# HIGHWAY 407 / BRONTE CREEK CROSSING
# AN INCREMENTALLY LAUNCHED PRECAST CONCRETE GIRDER DESIGN

Mr. James C. Sherlock, P. Eng., McCormick Rankin Corporation, Canada
Mr. John. Otter, Otter Brown Engineering Limited, Canada

## ABSTRACT

*The western extension of Highway 407 in Ontario, Canada, from Mississauga to Hamilton required a major crossing of the environmentally sensitive Bronte Creek Valley. Commitments made to minimize the impacts of construction on the valley and creek led to the conclusion that a bridge structure, launched from the top of the valley, was the optimal solution. This paper addresses some of the key components of interest during design and construction of the resulting incrementally launched, precast girder design.*

**Keywords:** Precast, Spliced, Launch, Construction, Erection, Incremental.

## INTRODUCTION

In 1993, the planned extension of Highway 403, in Ontario, from Mississauga to Hamilton required a major crossing of the environmentally sensitive Bronte Creek Valley. At the crossing location, the Bronte Creek Valley is characterized by very steep slopes, such that access into the valley is not feasible without extensive environmental impacts. Commitments made to minimize the impacts on the valley and creek led to the conclusion that a bridge structure, launched from the top of the valley, was the optimal solution.

Preliminary studies were completed for both steel and concrete bridges, ultimately leading to the completion of contract drawings for both, a launched steel box girder bridge and a launched, precast concrete girder bridge. However, at this point, the Province of Ontario started investigating the concept of a toll highway across the top of Toronto that would eventually pass through the Highway 403 site, and the contract drawings were shelved.

In 1999, the Province of Ontario decided to extend the Highway 407 Toll Road westerly, and the planned extension of Highway 403 became the westerly extension of Highway 407. A design-build contract was awarded to SLF Joint Venture. The Bronte Creek crossing was recognized as a key component of the work, since it was required to provide access for the construction of the remainder of the highway. Because of geometric changes in the highway, however, a complete redesign was required.

Preliminary studies were undertaken for both precast concrete and structural steel alternatives. Preliminary quantities were developed for costing by the design-build contractor, with the result that the precast concrete spliced-girder alternative was determined to be the more economical structure.

This paper addresses some of the key issues of interest regarding the design and construction of the resulting incrementally launched, precast concrete spliced-girder bridge.

## DESCRIPTION OF THE BRIDGE

The bridge crossing actually consists of two separate structures, supported on common abutments, but independent piers. The span arrangements of the two structures is controlled by the alignment of the Bronte Creek, which flows diagonally to the highway, resulting in non-symmetrical span arrangements for the eastbound and westbound structures. To simplify construction, however, the same 35m – 60m – 55m span arrangement was chosen for both structures, and simply reversed for the eastbound and westbound bridges, respectively. The heights of the bridges rise about 23m above the valley floor. For a general view of the twin structures, refer to Figure 1.

The cross-sections of the twin bridges each consist of a 225mm thick concrete deck slab supported on seven, precast concrete girders. The center to center girder spacing for both structures is 3.1 m. Each bridge deck carries four lanes of traffic (eastbound and westbound Highway 407, respectively) and is approximately 21 m in width (see Figure 2).

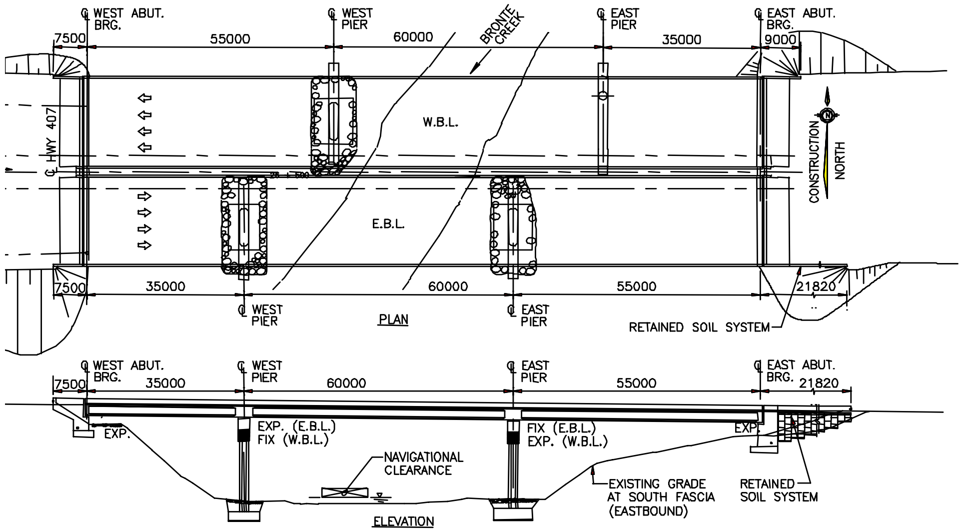



Fig. 1

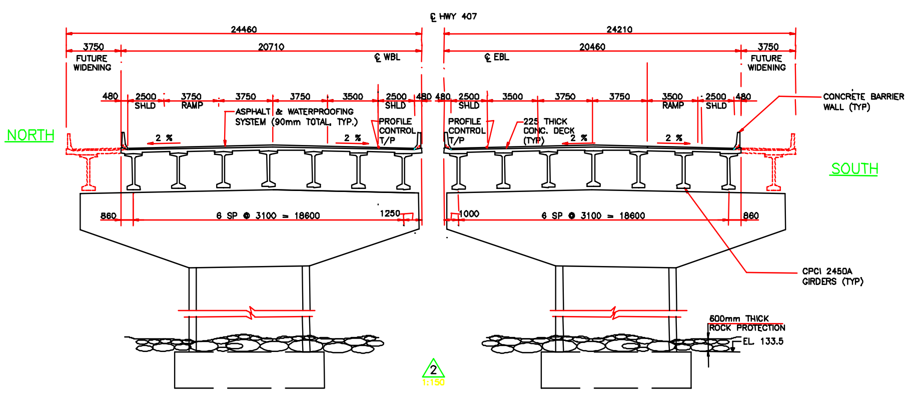



Fig. 2

## DESIGN ISSUES

The design of the twin bridges was undertaken in accordance with the Ontario Highway Bridge Design Code (3rd Edition). A discussion of some of the key design issues follows.

