Jointless Abutment Details for Two Structures with Abnormal End-Regions

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ABSTRACT

Use of jointless abutment structures continues to be seen as a beneficial method for the construction of roadway bridges and grade separations. Along with this current popularity also comes an increased challenge to accommodate the basic operational philosophy and typical details to wider numbers of geometric scenarios that are imposed by the placement of the roadway, local topography, or other special considerations.

This paper will look at the circumstances of two individual grade separation structures that were chosen specifically to be jointless style because of the recognized potential for life-cycle cost reduction. However, in both cases, the imposed roadway geometry required departure from the typical set of jointless structure details and consideration of alternate means of accommodating structure movement.

Both structures were designed with precast prestressed concrete girders and precast concrete MSE walls to achieve the desired results. One of the structures was also longitudinally post-tensioned with semi-integral abutments on cast in place drilled caissons, while the other was fully integral on driven steel pipe piles. One structure has been in service for over three years while the second is expected to start construction in the near future.

Keywords: Jointless Abutment, Precast Girder, MSE Wall, Semi-integral, Fully-integral.

INTRODUCTION

Conventional bridge expansion joints are considered by many transportation authorities and managers to be key agents of deterioration in a structure and, as a result, may experience significant life-cycle cost[1]. The total life-cycle cost is a combination of the range of movement expected, the severity of the local environment, volume of traffic and expected level of maintenance applied. Exposed expansion joints, regardless of their configuration or style, are seen to only have a limited service life given the physical and chemical attacks that they face. The potential damage to the structure, caused by a failed or ineffective joint, is also not limited to the joint itself and could also include damage to bearings, girders, and substructure as well.

As an alternative to exposed expansion joints, so-called “jointless” structures that still allow the two opposing ends of a structure to move but using a less obvious means of operation have gradually become more popular in recent decades[2]. Essentially the movement joint is relocated from the superstructure-to-substructure interface to the end of the approach slab where the structure transitions onto the roadway approach fills. At this location, should the joint fail, other structural elements would not be at risk and repairs become less costly. This type of joint is also less expensive to construct initially and is often detailed as being little more than a flexible material that permits the range of movement expected between the structure and the roadway. Jointless structures are generally constructed according to one of two basic styles: full- or semi-integral abutment in reference to how the superstructure is connected to the substructure.

A fully integral abutment structure requires that both abutments be monolithically connected to the superstructure so as to form a rigid frame consisting of two flexible legs, e.g. steel piles, connected by the rigid superstructure as seen in Figure 1a. To ensure maximum flexibility, steel piles are driven along a single vertically positioned row. Battered piles and offset pile rows would both contribute to an undesirable increase in longitudinal stiffness in the system and thus are not favored[3]. Then, assuming that the piles develop fixity at some specified depth below the pile cap (abutment), the structure becomes free to translate laterally at the top without risk to its global stability. Fully-integral abutment piles are also often constructed with partially- or fully-voided “sleeves” at the upper portion of the pile to help ensure unrestricted pile movement. So, as the superstructure moves according to thermal (or other) demands, the substructure moves in concert. Steel piles offer the advantage of accommodating large flexural strains and high axial capacity. Concrete piles are generally too stiff and lack ductility to be considered effective for fully-integral style abutments.

Semi-integral abutment styles on the other hand, provide opportunity for use of either steel or concrete piles or spread footings because the abutment is divided into two operationally distinct sections. The upper half remains monolithically connected to the superstructure and approach slab, while the lower half remains static as seen in Figure 1b. The two abutment halves are then connected by conventional bridge bearings that can accommodate the range of movements and rotations required. There are also two potential variants of the semi-integral style by considering how the wingwalls will be connected to the structure. In the fully-integral case, wingwalls are made to be monolithic with the structure mass; but in the semi-integral instance, they could be connected to either the upper half or lower half. The nature of the wingwall connection is generally arrived at based on previous experience, departmental policy, or if nothing else, the individual designer's preference.

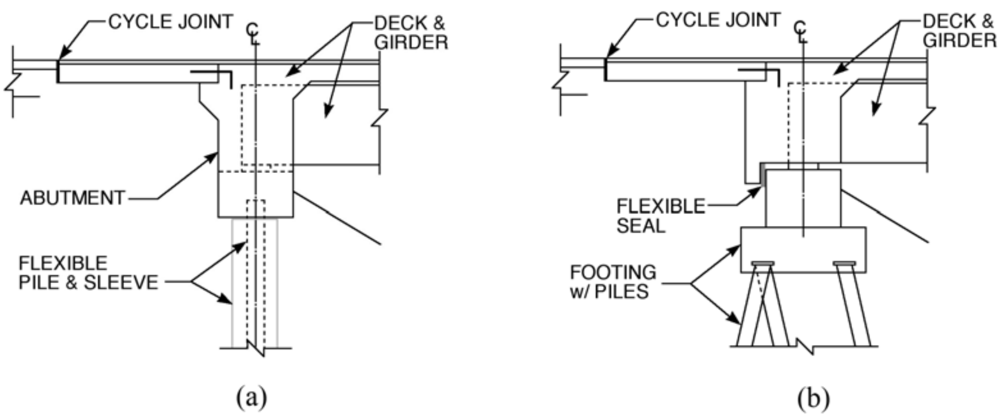

Figure 1. Typical jointless configurations for (a) fully and (b) semi-integral abutments.

While there are no rigorous guidelines to preparing jointless-style bridge structures, there are some basic rules and limits that have been proposed. In Canada, the Ministry of Transportation Ontario (MTO) has published a guiding document SO-96-01[4] that may aid the designer in ascertaining the appropriateness of using integral abutment structures. Additionally, these guidelines make reference to preferred details and arrangements to efficiently utilize the structure including a strong recommendation to not have any splayed wingwall arrangements. By observing this condition, the structure ends (abutments) do not engage the approach fills through lateral bearing in the soils. The ideal arrangement then for any integral structure is to have wingwalls aligned with the axis of the superstructure. This requirement is similarly extended to the approach slabs. *Alberta Transportation* (AT) has prepared, as part of its design manual, guidelines for the design of integral abutment joints as a summary of local experience and other provincial or state findings and recommendations[5].

