

The Twelve Mile Creek Precast Prestressed Segmental Bridges



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The Twelve Mile Creek Bridges, located on Highway 406 linking the City of St. Catharines, Ontario, to the Queen Elizabeth Way, were completed in 1982. For the last two years the bridges have been used for access and hauling fill before being opened to the public this past October.

The route of the new highway, which follows the valley through which the creek flows (as shown in Fig. 1), was selected for economy and minimum social and environmental impact. The Twelve Mile Creek originally formed part of the Welland Canal, connecting Lake Erie to Lake Ontario. Today, it is part of a fast flowing tailrace for a generating station.

The twisting alignment and profile of the new divided highway at the creek crossing was dictated by an adjacent interchange, site terrain, poor soil conditions, and property constraints. The size, number, and location of piers in the waterway also were restricted and the requirement for a minimum vertical clearance above the creek had to be met.

In addition, the proximity of the adjacent interchange required part of the roadway on the bridge to be flared to accommodate the ramps. Therefore, where the divided highway crosses the creek, two bridges, one for the north-bound lanes and the other for the south-bound lanes, were necessary to meet the specified requirements.

Synopsis

Describes the overall design of the bridges plus the fabrication and erection of the segments. A special feature of these structures is the use of transverse ribs along both segment faces to accommodate the deck flare. Problems encountered during construction and their method of solution are also discussed.



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Several alternative types of bridges were investigated during the preliminary design stage. The precast post-tensioned segmental concrete box girder bridges were selected for their ability to economically satisfy the complex geometry, restricted construction depth, and the variable roadway width requirements.

Description of Bridges

The bridges consist of a 594 ft (181 m) long northbound lanes structure and a 1310 ft (399 m) long southbound lanes structure. The northbound lanes structure is on a horizontal curve with three continuous spans of 172, 250, 172 ft (52,

76, 52 m) while the southbound lanes structure is on a horizontal S-curve with six continuous spans of 155, 250, 250, 250, 250, 155 ft (47, 76, 76, 76, 76, 47 m). The southbound lanes structure is considerably longer than the northbound lanes structure because neither fill or an embankment retaining wall at the south approach was permitted to encroach on the waterway. The layout of the bridges is given in Fig. 2.

Both bridges were designed for three 12 ft (3.66 m) lanes of traffic; two 3 ft 3 in. (0.99 m) shoulders; and 1 ft 8 in. (0.51 m) safety barriers giving an overall width of 45 ft 10 in. (13.97 m). However, to accommodate the on and off ramps from the adjoining interchange, the



Fig. 1. Aerial view of the completed bridges.

decks are flared towards the north abutments to a maximum of 48 ft 8½ in. (14.85 m) for the northbound lanes structure and 54 ft 4 in. (16.56 m) for the southbound lanes structure. Both bridges are on a slight grade towards the north.

Each bridge consists of a single cell box girder having a constant depth of 9 ft 3 in. (2.82 m) and a constant out to out web width of 25 ft (7.62 m). The cross section of the superstructure is given in Fig. 3. The extremely shallow construction depth (span to depth ratio of 27:1) was dictated by the roadway profile which was controlled by both the

nearby interchange and the restricted soffit elevation required for creek flow.

Two spherical rotational bearings with TFE sliding surfaces were provided at each abutment and pier with the exception of the bearings at Pier 1 for the northbound lanes structure and Pier 4 for the southbound lanes structure. Those bearings were fixed for translation.

A special design feature of the bridges is the transverse ribs located along both segment faces as shown in Fig. 4. The decision to use transverse ribs was primarily due to the need to flare the decks towards the north abutments