GIRDER FABRICATION/POST-TENSIONING SEQUENCE

The girders were fabricated in the Pre-Con Company precast plant in Brampton, Ontario. The 60m and 55m span girders were too heavy to be transported on municipal/provincial roads and therefore each girder had to be fabricated and shipped to the site in three segments (see Figure 3). The 35 m girder segment was just within the permissible weight restrictions and as a result was fabricated and shipped as a complete unit.

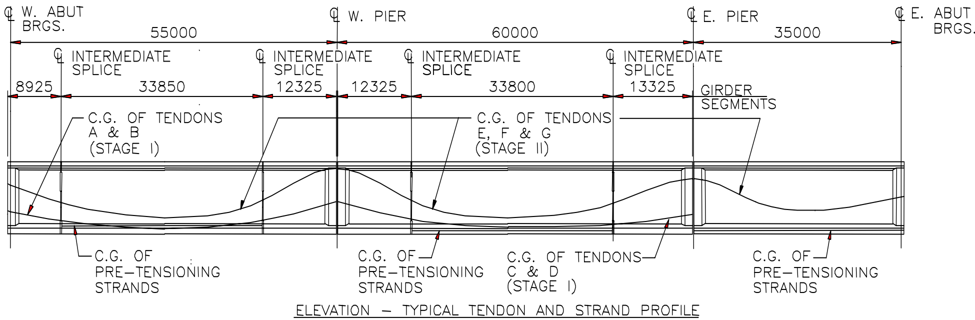



Fig. 3

The centre segments for the 55m and 60m girders were pre-tensioned for self-weight alone while the shorter, end segments were only reinforced with normal reinforcing steel. The girders were fabricated with five, draped post-tensioning ducts, two of which would be used to splice the individual segments of the girders into complete girders (Stage I Post-Tensioning) and three of which would be used to splice the individual spans into one continuous structure (Stage II Post-Tensioning). The 35m span girders were pretensioned both for self-weight and the slab dead load. Accordingly, these girders were fabricated with only the three, draped post-tensioning ducts required for Stage II Post-Tensioning. In order to keep the girder webs clear for the draped post-tensioning ducts, prestressing strand was located in the bottom flanges of the girders and away from the web of the girder. Consequently, the pretensioned strands were all straight.

Once on site, the individual segments of the 55m and 60 m spans were placed on concrete pedestals, and aligned. The post-tensioning ducts of the adjacent segments were then coupled and the girders were spliced with cast-in-place, in-fill closure pieces. Each assembled girder was subsequently post-tensioned (Stage I Stressing) for the assembled girder dead load and for the slab dead load, and the ducts were grouted. All of this was carried out in an assembly area located approximately 200m to the west of the bridge site.

Once assembled, the individual girders were transported along a girder delivery system to the structure and along a launching truss running between the twin bridges to their respective span locations. The girders were transported laterally along another delivery system from the launching truss to their final resting positions on the piers and abutments. Once in position, each girder was transferred onto its permanent bearings and steel angle cross frames at third points along the girder spans were install for stability.

It is a requirement of the Ministry of Transportation of Ontario that all bridges be constructed in a manner that permits future deck slab replacement. Consequently, the continuity (Stage II) post-tensioning was carried out prior to the placement of the deck slab. Accordingly, once the girders were in-place, the ducts for the through cables running the full length of the structure were then coupled at the piers. Support diaphragms were cast at the piers enclosing the ends of the girders in adjacent spans. The fully continuous girders were then post-tensioned (Stage II Stressing) with cables running full length of the structure. The ducts were grouted and the deck slab was subsequently cast.

The Bronte Creek bridges were designed utilizing a traditional, cast-in-place deck slab design. Given the difficult access conditions and schedule constraints, however, the Contractor decided to use normally reinforced precast concrete deck panels, with a reinforced cast-in-place top surface. The total thickness of the panels and cast-in-place topping matched the original design 225mm cast-in-place slab (90mm thick deck slab plus 135mm thick cast-in-place topping). Because the Ontario Highway Bridge Design Code does not address precast panel design, the panels were designed in accordance with the requirements of the, draft version of the new Canadian Highway Bridge Design Code (the Canadian Highway Bridge Design Code has since been adopted in Ontario and now replaces the Ontario Bridge Design Code).

## PRECAST CONCRETE GIRDER

The original 1993 design was based on a modified version of the largest commercially available precast girder of its day. In order to stretch the capacity of the 2.35m deep girder to reach the 60 m maximum span in this structure, a very complex, multi-stage post-tensioning sequence was required, including supplemental post-tensioning at the piers (short, local cables over the pier that protruded through the top flange of the girders with anchorage blisters), and stressing and de-stressing of external Dwidag threaded rods near the bottom flange at the pier (used to control the extreme fibre stresses during the post-tensioning sequence).

