# Strengthening of a Long Span Prestressed Segmental Box Girder Bridge

**Bruno Massicotte**
**Ph.D., P.Eng.**
Associate Professor
Department of Civil Engineering
École Polytechnique de Montréal
Montréal, Québec, Canada
Formerly, Bridge Design Engineer at the Québec Ministry of Transportation

**André Picard**
**Ph.D., P.Eng.**
Professor
Department of Civil Engineering
Université Laval
Québec City, Québec, Canada

**Yvon Gaumond, P.Eng.**
Bridge Design Chief Engineer
Bridge Department
Québec Ministry of Transportation
Québec City, Québec, Canada

**Claude Ouellet, P.Eng.**
Senior Bridge Engineer
Bridge Department
Québec Ministry of Transportation
Québec City, Québec, Canada

*The Grand-Mère Bridge in the province of Québec, Canada, is a 285 m (935 ft) long, cast-in-place, segmental box girder bridge that experienced several problems which resulted in distress characterized by an increasing deflection combined with localized cracking. These defects were due mainly to insufficient prestressing causing high tensile stresses in the deck and possible corrosion of the prestressing steel. To remedy this situation, the Québec Ministry of Transportation strengthened the bridge by adding external prestressing equivalent to 30 percent of the remaining internal prestressing. The paper describes the causes of the distress and focuses on the assumptions adopted in the analyses to determine the current state of the bridge. The technique and design criteria used in strengthening the Grand-Mère Bridge are described. Also, the construction aspects and the various problems met during the external prestressing operation are discussed. The new technology and experience gained in strengthening this structure can be applied to both pretensioned and post-tensioned concrete bridges.*

The Grand-Mère Bridge, a 285 m (935 ft) long, cast-in-place, post-tensioned segmental structure built in 1977 (Fig. 1), consists of a single-cell box girder. It is located 200 km (125 miles) northeast of Montréal on Highway 55, where it crosses the St. Maurice River near the town of Grand-Mère.

An article on the design and construction of the Grand-Mère Bridge appeared in the January-February 1979 PCI JOURNAL[1] and the project won a Special Jury Award in the 1978 PCI Awards Program. This long and slender bridge is an elegant structure which blends in well with the surrounding landscape.

Unfortunately, this bridge experienced various problems and distress during and after its construction, resulting in localized cracking and increasing deflection in the 181.4 m (595 ft) central span. These problems were due mainly to insufficient prestressing as a result of construction problems, optimistic design assumptions, and a limited state of knowledge at that time, especially regarding thermal stresses.

Numerous studies showed that the short-term bridge safety was adequate. However, because of the potential risk of cracking in the deck, the studies also indicated that the long-term integrity could be affected if corrective measures were not taken immediately.

Based on these technical evaluations and considering the structure's importance, the owner, the Québec Ministry of Transportation (QMT), decided to strengthen the bridge. Additional longitudinal prestressing corresponding to 30 percent of the remaining amount corrected the lack of sufficient prestressing. The strengthening design, in which the authors of this paper played an active role, was done by the QMT Bridge Department.

Construction began in May 1991 and the additional prestressing was finally applied to the bridge in November of the same year. The technology used in the strengthening of the bridge can be applied to all prestressed concrete structures, whether pretensioned or post-tensioned.

Fig. 1. The Grand-Mère Bridge.

## SCOPE OF THE PAPER

This paper is the first of two companion papers describing the corrective measures applied to the Grand-Mère Bridge. The scope of this first paper is to present the history of the bridge, describe the various problems met during construction, and discuss the subsequent distresses, their causes and their remedies. It presents the assumptions used in the analyses to establish the bridge state and later to design the strengthening program. The use of individually lubricated strands and details of the cable anchorage system to the existing structure are described. The construction aspects and problems faced during the strengthening process are also discussed. The paper deals with practical considerations and is addressed to bridge engineers who may face similar problems.

The companion paper,[2] on the other hand, presents the details of an important monitoring program carried out on the bridge during the strengthening process. This program studied the various aspects of the bridge's behavior before, during and after the strengthening operation.

## BRIDGE DESCRIPTION

The Grand-Mère Bridge, a 285 m (935 ft) long structure, consists of three continuous spans of 39.6, 181.4 and 39.6 m (130, 595 and 130 ft), completed with a wedge-shaped, solid cantilever of 12.2 m (40 ft) at each end acting as counterweights (Fig. 2). The length of the central span was a remarkable achievement in 1977, and is still among the longest for this kind of bridge in North America. The bridge is an elegant, slender structure spanning the St. Maurice River.

In the central span, the depth of the box girder varies parabolically from 9.75 m (32 ft) at the piers to 2.90 m

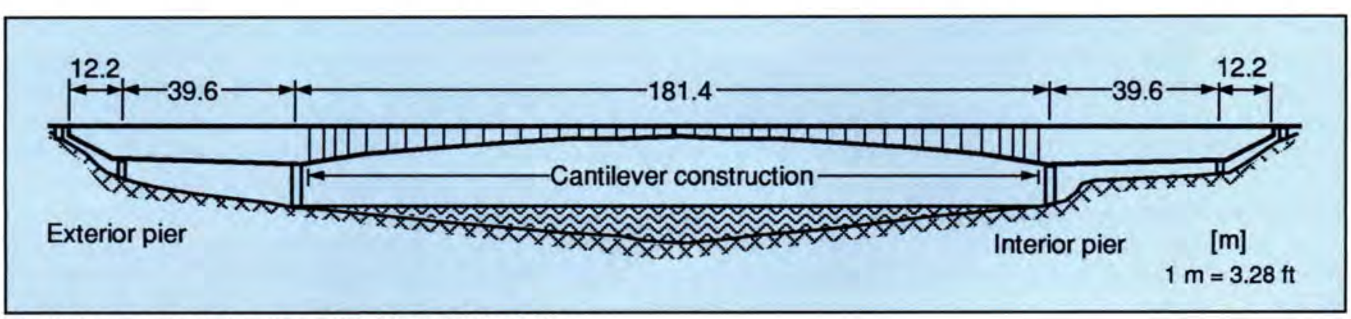


Fig. 2. Longitudinal elevation of the Grand-Mère Bridge.

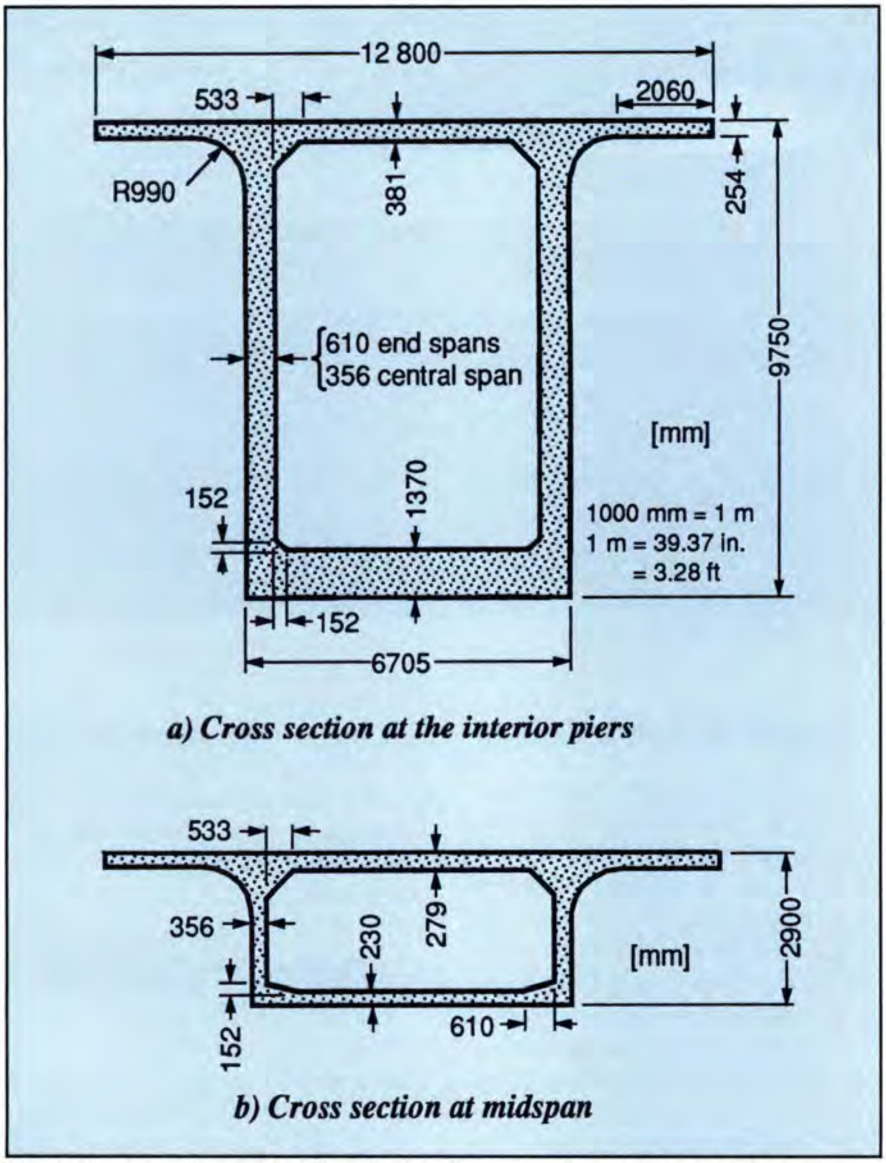

Fig. 3. Cross sections of the Grand-Mère Bridge at interior piers and midspan.