In most situations the provisions of these documents coupled with practical experience are sufficient to allow for construction of a large number of integral and semi-integral abutment structures. The remainder of this paper will explore two case studies where typical details were modified so as to allow for the design of one integral and one semi-integral structure where otherwise each may have been designed with conventionally jointed abutments owing to extreme angles required in the wingwalls and approach slabs.

PROJECT LOCATIONS

Although the two projects discussed in this paper are from geographically diverse parts of Canada, they do have several physical features in common. One project is located on the Atlantic coast in the city of Dartmouth, Nova Scotia where the influence of high natural moisture along with airborne and applied roadway salts are considerations that lead the *Nova Scotia Department of Transportation and Public Works* (now renamed to Transportation and Infrastructure Renewal) to use jointless construction where possible for new structures. In contrast, the second project is located in the city of Calgary at approximately 1100 m (3,600’) geodetic elevation in the rain-sparse prairies east of the Canadian Rockies. The roads in this city are sanded and salted during the winter months, but not to the same degree that central and eastern Canadian cities are. However, the life-cycle cost benefits of jointless construction are not lost on the *City of Calgary Roads - Transportation Infrastructure* group which also endorses jointless construction where opportunity exists.

The subsurface conditions are also generally different between these two cities. Dartmouth soils typically consist of clays and tills intermixed with cobbles and boulders overlying hard rock at shallow depths. The area is also noted as having numerous lakes and groundwater is generally present. Typical pile installations are driven steel H-pile or pipe pile that usually do not have to be advanced any greater than 10 m (33’) to achieve refusal - either in the compact till layer or at the rock surface. In Calgary, the soil stratigraphy generally consists of clays and lesser amounts of sands and tills. Rock, if present, is usually weak and fractured and unable to support significant load. Generally speaking, drilled concrete piles with belled ends are common practice for transportation structures with typical pile diameters ranging from 760 mm (30”) to 1830 mm (72”). The depth required to locate suitable bearing capacity values may be between 10 and 20 m (33’ to 66’).

The seismic characteristics of central Nova Scotia are modest with zonal acceleration and velocity values (A and V) that permit classification of most sites to be Seismic Performance Zone (SPZ) 1 or 2 as per the definitions and data provided by the *Canadian Highway Bridge Design Code* CAN/CSA S6[6,7], or CHBDC. It is this document which guides highway structure design in Canada and provides data to describe all areas of the country as being in one of four zones of potential seismic activity - ranging from 1 (negligible) to 4 (severe). The prairie regions almost always have the lowest SPZ values and this was the case for the project site in Calgary. In addition to seismic performance, the CHBDC also defines a given structure as being one of three types to administer the type and level of survivability that a structure ought to have: *Lifeline*, *Emergency Route*, or *Other* in decreasing order of survivability. In both project cases, the structures were designated as *Emergency Route*, meaning that they need be able to permit passage of emergency vehicles, underneath or on top, after the design event.

A summary of the basic climatic data used for the preparation of design at these locations is summarized in Table 1 based on information presented in the CHBDC.

Table 1. Summary of project climatic data.

Project Location	Zonal Accel’n, A	Zonal Velocity, V	Peak Wind 1:25 yr (kPa)	Min. Daily Temp. (C)	Max. Daily Temp. (C)	Mean Annual R.H. (%)
Dartmouth	0.05	0.05	0.505	-22	+26	80
Calgary	0.00	0.05	0.455	-38	+26	55

PROJECT TIMELINES

The Wright Avenue / Highway 118 project in Dartmouth began with preliminary engineering approximately December 2004 followed by detailed design and project tender in the fall of 2005. Work progressed on the site through the following winter and spring with completion by May 2006. At this time, the current edition of the CHBDC was the 2000 edition (CAN/CSA-S6-00[6]) and the project followed NS-TPW Standard Specifications (2004).

The 64 Avenue North / Metis Trail project in Calgary began approximately in January 2008 with integrated preliminary engineering. Detailed design continued through to the end of the year with a tender expected early in 2009 and a specified completion date of September 2009. However, considering the current economic downturn, budgetary commitments have been constantly revised during recent months and tender for this project may not occur until the summer of this year, or even later, with completion one year afterward. Regardless, the design standard used for this project is the 2006 edition of the CHBDC (CAN/CSA-S6-06[7]) with additional reference to the City of Calgary Design Guidelines for Bridges and Structures.

DARTMOUTH: WRIGHT AVENUE / HIGHWAY 118

Provincial arterial Highway 118 was constructed in the mid-seventies as a four lane divided highway to the east side of Halifax Harbour to connect Highways 102 and 111 to the north and south respectively. Although the highway originally travelled through sparsely populated lands, the current expansion of the city along this corridor prompted consideration for a new interchange to improve local traffic congestion and provide alternate points of access (including municipal road extensions) to new and expanding commercial and industrial areas to the west. Wooded park space existed immediately beyond the east side of the proposed interchange and one of the directives of the project was to ensure that no new construction extended into this area. Geographically, the site is constrained by elevated terrain and rock outcroppings to the west and wooded parkland and lakes to the east. In order to achieve the smallest footprint possible for the dual lane on- and off-ramps at the east side of the interchange, mechanically stabilized earth (MSE) with precast panels were employed. Originally, MSE wall was envisioned along both edges of the ramps, however natural slopes were preferred by the park-going public in order to make a more natural visual and physical transition into the wooded park space and pathways. The location of these ramps to the edge of the roadway meant that little working room was left to introduce an abutment.