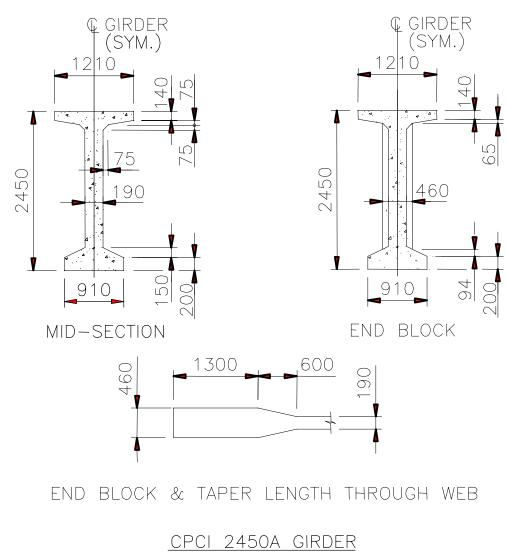



Fig. 4

The new design completed in 1999, employed the new CPCI 2450A girder (Figure 4). The benefits realized by replacing the original girder, with the new girder included reduced post-tensioning requirements, and a greatly simplified post-tensioning sequence, resulting in significant cost savings. This project is only the second application for these girders in Ontario (the other application was for the Perly Bridge replacement over the Ottawa River, between Hawkesbury, Ontario and Grenville, Quebec, where the girders were combined with haunched, precast sections at the piers to achieve a maximum span of 68 metres). These girders permitted the structure to be built with the longest precast girder span for a constant depth section, in Ontario. The center-to-center girder spacing was 3.1 m. The girders were constructed with 50 MPa concrete. The required at-transfer concrete strength was 30 MPa.

## STRESSING SUMMARY

The following table summarizes the prestressing and post-tensioning requirements for the individual girders:

| Girders | 55m | | | 60m | | | 35m |
|---|---|---|---|---|---|---|---|
| Segments | 8.925m | 33.85m | 12.33m | 12.325m | 33.8m | 13.325m | - |
| Pre-Tension[1] | - | 14 | - | - | 18 | - | 14 |
| | - | 1383 | - | - | 1778 | - | 1383 |
| Post-Tension (Stage I)[2] | 2–12/15 | 2–12/15 | 2–12/15 | 2–12/15 | 2–12/15 | 2–12/15 | - |
| | 3360 | 3360 | 3360 | 3360 | 3360 | 3360 | - |
| Post-Tension (Stage II)[2] | 3–12/15 | 3–12/15 | 3–12/15 | 3–12/15 | 3–12/15 | 3–12/15 | 3–12/15 |
| | 5040 | 5040 | 5040 | 5040 | 5040 | 5040 | 5040 |

[1]Prestressing strand is low-relaxation seven wire strand, size designation 13 (0.5”), grade 1860MPa. No. of strand and total strand area given.

[2]Post-tensioning strand is low-relaxation seven wire strand, size designation 15 (0.6”), grade 1860MPa. No. of cables, cable designation, and total cable area given.

Table 1 – Prestress and Post-Tensioning Requirements

## GIRDER SPLICE DETAILS

The structure employed two different types of girder splices, the intermediate girder splices connecting together the girder segments within the 55m and 60m spans, and the splices over the piers which provide continuity between spans. A description of each follows:

*Intermediate Splices* - In order to minimize the flexural stress on the spliced section, the splices were located close to the inflection points within each span. Rather than using a match cast approach to splicing the girders, splicing was achieved by casting a 150 mm wide, keyed, concrete in-fill section between the girder segments (Figure 5). This approach offered the advantage of ensuring a perfect fit between girder segments regardless of the deformation of the individual segments under dead load and the initial prestress. The splice concrete consisted of a superplasticized High Performance (50 MPa) Concrete Mix, with 10mm aggregate. The splices were reinforced with 15M hairpin bars embedded in the webs and bottom flanges of the girders. The bars were offset 15mm from the centerline of the girder to facilitate lapping of the bars when the girder segments were placed end to end. After Stage I Post-Tensioning, four 30M bars were spliced (welded to an overlapping angle) within a blockout in the top flange.

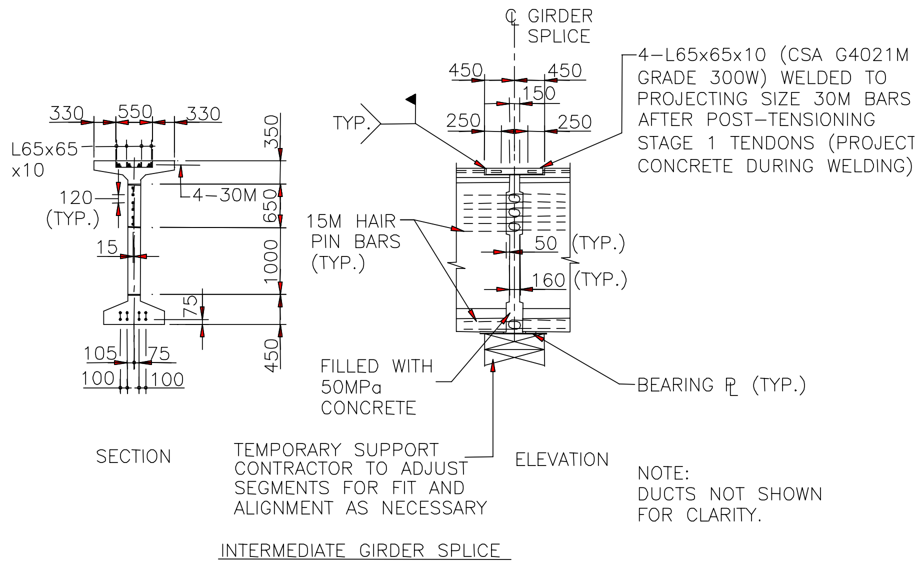



Fig. 5

*Pier splices* -The splices between the ends of the girders at the piers were carried out after the girders were erected in-place. The concrete diaphragms along the support lines provided the infill concrete between the ends of the girders. Positive moment reinforcement was provided in the bottom flange primarily to control cracking in the bottom flange during stressing of the continuous (Stage II) continuity post-tensioning. A sectional analysis for a cracked section was carried out to determine the stresses in the concrete and the required positive moment reinforcement. The resulting crack widths were within code limitations, and any cracks that formed were subsequently closed up by the addition of the slab dead load.