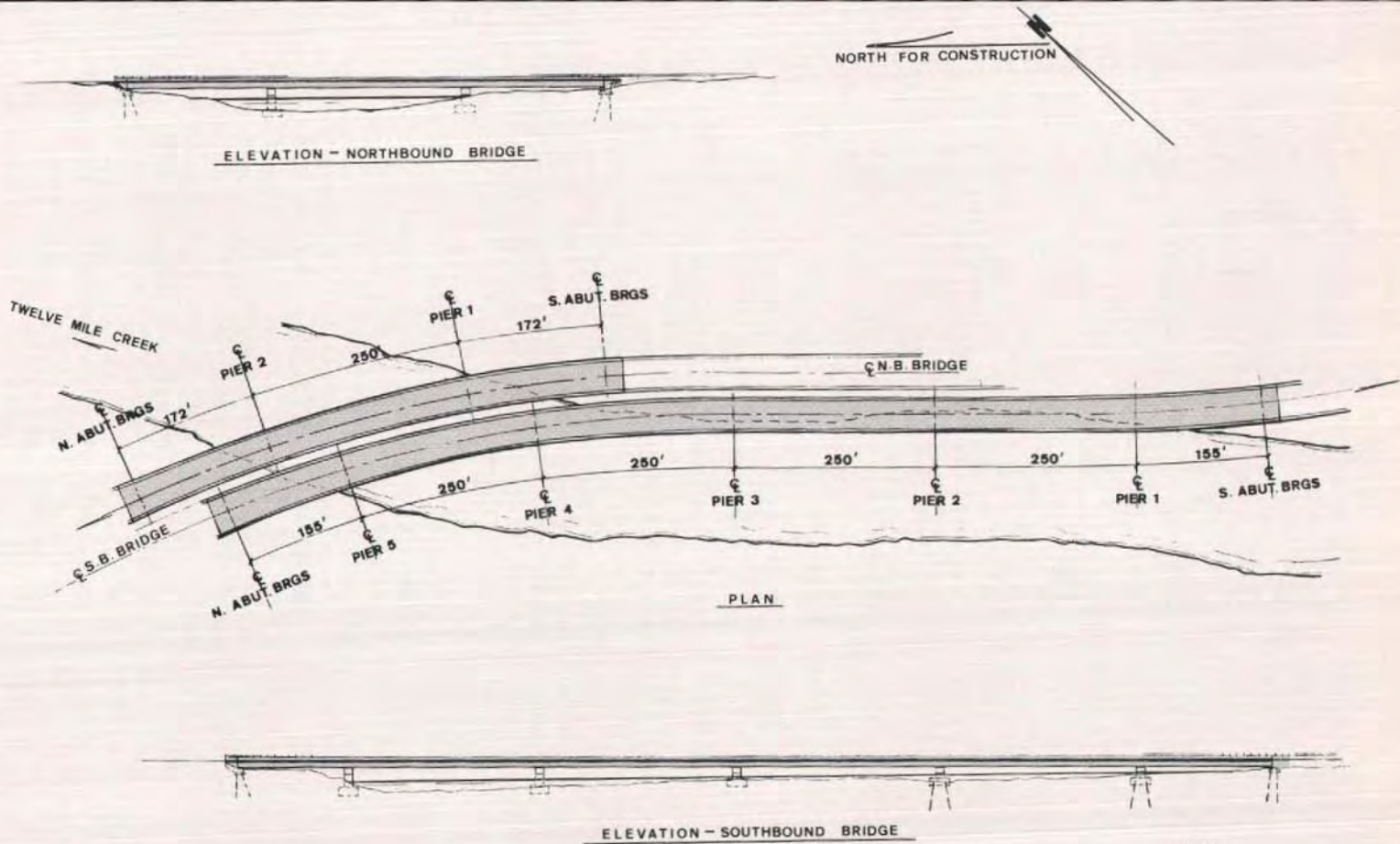


Fig. 2. General layout of bridges; plan and elevation.

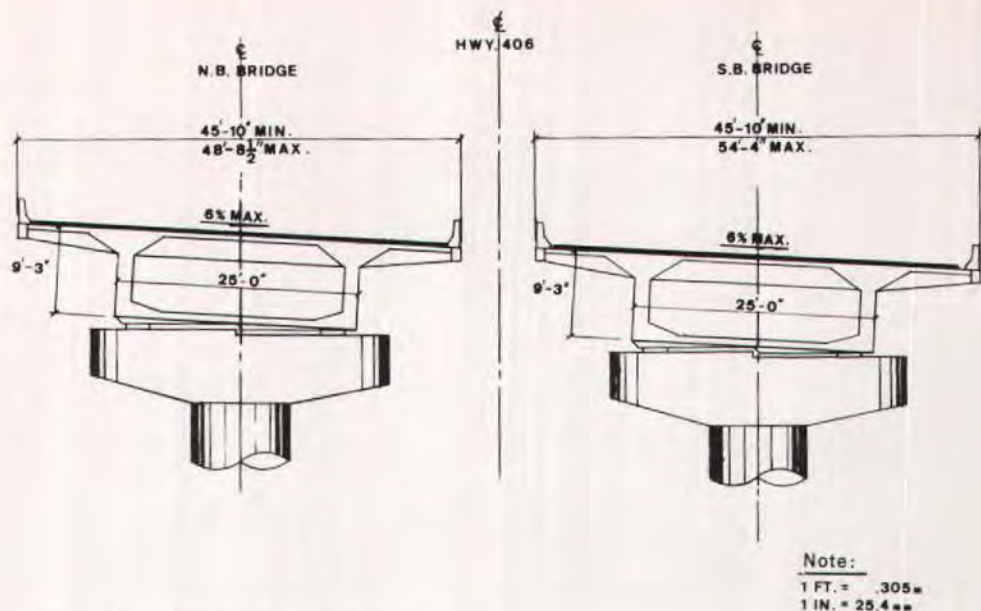


Fig. 3. Typical cross section of bridges.

where the normal deck cantilever overhang of 10 ft 5 in. (3.17 m) increases to a maximum of 14 ft 8 in. (4.47 m). The transverse post-tensioned ribs with a 7 in. (178 mm) thick top slab were extended to accommodate the deck flare and resulted in a 10 percent reduction in the segment weight. Without the ribs, the deck flare would have required a considerably thicker top slab.

The two bridges were constructed by the balanced cantilever method. The northbound lanes bridge has 74 precast segments and three cast-in-place closure segments. The southbound lanes bridge has 163 precast segments and six cast-in-place closure segments.

