Special Report

Aerial Guideway for the Vancouver ALRT Project

- **Part 1 — Design Overview**
  by Terry A. Nettles

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  by Paul A. R. Lowe

One of the most striking features of the nearly C$1 billion Vancouver Advanced Light Rapid Transit (ALRT) system is the aerial guideway. This 16 km (10 miles) long ribbon of precast and prestressed concrete follows the curvature of the track profile with long spans, in-depth crossheads and a minimum visual impact on the urban areas through which it passes.

In Part 1 of this report, the project engineer provides the background leading to the selection of the structural system and then gives an overview of the design features of the structure and the major precast prestressed components.

In Part 2, the precaster discusses the production, quality control, post-tensioning operations, transportation, and erection of the more than 1000 precast prestressed beams used in the project.

The aerial guideway system is now fully operational.

Part 1 — Design Overview

by

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The "SkyTrain" automated rail rapid transit system (Fig. 1) in Vancouver, British Columbia, Canada, was opened to revenue service on January 3, 1986, 3 months ahead of the original schedule. The system was designed, constructed, acceptance tested, and operated for 2 months of free public ridership in less than 5 years. It is a 21.4 km (13.4 mile), dual lane single route system running from the Burrard Inlet waterfront in downtown Vancouver southeasterly to the city of New Westminster on the Fraser River. A southern extension crossing the Fraser River and extending to the municipality of Surrey will open in 1989 (Fig. 2). Extensions to other municipalities in the greater Vancouver area are also planned, at which time the system will become a branched network.

From the SeaBus Terminal on the Burrard Inlet through the heart of downtown Vancouver, the system is double tiered for 1.3 km (0.8 mile) in a refurbished railway tunnel. The relatively small height of the vehicles and lowering the invert of the tunnel by 1 m (3.25 ft) made double tiering possible in the previously abandoned, single track tunnel. The system emerges above ground near BC Place Stadium and is then elevated throughout on a concrete guideway, except for approximately 3.5 km (2.2 miles) where the line is at grade. The total length of the elevated route is 16.6 km (10.4 miles). The original system has 15 stations. There are two in the tunnel and the remainder are at grade or elevated, varying between center platform and side platform types.

DEVELOPMENT OF THE GUIDEWAY CONCEPT

The preliminary and final designs of the SkyTrain guideway were preceded by a conceptual design of a guideway that paralleled the development of the Intermediate Capacity Transit System (ICTS), which was to become "SkyTrain." In 1975, the Urban Transportation Development Corporation (UTDC) Ltd. began to develop a transit system that would be intermediate in both its line haul capacity and its total system cost as compared to all other forms of existing public mass transit.

From the inception of the technology

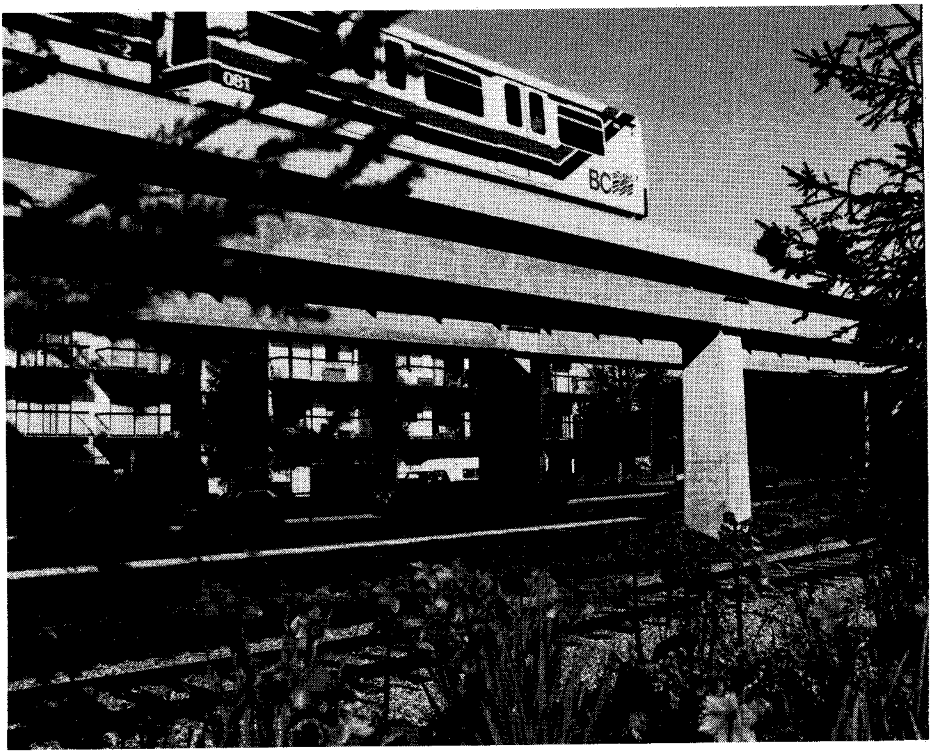


Fig. 1. Partial view of Vancouver rapid transit SkyTrain structure.

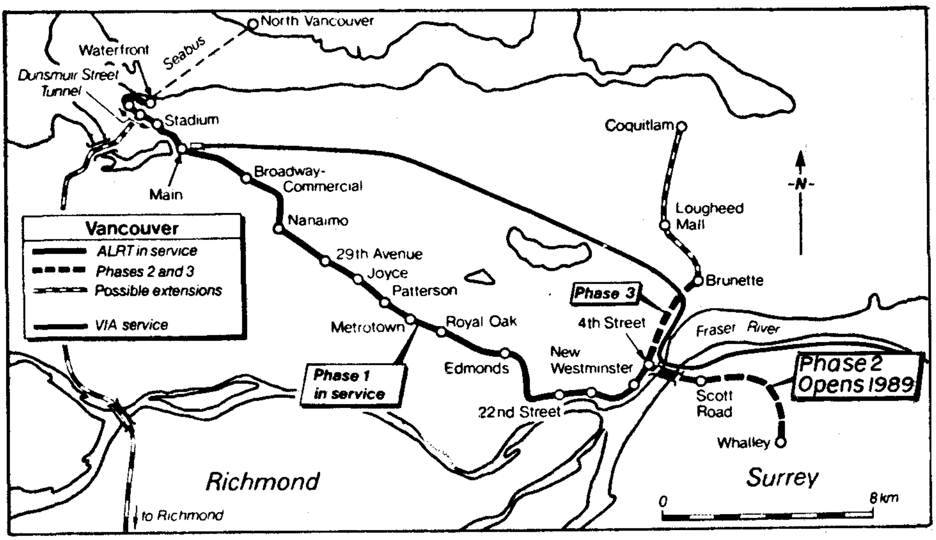


Fig. 2. Current and future routes of Vancouver guideway transportation system.

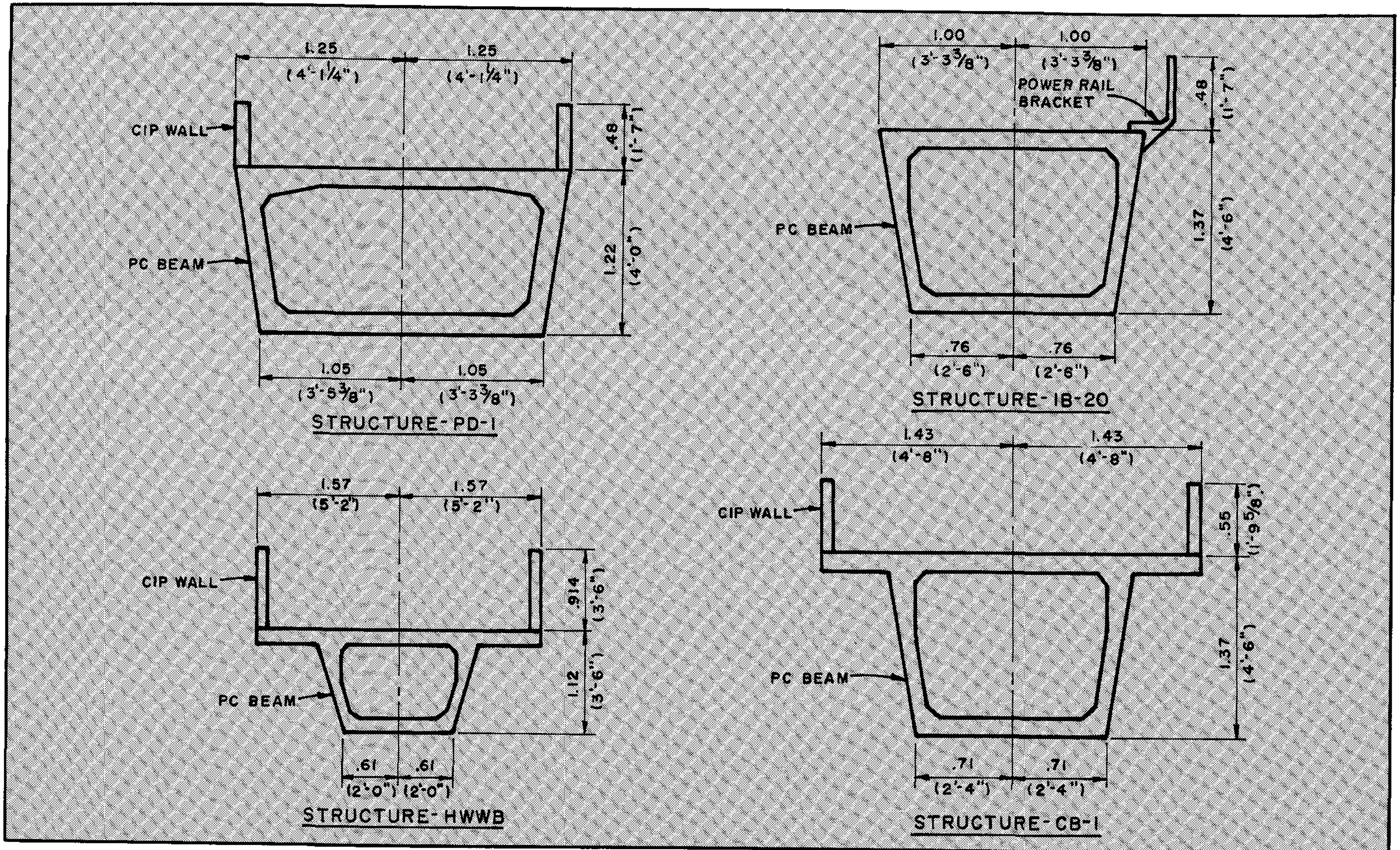


Fig. 3. Preliminary box beam sections for comparison (Structures PD-1, IB-20, HWWB and CB-1).

development program, it was understood that the guideway component of the system would be a significant cost element. In order to achieve the optimum technology solution for both performance and cost, it was further recognized that the guideway had to be developed with rigor comparable to that which would go into the vehicle. The guideway would not only have to support and guide the vehicle, but it would be required to successfully integrate all of the subsystems (running rail, propulsion power, linear induction motor reaction rail, communication cabling, signal cabling, walkways, etc.). In addition, it had to be acceptable to the public in terms of aesthetics and superior ride comfort.

To execute the above, a five-phase development program was established by UTDC. The first phase started with a review of all technology types, with comparisons of performance and costs. The final phase culminated in a prototypical revenue system application.

The guideway development began with parametric studies of a variety of structural schemes, determining relative costs between varying components. This included superstructure cross sections and materials, span lengths and beam depths, single and double column configurations for dual lane routes, column heights, foundation types, etc. Each of these was developed for varying sizes and types of vehicles with particular attention to variable support and propulsion forms (i.e., rubber tired, steel wheeled/steel rail, air cushioned, magnetic levitation, and bottom versus top supported). Concrete was the predominant material of choice and precast, prestressed methods of construction were emphasized throughout the planning phase.

These studies provided not only a thorough understanding of the cost of each structural component, the cost influence of one component on other components, and the cost of the integrated structure for each candidate system, but also a thorough evaluation of the value of each vehicular characteristic upon the technology package as a whole.

Following the parametric studies and a definition of technology characteristics, preliminary design criteria were prepared to define the geometry, loads, materials, and operations of a model system application. At this point, only precast concrete beam solutions remained as viable choices. Several designs were then produced, with the primary difference being in the beam cross sections. These designs (Fig. 3) allowed a more refined analysis and comparison of the constructability, construction costs, and variations of architectural impact in an urban environment.

A model urban route scenario was developed as a basis for comparison of the different designs. This provided an objective assessment of the benefits and shortcomings of each of the designs, the purpose being to lead to an ultimate selection of the optimum guideway choice. The beam cross section, spans, and supporting column/crosshead configurations finally selected were not the least cost solutions for the chosen vehicle system. However, they were believed to be the best all around choice for a combination of economy, appearance, and ride comfort features.

The structural system chosen (Fig. 4) provided these principal attributes:

■ A single trapezoidal cross section used for tangent and curved guideways, special structures, and spans up to 45 m (150 ft). The uniform cross section enhanced the advantages of precasting and mass production. The versatility of the same trapezoidal cross section provided freedom of column positioning in the urban environment.

■ Crossheads concealed within the depth of the beam. This fulfilled a major aesthetic consideration.

■ Structures continuous in two- to six-span segments. Continuity mini-

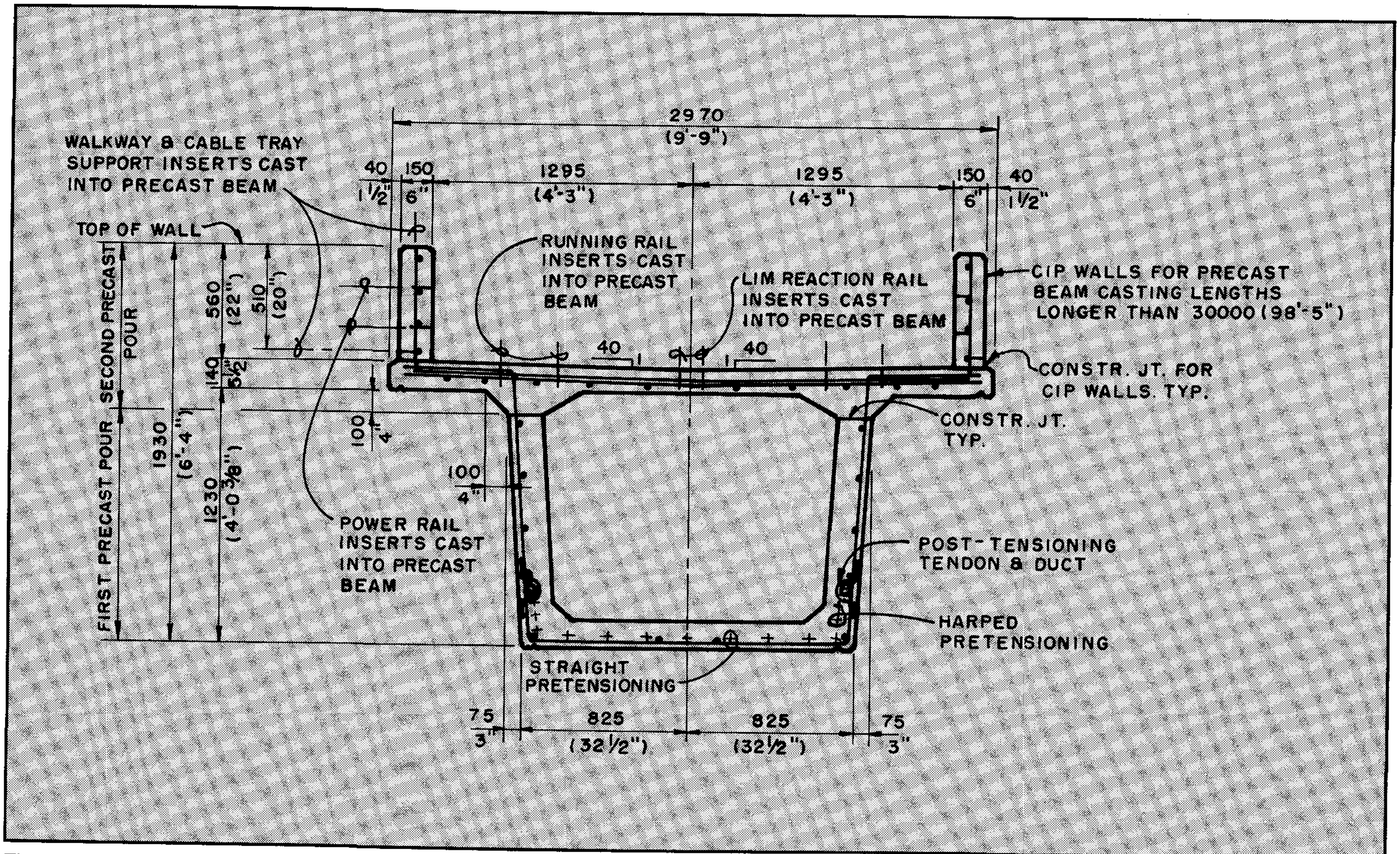


Fig. 4. Typical SkyTrain beam section showing major dimensions and reinforcement details.