(9.5 ft) at midspan (Fig. 3), giving span-to-depth ratios of 18.6 and 62.6, respectively. The depth of the box girder in the end spans varies slightly, from 9.75 m (32 ft) at the interior piers to 8.53 m (28 ft) at the exterior piers. The total width of the deck is 12.8 m (42 ft), including a 6.7 m (22 ft) wide top flange of the single-cell box girder and two 3.05 m (10 ft) cantilevers. This torsionally stiff box girder bridge has only 1.83 m (6 ft) thick diaphragms over the main piers.

The designer planned to place 930 $m^3$ (1216 cu yd) of gravel ballast in the chambers of both end spans. The ballasted chambers and the wedge-shaped, solid cantilevers would counterbalance the weight of the main span during the cantilever construction and avoid any uplift of the box girder at the exterior piers as a consequence of the unusual center-to-end span ratio of 4.58.

### Construction and Materials

The bridge construction lasted two years, from 1976 to 1977, interrupted during the winter season to avoid cold weather concreting. The end spans were erected the first year and were cast on scaffolding. The central span was built in 1977, using the progressive cantilever method with traveling forms for the cast-in-place segments. After construction of the two segmental cantilevers of the central span, a keystone segment was cast and the continuity prestressing in the bottom flange and webs was added. Gravel ballast in the end spans was applied progressively, in three stages, as the cantilever construction proceeded.

The box girder was prestressed using 32 mm (1.25 in.) diameter prestressing bars with 1030 MPa (150 ksi) ultimate stress. The bridge was prestressed longitudinally during the cantilever construction with 284 straight bars located in the upper deck. The continuity prestressing comprises 80 slightly curved bars in the bottom flange and 48 draped bars in the webs (Fig. 4).

Shear reinforcement consists of straight prestressing bars, either vertical or inclined, and of conventional mild reinforcing steel. The deck is prestressed transversely with straight bars. Moreover, passive reinforcement of each segment was extended through adjacent segment joints. The specified concrete strength was 34 MPa (5000 psi), except in the middle portion of the central span, where 38 MPa (5500 psi) concrete was specified.

## INITIAL PROBLEMS AND SUBSEQUENT DISTRESS

The bridge had experienced several problems originating from three sources: construction problems, design assumptions, and the limited state of knowledge at that time. The bridge design included two distinguishing features: the slenderness of the 181.4 m (595 ft) central span and the exclusive use of straight and curved prestressing bars for the longitudinal and transverse prestressing.

### Construction Problems

Construction problems originated from three sources: coupling of numerous bars, concreting, and grouting the prestressing bars.

First, to join 10 to 15 m (33 to 50 ft) long bars over a length of up to 100 m (330 ft), the couplers used were sometimes unevenly screwed on two adjacent bars, causing some to fail. Damaged ducts, due to insufficient care during concreting, restricted the free sliding of bars and couplers during stressing. To overcome these two problems, holes had to be made in the top flange to replace or free some couplers and allow adequate prestressing. These necessary corrective measures

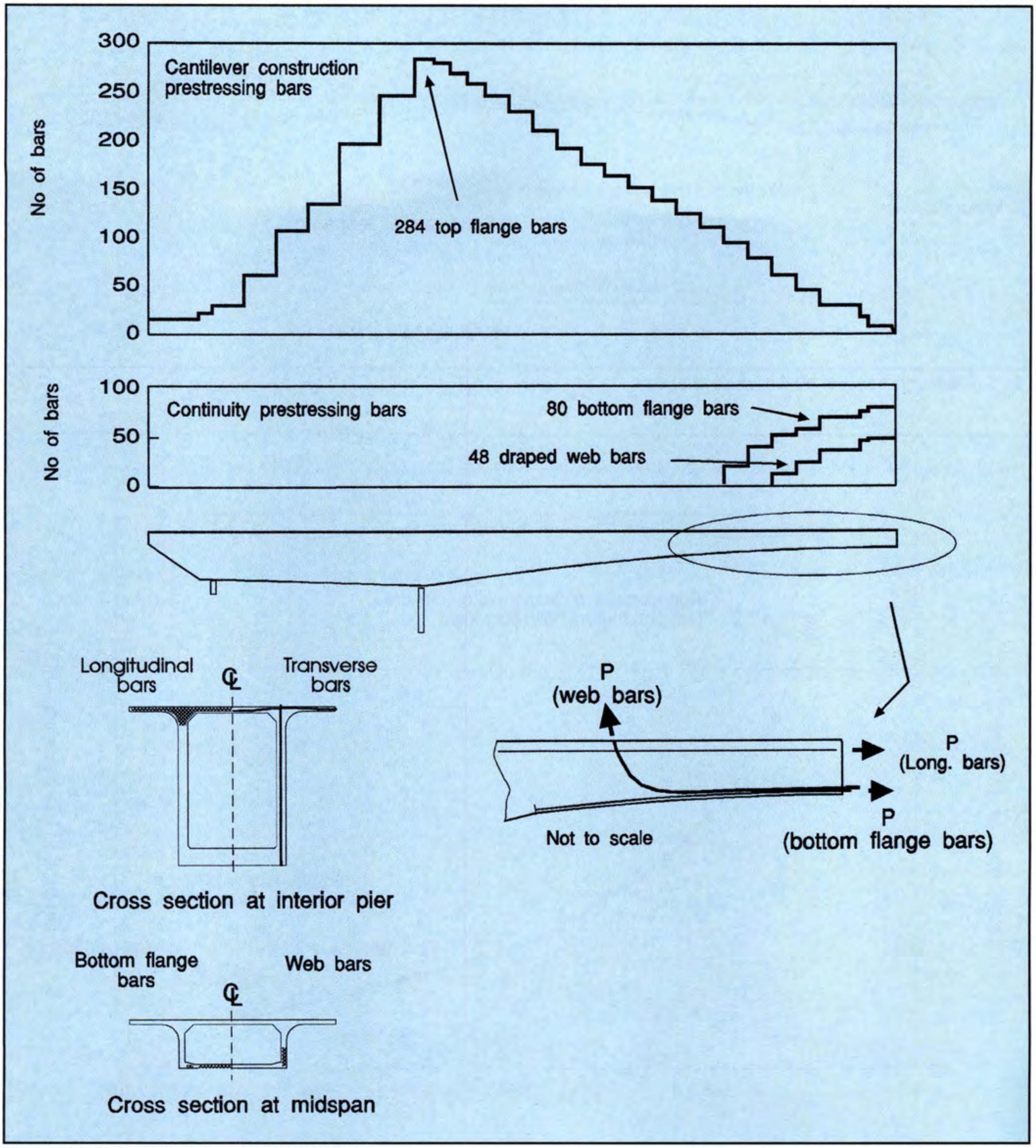


Fig. 4. Original prestressing layout of bridge.

reduced the concrete quality in portions of the top deck.

Second, concrete casting problems in the webs of the ballast chamber on the west side forced the designer to modify his original design due to weakened walls. The retained option consisted of widening the wedge-shaped cantilever span acting as counterweight with concrete, flush to the top deck, eliminating the need for gravel ballast at the west end. However, this solution required additional prestressing and thickening of the bottom flange (Fig. 5) in the end span.

Finally, duct injection of grout after prestressing was completely achieved with certainty only on 80 percent of the bars, the remainder being partially filled or not injected at all. Therefore, up to 20 percent of the top bars remained virtually unprotected against corrosion problems in the longer term.

Although the construction technique that was used was adequate in principle, the various construction problems reduced the durability potential of the structure, and the required quality level for this type of construction was not reached.