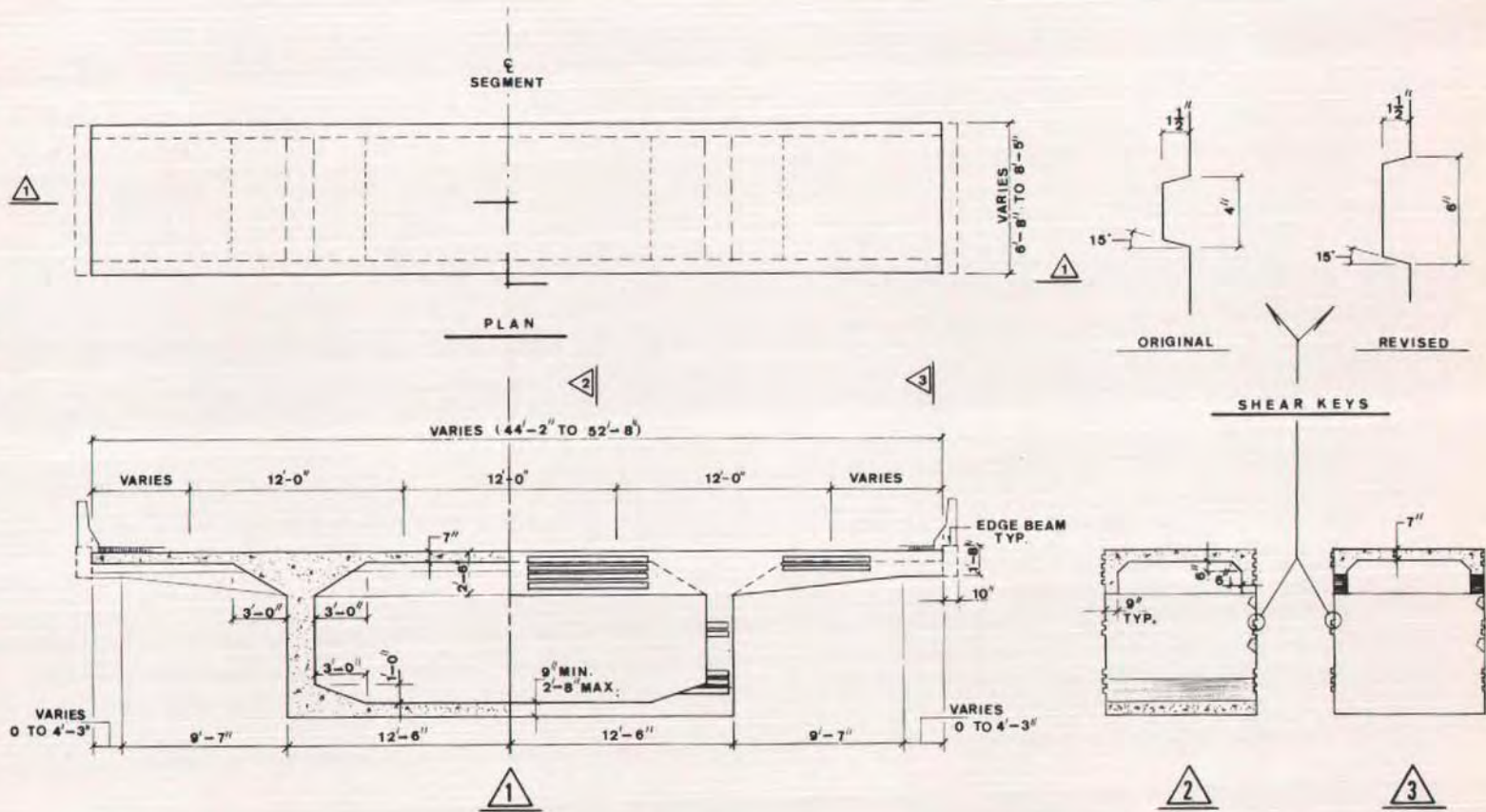
The pier and abutment segments are 6 ft 8 in. and 6 ft (2.03 and 1.83 m) long, respectively. The first segments adjacent to the pier segments are also 6 ft 8 in. (2.03 m) long. They are followed by four segments, each 7 ft (2.13 m) long. The thickness of the bottom slab for the five segments adjacent to the pier segment varies from 2 ft 4 in. to 9 in. (686 to

229 mm) and then remains constant for the remaining segments which are 8 ft 5 in. (2.57 m) long.

The abutment and pier segments include transversely post-tensioned solid diaphragms with access openings. The concrete in the diaphragms was placed after erection of these segments to keep the segment weights within the capacity of the launching truss.

Multiple shear keys without any reinforcement and spread over the segment faces were provided to resist the vertical shear forces during construction. They were chosen over a single large shear key due to the lack of space in the shallow webs where anchorages for the draped longitudinal tendons also were located.

Longitudinal cast-in-place fascia beams were provided at the edges of the deck to cover the transverse post-tensioning anchorages and to give the structure an aesthetically pleasing appearance. The fascia beams also help to distribute the wheel live loads.



