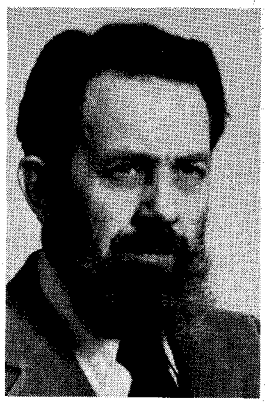
Design-Construction Feature

# The Islington Avenue Bridge



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The Islington Avenue Bridge is of interest not so much because of its segmental cantilever construction, but because use of this technique is made in an urban setting over a large and very busy railway yard. It is believed to be the first application of precast segmental construction in a major urban area in North America.

Islington Avenue is one of Toronto's many north-south arteries that run from the shores of Lake Ontario, right through the city, into the open country beyond (Fig. 1). The roadway differs from others in that its access to the peripheral Lakeshore Boulevard was prevented by a large railyard belonging to Canadian National Railways. Traffic predictions were sufficient to convince the planners that it would be prudent to provide this access, before the adjacent parallel streets became overloaded, and that the best route would be right across the yard at one of its widest points.

This choice was not a popular one with Canadian National (CN). The railway, quite understandably, did not welcome interference with their operations in an area that was solely theirs and where there had been no road crossing

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NOTE: The author was responsible for designing the Bear River Bridge, an eight-span nearly 2000-ft (610 m) long precast prestressed segmental structure completed in December 1972 near Digby, Nova Scotia. Although a few short-span segmental bridges were built in Canada in the sixties, the Bear River Bridge was the first major bridge built in North America using the precast segmental technique. The bridge played a significant role in the introduction of segmental construction in America. In 1973, Mr. Lovell was honored by the Prestressed Concrete Institute in the PCI Awards Program for his design of the Bear River Bridge—EDITOR

Describes the design considerations and the production and erection techniques used in building the Islington Avenue Bridge—a 491 m (1610 ft) long precast prestressed segmental structure recently completed in Toronto, Ontario.

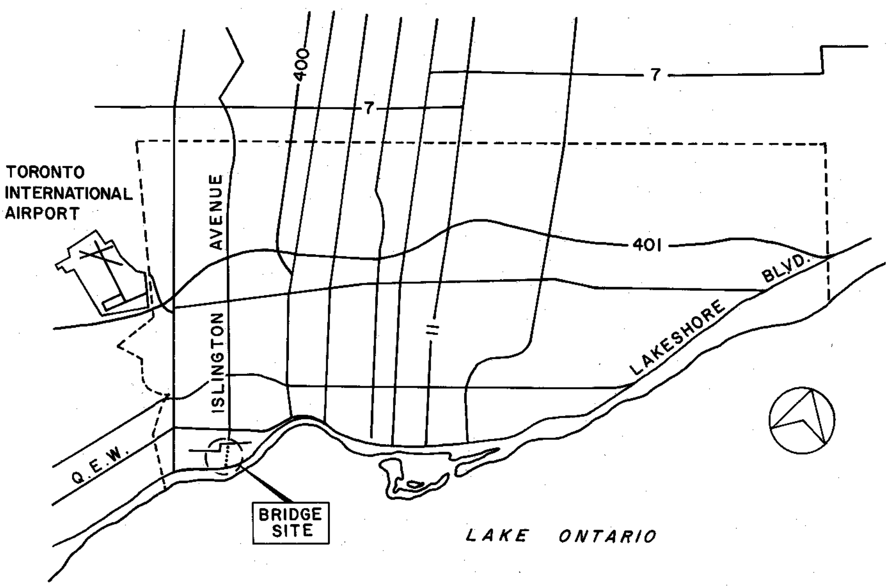



Fig. 1. Map of Metro Toronto showing location of bridge.

before. Their yard at this location is some 500 m (1640 ft) wide and contained between 35 and 40 tracks (depending on just where they were counted). Not all of these tracks were busy; some were used for storage, others were used for parking and the light maintenance of a Government-operated transit system. However, three of them, approximately in the middle of the yard, were (and still are) very busy high-speed mainlines (Fig. 2).

The problems of placing piers in such a concentration of tracks were many, and their resolution became possible only when the concept of segmental construction was demonstrated to the railway company. Inherent in that concept are the advantages of longer spans, and therefore fewer piers, and the opportunity to build the superstructure with very little contact with the ground underneath; advantages that proved to be powerful arguments. Once the railway company's acceptance had been obtained, their cooperation was immediate and they were generous in the amount of access and construction space they allowed.

The purpose of this paper is to present the design considerations and the production and erection techniques used in building the Islington Avenue Bridge. Special attention is devoted to the treatment of deformations due to temperature effects and volume changes. Detailed examples are provided in the Appendix, demonstrating how to calculate temperature and continuity effects and how to predict the "droop" curve. Lastly, some of the problems and solutions encountered during construction are related.

## Design Considerations

The client, the Municipality of Metropolitan Toronto, required that the structure be designed according to HS.25 (AASHTO HS20-44 multiplied by 1.25). To accommodate six lanes, two sidewalks and a narrow median, a total structure width of 28 m (92 ft) was required. Two single-cell boxes were used (Fig. 3). The final span arrangement agreed upon by CN was, reading from the north: 49, 61, four spans at 83, and 49 m (161, 200, four spans at 272, and 161 ft) for a total

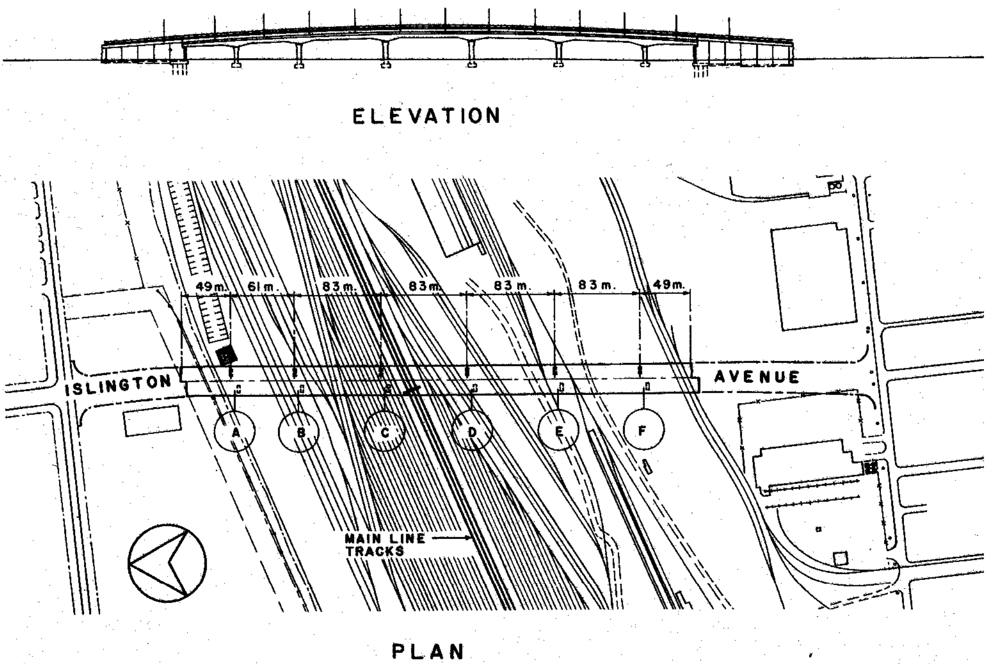



Fig. 2. Plan and elevation of Islington Avenue Bridge.

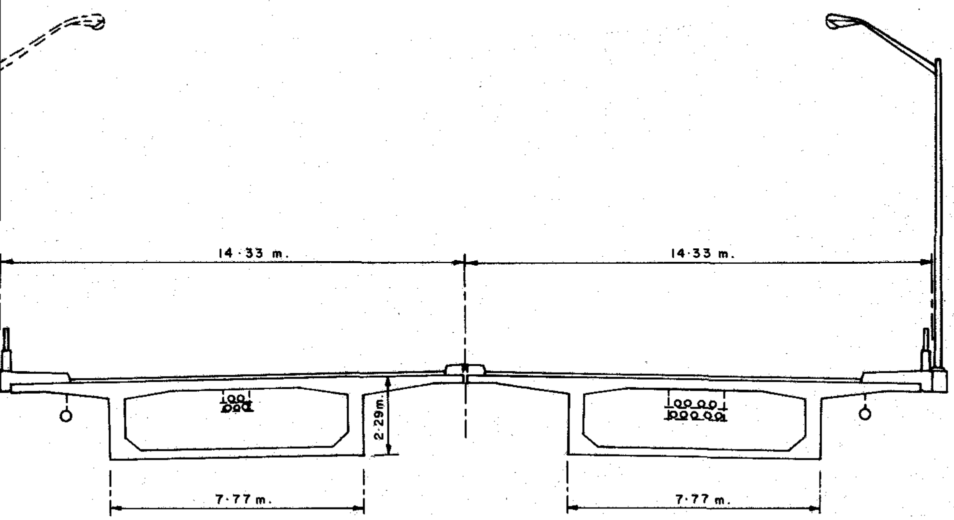



Fig. 3. Cross section of bridge at midspan.

structure length of 491 m (1610 ft). Fig. 2 shows the principal longitudinal dimensions of the bridge.

The lack of symmetry in this span arrangement provided a few problems that tended to complicate the analysis, increase the complexity of tendon profiles and, in some cases, require precast units of non-standard dimensions. However, these problems, which needed some further calculations to solve them, were secondary to the vertical alignment constraints. The normal railway clearance of 7.16 m (23.5 ft) from base of rail was easily attained in the interior spans. However, in the end spans, it was much more difficult.

Gradients on the approaches to the bridge had to be limited to 6 percent. It was necessary to raise the intersections with adjacent cross-streets by amounts that seriously interfered with access to local properties and that were considered undesirable by the local municipality. To accommodate the conflicting requirements of maintaining adequate rail clearance without raising the profile any more and to leave the designer some construction depth to work with, the end spans were tapered (Fig. 4).

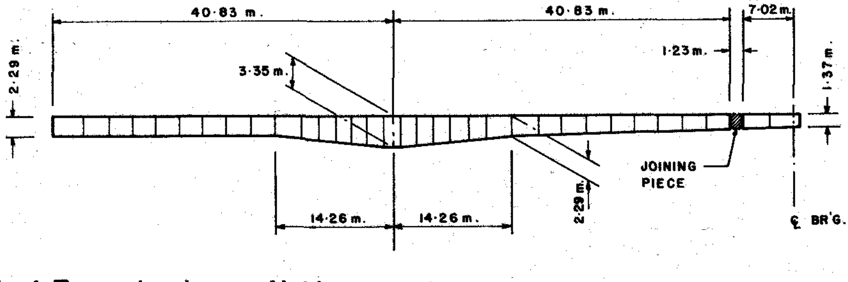



Fig. 4. Tapered end span of bridge.

The structural design was based upon classical elastic theory for continuous structures. It was assumed that the bridge would be built by starting at one end and working methodically to the other. The balanced cantilevers, of course, are statically determinate and are not complicated by their non-prismatic properties.

However, after the free ends of the first pair of adjacent cantilevers are joined, all subsequent calculations must recognize the non-prismatic nature of the structure and the ever-increasing number of spans involved.

## Deformations

A feature of the permanent change in the statical system after the joining of the cantilevers is the effect of creep on the structure. At the time of joining, both cantilevers have deflected and rotated under the combined effect of dead load and prestress. After the concrete that fills the gap between adjacent cantilevers has hardened, the cantilever tips are effectively locked together and further relative rotation between their ends is prevented. Under the influence of creep, which tries to increase any existing deformations that have taken place elastically, the two cantilever tips attempt to rotate and deflect even further.

While there is little or no restraint to additional deflection, since both ends are attempting to deform in the same direction and by the same amount, there is restraint to further rotation because the sense of each rotation is the opposite of the other. It is the restraining of these attempted rotations that brings into play secondary moments that are distributed right through that part of the structure that is continuous at that time. Note that a sample calculation showing the distribution of such secondary moments is given in Appendix C.

It can be shown that the moments induced at the supports of the now continuous span by such a restraint can be determined from:

$$M_s = (M_o - M_c)(1 - e^{-\phi})$$

where

$\phi$ = creep factor

$= \dfrac{\text{strain due to creep at time } t}{\text{elastic strain at time zero}}$

$t$ = time when creep is being considered

$M_o$ = actual support moment at time of joining

$M_c$ = moment at support caused by the same loading condition, assuming the structure had been continuous in the first place

The sense of the corrective moment is to move from the cantilever condition to that which would have existed had the structure been initially continuous. Such corrective moments have to be distributed across the continuous structure.

