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# Vancouver ALRT Special Guideway Structure

Structure spéciale pour le métro de Vancouver

Spezielle Hochbahn-Tragstruktur des ALRT-Systems in Vancouver

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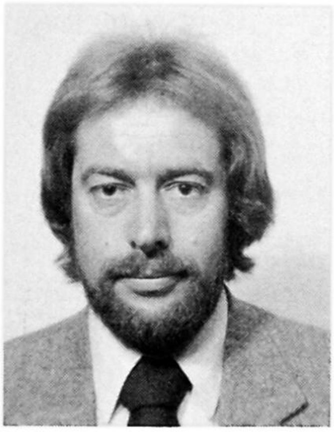

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## SUMMARY
Vancouver's Advanced Light Rapid Transit (ALRT) System is an ambitious transit project that integrates structural efficiency, environmental sensitivity, and electronic control technology. This paper describes the design and construction of a three-span continuous cast-in-place post-tensioned box girder structure that comprises a segment of the concrete elevated guideway.

## RESUME
Le métro de Vancouver est un projet ambitieux qui intègre les problèmes liés aux structures, à l'environnement et au système de contrôle électrique. Cette contribution décrit le projet et l'exécution d'un caisson continu sur trois travées faisant partie du tracé surélevé.

## ZUSAMMENFASSUNG
Das Advanced Light Rapid Transit (ALRT)-System ist ein ehrgeiziges Vorhaben eines Transportsystems, das bauliche Leistungsfähigkeit, Rücksicht auf die Umweltbelastung und elektronische Kontrolltechnologie vereint. Der Beitrag behandelt den Entwurf und den Bau eines an Ort betonierten, vorgespannten, dreifeldrigen Kastenträgers, der Teil der erhöhten Betontragstruktur ist.

## 1. INTRODUCTION

In the early 1970's the sharp increase in the price of non-renewable fossil fuels, traffic congestion, and environmental pollution in large cities stimulated keen interest in alternative and more energy efficient forms of public mass transit. The Urban Transportation Development Corporation Ltd. (UTDC) was created by the Ontario Government in 1972 to investigate and develop urban transportation alternatives. Vancouver's Advanced Light Rapid Transit (ALRT) System is the culmination of the UTDC developmental program and is the first full-scale application of this rapid transit technology. Prime contractor for the Vancouver ALRT is Metro Canada Ltd., a subsidiary of UTDC. Owner and operator of the Vancouver ALRT is B.C. Transit, a crown corporation of the Province of British Columbia.

The first phase of construction comprises a 21.4 km long dual guideway that is completely separated from all conflicting traffic to form a continuous, uninterrupted, dedicated right-of-way that will allow fast, efficient service. Thirteen kilometres of the guideway is divorced from traffic conflicts by an elevated structure that varies in height from six to eight metres above the ground. The elevated guideway superstructure is composed primarily of standardized precast concrete beam elements, the only exception being where geometric complexities dictate that a different type of construction be used. One such exception is "Special Switch Structure S232 Outbound". This latter structure is the subject of this paper which describes the design and construction of this complex segment of the concrete elevated guideway.

## 2. STRUCTURE LOCATION AND CONFIGURATION

Special Switch Structure S232 Outbound comprises the initial segment of the outbound elevated guideway immediately east of the Stadium Station. The special guideway structure is adjacent to the site that is being developed for Vancouver's world fair Expo '86. The special guideway structure passes over Pacific Boulevard, the main arterial route to Vancouver's domed stadium.

The Stadium Station has one storage outbound track in addition to the primary outbound track. The storage outbound track converges with the primary track east of the station and the switch is located on this special guideway structure. As a consequence, the structure is bifurcated, with the deck tapering in plan in the vicinity of the switch. The total length of the structure is 97 m.

## 3. DESIGN CRITERIA

Since the ALRT system incorporates specialization in the vehicles, structural system, trackwork, and communication technology, UTDC established design criteria to supersede or supplement existing design codes. The structural design was completed in accordance with the "Vancouver Advanced Light Rapid Transit System Design Manual". Aspects of the structural design not covered by the latter design manual conform to the Canadian code "Design of Highway Bridges" CAN3-S6-M78.