NOTE:
 1 FT. = .305 m
 1 IN. = 25.4 mm

35 Fig. 4. Details of segment showing shear keys.

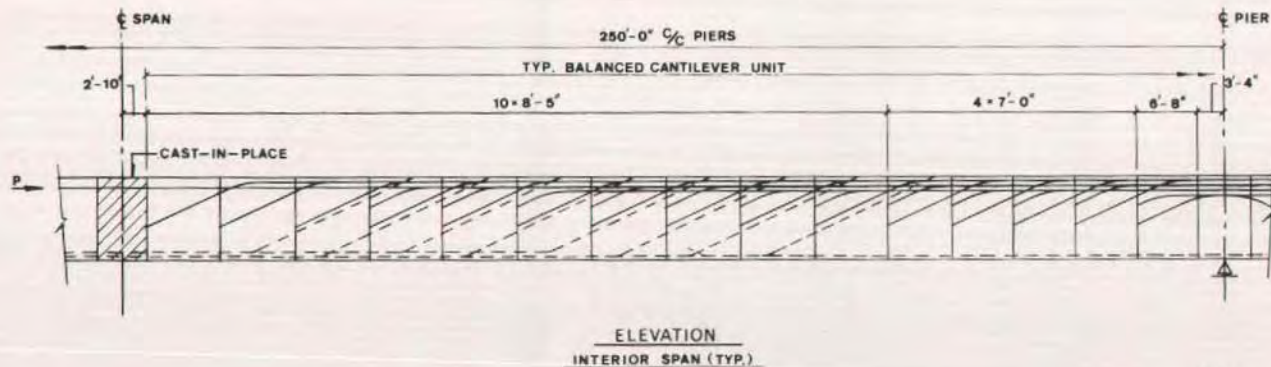
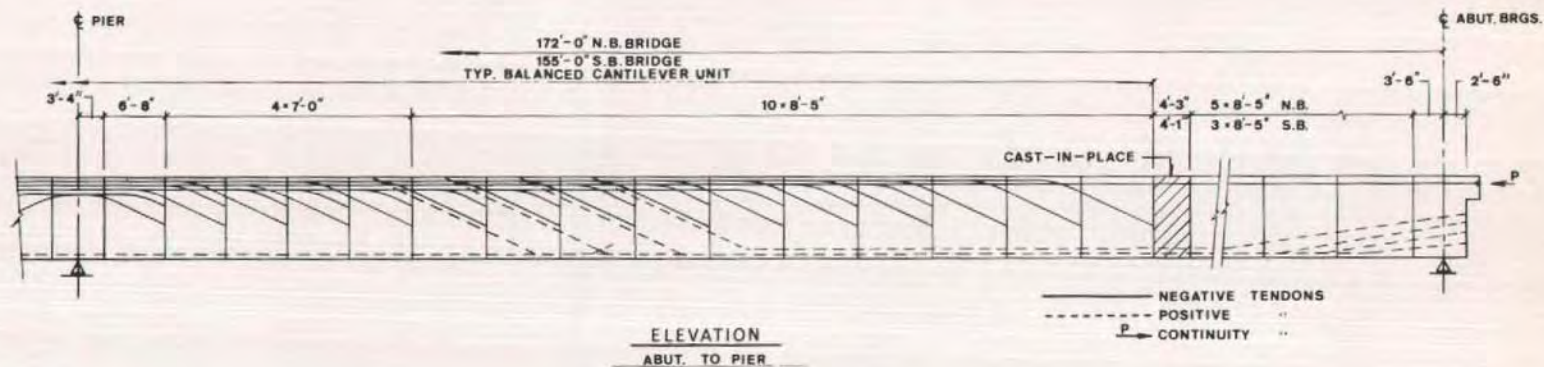


Fig. 5. Details of span segments showing location of post-tensioning.

Note:
 1 FT. = 0.305 m
 1 IN. = 25.4 mm

Design Considerations

The bridges were designed for all normal AASHTO¹ loads. The longitudinal analysis of the box girders was carried out using the BC computer program.² Three groups of tendons (19/0.6 strands) were used for longitudinal post-tensioning (Fig. 5). The function of the first group, where the tendons are draped and anchored in the webs, was to resist the negative moments during construction by the balanced cantilever method. The tendons for the second group, located in the girder bottom to resist the positive moments for the continuous structure, are draped and anchored in the deck slab. The third group of tendons, located in the top slab, were provided to resist negative moments along the span arising from live load in adjoining spans.

The transverse design was based on a finite element analysis and a modified version of the procedures given in the *Precast Segmental Box Girder Bridge Manual*.³ Transverse post-tensioning tendons (1/0.6 strands) were provided in the ribs as well as in the deck slab. The bottom slab of the box girder was reinforced with mild steel reinforcement.

The design for shear in the webs of the box girder was based on the *CEB-FIP Model Code for Concrete Structures*.⁴ A combination of vertical post-tensioning consisting of 1 in. (25 mm) diameter Dywidag bars, stressed from the top, and mild steel reinforcement was used in the first six segments on each side of the pier to resist shear. The remaining segments were provided with mild steel stirrups.

All piers consist of 10 ft (3.05 m) diameter round columns with post-tensioned pier caps. Circular single column piers were chosen to suit the skew crossing and to minimize any effect on the creek flow, a concern of the nearby upstream generating station. Piers 1 and 2 of the northbound lanes structure were aligned with Piers 4 and 5, respectively,

of the southbound lanes structure as they are located in the fast flowing water. The construction sequence of these piers was established from a scale model study of creek flow to minimize flow restriction and scour.

The piers are supported on either spread footings or footings with piles driven to bedrock. The abutment footings are supported on piles driven to bedrock.

The design strength of the concrete was 6000 psi (41.3 MPa) for the precast segments and 5000 psi (34.4 MPa) for the pier caps. No tension was allowed in the design of the prestressed concrete box girder and pier caps during construction and under AASHTO¹ loads.

Fabrication of Segments

The segments were precast in a temporary all-weather building near the bridge site. Two short-line casting beds located on the same base line, one to produce the standard 8 ft 5 in. (2.57 m) long segments and the other to produce nonstandard segments, were used. The survey tower to control the horizontal and vertical alignments of the segments was located between the two casting beds.

Each casting form consisted of a fixed vertical bulkhead, soffit, two wing forms, and a retractable mandrel carrying the expanding interior forms. Each segment was cast between the fixed bulkhead and its adjacent or counter-cast segment. Horizontal and vertical alignment was achieved by adjusting the orientation of the counter-cast segment prior to casting the new segment.

A tolerance of $\pm 1/16$ in. (1.59 mm), considered to be too fine by the contractor, was enforced both horizontally and vertically for the setting up of the counter-cast segment prior to placing concrete in the new segment. Due to the movements caused by the concreting operation, the resulting as-cast tolerance was $\pm 3/16$ in. (4.76 mm). Although it was time

consuming to meet the initial tolerance of $\pm 1/16$ in. (1.59 mm), the resulting bridge alignment was satisfactory.