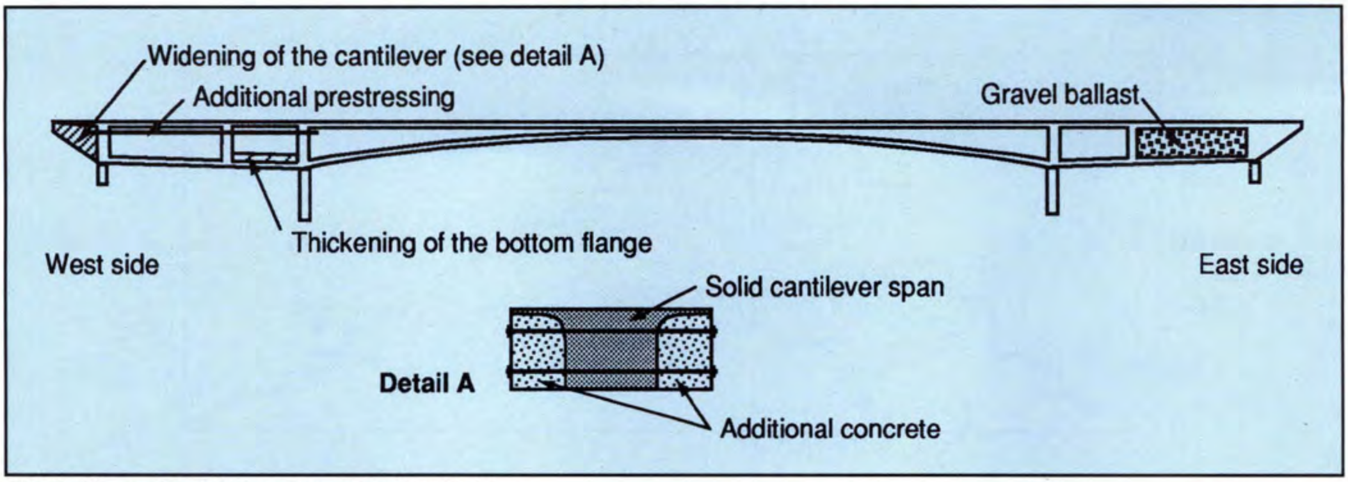

Fig. 5. Alternative scheme for west side of bridge.

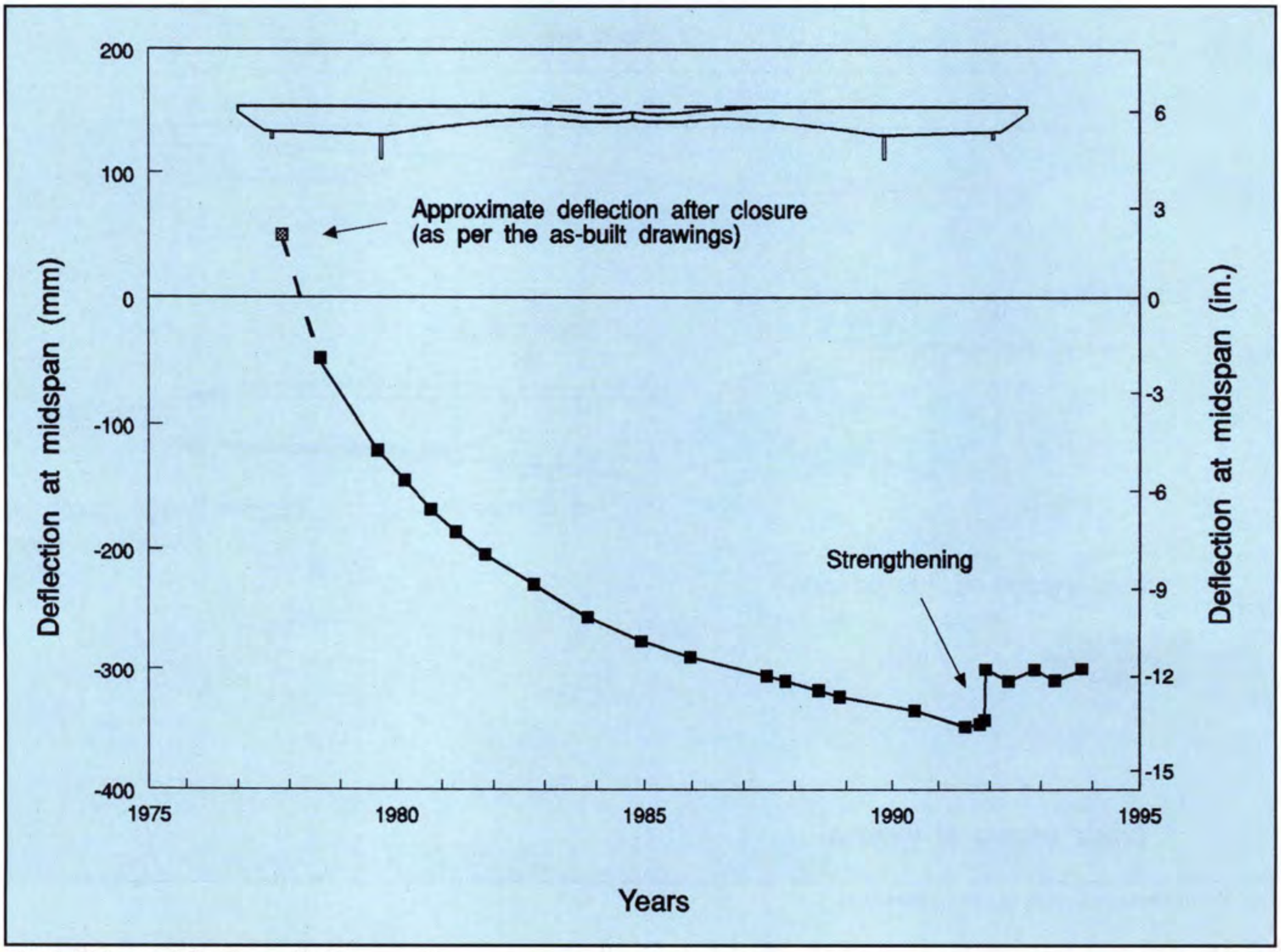

Fig. 6. Midspan deflections of bridge over time.

## Design Assumptions

Some of the problems in this bridge were due to the adoption of overly optimistic design assumptions. Wobble and curvature coefficients of 0.0007 per m (0.0002 per ft) and 0.30 per radian, respectively, were used in the original design. At that time, the values recommended by the Canadian Bridge Design Code[3] were 0.0026 per m (0.0008 per ft) and 0.30 per radian, respectively, very close to the prescribed values by CEB-FIP[4] and AASHTO.[5] Moreover, the elastic modulus of 213000 MPa (30,892 ksi) measured on short bars and used in the design[6] does not reflect the actual behavior of long bars, which contain up to eight couplers in some cases.

A more appropriate value of 193000 MPa (28,000 ksi)[3,4] would be recommended in such a case. If the theoretical curvature coefficient is assumed to be adequate, the calculated wobble coefficients, according to measurements during construction,[6] were approximately 0.0053 per m (0.0016 per ft) in the top and bottom flanges and 0.0145 per m (0.0042 per ft) for the web prestressing bars. These values are, respectively, 7 to 20 times larger than the design value.

### State of Knowledge

The knowledge related to long span, segmental prestressed concrete bridges was limited or not yet published at that time. The length-to-midspan depth ratio of 62.6 for the main span is significantly higher than a more recent maximum value of 50 recommended by Podolny and Muller.[7]

The most slender structure in Europe, as reported by Mathivat,[8] has a slenderness ratio of 47. Although the slenderness at midspan can theoretically reach values up to 60, Mathivat recommends a much smaller maximum value to reduce the creep and thermal gradient effects.

The Post-Tensioning Institute (PTI) suggests a span-to-depth ratio at midspan of about 42.[9] It is obvious that the 2.90 m (9.5 ft) bridge depth at midspan was too small and a value of at least 3.70 m (12 ft) would have been more appropriate.

An important design consideration of concrete box girders concerns stresses induced by thermal gradients between the top and bottom slabs. Little was known about thermal stresses in 1976 and they were not accounted for in the bridge design. For the Grand-Mère Bridge, a linear thermal gradient of 10°C (18°F) creates a positive bending moment at midspan of the same order of magnitude as a full two-lane live load on the central span. Such loading is included in modern box girder bridge design.

### Observed Distress

Shortly after its completion, the central span of the bridge underwent unexpected deflection. Measurements were then taken regularly, and, by 1986, the average midspan deflection, fluctuating with seasonal temperature changes, had reached 300 mm (12 in.) (Fig. 6). This was considered sufficiently abnormal to proceed with extensive studies.