The design vehicle is a light automated aluminum and fibreglass car that has a normal load of 76 passengers and a crush load of 105 passengers. The live load adopted for the design is for the crush passenger load and is equivalent to 16.34 KN/m. Impact is 10% in all areas other than switch areas where the impact is increased to 30%. Each train comprises pairs of vehicles with up to six vehicles per train. The vehicle length is 12.7 m between couplers.

In addition to the considerable contribution from the girder self-weight, the superstructure dead load includes the 180 mm thick concrete deck overlay. The dead load was the dominant force in the design of the superstructure girder.

The use of continuous welded rails that are fastened directly to the deck introduces several unusual design forces and modifies the distribution of other superstructure design forces. Variations in temperature above or below the temperature at rail installation develop significant rail/structure interaction forces that must be considered in the design of both the superstructure and substructure. The presence of a switch on the special guideway structure provides an additional imbalance in rail temperature forces. The rare occurrence of a rail breaking is also considered, the loading shock being partially distributed to adjacent structures through continuity of the remaining unbroken rails. Similar force distribution to adjacent structures also occurs for localized forces such as normal or emergency braking.

In addition to rail/structure interaction forces, temperature load arises from a thermal gradient in the deck and a temperature differential between ambient air temperature and the air temperature of the superstructure's box girder void. The top surface of the deck was assumed to be 22°C higher than the underside of the top flange and underlying girder. The box girder void was assumed to be 22°C higher or 28°C lower than the ambient air temperature.

Vancouver's proximity to active seismic zones is reflected in the severe earthquake design provisions that are incorporated in the structure. The stratigraphy of the ground in the vicinity of the structure consists of a silt zone varying in depth from 3 m to 10 m overlying a stiff conglomerate till. The combination of this foundation material and the stiffness of the elevated structure resulted in a seismic design force equivalent to a static horizontal load of 20% of the vertical dead load.

## 4. STRUCTURAL SYSTEMS

### 4.1 Superstructure Structural System

The special guideway structure is illustrated in Figure 1. The superstructure consists of a post-tensioned concrete box girder supporting a sidewalk on one or both sides of the girder. The girder carries a concrete overlay, running rails and fasteners, linear induction motor reaction rail, power rails on the inside face of the parapet wall, cable conduits in a cable tray underneath the sidewalk, and an automatic train control data communication loop between the running rails.

The standard superstructure module for the elevated guideway is a two-span continuous structure comprised of two precast post-tensioned concrete box girders of span lengths up to 33 metres. Because of the non-uniform deck shape tapered in plan, the long centre span, and clearance limitations over Pacific Boulevard, a three-span continuous cast-in-place concrete box girder was adopted for the superstructure. The bifurcation of the girder to accommodate the storage track changes the box configuration from single-celled to triple-celled at the station end.

As noted previously, the rails are fastened directly to the deck. Presence of the switch requires considerable clearance underneath the rails to accommodate the switching equipment. Allowance for the switch clearance envelope led to the introduction of the 180 mm concrete overlay on top of the box girder top flange.

### 4.2 Substructure Structural System

The superstructure is supported by three expansion piers and a fixed pier. The expansion pier at the west end is shared with the Stadium Station superstructure. The two expansion piers east of the Stadium Station have single columns with uni-directional bearings, the columns being positioned along the girder centreline. The fixed pier has two laterally-spaced columns that are cast integrally with the girder to form a rigid connection. Adoption of one fixed pier rather than a fixed pier either side of the centre span minimized secondary stresses due to live load, post-tensioning, shrinkage, creep, and temperature change.

All pier columns are supported on piled footings, the piles being required to reduce differential settlements in the underlying silt. All piles are tubular steel piles and are filled with concrete. Footings and columns are conventionally reinforced.