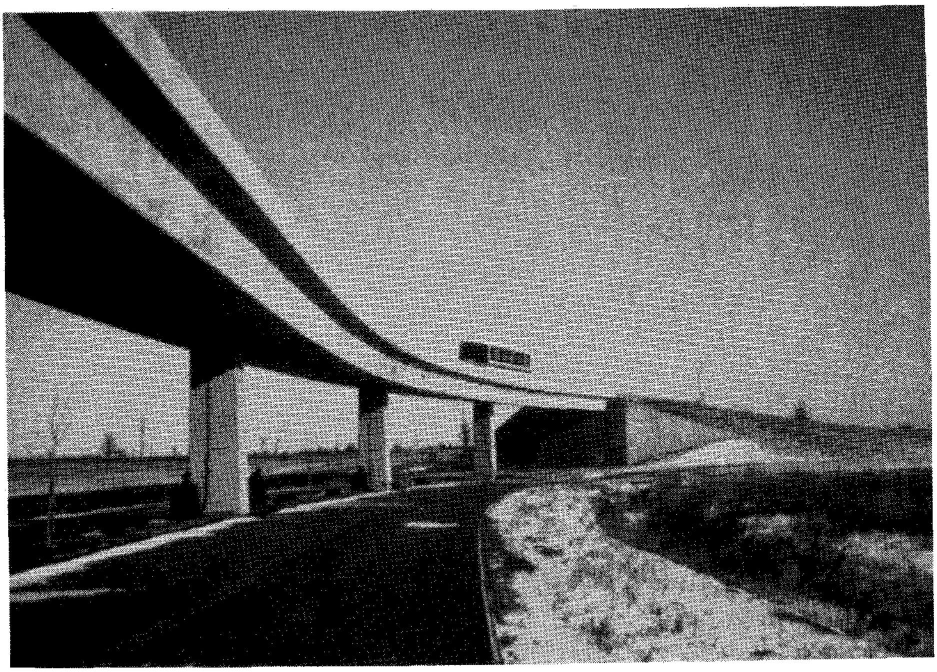


Fig. 5. System test facility demonstration guideway.

mized the number of joints in the riding surface and stiffened the structures, allowing greater control over deflections, vibrations, and ride smoothness. It also provided the benefit of structural redundance for the seismic environment of Vancouver.

■ Continuous welded rail attached directly to the supporting structure. Noise suppression and ride smoothness were the resulting benefits.

Constructability studies, performed concurrently with the structural designs, indicated that regardless of the structural scheme eventually adopted, significant cost and system performance benefits could accrue if the trackwork, linear induction motor, and other operational subsystems could be attached directly to the precast concrete without the need of a second concrete tolerancing pour. However, full scale verification testing was required before this feature could become part of the final design.

The final stage in the development program was the design and construction of a Transit Demonstration Center, including a full operational test track in Kingston, Ontario. The primary purpose of the track was to provide a testing facility for the vehicles and system operations. Construction of the facility, however, also afforded the opportunity to verify certain design assumptions with respect to the guideway's constructability and performance.

An elevated six-span, dual lane structure was constructed (Fig. 5) at the test track in a portion of the overall alignment. It was positioned in the test loop in such a way as to have two spans on the tangent, two in a spiral transition curve, and two on a circle curve. Although the prototype guideway beams were planned to be precast concrete, the test track beams were constructed by the cast-in-place method to be most cost effective for the scale of production.

Installation of the trackwork, linear induction motor reaction rail and other subsystems, combined with physical tests and extensive measurements before and during train operation, verified that the inserts could be successfully installed in a single concreting operation with the methods developed. There are, on average, ten inserts placed in the concrete beam every meter (3.3 ft) of its length. The ability to do this reliably and accurately to close tolerances for a variable geometry became a major focus of the final design.

INITIAL DESIGN OF SKYTRAIN

When the project was awarded to Metro Canada Limited (MCL, the implementation arm of UTDC) in 1981, there was a firm completion date. The transit system had to be open to revenue service in May 1986 to accommodate the Expo 86 event. There was a need to have the project groundbreaking as early as possible. Yet, there were many unsettled issues, one of which was the final identification of the alignment rights of way.

The owner, together with MCL, determined that a short section of right of way was defined with certainty and was immediately available, and that there would be benefits to all if a starter (demonstration) section of alignment was built in advance of the balance of the system. Consequently, a 1.1 km (0.7 mile) segment of the line and a station were designed and constructed before the final design of the entire line was complete. This section of the line, called the Prebuild Section, was later integrated into the system.

The purpose of the Prebuild Section was to demonstrate the system to the public, validate numerous design and construction techniques, complete vehicle testing, and verify operational performance. Immediately following completion of this section (from the end of June through November of 1983), nearly 300,000 passengers rode the demonstration shuttle. All of the Prebuild Section objectives were achieved and the public, which had been generally skeptical beforehand, became enthusiastic proponents of the system.

In order to fast track the Prebuild Section design, the design developed at the Transit Demonstration Center in Kingston was reviewed and adapted to it. Vancouver project specific design criteria were prepared. The resulting foundations, columns, and 76 beams were individually designed and detailed for the fabrication. In the process of doing so, the basis for a set of project standards was established. Standards would be fully developed in the beginning of the remainder of the alignment final design.

SYSTEMWIDE FINAL DESIGN

A systematic rationale for the design of all aerial structures was established to limit redundant or inconsistent engineering efforts. An analysis approach was defined, segregating the work into three categories — definition of static and dynamic load cases, definition of structural model geometries, and development and definition of a consistent set of rules by which computed elastic forces could be translated into design values for the various member types.

Regularity of span lengths and column heights over most of the route meant that a small family of typical models would adequately represent most field conditions. However, due to real world conditions, geometric deviations from the standards occurred over 10 to 20 percent of the alignment. A considerable number of "special" structures, requiring individual analyses, were also identified.

To take maximum advantage of the

Table 1. Skytrain Project Costs (Canadian dollars).

Principal items:		
Vehicles (114)		C$149,000,000
Automated controls, communication, ticketing, and administration		93,000,000
Power distribution		29,000,000
Maintenance facility		16,000,000
Stations		62,000,000
Guideway		249,000,000
Engineering design and management		147,000
Right-of-way acquisition and other related items		57,000,000
Total project cost		C$802,000,000
Breakdown of costs for "Guideway" element listed above:		
Tunnel rehabilitation — 1.3 km		C$25,000,000
Prebuild Section, complete — 1.1 km (0.7 mile)		12,200,000
At-grade guideway — 3.5 km (2.2 miles)		14,600,000
Elevated guideway (less Prebuild Section) — 15.5 km (9.6 miles):		
Beam production	C$54,600,000	
Beam erection and delivery	9,400,000	
Site construction	41,500,000	
Elevated guideway subtotal		105,500,000
Trackwork (50 track km), installation and materials		63,000,000
Other items		28,700,000
Total Guideway		C$249,000,000

computer, a systematic approach involving automated generation of structural models and rigorous naming conventions for the various load cases was adopted for typical and special structures alike. This provided a high degree of consistency in the format of analysis printouts, making it easy to cross-check results from different models to reveal trends or suspected errors.

The procedures by which elastic analysis results were applied in the sizing of members and the detailing of reinforcement were equally important. A design philosophy was adopted, resulting in a hierarchy of member strengths. Under extreme lateral loads, as in earthquake or street level vehicle impact, the columns (which are readily detailed for ductile inelastic behavior) are designed to yield first; factored design capacities of foundations and crossheads exceed column ultimate strengths, ensuring that these members will remain elastic.

The total cost of the SkyTrain project (see Table 1 for the details) was $802 million (Canadian). The breakdown of costs of the Guideway element alone is shown in Table 1, adding up to a total of $249 million (Canadian). The above amounts to a dollar per square unit price of C$61 per sq ft for the entire project combined, but only C$53 per sq ft excluding the Prebuild Section. This figure reduces to a precast beam cost of C$1325 per cubic yard project wide or C$1161 per cubic yard excluding the Prebuild Section.

DESCRIPTION OF THE DESIGN

The basic guideway structure consists of a pair of trapezoidal precast concrete beams resting on cast-in-place crossheads and columns (Fig. 6). Typical spans are 30 m (98 ft). The most common structure is continuous over two spans. The center column has a fixed connection to the beams and reacts longitudinal loads. All columns share transverse loads (Fig. 7).

Typically, expansion columns and crossheads alternate with fixed columns and crossheads (Fig. 8). Both types of crossheads were designed to be constructed within the depth of the guideway beams, providing a uniform depth structure over the entire length of the system (Fig. 9). This produces a very clean continuity of appearance.

Sidewalls of the guideway are typically precast integrally with the beams for lengths up to 30 m (98 ft). For the few cases where longer beams were required, transportation weights in excess of the maximum dictated that sidewalls be cast in place following transportation and erection of the precast beam at the site.

Several special operational requirements and urban conditions prevented the universal use of typical structures. There are four types of special structures:

- Structures requiring special support(s)
- Structures requiring long spans
- Structures requiring continuity over three and four spans
- Crossover and switching structures

To accommodate these situations, special purpose structures were designed that allowed the use of the standard trapezoidal beam cross section and

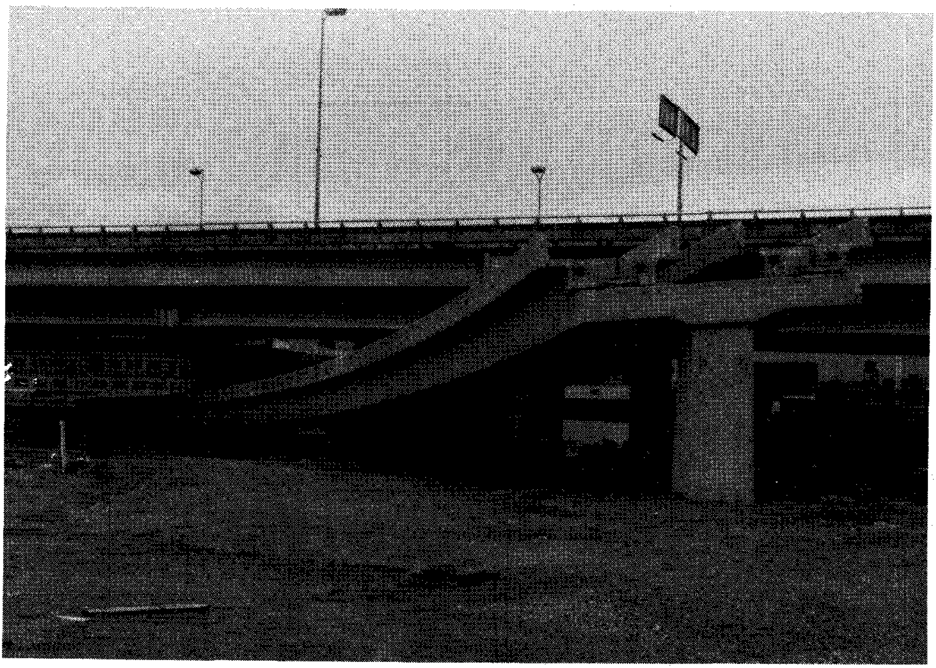


Fig. 6. Dual lane guideway scheme.

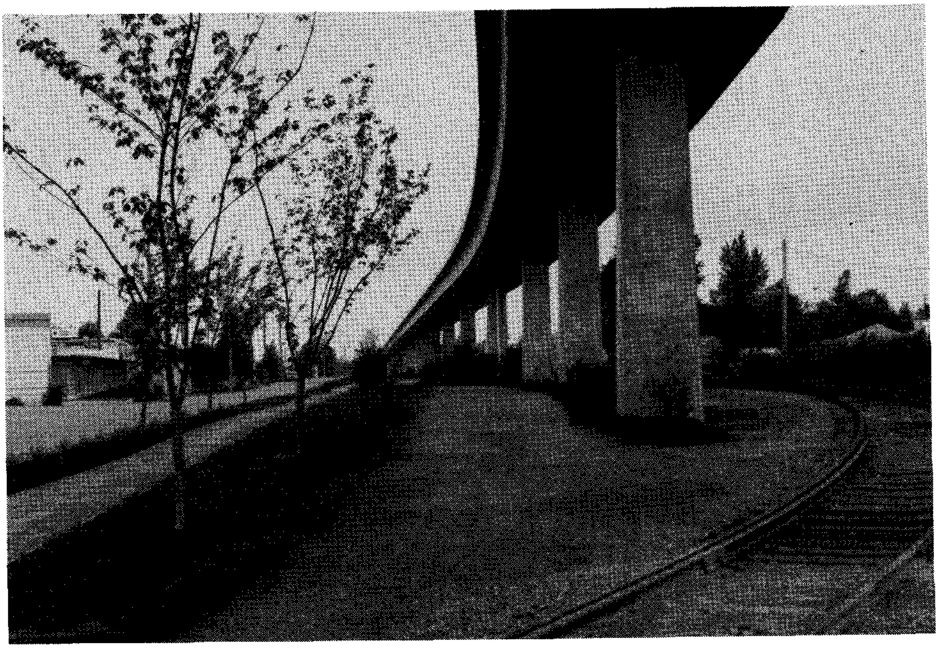


Fig. 7. Continuous spans provide redundant load paths.

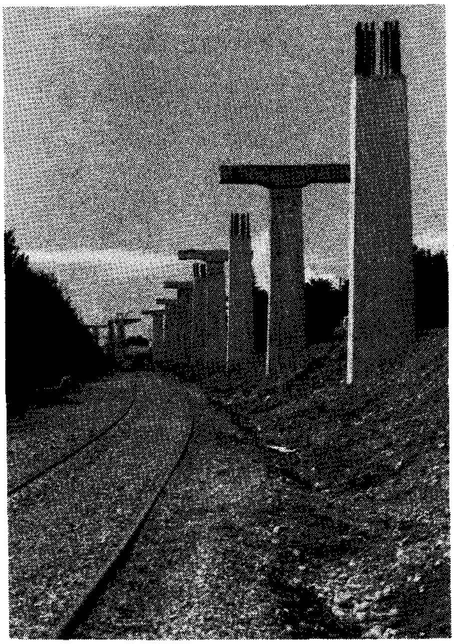


Fig. 8. Alternating column types for two-span structures.

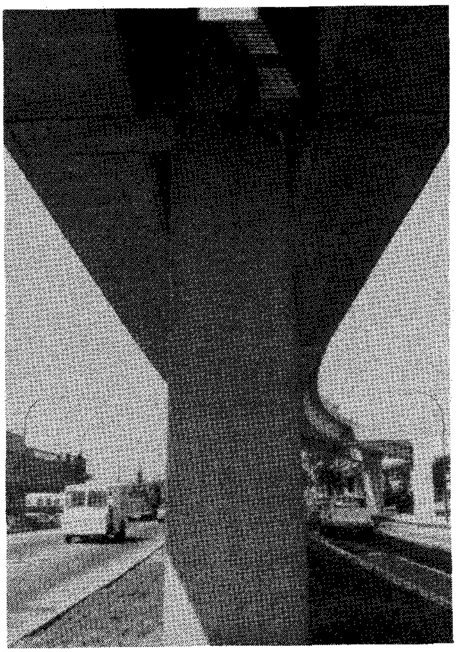


Fig. 9. Uninterrupted soffit line of beamway.

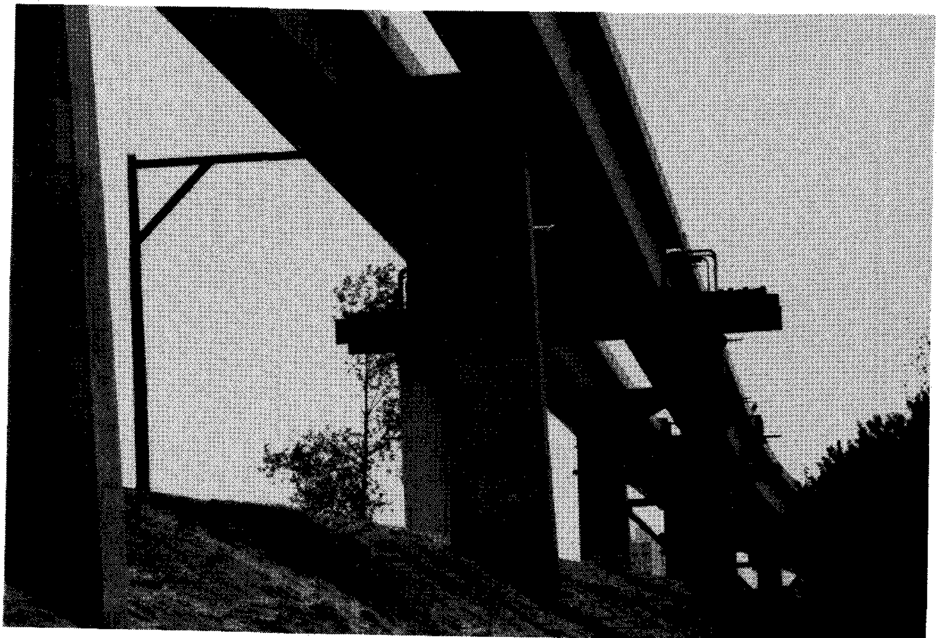


Fig. 10. Special structure supports.