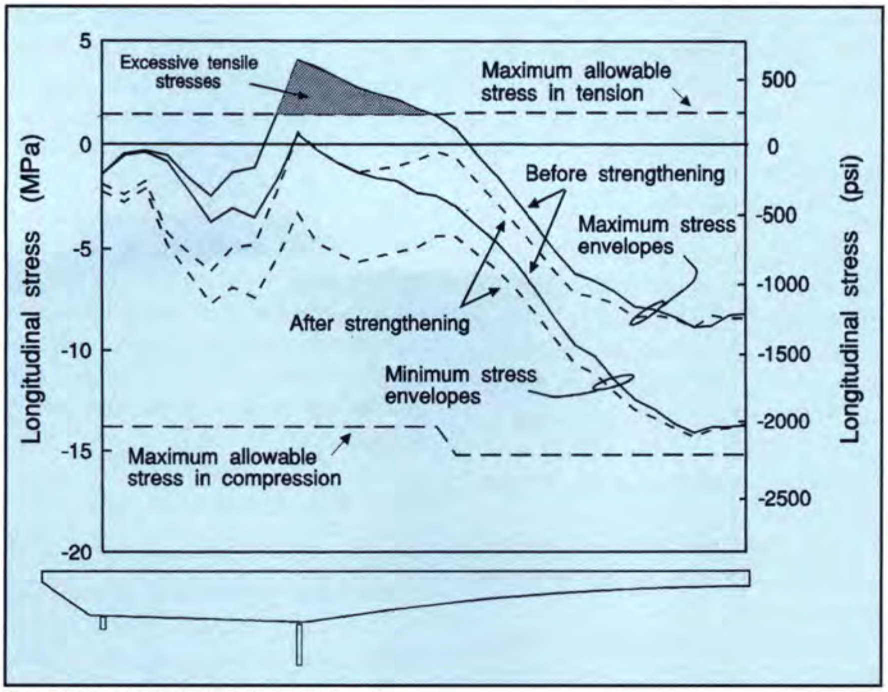

Fig. 7. Stress variation in top flange.

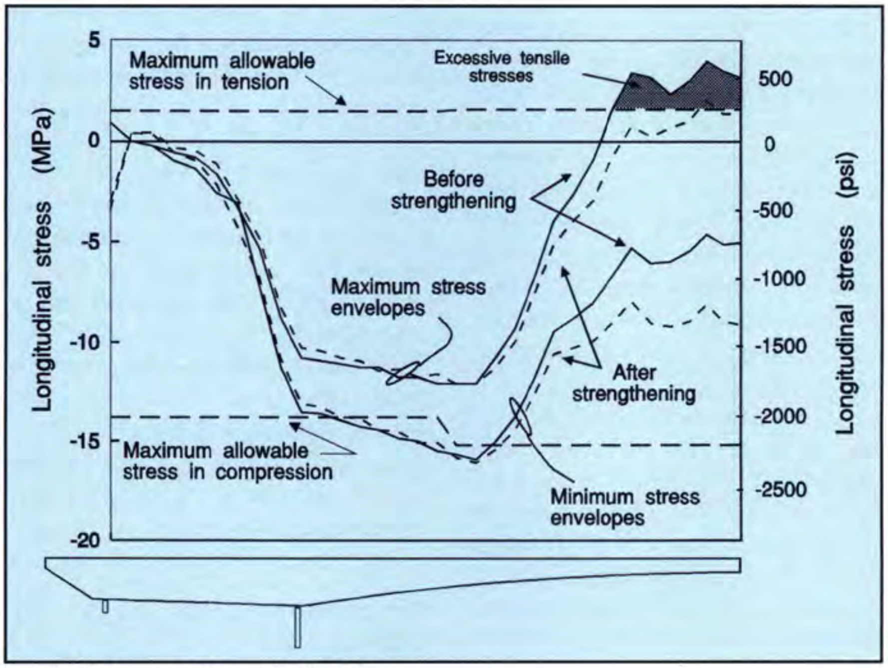

Fig. 8. Stress variation in bottom flange.

Despite this unusual deflection, a careful inspection of the bridge in 1988 did not show any evidence of significant distress and cracking was observed in only two areas. At the third points of the central span, tiny fishbone cracks were found on the internal face of the bottom flange, at the location where the unexpected deflection begins (Fig. 6). However, these cracks were fine, their width being about 0.1 to 0.2 mm (0.004 to 0.008 in).

Wider transverse cracks were found in the top slab of the east end span, with some traces of chloride efflorescence. In 1985, coring of concrete in various locations on the top and bottom slabs indicated average compressive strengths of 51 and 43 MPa (7400 and 6200 psi), respectively. The cored concrete showed little sign of deterioration and the waterproofing membrane underneath the asphalt appeared to be in good condition, except along parapets.

### Structural Safety and Behavior

Cracking observed in some areas was not considered to impair the bridge safety. The location of the first set of cracks coincides with the dead-end anchorage of the bottom flange continuity prestressing bars (Fig. 4) and was caused by the prestressing force being transferred to the concrete. The second set of cracks was attributed to differential shrinkage between the webs and the top flange of the box girder.[10]

Nonetheless, the excessive deflection of the central span was a major concern for the QMT authorities. Numerous analyses of this structure by the QMT and various independent consulting firms concluded that structural safety was not compromised in the short-term period. The most recent study[6] showed that the prestress losses due to shrinkage, elastic shortening and relaxation were 18 percent larger than assumed in the original design. The average final prestressing steel stress was estimated to be 48 percent of the ultimate prestressing bar strength ($0.48f_{pu}$), whereas the design value was $0.59f_{pu}$.

This significant reduction of prestressing force does not impair bridge safety. However, it enhances creep effects and it can affect bridge behavior and safety in the longer term. The high tensile stresses in the upper flange over the interior piers and in the bottom flange at midspan (Figs. 7 and 8) are sufficiently in excess of the allowable values to cause concrete cracking and probably lead to serious corrosion problems in the future. Moreover, the increasing unusual deflection of the central span, caused by reduced prestressing, was not showing any sign of stabilization.

Various groups conducted studies on the effect of creep using time-step analyses and nonlinear models. Based on calibration with measured deflection, they concluded that the ratio of the final creep elastic modulus ($E_f$) to the initial elastic modulus ($E_i$), given by $\alpha = E_f / E_i$, varies between 20 and 30 percent. These values correspond to creep factors of 4.0 and 2.3, respectively — not surprising for concrete prestressed at an early age. Such high values would be predicted by the 1970 CEB-FIP Model Code,[4] as reported in Ref. 11. Thus, the final state of stress, due to dead load and prestressing forces applied before closing the structure, can be obtained as the average between the cantilever state and the continuous state with the following equation:[7]

$$\sigma_D = \alpha\sigma_{\text{cantilever}} + (1 - \alpha)\sigma_{\text{continuous}} \tag{1}$$

This equation takes into account the effect of creep and allows subsequent easier computation of stresses. In the various studies on the Grand-Mère Bridge, different professional opinions were given about the amount of creep that had already taken place in 1986. Depending on the assumptions used in the time-dependent analysis to evaluate the current state of stress, the total creep in 1986 ranged between 65 and 85 percent of the long-term value. Nevertheless, all the analyses were calibrated on the observed deflection at that time and, except for the remaining creep, they indicated, with a sufficient degree of confidence, the current state of stress before strengthening. After these studies were completed, the deflection increased by more than 90 mm (3.5 in.) in four years (1986 to 1990), showing that creep was still occurring.

### Conclusions on the Current State

From the various studies conducted on the Grand-Mère Bridge, it can be concluded that the main source of problems was the insufficient prestressing due to the following factors (in order of importance):

**1.** Overly optimistic design assumptions about friction coefficients and an underestimation of prestress losses.

**2.** Construction problems which worsened the design unconservatism.

**3.** Lack of code specifications about thermal gradients and a maximum span-to-depth ratio.

### Québec's Past Experience in Strengthening Long Span Prestressed Concrete Bridges

In 1977, the Grand-Mère Bridge was the third long span prestressed concrete bridge built in Québec, having the longest span. The first one, the St. Adèle Bridge, was, in 1964, the first North American cast-in-place segmental bridge.[9] This bridge suffered from insufficient prestressing causing excessive deflection and cracking. It was strengthened in 1988 by external prestressing.[13] Since then, the bridge deflection rate and cracking has stabilized.

The second one, the Lièvre River Bridge, built in 1967, was the first North American precast segmental bridge.[14] This bridge was strengthened in 1987 because insufficient prestressing led to the opening of joints between segments. In 1968, the joints were not glued with epoxy as recommended today.[15] However, that bridge did not suffer from creep deflection as much as the Grand-Mère and the St.Adèle bridges, as expected for precast segmental construction. The adequate behavior of the bridge after strengthening indicates the success of the operation.

The last two studies on the Grand-Mère Bridge[6,16] indicated the need for early corrective action on the bridge, i.e., additional prestressing, before more distress develops. Based on the experience gained in the strengthening of the first two bridges, considering the advice from experts indicating corrective actions, and due to its importance, the QMT decided to strengthen the Grand-Mère Bridge.

## STRENGTHENING PROGRAM

### Design Assumptions

Code provisions or guidelines for the strengthening of existing bridges are not covered in any bridge code. The strengthening was based on requirements of various recent bridge design codes such as the Canadian Code[17] and the Ontario Code.[18] The expertise of the French Department of Transportation (SETRA) in the strengthening of segmental prestressed

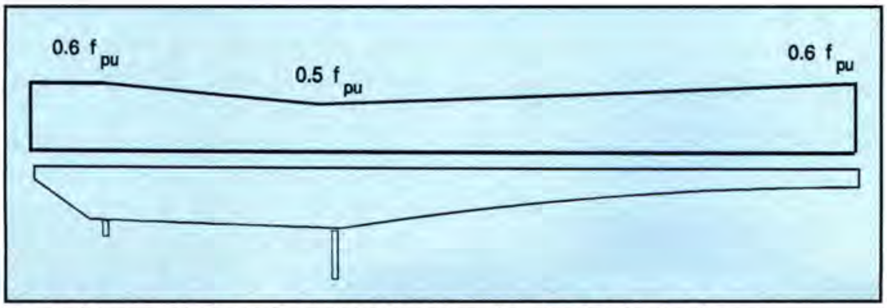

Fig. 9. Stress variation in the original prestressing steel of the top flange.

concrete bridges was also considered in establishing the general concept.

