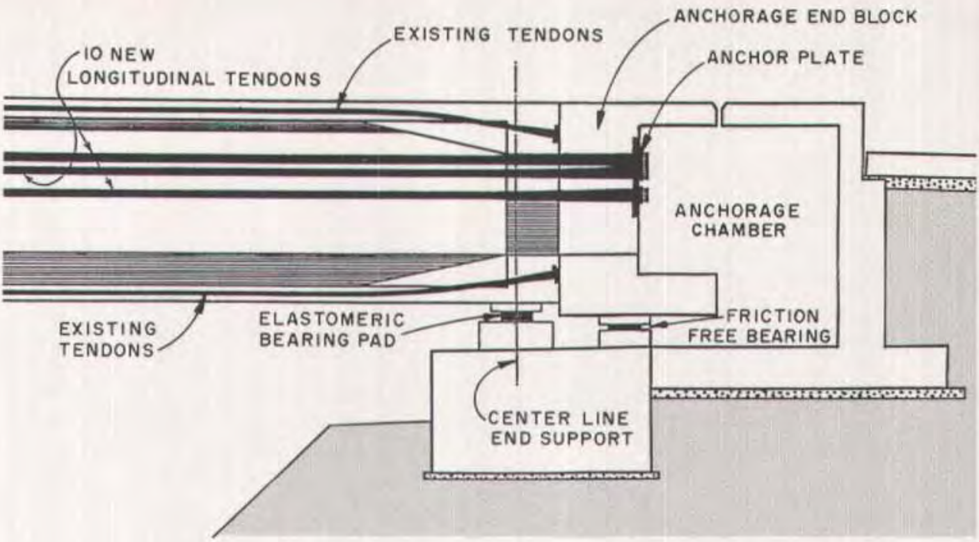

Fig. 14. Longitudinal section at end anchorage block showing new longitudinal tendons.

REQUIREMENTS FOR ADDITIONAL TENDONS

To rehabilitate and strengthen the structure, it was necessary to install new tendons in such a manner that the resulting concrete stresses, produced by the combination of the rehabilitated *existing* tendons and the *new* tendons, would remain within the allowable limits in each and every section of the box girder throughout the entire length of the viaduct (see Fig. 12).

To achieve this goal, it was necessary to first re-establish the strength and structural integrity of the junctures of webs and slabs and then to add, over the entire cross section of the box girder for the entire length of the viaduct, compression forces which would produce compressive stresses of about 570 psi (40 kg/cm^2).

Installing appropriately located vertical, transverse and longitudinal tendons would produce, when stressed, the necessary forces for the required compressive stresses. To re-establish the effectiveness of the juncture of the slabs and webs, and at the same time increase the shearing strength of the webs, new transverse and vertical tendons (see Fig. 13) near the pier supports for a distance of about 42 ft (13 m) would produce the required vertical compressive stresses.

Lastly, the installation of appropriately located longitudinal tendons (see Fig. 14) from one end of the viaduct to the other would produce the additional horizontal compressive stresses desired. The new tendons, when combined with the restored, existing longitudinal tendons in the slabs (the only existing flexural reinforcement so far) would make the structure capable of carrying the superimposed design loads which the originally constructed viaduct could not.

Based on the assumption that the above forces could indeed be furnished in the field and that the required restoration could be carried out, the actual structural strength of the rehabilitated structure was recomputed by the finite element program mentioned previously.

These computations indicated that the combined compressive stresses thus produced would be about 3100 psi (220 kg/cm²) in the high stress concentration areas near the pier supports.

To determine whether the existing concrete in the bridge could safely sustain such high compressive stresses, sonic tests at 4500 points throughout the structure were made. Note that the actual concrete strengths varied from 6260 to 10,700 psi (440 to 750 kg/cm²).

The probability of finding concrete compressive strengths of less than 6400 psi (450 kg/cm²) in areas of high stress concentrations was considered minimal (less than 1 percent).

Required Transverse and Vertical Prestressing Forces

The required transverse and vertical prestressing of the box girders was to be accomplished by means of external transverse tendons along the bottom slab and inclined vertical tendons along the webs as shown in Fig. 13. This formed a type of envelope defined herein as a "bridle."

At the top slab the inclined tendons were to be anchored in a pocket under the wearing course (Fig. 13) while at the bottom a steel shoe arrangement, anchored to the concrete by means of a resin mortar, was to be furnished. The design/analysis indicated that a total of 156 bridles would be required for the entire structure. Each bridle was designed to produce a force of 176 kips (85 short tons).