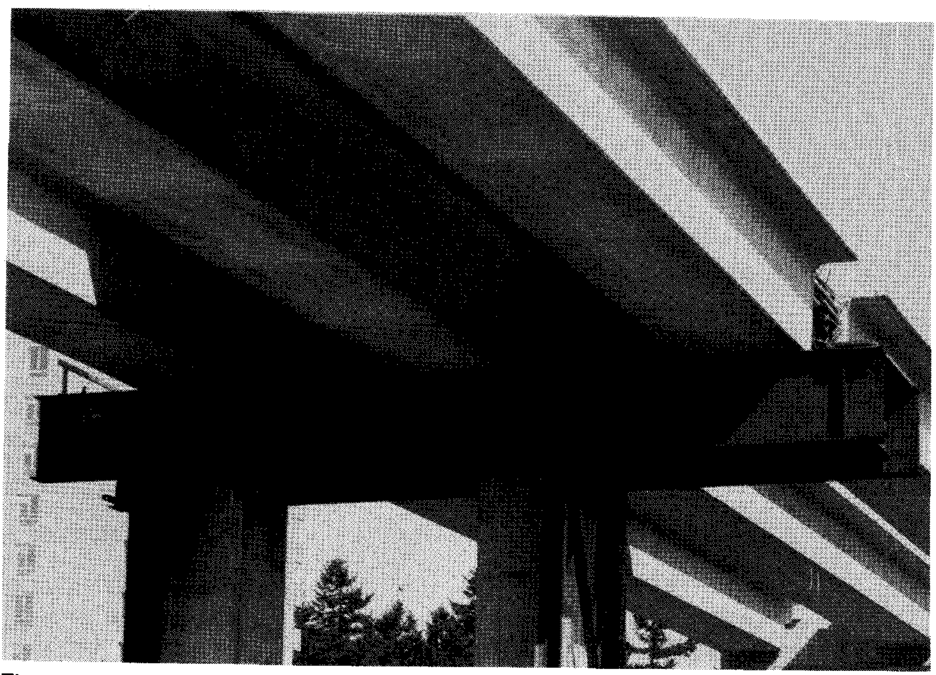


Fig. 11. Three-beam deck of pocket track with offset columns.

Fig. 12. Long span requirements create special structures.

Fig. 13. Standard beams for special track work.

Fig. 14. Completed SkyTrain structure in operation.

hence the standard forms. Examples of special supports are shown in Figs. 10 and 11. Spans of 45 m (148 ft) were created using three-span structures with a center drop-in beam (Fig. 12). Three-span structures generally have nominal span ratios of 1:1.5:1.

For crossovers and switches, standard guideway beams were lowered 167 mm (6.57 in.) on special crossheads (Fig. 13) and a cast-in-place slab was placed over the beams and in the gap between them. In these cases, inserts for subsystems were installed in the field and the overall depth of the structure became 1.097 m (3.6 ft) rather than the standard 0.930 m (3 ft).

The standard deck thickness was reduced by 40 mm (1½ in.) in the precast beam to act as a deck form for the field cast special slab, and the sidewalls were not precast. Flange edges were varied in width to accommodate geometry and the final deck was reinforced and field cast to integrate all beams into a composite system. A second pour track fastening slab was then placed on top of the structural slab, which allowed for variable placing of turnout hardware.

CONCLUDING REMARKS

Much thought and planning went into the development of the guideway components in an effort to maximize standard precasting elements. The designers strongly felt that the additional effort to achieve demanding tolerances in the precast beams would be more than repaid by the simplified installation and labor and time reduction experienced by the field contractor. This in fact was the result. The actual tolerances were well within the specified limits.

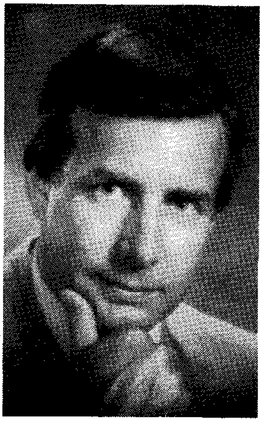
Construction disruption to neighborhoods along the alignment was substantially reduced for both erection and rail installation. The now record economies of this project (Fig. 14) were due in large part to this design approach and to the cooperative efforts of all parties involved to make it work.

Aerial Guideway for the Vancouver ALRT Project

Part 2 — Construction Highlights

by Paul A. R. Lowe

Part 2 — Construction Highlights



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In May 1981, British Columbia Transit awarded a prime system contract for the construction of a 22 km (13 mile) long light rapid transit line (Fig. 1) in Vancouver to Metro Canada Ltd., a subsidiary of the Urban Transportation Development Corporation of Ontario, Canada. The May 2, 1986, opening of Expo 86 was set as the deadline for completion of the project.

The owner elected to construct a 1 km (⅝ mile) long Prebuild Section before proceeding with the main part of the project for three main reasons: (1) to demonstrate the appearance and operation of the system to the public and to win its confidence and approval, (2) to evaluate design and construction concepts for the civil works, and (3) to accelerate the testing of electrical and mechanical systems under site service conditions.

It is generally accepted that the premium associated with the construction of the Prebuild Section was repaid several times in cost and time savings during the construction of the remainder of the project. The prebuild guideway was constructed by Commonwealth Construction during the fall and winter of 1982, and vehicle testing was commenced in May 1983. As a part of the public relations objectives, complimentary rides on the Prebuild Section were provided to the public throughout the summer of 1983. Public reaction was extremely positive.

Concurrent with the construction of the Prebuild Section, on June 30, 1982, bids were called for the fabrication, delivery and field post-tensioning of 1040 guideway beams for the major section of the aerial guideway. As final alignment had not been completed at that time, the bid drawings were of a generic nature with final beam geometry to follow later. This beam supply contract was awarded to the Supercrete Division of Canada Cement Lafarge Ltd. on October 13, 1982.

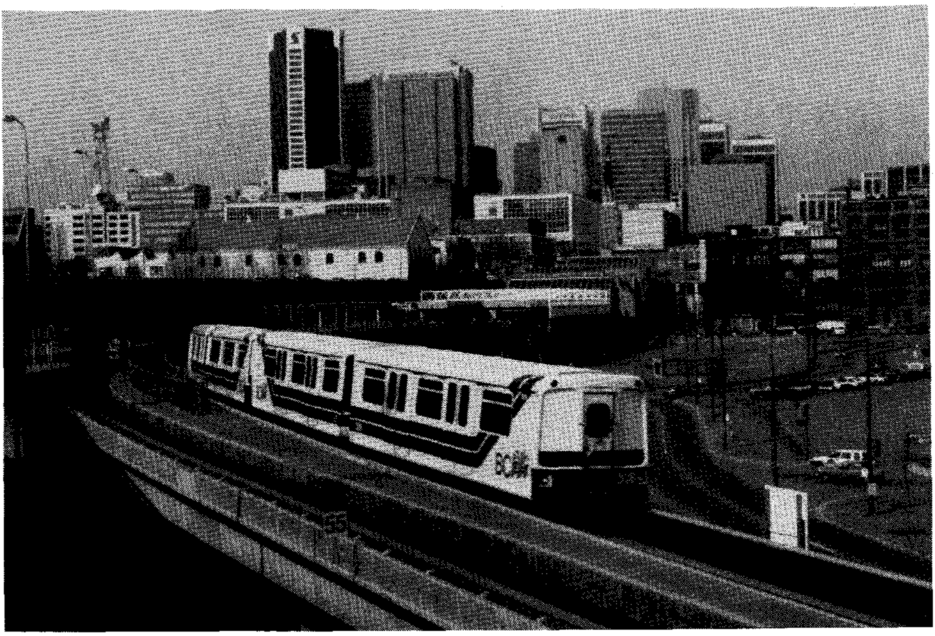


Fig. 1. Panoramic view of Vancouver ALRT aerial guideway system with city skyline in background.

Highlights of the two precast beam contracts are shown in Table 1.

As the final design was completed, eleven separate contracts were awarded for the construction of the tunnel section, the maintenance yard, at grade sections and the substructure portions of the aerial guideway. At a later date, a single contract was awarded to Peter Kiewit & Sons Ltd. for the installation of all guideway beams. Further contracts were awarded for station construction, rail installation, and electrical and mechanical installations.

SUBSTRUCTURE

Foundation Construction

Foundation design varies along the length of the route. The majority of foundations consist of cast-in-place concrete spread footings; in limited areas, post-tensioned rock anchors are also used. There are three types of pile foundations in this project: concrete filled steel pipe piles, steel H-piles and cast-in-place expanded base driven piles.

Columns and Crossheads

All columns, bents and crossheads are of cast-in-place concrete construction. Four types of columns support the aerial guideway: single columns with cantilever crossheads that support dual lanes (Fig. 2), isolated columns that support single lanes near stations where the lanes diverge, column bents that span over roads and railway tracks, and station bents that carry station platforms in addition to the guideway beams. There are 376 dual lane columns, 122 single lane columns, 39 spanning bents and 31 station bents.

All dual and single lane columns have a constant 1:40 taper in the transverse direction and are of a constant width in the longitudinal direction. The majority

Table 1. Precast beam data.

Item	Prebuild		Phase 2	
Total number of beams	74		1040	
Number of tangent beams	56		484	
Number of curved beams	18		556	
Total length of beams	2062 m	(6,763 ft)	29 043 m	(95,261 ft)
Length of longest beam	34.3 m	(113 ft)	35 m	(115 ft)
Length of longest span	34.3 m	(118 ft)	45 m	(148 ft)
Mass of heaviest beam	95 t	(104 tons)	106 t	(116 tons)
Volume of concrete	2633 m^3	(3,440 yd^3)	36 000 m^3	(47,000 yd^3)
Nonprestressed reinforcement	336 t	(370 tons)	5200 t	(5,700 tons)
15 mm pretensioning strand	31 km	(19 miles)	290 km	(180 miles)
13 mm plant post-tensioning	17 km	(11 miles)	560 km	(350 miles)
13 mm field post-tensioning	50 km	(31 miles)	970 km	(600 miles)
Number of inserts cast in	32 000		300 000	
Contract awarded	82-05-25		82-10-13	
First beam cast	82-08-20		83-04-11	
Last beam stripped	82-11-24		84-10-03	
First beam erected	82-10-15		84-01-09	
Last beam erected	82-12-10		84-11-09	
Contract value	Can. $7.6 million		Can. $42 million	
System owner	B.C. Transit		B.C. Transit	
System contractor	Metro Canada Ltd.		Metro Canada Ltd.	
Elevated guideway designer	ABAM Engineers Inc.		ABAM Engineers Inc.	
Beam precaster	Con-Force Structures Ltd.		Supercrete, A Division of Canada Cement Lafarge Ltd.	

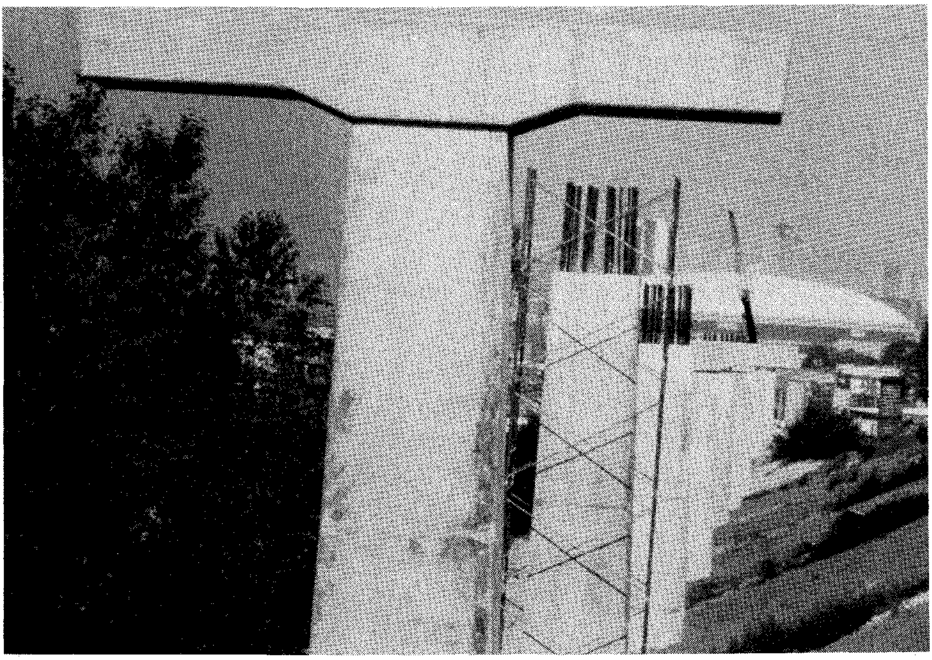


Fig. 2. Typical expansion and fixed columns.

of columns have a constant cross section at the top which simplified forming and permitted the reuse of forms; most substructure contractors elected to use steel forms. Although the use of precast concrete columns was investigated, it was found to be more economical to utilize cast-in-place construction.

One of the most aesthetically pleasing aspects of the guideway design is that all crossheads are within the depth of the guideway beams (Fig. 29). This required the use of dapped beam ends at expansion bearings.

Because of the development of small cracks in the relatively shallow expansion crossheads on the Prebuild Section, all expansion crossheads were field post-tensioned for Phase II. The fixed crossheads, however, because of their greater depth, did not require post-tensioning.

Expansion crossheads have good geometrical repetition. Some contractors elected to support the crosshead formwork on falsework, while others supported these forms from the columns and tied the forms down with deadmen and come-alongs. The lower plates for the beam bearings were cast directly into the expansion crossheads.

The fixed crossheads were constructed after the erection of the guideway beams. This operation is described in detail later in this paper.

BEAM PRODUCTION

Prebuild

A casting plant was built by Con-Force Structures Ltd., in the summer of 1982 to produce the 74 beams required for the Prebuild Section. Two tangent and one curved custom built concrete casting beds, founded on timber piles, were constructed on a site near the North Arm of the Fraser River in South Vancouver.

The prime contractor and the guideway designer, ABAM Engineers Inc., had, during the initial stages of the development of the system, identified the design and procurement of the beam formwork as a critical factor in the economical achievement of the required beam fabrication tolerances. Consequently, ABAM worked closely with the successful form supplier, Burke Canada Ltd., to develop the forming concept and the form design details. Three forms, two tangent and one curved, were provided by the system contractor to the Prebuild beam precaster as an element of the contract.

Plant Design

During the 6-month period following the award of the Phase II beam supply contract, a $9 million (Canadian) precast plant was designed and constructed on an open site adjacent to Canada Cement Lafarge's cement plant in Richmond, a suburb of Vancouver. This is approximately 20 km (12 miles) south of the downtown core, which is the western end of the transit line.

To meet the schedule, eight forms were required — three tangent and five curved. Three of these forms were used during the casting of the Prebuild beams; the remaining forms were acquired specifically for Phase II.

Because of the large quantities of materials and form parts to be handled during the 18-month production period, the design of the plant was approached as a materials handling problem, and considerable thought was given to the optimization of hoisting equipment selection and design. The solution was to locate the form component storage and reinforcing bar fabrication areas adjacent to each respective casting bed.