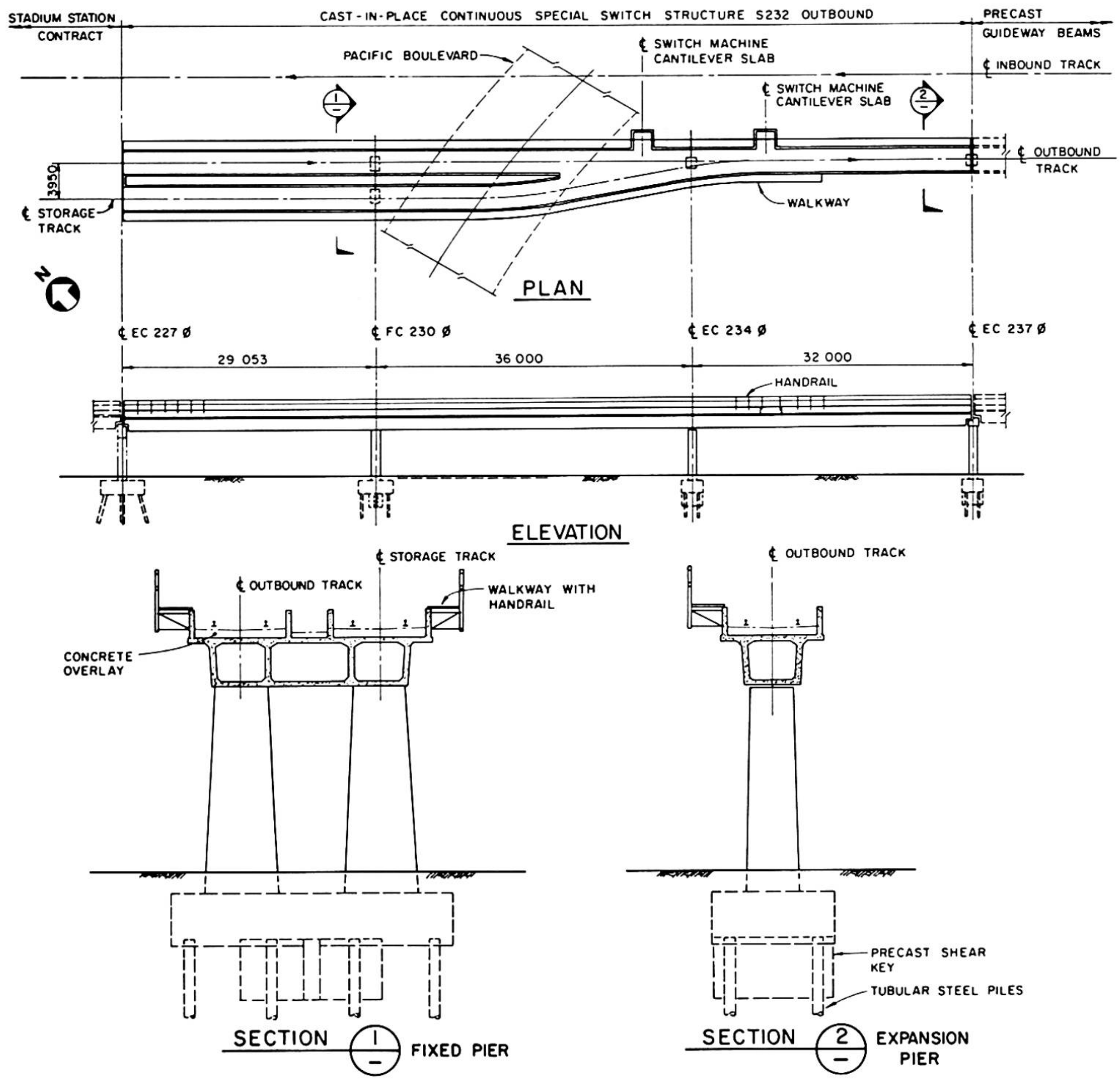


Fig. 1 Special Switch Structure S232 Outbound

## 5. SUPERSTRUCTURE DESIGN

The geometric configuration of the cast-in-place concrete box girder superstructure was determined by the alignment of the primary and storage tracks, clearance provisions over Pacific Boulevard, and adherence to the critical system-wide geometry requirements. Small cantilever extensions of the girder top flange were required to support the switch machines positioned outside the girder parapet walls.

Design of the continuous post-tensioned box girder was dominated by dead load and temperature forces. The magnitude of the live load was considerably less than the latter two forces. In addition to the principal vertical loads, secondary horizontal design forces due to rail-structure interaction and switch forces were also considered in combination with the vertical loads. The level of post-tensioning was chosen to prevent cracking in the box girder top flange but permit cracking in the parapet walls. Vertical cracking of the parapet walls is permitted as their prime function is to support the power rails and to absorb the energy from a derailed vehicle.