1. Makeup of Transverse and Vertical Tendons — To produce the necessary 176 kip (85 short tons) force, six 1.45 in. (36.8 mm) diameter bars were required for each of the 156 bridle tendons installed over the entire viaduct. The two bottom transverse bars were 24 ft 4 in. (7.41 m) long and the four shorter inclined bars were 10 ft 4 in. (3.15 m) long (see Fig. 15).

2. Criteria for Maintaining Stresses by Means of External Tendons — The following three basic criteria had to be satisfied:

(a) *Prevention of Corrosion.* Protection against corrosion must be highly efficient and long lasting because, in restoration work particularly, access to many tendon areas, especially at the anchorages, may be difficult. Thus, a reliable means of corrosion protection had to be developed.

(b) *Minimizing of Undesirable Strains.* Since stress can only be induced in tendons by strains which are a function of tendon length, any variation in strain in short tendons, due to seating of the anchorage for example, can seriously affect the magnitude of the stress required to be maintained. Note that percentage wise, for the same amount of strain, the stress losses will be much greater in short tendons than in long tendons. Thus, quality assurance control becomes a very important factor.

(c) *Maintenance of Stress Induced.* To ascertain that the stresses induced in the tendons proper and in their anchorages are maintained, devices for checking stresses at regular intervals had to be developed.

3. Corrosion Protection — Selection of Type of Steel for Tendons — Since corrosion had set in the bars of the bridles installed prior to COFIROUTE's surveys, it was considered imperative that the new exposed bars had to be protected from corrosion, if indeed bars were to be used as tendons. It was necessary to develop bars made of a steel which would be inherently resistant to corrosion while at the same time have the appropriate mechanical properties which are required in prestressing work. Corrosion resistance of such a steel fundamentally results from the formation on its surface of a thin film of inert oxides, which is capable of spontaneous reconstruction in case of damage.

Meeting these criteria was an absolute necessity in the case of the Boivre Viaduct because its lifetime expectancy de-

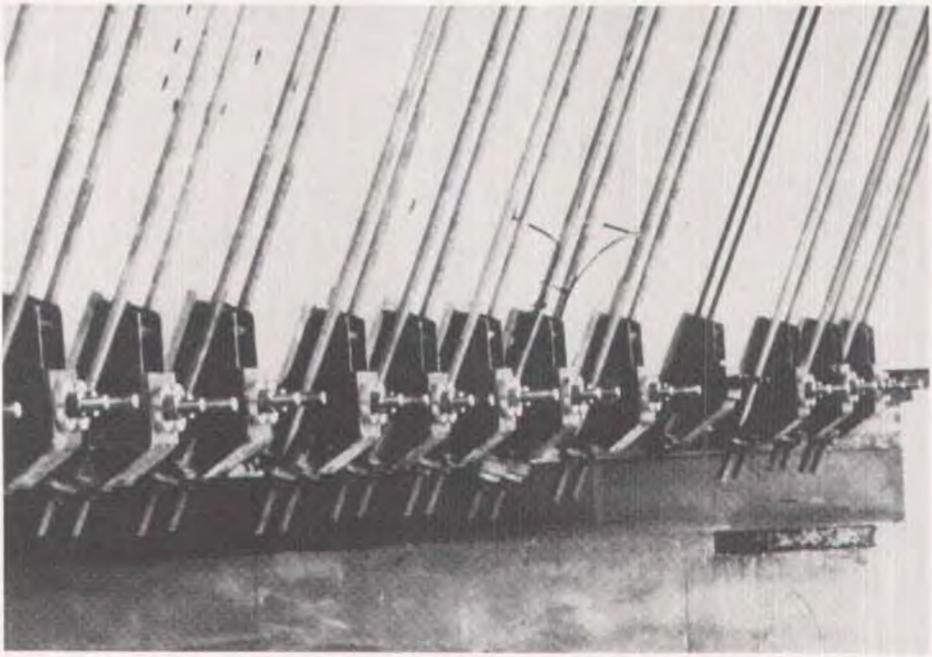

Fig. 15. Steel shoes serving as anchorage for pairs of stainless steel bars.

pended on the accessibility to member parts such as bar threads and anchorages.