The resulting plant area of 18 000 m^2 (200,000 sq ft) (Fig. 3) economically precluded the construction of a covered building with conventional bridge cranes. Instead, to provide protection from rain, snow and sunshine, three movable shelters running on tracks were

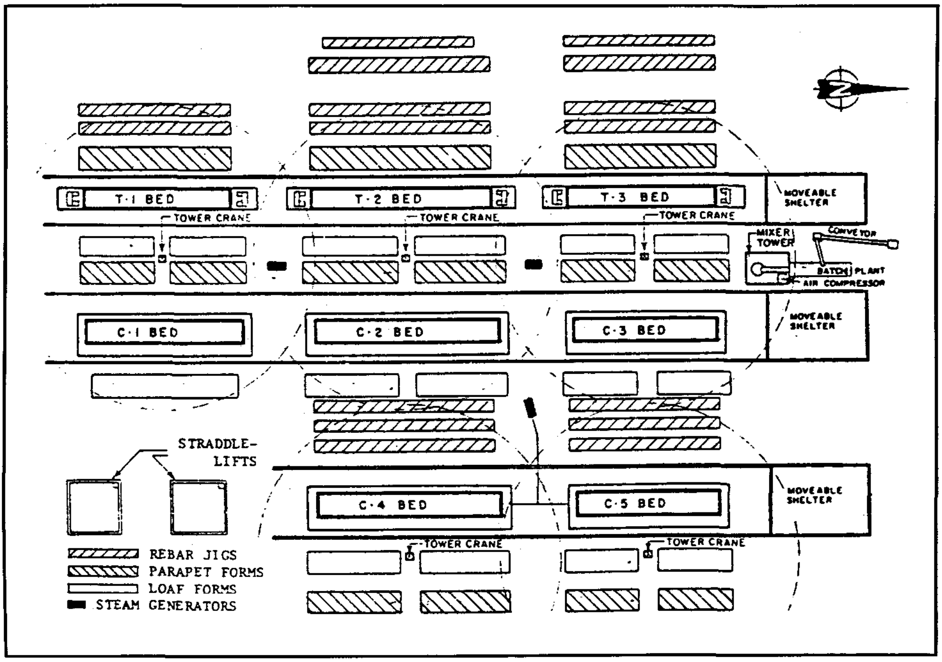


Fig. 3. Schematic diagram of precast plant layout.

constructed. The buildings were fabricated from standard pre-engineered components, and each consists of two telescoping sections, each 18 m (60 ft) long (Fig. 4).

The building for the tangent line is 11 m (36 ft) wide, and the two buildings for the curved lines are each 15 m (45 ft) wide; all buildings have an inside clear height of 8 m (26 ft). Moved with electric motors, the buildings have ceiling mounted lighting; this provides excellent illumination for night time operations. Additional lighting is mounted on the tower crane masts and booms.

Many precast plants operate with one or more bridge cranes which are used to handle form parts, reinforcing cages and concrete buckets, and to strip and yard the finished products. The key to the efficient operation of this plant was the decision to provide three completely independent materials handling systems. Five tower cranes were used to hoist form parts and most of the reinforcing cages. Two concrete pumps and a slick line and placer boom system were used to deliver concrete, and two rubber tired gantry cranes were used to strip, yard and load the beams.

A trailer complex was constructed to provide office accommodation, washroom, lunchroom and storage facilities for the 300 persons working in the plant.

Fabrication Concept

The guideway designer had conceived a beam design and forming system that utilized a two-stage casting process (Fig. 5). The first stage consisted of the bottom flange and webs of the beam, together with the reinforcement, pretensioning strand and post-tensioning ducts. Following overnight accelerated curing of the concrete, the loaf forms were removed, and a soffit form, the top reinforcing cage, and an insert

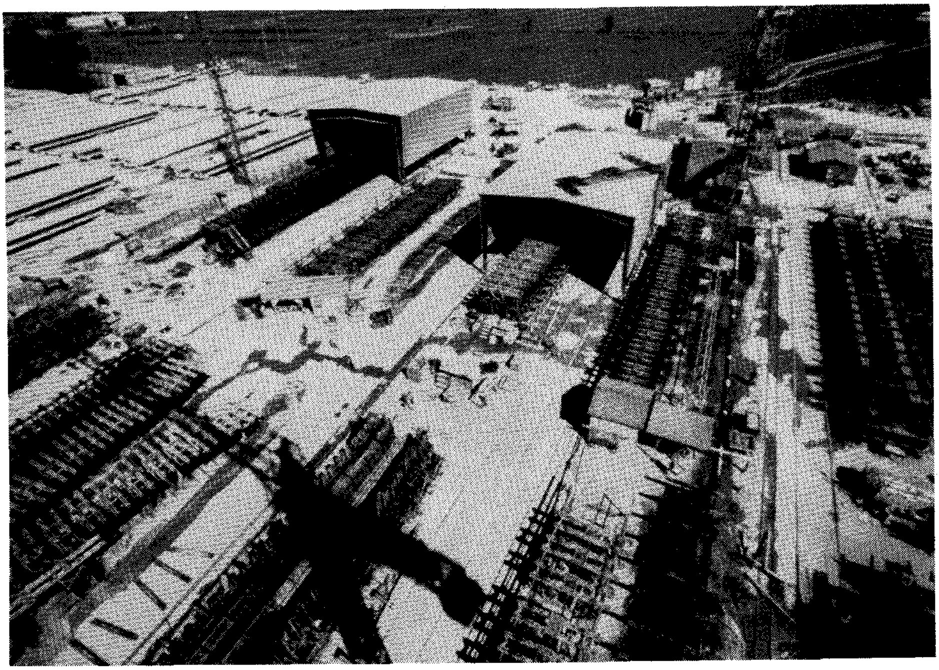


Fig. 4. Overview of precast plant.

jig complete with threaded inserts were installed. Second stage concrete was placed and cured overnight, ready for beam stripping the following day.

The design concept also involved the casting of curved beams, comprising approximately 50 percent of the project, to the continuously varying superelevation and horizontal and vertical profile of the track. This permitted direct fixation of the track to the guideway beams without the requirement for secondary field construction of support plinths for the rails.

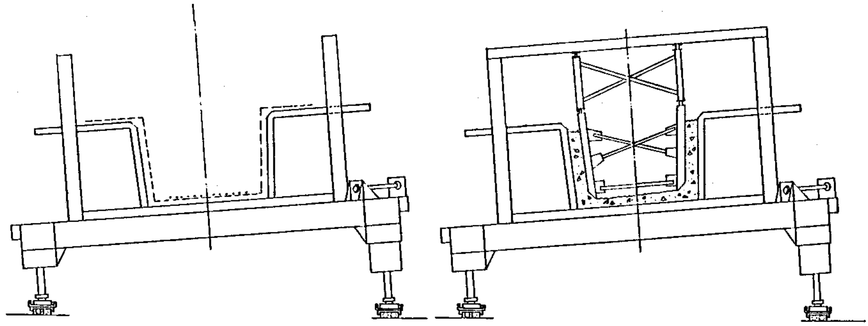
Shop Drawings

The generation of shop drawings for over 1000 beams, each of which was similar to yet different from the others, provided a challenge to the precaster, particularly due to the very tight production schedule.

The least cost solution was the utilization of CAD generated drawings, prepared by Advenco Consultants Ltd. Drawings were of two types, i.e., standard detail drawings applicable to all beams, and unit production drawings. Standard detail drawings were prepared on a Holguin interactive graphics system. These drawings not only had the benefit of clarity afforded by mechanical lettering and the use of inked transparencies, but provided early warning of possible duct conflicts because of the system's ability to rapidly draw cross sections and duct locations to scale.

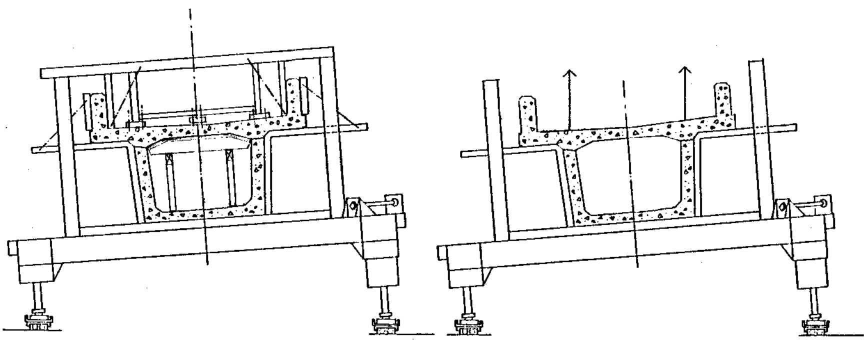
An innovative approach was used by the consultant to produce the two-sheet production drawings for each beam. The first sheet showed a plan of the beam drawn to scale, formset information, bulkhead locations, and other form dimensions. The second sheet showed all of the reinforcement details. These drawings were generated using the highly intelligent batch-run Sommel system which automatically combined various "patterns" to create the drawings. The plot file from this system was then transferred to the Holguin interactive system for final fine tuning.

As the drawings were used as a tool in the production of the beams, the temptation to produce works of computer art was resisted. Upon approval, minor corrections to the drawings were made manually. No attempt was made to archive the as-built changes onto the computer records.



FIRST STAGE

1. CLEAN AND OIL FORM
2. SET FORM & CAULK JOINTS
3. PLACE REINFORCEMENT CAGE
4. STRESS STRANDS (TANGENT BEAMS ONLY)
5. INSTALL VOID FORM
6. PLACE CONCRETE
7. CURE CONCRETE



SECOND STAGE

1. REMOVE VOID FORM
2. INSTALL SOFFIT FORM
3. PLACE REINFORCEMENT CAGE
4. INSTALL INSERT JIG
5. PLACE CONCRETE
6. CURE CONCRETE
7. REMOVE INSERT JIG
8. REMOVE BEAM
9. YARD BEAM ON BUNKS
10. PLANT POST-TENSIONING (CURVED BEAMS ONLY)
11. FINISH BEAM

Fig. 5. Casting concept.

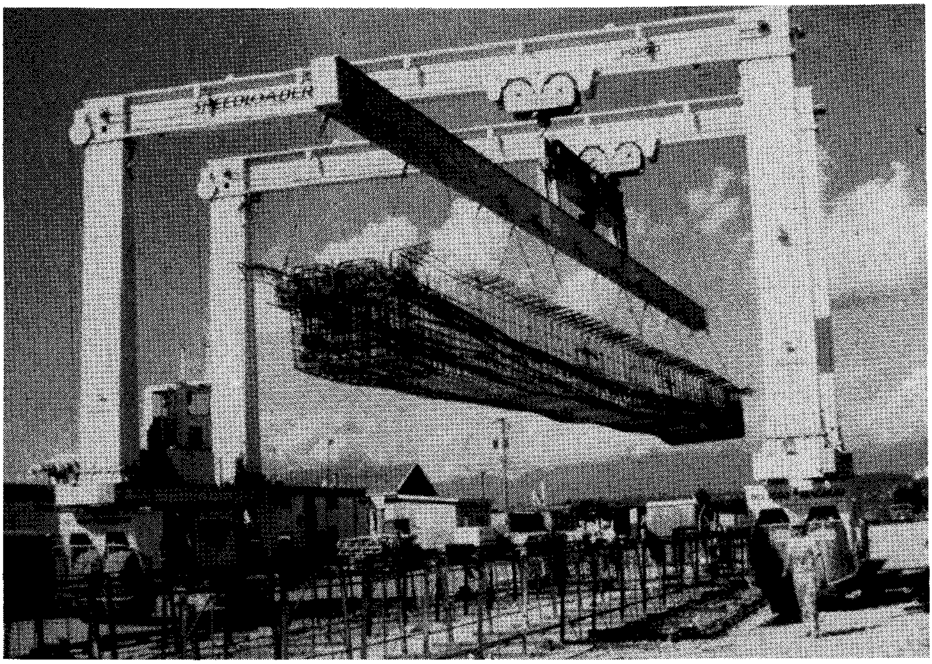


Fig. 6. Reinforcement jigging and cage.

Reinforcement

Reinforcing bars, which were all Grade 400 (60 ksi), were cut and bent at the fabricating shop of Lafarge Concrete Ltd. and then trucked to the precast plant. Bars were grouped for each beam and unloaded directly at the appropriate jigging area. Individual jigs (Fig. 6) were custom fabricated from reinforcing bars and light steel angle framing.

Jigs for curved cages were set to the average radius for all beams; it was not found necessary to adjust the radius of the jigs. However, because of the need to pre-position deck reinforcement perpendicular to the longitudinal axis of the beams to allow adequate clearance for the placement of deck inserts and to avoid a splaying of bars at the ends, the longitudinal location of control stirrups and deck reinforcement was precalculated at 1500 mm (5 ft) centers and shown on the shop drawings. The length of longitudinal bars was also varied across the width of curved beams.

Because of the tight setting tolerances required at the dapped ends of beams to provide space for bearing blockouts, deck joint recesses and deck insert placement, dapped end cages were prefabricated on special jigs inside a shelter. These cages were then set into the main jigs in one piece.

Most cages were fabricated within the boom radius of the tower crane serving the related form. These cages were hoisted in half length sections. The remaining cages were transported to the forms in complete units (Fig. 6) utilizing a 28 m (92 ft) long strongback and one of the mobile gantries.

No welding of the design reinforcement was permitted by specification. Consequently, post-tensioning duct support hoops were welded to additional No. 10 (3/8 in.) control stirrups at 1500 mm (5 ft) centers. These control stirrups were fabricated on a special jig and moved to the forms in complete sets for each beam.

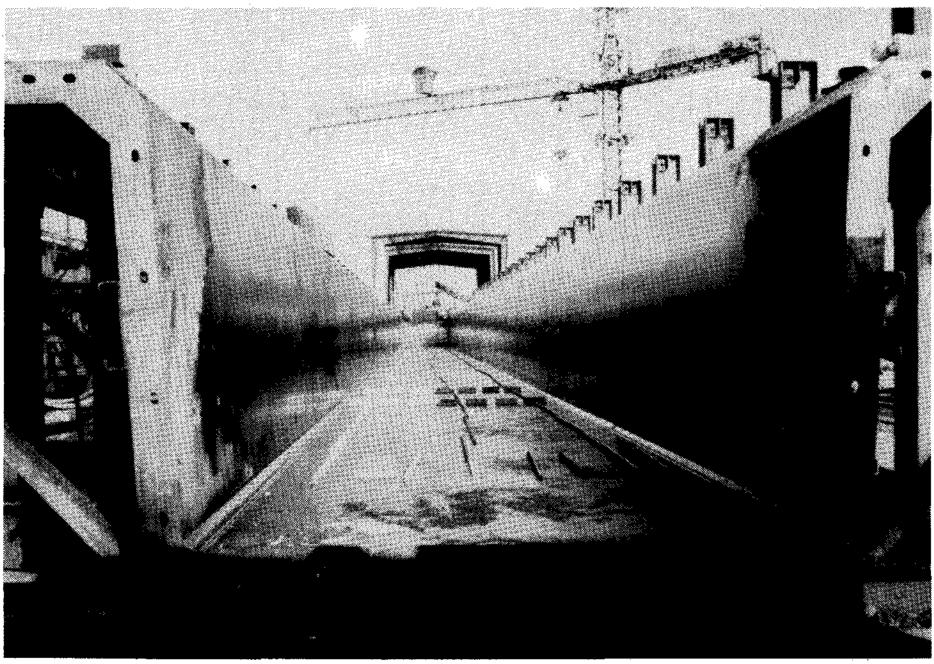


Fig. 7. Tangent form.

The 67 mm (2⅝ in.) diameter ducts for the plant post-tensioning of curved beams and the 76 mm (3 in.) diameter ducts for field post-tensioning were then pulled through the support chairs, and additional support and lateral tendon restraint bars were tied to the stirrups at 500 mm (20 in.) centers. The post-tensioning anchorages were preset in the forms prior to the placement of the reinforcing cages.

There was congestion of deck reinforcement at the intersection of the webs and the deck in tight radius beams, and several deck bars had to be individually tied into the cage in the form. On future projects, the interrelationship between the two cages could be further studied in the design stage and this problem may be overcome.

Beams were lifted and erected using 26 mm (1 in.) diameter Grade 1030 (150 ksi) Dywidag bars. These bars, complete with bearing plates and conical recess formers, were tied to the web reinforcement in the reinforcing jigs. Two pairs of these bars were used at each end of each beam. Four additional single bars were provided at approximately the one-third points for stripping and yarding of curved beams.

Despite the rigid quality control procedures exercised on this project, one shipment of reinforcement below the specified strength found its way into the precast plant, and 25 beams were produced before the problem was detected. Engineering analysis, fortunately, determined that the beams had adequate structural capacity and they were not rejected. The problem was traced to a testing procedure error in the steel mill.

Pretensioning

Tangent beams were prestressed with 15 mm (0.6 in.) diameter strands. There were up to eight strands in each web and 16 straight strands in the bottom

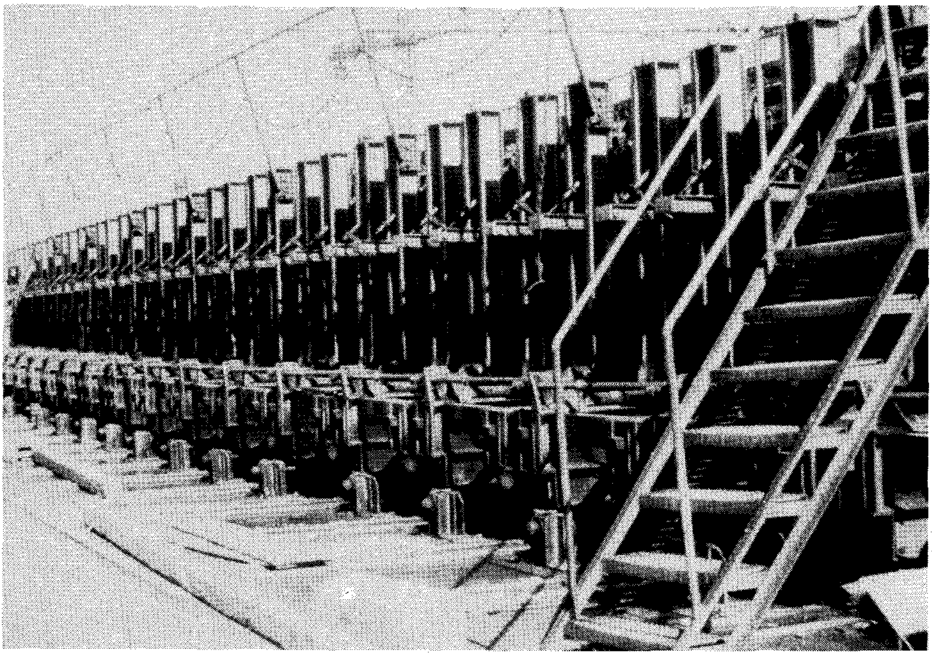


Fig. 8. Curved form.

flange of each beam. Harped strands were held down with roller assemblies at approximately the one-third points of the span. For hog and sag beams, additional concrete block spacers were used to deflect the strands in the bottom flange.