## GIRDER ERECTION

### SPECIALIZED CONSTRUCTION ENGINEERING

The incremental launching of the girders had a dramatic effect on the complexity of the erection process. The equipment required to carry out the work was almost exclusively purpose-built equipment and the logistics of coordinating the girder assembly, the girder delivery systems to the bridge, the launching equipment across the bridge, and the girder placement required a high degree of planning. Some of the key components that required specialized construction engineering were as follows:

- The girder assembly yard
- Three separate carriage delivery systems to transport the girders from the assembly yard to their final resting positions on the piers and abutments. These included:
    - Side-shifter carriages in the assembly yard to transport the girders laterally from the position in the yard where they were assembled to the longitudinal delivery system leading to the bridge.

    - Transporter carriages to carry the girders along the longitudinal delivery system from the assembly yard to the bridge and across the launching truss.
    - A second, different, side-shifter carriage system to transport the girders laterally from the launching truss (located in the median between the eastbound and westbound structures), across the tops of the piers and abutments, to their final resting positions on the piers and abutments
- Five individual launching trusses (three unique span lengths, 35m, 20m and 40m) to span between the piers and abutments
- Various stabilizing measures required to prevent the girders from tipping over under construction wind loads, during assembly, during transportation on three different carriage systems, and, once erected, prior the deck slab being cast.

## GIRDER ASSEMBLY YARD

The fact that the 55m and 60m girders were fabricated in multiple segments and had to be spliced together on site led to the establishment of a large assembly area to the west of the bridge site. In order to maintain a reasonable girder assembly speed, it was decided to create a large enough working area to assemble two spans of girders simultaneously (i.e. 2 bays of 7 girders). This allowed assembly and pouring of splices to take place on the girders for one span while previously spliced girders were having their splices cured and Stage I post-tensioning carried out.

The assembly area for each bay of girders consisted of four concrete sleeper slabs to support the ends of the three girder segments in each spans (i.e. for the 55m and 60m spans). The two interior sleeper slabs in each bay (at the girder splice locations between the girder segments) supported the adjacent ends of the girder segments to be spliced, on a single raised concrete pedestal. The two sleeper slabs at each end of the bay not only supported the ends of the exterior girder segments on raised concrete pedestals, but also provided a running surface for the side-shifter carriages that would carry the girders transversely from their assembly position in the yard to the longitudinal delivery system that would transport the girders to the structure.

In order to provide a traversable slope for the longitudinal girder delivery system from grade at the assembly area to the elevation at the bridge from which the girders would be erected (i.e. the top of the abutment bearing seat), the assembly yard had to be set back about 200 m from the abutment. The resulting longitudinal grade on the delivery system was about 3%.

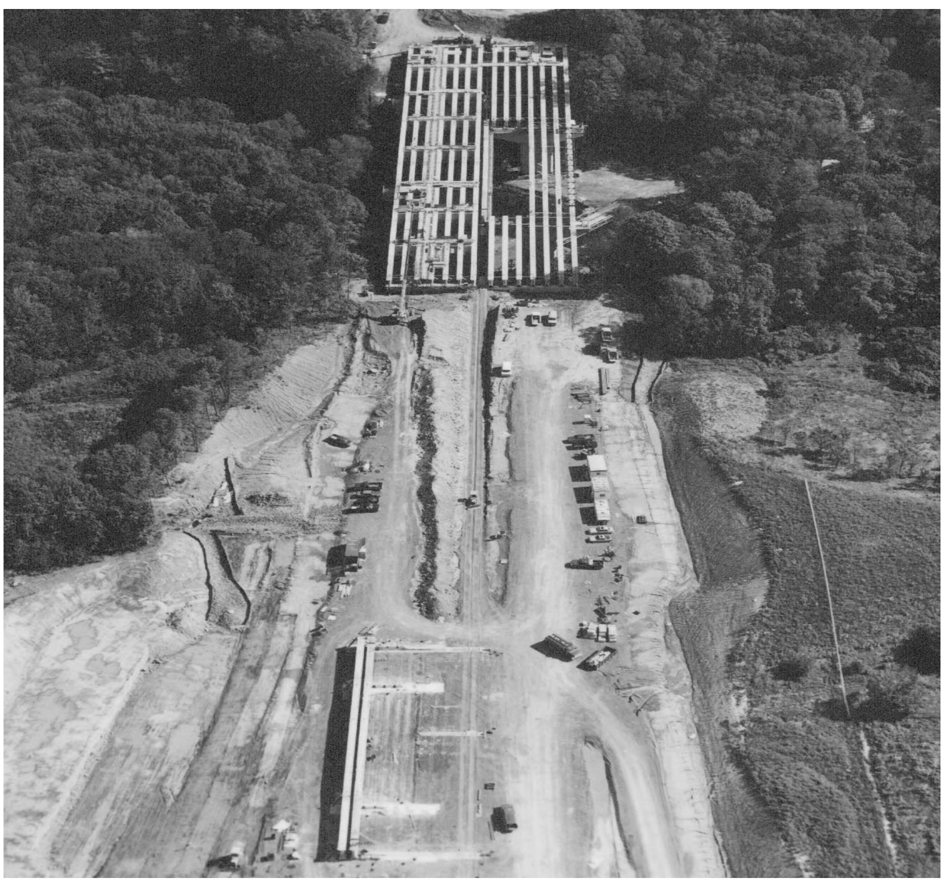
## INCREMENTAL LAUNCHING OF GIRDERS

The erection/launching sequence was as follows (work in the area of the bridge is shown schematically in Figure 6; side-shifter carriages and longitudinal transporters are shown in Figures 7 to 9):