Corrosion resistance of stainless steel, for example, is essentially due to the presence of chromium in the steel, the percentage of which must be greater than 12 percent. The magnitude of such resistance is then proportional to the percentage of chromium. The amount of nickel also considerably reduces corrosion as shown in Fig. 16.

In addition, the chosen type of steel must have relatively high mechanical characteristics and be capable of resisting an environment of chlorides, nitrites and sulfates.

An austenitic low carbon steel of high chromium and nickel content with increased mechanical characteristics achieved by nitrogen hardening was eventually selected as meeting the basic criteria.

The chemical composition and the mechanical characteristics of the steel used are given in Tables 1 and 2.

Corrosion tests at 90 percent of the elastic limit were performed at the Public Works Central Laboratory in Paris. The results showed no evidence of corrosion.

The electricity of the steel was found to be 500 millivolts. However, since there was no precedent for using this steel in prestressed concrete work, additional precautions were taken as follows:

(a) No strain hardening was to be allowed. The steel was rough forged and peeled.

(b) The stress at the base of the thread was not to exceed 60 percent of the elastic limit.

4. Anchorages — With no anchorages available in France for stainless steel bars, the Société Francaise pour la Précontrainte (SFP) developed a new anchorage in which:

(a) The threads were obtained by "knurling," i.e., bars are pressed be-

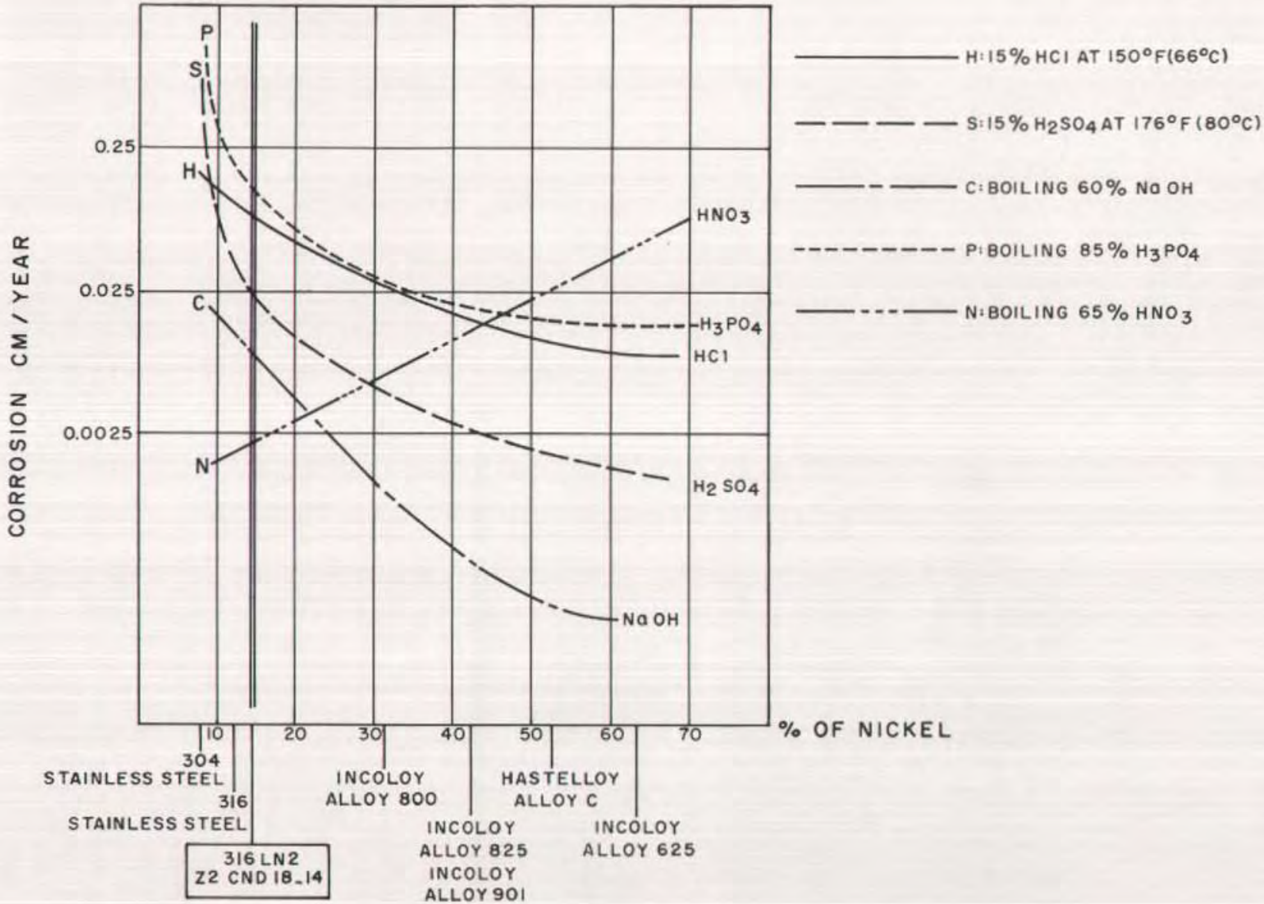

Fig. 16. Corrosion resistance of various steels.

tween rollers which indent the threads.

(b) Stainless steel and carbon steel remained separate.

(c) Nut and washer interfaces are spherical, to minimize undesirable bending stresses. Thus, the anchorage consists of a bar threaded at its ends, a nut with a spherical base, a circular washer with a spherical upper face, an insulated washer, and a nylon washer (see Fig. 17a). A detail of the threads is shown in Fig. 17b.

5. Quality Assurance Control for Fabrication of Bars — Four castings produced about 38.5 short tons of stainless steel bars of the quality required. Four complete corrosion tests, a tensile test on 1.45 in. (37 mm) diameter bar, three series of tensile controls on test specimens and a complete chemical analysis were made for each casting. Each of the nuts, bars and washers were individually checked.

All bars with their accessories were delivered to the construction site in crates to which control certificates were attached.

6. Laboratory Quality Assurance Control of Bars — The laboratory tests were made in the Central Public Works Laboratory. Recording strain gauges, giving directly the stress-strain curves, were attached to each bar together with a mechanical extension gauge and Gloetzl cell, which is a calibrated hydraulic flat jack.