This and many other features of segmental design are explained and illustrated in the *Precast Segmental Box Girder Bridge Manual.*[1]

## Temperature Effects

One of the topics covered in the *Segmental Manual*[1] worthy of further comment is the longitudinal effect of temperature differential through the precast segmental box. In most design examples cited (in the *Segmental Manual* and elsewhere), it is usually assumed that the rectangular section of the top slab of a simplified box is at a uniform temperature that is different from the rest of the box (see Fig. A1 in Appendix A).

It is extremely unlikely, however, that such a temperature distribution would ever exist in an actual situation. Priestley[2] discusses the problem and gives diagrams that follow a triangular distribution. For purposes of compari-

Table 1. Summary of temperature stresses at midspan for a simple span and continuous span structure. (see Appendix A).

| Loading condition | Case I | | Case II | |
|---|---|---|---|---|
| Stress condition at center span | Simple span | Span D-E continuous structure | Simple span | Span D-E continuous structure |
| Top fiber | 0.018 | −1.739 | −1.382 | −2.581 |
| Bottom fiber | −0.691 | 2.120 | −0.547 | 1.371 |

Note: Stresses are given in MPa (1 MPa = 145 psi).
A negative sign denotes a compressive stress.

son, the top and bottom fiber stresses were calculated for a temperature differential of 10 C (18 F), where the distribution was uniform for Case I and triangularly distributed over the same cross section for Case II (see Fig. A2 in Appendix A).

These temperature stress calculations are performed for both a simple span and continuous seven-span structure. Table 1 summarizes these stresses. Note that a negative sign denotes a compressive stress. For calculation details leading to the stress values shown in Table 1, refer to Appendix A.

Thus, in the simple span condition, the tensile stress is negligible. In the continuous structure, however, tension becomes more important. It is true (as pointed out in the *Segmental Manual*[1]) that the increases in permissible stresses that are allowed for the less probable loading combinations, make stresses of these magnitudes relatively unimportant in the total stress picture. However, if the resultant stress is tensile, the allowable tensile stress is still zero and the factors have no meaning.

Considering the distribution of the temperature differential to be triangular has a beneficial effect and it is important that the actual distribution be known. Steps are being taken to have the Islington Avenue Bridge instrumented so that the temperature differentials may be measured.

## Segment Design

From preliminary design considerations, it was established that a depth of segment section of 2.29 m (7.5 ft at midspan and 3.35 m (11 ft) at the piers would be adequate. The transition from one depth to the other was achieved by means of a straight haunch section (Fig. 5). A curved soffit to the haunch was considered but was rejected because it would have complicated the formwork considerably.

For practical reasons, the haunch point, that is, the place where the section becomes constant, was located at a joint between segments. Initially, its location was selected intuitively. However, as the design developed and segment sizes became more refined, the haunch point moved a little. Its final location produced some minor compression problems in some haunches, and although these have satisfactorily been resolved, it is now evident that a launch point a little further from the pier would have been advantageous.

The sides of the box section of the segments were kept vertical because sloping sides would have resulted in a narrower soffit at the piers than in center span. Enhanced section prop-

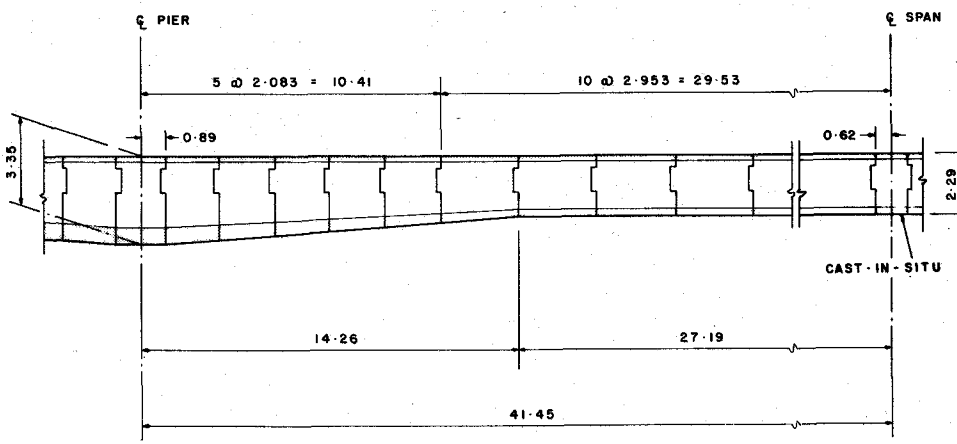



Fig. 5. Details of segments in one cantilever of bridge.

ALL DIMENSIONS IN METRES

erties at the piers were achieved partly by the increased depth and partly by increasing the thickness of the bottom slab linearly from 0.23 m (0.75 ft) at the constant depth section to 0.68 m (2.3 ft) at the pier.

A single cell unit was used for each of the two structures. A twin cell unit of the same overall width as the single cell unit was considered, but rejected in favour of the more elegant and cheaper unit whose two webs were adequate for the job. If tendon anchorages are to be placed in the web, the web cannot be reduced beyond a certain minimum thickness required to accommodate the anchorage. Thus, if the required prestressing can be fitted into two webs comfortably and three webs are not required for shear, the single cell unit is the more economical segment.

With only two supports for the top slab, the cantilever arms become longer and the top reinforcement over the webs becomes relatively heavy. The anchorages for the continuity tendons are located in the top slab immediately over the webs. The recesses for these anchorages are long and they effectively cut the top reinforcing steel for the cantilever at the location where the moment is maximum.

To circumvent this problem, stressing (using 1/0.6 Freyssinet tendons) was used with the tendons passing right through the anchorage recess. This required the tendon sheaths to be cut at the recess until stressing was completed. The sheaths were patched, the recesses filled with concrete having a strength of 41.4 MPa (6 ksi), the tendons threaded through their sheaths and tensioned.

Mild steel reinforcement was trimmed around the recesses in a normal manner. Since the deck slab had been designed to be under partial prestress (using 1/0.6 Freyssinet tendons) all that was necessary, in most cases, was to move a nearby tendon in order to make it pass through a recess. In a few instances, additional tendons had to be provided.

At this point, a comment on the deck stressing is worthwhile. The deck slab, including the cantilever arms, was partially prestressed in the transverse direction and the requirements for cracking stresses were met. However, it now appears that it would have been prudent to provide full transverse prestress. Winter weather in the Toronto area is neither warm enough nor cold enough to cope with the normal snowfall without the liberal application of de-icing salt. Although the deck is provided with a water-proofing membrane of hot rubberized mastic asphalt under the asphalt road surface, it is obviously better to have the top concrete stressed permanently in compression so that there are no cracks for the salt to penetrate the deck.

## Segment Size

Segmental units are usually cast by the "long-line" method which essentially uses a fixed soffit form, shaped like the underside of the bridge, with movable side forms that travel along it. The other technique is the "short-line" method which uses a single form in which each segment is cast individually.

The length to which the segments would be manufactured was a controversial point. Two criteria had to be considered, namely, constant length and constant weight. A constant length unit would have definite formwork advantages, especially if casting were done by the short-line method. Constant length units of varying depth would obviously vary considerably in weight.

A constant weight unit would have advantages when hoisting equipment was considered. The agreement with the railway company (in which little or no erection from the ground was stipulated) made it almost certain that some form of launching truss would be employed. Such a launcher would have to be kept as light as possible to reduce erection loads on the structure to a minimum. The cost of the launcher would be sensitive to its own weight and its lifting capacity. Thus, economy would be achieved by lifting many loads of approximately equal magnitude rather than sizing the equipment to lift a few heavy loads and having it under-utilized for most of the time. Constant weight segments of varying depth would obviously vary in length.

After consultation with highway and municipal authorities, it was agreed that, for segmental units of the shape chosen, the weight should be limited to 64 tonnes (70.5 tons). The target was set at 50 tonnes (55 tons) and later increased to about 55 tonnes (60.6 tons) in an effort to reduce the number of units. The lower maximum weight was selected in the hope that unrestricted passage through the city would be permitted. (As it turned out, there was very little trouble transporting the segments although a police escort was provided for some of the taller units). The segmental units were designed for constant weight although, as explained later, this decision was reversed.

## Design Adjustments

Final stress checks showed that, under adverse temperature effects, there was no residual compression in the bottom fibers at some midspan sections. The magnitude of the tensile stress was well within the cracking strength of the concrete but since the structure was segmental, i.e., there was no continuous bonded unstressed reinforcement across the joints, absolutely no tension was allowed.

At this time, the production of final drawings was in progress. Therefore, because the problem was minor, no change in geometry or prestress was contemplated. Instead, four additional tendon sheaths were placed in the bottom slab over a number of midspan segments, with the sheaths coming to the surface of the bottom slab in designated segments at approximately the quarter span points. After erection, unstressed single strand tendons were threaded through the sheaths and grouted in.

Table 2. Comparative costs of bridge as bid.

| Bid | Prices in millions of dollars | | | Cost in dollars per square foot for total structure |
|---|---|---|---|---|
| | Total contract | Concrete bridge | Steel bridge | |
| 1 | 9.239 | 6.309 | — | 42.59 |
| 2 | 9.432 | 6.096 | — | 41.16 |
| 3 | 9.582 | 6.430 | — | 43.41 |
| 4 | 9.814 | 6.575 | — | 44.39 |
| 5 | 10.015 | — | 6.640 | 44.83 |

Note: Costs are given in Canadian dollars (1978).

A partial prestress, cracked section analysis showed that there was negligible stress in the unstressed tendons and an acceptably low tension stress in the concrete. Longitudinal main stressing used 12/0.6 Freyssinet tendons and transverse stressing was by 1/0.66 Freyssinet strands. Concrete strength in the precast segments was 41.4 MPa (6 ksi). An option to change the prestressing system was not exercised by the contractor.

## Bridge Costs as Bid

The contract documents included alternative designs in both precast segmental cantilever construction and structural steel; only one design could be bid. Both designs were based on identical span arrangements and loadings, both designs had to be launched. The steel structure was to be incrementally launched from one end.

Seven sets of documents were taken out. Five bids were returned; four in concrete, one in steel. Steel was the highest bid (see Table 2). Note that the cost figures are for 1978 and are given in Canadian dollars.

The contract was divided into eight parts, of which two were the steel and concrete alternative designs.

Bid No. 1 (a precast segmental alternative design) was awarded the bridge project.

The total contract was for $9.239 million of which the bridge portion was $6.309 million or $42.59 per sq ft for total structure (Table 2).

## Segment Manufacture

The contractor chose the short-line method for the manufacture of the segmental units, which were to be match cast. Two forms, or casting cells, were developed, one of which is very flexible and adaptable, the other being less so. The former casting cell is intended primarily for the production of the tapered units and any other non-standard unit, while the latter is intended for the constant depth units. Both forms are of all-steel construction and are the end product of much thought and careful design.

As a consequence of his choice of forms, the contractor decided to use a constant length unit rather than the constant weight unit of the contract drawings. He redesigned the superstructure and finished up with two segments fewer per span. This resulted in only little change in the basic design. The overall shape of the span was the same except for a slight change in the haunch point. The amount of stressing was the same; only its disposition was slightly altered.

Pier units, i.e., the segmental units that sit directly on the pier, were cast first and are the only units not match cast. Each one acted as an end form for one side of the adjacent units.

The method of match casting, as used for the manufacture of the Islington Avenue segments, was as follows:

The casting cells are equipped with only one end form, or bulkhead. The other end form is provided by a previously cast unit that is clamped between the side forms after having been very carefully positioned for attitude and direction. The accurate location of the unit is achieved by the use of surveying instruments that are set firmly on a permanent base adjacent to the form. The steel bulkhead at the other end is equipped with a pattern of holes that will satisfy all possible combinations of tendon patterns; unused holes being closed by special inserts.

A reinforcing cage that also contains all tendon sheaths is lowered into the form. When the cage is secure, final adjustments are made to the sheaths to ensure their correct alignment. All inserts such as tendon anchorages, holes for deck drains, manholes, and other auxiliary items are set at this time.

The inner form is cantilevered out from a trolley that runs on rails and it is pushed into position. The side walls and bottom of the casting cell are now securely clamped to the previously cast unit at one end and the end bulkhead at the other. Inflatable tubes are inserted into all tendon sheaths and inflated to ensure good alignment of the ducts and to keep out cement paste. Concrete with a strength of 4.14 MPa (6 ksi) is placed within the form and very carefully vibrated. It is imperative that the concrete be placed with great care. Cross sections are thin and are congested with many different types of reinforcement. Some blockouts provide re-entrant corners that encourage honeycombing.