The reinforcing steel cages, with the post-tensioning ducts in place, were prefabricated in a wooden mock-up of the segment before moving them into the casting beds.

The forms were stripped when the concrete strength reached 1500 psi (10.3 MPa). The counter-cast segment was moved into a storage area and the newly cast segment was then moved into the counter-cast position.

The segments were vertically and transversely post-tensioned and grouted in the storage area after the concrete reached a strength of 5000 psi (34.4 MPa).

An average of 1.5 segments per day were produced over a period of about 10 months. The maximum segment weight, excluding the cast-in-place diaphragms, was 72 tons (65 t).

Erection of Segments

The abutment segments and the adjacent segments (up to the first cast-in-place closure segment) were erected on falsework using a crawler crane. The entire balanced cantilever over Pier 1 of the southbound lanes structure, which is situated over the creek shoreline, was erected by a crawler crane using a movable temporary support and stabilizing concrete blocks.

The remaining segments were erected by a launching truss. The steel launching truss which had previously been used on two other straight segmental bridges in Ontario had a length of 377 ft (114.9 m) and an approximate weight of 463 tons (420 t).

Major modifications were required to enable the truss to travel and erect segments on the horizontally curved bridges. The launching truss stabilized the balanced cantilevers, delivered segments, and supported the work platforms and post-tensioning hardware.

Initially, two segments were erected during a normal working day. This increased to four segments per day by the time the northbound lanes structure was completed.

The temporary stressing system used to clamp the segments onto the growing cantilevers, until permanent tendons could be installed, was a combination of strand and high strength bar tendons located in either existing ducts or in ducts especially provided for this purpose. Due to the presence of the transverse ribs, the large contact area at the segment joints was provided with additional short prestressing bars to provide the required 30 to 50 psi (0.21 to 0.34 MPa) temporary contact pressure.

Epoxy adhesive was used to bond the segments during erection. Two formulations were used: one for faster setting during cool weather and the other for setting during warmer weather. A diligent inspection, testing, and curing procedure in accordance with AASHTO specifications¹ was followed to ensure success.

After completion of cantilevers in each span and removal of all loads from the spans, stabilizing clamping beams were used to connect the two cantilever tips or the cantilever tip in the end spans to segments resting on temporary falsework. The concrete in the closure segments was placed during periods of stable ambient temperature, which was usually around midnight. When the concrete in the closure segment reached a strength of 1000 psi (6.9 MPa), four of the tendons were stressed. The remaining tendons were stressed when the concrete strength reached 4000 psi (27.6 MPa).

Problems and Solutions

During precasting of the segments and erection of the superstructure several problems were encountered. Discussed below are the difficulties and their method of solution.

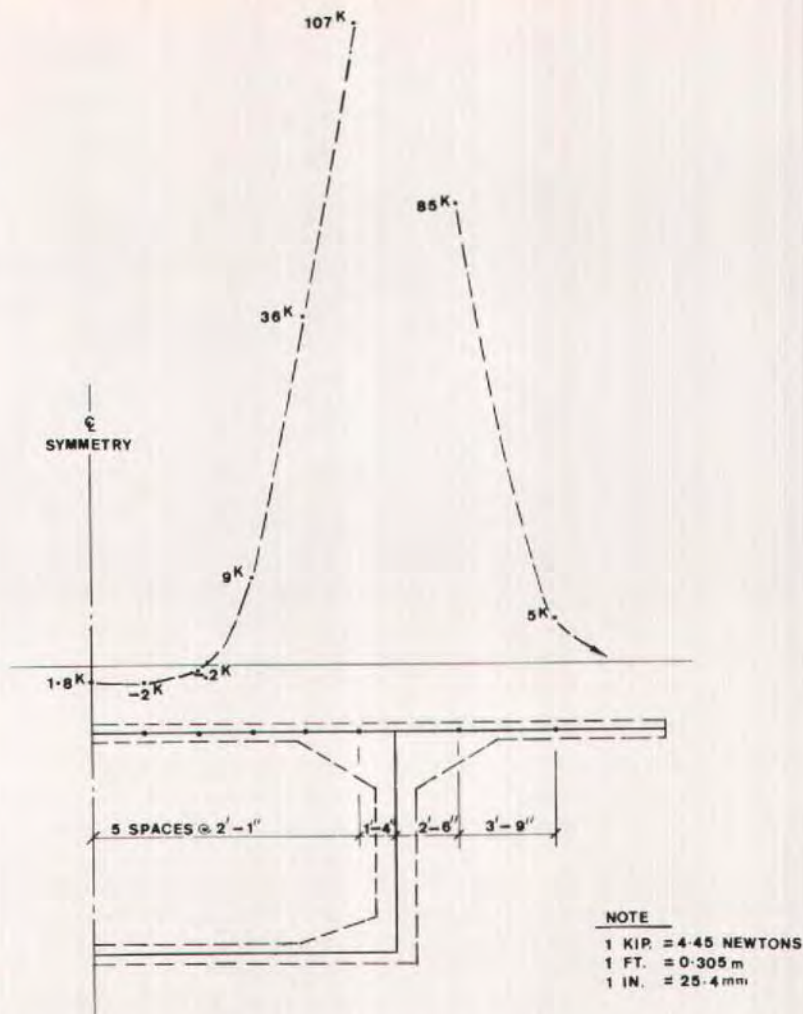


Fig. 6. Shear distribution due to segment weight and construction load.