During the tests the required stress-strain procedures were defined and the steel's mechanical characteristics were verified.

7. Installation and Stressing of Transverse and Inclined Tendons

(a) Maintenance of Traffic. Since the Boivre Viaduct could not be closed to vehicular traffic, the installation and stressing of the transverse and inclined tendons was difficult and therefore schemes for maintaining traffic had to be devised.

The requirement of maintaining three lanes of traffic open at all times could only be met after the completion of a second bridge adjacent to the Boivre Viaduct. Therefore, any rehabilitation of the viaduct could not commence until 1976.

Two of the three lanes required to maintain traffic became available on the adjacent new structure.

The third lane (on the original viaduct) was made available through the adoption of a three-stage rehabilitation procedure:

During the first stage, the center lane remained open to traffic. This permitted all construction related to the installation and stressing of the transverse and inclined tendons to be carried out.

During the second and third stages, traffic was permitted on either one of the outside lanes, thus allowing construc-

Table 1. Chemical composition of stainless steel bars used in this project.

Element	Percent	Element	Percent
Chromium	18.0	Silica	1.0
Nickel	14.0	Sulfur	0.036
Molybdenum	3.0	Phosphorus	0.030
Manganese	2.0	Nitrogen	0.20
Carbon	0.05	Iron	62.0

Table 2. Mechanical characteristics of stainless steel bars used in this project.

Ultimate stress: 142,200 psi (100 kg/cm^2)
Elastic limit: 120,900 psi (85 kg/cm^2)
Minimum elongation (necking free): 6 percent
Minimum elongation at rupture: 24 percent
Resilience at low temperature: 120 j/cm^2
Stress relaxation at 90 percent of elastic limit: 2.4 percent
Modulus of elasticity: 25,600,000 psi (18000 kg/cm^2)

Note: The above mechanical characteristics could be considerably increased by strain hardening.

tion of the required new anchor blocks for the new longitudinal tendons.

(b) Construction of Bridles. Since construction of the bridles required simultaneous access to both the top and bottom of the structure, two walkways were required.

The most difficult operation was the boring of the holes through the top slabs since contact between the concrete and the tendons had to be avoided. This required a high degree of precision. The boreholes were drilled with diamond core drills.

Fig. 17a. Bar anchorage developed in France for stainless steel of the type used in the rehabilitation of the viaduct.

(c) Stressing of Tendons. Six hydraulic jacks fed by six synchronized pumps were used to stress the tendons. Stressing was to be executed in two steps separated by an interval of time to allow total relaxation, i.e., to allow compensation for initial creep losses. It was indeed determined that the initial tensile losses would not exceed 3 percent of the initial stresses induced in the bars.