In contrast to the east side of the interchange, the west side had few obstructions and allowed for the placement of large radius on- and off-ramps which did not otherwise influence the structure geometry. In addition to Wright Avenue, Commodore Drive was also selected to be extended to the Highway 118 corridor as a simple right-in/right-out arrangement approximately 700 m (2,300’) to the south. Given that there would now be three exits located on the southbound lanes in relatively short distance (including the exit to westbound

Highway 111), a collector-distributor (CD) lane was deployed alongside the southbound mainlanes to better direct traffic. This area of the city was also expected to continue experiencing growth and municipal planners required that provision for future lane additions be made for both northbound and southbound Highway 118 traffic.

SUPERSTRUCTURE GEOMETRY

In order to bridge all seven lanes, including the future widening, of Highway 118, two prospective zero-skew span arrangements were studied: a 56.0 m (184') two-span using prestressed precast concrete girders and a 52.2 m (171') single span using welded steel plate girders. Each option had advantages over the other. The steel structure would be faster to build while the concrete girder had a lower construction cost. Although the construction schedule was to be very aggressive, the two-span option (as shown in Figure 2) could also be built within the required timeframe and thus it was selected as the preferred option.

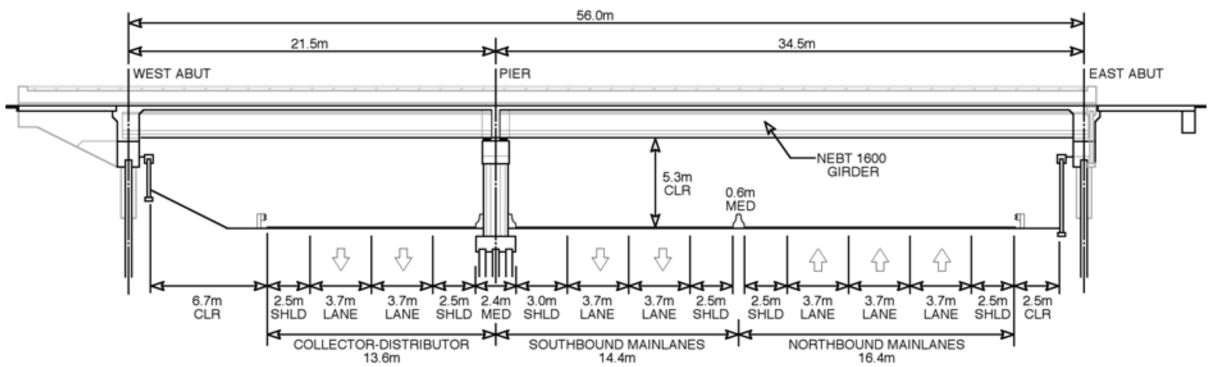

Figure 2. Wright Avenue / Highway 118 Underpass two-span precast concrete girder (looking north) with ultimate lane configuration shown.

The most practical location for the center pier was the exterior edge of the southbound mainlanes with the east abutment then placed as close as possible to the to the northbound lanes yielding a minimum permissible span of 34.5 m (113') to the pier. All clearances, medians, guiderail location, etc. were as specified by the *Transportation Association of Canada* (TAC) Manual[8]. The adjacent span would then cross the southbound CD lanes and could have been as little as 16.0 m (52') if MSE wall were also used to eliminate the foreslope at the west abutment. Alternatively, if no MSE wall were used at the west abutment, the span would have been 25 m (82') with 8 m (26') wingwalls. It was decided to use a secondary span that was in between these limits using a combination of foreslope and MSE wall. This allowed the secondary span to be 5/8 of the longer primary span, to better balance the potential for live load uplift, and create a smaller 5.5 m (18') wingwall, which follows the MTO guideline suggesting smaller end walls are more practical for integral abutment structures. The secondary span thus became 21.5 m (71') making the total span 56.0 m (184').

The live load standard was the CL-625 (Ontario configuration) as described in the CHBDC including lane load. This load is essentially a 5-axle 625 kN (67.5 ton) tractor-trailer 18 m (60’) in length in isolation or combined at 80 percent plus a 9 kN/m (0.62 klf) lane load to produce a worse live load effect.

The superstructure cross section was determined by municipal standards including four 3.5 m (11.5’) lanes, shoulders, medians, and a pedestrian sidewalk to be integrated with the city pathways network. The dual-lane turning movements on and off the structure over the east abutment were found to be critical to the overall structure geometry. The WB20 tractor-trailer was selected as the design movement (three axle groups having a total length of 20 m or 66’) in the outside lane in combination with a city bus in the inside lane. By providing a 1.0 m (39”) center median (painted) and outside shoulders also 1.0 m wide, the anticipated turning movements could be accommodated. Cast-in-place traffic barriers were designated PL2 and included open steel rail along the pedestrian way. The total width of the superstructure is 21.11 m (69’-3”) as seen in Figure 3.

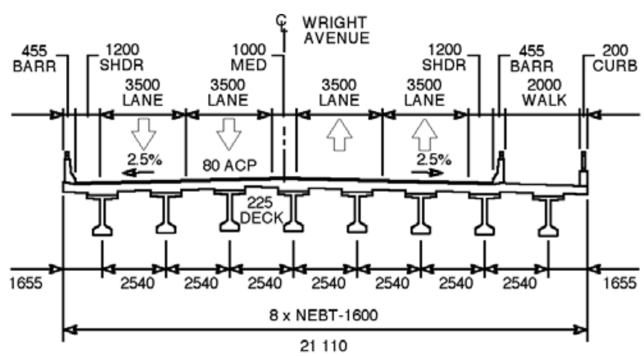

Figure 3. Wright Avenue deck cross-section (looking east) showing 8 New England bulb-tee (NEBT) girders supporting a 225 mm deck and 80 mm asphalt wearing surface.