Next day, before the forms are eased, all control points on the newly cast unit are resurveyed now that the concrete has hardened. The positions of these control points relative to those of the previously cast unit are analyzed and any necessary corrective action is calculated and incorporated into the setting-out data of later segments. At the same time, an exaggerated plot of the "errors" is kept for monitoring the erection of the units later. Careful attention to accuracy at this time shows its rewards when the units are being erected because any errors cast into the unit can be almost impossible to correct later.

The previously cast unit is now removed from the form and any defects in casting are promptly patched up where possible. In any event, the freshly cast segment is taken for wet curing and eventual storage in the storage yard. The newly cast unit now becomes the previously cast segment and is moved over to be clamped in the open end of the form. The cycle continues at about 24-hour intervals.

## Erection Procedure

The following erection procedure was used for the Islington Avenue Bridge:

The launching truss (or launcher) is critical to the placing of the segmental units in their final position in much the same manner as the casting cells were critical to their manufacture. The launcher performs the functions of a temporary bridge between successive supports, a travelling crane by which every unit is placed in position and the stabilizer which balances the growing cantilevers on their piers under the eccentric loading of erection. Many different types of launchers exist around the world but there was nothing suitable or readily available in Canada. The economy of bringing a foreign launcher into the

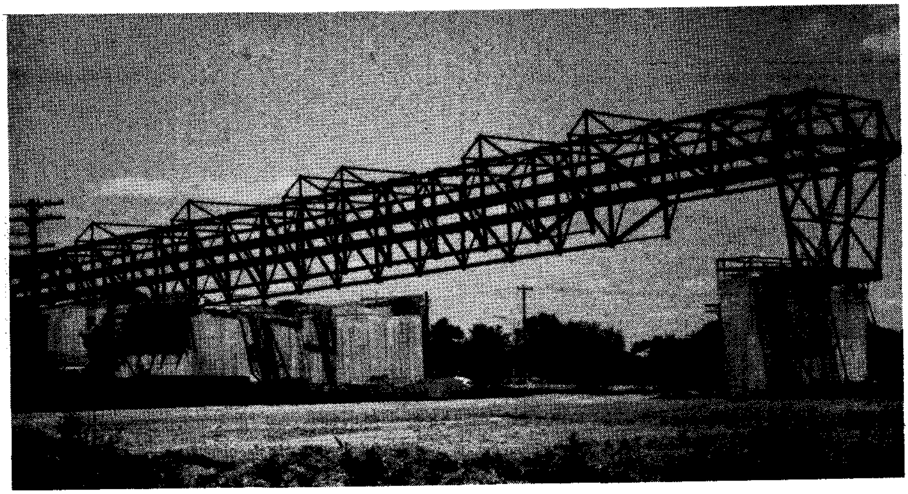


Fig. 6. Launching truss resting on piers.

country was investigated but was found unfeasible. In the end, the launcher shown in Fig. 6 was designed, fabricated and erected in Toronto. Fig. 7 shows a schematic shot of the launcher.

The construction of each balanced cantilever starts with the launcher supported with its front end on a pier and with its rear end on previous construction. The truss overpasses the pier and reaches backwards with two fingers that are supported in specially prepared seats in the forward face of the pier.

These fingers are designed to carry not only the front reaction from the launcher, but also the segment that eventually will be attached to the forward face of the pier segment. It is necessary to place this segment first because there is no room in which to

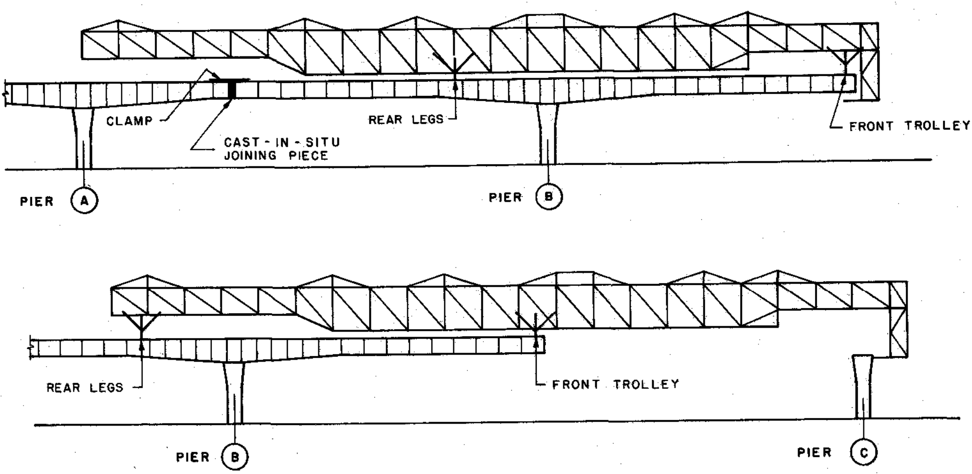



Fig. 7. Schematic drawing of launching truss and launching procedure.

pass the pier segment once the latter is in position.

The pier segment is then placed on its permanent bearings, carefully located and temporarily blocked in position. The mating surfaces of this and the waiting forward segment are coated with epoxy adhesive and then gently pulled into contact by means of the temporary holding device. The temporary holding device consists of tensioning rods that are held in specially designed brackets bolted to the top of the top and bottom slabs of each segment.

By means of small hydraulic jacks, the rods are put into tension and the two segments are slowly pulled together. As this happens, excess epoxy adhesive is squeezed out of the joint and then, when concrete to concrete contact is obtained, the two segments are securely locked together. They will remain this way until the permanent stressing is applied. In an attempt to get a uniform thickness of adhesive in the joints, the forces applied to the top and bottom tensioning rods have to be varied as the shape and weight of the units vary.

The rear segment is then brought up and similarly treated. With both units secured, the main tendons are threaded through all three units and tensioned. The temporary holding devices are detensioned and prepared to accept the next two segments.

At this stage, the launcher is still supported by its two fingers from the front of the pier and there are now three segments stressed together so that they act as one unit, called the "hammerhead." The hammerhead rests on the permanent bearings but is stabilized by temporary hardwood blocks between it and the top of the pier.

The bottom slab of the pier segment is 0.69 m (2.26 ft) thick and it has a square hole in it, centered over the pier. A truncated pyramid of reinforced concrete, formed as part of the pier, protrudes into the hole and acts as a stop block (Fig. 8).

The pier segment, which had been placed accurately into position, is secured in that position by inserting hardwood wedges between the vertical walls of the hole and the sloping sides of the stop block. When the hammerhead has been adjusted so that it is heading in the correct direction, both horizontally and vertically, mortar wedges are cast between the hardwood blocks. These wedges have handles cast into them so that the wedges can be eased or removed when required. The contact surfaces between mortar and hardened concrete are lined with sheet neoprene so that, although the hammerhead is effectively locked against translation, it is capable of limited rotational movement (Fig. 8).

With the hammerhead thus secured, the launching truss has to launch itself in order to erect more segments. The launcher is equipped with a gantry crane that runs from one end of the truss to the other, and it is the trolley of this crane that provides the motive power for the auto-launch.

Two removable legs are attached to the trolley, one on each side of the truss. Each leg is equipped with a hydraulic jack and the two jacks are capable of lifting the weight of the launcher. The legs, now an integral part of the trolley, are centered on the pier segment directly over the bearings.

A similar pair of legs are attached to the rear of the truss and they are equipped with wheels that bear on both the top and bottom of the crane rail. All four hydraulic legs are extended until the weight of the launcher is borne on the legs alone. The supporting fingers which rested on the forward face of the pier are now free of their bearing seats.

By attempting to move the trolley

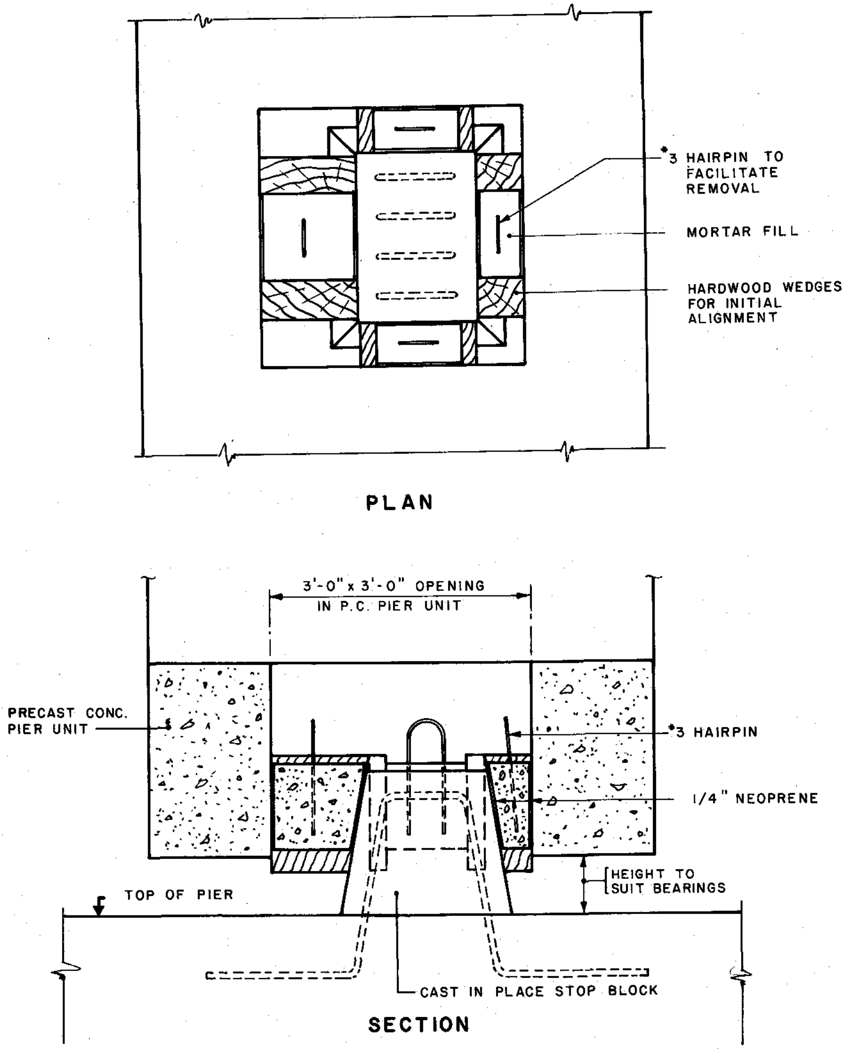



**Fig. 8. Plan and section details of stop block.**

towards the rear of the launcher, the truss portion is moved forward on the wheels of the stationary supports. The movement is very gentle and under the control of one person situated in the control cabin on the crane trolley. When the leading edge of the launcher is past the midspan point the movement ceases and special feet, incorporating screw jacks, are attached to the bottom chord of the truss. By lowering the front legs, the weight of the forward end of the launcher is transferred to four feet, one at each end of each side of the three-segment hammerhead. Similarly, the rear end of the launcher is lowered onto a pair of feet.

At this stage, the front end of the launcher is supported from the hammerhead which in turn is stabilized by the launcher. The temporary blocking under the pier segment may now be removed and the stop block remains properly wedged.

Precast deck segments, trucked in from the plant, are backed up under the rear end of the launcher. The trolley, once again in its role of a travelling crane, picks up the segment and carries it through the truss until it is over one end of the cantilever. The segment is lowered, turned through a right angle, epoxied and temporarily stressed to the hammerhead. Once the segment is supported by the temporary holding device, the crane retreats, picks up the next segment and lowers it on to the other end of the cantilever.

It is during this procedure that the unbalanced moment, caused by only one of the segments being supported on the hammerhead, is stabilized by the launcher. As the cantilever arms grow, the magnitude of the unbalanced moment increases. However, the four stabilizing feet are constantly being moved outwards so that maximum attitude control is maintained at all times. The launcher is an elastic structure and will allow the hammerhead to tilt a little until the balancing segment is placed. This stage is a time when a very close watch must be kept on the geometry of the growing hammerhead.

The double cantilever, as has been explained previously, is capable of limited rotational movement because of its neoprene cushioning at the pier stop block. At this stage in the construction, the whole hammerhead is extremely susceptible to warping and deflection in all directions when exposed to the intensity and direction of sunlight. [For example, a diurnal horizontal oscillation of about 50 mm (2 in.) amplitude was measured.] Direct sunshine will cause the ends of the cantilever to droop.