(d) Field Quality Assurance Control

(1) Boreholes

An independent surveyor checked the accuracy of the size and location of the boreholes.

(2) Stressing of Tendons

A detailed quality assurance control program for the stressing of the tendons was developed. It included the following elements:

a. Mechanical strain gauges were used to check the accuracy of the stressing procedures on 5 percent of the total number of ten-

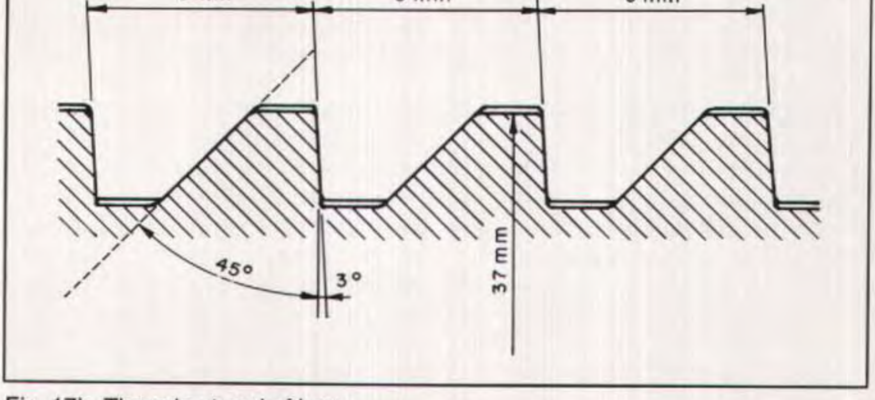

Fig. 17b. Threads at end of bars.

dons, which represented 10 percent of the entire bridle tendons.

b. In each span one bridle tendon was chosen at random to be subjected to the following checks:
   - Elongations by means of mechanical strain gauges placed on the bars making up the tendon.
   - Four displacement gauges to be set on the cracks.
   - A series of 11 strain gauges to be placed on the concrete. In addition, on one bridle in each span, a series of Gloetzl cells were installed.

To further insure accuracy in the stressing operation, the elongations of the bars were isolated from all other possible related movements, such as settlement of the top slab and compression of the anchorage components.

Finally, in order to observe the long-term behavior of the gauges and Gloetzl cells, these devices were installed to remain in place permanently.

In general, it was found that the control values attained in the field agreed fairly well with the predicted values obtained from the design calculations.

Longitudinal Prestressing

1. Composition and Number of Tendons — To withstand the normal load conditions, a compressive stress of approximately 600 psi (42 kg/cm^2) had to be produced through additional longitudinal prestressing.

This stress corresponds in this case to a force of 6600 kips (3300 short tons) over the entire cross section of the box girders. Ten tendons, each ±965 ft (±294 m) long and each made up of seventeen ⅝ in. (15.7 mm) diameter galvanized strands totaling 3.95 sq in. (2550 mm^2), were required to produce this prestressing force.

2. End Anchor Blocks — Computations determined that the existing end cross beams could not carry the 6600 kip (3300 short tons) force. Therefore, new reinforced concrete end anchorage blocks had to be designed and constructed so that the forces could result in a uniform stress distribution. The existing anchorages of the longitudinal tendons made the need for uniformly distributed stresses of the new tendons imperative to avoid major problems.

The presence of the bridles dictated the location of the longitudinal tendons and left the designer with no choice but to locate these inside of the box girders. To obtain the desired uniform compressive stress distribution in each section throughout the entire length of the structure, the resultant of the prestressing forces produced by these additional tendons had to be located as closely as possible to the center of gravity (neutral axis) of the cross section of the box girders.

The problem is a relatively simple one in the case of a noncracked beam but more complex for a cracked beam. To obtain a uniform stress distribution, the anchorage blocks were designed with stiffeners.

The design called for the construction of 3 ft 3 in. (1 m) thick reinforced concrete end blocks to be stiffened by three 6 ft 6 in. (2 m) thick beams and reinforced with 500 lb of mild reinforcing steel per cubic yard (about 300 kg/m^3). For details refer back to Fig. 14. The ten anchorages bear on two 4 ft x 3 ft 3 in. (120 x 1 m) steel plates (see Fig. 18).