Although the CHBDC also suggests 175 mm (7”) as a minimum deck thickness, it is generally easier to accommodate a 225 mm (9”) thick deck given other bar spacing and cover requirements. The cast-in-place deck section was supported on eight precast prestressed New England Bulb Tee (NEBT) sections 1.6 m (63”) in depth using 36- and 54-13 mm (0.5”) low relaxation strands (f_{pu} =1862 MPa, 270 ksi) for the short and long span girders respectively. Girder spacing was 2.54 m (8’-4”) with 1.665 m (66”) cantilevers. Concrete strength at release was specified as 40 MPa (5,800 psi) and 55 MPa (8,000 psi) at 28 days - both of which were easily achieved by the precaster *StressCon Ltd*. The NEBT was specifically designed to be used in the harsh climate of the New England and Northeast states and has become a popular and preferred precast bridge girder shape in eastern Canadian provinces as well. The girder spans are made continuous for live and additional dead load(s) by projecting a series of 20M (No. 6) hairpin rebar that overlap from each girder end at the central pier diaphragm.

Having the intermediate pier offset from the center of the superstructure, meant that there would be some differential rotation and translation at this support. The design provided for steel laminated elastomeric bearings which were restrained against longitudinal and lateral movement by use of steel pintles (2 per bearing) grouted into the pier cap beam, effectively providing a point of fixity. Due to the rotational and translational flexibility of the abutment pile system, the ends were considered as near-perfect roller (guided free) supports.

ABUTMENT GEOMETRY

The extent of the critical turning movements were plotted in plan view and indicated that the east abutment wingwalls would be required to be splayed from their normal longitudinally directed position by as much as 60^{o} as seen in Figure 4. However, the west end of the structure remained with normally orientated wingwalls and approach slab of 6.6 m (22') length. The length of the approach slab was then correspondingly matched on the east side, but the outside corners were truncated and aligned so as to be directed orthogonally to the traffic flow at each side of the roadway intersection.

It was realized at this point in the design process that a jointless structure may have to be abandoned for one simple reason: the lack of an appropriate detail to permit large splay angles at the end region of the structure. However, the solution to the problem appeared simply to disconnect the wingwall and approach slab from the structure-proper and introduce modified joint details suited to their relocated position. The only other alternative was to use conventional joint details or possibly a hybrid joint system (e.g. integral at the west and conventional at the east), but either of these alternatives would mean adopting some additional long-term maintenance liability for the structure.

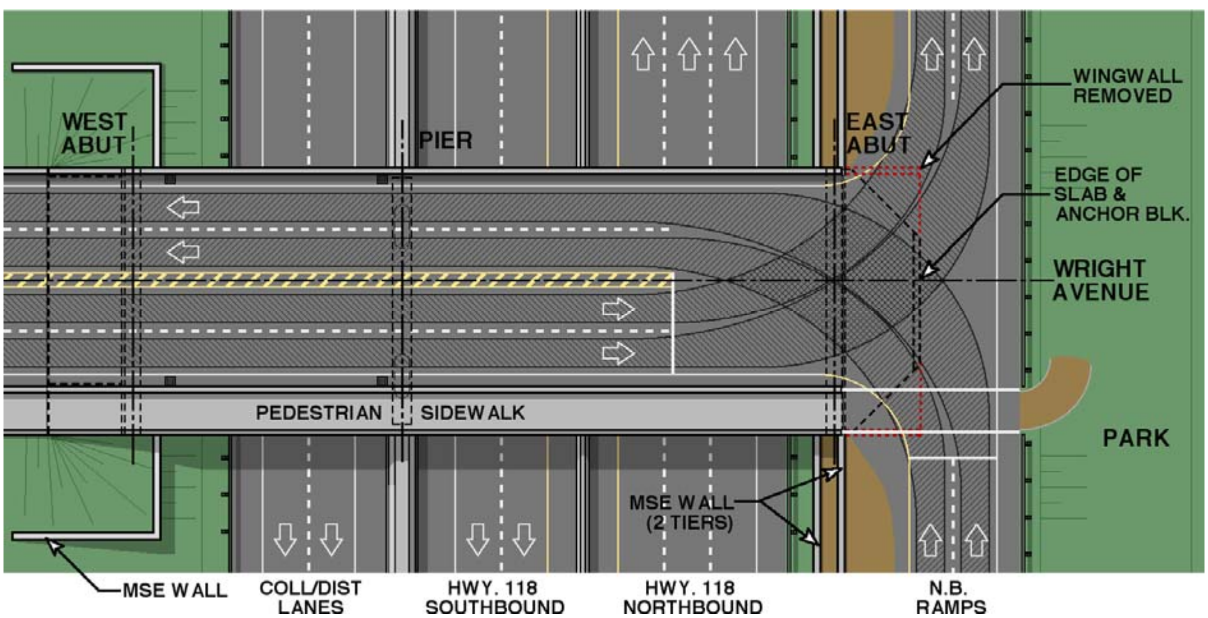

Figure 4. Wright Avenue structure configuration with respect to Highway 118 and associated northbound (NB) ramps.

MODIFIED CYCLE JOINT DETAIL

The cycle joint was repositioned to be at the front edge of the approach slab - i.e. where it connects to the back of the abutment ballast wall. By locating the joint here, the approach would become a stationary element with respect to any longitudinal movements in the structure. Details related to the joint however were generally unchanged as shown in Figure 5 which compares the unchanged joint at the west end and the modified joint at the east end.

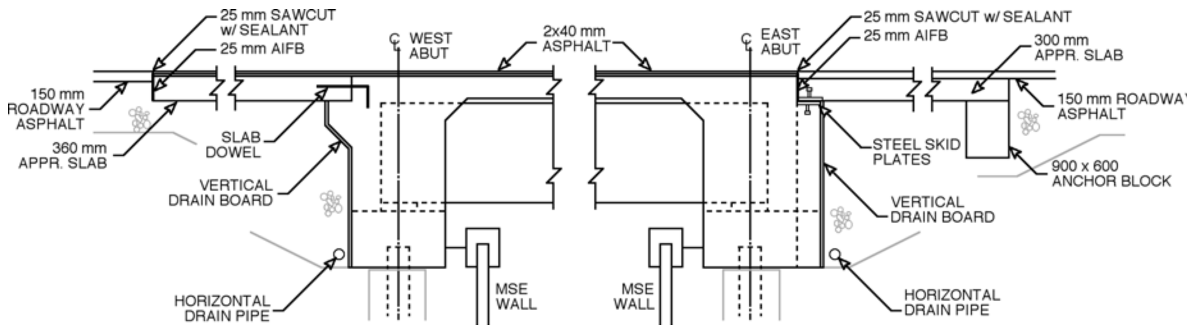

Figure 5. Wright Avenue west and east abutment details including cycle joints.