Thus, it is a seeming paradox in that it is possible to have a correct alignment, both horizontal and vertical, but at any one time measurements might indicate a different attitude. Every effort, therefore, must be made to check measurements when the sun is not shining. Any readings taken in strong sunlight must be treated with due caution.

In an attempt to eliminate possible errors due to the whole hammerhead being tilted, the checking was done in the following manner:

The design calculations gave the elevations of all joints in a balanced cantilever after the addition of each pair of segments. Not included, at this stage, were deflections attributable to the weight of the launcher.

From these data a curve of elevations for each joint was plotted [see solid line in Fig. 9 (a)]. The value used for each joint was the elevation of that joint, when it was the free end of a newly erected segment, with the deflection caused by the launching truss having been taken into account.

The broken line represents the attitude the structure could have due to a tilt at the pier. It is quite practicable to take this tilt out when joining the "free" cantilever to the already completed structure. Thus, the broken line must be recognized as the actual alignment, even though it looked wrong in the field.

If the base line in Fig. 9(a) is swung to the horizontal as in Fig. 9(b), then the distance $\delta$ can be calculated from:

$$\delta = \text{El.Pier} - \frac{\text{El.A} + \text{El.B}}{2}$$

where A and B are the two ends of the hammerhead at any given time. El. Pier is the elevation at the centerline of the pier and is constant regardless of the attitude of the hammerhead; $\delta$ is the average "droop" of the two ends and is the same as $\delta$ in Fig. 9(a).

The same calculation for measured elevations will give the measured

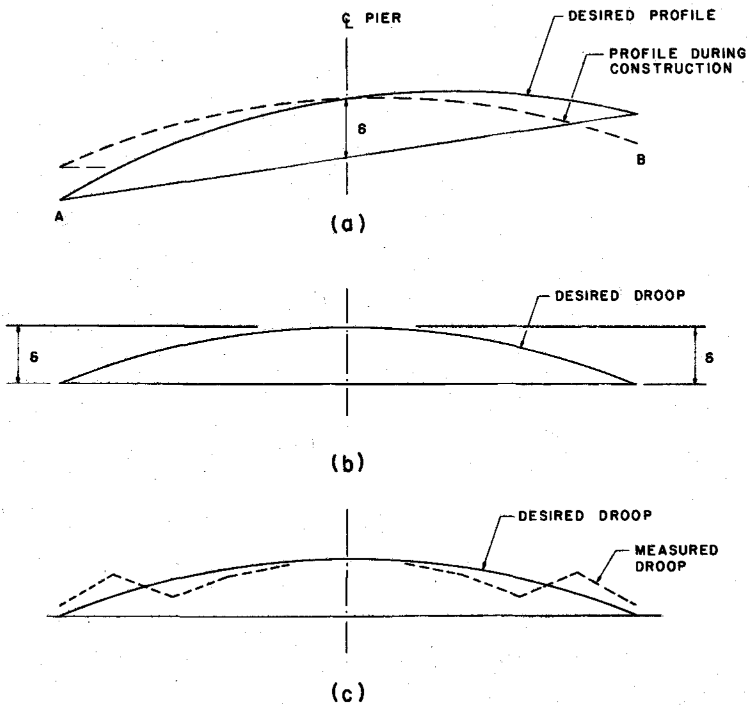



Fig. 9. Droop curves.

droop. Fig. 9(c) shows measured droop in relation to required droop. The nature of the calculations is such that the resultant droop curves are symmetrical about the centerline of the pier and only half of the curves need be considered (as in Fig. 10). The relationship between the measured and the desired droop makes the direction of any desired correction obvious but the magnitude of the correction is more difficult to assess.

To make the correction, a series of shims must be inserted in either the top or bottom joints between slabs. In essence, the shim is a thin obstruction device that prevents the two concrete surfaces of a joint from coming into contact before the epoxy adhesive has hardened. Shims thus provide a joint which tapers from top to bottom (or bottom to top as the case might be).

In practice, the extra waiting time, to allow the epoxy to harden, was sometimes impossible to achieve because of restraints imposed by railway operations. In these cases, the opportunity to shim had to be forgone and an attempt made to compensate for the error in the next joint.

For a number of reasons, such as varying contact pressures between faces at the joint due to variations in the temporary holding device forces, or the fact that the extra thickness of epoxy was not hard enough to refrain from some deformation under permanent prestressing, the effects of shimming were difficult to predict.

If, despite this vigilance, the end of the free cantilever of the finished part of the structure turned out to be in error, then a correction could be applied to the "Desired Droop" values.

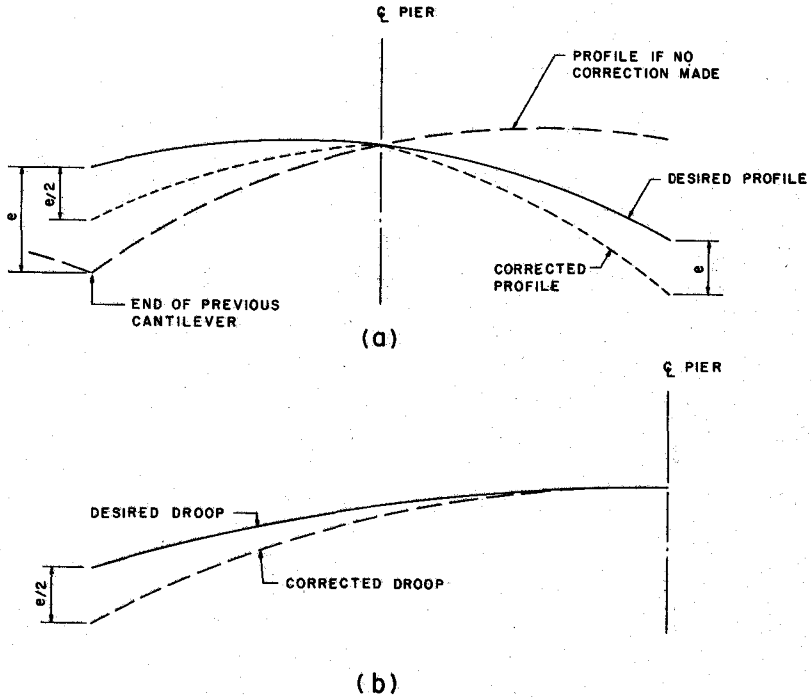



Fig. 10. Corrected droop curves.

In Fig. 10(a), an error $e$ was found to exist at the end of the previous cantilever. To allow for this error, a correction varying linearly from $e/2$ at the end to zero at the pier is applied to the required profile, the sense of the correction being the same as the error $e$.

The resultant curve will differ from the required curve but when the joining end of the hammerhead is lowered to meet the existing structure, the free end will rise to meet the desired elevation. The corrected droop curve is shown in Fig. 10(b).

The preceding method appears to be very much of a hit-and-miss approach and the exaggerated scale sketches add to this impression. Actually, the errors are usually very small. However, if allowed to remain uncorrected, they can become an appreciable, and often visible, eccentricity in the alignment of the structure. If attended to promptly, the aberrations are not obvious; in fact, normally only the field staff are aware of their existence.

The Islington Avenue Bridge developed in much the same way as just discussed. An accurate record of the casting results had been kept and any segment that was known to be not quite standard was watched carefully. Extreme vigilance is imperative at all times and the vital importance of this fact must be recognized right from the start of erection. Perfectly normal segments would sometimes behave abnormally when incorporated in the structure. Erection would be in progress with each segment conforming to the desired profile when the addition of a segment would cause an erratic deviation from that profile. The deviations were always small but small

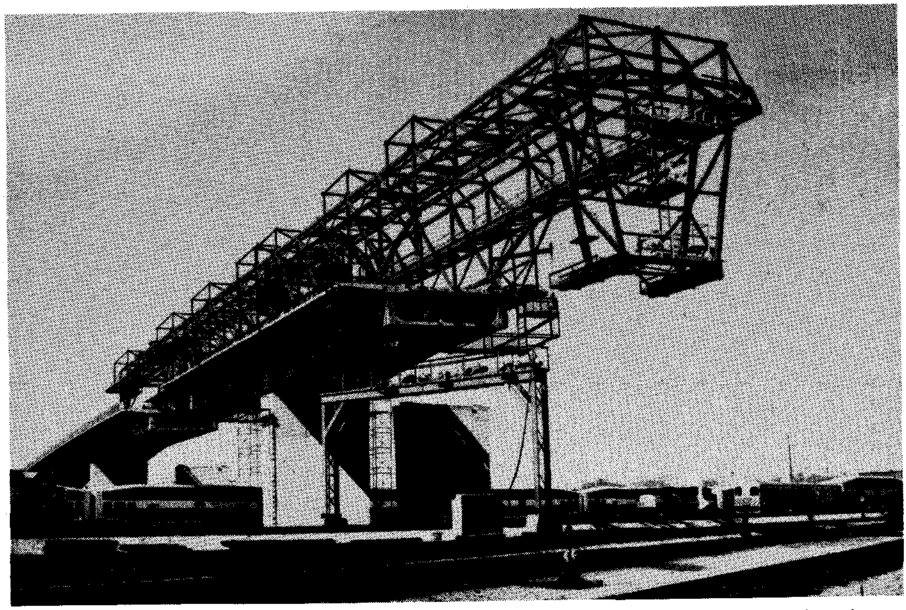


Fig. 11. Construction of bridge over main line tracks. Note that signals on signal gantry have been lowered temporarily.

changes can accumulate into much larger errors after the addition of further segments.

Several factors were identified that could have contributed to these deviations. One possible cause was a variable epoxy joint thickness. It had been recognized that, for a uniform joint, a uniform contact pressure would be required. Segments of different shapes and weights required different tension forces in the temporary holding devices, something that was allowed for in the contract drawings.

The type of temporary holding device selected by the contractor differed from that in the drawings and failed to provide the required uniform glue pressures. It was only later, when alignment difficulties were being encountered, that this fact was recognized. The forces were immediately altered to give even pressure but it is difficult to say whether any appreciable effect was noted.

In the early stages of the Islington Avenue Bridge, some of these divergences from the true profile were suspected but not accurately identified. It was only when the first joint between two cantilevers was completed that it became apparent that the free end of the new cantilever was noticeably below required profile. Although a determined effort was made to monitor every stage in the next cantilever and to ensure that joints were correctly aligned and epoxied, it was not possible to eliminate the first "dip." The free end of the new cantilever was high by an amount almost equal to that of the previous "dip."

Although it was discouraging not to have eliminated the misalignment, it was, nevertheless, encouraging to know that the care taken had resulted in a double cantilever that had behaved as expected—one end was a little low, the other end was a similar amount high.

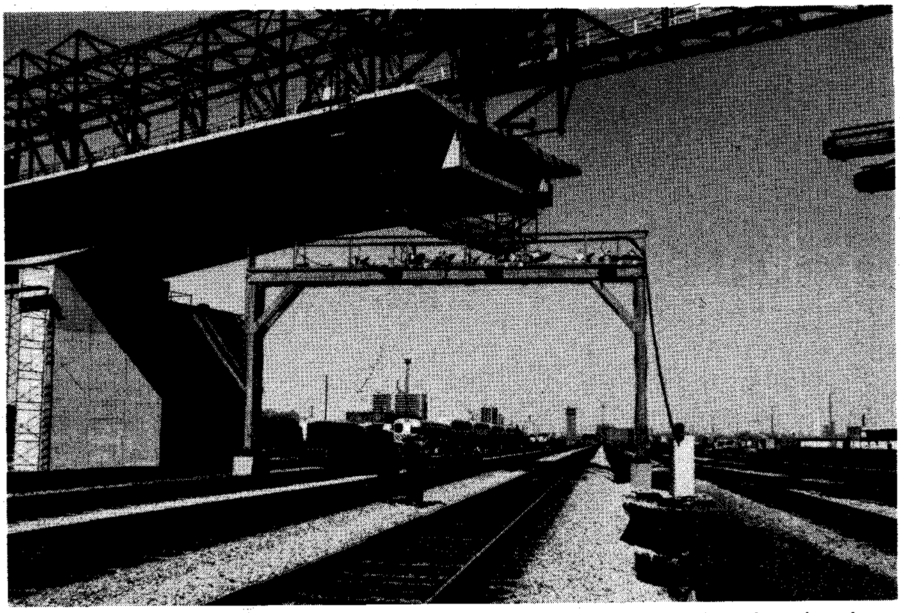


Fig. 12. Another construction shot of bridge over main line tracks. Note that signals on signal gantry have been lowered temporarily.