Pretensioning was accomplished by single strand jacking between fixed stressheads for each form. The stressheads consist of large steel box section uprights, horizontal fin plates and vertical backing plates. The fin plates were adjusted vertically to accommodate the change in elevation of the end of the forms due to vertical curvature of some of the beams. The uprights were restrained in pockets in the pretensioning beds. The beds themselves were reinforced concrete slabs 4500 mm (14 ft 6 in.) wide, founded on concrete piles.

Tangent Forms

Three forms were utilized for the production of tangent beams; one form was 36 m (118 ft) long, the other two were 30 m (98 ft) long. Forms were fabricated in 3000 mm (10 ft) long sections (Fig. 7) to permit the introduction of vertical curvature; a neoprene gasket was used to seal the joints between form sections. Hog or sag, with a maximum midspan value of 200 mm (8 in.), was achieved by the use of screw jacks. Formset consisted of setting the height of a punch mark on each cross frame above a plate set in the bed. The majority of end bulkheads were of steel construction, while some of the special beams had plywood sections with steel backup framework.

Curved Forms

Of the five forms used for curved beam production, two were 36 m (118 ft) long and three were 30 m (98 ft) long. The design of the curved forms (Fig. 8) was one of the most interesting aspects of the project. Form sections [1500 mm (5 ft) long] traveled on transverse rails cast into the beds; these rails were set to

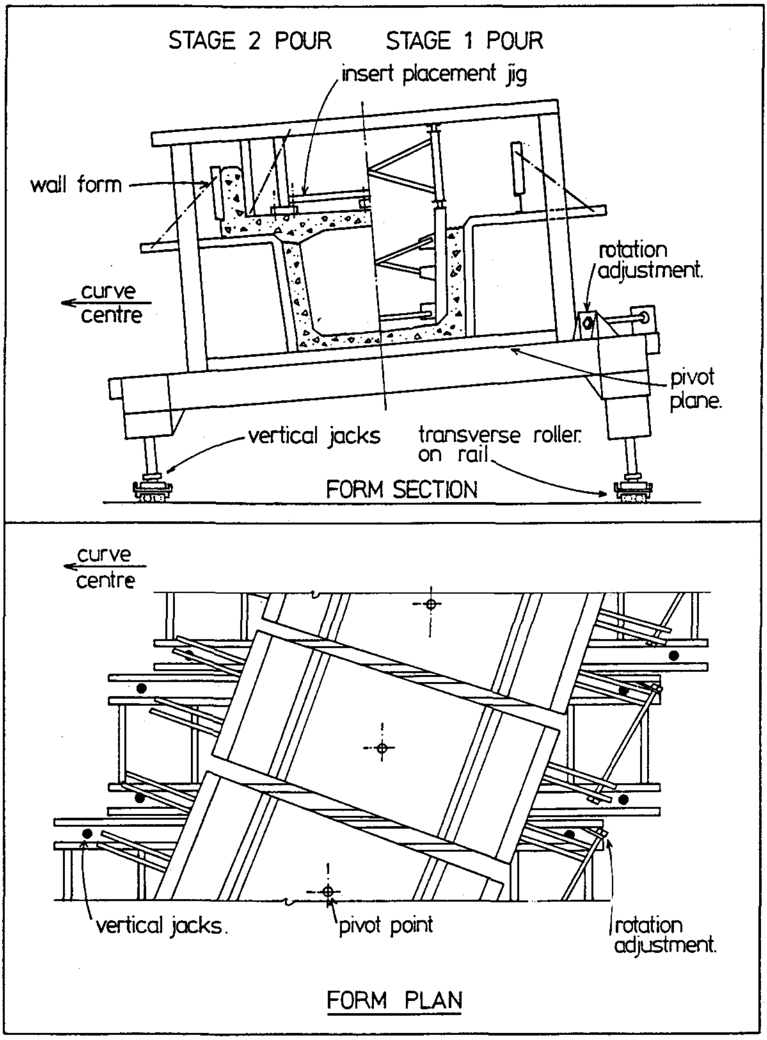


Fig. 9. Curved form arrangement.

a vertical and horizontal tolerance of 2 mm ($^1/_{16}$ in.) to achieve the required production tolerances. Each section was supported on two rails by four roller assemblies and was pushed across the beds with a large screw jack. The subframe of each section (Fig. 9) could be adjusted vertically with four screw jacks. This frame had sufficient torsional flexibility to enable it to be warped to the required profile of the beam soffit.

Finally, each tub section sat on the subframe and was rotated about a central vertical pin by a sixth screw jack. The form was set transversely by plumbing down from control marks on each section to the rail, and measuring the offset from a previously scribed line. Vertical adjustment was measured between four gage marks on the subframe and the rails, using plumbed measuring rods. Rotation was set by measuring between calibration marks on the tub and subframe, respectively.

After each tub had been set to its required geometry, filler plates were installed in each joint (Fig. 10) and the remaining gap was filled with silicone caulking compound.

All forms were recalibrated after each 25 casts by returning them to their tangent position and resetting calibration points, if necessary. The cross section of each tub and the location of each insert setting plate was set to within a tolerance of 3 mm (1/8 in.) and minor repairs and adjustments were made if required. No major repairs to the forms were required during the project.

Formset data were provided by the guideway designer and were incorporated into the precast shop drawings. A coordinate transformation was executed to roll the curved beams into an optimum casting configuration. This not only minimized the absolute extremes of form adjustment required, but also minimized the form changes between sequential casts; two beams that appear radically different in geometry in the finished guideway due to different grades and superelevation might be quite similar when rolled into a neutral position.

Beam production scheduling was recorded using a custom designed computer program, so that actual and projected casting dates could be updated weekly. The task, however, of optimizing the scheduling of beams with varying length, curvature, end condition and special requirements, to meet the required delivery was found to be more easily managed by using a manual system of cards displayed in a wall rack; these cards graphically portrayed the important features of each beam and could be readily reorganized as required.

The importance of scheduling of curved beams cannot be overemphasized. When the production sequence for a form resulted in sequential beams with only small changes of geometry, it was not necessary to remove the form filler strips and recaulk the joints; such beams could be produced in a 2-day cycle. However, larger changes of geometry requiring such resetting and recaulking, led to a 3-day cycle with its attendant extra cost.

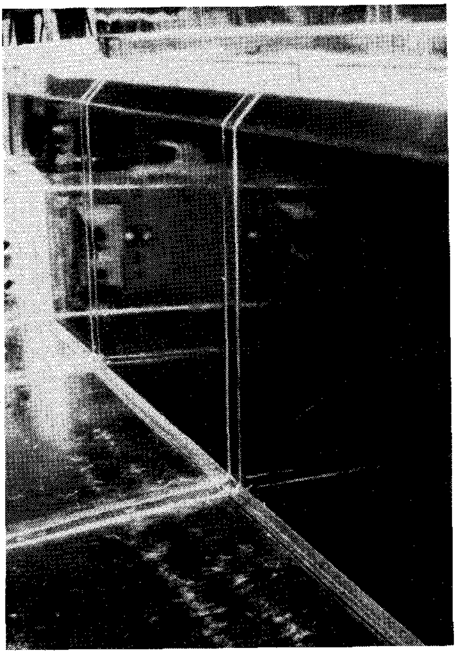


Fig. 10. Curved form filler plates.

Loaf Forms

The loaf forms used to form the inside faces of the webs were fabricated in 1500 mm (5 ft) long sections for curved beams (Fig. 11) and in 3000 mm (10 ft) long sections for tangent beams. While on paper the loaf forms had adequate horizontal stiffness to achieve the required web wall thickness tolerance, provided that the differential concrete level in the webs during concrete placement did not exceed 300 mm (12 in.), in practice, it was found that some modifications to the curved loaf forms were necessary, particularly for cases of significant cross slope. Because workers had to climb down into the loaf forms to

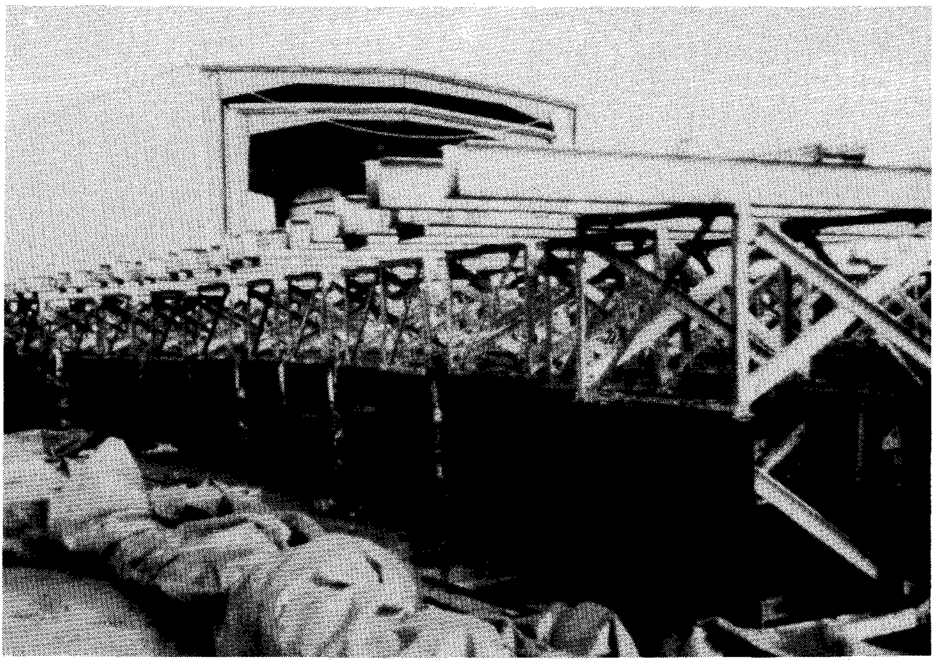


Fig. 11. Curved loaf forms.

trowel the surface of the bottom flange, some of the cross bracing had to be removed and additional restraint had to be provided by using coil rod spacers between the loaf forms and the outer tub form. These rods were withdrawn after the concrete had been fully placed.

The space between loaf form sections for the curved forms was filled with steel filler plates in a manner similar to that used for the outside forms. Caulking of the joints, however, was not required.

Soffit Forms

Approximately two-thirds of the beams were produced using hydraulically actuated collapsible steel soffit tables (Fig. 12) that were subsequently withdrawn from the open end of each beam. The performance of these tables did not live up to expectations, for the following reasons:

1. The edges of the tables would occasionally hang up on a concrete lip and excessive hydraulic pressure was required to retract the tables.

2. The forms were not robust enough to withstand the rough treatment, especially while being moved back to the casting bed over a gravel surface.

3. On several occasions, despite the fact that the tables had been designed with a theoretically failsafe over-center mechanism, fate led to premature collapse of the table before the deck concrete had set up and expensive repairs were required.

4. Withdrawal of the tables was on the critical path and valuable gantry time was used up waiting for the tables to be withdrawn, as they were needed almost immediately for the next beam.

When the tables were reaching the end of their useful life, production personnel had no difficulty in switching to one time use wooden soffit forms. These forms (Fig. 13), which were supported on top of the first stage web with galvanized steel fingers, were light, could be

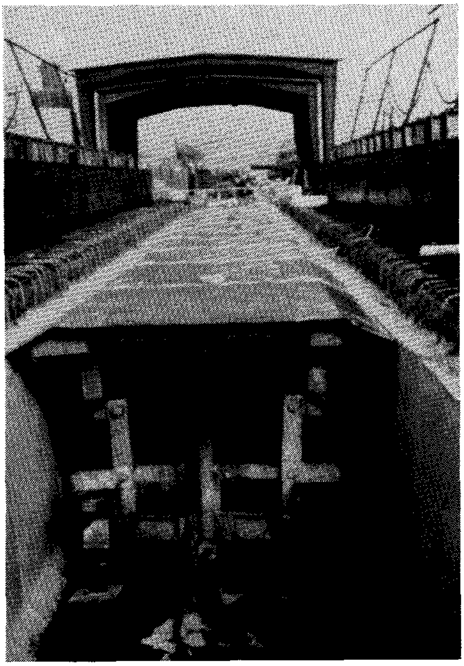


Fig. 12. Steel soffit forms.

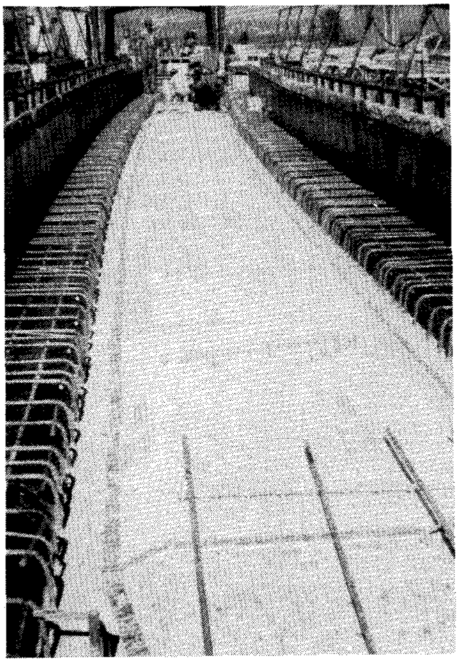


Fig. 13. Timber soffit forms.

manhandled and were installed in a fraction of the time required for the steel forms. Joints between forms were sealed with tape.

The cantilever ends of Type II side span beams did not lend themselves to removable steel loaf forms. Consequently, wooden voids, of somewhat complex geometry to accommodate the varying web and flange thicknesses, were used to form the inside surfaces of these beams. A 100 mm (4 in.) wide open space was left down the center of the bottom of these forms and inspection hatches were left in the top to check that concrete had fully filled the bottom flanges; no problems were encountered in this respect.

Insert Jigs

Specified tolerances for the location of inserts for the running rails, for the central linear induction motor (LIM) reaction rail and for the power rail in the parapet walls, were extremely demanding (Table 2). To meet the gage requirement of ±0.8 mm (1/32 in.) between pairs of running rail inserts, elaborate insert jigs were designed and fabricated (Fig. 14). The surfaces of these plates were machined and the bolts that held the threaded inserts had machined shanks. As previously stated, the location of each of these insert holding plates was checked during the 25 cast recalibration operation.

In addition to positioning the inserts, the tangent jigs carried both sides of the vertical parapet wall forms. These forms were pulled away from the wall surfaces prior to stripping with ratchet screw jacks. Some reworking of the pin sizes and brackets was required in order to achieve the specified parapet wall tolerances, which were critical because of the power rail bracket locations.

The insert jigs for the curved forms were similar in concept to those for the tangent forms, with the exception that each form was only 1500 mm (5 ft) long, and the outside parapet wall form was a part of the tub assembly not the insert jig assembly. Filler plates were required between parapet wall sections and caulking was required on exterior faces.

Tower Cranes

Five Linden L75 tower cranes (Fig. 4) with a capacity of 2 t (2.2 tons) at 34 m (112 ft) maximum radius and 5.5 t (6 tons) at 17 m (56 ft) radius were used to hoist lighter loads. By raising the height of two of these cranes, operating radii could be overlapped and almost total coverage of the plant was achieved (Fig. 3). These cranes saw heavy use, particularly on curved forms, where 24 tub sections and 24 insert jigs had to be either installed or withdrawn from each form, each day.

Concrete Mix Design

The mix design had to achieve strengths of 25 and 28 MPa (3600 and 4000 psi) at 14 hours and 42 MPa (6000 psi) at 28 days. The production procedure required a pumpable mix and a mix that would hold its workability during the finishing of the top deck. A Type 10 (ASTM Type 1) cement mix with a Master Builders superplasticizer and nonchloride accelerator was used. Initial trial batches using Type 30 (ASTM Type 3) cement had demonstrated too rapid an initial set and a degree of stickiness that hampered mixing, pumping and finishing.