Design and layout of the post-tensioning was complicated by the changing box girder cross-section from three cells at the Stadium Station end to one cell at the other end. The highest level of post-tensioning was required over the fixed pier with lesser post-tensioning required at mid-span of the centre and end span adjacent to the Stadium Station. To reduce friction losses and achieve the high level of post-tensioning over the fixed pier, deck openings were provided 6.75 m either side of the fixed pier centreline to terminate and jack the tendons originating from opposite ends of the girder and crossing over the fixed pier.

As stated previously, the principal purpose of the 180 mm concrete overlay on the box girder's top flange was to accommodate the trenches beneath the running rails for the switching equipment. Since the trenches were required across the full width of the deck, the overlay was considered non-structural and expansion joints were located at regular intervals in the overlay to discourage composite action and reduce the size of overlay tension cracks over the interior pier supports.

The presence of continous parapet walls and a deck profile that slopes toward the centre of each track provides a large enclosed catchment area to be drained. The vertical alignment of the structure is a 0.5% slope toward the Stadium Station. This longitudinal slope was considered sufficient to require only one set of drains adjacent to the fixed pier. One hundred millimetre diameter PVC drain pipes have been cast in the girder and these connect with 150 mm ductile iron drain pipes at the underside of the box girder.

## 6. SUBSTRUCTURE DESIGN

The bent that supports the special guideway superstructure at the Stadium Station interface is not discussed as it was constructed in a separate contract. The remaining supports comprise two expansion columns and a pair of fixed columns as noted previously. Twin fixed columns are required to resist the severe longitudinal earthquake forces. In calculating column bending moments due to longitudinal forces, account was taken of the significant flexibility in the piled footings. Double uni-directional bearings were provided at the expansion column EC 237 ∅ to provide girder torsional fixity at the expansion joint. This measure was necessary as the excessive box girder torsional rotation at the east deck expansion joint would have developed large secondary stresses in the continuous running rail. The east end of the box girder is solid for the last 5 metres to prevent bearing uplift.

All columns are supported on rectangular, reinforced concrete footings of 1.5 m thickness. In addition to meeting the piling requirements, the footing size was influenced by the need to restrict footing movement under the action of horizontal, in-service forces. These forces included emergency braking and broken rail forces. The footing movement was considered acceptably small for the in-service condition if the lateral soil pressure did not exceed the at-rest value. Where the footing size would have become unacceptably large to meet this requirement, precast reinforced concrete shear keys were introduced at the underside of the footing slab. This increased the effective footing area in contact with the soil. The shear keys extended 1.5 m below the underside of the footing. The shear keys were precast as the groundwater elevation fluctuated considerably and was anticipated to be close to the bottom of the footing. Footing reinforcement was increased by 10% beyond that required to resist the column forces to ensure that a ductile hinge would form in the column in a failure loading condition.

The piles supporting the footing are 360 mm diameter tubular steel piles of 12.7 mm wall thickness. The pile driving specification required the piles to be driven through the 3 m to 10 m silt layer and penetrate 3 metres into the hard till. This penetration into the till provided design pile capacities of 1700 KN in compression and 180 KN in tension. Although vertical column loads were substantial, due primarily to the high dead load, horizontal design forces were significant in the design of the piling layout. The initial approach adopted was to resist all horizontal forces, including the large earthquake forces, by the use of battered piles. However, the earthquake design shears and moments required an excessive number of battered piles, the pile strength limitation being the low tensile capacity rather than the high compressive strength. In conjunction with the use of battered piles, there was also concern that the thick silt layer might experience a substantial seismic movement in the order of 75 mm. Such silt movements would generate large soil passive pressures upon encountering the relatively rigid, battered-piled footing, raising the possibility of failure in the battered piles. To resolve these two serious problems in a battered pile approach, all piles were made vertical and in-service horizontal forces were resisted by at-rest soil pressure. Under seismic conditions, the vertical piles are sufficiently flexible to move with the silt should large soil movements occur. The vertical pile arrangement resisting all vertical forces and column moments was considerably less expensive than that using battered piles. The top of pile to footing connection is designed as a hinge to further increase lateral pile flexibility under seismic conditions.

## 7. AWARD OF CONTRACT

The contract for the construction of Special Structure S232 Outbound was awarded to Farmer Construction Ltd. of British Columbia in September 1983. The contract included work on an additional segment of the elevated guideway. The bid price for the special guideway structure was estimated at $350,000. Notice to proceed was given on 4 October 1983 and the contract completion date was set at 1 July 1984.