The cycle joint is essentially little more than 25 mm (1”) expanded asphalt impregnated fibre board (AIFB) capped with a flexible polyurethane sealant. The roadway pavement structure consisted of 100 mm (4”) of provincial standard Mix-B-HF followed by 50 mm (2”) of Mix-C-HF. The bridge deck wearing surface consisted of 10 mm waterproofing membrane followed by 30 mm (2-1/4”) and 50 mm (2”) of Mix-C-HF. To permit cyclic movement within the wearing surface, the asphalt was saw-cut over the joint and filled with a rubberized joint sealing compound.

To help ensure that the approach slab does not move as a result of the continuous action of superstructure expansion and contraction, an anchor block is located at the back of the approach slab (where it abuts the roadway). The block was detailed to be backfilled using compacted free-draining granular material - an important consideration given the cold winter temperatures that will occur. The depth of the block is sufficient to be below the expected level of frost and the use of free-draining (low fines) granular material reduces the potential for moisture to accumulate and freeze in the subsurface.

Another consideration at the joint implemented two opposing steel “skid” plates, anchored into the approach slab and the approach slab seat respectively, to reduce the level of sliding friction between these two surfaces. Water intrusion into the interstitial gap left between abutment and the approach slab could not be ruled out but it was reasoned that, because the approach slab seat also followed the 2.5% roadway crown and the gap was open at each end, no significant water accumulation would occur. Similarly, corrosion of the steel skid plates was not considered to be a deterioration concern. The final consideration in the modified joint detail was to remove the anchored steel rebar normally used to join the approach slab to its abutment. Obviously inclusion of these bars, into the east approach slab, would unduly restrict the movements otherwise required.

MODIFIED ABUTMENT DETAILS

Both abutments were founded on fourteen 273 mm x 13 mm dia (10.75” x 1/2”) open ended steel pipe piles driven in a single row to refusal for use in an integral abutment configuration. These piles were then sleeved using 0.8 m dia x 3.0 m (32” x 10’) corregated steel pipe and infilled with loose sand followed by casting of the abutment diaphragm to the bridge seat. Because MSE wall was anticipated to be required along the northbound rampways, which in turn were to be close to the east end of the structure, the walls were easily incorporated into the design of the east abutment. Further, it was possible to divide the wall into two sections near the abutment: the lower portion of the wall would run in front of the abutment while a separate upper portion would intersect the abutment face at the centerline of bearings, as seen in Figure 6. Doing this would allow the removal of wingwalls from the structure altogether - in essence they were to be replaced by the MSE walls. With the splayed wingwalls removed and the introduction of non-participatory MSE walls, the structure would become free to translate and rotate as required without hinderance from adjoining elements.

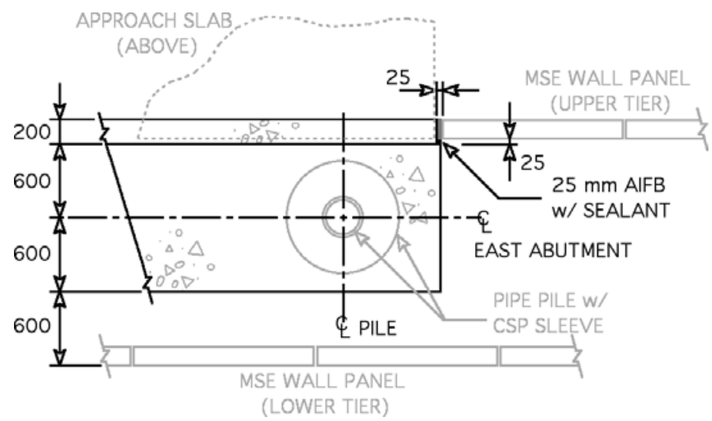

Figure 6. East abutment to MSE wall joint detail, plan view.

However, differential movement and rotation would still need to be accommodated at the interface between the wall panels and the abutment’s lateral faces. Each side of the abutment was detailed with a small (25 mm or 1”) recessed face in which to create a clean termination line for the wall panels. The recess then allowed for movement of the abutment to occur that would not affect the wall system. The intermediate gaps were infilled with a compressible expanded polystyrene product and dressed with a flexible polyurethane joint sealant. Little change was otherwise required in the abutment details as compared to traditional projects.

CALGARY: 64 AVENUE / METIS TRAIL

Ongoing extensions of the City of Calgary’s light rail transit (LRT) system often leads to requirements for additional roadway infrastructure alongside it as well. That was the case when the city planned to extend the northeast line further from the current terminus at

McKnight-Westwinds station (opened in late 2008) and add two new stations at Martindale and Saddle Towne. To accomplish the extension, the LRT line would need to cross 64 Avenue North - a municipal arterial - near its at-grade signalized intersection with another arterial - Metis Trail. Several arrangements were reviewed including maintaining the at-grade intersection and pushing the LRT profile down to be under 64 Avenue, but traffic studies pointed to the necessity of a grade separated interchange between the two major roadways by 2015. Numerous interchange arrangements were then studied with one being the clear choice for a preferred alternative - a “tight diamond” configuration where the LRT would remain at grade and 64 Avenue would rise over both tracks and Metis Trail.

Final arrangement for the project then promoted a slight realignment of 64 Avenue in addition to requiring two new structures to span over Metis Trail and the LRT right-of-way - referred to as Structure 1 and 2 respectively. Construction of a single multi-purpose structure was not found to be justified in terms of either the basic cost or the added complexity of adding ramps at mid-length. As with the project location in Dartmouth, the on- and off-ramps that would be required alongside the northbound Metis Trail mainlanes had little latitude for variation in alignment given the close proximity of the LRT routing immediately to the east. This in turn resulted in turning movements onto and away from the structure that would be overlaid on the east end of the structure and approach slab and require large splay angles in the wingwall arrangement. However, the desire to use jointless structure details was also expressed by city engineers and a similar opportunity to use modified joint details was presented as a solution to this situation as well.