Air entrainment of all concrete was in the range of 3 to 6 percent. All aggregates were completely tested for durability and potential reactivity prior to the start of beam production. Aggregate piles were sprinkled with water and cement had to be drawn from a special source in order to keep its temperature below the specified limits during the summer months. Aggregates and water were heated during the winter months.

Concrete Batching and Placement

The batch plant consisted of a Ross portable plant elevated on columns, an extra cement silo, additional screw conveyors and an aggregate charging conveyor. A custom designed mixer building complete with a 2 m^3 (2.6 yd^3) Smith pan mixer and a Western Scale automatic batch control system, was constructed.

For what is believed to be the first application in a North American precast plant, two stationary concrete pumps were used to distribute the concrete to the forms. During the design of the plant, consideration had been given to concrete placement using buckets, side winders or conveyors. However, the sheer size of the forms, each with a plan area of 6 x 36 m (20 x 118 ft) and a height of 5 m (16 ft), the need to cast in all eight forms every day, the distance between forms and the limited tower crane time available, led to the decision to use concrete pumps.

The performance of the system was extremely satisfactory; the system is currently being used for the production of other types of precast components with a very high productivity factor. The only drawbacks to the system are occasional line blockages if pumping is interrupted, and the relatively high concrete wastage.

The system consists of two holding hoppers beneath two gates in the mixer, with two Elba-Scheele high capacity Model 5016 trailer mounted pumps at ground level. Slicklines [125 mm (5 in.) diameter] from each pump feed the concrete to three Elba placing booms (Fig. 15). These units consist of standard, hydraulically actuated placer turrets mounted on custom designed carts which travel the length of the forms on rails. To connect the stationary end of the slickline to the moving placer boom, a knuckle jointed accordian arrangement of pipes was custom designed for the project.

Table 2. Summary of beam tolerances.*

Measurement	Specified tolerance†	Percent of measurements within specified tolerance: Verification beams	Percent of measurements within specified tolerance: Sample beams	95 percent of measurements within†	99 percent of measurements within†
Running rail insert offset		97			
Tangent beams	±6		98	±5.3	±6.9
Curved beams	±6		90	±7.4	±11.6
LIM rail insert offset		91			
Tangent beams	±6		99	±4.2	±6.0
Curved beams	±6		89	±8.0	±12.6
Power rail insert offset	±3 85% ±5 100%				
Tangent beams			68/86	±7.3	±10.2
Curved beams			57/80	±7.8	±10.2
Running rail insert vertically	±2°	99	100	±.5°	±.8°
LIM rail insert vertically	±2°	93	100	±.7°	±1.0°
Cross slope of rail seat plane	30:1 to 50:1 1.91° to 1.15°	—	98	±.25°	±.37°
Beam end depth					
Fixed end	−6 to +12	—	96	±9	±12
Expansion end	±3	—	88	±4	±6
Beam camber					
Tangent beam		—	—	±17	±23
Curved beam		—	—	±12	±18
Gage of LIM inserts	±6	100	100	±1.0	±2.3
Gage of running rail inserts	±0.8	89	97	±.7	±1.1

*Kirkness, A. J., and Groves, J. S., "Quality Assurance for a Major Transportation Construction Project," *Concrete in Transportation*, ACI SP-93, American Concrete Institute, Detroit, Michigan, 1986, 929 pp. Reproduced with the permission of the authors and publishers.

†Expressed in mm unless otherwise noted.

It is a credit to the concrete mix designer that 80 mm (3 in.) slump concrete could be pumped around these bends after having traveled 150 m (450 ft) from the pumps. The end of the drop pipe can be positioned to any height, in any position, in any form, by an operator standing on top of the form utilizing a control panel. Using the single mixer, two pumps and two boom placers concurrently, a concrete placement rate of 40 m^3/hr (52 yd^3/hr) was achieved.

Concrete vibration was by means of both internal and external vibration. All

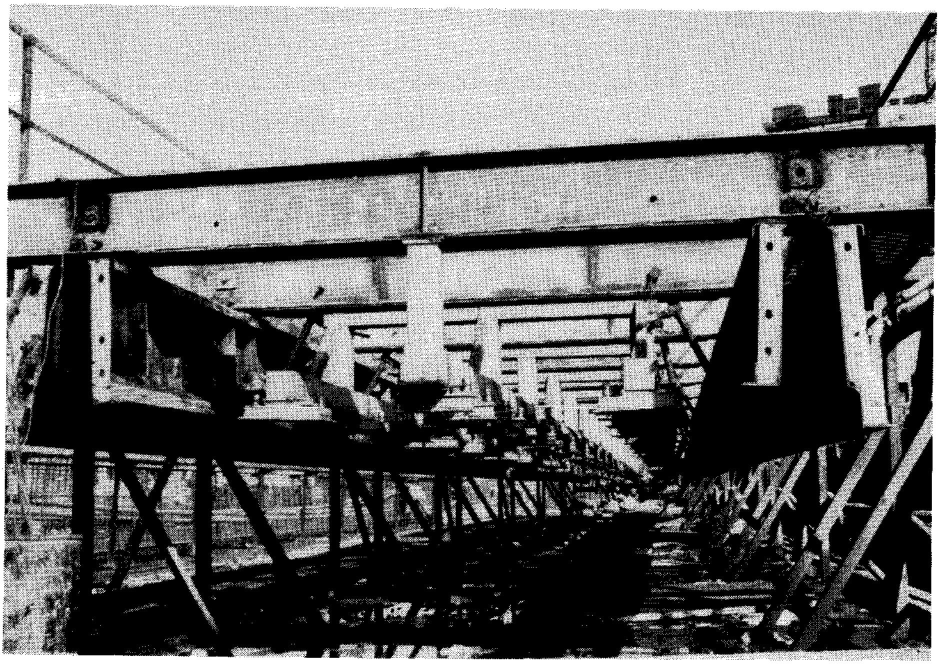


Fig. 14. Tangent parapet wall form and insert jig.

forms were equipped with Vibratrack vibrators on both sides of the form. Careful control of external vibration was required in order to ensure complete compaction while, at the same time, minimizing the quantity of concrete emerging from the top surface of the bottom flange; this excess concrete had to be shoveled back into the top of the webs by hand.

Accelerated Curing

The use of insulated tarpaulins was deemed to be impractical due to the size of each enclosure. Conventional tarps were "flown" into position using spreader bars and the tower cranes. During some windy days, this was a difficult task and a relatively high amount of damage to the tarps resulted. Tarps were held in place by 200 x 200 mm (8 x 8 in.) timbers.

Steam was produced using three Johnson 1 MW (3.5 x 10^6 Btu/hr) low pressure steam generators. Steam was piped underground to the beds using 150 mm (6 in.) diameter pipes. Following a 3½ hr preset period, the curing temperature was increased to 60°C (140°F). Temperatures were monitored with a series of thermocouples and a 24 track temperature recorder.

In retrospect, the steam curing system was slightly undersized during periods of low temperature; several days of continuously subfreezing weather were experienced.

Stripping and Yarding

Beams were stripped from the forms using two rubber tired Ropco Speedloader gantries. These machines each have an inside clear width of 12 m (40 ft), a wheel base of 6 m (20 ft), a hook height of 9 m (30 ft) and a capacity of 68 t (75 tons). Tangent beams were stripped using the cast-in Dywidag lifting bars at each end.

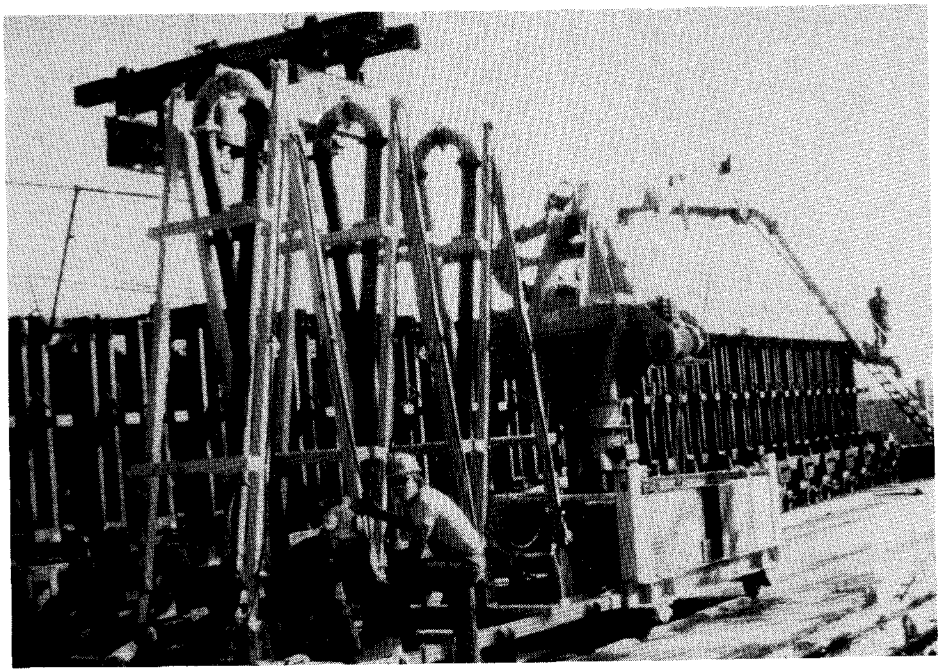


Fig. 15. Concrete placing boom.

Curved beams, however, because they had not been prestressed prior to stripping, were stripped from eight points on the deck. To ensure equal load distribution, elaborate "teeter totter" swivel spreader beams were designed (Fig. 16). The exact positioning of these spreaders, and the offset of the center of gravity from the centerline of the beams, were predetermined to ensure that the beams did not bind in the forms during stripping.

Unfortunately, the design of this lifting hardware was not completely foolproof. The fouling and failure of the lifting gear during loading resulted in the writeoff of two beams. Fortunately, nobody was injured.

All beams were yarded onto timber cribs. Curved beams were placed with an additional bunk at midspan (Fig. 17); these bunks had been previously surveyed into position to prevent cracking of the beams. Before production was commenced, load tests of the proposed bunking system were undertaken and settlements of less than 6 mm (¼ in.) were experienced, despite the fact that the plant is situated on extremely poor soil conditions.

At the commencement of erection in January 1984, 430 beams had been produced. The storage of these beams occupied an area of 8 ha (19 acres), approximately three times the area shown in Fig. 18. This was a severe test of the confidence that everybody associated with the project had placed in the designer, the computer generated formset data, the precaster's quality control program and the system contractor's quality assurance program.

The last beam was stripped from its form on October 3, 1984, 18 months after the casting of the first beam.

Plant Post-Tensioning

Curved beams were post-tensioned in the yard prior to shipment. A multiple

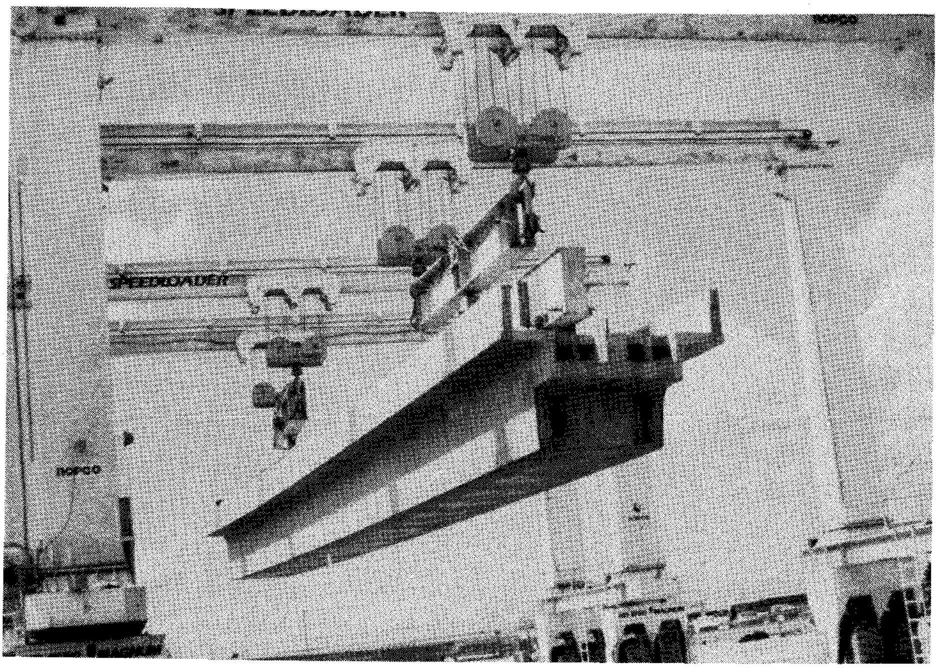


Fig. 16. Yarding a curved beam.

strand jack was utilized (Fig. 19). Up to six Freyssinet 7-13 mm (½ in.) diameter tendons were required in each beam. All ducts were grouted in the yard using a prebagged expanding portland cement grout. Very few problems were experienced with either the stressing or the grouting operations.

Finishing

Because of the high visibility of the guideway in the downtown Vancouver area, all surfaces of the beams were sack rubbed. To prevent rust staining prior to erection, the protruding bars at fixed ends were painted with zinc rich paint.

Although the form joint marks are visible at close range when viewed from ground level (Fig. 19), joints on beams in the guideway are only noticeable to precasters, architects and others intimately involved in the project.

Plywood bulkheads were installed in the open fixed ends of beams to contain the field placed crosshead concrete.

An early concern of the owner was the presence of air bubble voids on the surfaces formed by the insert holding plates. Particular concern was expressed at the possibility of reduced structural bearing capacity at the running rail bearing plates, which were designed to bear directly onto the deck (Fig. 28). A project was undertaken jointly by the system contractor and the precaster during the mobilization stage to develop a machine that would insert the threaded inserts into the freshly screeded deck concrete.

Promising results were obtained with the machine with respect to the LIM inserts, for which a positional tolerance of 6 mm (¼ in.) was specified. However, when the type of running rail pad was finally selected, with its attendant gage tolerance of 0.8 mm ($^1\!/_{32}$ in.), it became apparent that it was unlikely that the in-

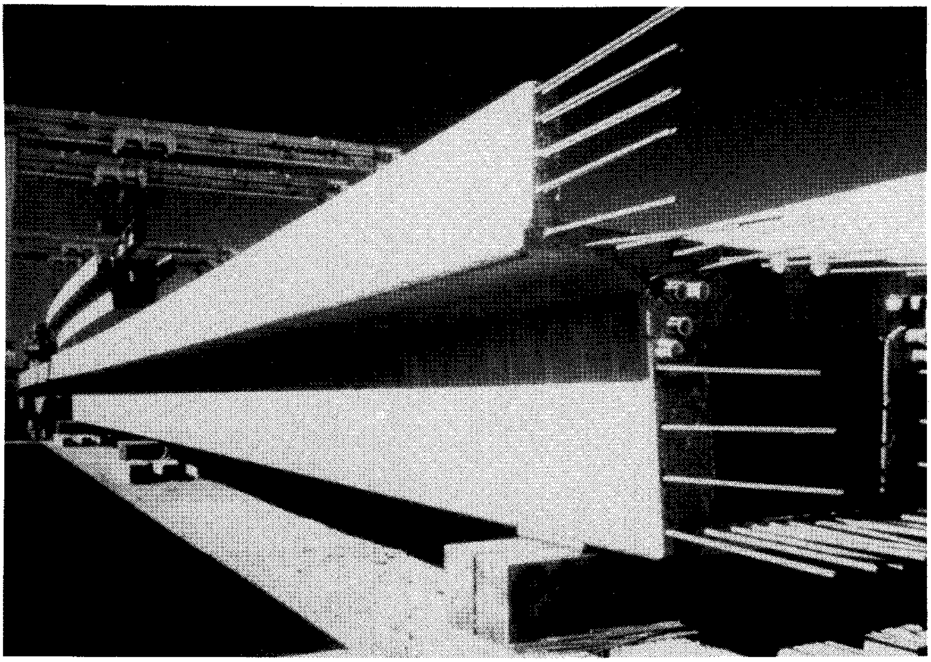


Fig. 17. Bunking of a curved beam.

Fig. 18. Portion of beam storage yard.