As progress continued and the "droop" method of estimating corrective measures was developed, the aberrations diminished to acceptable limits but never entirely disappeared.

The question of when and how much to shim was always a vexing problem that exercised the minds of the contractor's and engineer's field staffs on many occasions. The degree of precision finally attained showed that both the method adopted and the field cooperation were successful (see Appendix B).

A digression on the subject of epoxy adhesive is in order. The original intent was to provide a uniform contact pressure of 207 to 276 kPa (30 to 40 psi). In practice, in order to obtain a uniform pressure with the equipment being used, the pressures were mainly in the low end of that range. Had the epoxy always been fully hardened when the final stressing was applied, this would not have mattered. As it turned out, there were often occasions when the epoxy was slow in gaining strength so that when, rather than delay erection, it was decided to apply full prestress, secondary extrusion occurred. Around the outside and inside faces of the joints this extrusion was little more than a nuisance, i.e., the epoxy had to be cleaned off. However, in the sheath joints, it was a more serious problem.

After the initial extrusion, on tensioning the temporary holding devices, all sheaths were cleaned by causing an "obstruction" to be passed through the joint area several times. Typically, the "obstruction" was a metal blob on the end of a rod long enough to reach the joint in question. In this way, any extruded adhesive was spread thinly over the inside surface of the sheath. However, with the secondary extrusion, this was not always done, resulting in an inward corona of hardened epoxy. This obstruction right at the place where any kink or misalignment of the duct

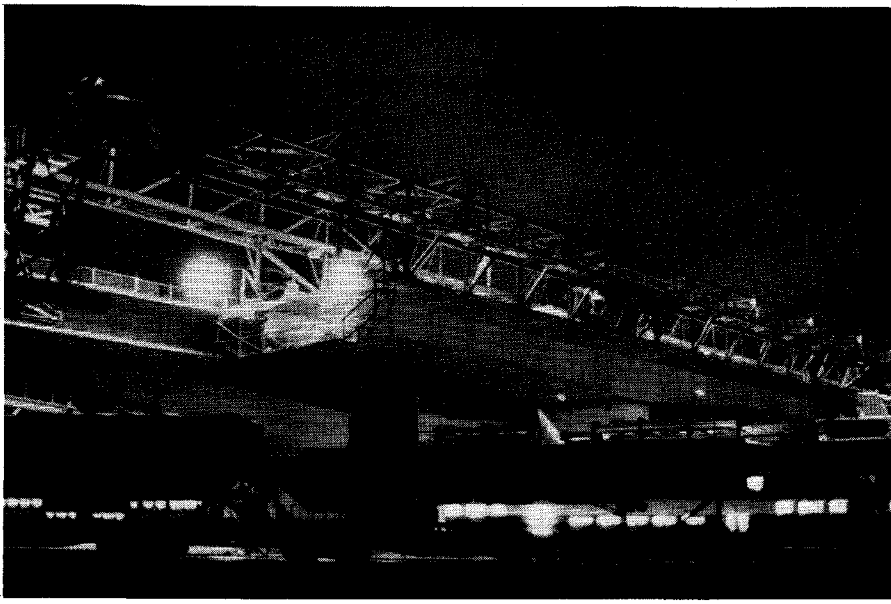


Fig. 13. Night construction.

is likely to occur, does little to improve the friction characteristics of the sheath.

When the double cantilever has reached full length, one end of it should be about 1.22 m (4 ft) from the end of the previously constructed cantilever. This gap is large enough to get a jack in place for the last cantilever tendons. It is at this point in the construction when the accuracy of the work becomes evident. The two cantilever tips should be ready for alignment.

As mentioned before, it may be necessary to rotate the new segment through a small angle either vertically, horizontally or both, or to move it bodily along the centerline of the bridge. By taking advantage of the unique flexibility of the launcher, this may be done relatively easily. If the movements are small, all three may be achieved by manipulating the rear legs.

Lateral movement, giving a horizontal rotation of the cantilever, may be obtained by screw jacks in the feet of the rear legs. Vertical movement may be obtained by extension of the hydraulic legs. Translation may be achieved by releasing the brakes on the wheels of the rear legs (bearing on the underside of the crane rail) and pulling the cantilever over its sliding bearings by means of the tensioning rods used for the temporary holding device during the erection of a new unit.

After the joint has been satisfactorily aligned, it is reinforced, tendon sheaths are installed and it is filled with concrete having a strength of 41.4 MPa (6 ksi). When the concrete has reached a strength of about 10.3 MPa (1.49 ksi), a small amount of prestress is applied across the joint to prevent thermal movements of the structure pulling the newly cast joint apart.

This initial partial prestress is applied by stressing some of the continuity tendons, the positive moment tendons that are threaded from the top of the units, through the bottom at midspan and back to the top again. Before initiating the second phase of the auto-launch of the launching truss, the remaining continuity tendons, up to half their total number, are stressed.

The new double cantilever is now firmly connected to the previous work and there is one more span to consider in the continuity calculations. The final stages of stress analysis indicated that it would be advantageous to stress only half the continuity tendons at the time of joining, leaving the remainder to be stressed when making the next joint.

The first move of the second stage of the auto-launch is to position the trolley, with its hydraulic legs again attached, at the forward tip of the new cantilever. The legs are lowered to take the weight of the launcher off the four feet that were stabilizing the newly completed cantilever. The rear legs are brought forward to be located at the launch point on the far side of the last pier. These legs are also lowered to take the weight off the rear feet (see Fig. 7).

At this stage, the launcher is supported only by the wheels, on top of the legs, bearing on the underside of the crane rail. As before, the trolley on the front legs attempts to move backwards with the result that the whole launching truss moves forward until, once again, the front fingers sit on their seats on the forward face of the next pier. The three hammerhead units are placed and the cycle repeats itself.

Those parts of the structure not erected in cantilever, i.e., at the abutments, were supported on falsework and are exceptions to the rule of no support from the ground. In each case, there was fortunately no problem with tracks and the segments were erected and glued together in much the same way as before. At the north end, where the length of the structure to be supported was ten segments, nine of them were supported on falsework and the remaining one was cantilevered out.

Figs. 11 through 17 show various stages of the segmental construction over the rail yard, including two night shots.

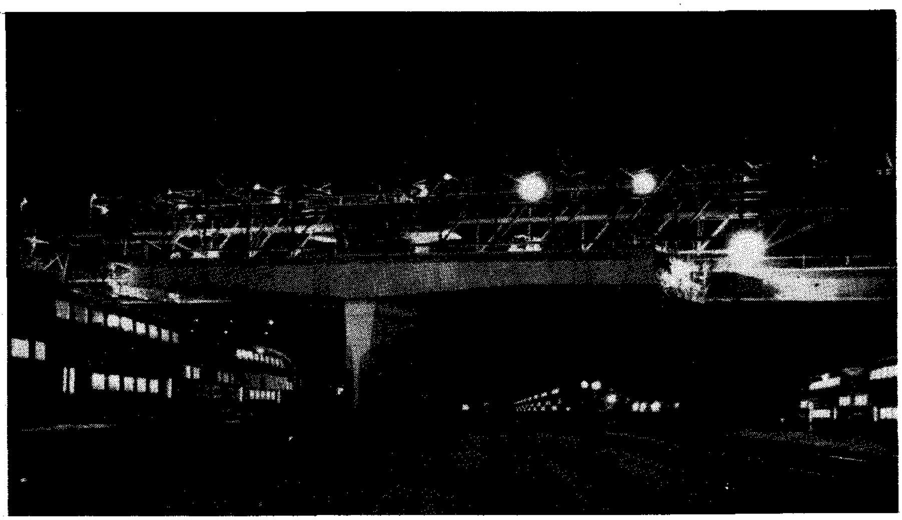


Fig. 14. Night construction.

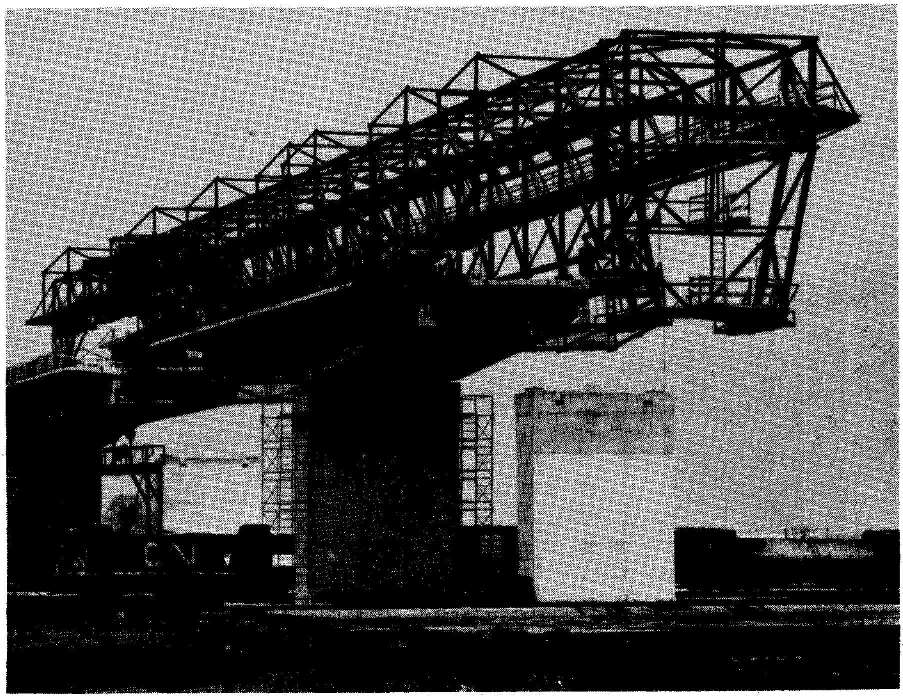


Fig. 15. Partially built cantilever balanced by truss on its permanent bearings.

## Problems and Solutions During Construction

One serious restraint on the contractor's operations was the busy train schedule. Over most of the yard, tracks were not dedicated to an inflexible workload and liaison with the railway operating personnel usually provided ample working time over these tracks. Over the three mainline tracks, however, this flexibility was not available. Not only is normal railway traffic heavy, but the morning and evening commuter traffic compounds the problem. Unfortunately, no work over the tracks could be allowed during a large portion of the working day.

Any work that was not confined to the top deck area between the side safety fences (temporary erections at the extreme edges of the deck) or to within the structure itself was prohibited. Movements of the launcher were not allowed and movement of the travelling crane itself were viewed with alarm by the railway safety personnel at track level. The times covering these prohibitions were from 6:30 to 9:30 a.m. and from 3:30 to 5:30 p.m.

At other times, erection could proceed with prior arrangement with the railway operating staff. Radio control of bridge operations was maintained at all times by means of "walkie-talkies" carried by the resident engineering staff. The railways had their own radio system. All that was necessary was to have a railway and a resident engineering representative within talking distance of each other and cooperation was assured.

Schedules were arranged so that a segment would be erected over the

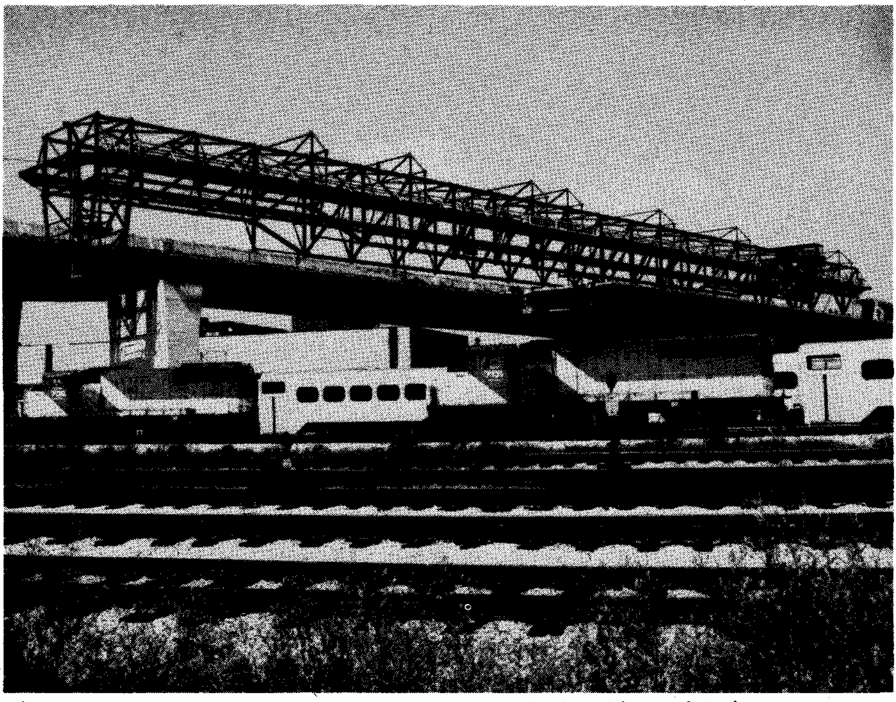


Fig. 16. Construction of second structure over parked rapid transit trains.

tracks in a "safe" period and the balancing segment, on the other side of the pier, could be erected during the prohibited period. Stressing, however, would have to wait until the next "safe" period.