Precasting

During the separation of the counter-cast segments from the new segments, some of the shear keys were damaged. This was due to the method of separation employed which caused the segments to slightly rotate relative to one another, resulting in unanticipated forces on the shear keys when the concrete strength was only 1500 psi (10.3 MPa). This problem was resolved (but

not until about 20 percent of the segments had been cast) by adjusting the method of separation, increasing the depth of shear keys from 4 to 6 in. (102 to 152 mm), and careful application of the bond breaker.

The damage to the shear keys necessitated a reassessment of the capacity of the remaining undamaged shear keys. Tests were carried out on a number of concrete blocks with 4 in. (102 mm) deep shear keys. A minimum shear



Fig. 7. Completed northbound bridge with southbound bridge under construction. Note mislocated Pier 4 with Pier 5 in the foreground.

capacity of $2.5 \sqrt{f'_c}$ psi ($0.2 \sqrt{f'_c}$ MPa) was established from these tests and was used in assessing the shear capacity of the undamaged shear keys.

Most of the damaged shear keys were in the webs. A finite element analysis was used to check the shear distribution at the remaining keys located in the top and bottom slabs and is shown in Fig. 6. The analysis confirmed that only about 20 percent of the shear force was transferred through these keys. It was, therefore, decided to hold the segments with the damaged keys in place until the epoxy cured since the keys could not be relied upon to transfer shear forces across the joint. This resulted in a slow-down of segment erection.

The fixed bulkhead had a 5 in. (127 mm) protruding ledge which formed part of the bottom pallet for the segment forms. During separation, large pieces of concrete often were broken off by the ledge. These breaks were subsequently repaired but caused fit-up problems during erection and led to leakage dur-

ing grouting of the tendons.

The longitudinal tendon ducts were displaced between the segment faces during concrete placement even though they were well fixed and inflatable rubber hoses were inserted into the ducts. This resulted in excessive friction during post-tensioning.

The transverse tendons in two segments were accidentally grouted before they were stressed. In the process of removing these segments from the storage area, the boom of the lifting crane collapsed and severely damaged another two segments. Since these four segments were to be located near the tip of one of the cantilevers, it was only necessary to recast these four segments.

The original design called for the transverse reinforcing bars crossing the anchorage recesses for longitudinal post-tensioning in the top slab to extend into the recesses and be spliced by field welding after longitudinal post-tensioning. However, for construction it was decided to place prebent bars through