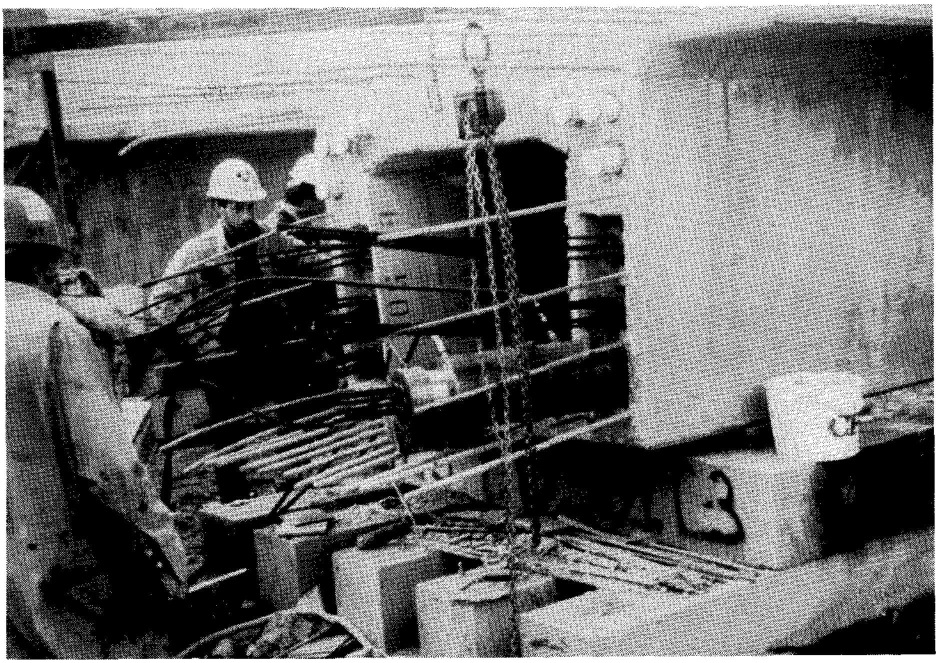


Fig. 19. Plant post-tensioning.

sert placing machine could ever be developed to this accuracy, and therefore the project was abandoned. Several other attempts were made to alleviate the air void problem, but eventually, the solution selected was to patch the voids with an epoxy grout.

Special Beams

All guideway beams were produced in the forms previously described; no penetration of the exterior forms was required. However, there was a requirement for several special types of beams.

Beams for Type I, three-span continuous structures are very similar to standard beams. The center span has fixed end details and deck manholes at both ends. However, beams for Type II, three-span continuous structures, with midspans of up to 45 m (148 ft), have added features. The side span beams for dual lane guideway structures are supported on typical bearings at the expansion end, but are cantilevered from columns at approximately 5 m (16 ft) from the other end.

This fixed end connection (Fig. 20) required that ducts and anchorages for 14 - 36 mm (1⅜ in.) diameter Grade 1030 (150 ksi) Dywidag lateral post-tensioning bars and 21-35 mm (#11) diameter Grade 400 (60 ksi) bars and couplers be cast into a diaphragm in the beam. This was a detailing challenge to fit this hardware into the beam, while at the same time clearing the longitudinal post-tensioning ducts and all of the torsional reinforcement.

Close communication and cooperation between the designer and the precaster enabled these details to be resolved in an expeditious manner. At the point between the cantilevered end of the side spans and the drop-in beams, two large 975 x 1000 x 100 mm (38 x 40 x 4 in.) steel plates (Fig. 21), each with

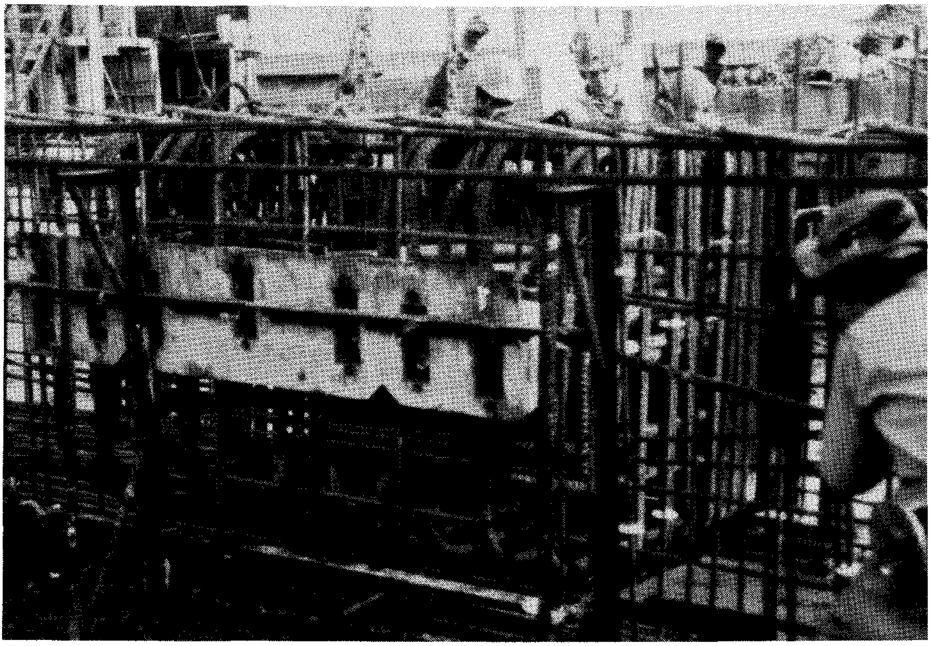


Fig. 20. Crosshead connection hardware for Type 2 cantilever beam.

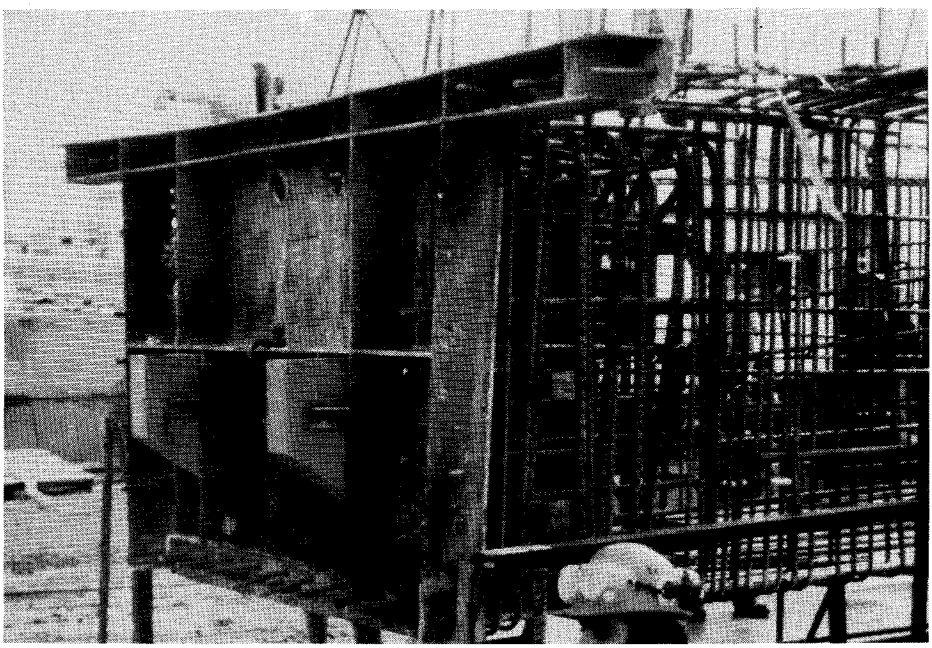


Fig. 21. Support bracket for drop-in beam.

100 headed studs, were cast into each web. Special steel end bulkheads were attached to these end assemblies before the reinforcement cage was placed in the form.

To support the roof structure in some of the stations, hoop truss anchor plates consisting of triangular plates 1300 mm (51 in.) long on each side and 25 mm (1 in.) thick with 3 - 36 mm (1⅜ in.) diameter Dywidag bars were cast into special diaphragms in the beams. This required the use of special loaf form sections which, however, remained modular.

Special trackwork beams (Fig. 22) for track switches and crossovers were fabricated by substituting 100 mm (4 in.) high side forms for the top flange in place of the insert jig and parapet wall forms. Protruding shear reinforcement and sandblasting of the deck were used to provide the required composite action for the cast-in-place deck slab.

In addition to the guideway beams through the stations, a separate contract was awarded for the construction of platform beams. These simple span beams, which were slightly deeper than the guideway beam cross section, were cast in a separate special form by Con-Force Structures Ltd.

Quality Control and Quality Assurance

There is no doubt that the success of this project was largely due to the emphasis placed on quality assurance by the owner and the system contractor, and on quality control by the precaster and his subcontractors.

The precaster's quality control team consisted of a quality control engineer and a staff of twelve surveyors and inspectors. In addition to routine tests on fresh concrete, cylinder testing, reinforcement inspection and finished beam inspection, considerable effort was expended to ensure that the specified geometrical tolerances were achieved. During each form recalibration, there were over 700 points on the form which were surveyed, adjusted and double checked.

For each tangent beam there were 50 formset measurements and for each curved beam there were 144 formset measurements which were set by production personnel. Each measurement was then checked and recorded by a quality control inspector and random samples of measurements were double checked by a quality assurance inspector. Prior to casting, the alignment of the form and the insert jig were checked with a transit and a level mounted on survey walls at each end of each bed (Fig. 23). Prior to stripping, the elevation and offsets of six pins placed in the beam decks were surveyed. Finally, as-built checks of beam geometry, insert location and camber were surveyed while the beams were in the storage yard.

The quality assurance program utilized a signoff procedure in which production could not proceed beyond certain stages until inspections had been performed by the precaster, and confirmed and signed off by the resident engineer's staff. Nonconformance reports were raised for each nonconformance and a system of remedying, checking and signing off these reports was instituted. Major and minor nonconformances were resolved at the appropriate level of management, thus preventing the project from becoming bogged down in paperwork. In order to meet the deadline imposed for completing the guideway, the system contractor set rigid time limits on the resolution of any problem.

BEAM TRANSPORTATION, ERECTION AND COMMISSIONING

Transportation

The erection schedule called for the precaster to deliver six beams per day to

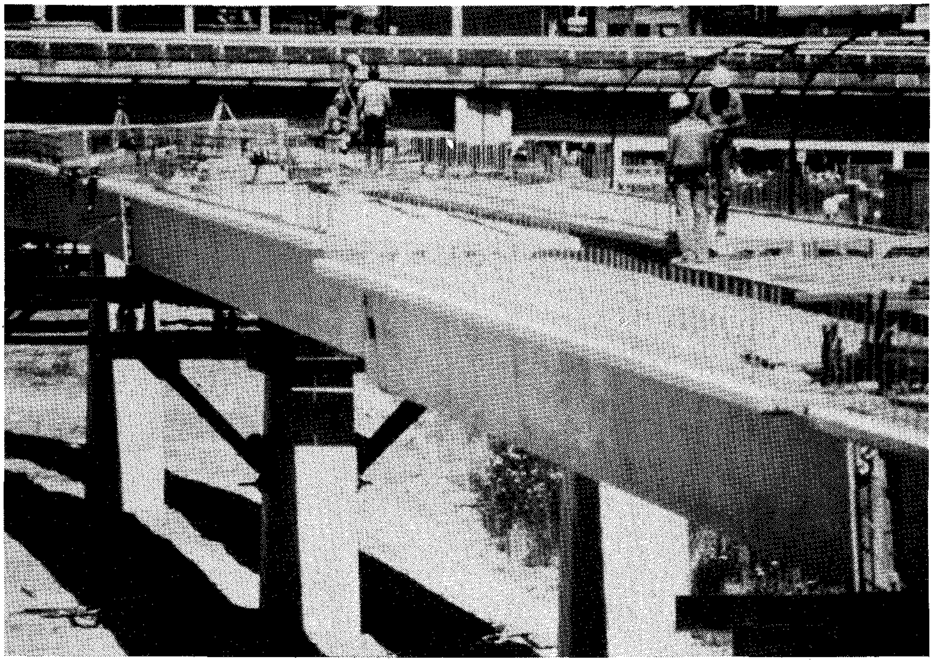


Fig. 22. Special trackwork beam.

Fig. 23. Geometrical control.

Fig. 24. Beam transporter.

any location on the construction right of way. In order to meet the highway loading regulations of the Province of British Columbia and to negotiate the street corners in the downtown area, carrying beams up to 36 m (118 ft) long and 100 t (110 tons) in weight, six special 13-axle transporters were built (Fig. 24) by the precaster's hauling subcontractor. Two of these units had two additional booster axles to carry the heaviest beams.

The rear assembly was steered by an operator in a cab, but the unit was not self powered. The total length of each loaded transporter was 59 m (194 ft) and the transporters had to be returned to the yard in two separate parts because the units were overlength, even when the two halves were drawn together. The offsets of bunking points, to accommodate beam curvature, were precalculated.

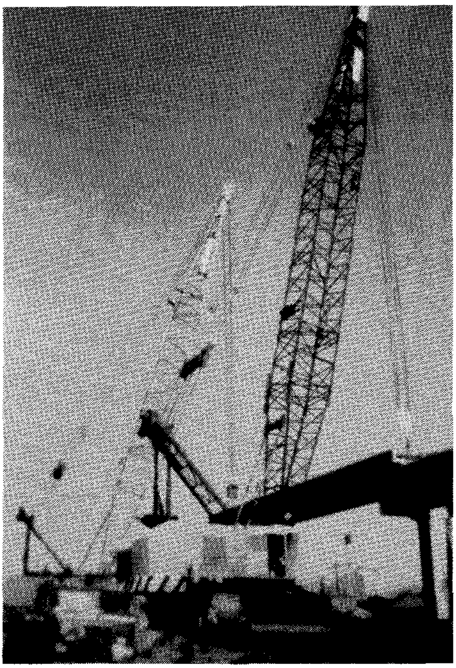


Fig. 25. Beam erection.

Beams were loaded in the evening

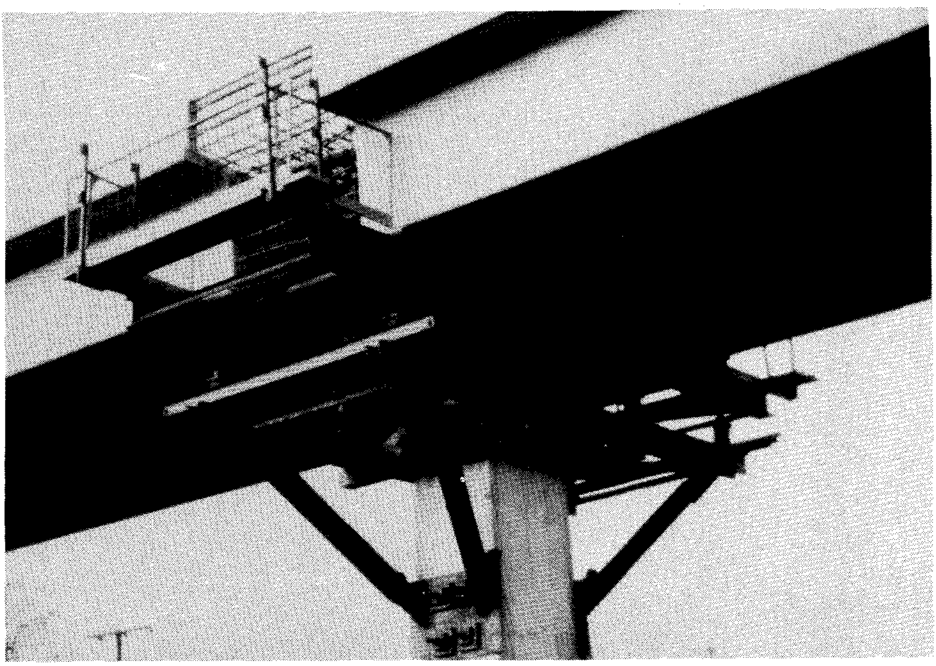


Fig. 26. Fixed end falsework.

and transported to the site between the hours of 2 a.m. and 5 a.m. with the assistance of pilot cars and a police escort. Despite the fact that one of the approach roads has a 9 percent grade, no difficulties, accidents or incidents were encountered during beam transportation.

Erection

The erection contractor utilized two Manitowoc Model 4100, 210 t (230 ton) capacity crawlers to erect all of the beams on Phase II (Fig. 25). Beams for typical two-span structures were supported on sliding bearings at the expansion ends and on steel falsework at the fixed end (Fig. 26). These falsework assemblies, which were clamped onto the fixed columns with Dywidag bars, provided for vertical adjustment of the beam bearing points and acted as the soffit form for the beam closure concrete.

Although this concept worked extremely well from a construction viewpoint, the time required to complete the closure, strip the falsework and recycle it to its next location proved, in fact, to be longer than anticipated and many more sets of falsework than originally planned had to be acquired. There would be considerable merit in investigating alternative fixed end connection details that would eliminate the requirement for this falsework on future projects.

The only means of adjusting the vertical height of the expansion bearings was by means of shims. Lateral tolerance was taken up by oversizing the pocket into which the upper bearing plate dowels protruded. Final erection tolerances were achieved, but at the expense of a considerable survey cost. A more forgiving wet type bearing connection might have proved to have been more economical.

Six beams with a radius of curvature of approximately 75 m (250 ft) (Fig. 31) required counterweights during erection to maintain their stability. Large tanks filled with gravel were suspended from outrigger beams bolted to the deck for this purpose.