In general, the arrangement worked well even if, from the contractor's viewpoint, frustratingly slowly. The second structure went up even more smoothly. It did require some work at night and very early in the morning. Floodlighting on the launcher helped but was obviously no substitute for daylight and such night work was avoided whenever possible.

Application of the epoxy adhesive required careful surveillance. The components were mixed at the time the glue was required. It was applied by hand, using protective gloves. Both mating surfaces were coated. It was assumed that using this method would ensure complete and generous application such that after the segments were pulled together, the joint would be completely filled. This proved not to be the case when firstly, rain seeped through some joints in the top slag and secondly when "manifolding" took place during grouting of the tendon ducts.

A possible reason for the latter is the close spacing of the tendon ducts. At a center-to-center spacing of 127 mm (5 in.) with a duct 81 mm (3.2 in.) in diameter, there is little space between the ducts and this leads to a susceptibility to honeycombing. In general, the concrete had a relatively smooth surface finish but in some cases there was a surface deterioration between the ducts. Although the persons applying the epoxy were asked to pay particular attention to these surface areas, they were not always to-

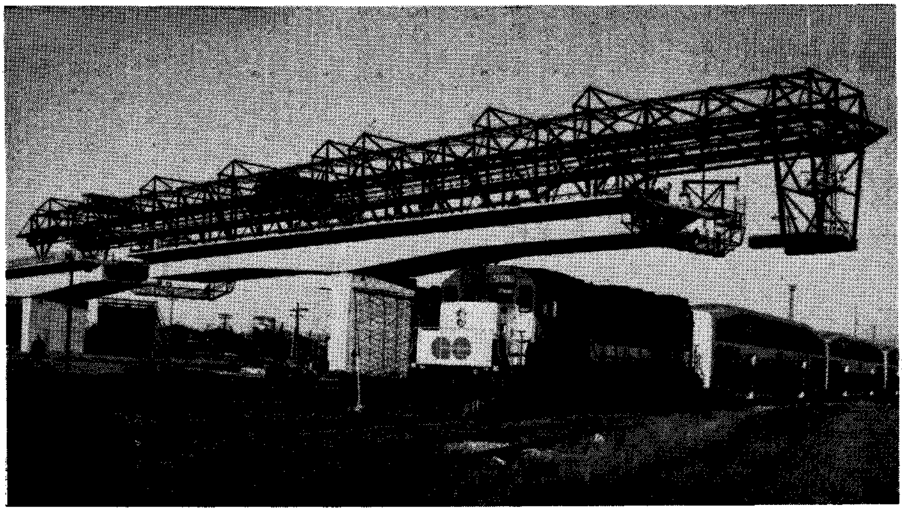


Fig. 17. First full size cantilever nearing construction, rapid transit trains parked underneath.

tally diligent. It is therefore probable that a grout path between one duct and another was provided this way. Whatever the reason, interconnecting grout paths between ducts, or "manifolding," did exist and were a nuisance.

The cantilever tendons were mostly anchored in the webs. A block-out was provided for the Freyssinet anchor and this was entirely on one side of a joint. When the adjacent segment was attached, a grout pipe was pushed through the hole in the male cone and carried back into the interior of the segment by means of a prepared notch in the inside of the web wall. It was assumed that the block-out would be filled with grout during the tendon grouting operation. This proved not to be the case.

A damp joint aroused the suspicions of an inspector who ordered a small hole to be drilled into the block-out. Water emerged from the drill hole. A thorough inspection of both structures provided other examples of water in the block-outs. Since the weather was now fairly consistently below freezing, it was important to ensure complete drainage.

The apparent remedy was to mortar up the block-out as soon as the tendon has been stressed, anchored, inspected and cut-off. If the mortar is not fully hardened at the time of attaching the next segment, no harm will come, provided the finished face of the mortar is not proud.

The Islington Bridge consists of two separate bridges whose supports are staggered about 6.25 m (20.5 ft) in the direction of the bridge. This is dictated by the need to accommodate the tracks below whose skew angle is 24 deg. There is a central 1.22 m (4 ft) wide median with a longitudinal joint about 25 mm (1 in.) wide running the length of the structure. This geometry was used since any separation between the structures would require a wider right-of-way and an extra two sets of handrails.

It is impracticable to place precast units within 25 mm (1 in.) of one another. A wider gap, 0.3 m (1 ft), was therefore left between the precast units which was later reduced to 25 mm (1 in.) when the median slabs were cast.

Because the profile of the finished bridge was not perfect due to the small deviations that have already been discussed, and because deflections of adjacent points at the central longitudinal joint will not behave in quite the same way (they are at different locations in their own span, due to skew) there is frequently an elevation discontinuity across the median at the joint. These differences in elevation are small, not too noticeable and probably inevitable. A wider separation would appear to be the best solution.

The problem of "manifolding," where grout is pumped into one duct and comes out of another, has been mentioned. It is possible that the Islington Avenue Bridge achieved a world "first" when it managed to produce tendon manifolding. The tendons involved were in the north approach span of one of the structures only. They were secondary tendons in that they performed no permanent function but were used for temporary stressing only. When the tendon was threaded through its duct, it came out of the wrong hole at the other end. Fortunately, the problem was easily solved and no harm occurred. Neither

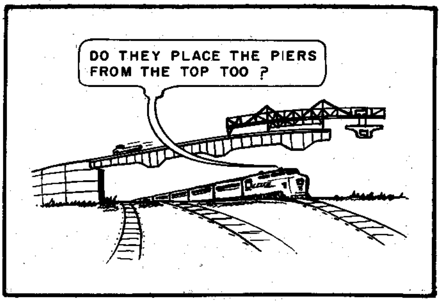
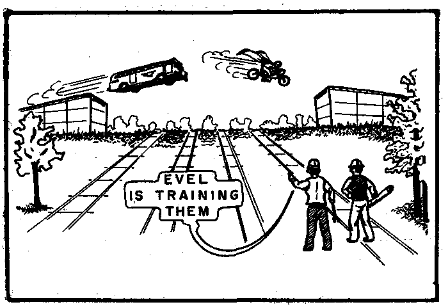
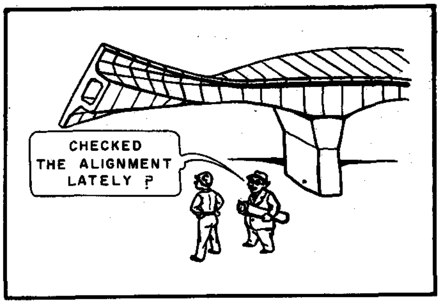



Fig. 18. Aerial view looking east at finished structure.




the contractor nor the inspectors had noticed the transposition of two ducts!

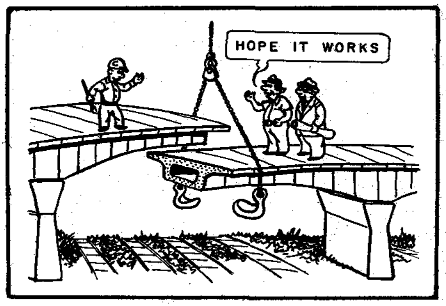
In a more serious vein, however, it is thought that the Islington Avenue Bridge can claim a much more creditable "first" when the contractor added eight segments to the structure in one working day. It was, indeed, a long day!

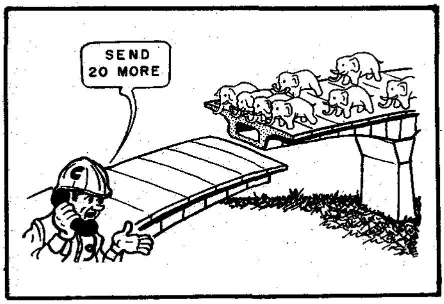
Figs. 18 and 19 show two aerial views of the completed bridge.

## Concluding Remarks

The Islington Avenue Bridge offers no great novelty so far as segmental bridge technology is concerned, although its launching truss was innovative. It is significant in its own locality because it is the first one to be completed in the Province of Ontario. The Islington Bridge is the first Cana-

Fig. 19. Aerial view looking northwest at finished structure.




dian segmental structure to be built in an urban area and over a very busy railway yard.

This bridge is probably one of the last to be designed by hand. Computers were used as tools. Programmable hand calculators did a great deal of the work.

However, the design was given an independent check using one of the large computer programs and found to be satisfactory. Most of the original design was pulled together by hand.

An future design would almost certainly be carried out by computer with only small amounts of work done by hand.

The bridge was opened to traffic on December 7, 1979, and for the past 6 months has been performing with total satisfaction.

## Acknowledgment

The author wishes to express his appreciation to W. Jansen for permission to use his photographs in this article.

The author is also very grateful to Robert L. Jefferies for the use of his cartoons, which bring welcome relief to a job that at times can be vexing.

## Credits

Structural Engineers: Planmac Consultants Ltd., Toronto, Ontario.

General Contractor: Pitts Engineering and Construction Ltd., Toronto, Ontario.

Precasting Subcontractor: Beer Precast Concrete Ltd., Toronto, Ontario.

Owner: Municipality of Metropolitan Toronto, Ontario.

## References

1. *Precast Segmental Box Girder Bridge Manual,* Prestressed Concrete Institute, Chicago, Illinois, 1978, 124 pp.
2. Priestley, M. J. Nigel, "Design of Concrete Bridges for Temperature Gradients," *ACI Journal,* Proceedings V. 75, No. 5, May 1978, pp. 209-217.

**NOTE:** Detailed sample calculations for determining temperature stresses, droop curves, and moment distribution are given on the following pages in Appendices A, B, and C, respectively.

* * *

# APPENDIX A—SAMPLE CALCULATION OF TEMPERATURE STRESSES

All calculations given in this and the following appendices are in metric (SI) units; a table of conversion factors is included herein.

The strain in the concrete due to a 10 deg C temperature change is:

$$10 \times 12 \times 10^{-6} = 12 \times 10^{-5}$$

which induces a stress equal to:

$$12 \times 10^{-5} \times 34{,}220 = 4.106 \text{ MPa}$$

The temperature stresses are now calculated assuming a uniform stress block (Case I) and a triangular stress block (Case II).

## Case 1: Uniform stress block

With reference to Fig. A1, the force in the concrete (considering a total section) is:

$$4.106 \times 2.1155 \times 2 = 17.372 \text{ MN}$$

The moment of this force about the centroid of the section is:

$$17.372\ (0.879 - 0.162) = 12.456 \text{ MN} \cdot \text{m}$$

Since there are no external forces, the induced force of 17.372 MN must be resisted by an equal force at the centroid of the section. Therefore:

Top fiber stress:

$$-4.106 + \frac{17.372}{7.645} + \frac{12.456}{6.728} = 0.018 \text{ MPa (tens)}$$

**Conversion Factors**

| | |
|---|---|
| 5 deg C | = 9 deg F |
| 1 mm | = 0.03937 in. |
| 1 m | = 3.281 ft |
| 1 MPa | = 145 psi |
| 1 MN | = 2248 kips |
| 1 MN • m | = 737.6 ft-kips |

Bottom fiber stress:

$$\frac{17.372}{7.645} - \frac{12.456}{4.204} = -0.691 \text{ MPa (comp)}$$

## Case II: Triangular stress block

With reference to Fig. A2, and assuming that $p$ is the maximum stress, take moments about the base of the haunch (use enclosing rectangle and deduct areas as shown in Table A1).