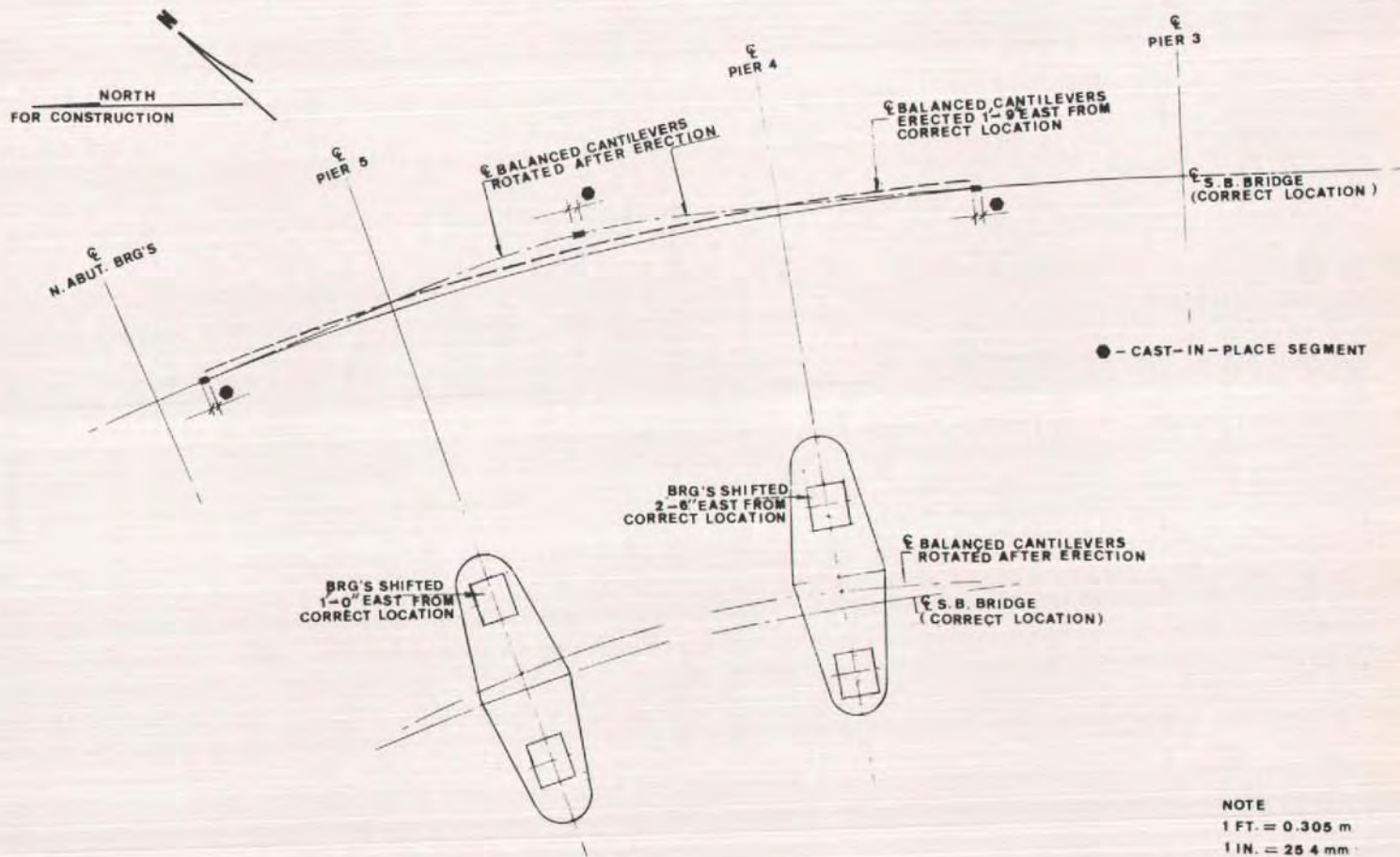
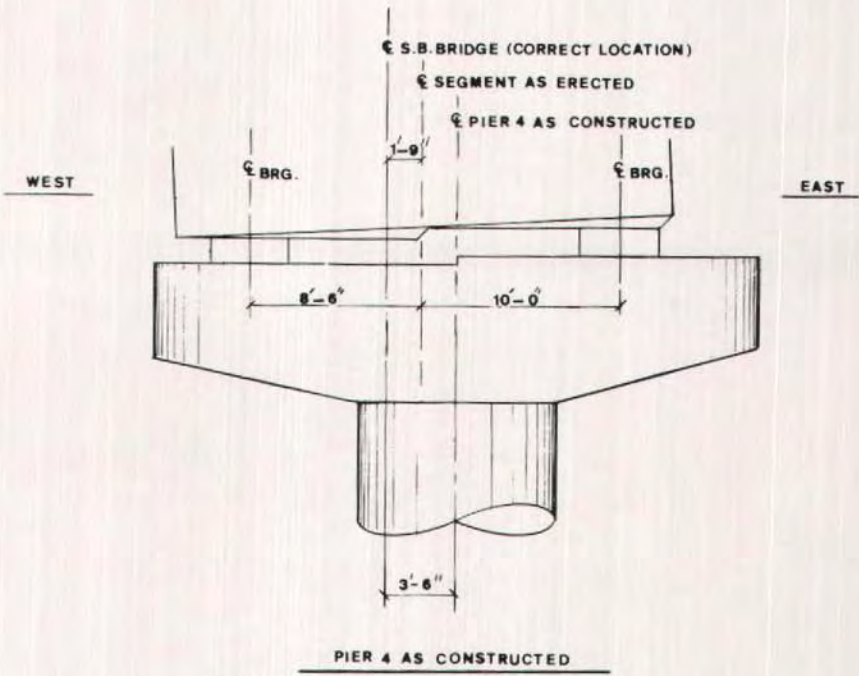
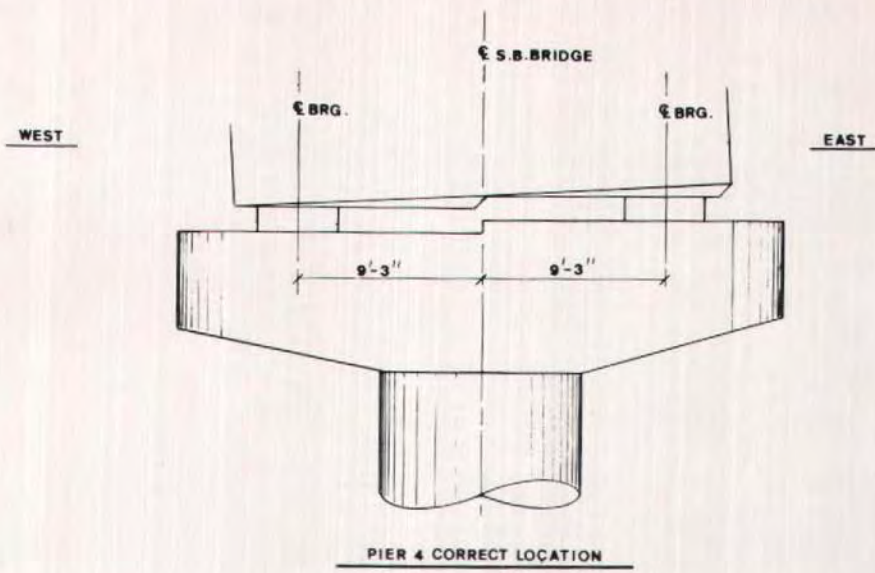


Fig. 8. Alignment adjustment for mislocated Pier 4.



NOTE
 1 FT. = 0.305 m
 1 IN. = 25.4 mm

Fig. 9. Pier segment and bearings at mislocated Pier 4.



Fig. 10. Typical pier segment with cast-in-place diaphragm and erection of adjacent segment.

the recess formers and bend them down to form a lap splice after longitudinal post-tensioning. This caused the bent-up bars to interfere with construction and many of them were broken off. In retrospect, the original design detail would have been better.

Erection

As noted earlier, the longitudinal tendon ducts had displaced between the segment faces during precasting and the friction loss was higher than anticipated. Since low-relaxation strands were used on this project, it was decided to increase the jacking force from $0.08 f'_s$ to $0.85 f'_s$ for subsequent tendons to compensate for the higher friction loss.

Due to sharp kinks at some segment

joints, some strand wires did break. Where it was not possible to increase the jacking force for subsequent tendons, especially near the ends of the cantilevers, the temporary prestressing tendons connecting the last few segments were left in place and grouted.