1. Deliver the girder segments to the yard and unload by conventional crane.

2. Assemble individual girder segments into full-length girders, cast and cure cast-in-place intermediate field splices; and, stress Stage I post-tensioning.
3. Shift the assembled girders laterally across the yard to the delivery rail on side-shifter carriages.
4. Transfer the girders from the side-shifting carriages to the longitudinal transporters that run on the delivery rails.
5. Push the girders down the delivery rails to the bridge using a small bulldozer.
6. Connect a winch from the far side of the bridge to the longitudinal transporters to winch the girder along the suspended launching truss to the girder's respective span location.
7. Transfer the load of the girder from the longitudinal transporters to side-shifter carriages running across the tops of the piers and abutments.
8. Pull girder on the pier/abutment side-shifters laterally across the piers and abutments to its final position.
9. Lower girder onto its bearings and temporarily brace.

The following photograph (Photograph 1) shows an aerial view of the site. Only the remaining three girders in the center span of the eastbound lane structure remain to be erected (east is up in the picture). Only half of the assembly area is visible at the bottom of the picture (the second bay of girders is just out of the picture). The delivery rail system runs



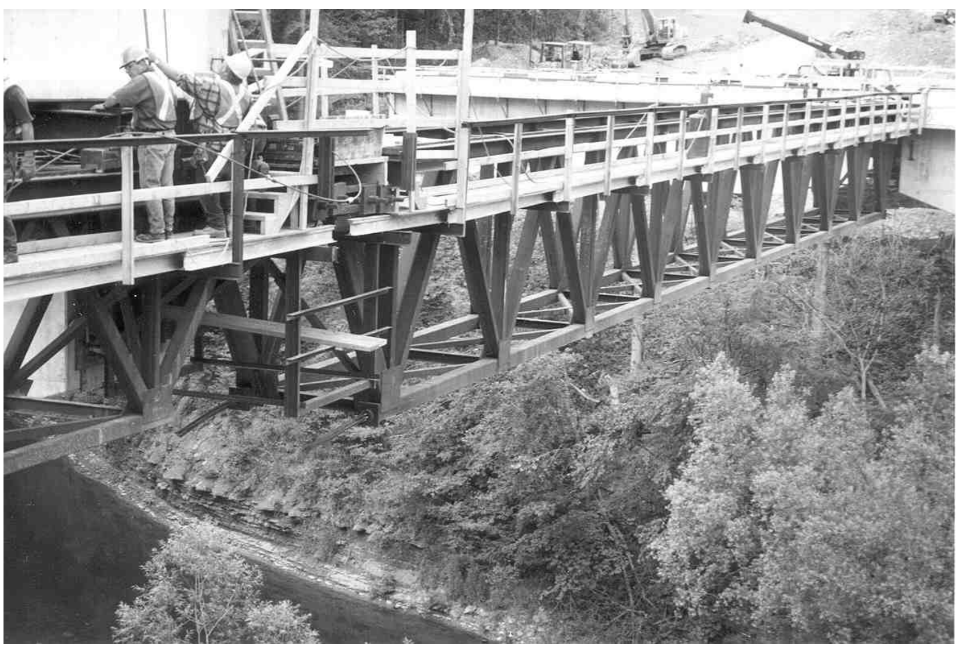
Photograph 1

longitudinally from the assembly area to the structure, between the westbound and eastbound structures. Two girders remain in the assembly bay, while the remaining girder is sitting on the launching truss in the middle of the bridge, waiting to be transported laterally to its final resting position on the eastbound bridge.

## LAUNCHING TRUSS

The launching truss (see Figure 6) consisted of a series of custom-made steel trusses, erected in the median between the twin bridges. The trusses were largely constructed from hollow structural steel members and were supported on brackets cantilevered off the sides of the piers.

The staggered pier arrangement greatly reduced the spans of the individual trusses between supports in comparison with the girder spans. The launching truss ran the full length of the two bridges offering maximum flexibility in sequencing the placing of the girders in any span. The longest truss span between the piers was 40m. For this span, it was necessary to post-tension the bottom chords of the truss (see Photograph 2).