Dividing the summation of the respective moments and forces, the centroid is:

$$\bar{x} = \frac{0.454\,p}{1.365\,p} = 0.333 \text{ m}$$

or measured from the top:

$$0.457 - 0.333 = 0.124 \text{ m}$$

Therefore, the total force is:

$$1.365 \times 4.106 \times 2 = 11.209 \text{ MN}$$

Moment of this force about centroid:

$$11.209\ (0.879 - 0.124) = 8.463 \text{ MN} \cdot \text{m}$$

Therefore, top fiber stress:

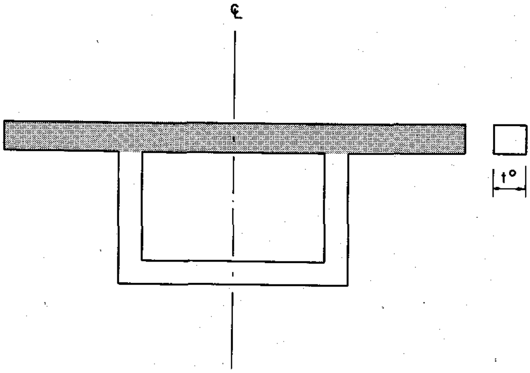
$$-4.106 + \frac{11.209}{7.645} + \frac{8.463}{6.728} = -1.382 \text{ MPa (comp)}$$

and bottom fiber stress:

$$\frac{11.209}{7.645} - \frac{8.463}{4.204} = -0.547 \text{ MPa (comp)}$$

The above results are for a section in the constant depth part of a simple span.

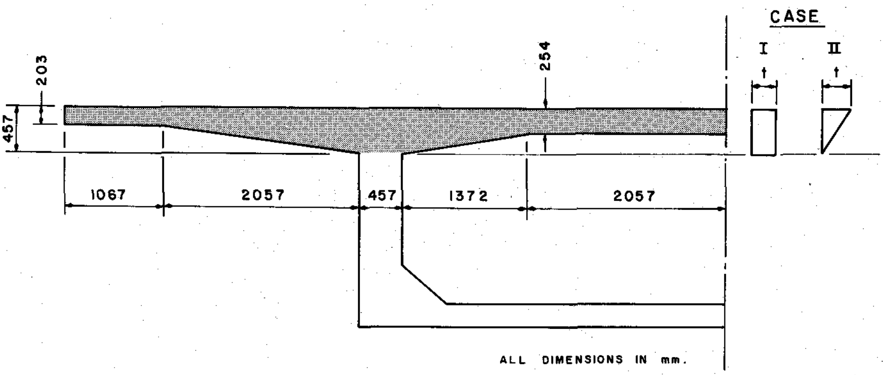
For the complete structure, the effects of continuity have to be considered. In order to distribute moments across all spans, the fixed-end moments (FEMs) caused by the thermal stress effects were calculated. To do this, recourse was made to the classical methods of the $M/EI$ diagrams. Since $E$ is assumed to be constant, the values of $M/I$




ASSUMPTIONS :

AREA OF TOP SLAB ( HATCHED ) IS
AT A CONSTANT TEMPERATURE
DIFFERENCE t° ABOVE OR BELOW
REST OF BOX .

Fig. A1. Simplified example of temperature variation through bridge segment.




ASSUMPTIONS :

AREA OF TOP SLAB ( HATCHED ) IS SUBJECTED TO
A TEMPERATURE DIFFERENCE OF :-

CASE I , CONSTANT VALUE t°
CASE II , VALUE VARYING LINEARLY FROM t° AT OUTER
FIBRE TO ZERO AT BASE OF SLAB

| | |
|---|---|
| MAX. TEMPERATURE DIFFERENTIAL = t | 10 °C |
| COEFFICIENT OF LINEAR EXPANSION | $12 \times 10^{-6}$ / °C |
| $E_c$ | 34220 MPa. |
| SECTION PROPERTIES AT MID - SPAN : | |
| AREA OF CROSS - SECTION | 7·645 $m^2$ |
| CENTROID FROM TOP FIBRE | 0·879 m . |
| TOP SECTION MODULUS | 6·728 $m^3$ |
| BOTTOM SECTION MODULUS | 4·204 $m^3$ |

Fig. A2. Practical example of temperature variation through bridge segment.

Table A1. Summary of moments using triangular stress block.

| Quantity | Force | Lever arm | Moment |
|---|---|---|---|
| $(p/2)\ (0.457)(7.010)$ | $1.602\,p$ | $0.457\ (2/3)$ | $0.488\,p$ |
| $(p/2)\left(\frac{254}{457}\right)(0.254)\ (1.067)$ | $-0.075\,p$ | $0.254\ (2/3)$ | $-0.013\,p$ |
| $(p/2)\left(\frac{254}{457}\right)(0.254)\left(\frac{2.057}{3}\right)$ | $-0.048\,p$ | $0.254\ (1/2)$ | $-0.006\,p$ |
| $(p/2)\left(\frac{203}{457}\right)(0.203)\ (2.057)$ | $-0.093\,p$ | $0.203\ (2/3)$ | $-0.013\,p$ |
| $(p/2)\left(\frac{203}{457}\right)(0.203)\left(\frac{1.372}{3}\right)$ | $-0.021\,p$ | $0.203\ (1/2)$ | $-0.002\,p$ |
| | $\Sigma = 1.365\,p$ | | $\Sigma = 0.454\,p$ |

Table A2. Summary of section properties and moments for one end span.

| Distance | $\bar{y}$ | $M$ | $I$ |
|---|---|---|---|
| 0 | 1.815 | 18.954 | 22.387 |
| 1.524 | 1.726 | 17.957 | 20.350 |
| 3.048 | 1.627 | 16.847 | 18.201 |
| 4.877 | 1.508 | 15.513 | 15.777 |
| 7.010 | 1.367 | 13.933 | 13.156 |
| 9.144 | 1.224 | 12.330 | 10.754 |
| 11.582 | 1.059 | 10.480 | 8.267 |
| 14.173 | 0.879 | 8.463 | 5.914 |
| 16.840 | 0.851 | 8.149 | 5.488 |
| 19.507 | 0.824 | 7.846 | 5.079 |
| 20.726 | 0.811 | 7.701 | 4.898 |
| 23.393 | 0.784 | 7.398 | 4.515 |
| 26.365 | 0.757 | 7.095 | 4.149 |
| 28.727 | 0.730 | 6.793 | 3.800 |
| 31.394 | 0.703 | 6.490 | 3.468 |
| 34.061 | 0.676 | 6.187 | 3.153 |
| 36.728 | 0.650 | 5.896 | 2.854 |
| 39.395 | 0.623 | 5.593 | 2.571 |
| 42.062 | 0.597 | 5.302 | 2.305 |
| 44.729 | 0.571 | 5.010 | 2.055 |
| 47.396 | 0.545 | 4.719 | 1.820 |
| 49.073 | 0.529 | 4.540 | 1.681 |
| FEM L = 12.4105<br>FEM R = 3.8477<br>CO L→ R = 0.3160<br>CO R→ L = 0.9178<br>Stiff. L = 0.6505<br>Stiff. R = 0.2253 | | | |

for each segment joint were calculated, and by using straight lines between each value of $M/I$, areas and moments of areas were calculated by using semigraphical integration.

This admittedly tedious task was reduced to manageable proportions by the use of programmable hand calculators. Moments were calculated as follows:

For Case II: $11.209\ (\bar{y} - 0.124)$ MN • m where $\bar{y}$ is the depth of centroid from the top fiber. (Section properties of every joint were, of course, known.)

A program is available which, given a progressive distance from one end of a beam, the moment existing at that section and the moment of inertia of that section, will produce fixed-end moment left (FEM L), FEM R, carry-over factor left to right (CO factor L-R), CO factor R-L, and the stiffness at each end.

One end span is demonstrated (Table A2); the same results for other spans are merely stated.

For the short (61 m) span:

FEM L = 9.7390; FEM R = 9.7390

For the standard (83 m) span:

FEM L = 9.3357; FEM R = 9.3357

It has already been stated that the design was performed by hand and various artifices were used to simplify the calculations. One such artifice was used to distribute the moments caused by thermal effects across the full structure.

Table A3. Support moments for a central unit point load in each span (refer to Fig. A3).*

| Description | $M_A$ | $M_B$ | $M_C$ | $M_D$ | $M_E$ | $M_F$ |
|---|---|---|---|---|---|---|
| Span 1 | −7.284 | 2.046 | −0.672 | 0.221 | −0.073 | 0.027 |
| Span 2 | −5.966 | −4.906 | 1.612 | −0.530 | 0.176 | −0.065 |
| Span 3 | 2.994 | −9.246 | −7.316 | 2.407 | −0.801 | 0.294 |
| Span 4 | −0.983 | 3.036 | −7.946 | −7.753 | 2.579 | −0.945 |
| Span 5 | 0.322 | −0.993 | 2.599 | −7.773 | −7.897 | 2.894 |
| Span 6 | −0.101 | 0.314 | −0.821 | 2.455 | −7.458 | −8.816 |
| Span 7 | 0.026 | −0.083 | 0.216 | −0.647 | 1.965 | −5.980 |

*Note that values obtained in this table are obtained from influence lines. Hence, the signs are based on a downward loading. Note also that the unit loads act at the centerline of each span (Spans 1 to 7).

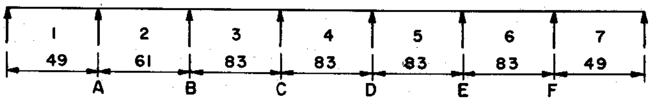



Fig. A3. Seven-span structure showing support (refer to Table A3).

Table A4. Distribution of fixed-end moments in seven-span structure (refer to Fig. A3 and Table A3).

| Description | $M_A$ | $M_B$ | $M_C$ | $M_D$ | $M_E$ | $M_F$ |
|---|---|---|---|---|---|---|
| 1.1342 × Line 1 | 8.2615 | −2.3206 | 0.7622 | −0.2507 | 0.0828 | −0.0306 |
| 1.0731 × Line 2 | 6.4021 | 5.2646 | −1.7298 | 0.5687 | −0.1889 | 0.0698 |
| 0.7841 × Lines 3 to 6 | −1.7501 | 5.4017 | 10.5728 | 8.3616 | 10.6457 | 5.1539 |
| 1.1342 × Line 7 | −0.0295 | 0.0941 | −0.2450 | 0.7338 | −2.2287 | 6.7825 |
| Σ | 12.8840 | 8.4398 | 9.3602 | 9.4134 | 8.3109 | 11.9756 |

A commercially available computer program was used to produce influence lines for moment in the seven-span structure (Fig. A3). Table A3 shows the support moments for a central unit point load in each span.

In interior spans, FEMs for a central load are equal for any given span. Similarly, the FEMs for temperature effects in the interior spans are equal at each end of the span. (In general, this statement applies equally to cantilever and continuity tendons, dead load, creep and shrinkage, and all these effects were treated similarly.) If the FEMs for the central point loads can also be calculated, the ratio FEM temp to FEM unit load may be made to modify the values of the first table to obtain the second (see Table A4).

Using the same technique as described above, the FEMs of the central unit point loads are:

End span: FEM L = 11.1532; FEM R = 3.1623

Short span: FEM L = FEM R = 9.0757

Standard span: FEM L = FEM R = 11.9070

In the end span, FEM R becomes zero on release. Hence, the same result for distribution can be obtained by modifying FEM L thus:

New FEM L = FEM L + FEM R × CO R to L

Table A5. Summary of stresses for Cases I and II.*

| Loading condition | Case I | | Case II | |
|---|---|---|---|---|
| Stress condition at center span | Simple span | Distributed | Simple span | Distributed |
| Top fiber† | 0.018 | Not calculated here | −1.382 | −2.581 |
| Bottom fiber† | −0.691 | | −0.547 | 1.371 |

*See also Table 1 in early part of text.
†Negative sign denotes compression.

Thus: Thermal FEM L =
$12.4105 + (3.8477)(0.9178) = 15.9419$
Central unit load FEM L =
$11.1532 + (3.1623)(0.9178) = 14.0556$

Therefore, the ratio of FEM applied load to FEM unit point load are:

End span: $\dfrac{15.9419}{14.0556} = 1.1342$

Short span: $\dfrac{9.7390}{9.0757} = 1.0731$

Standard span: $\dfrac{9.3357}{11.9070} = 0.7841$

Check the stresses at the centerline of Span DE.
Simple span moment:

$$11.209\,(0.879 - 0.124) = 8.463 \text{ MN} \cdot \text{m}$$

Resultant moment:

$$-8.463 + \frac{(9.4134 + 8.3109)}{2} = 0.399 \text{ MN} \cdot \text{m}$$

Top fiber stress:

$$-4.106 + \frac{11.209}{7.645} + \frac{0.399}{6.728} = -2.581 \text{ MPa (comp)}$$

Bottom fiber stress:

$$\frac{11.209}{7.645} - \frac{0.399}{4.204} = 1.371 \text{ MPa (tens)}$$

Table A5 summarizes the temperature stresses for Case I (uniform stress block) and Case II (triangular stress block). From this table and the above sample calculations, the values in Table 1 can be compiled.