SUPERSTRUCTURE GEOMETRY

Metis Trail is currently a four lane divided urban throughway but was planned to be improved to six 3.7 m (12’) lanes as a result of this project work. By including the median width, shoulders, clear recovery zones, and fill slopes on each side of the roadway, the total span required was optimized to 61.4 m (202’). This underpass could then be made to be either a single span or a two span structure given that 65 m (213’) precast concrete girders had been successfully used previously in the city[9]. The most notable drawback to using such a lengthy single span arrangement would be the deeper girder required - at least 2.0 m if not 2.4 m (79” and 95” respectively) - which would also unfavorably increase the height of fill required not only at the two structures but along all the adjoining rampways as well. Two spans of 30.7 m (101’) became the preferred alternative and were modeled using both welded steel plate girder and precast concrete girders. The precast option, shown in Figure 7, was only slightly more expensive than the steel girder option based on initial capital cost, but was found to be much less costly after total lifetime maintenance costs were assessed via life cycle cost analyses for the two options.

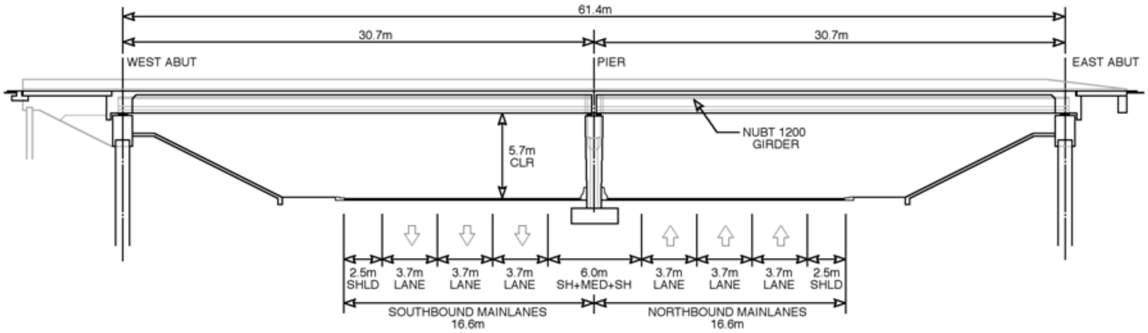

Figure 7. 64 Avenue / Metis Trail Underpass two-span precast concrete girder (looking north).

In Alberta, owing to the anticipation of heavy oil and gas industry truck loads, the design live load is often increased beyond the minimum condition described in the CHBDC, specifically to that of a CL-800 (90 ton) truck model having the same axle spacing of the CL-625. The CL-800 truck has also been adopted for use in the City of Calgary design model although the code-specified lane load of 9 kN/m (0.62 klf) does not change.

The 64 Avenue deck section was designed to have two through-lanes in each of the primary directions (east-west) plus two dual left-turn lanes available to eastbound traffic and a single left-turn lane for westbound traffic as shown in Figure 8.

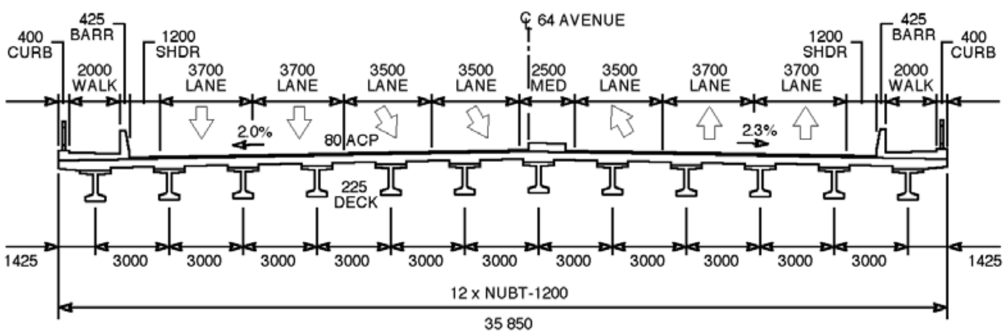

Figure 8. 64 Avenue deck cross-section (looking east) showing 12 Nebraska University bulb-tee (NUBT) girders supporting a 225 mm deck and 80 mm asphalt wearing surface.

The structure cross-section also includes a raised center median in addition to sidewalks, pedestrian railings, and PL2 concrete barriers on each side of the roadway for a total out-to-out deck width of 35.85 m (118’). The 225 mm (9”) cast-in-place deck will be placed on twelve lines of 1200 mm (47”) deep Nebraska University (NU) style bulb-tee precast concrete girders each prestressed with 54-16 mm (0.6”) low relaxation strands (f_{pu} = 1862 MPa, 270 ksi). The girder spacing used is 3.0 m (9’-10”) with 1.425 m (4’-8”) cantilevers. Specified concrete strengths for the girder are 49 MPa (7,100 psi) at release and 70 MPa (10,150 psi) at 28 days.

Continuity of the deck will be achieved by post-tensioning the total span following deck placement using a single line of 12 post-tensioning strands inside an 80 mm (3.15”) o.d. corrugated metal p.t. duct set to a parabolic profile and grouted following tensioning (note: the same strand type as used for prestressing). The narrow width of the pier section - so required to keep inside the permissible limits of the Metis Trail roadway median below - also means that the intermediate space between girder ends will be limited to 300 mm (12”) in which to develop a positive moment girder connection.