In the strengthening of existing structures, the assumptions determining the current state of stresses should account for some degree of uncertainty. In stress computations, a range of stress values was used rather than specific values. Variable load factors and load combinations were considered. Such an approach could also be adopted for the design of new bridges considering construction problems and some degree of uncertainty often met in long span, prestressed concrete bridges. The following assumptions about the Grand Mère Bridge were made.

**State of stresses before strengthening** — Based on observed evidence, a linear effective stress varying from $0.5f_{pu}$ to $0.6f_{pu}$ in longitudinal prestressing steel contained in the top slab was assumed (Fig. 9). For the additional 20 bars on the west side, an effective stress of $0.6f_{pu}$ was assumed. The corresponding forces over the interior piers are equal to 117600 and 127500 kN (26,440 and 28,670 kips) for the east and west sides, respectively. On the other hand, the effective stress level in the continuity prestressing steel was taken as $0.5f_{pu}$, whereas for the vertical and inclined web bars, a value of $0.6f_{pu}$ was considered. The stress redistribution due to creep, for loads acting on the structure in the cantilever stage, was considered using Eq. (1) with values of $\alpha$ ranging from 0.2 to 0.4.

**Live load** — For live load, the current CSA loading CS-600,[17] a four-axle 600 kN (67.4 ton) loading model, was used. The dynamic load allowance (DLA), or impact factor, applied to the CS-600 loading is a function of the first natural frequency of the bridge. It varies between 0.20 and 0.40. Based on Refs. 6 and 16, the selected DLA was 0.20 and 0.40 for the negative and positive bending moments, respectively. The lane load was the governing loading for this bridge. It consists of a CS-600 loading model, with each axle load reduced to 60 percent (with the corresponding DLA), superimposed on a uniform load of 12 kN/m (822 lb per ft) with a DLA of 0.10.

**Thermal gradient** — The most recent French regulation recommended a short-term linear thermal gradient of 12°C (21.6°F) between the top and bottom flanges and a reduced long-term thermal gradient of 6°C (10.8°F). The short-term gradient is applied with dead load only, whereas the long-term gradient is combined with live load. In the analysis for the strengthening operation, either 100 or 50 percent of the positive gradient [12°C or 6°C (21.6°F or 10.8°F)] was considered for positive bending moment calculations. For negative bending moments, when a negative thermal gradient occurs, minus half of these values were considered, giving gradients equal to -6°C and -3°C (-10.8° and -5.4°F) for short- and long-term occurrences, respectively.

**Load cases** — According to the CSA-S6,[17] a unique load combination ($D + L + 0.8T$, where $D$ = dead load, $L$ = live load, and $T$ = temperature) shall be considered as the governing condition for stress computations at service load level. Although such an approach may be acceptable for the design of new bridges, its application becomes questionable for existing bridges where the uncertainty about prestressing forces is larger. It was felt that a broader range of values should be assumed in the strengthening design.

Table 1. Load cases and load factors considered in the strengthening design.

| Load case number | Nature of loads | | | | |
|---|---|---|---|---|---|
| | $D$ | $L_{Mmin}$ | $L_{Mmax}$ | $T$ | $P$ |
| 1 | 1.0 | | | | 1.0 |
| 2 | 1.05 | | | | 0.95 |
| 3 | 0.95 | | | | 1.05 |
| 4 | 1.0 | 1.0 | | | 1.0 |
| 5 | 1.0 | | 1.0 | | 1.0 |
| 6 | 1.0 | 1.0 | | -0.4 | 1.0 |
| 7 | 1.0 | | 1.0 | 0.8 | 1.0 |
| 8 | 1.0 | | | -0.5 | 1.0 |
| 9 | 1.0 | | | 1.0 | 1.0 |

$D$ = dead load effects
$L_{Mmin}$ = live load effects associated with the negative bending moment
$L_{Mmax}$ = live load effects associated with the positive bending moment
$T$ = temperature gradient effects
$P$ = prestressing force effects

Several loading cases were used in the analyses to determine the worst loading conditions, as listed in Table 1.

**Allowable stresses** — The allowable tensile stresses are $0.25\sqrt{f_c'}$ and $0.50\sqrt{f_c'}$, expressed in MPa ($3.0\sqrt{f_c'}$ and $6.0\sqrt{f_c'}$ in psi), for severe and normal exposure, respectively — the first applying to the top slab and the second, to the bottom slab. These limits are given in OHDBC-83,[18] which allows some tensile stress in precompressed joints if waterproofing is present, as in this case. The stresses were calculated considering the average of the transformed and non-transformed section properties (including or not including reinforcing steel at joints, respectively).

## Strengthening Prestress

**Strengthening work constraints** — In strengthening this bridge, several constraints had to be considered. First, access to the inside of the box girder was restricted to an existing 600 x 600 mm (2 x 2 ft) opening in the bottom flange of the east side end span. The possibility of cutting an access of approximately 1 x 2 m (3 x 6 ft) in the bottom flange of the west side end span was studied and allowed. In the top flange, only localized small holes, with diameters not

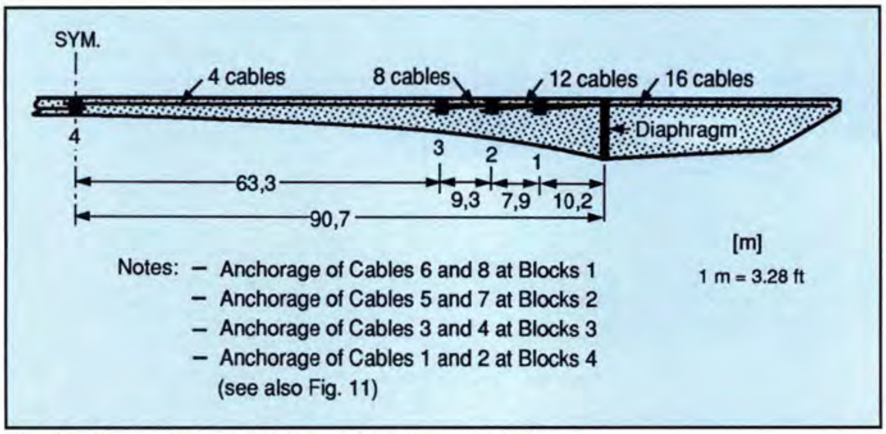

Fig. 10. Strengthening cables and anchorage blocks.

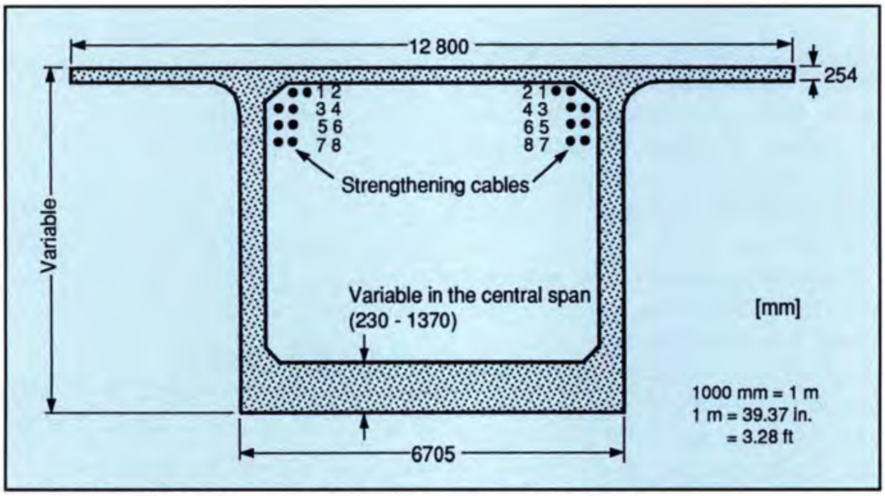

Fig. 11. Location of strengthening cables inside cross section.

exceeding 180 mm (7 in.), were drilled for pouring concrete.

For practical purposes and aesthetics, the prestressing cables were localized inside the box girder. Moreover, the bridge had to remain open, with minimum interference to traffic; the bridge is on a main access road to a northern industrial region of Québec.