# APPENDIX B—EXAMPLE OF DROOP CURVE CALCULATION, PIER B, EAST STRUCTURE

A form similar to that shown in Table B1 was used in the field. It was supplied with the first three columns completed; the remainder were filled in by field staff.

The spans flanking Pier B (shown in Fig. B1) were selected for several reasons:

1. Field staff were convinced that all cantilevers drooped too much. Shims were therefore applied at a first joint—with rather greater success than was anticipated.

2. At Nodes 37-59, shims were again applied to increase the droop with satisfactory results. The previous node would have been shimmed had the delay for the hardening of the epoxy not been too inconvenient because of train schedules.

3. The third set of shims at Nodes 35-61 illustrate the maddening perversity of the shimming process, i.e., the droop increased instead of decreasing as planned.

4. The story had a happy ending with a residual "error" of about 5 mm (less than 0.2 in.). It was not always thus; this span started with an error of 24 mm (nearly 1 in.) Happily, most of that was eliminated, as shown. Sometimes, in spite of all efforts, shimming did not eliminate all of the droop "error."

Table B1. Form used to monitor droop (see Fig. B1).

| Node | Mean droop | Launcher legs | Previous span error | Corrected droop, mm | Measured droop, mm |
|---|---|---|---|---|---|
| 48 | 0 | 0 | 0 | 0 | 0 |
| 47-49 | 0 | 0 | − 0.27 | 0 | 0 |
| 46-50 | − 0.24 | − 0.03 | − 0.89 | − 1 | 0 |
| 45-51 | − 2.90 | − 0.06 | − 1.51 | − 4 | 0 |
| 44-52 | − 5.73 | − 0.12 | − 2.13 | − 8 | 0 |
| 43-53 | − 9.48 | − 0.18 | − 2.75 | − 12 | − 3 |
| 42-54 | − 14.05 | − 0.21 | − 3.38 | − 18 | − 3 |
| 41-55 | − 21.61 | − 0.30 | − 4.26 | − 26 | 0 |
| 40-56 | − 30.33 | − 1.04 | − 5.14 | − 37 | − 6 |
| 39-57 | − 40.54 | − 1.28 | − 6.02 | − 48 | − 14 |
| 38-58 | − 52.64 | − 3.11 | − 6.90 | − 63 | − 23 |
| 37-59 | − 67.36 | − 6.25 | − 7.78 | − 81 | − 49 |
| 36-60 | − 85.37 | − 7.35 | − 8.67 | −101 | − 82 |
| 35-61 | −107.72 | −12.80 | − 9.55 | −130 | −116 |
| 34-62 | −139.45 | −14.66 | −10.43 | −165 | −140 |
| 33-63 | −180.50 | −16.52 | −11.31 | −208 | −210 |
| 32-64 | −232.71 | −18.38 | −12.19 | −263 | −258 |

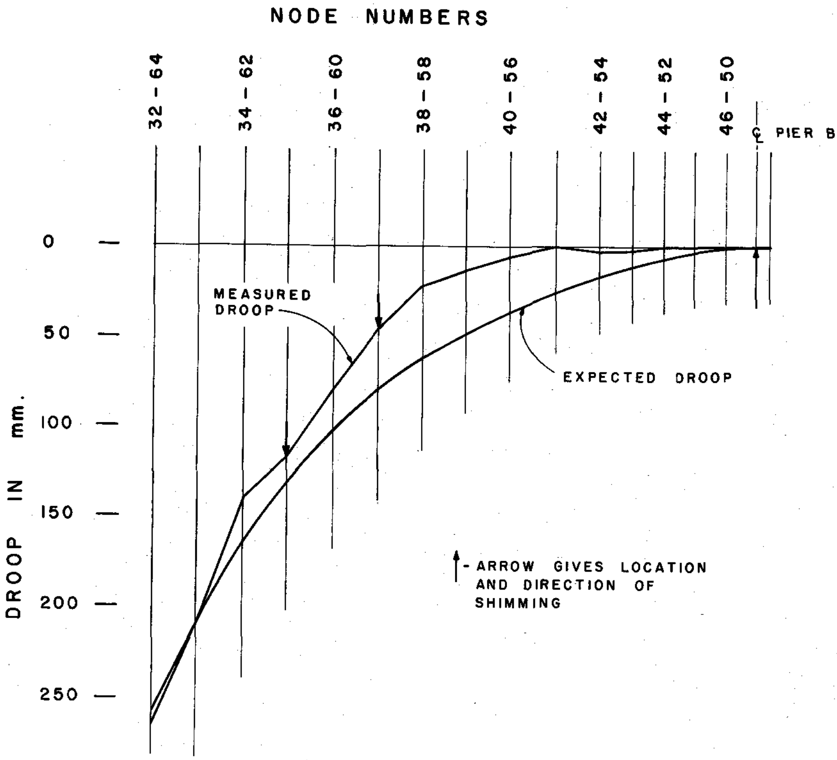



Fig. B1. Measured and predicted droop (see also Table B1).

# APPENDIX C—SAMPLE CALCULATION SHOWING MOMENT DISTRIBUTION IN CONTINUOUS STRUCTURE

Mention has been made of distributing moments across a structure whose number of spans is constantly growing. It has also been stated that the Islington Avenue Bridge was not designed with the aid of any of the large bridge computer programs that are now becoming available. Recourse was therefore made to an extension of the artifices described in Appendix A.

Fortunately, most of the forces acting upon the structure are symmetrical, e.g., dead loads from self-weight to asphalt road surface, cantilever post-tensioning, continuity tendons, and thermal effects. All these effects produce symmetrical fixed-end moments. The tensioning of the continuity tendons is an example of staged construction that will be followed here.

The stressing of continuity tendons, which takes place in an end span (the leading end span), not only causes secondary moments throughout the structure, but it causes the leading free cantilever to rotate downwards; this is of prime interest in the vertical alignment calculations.

When the continuity tendons are tensioned at the time of the first closure strip in Span 1, the deck cambers up between the piers but since it is a single simple span, it is statically determinate. Therefore, the moments are calculated without difficulty. Of course, $M_A$, the moment at the leading support (Pier A) is zero.

When the closure strip in Span 2 is made, it is in the leading span of a two-span structure and $M_B = 0$. Similarly, when

Table C1. Support moments in seven-span structure.

| Description | $M_A$ | $M_B$ | $M_C$ | $M_D$ | $M_E$ | $M_F$ |
|---|---|---|---|---|---|---|
| Unit load on Span 3* | 2.994 | − 9.246 | −7.316 | 2.407 | − 0.801 | 0.294 |
| Unit load on Span 4* | −0.983 | 3.036 | −7.946 | − 7.753 | 2.579 | − 0.945 |
| Modified load on Span 4* | 0.905 | − 2.795 | 7.316 | 7.138 | − 2.375 | 0.870 |
| Σ | 3.899 | −12.041 | 0 | 9.545 | − 3.176 | 1.164 |
| Unit load on Span 3* | 2.994 | − 9.246 | −7.316 | 2.407 | − 0.801 | 0.294 |
| Modified load on Span 6* | 0.900 | − 2.798 | 7.316 | −21.877 | 66.458 | 78.560 |
| Σ | 3.894 | −12.044 | 0 | −19.470 | 65.657 | 78.854 |

*Note that the unit loads are applied at the centerline of the span.

Table C2. Staged construction: Moment distribution.*

| Description | $M_A$ | $M_B$ | $M_C$ | $M_D$ | $M_E$ | $M_F$ |
|---|---|---|---|---|---|---|
| Unit load on Span 1* | 0 | 0 | 0 | 0 | 0 | 0 |
| Unit load on Span 2† | −7.555 | 0 | 0 | 0 | 0 | 0 |
| Unit load on Span 3† | 3.899 | −12.041 | 0 | 0 | 0 | 0 |
| Unit load on Span 4† | −1.304 | 4.026 | −10.538 | 0 | 0 | 0 |
| Unit load on Span 5† | 0.429 | − 1.325 | 3.468 | −10.373 | 0 | 0 |
| Unit load on Span 6† | −0.139 | 0.436 | − 1.139 | 3.409 | −10.355 | 0 |
| Unit load on Span 7† | 0.026 | − 0.083 | 0.216 | − 0.647 | 1.965 | −5.980 |

*Moments are in meter units.
Signs are for downward load.
†Note that the unit loads are applied at the centerline of the span.

Table C3. Moment distribution in spans due to continuity tendons and central unit point loads.

| Description | Continuity tendons, MN • m | Central unit point load, MN • m | Ratio of $\frac{\text{Continuity tendon}}{\text{Point Load}}$ |
|---|---|---|---|
| End span* | 5.2625 | 14.0614 | 0.3743 |
| Short span | 11.4563 | 9.0757 | 1.2623 |
| Standard span | 11.8516 | 11.9070 | 0.9953 |

*Note that the moment given is the propped cantilever moment at the first support.

Span 3 is closed, it is the leading span of a three-span structure, and $M_C = 0$.

Consider the above case: from Table A3 in Appendix A, the moments at each support for a central point load in Span 3 are as shown in Table C1. If a load, not necessarily a unit load, be applied simultaneously in a span beyond Span 3, such as Span 4, so that the value of $M_C$ for that condition was equal and opposite to the $M_C$ value above, then the resultant $M_C$ would become zero.

A zero moment at a support in a continuous structure is tantamount to a hinge, and the structure becomes discontinuous at that support—resulting in a three-span continuous structure with a central unit point load in Span 3. This is analogous to the condition under discussion; closure in Span 3.

Table C4. Staged support moments due to stressing continuity tendons*.

| Continuity tendons stressed in | $M^A$ | $M^B$ | $M^C$ | $M^D$ | $M^E$ | $M^F$ |
|---|---|---|---|---|---|---|
| Span 2<br>Line 2 × 1.2623 | 9.537 | | Moment contribution this stage → 1.298<br>Sum to date → 6.954 | | | |
| | 9.537 | | | | | |
| Span 3<br>Line 3 × 0.9953 | −3.881 | 11.984 | | | | |
| | 5.656 | 11.984 | | | | |
| Span 4<br>Line 4 × 0.9953 | 1.298 | − 4.007 | 10.488 | | | |
| | 6.954 | 7.977 | 10.488 | | | |
| Span 5<br>Line 5 × 0.9953 | −0.427 | 1.319 | − 3.452 | 10.324 | | |
| | 6.527 | 9.296 | 7.036 | 10.324 | | |
| Span 6<br>Line 6 × 0.9953 | 0.138 | − 0.434 | 1.134 | − 3.393 | 10.306 | |
| | 6.665 | 8.862 | 8.170 | 6.931 | 10.306 | |
| Span 7<br>Line 7 × 0.3743 | −0.010 | 0.031 | − 0.081 | 0.242 | − 0.735 | 2.238 |
| | 6.655 | 8.893 | 8.089 | 7.173 | 9.571 | 2.238 |

*Line numbers in the table above refer to lines in Table C2.
Signs are correct for continuity tendons.

The second line in Table C1 gives the values for Span 4. Change the magnitude of the load to produce $M_C = 7.316$, i.e., multiply throughout by $-7.316/7.946 = -0.9207$. In theory it does not matter which span is used for the modifying force, provided it is to the right of Pier C.

It must be that way to maintain only one load in the three spans under consideration; a central unit point load in Span 3.

To check with a load in Span 6: factor loads using $-7.316/-0.821 = 8.9111$. In practice, there are small differences in values because of normal arithmetic inaccuracies during the calculations of moment influence lines for a seven-span structure. The values in the table that are to the right of $M_C$ are of no interest. Using the procedure described above, Table C2 was obtained.

Using small in-house computer programs, the force in each tendon at each joint was calculated. The horizontal components of the forces and their distances from the neutral axis were used to compute the moment at each joint. As in Appendix A, these moments and the known section properties at each joint were used to calculate the fixed-end moments produced by stressing the continuity tendons.

The corresponding fixed-end moments for unit loads at centerline of each span were similarly calculated. Using these techniques, the values in Table C3 could be obtained.

Once the support moments are known, the actual moment at each joint in the structure can be calculated (see Table C4). Thereafter, together with the known section properties, the moments may be used to calculate deflections and rotations at each joint. It is the rotation at the forward free cantilever support that is of interest when predicting the behavior of that cantilever before closure.

Apart from the computer program used to calculate tendon forces, all the above calculations were done with programmable hand calculators.

* * *

Discussion of this paper is invited.
Please forward your comments to
PCI Headquarters by January 1, 1981.