Photograph 2

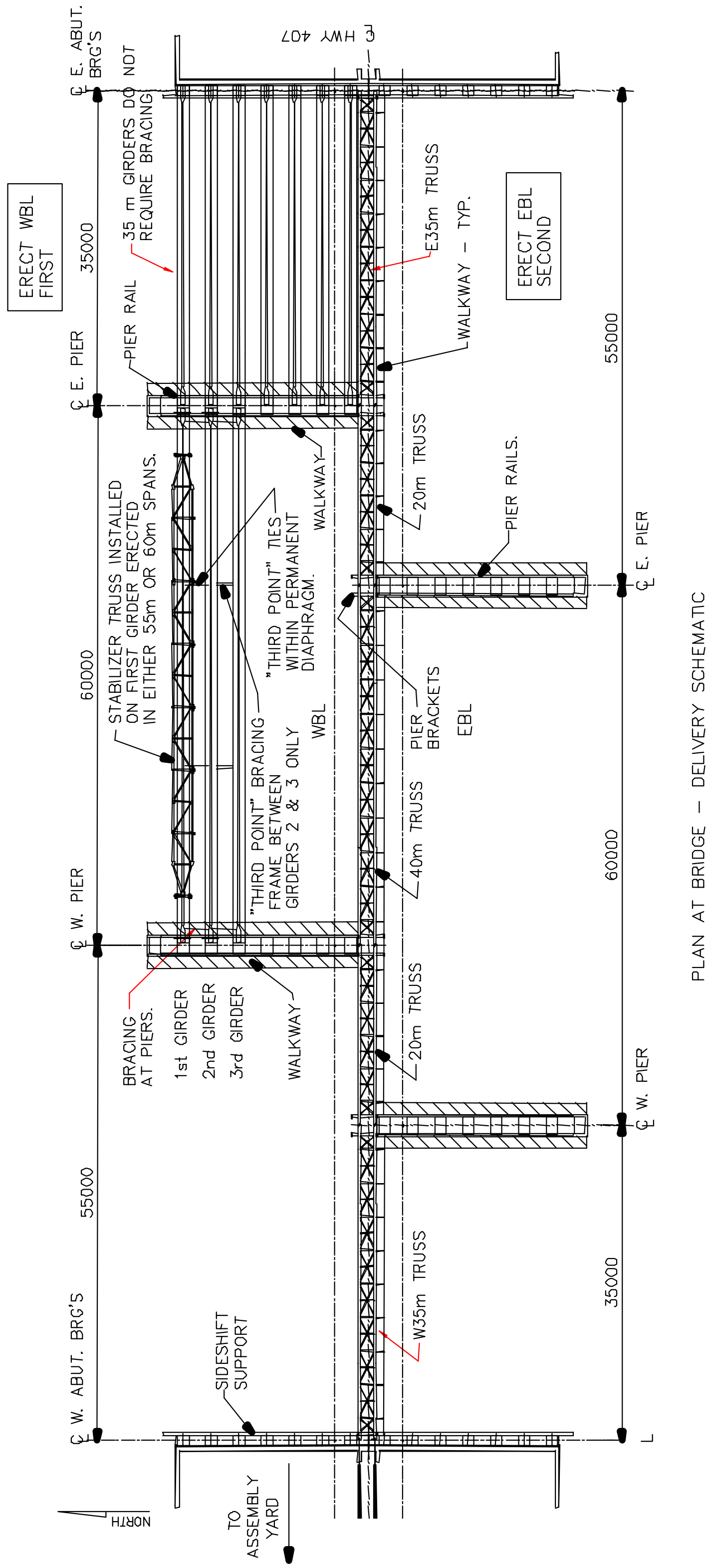

PLAN AT BRIDGE – DELIVERY SCHEMATIC
ERECT WBL FIRST
ERECT EBL SECOND
℄ E. ABUT. BRG'S
℄ E. PIER
℄ W. PIER
℄ W. ABUT. BRG'S
℄ HWY 407
35000
60000
55000
35 m GIRDERS DO NOT REQUIRE BRACING
PIER RAIL
E35m TRUSS
WALKWAY – TYP.
20m TRUSS
40m TRUSS
W35m TRUSS
PIER RAILS.
PIER BRACKETS
STABILIZER TRUSS INSTALLED ON FIRST GIRDER ERECTED IN EITHER 55m OR 60m SPANS.
"THIRD POINT" TIES WITHIN PERMANENT DIAPHRAGM.
"THIRD POINT" BRACING FRAME BETWEEN GIRDERS 2 & 3 ONLY
WALKWAY
WBL
EBL
BRACING AT PIERS.
1st GIRDER
2nd GIRDER
3rd GIRDER
SIDESHIFT SUPPORT
TO ASSEMBLY YARD
NORTH

Fig. 6

CARRIAGE DELIVERY SYSTEMS

As previously noted, the delivery of the girders from where they were assembled in the assembly yard, to their final resting position on the structure required three different carriage delivery systems. These were as follows:

*Assembly Yard Lateral Side-shifter System (Fig 7)* – The side-shifter system in the assembly yard consisted of a pair of support carriages in each bay, which would transport the assembled girders laterally to the longitudinal delivery system, which would transport the girders to the bridge. Each carriage was supported by four Hillman Roller bearings. The rollers ran on two steel bars embedded in each of the two exterior concrete sleep slabs. The carriages were powered by long stroke hydraulic jacks pulling prestressing strand, which was attached to the carriages.