Erection of the first beam on Phase II turned out to be a public relations nightmare. A large crowd was present and the press were out in full force. Approximately 20 minutes after the beam had been lifted from the transporter and the transporter driven from under the load, a failure of one of the lifting devices resulted in the beam crashing to the ground and being completely destroyed. Fortunately, no one was injured.

The cause of the accident was attributed to the mistaken use of a left handed Grade 400 (60 ksi) Dywidag bar being screwed into a right handed Grade 1030 MPa (150 ksi) coupler. While the lower strength bar would have had sufficient tension capacity, the crossing of threads did not provide sufficient thread bearing area. Nobody associated with the project had realized that it was possible to mix the right and left hand threads.

The accident investigation was completed in record time. A replacement beam was fabricated within one week, the quality control of lifting devices was intensified and the erection contractor proof loaded each lifting bar with an hydraulic jack for the remainder of the project.

Two of the drop-in beams at the New Westminster end of the project posed a particular challenge as they had to be erected over a 40 m (131 ft) wide building. The cantilever spans were erected, a steel truss was placed over the 36 m (118 ft) wide opening and each beam was launched with the use of two cranes. One crane was then moved to the opposite end, the truss was skidded laterally on tracks and the beam was lowered into position. These particular beams form part of a special five-span continuous structure.

The first beam was erected on January 9, 1984, and the last beam was "topped out" on November 9, 1984, exactly 10 months later.

Closures

The erection contractor placed reinforcement in the fixed end closures (Fig. 26) and the post-tensioning subcontractor placed ducts and fed the strands into the ducts. The erection contractor then formed the outside surfaces, installed cans for the deck inserts and placed the concrete for the beam closures with the aid of a concrete pump and boom truck.

For beams over 30 m (98 ft) in length, parapet walls were field constructed in order to limit the beam transportation weight to 100 t (110 tons).

Field Post-Tensioning

Field post-tensioning consisted of either two or four tendons of 12-13 mm (½ in.) diameter strands in each span, arranged in a cross-over pattern. Each tendon ran from a static anchorage at the top of the dap at the expansion end, down to the bottom of the web at midspan, up to the intersection of the top flange and web at the fixed end and over to a live end anchorage in a manhole (Fig. 27) in the deck at approximately the one-fifth span point of the adjacent beam. Tendons were stressed with a multistrand jack.

A similar scheme was used for three-span continuous structures, with the exception that tendons for the center span were anchored in the deck of both adjacent side spans. These tendons were stressed from both ends.

The only problem encountered during stressing was the failure of one anchorage block. The cause of this failure was traced to a problem with steel chemistry at the foundry.

All tendons were grouted within 7 days of stressing with a portland cement grout using an expansive admixture.

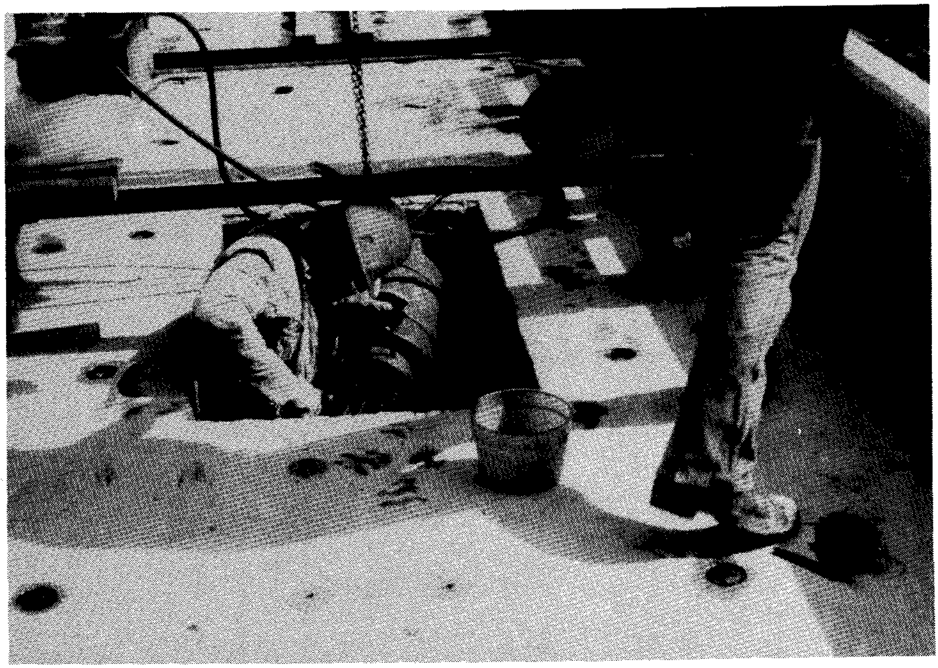


Fig. 27. Field post-tensioning.

Track Laying

The rail contractor electric flash butt welded 23.8 (78 ft) long sections of rail into 450 m (1476 ft) long strings and pulled them down the guideway on rollers. Lord running rail pads (Fig. 28) were laid on 500 mm (20 in.) centers on curves with a radius of less than 1400 m (4600 ft) and on 1000 mm (40 in.) centers for the remainder of the guideway. Each pad consists of a steel and elastomeric assembly which was bolted directly to the guideway beam inserts. The rail was then clamped to these pads with bolts, with provision for a lateral adjustment of up to 19 mm (¾ in.). Vertical adjustment was obtained using steel shims. While the majority of the shims were in the range of 5 to 10 mm (3/16 to ⅜ in.) thick, a maximum shim height of 22 mm (⅞ in.) was required.

During the initial stages of the project, many people believed that the direct fixation of the track to the beams was neither practicable nor economic. However, with the successful completion of track laying in record time, the consensus is that this is the direction that track fixation will take in the future.

Commissioning

Commissioning and testing of the first section of the guideway was commenced in the summer of 1984 (Figs. 29 and 30). Integration of the remaining sections of the aerial guideway, the 1.3 km (0.8 mile) long underground section in the downtown area, the 3.5 km (2.2 miles) of at grade section and the maintenance yard continued through 1985. The system was open for revenue service on January 3, 1986.

The first "extension" to the system is currently under construction. A 300 m (1000 ft) span, precast concrete cable stayed bridge, the largest such span in the world devoted to rapid transit, is being built over the Fraser River. The

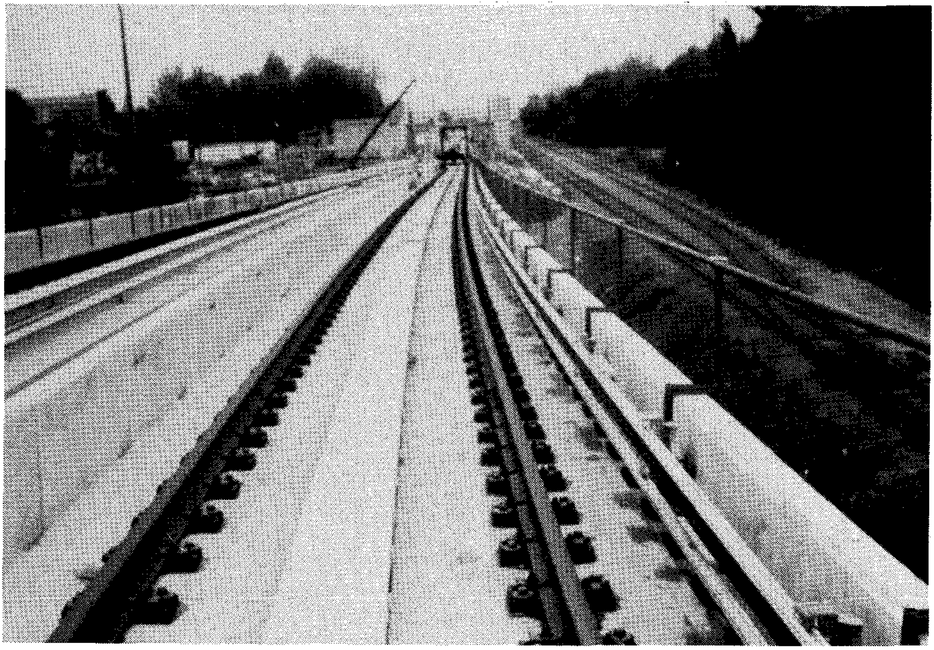


Fig. 28. Power, LIM and running rails.

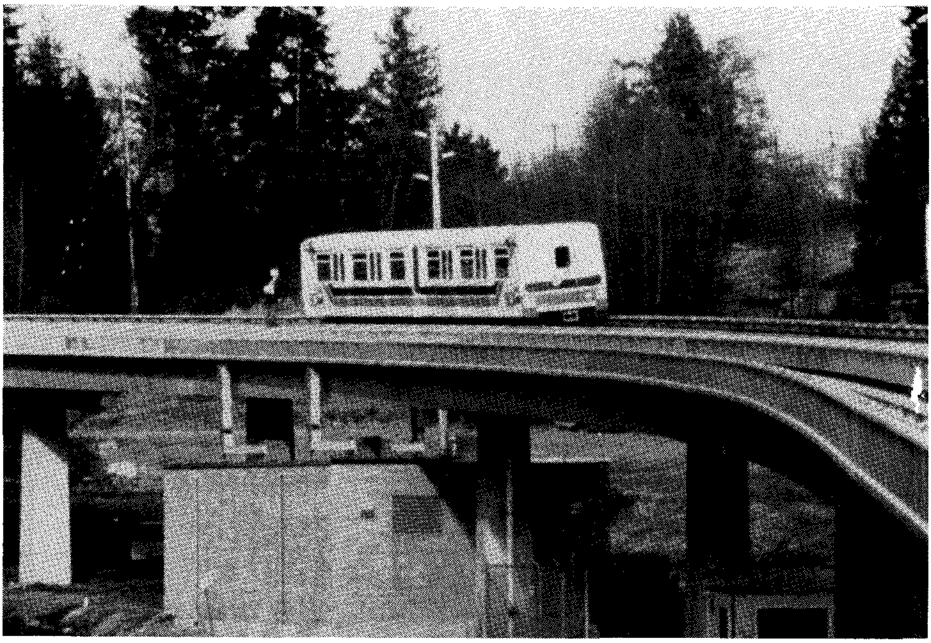


Fig. 29. Vehicle and system testing.

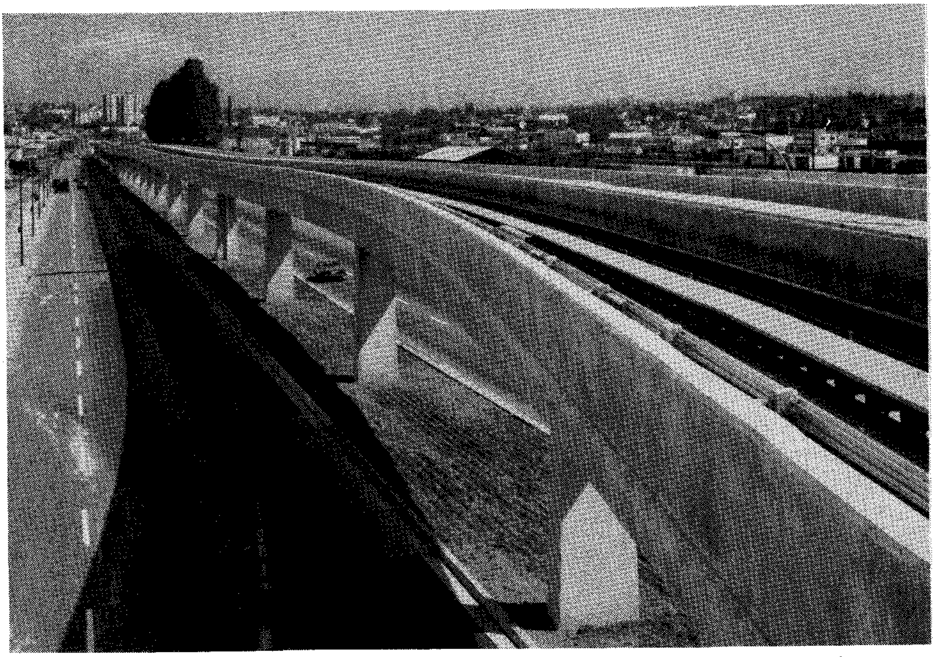


Fig. 30. Modular design concept reduces guideway to assembly of standard products.

north and south approaches to this bridge, a 77-beam section of guideway and two additional stations will complete the second phase into the Scott Road area of Surrey. Planning of extensions into Whalley, Coquitlam and Richmond is also in progress.

Figs. 31 and 32 show completed portions of the aerial guideway for the Vancouver ALRT project.

CONCLUDING REMARKS

Precast, prestressed curved box girders were found to be the most economical structural solution for the aerial guideway for the Vancouver Advanced Light Rapid Transit (ALRT) project. The design has the additional benefit of an extremely attractive physical appearance based upon the long clear spans, the continuously curved geometry of the beams, the in-depth crossheads and the well proportioned columns.

The precasting of the beams to the very tight, nonnegotiable schedule was a major challenge to the precaster.

Although problems, both minor and major, developed on the project, the overwhelming dedication of the owner, the systems contractor, the consultants, and the contractors to complete the project "on time" and "on budget" led to highly effective decision making procedures. The quality control and quality assurance programs developed could well act as models to other jurisdictions which find themselves getting bogged down in a paper war, schedule delays, cost overruns and difficulty in obtaining decisions.

The plant design, the formwork concept and the concrete placing innovations were all extremely successful. Numerous small improvements will no doubt be made on the next project. The

Fig. 31. Finished portion of the aerial guideway for Vancouver ALRT project.

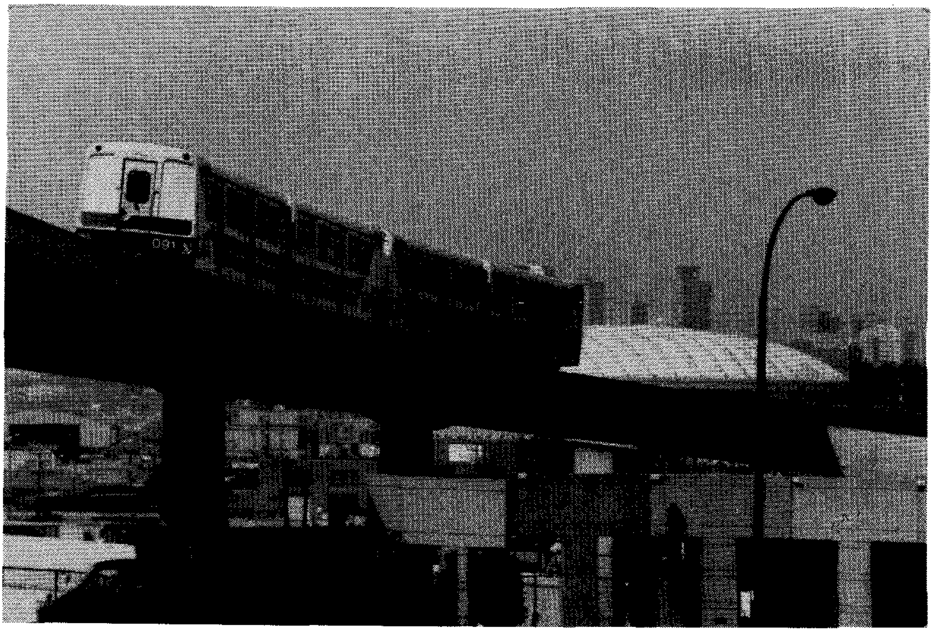


Fig. 32. Another view of aerial guideway with city skyline in background.

elimination of erection falsework and the use of a single stage cast appear to be areas in which cost savings could be achieved.

The actual casting and erection tolerances achieved on this project could be used as a basis for future specifications. In particular, the specification should clearly distinguish between the tolerances which are critical for the subsequent installation of track systems, and others which are of secondary importance. With the complex geometry of such guideways, one often cannot see the forest because of the trees.

In the author's opinion, it has been demonstrated that the direct fixation of trackwork to the guideway beams is not only a possible alternative, but can also be a cost effective solution.

Every indication is that this type of guideway could be the optimum choice on future projects, particularly those in urban areas, with tight radii and where aesthetics is a major criterion for design.

CREDITS

Owner: British Columbia Transit, Vancouver, British Columbia.

System Contractor: Metro Canada Ltd., Vancouver, British Columbia.

Guideway Designer: ABAM Engineers, Inc., Federal Way, Washington.

Prebuild Precaster: Con-Force Structures Limited, Vancouver, British Columbia.

Phase II Precaster: Supercrete, A Division of Canada Cement Lafarge Ltd., Vancouver, British Columbia.

Prebuild Erector: Commonwealth Construction Ltd., Vancouver, British Columbia.

Phase II Erector: Peter Kiewit Sons Co. Ltd., Vancouver, British Columbia.

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