The superstructure support arrangement at the pier is intended to behave similar to what was used at Dartmouth, except in this case, the pier will be located at the center of the span and unbalanced forces are less of a concern. The pier provides a means of longitudinal and translational fixity by using anchored keeper plates against the steel laminated elastomeric bearings. Opposing keeper plates are welded or bolted into both a bottom sill plate, anchored into the pier cap beam, and a top shoe plate upon which the girder would rest (note - the embedded girder sole plate would then be field-welded to the bearing top plate). Each end of structure rests on a similar set of elastomeric bearing assemblies and treated as a free end with respect to translation and rotation.

ABUTMENT GEOMETRY

The features of the east abutment were similar to those at the Dartmouth project specifically the requirement to have vehicle turns occurring over the approach slab due to the proximity of the ramps associated with the interchange. In this case however, the rampways were constrained by the extending LRT trackworks instead of parkland as seen in Figure 9.

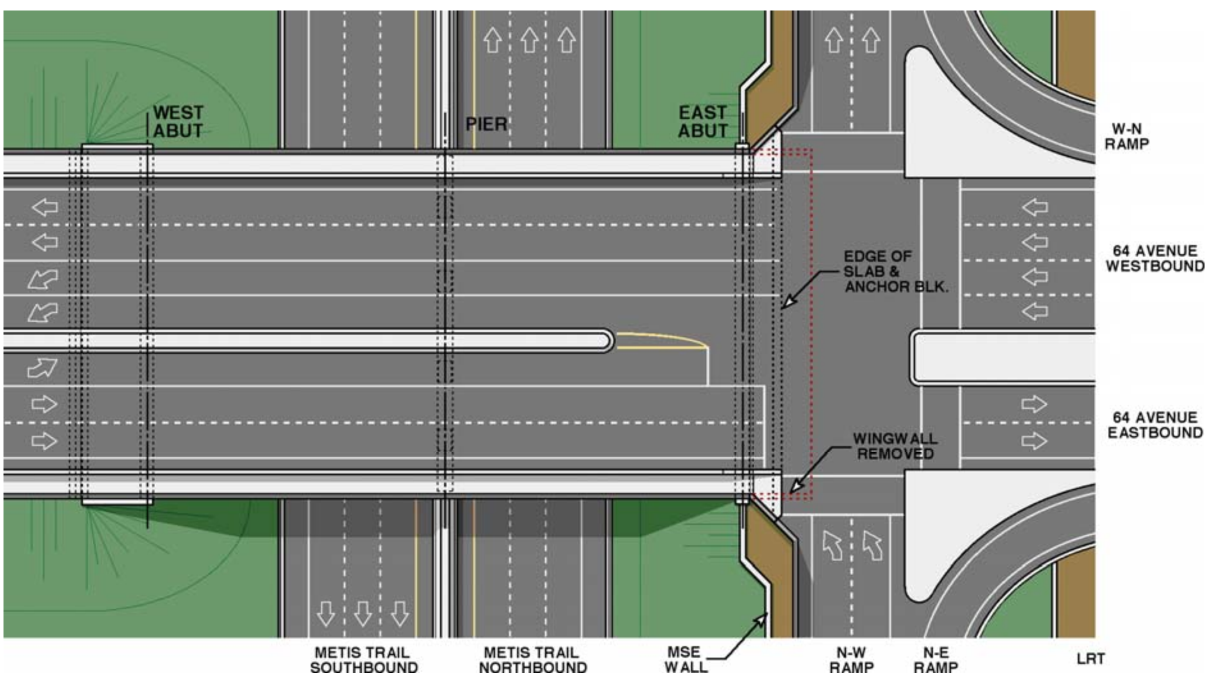

Figure 9. 64 Avenue structure configuration with respect to Metis Trail. Ramps are aligned between mainlanes and LRT right-of-way. A second structure crosses over the tracks.

Additionally, both sidewalks would also need to have the typical City transition to roadway detail to permit access for persons with disabilities, which among other things, requires that the sidewalk flare open from 2.0 m (6'-6") to a width of 3.9 m (13') while also tapering down to grade at the curb. This detail alone would require that the wingwalls and approach slab have a splayed shape in addition to the requirement from the turning traffic. However, the amount of splay was not as great as the Dartmouth project - only about 45^{o} as seen in Figure 9. One other consideration was to shorten the length of the approach slab at the east end to 4.0 m (13') as compared to the 6.75 m (22') length used at the west abutment. This was necessary in order to prevent the joint from straying into the region of the intersection (and experience traffic traveling transversely to the direction of the slab).

MODIFIED CYCLE JOINT DETAIL

The cycle joint designed for this project is similar to that used for the Dartmouth project. It is positioned to be a the front edge of the approach slab and it accepts differential movement between the abutment and the slab. The width of the joint however is wider, being increased to 40 mm (1½") to allow for the greater range of movement created by the slightly longer span and the slightly colder winters. Both the east and west end cycle joint details, shown in Figure 10, have been detailed to be filled with a flexible non-adsorbent closed-cell foam material (Ceremar) to permit the range of movement expected. The joint filler can also be used with hot- or cold-applied sealants. In this case, the joint was detailed to be topped with a flexible polyurethane sealant/adhesive. The roadway wearing surface would then be applied to the deck and approach slab in two lifts of 40 mm (1-1/2") B-mix asphalt that are continuous over the joint. To allow for the reflective movement in the asphalt from the joint below, the asphalt will be saw-cut and filled with a hot-mix asphalt sealer.

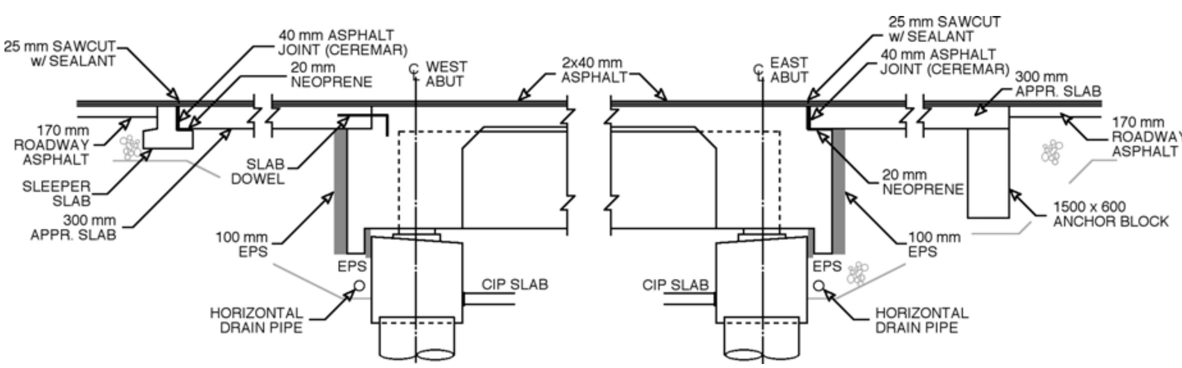

Figure 10. 64 Avenue west and east abutment details including cycle joints.