3. Installation of Longitudinal Tendons and Their Protection From Corrosion — Metal duct supports were used to hold in place the 4½ in. (114 mm) diameter reinforced glass fiber duct which enclosed the galvanized steel strands. The supports were bolted to reinforced concrete blocks spaced on 13 ft (4 m) centers in order to prevent possible vibrating resonance problems between the tendons and the structure (see Fig. 19).

Although the strands were galvanized

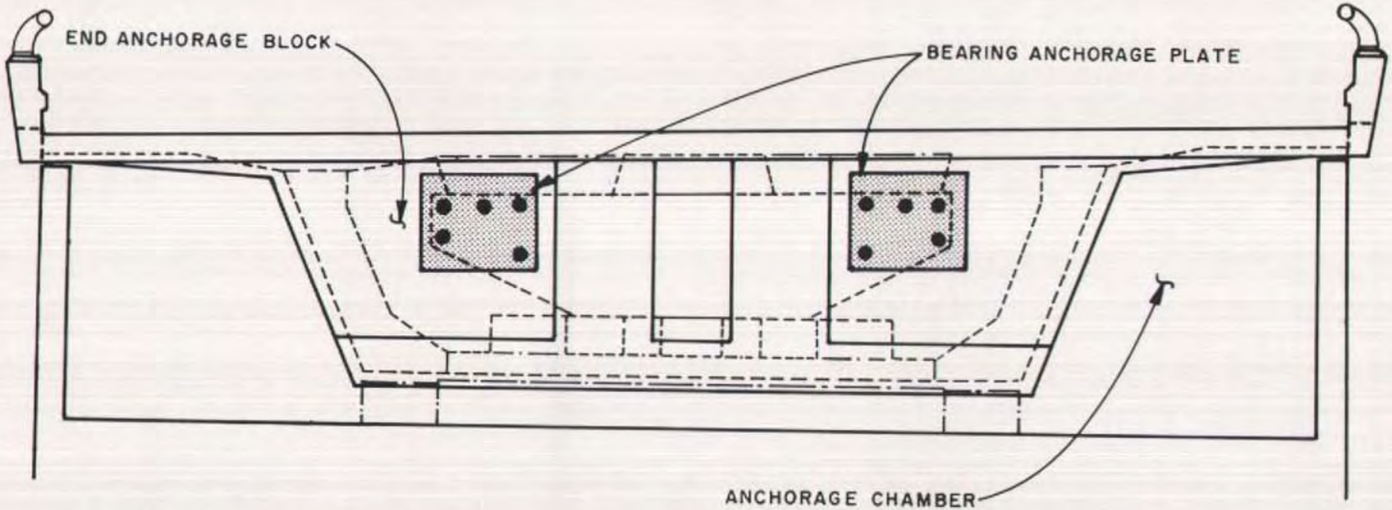

Fig. 18. End view of end anchorage block showing location of tendon bearing plates.

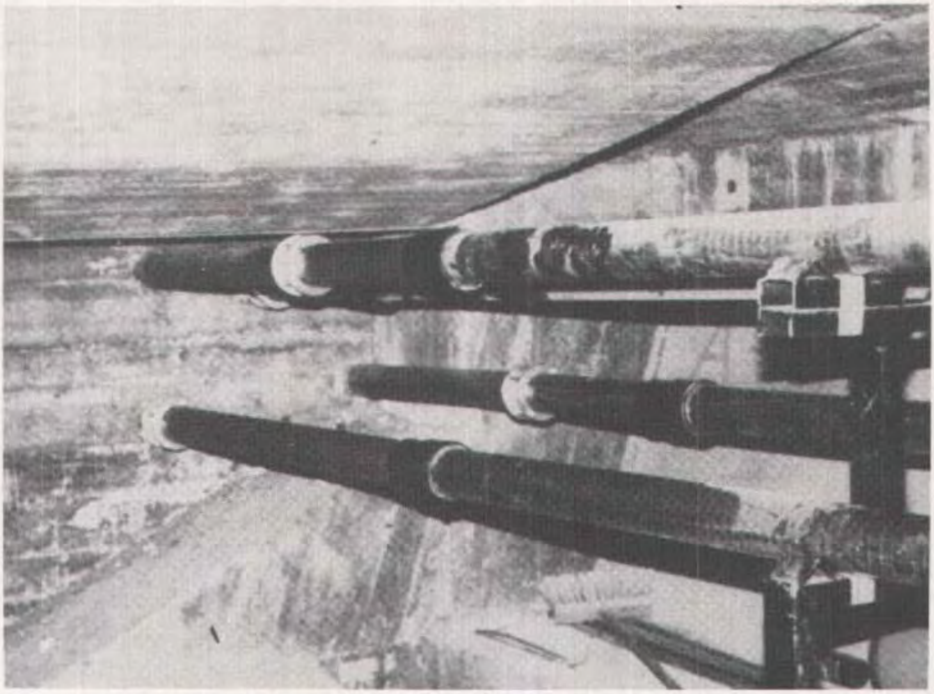
Fig. 19. Reinforced glass fiber duct inside box girder.