**Prestressing cable layout** — The strengthening operation was performed using 32 cables, 16 from each end (Fig. 10). These straight cables were placed just underneath the top slab near the webs, eight on each side (Fig. 11). Cables 3 to 8 are 12S15 cables [twelve 15 mm (0.6 in.) diameter strands], each strand having a nominal section of 140 mm² (0.217 sq. in.); 15S15 cables were used for Cables 1 and 2. Strands were stressed at 70 and 82 percent of their nominal ultimate strength, equal to 1860 MPa (270 ksi), for 12S15 and 15S15 cables, respectively.

The 12S15 cables transfer a prestressing force of 2190 kN (492 kips) each, whereas the prestressing force is 3205 kN (720 kips) for 15S15 cables. At the section over each interior pier, the total force added by the 16 strengthening cables is equal to 39100 kN (8790 kips), which corresponds to 31 and 33 percent of the prestressing force of the original design, at the west and east interior piers, respectively.

**Final prestressing** — Creep, relaxation, friction and elastic losses were estimated at an average of 11 percent of the added prestressing. Creep losses for load application to an old concrete were assumed equal to 20 percent of the creep losses of a 28-day concrete, based on extrapolation of experimental data reported in Ref. 11, leading to creep losses of 0.3 percent of the added prestress. The elastic shortening of the existing internal prestressing bars due to the external prestressing produced losses of about 7 percent. Relaxation and friction losses were estimated at 2 percent each.

Individually lubricated sheathed strands were used. All cables, made of either 12 or 15 strands, were inserted in individual PVC ducts which were later grouted prior to tensioning.

**Final stresses** — Final stresses, computed with the above assumptions,

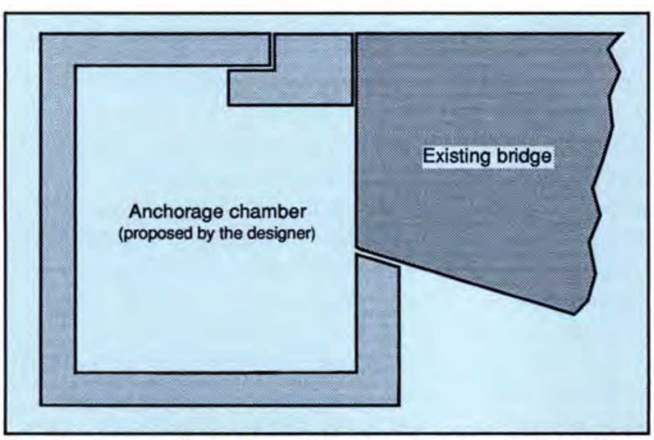

Fig. 12. Schematic section of dead-end anchorage chamber.

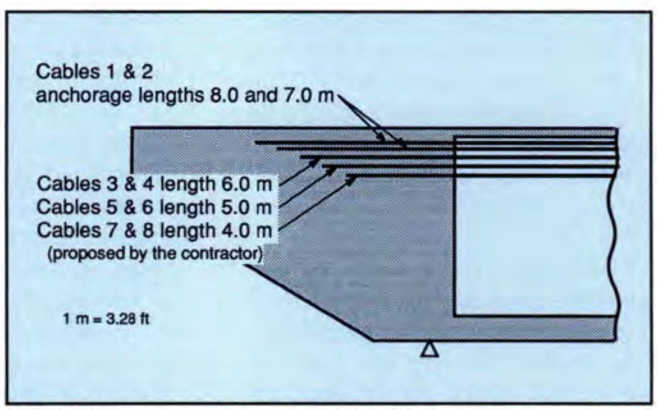

Fig. 13. Dead-end anchorages in solid trapezoidal cantilevers.

Fig. 14. Anchorage device at dead end of each strand.

were equal to $0.96\sigma_{all}$ and $1.08\sigma_{all}$ for the top and bottom slab, respectively, within the allowable limits in tension and compression for most sections along the bridge. In compression, the allowable stress was exceeded by about 10 percent over a very limited area. Allowable stresses were computed based on original concrete design strength at 28 days. Final stress variations are shown in Figs. 7 and 8 for the top and bottom slab, along with the corresponding values before strengthening.

## CABLE ANCHORAGES

### Dead-End Anchorages

The cable dead-end anchorages were located at both ends of the bridge. In the QMT design, anchorage chambers (Fig. 12) were planned at both ends. To minimize traffic disturbance, the contractor suggested anchoring the cables in drilled holes, subsequently grouted, in the 12 m (40 ft) long solid trapezoidal cantilevers (Fig. 13). This accepted alternative required the removal of the sheath and grease on all strands and the use of a special anchorage device squeezed on each strand (Fig. 14) to increase bond.

During drilling, it was found that the concrete of the solid cantilevers was in questionable condition. This required additional grout injection to obtain a sound concrete mass. Unfortunately, this alternative delayed the strengthening operation by more than two months but minimized traffic interference.

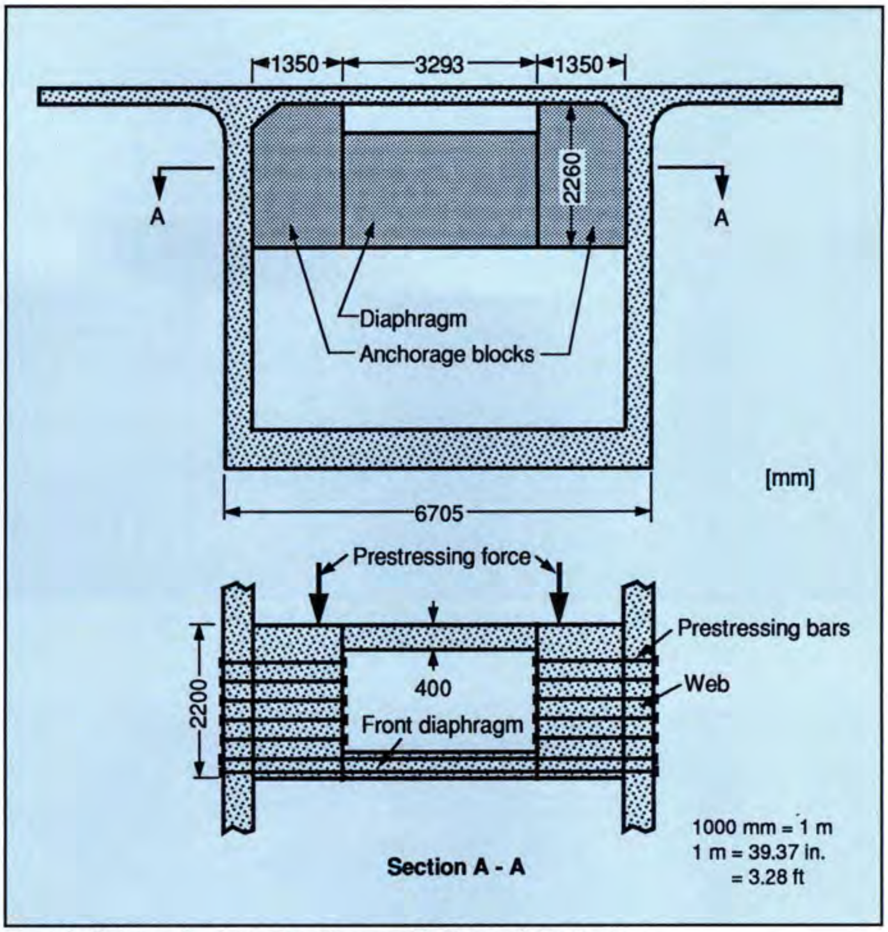


Fig. 15. Schematic view of anchorage blocks and diaphragms.

### Anchorage Blocks

Inside the box girder, the 32 added cables were anchored to the webs by means of 14 blocks distributed along the bridge (Fig. 10). At each anchorage section, the concrete blocks were linked by two diaphragms (Fig. 15) to eliminate any bending moment in the box girder webs. Tensile stresses produced in the front diaphragm by the couple due to the longitudinal prestressing were eliminated by prestressing the diaphragm with Dywidag bars running from one web to the other. The back diaphragm, subjected to a compression force, was only reinforced.

At cable tensioning, a total force of 5315 kN (1195 kips), corresponding to $0.85f_{pu}$, was transferred to the webs by each anchorage block, except for the central ones. Dywidag prestressing bars stressed at 412 MPa (60 ksi), 40 percent of their ultimate strength, were used to fasten the blocks to the webs. The webs at the block location were chipped 25 mm (1 in.) deep before concreting the blocks.

To compute the transverse prestressing force, the friction coefficient at the interface was assumed equal to 1.0. Some details of the blocks and diaphragms are shown in Fig. 16. The outside view of the Dywidag bars, before they were protected by a 250 mm (10 in.) concrete cover, is shown in Fig. 17. A significant amount of reinforcement was used in the blocks. Reinforcing bars were placed in the forms in three main directions, giving a total reinforcement volume varying from 3.0 percent in Block No. 1 (Fig. 18) to 2.4 percent in Block No. 4.