Anchorage of the approach slab is also required to ensure that it does not drift or ratchet out of place as a result of the longitudinal movements of the superstructure. As was previously used at the Dartmouth project, the approach slab will be anchored along the back edge using a deep c.i.p. concrete block. Instead of using skid plates, the front edge of the slab will be fitted with a simple movement bearing using a continuous strip of 20 mm (3/4") neoprene along the approach slab seat as per typical City detailing. It is also a preferred detail for the City to incorporate a "sleeper" slab at the ends of their approach slabs as a means to promote consistent settlement between the approach fills and the slab (i.e. to avoid unwanted bumps in the roadway surface). The anchor block serves this purpose as well.

The backwall and interstitial space were fitted with up to 100 mm (4") of expanded polystyrene (EPS) as a means of providing compressible material against which the structure can move without transferring deformation(s) into the adjoining soil.

MODIFIED ABUTMENT DETAILS

The new ramps along northbound Metis Trail (aligned with the east end of the structure) would also be required to have a modest height MSE wall (e.g. less than 1 m, or 3') which could then be integrated into the east abutment similar to what was used in Dartmouth. The ramps at the west end of the interchange, however, would be sufficiently far enough away from the structure that end-region splay is not a concern.

Because the east-side walls were generally short, only one level of wall was required and it would simply align with the center of bearings at the east abutment. Both ends of the structure would be constructed as semi-integral abutments founded on drilled cast-in-place concrete piles. Each abutment would have 12 lines of 1067 mm (42") dia. piles with belled ends to increase bearing capacity. Preference was also expressed by city engineers to have wingwalls connected to the bridge seat as opposed to hanging down from the abutment diaphragm and overlapping the bridge seat as shown in Figure 11.

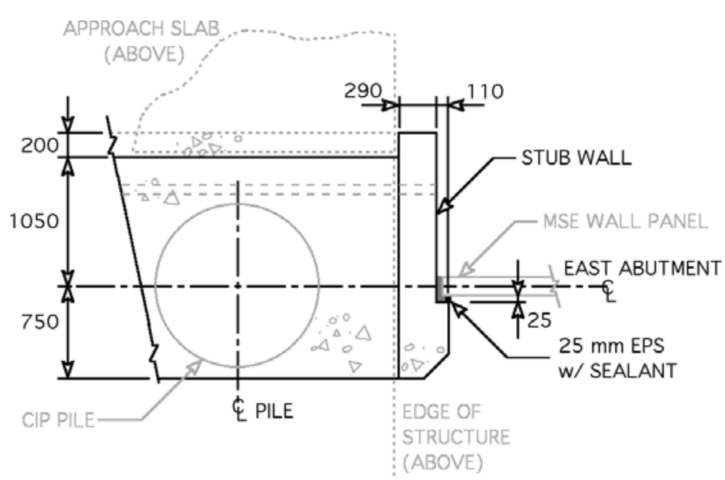

Figure 11. East abutment to MSE wall joint detail, plan view.

In the case of the east abutment however; there would be no actual wingwalls extending back into the fill, but rather truncated "stub" walls where the MSE panels would abut. Both of these elements would be stationary and require only a simple sealed joint and backing material.

Any differential movement and rotation would now occur between the abutment diaphragm, made monolithic with the superstructure, and the lower half connected to the stationary pile group. These two halves are to be separated by a row of steel-laminated elastomeric bearings under each girder. Similarly a 25 mm (1") vertical joint is to be established by filling the gap with expanded polystyrene between the two sections and capping with a flexible sealant.

CONCLUSION

As was previously noted, the concept of jointless bridge abutments centers on removing joints from locations where they only induce higher lifetime maintenance and relocating them to the end of the structure approaches in order to reduce operational costs. To do this effectively, structures are built to accommodate thermal and other longitudinal movements by acting as monolithic elements with emphasis placed on ensuring that the approach structure and abutments are not impeded in their movements. All differential movement between the structure and the adjacent roadway will now occur at the repositioned (cycle) joint ideally incorporating a jointed wearing surface over the approach slab. To further aid the structure in undertaking its requisite movements, the wingwalls are suggested to be both minimal in scale and directed along the principal axis of the structure. Adopting details that are otherwise could offer resisting forces to the longitudinal movements of the structure that only serve to defeat the original objectives of the design by imparting damage and increasing the maintenance commitment required.

In the two design cases presented in this paper, the over-ruling condition of the roadway geometry required that wingwalls be significantly splayed in order to accommodate turning movements at the structure ends. The Wright Avenue project in Dartmouth was constrained by parks and water while the 64 Avenue project in Calgary was constrained by the adjacent ramps and LRT right-of-way. These situations would have otherwise precluded the use of integral abutment details for these projects except that modified joint details were devised to recapture the original intent of a jointless structure.

Perhaps the simplest way to remove the difficulty associated with highly splayed wingwalls on integral abutment structures, is to disconnect them fully from the structure. In this manner the wingwalls would not impede the longitudinal expansion and contraction of the structure by developing bearing action in the soils surrounding the abutment. Incorporation of MSE walls using precast concrete panels is a simple and effective means of accomplishing this objective,center which coincidentally, were required as a result of the confining and restrictive site geometry as well.

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