Fig. 20. Wax-based product as developed by the ELF Corporation for corrosion protection of stainless steel bars.

Fig. 21. Laboratory test for corrosion of waxed tendon in duct.

and the ducts were theoretically unnecessary, it seemed prudent to leave the ducts permanently in the structure. This was not only to insure against corrosion but also to protect the tendons from shock since stressed tendons are highly susceptible to failure due to shock.

For the following specific reasons, it was decided to nevertheless protect the tendons from corrosion:

1. At the time of rehabilitation, sufficient experience had not been obtained regarding the protection or longevity of galvanized strand tendons stressed beyond 171,000 psi (120 kg/cm²). Also, sufficient data had not been obtained about the toughness of galvanization in friction. Provision against unexpected accidents which could occur during the placing of the tendons in the ducts and anchorages had to be made.

2. Cement grout (the material usually used for corrosion protection) has been found to be unsatisfactory in many cases. Therefore, the Public Works Central Laboratory undertook the investigation of other grouting materials. Most of these materials were also found to be unsatisfactory.

To insure complete corrosion protection, it was speculated that a basic soft product would be suitable. Several oil companies were approached which proposed the use of their products. These were all greases with a variety of emulsifiers such as calcium lithium hydrostearatic, aluminum complexes and other chemicals. Unfortunately, none of these products met completely the required specifications.

Eventually, based on further investigations, the ELF Corporation (the largest oil company in France) proposed the use of a wax-based product (see Fig. 20). Some of the advantages of wax-based products are as follows:

1. Absence of sweating.
2. Good penetration into all the interstices of the strands.
3. Good adhesion to the base material (wax-based products are highly sticking).
4. Good reversibility due to crystalline structure.
5. Hydrophobic nature of wax-based products.

The feasibility and practicality of infusing a wax-based product in a duct under site conditions remained to be investigated. Therefore, tests at temperatures of 54°F (12°C) were conducted and the material was infused in an 82 ft (25 m) duct. Under a pressure of less than 14.2 psi (1 kg/cm²), it was possible to fill the duct very easily within 3 minutes (see Fig. 21).

After the wax-based product had cooled, additional tests showed complete penetration of the material between the wires of the strands. Subsequently, a job site procedure, based on the use of specially equipped trucks, was developed to meet field site conditions (see Fig. 22).

As a result of the successful laboratory tests and the ease of delivering the material to the job site for immediate filling of the ducts, the wax-based product was used enthusiastically. Since the viaduct application, the wax-based product together with the application techniques, have been patented.

Major Construction Features

1. Tolerances — The installation of the longitudinal prestressing tendons presented major difficulties because of low permissible tolerances:

(a) Anchorage plates had to be placed with a dimensional variation in each direction not to exceed 0.04 in. (1 mm) and with a maximum angular tolerance of no more than 1 degree.

(b) Elevation differentials between two adjacent supports were not to exceed 0.04 in. (1 mm).

2. Anchorage Blocks — The construction of the anchorage blocks and also the anchorage chamber required the demolition of the abutment retaining walls.

The reinforcing steel cages for the anchorage blocks were prefabricated and positioned by cranes.

Tendons in the anchor blocks were placed in steel pipes up to the steel anchorage plates where the pipes were connected to reinforced resin pipes by flange points.

3. Thermal Variations — To control expansion due to thermal variation, expansion compensators were attached to the ducts at midlength of the viaduct.

4. Stressing — Each tendon was stressed in three steps of 220 kips (100

Fig. 22. Specially equipped truck for waxing tendons.

tons) each, at intervals of 1 month between each step.

Three stages of 2200 kips (1000 tons) each for the ten tendons were required. Stressing demanded the use of two 500 ton (454 t) capacity hydraulic jacks. An elongation of 4 ft (1.2 m) was required to attain the total desired prestressing force per tendon.