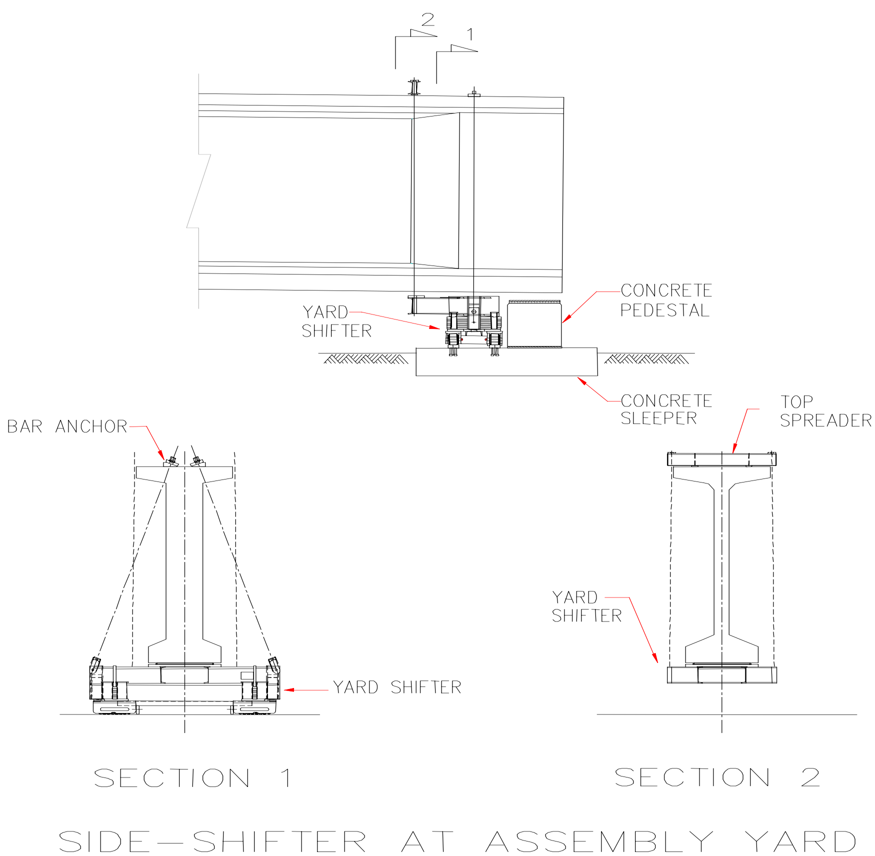



Fig.7

The steel grillage that comprised the carriage, housed a pair of jacks, which were used to jack the girder off its temporary support. Once the girder was raised off its supports, the jacks were blocked, and the girder was tied to the carriage using hi-strength thread bars. Longitudinal stability was provided to the girder (i.e. along the axis of the girder) by a second pair of thread bars, anchoring a spreader bar across the top flange of the girder to a stabilizer bar below the bottom flange of the girder. The stabilizer bar was rigidly connected back to the carriage. Once the girder was fully secured to the carriages, it could be side-shifted across the assembly yard.

*Longitudinal Transporter System (Fig.8)* – The longitudinal transporter system consisted of a second pair of carriages, one at each end of the girder. Again, each carriage was supported on four Hillman Roller bearings. Each pair of rollers ran on a steel bar running surface, which was welded to the top flange of a steel rail beam. The steel rail beams were attached to timber ties sitting on a granular pad. Once the grade was achieved on the running surface, granular was added to raise the pad level with the top flange of the rail beams. At the structure, the steel rail beams were fastened directly to the launching trusses.

The longitudinal carriages were a simpler design than the side-shifter carriages in two ways. First, it was not necessary to house jacks within them, because transferring of the girder from one delivery system to another was always achieved using jacks in the side-shifter carriages. Second, a wider wheel base could be used under the carriage, which provided the carriage with superior stability in comparison with the side-shifter systems – accordingly, only a single pair of hi-strength thread bars was required to tie the girder to the carriage.

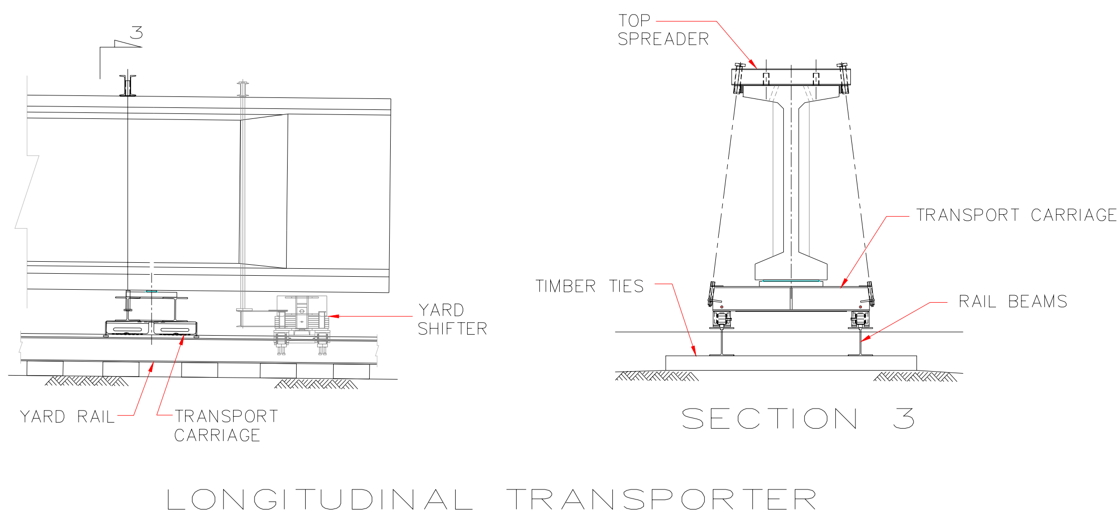



Fig.8

Once the girder was secured to the transporter carriages, a bull-dozer was used to push the girder along the rails, down the 3% grade to the bridge. A winch was used to pull the girder across the launching trusses to the span in which the girder was to be placed.

*Structure Lateral Side-shifter System (Fig. 9)* - The side-shifter system on the structure, again consisted of a pair of support carriages (one at each end of the girder), which would transport the girders laterally from the launching truss to their final resting position on the bridge. However, because of the limited room available at the piers and abutments to support a running surface for the carriage, each carriage was supported on only two Hillman Roller bearings (as opposed to four bearings for the other carriage systems). As a result, the side-shifters could not be made wide enough to be stable on their own. It was therefore necessary to stabilize the side-shifters to the girders themselves with tie rods connected to spreader bar across the top flange of the girder. The tie rods attached, in turn, to a stabilizing arm below the bottom flange. Additional lateral stability was provided to the stabilizing arm by a castor, running on a rail supported off the face of the pier.

The width of the pier caps was wide enough to accommodate the side-shifter rails as well as the permanent bearings. It was, however, necessary to increase the height of the pedestals in order to provide vertical clearance for the side-shift equipment as the girder was being transferred across the pier. At the abutments, the side-shifter rails were supported on steel brackets attached to the face of the abutment.