## 8. CONSTRUCTION

### 8.1 Foundations

From the geotechnical report the soil conditions were determined generally to be 2 m of fill overlying a variable layer of organic material, a thin layer of sand, and sound glacial till. From previous work in the area the fill was known to contain large timber and concrete obstructions and the till was known to contain large boulders of up to 1 $m^3$ in size. The sequence of construction for the piled foundations was excavation for the pile cap, installation of the precast shear key and backfilling, driving of piling, construction of pile cap, backfilling to top of pile cap, and lastly casting of the column. Observations from standpipes installed along the adjacent section indicated that the water table fluctuated between 2 m and 5 m below existing ground level. The installation of the 8.0 m (max) by 1.5 m deep by 0.5 m wide precast reinforced concrete shear keys, although below the water table, was achieved successfully by pumping, placing the shear keys and backfilling with granular backfill to the required 85% modified Proctor density.

## 8.2 Tendon Arrangement

To achieve maximum benefit from the post-tensioning, the tendons were designed for maximum eccentricities at mid-span and over the supports. Over the fixed support, 12 tendons from each end of the girder weave between each other through the support diaphragm reinforcement. The contractor was concerned that the tendon arrangement over the fixed support might cause congestion and thus hinder concrete placement and compaction. In response to the contractor's concerns, the section designer made a scale model of the box girder diaphragm over the fixed support to illustrate tendon clearance. This allayed the contractor's concerns and the model was kept in the engineer's office at the construction site for reference during tendon placement. To ensure efficient placing of concrete around post-tensioning ducts and reinforcement in the relatively narrow webs, super-plasticised concrete was used.

## 8.3 Grouting of Ducts

To remedy grout shrinkage voids at tendon high points, three vent hoses were installed at each high point and the shrinkage void was regrouted approximately 24 hours later. The second and third vent hoses were used for the second stage precautionary grouting. Steel rods were inserted in both tubes to avoid plugging during the first stage grouting.

## 8.4 Short-Term Shortening of Box Girder

The short term shortening of the concrete box girder due to post-tensioning, shrinkage, and creep was calculated to be 15 mm at column EC 227 and 35 mm at column EC 237. This shortening together with allowances for thermal expansion or contraction was compensated for during construction to obtain the required expansion joint gap setting.

## 8.5 Falsework

The subgrade supporting the falsework is progressively weaker from west to east along the line of the structure due to the increasing depth of the fill. To avoid conflicts with buried services and random obstructions in the fill the contractor elected to support the falsework on mudsills placed on compacted gravel and to monitor the structure for possible movements due to differential subgrade settlement. A jacking system was incorporated into the design of the falsework towers to permit vertical correction of the forms should settlement be experienced during the pouring of the box beam soffit slab. Upward adjustment, if required, was made prior to pouring the beam webs.

## 8.6 Formwork

The contractor elected to pour the box girder sections in four pours; bottom slab, webs, top slab and vertical parapets. The formwork was constructed of form ply and timber. As there was no designed access to the interior of the box sections the contractor was permitted to leave the forms for the top slab in the completed structure.

Conventional form construction was employed using cone ties, studs and walers. Curved surfaces were permitted to be constructed from straight segments with a maximum 6 mm offset from a true curve. To compensate for shortening of the structure under tensioning, removable timber strips were employed except for the top slab forms where compressible material was used. The tendons were tensioned in a predetermined sequence to minimize unbalanced effects. Falsework jacks were adjusted after stressing 50% of the tendons to compensate for movements under stressing and to avoid load concentrations developing on portions of the falsework.

### 8.7 Tolerances

To meet the stringent construction tolerances for the ALRT, the standard elevated guideway box beams were precast using an adjustable steel form under close shop supervision. For this cast-in-place box girder, a high level of survey accuracy was therefore required to maintain the same specified allowable variation in cross-section dimensions of -3 mm to +6 mm. The level of accuracy relative to the working survey control points was required to be:

- Any point on a tangent section to be established within ± 2 mm horizontally at 80% confidence level.
- On curved sections where control was from closest adjoining tangent, any point to be established within ± 5 mm horizontally at 80% confidence level.
- Elevation of any point to be within ± 2 mm of its theoretical elevation.

## 9. ACKNOWLEDGEMENTS

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