In computing load transfer from the blocks to the webs, various assumptions were made. The design was based on the shear-friction theory. Also, as a design check, the shear force produced in the transverse Dywidag bars by the longitudinal pre-

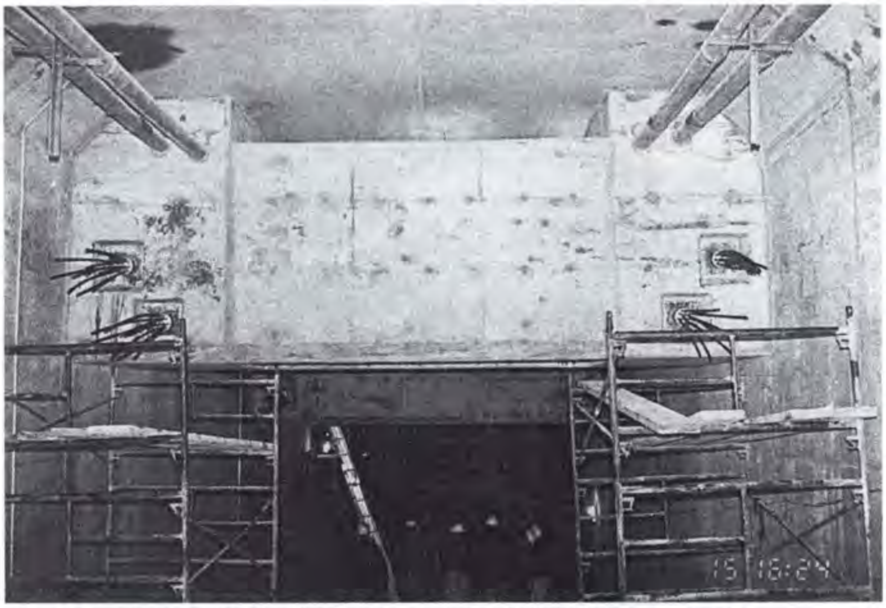

Fig. 16. Anchorage Blocks No. 3 at 27.4 m (90 ft) from interior support.

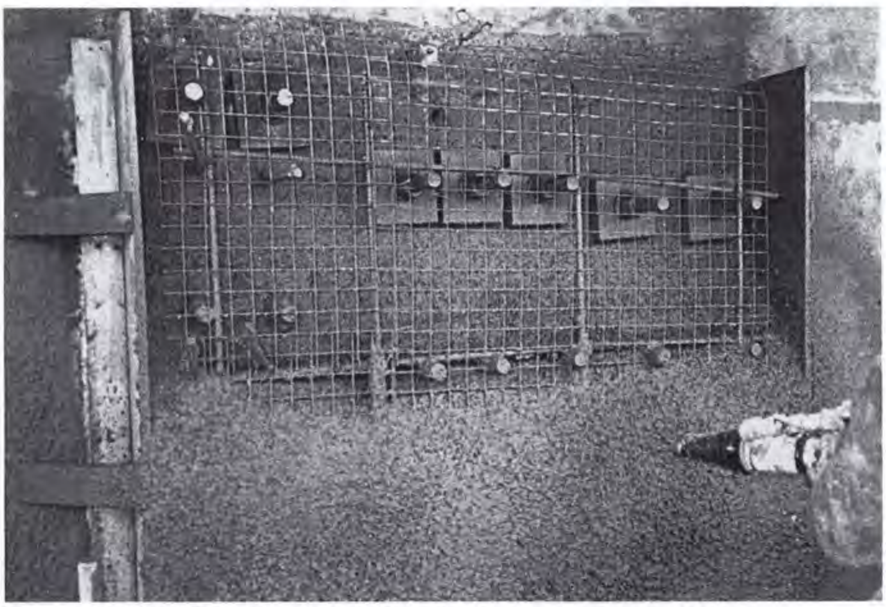

Fig. 17. Outside view of transverse Dywidag bars at anchorage block.

stressing was computed assuming an elastic force distribution between the bars, as one would do in shear-eccentric bolted connections in steel structures. Finally, in the verification process, the force induced in the webs and the corresponding bending moments were obtained using the simplified truss model shown in Fig. 19. From this figure, by geometry and equilibrium:

$$x = \frac{w + b_p - t}{1 + \tan\theta} \quad (2)$$

$$M_{web} = P\,e_w = P\,(w - x)/2 \quad (3)$$

$$T = C \approx P\,\frac{e - w/2 - e_w}{L - t/2 - (x/2)\tan\theta} \quad (4)$$

In this truss model, it is assumed that the overall bending moment induced by the eccentricity of the applied force with respect to the web centroidal axis is equilibrated solely by the additional force, $T$, developed in the prestressing bars of the front wall and the corresponding compression force in the back wall. Theoretically, the initial prestressing force in the front wall, $P_b$, should remain constant.

These verifications indicated that the bending moment in the web would create additional tensile stresses in the web equal to 70 percent of the allowable limit. This figure was obtained by assuming a vertical dispersion angle of 45 degrees of the prestressing force in the anchorage block. Calculations also indicated that the stresses in the compression struts and nodal zones were smaller than the maximum limits.

## PRESTRESSING APPLICATION

### Prestressing Sequence

Bridge strengthening work began in June 1991 and lasted until November 1991. Dead-end anchorage of the cables and slippage problems delayed the work. The prestressing was applied strand by strand with a monostrand jack.

To reduce as much as possible any lateral bending moment due to uneven prestressing in the bridge, pairs of Cables 8 to 3 were tensioned in the following sequence from anchorage Blocks No. 1 to No. 3 (see Fig. 10). Two cables were tensioned simultaneously, one on each side of the box girder. At each pair of anchorage blocks, one pair of cables was first tensioned at the west side of the bridge, followed by two pairs on the east side and, finally, the remaining pair on the west side. The prestressing progressed at the rate of four pairs of cables per day. The sequence is given in Table 2 for Cables 6 and 8.

The tensioning sequence for Cables 1 and 2 at Blocks No. 4, at the center of the bridge (Fig. 10), was different. There was no diaphragm between Blocks No. 4 for clearance reasons. Also, there were no Dywidag bars to fasten the blocks to the webs. However, the anchorage blocks were bearing longitudinally at their bottom against existing anchorage blocks in the bottom flange used to anchor the continuity prestressing (Fig. 20).

The blocks were also bearing against the top deck. Cables 1 and 2 were tensioned from both sides of each block simultaneously such that no horizontal force was transmitted to the web. The tensioning was done alternately from one web to the other, for each pair of cables. To avoid too much uneven prestressing, the cables were tensioned to

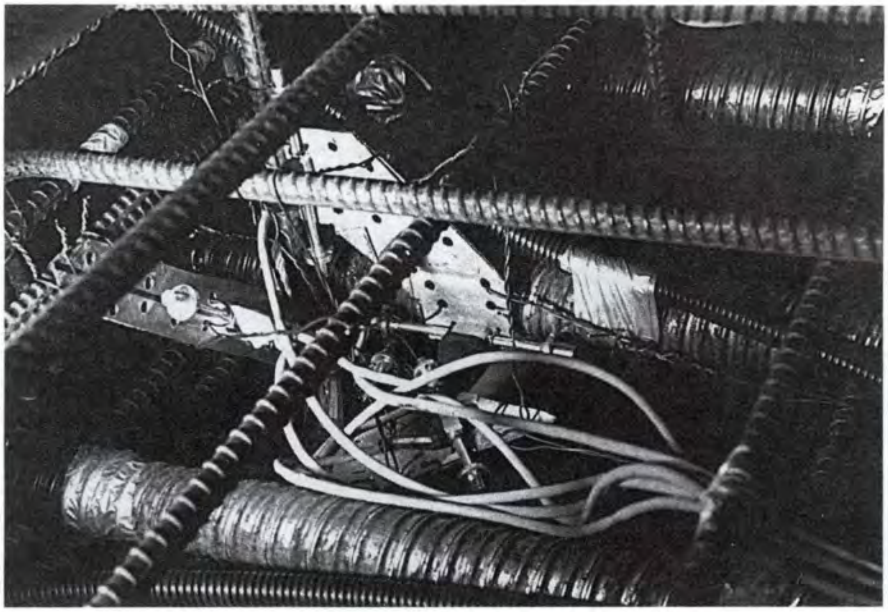

Fig. 18. Reinforcing steel in anchorage block.