5. Protection Against Corrosion — Immediately after completion of the stressing operation, the wax-based material was introduced under low pressure at the lowest point of the tendons. Note that the previously mentioned specially fitted trucks were used for this operation.

For each 965 ft (294 m) tendon, the anti-corrosion injection was completed in 40 minutes, including preparatory work.

To protect the protruding 4 ft (1.2 m) length of tendon so that it could serve as a means of restressing should the need arise at some later date, steel covers were used to seal the ends of the tendons.

Supervision of Construction

The supervision of construction included the following items of work:

1. Checking the characteristics of the material to insure that specifications were met.

2. Recording by means of strain gauges that the stressing of each pair of tendons was carried out accurately.

3. Measuring the size of main cracks after each 2200 kip (1000 tons) stage.

4. Measurement of force transmission ratios were made on the first two tendons even though the tendons were straight and the duct surfaces unusually smooth. The measured ratios ranged from 98 to 99 percent.

5. Checking jack pressures against elongations during the entire stressing of the 965 ft (294 m) long tendons, with the total elongation to be approximately 4 ft (1.2 m).

It was determined that the field work was carried out in accordance with the design computations.

EFFICIENCY OF THE REHABILITATION

To assess the efficiency of the rehabilitation work performed on the Boivre Viaduct, the structural and thermal behavior of the bridge were evaluated together with a review of the completion dates of the project.

Structural Behavior

To determine whether the rehabilitation measures described herein achieved their purpose, all the elements of the initial bridge inspected were reviewed.

The easiest, simplest and most rapid way was to reload the structure with the trucks as initially carried out and to record the stresses produced in the rehabilitated structure. This was done by connecting all the previously installed extensometric and strain gauges placed on the cracks to a computer with two scanners.

The truck loading gave excellent positive results:

1. The deformations, which were 25 percent larger than those indicated by the computations made prior to the repair, were now 15 to 20 percent less.

2. Rotations remained slightly less than those computed.

3. The transmission of forces from one span to the adjacent span became normal again.

4. Stress diagrams in the box girder section became linear again.

Thermal Behavior

Changes in stress due to temperature variations were recorded during a continuous 24-hour period. From these observations a temperature-stress relationship was calculated and a small computer program was set up. Note that thermal variations had to be distinguished from the effects due to loads.

Completion of Rehabilitation Work

The Boivre Viaduct was never closed to traffic. Reconstruction began in September 1982 and was completed in April 1984 even though no construction was permitted between April and September 1983 in order to accommodate summer travel.

CONTINUING OBSERVATION OF VIADUCT'S BEHAVIOR

In addition to the use of the checking devices which were left in the structure, additional detailed inspections were planned at 6-month intervals.

During the first such inspection, made in September 1984, the tensile stresses in the bridle bars were checked. Normally, such checking is made by means of hydraulic jacks. However, because of the large number of bridles, a new tensile check method based on vibration frequency of the bars was developed.

Prior to attaching the vibratory gauges to the bars in order to measure the tensile stresses in the bars, the gauges were calibrated with high precision. Then the bars were made to vibrate by means of a sledge hammer.

Measurements of the cracks indicated a substantial reduction in their widths. Several cracks which were 0.04 in. (1 mm) in size before the repairs measured now 0.004 to 0.008 in. (0.1 to 0.2 mm), and no new cracks appeared.

Strain gauges between the webs and slabs indicated that no movements of any kind such as sliding was taking place.

After making due allowance for temperature differential between stainless steel and concrete, the actual stress variations were found to be close to the values anticipated by the design computations.

CONCLUDING REMARKS

The normal behavior of the rehabilitated bridge at present, and as anticipated, justified the total cost of the repair work which was of the order of 15 million French Francs, i.e., about \$1,800,000,* including all truck loading tests and engineering studies. The cost of a new viaduct, excluding demolition costs, would have been 27 million French Francs or \$3,300,000.

One side benefit of this project is that the rehabilitation effort stimulated the development of new techniques and materials for this type of construction which resulted in reduced costs and greater efficiency. For example, strain hardened stainless steel can now be used at higher stresses and provide resistance to corrosion.

The wax-based corrosion protective material has been used effectively in three other major bridges near the town of Nick, and after numerous additional tests has performed to the satisfaction of all parties concerned.

*Based on 8.2 French Francs per U.S. dollar.

NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by January 1, 1987.