The side-shifters ran on a set of Hillman Rollers across the top of the pier in four specially installed channel tracks. As with the side-shifters in the assembly yard, the mode of power for the carriages was by long stroke hydraulic jacks pulling prestressing strand that was attached to the shifters.

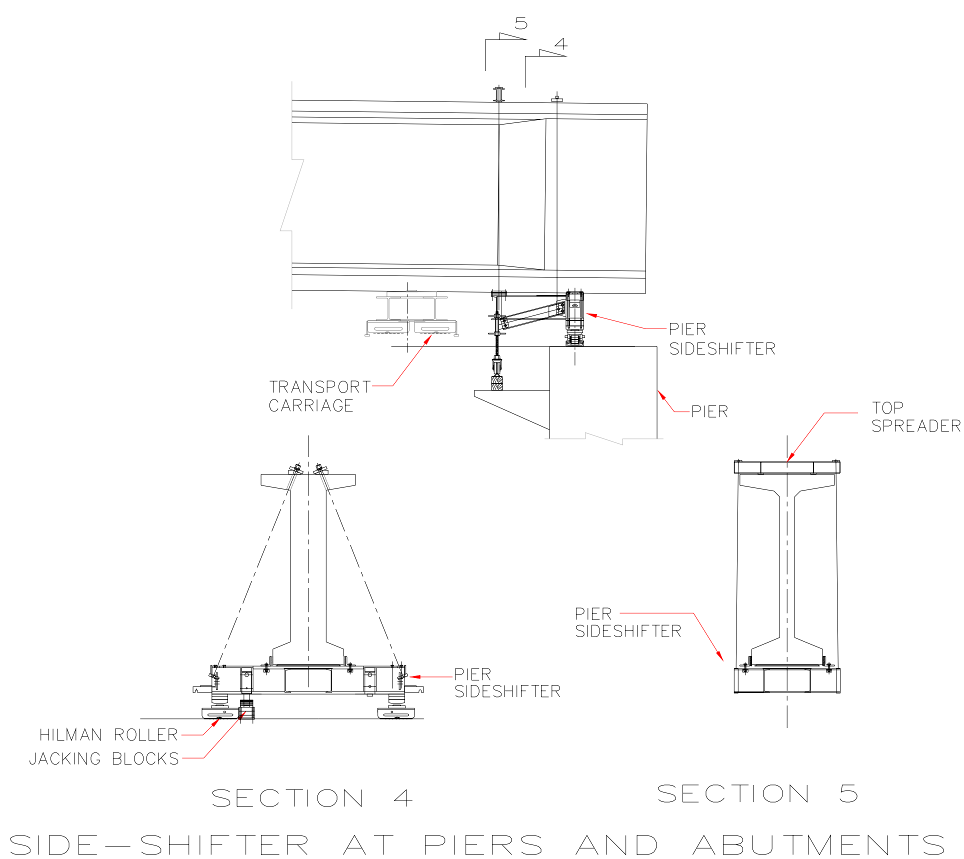



Fig. 9

*Girder Stability during Erection* – While on the various transportation devices, it was necessary to stabilize the girders against falling over sideways, particularly under the effect of wind load. This was achieved through threaded tie rods attached to the base of the transport carriage elements and tied either through the top flange of the girder, or to a spreader beam across the top flange of the girder. The tie rods provided sufficient stability to the girder for a 40km/hr wind speed, with gusts up to 60km/hr. The margin against instability was adequate but not excessive. Provisions were provided for tying off the girders, in the event the girders have to be stopped for an extended period on the transport carriage elements. The girder tie-offs were designed for a 1 in 5 year wind (81km/hr with gusts up to 115 km/hr).

Once erected on their bearings, additional measures were required to ensure the stability of the girders (see Photograph 3). The first girder erected in each of the longer spans (55m and 60m spans) was braced by a steel stabilizing truss attached to the top flange – the truss provided lateral support to the top flange. Subsequent girders were connected to the first girder and thus derived benefit from the additional stability provided by the stabilizing truss. In addition, the first three girders erected in each span were torsionally restrained at the piers and the abutments by post-tensioning strand connected to the top flange and anchored to the bearing seat. Once all the girders in a span were installed, the girders could be cross-braced in a way that allowed removal of the stabilizing truss before the forming and pouring of the deck. The design wind for the erected girders was a 1 in 10 year wind (88km/hr with gusts up to 125km/hr).

Photograph 3

## CONSTRUCTION SUMMARY

Construction of the Bronte Creek bridges was started in mid-November 1999. The substructure was completed in the spring of 2000. Precast girder fabrication started in February 2000, with delivery to the site starting in May. Fabrication of the temporary launching truss was started in March 2000. Field assembly was completed in June, with completion of installation of the launching assembly in July. The girders were launched commencing in early July and the launching operation was completed at the end of August. Placement of the precast concrete deck panels took place from early October to mid-November. One of the cast-in-place decks was placed in December. The remainder of the work was completed by April 2001.

## CONCLUDING REMARKS

The construction of the Bronte Creek was a highly successful project combining the proven economy of long-span, precast spliced-girder design, with launched girder technology. The use of the new CPCI2450A girder, made it possible to achieve the longest span for a bridge of this type in Ontario. And, although the launched girder construction scheme was highly specialized and is only appropriate for very specific applications, the scheme successfully ensured that the impact of construction on the environmentally sensitive valley floor was minimized. For this application, the contractor was satisfied that, despite the complexity of the project, he was constructing an economic structure for a project with challenging construction restrictions. The owner was satisfied that the impact of construction on the environmentally sensitive valley floor was minimized during the construction of a significant structure with long spans.

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