Provision should be made in all segmental bridge designs to provide additional prestressing to compensate for any excessive friction or for future strengthening.

Due to the breakage and patching of the concrete in the bottom slab at the segment joint during segment separation from the bulkhead and possibly other reasons, numerous interconnections and leaks were discovered during the process of water pressure testing the ducts prior to grouting. The original de-



Fig. 11. Delivering segment on completed cantilever for erection by launching truss.

sign called for the negative moment tendons in the cantilevers to be grouted after the completion of each balanced cantilever. However, due to the possibility of duct interconnection, the grouting was delayed and the tendons were protected against corrosion by coating them with a water soluble oil prior to installation.

The northbound lanes structure was grouted in one stage after completion of superstructure erection. The southbound lanes structure was grouted in two stages due to the approaching winter. The grouting for the first stage extended to the vicinity of Pier 3. The grouting for the second stage, which included the remaining tendons, was carried out in December during cold weather. The structure was heated from inside the box to prevent freezing of the grout. The required heat exceeded the temperature differential between the interior and exterior of the box girder assumed in design and fine cracks de-

veloped in some segments on each side of Piers 4 and 5.

Where heating for grouting purposes is likely to be required, the design should allow for the anticipated thermal gradient.

The vertical alignment of the cantilevers was controlled by insertion of stainless steel wire mesh between the segment faces and the resulting cantilever tip elevation was within 1 in. (25 mm) of the theoretical value. At one closure segment, the clamping beams between the cantilever tips moved during the concreting operation, resulting in a step of about 2 in. (51 mm) across the closure segment.

The alignment control pins in top of the deck along the segment centerline were set about 2 in. (51 mm) from the segment faces. Since the horizontal alignment computations were based on the actual segment lengths, rather than the pin to pin lengths, the erected curved cantilever tips curled inwards.



Fig. 12. Underside view of completed bridges showing the transverse ribs.

The maximum horizontal alignment deviation at the forward tip (with the rear tip in the correct position) was 6 in. (152 mm). This error was split between two closure segments by rotating the balanced cantilever in plan and the deviation is unnoticeable.

A serious problem on this project was the discovery of a survey error, which had located Pier 4 of the southbound lanes structure about 3 ft 6 in. (1.07 m) east of its correct location, as the cantilevers at Pier 3 were nearing completion (Fig. 7). Since the alignment of the bridge was curved, the problem could be resolved by altering the horizontal alignment of the bridge between the forward cantilever at Pier 3 and the north abutment. By allowing some tension in the top of the post-tensioned pier caps and reassessing the pier column and footing design, it was possible to shift the pier segment at Pier 4 eastward from its theoretically correct location by 1 ft 9 in. (0.63 m) with a further easterly

bearing shift of 9 in. (0.32 m) relative to the segment and completing the balanced cantilevers as usual, as shown in Figs. 8 and 9.

The completed cantilever at Pier 4 was then rotated to line up with the previously completed cantilever at Pier 3 and the closure segment was concreted. The pier segment on the correctly located Pier 5 was also shifted eastward by 1 ft 9 in. (0.53 m) from its correct location with an easterly bearings shift of 1 ft (0.30 m). The completed cantilever at this pier was then rotated to bring the forward cantilever tip to its correct position. On inspection, the shift in the curved alignment is not noticeable.

It was fortunate that the bridge was on a curved alignment and some adjustments could be taken up by the piers and the cast-in-place closure segments. With any other type of construction, it probably would have been necessary to demolish Pier 4 and reconstruct it in its correct location, resulting in increased



Fig. 13. View of completed bridge.

costs and considerable delay in completing the bridges.

Figs. 10 through 13 show various phases of the bridges during construction and after completion.

Closing Comments

The Twelve Mile Creek Bridges were the first precast segmental concrete bridges to be constructed by the Ontario Ministry of Transportation and Communications. The restricted construction depth and the curved and twisted alignment combined with flared decks near the north abutments, provided a challenge that was satisfactorily met by using precast prestressed concrete segmental construction.

The experience gained and the les-

sons learned from the design and construction of these bridges have enhanced the future of this type of construction in Ontario.

The total cost of the bridges (in Canadian dollars) was \$8,970,000 or \$100/ft² (\$1076/m²). The cost of the superstructure was \$88/ft² (\$947/m²).

In 1984 the Twelve Mile Creek Bridges received national awards from both the Prestressed Concrete Institute and the Post-Tensioning Institute.

The bridges were opened to traffic in October 1984 on completion of Highway 406 to the Queen Elizabeth Way.

Credits

Owner: Ministry of Transportation and Communications, Ontario.

Engineer: Structural Office, Ministry of Transportation and Communications, Ontario.

Special Consultant to Owner: Preconsult Canada.

General Contractor and Manufacturer of Precast Segments: Kilmer Van Nostrand Company Ltd., Downsview, Ontario.

Special Consultant to Contractor: DRC Consultants, New York City, New York.

Post-Tensioning Supplier: Canadian BBR (1980) Inc., Toronto, Ontario.

References

1. *Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, Washington, D.C., 1977.
2. Bridge Construction (BC) Computer Program, Europe Gecti Etudes, France.
3. *Precast Segmental Box Girder Bridge Manual*, Published jointly by the Prestressed Concrete Institute, Chicago, Illinois, and the Post-Tensioning Institute, Phoenix, Arizona, 1978.
4. *CEB-FIP Model Code for Concrete Structures*, Comité Euro-International du Béton, Paris, 1978.

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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by July 1, 1985.