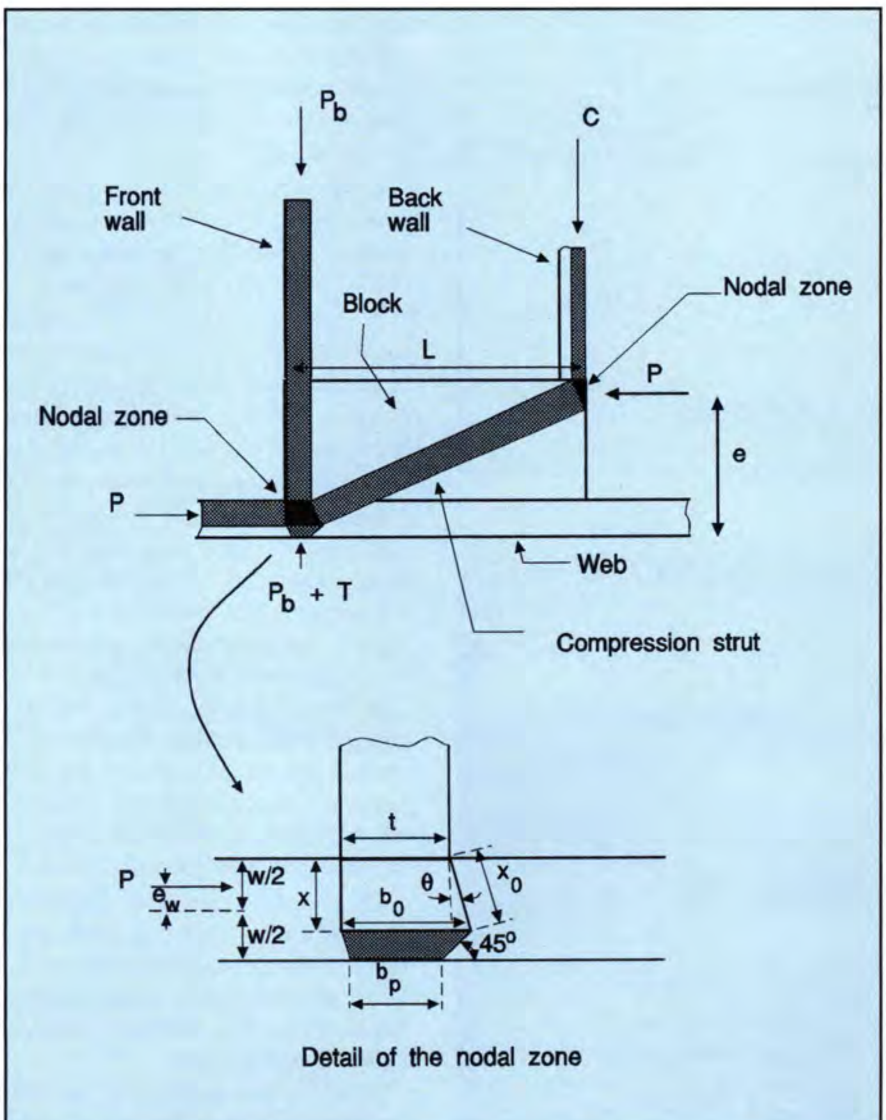


Fig. 19. Truss model at anchorage block.

50 percent of their final tensioning force and then brought to their final value of 0.82 $f_{pu}$. The sequence for Cables 1 and 2 is presented in Table 3.

## Duct Grouting and Prestress Losses

Grouting cable ducts when individually lubricated sheathed strands are used is controversial. Although grouting is not strictly required with external prestressing cables located inside the box girder, it may be beneficial. Past experience[19] indicates that strand sheaths can be damaged under contact stresses at cable deviators. Grouting before tensioning reduces contact stresses and avoids sheath damage. However, for cables longer than 30 m (100 ft), it is recommended to initially tension each strand at approximately $0.1 f_{pu}$ before grouting so as to align the strands inside the cable duct.

For the Grand-Mère Bridge, the contractor decided to grout the cable ducts before tensioning. For perfectly straight cables, this would not have caused a problem. However, the loose strands were not perfectly aligned in the ducts and some were probably twisted. It followed that friction losses were larger than expected for strands in individually lubricated sheaths.

As an example, the wobble coefficient for Cables 1 and 2, anchored at Blocks No. 4, computed from the measured force and elongation, was equal to 0.00124 per m (0.00038 per ft), close to 0.001 per m (0.0003 per ft), a value suggested in Ref. 19. However, the curvature coefficient was evaluated at 0.20 per radian, more than four times the value of the French regulations.[19] For all cables in this project, it would have been advisable to tension the strands at $0.1 f_{pu}$ prior to grouting.

## Construction Site

The width of the bridge deck permitted two lanes of traffic to be kept open for most of the construction period, except during the curing of the anchorage blocks (Fig. 21). During the first four hours after concrete pouring, only one lane was open on the bridge, and escorting vehicles limited the bridge speed crossing to 15 km/h (10 miles per hr).

Table 2. Prestressing sequence at anchorage Blocks No. 1.

| Side of the bridge | Side of the cross section | Cable | Prestressing force (percent of final value) |
|---|---|---|---|
| West | North and south | 6 | 100 |
| East | North and south | 6 | 100 |
| East | North and south | 8 | 100 |
| West | North and south | 8 | 100 |

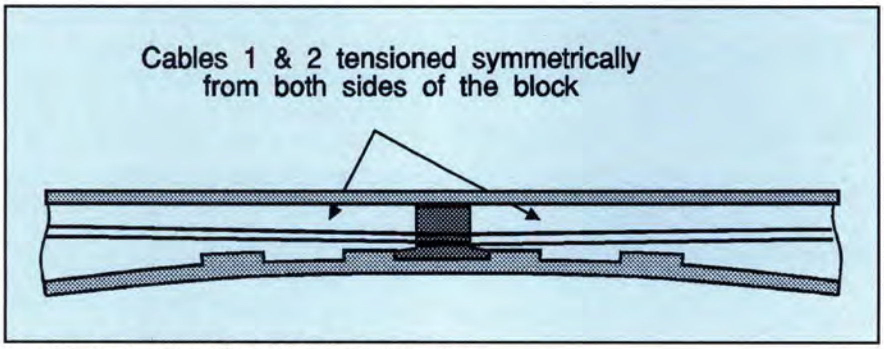


Fig. 20. Anchorage Block No. 4 at midspan.

Table 3. Prestressiing sequence at anchorage Blocks No. 4.

| Side of the bridge | Side of the cross section | Cable | Prestressing force (percent of final value) |
|---|---|---|---|
| East and west | North | 1 | 50 |
| East and west | South | 1 | 50 |
| East and west | South | 2 | 50 |
| East and west | North | 2 | 50 |
| East and west | North | 1 | 100 |
| East and west | South | 1 | 100 |
| East and west | South | 2 | 100 |
| East and west | North | 2 | 100 |

Fig. 21. Bridge strengthening operations, working from deck.

## CONCLUSIONS

The experience and findings gained from the Grand-Mère Bridge project — from the construction to the strengthening operation — are summarized here:

**1.** The span-to-depth ratio of box girder bridges at midspan should not exceed 50.

**2.** Thermal gradients should be considered in the design and analysis of long span concrete bridges.

**3.** Curved prestressing bars are not recommended.

**4.** Precast segmental construction, with epoxied joints, is preferable to cast-in-place construction because of a better control on materials and workmanship, of reduced creep effects, and of higher concrete strength.

**5.** Cables more than 30 m (100 ft) long made of individually lubricated sheathed strands inserted in ducts should be initially tensioned to 10 percent of their final tensioning force before grouting.

**6.** The strand-by-strand symmetrical application of the prestressing force worked satisfactorily for both construction purposes and bridge behavior.

**7.** Dead anchorage in the wedge-shaped cantilever span was not as successful as expected. Questionable concrete quality and slippage of cables delayed the work significantly. However, traffic disruption was avoided.

**8.** Cast-in-place anchorage blocks did not show any sign of distress. The anchorage block-and-diaphragm assembly and the transverse prestressing bars appeared to work efficiently.

**9.** The technology used for strengthening the Grand-Mère Bridge can be applied to both pretensioned and post-tensioned concrete bridges, either for the strengthening of existing bridges or for the construction of new structures.

**10.** The objectives of the strengthening operation — that is, to create a more favorable state of stress and stabilize the deflection — were achieved based on the first two-year deflection survey shown in Fig. 6.

**11.** The strengthening project, which cost $1.3 million, is expected to extend the bridge's useful life.

## ACKNOWLEDGMENT

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## APPENDIX — NOTATION

$C$ = additional axial force in back wall due to longitudinal prestressing force
$DLA$ = dynamic load allowance factor (impact factor)
$e$, $e_w$ = eccentricities of applied prestressing force and web reaction, respectively
$E_i$ = initial elastic modulus
$E_f$ = final elastic modulus accounting for creep effects
$f_{pu}$ = ultimate tensile stress of prestressing bars or strands
$P$ = longitudinal prestressing force
$P_b$ = initial prestressing force in front wall
$T$ = additional axial force in front wall due to longitudinal prestressing force
$\alpha$ = ratio of final creep elastic modulus ($E_f$) to initial elastic modulus ($E_i$)
$\sigma_{all}$ = code allowable stress
$\sigma_{cantilever}$ = longitudinal stresses computed in cantilever structure
$\sigma_{continuous}$ = longitudinal stresses computed in continuous structure
$\sigma_D$ = stresses due to dead load and prestressing acting on structure in cantilever construction