APPENDIX I

Structural Assessment

MEMO

TO: Ms. Carolyn Ryall, P.Eng., Regional Municipality of Niagara (Region)
Jack Thompson, Region
Jordan Frost, P.Eng., Region

FROM: Brad Schmidt, P.Eng., Dillon Consulting Limited (Dillon)

cc: Paul McLeod, P.Eng., Dillon
Stephen Peck, P.Eng., Dillon
Zahra Jaffer, Dillon

DATE: November 23, 2018

SUBJECT: Casablanca Boulevard at QEW Structural Evaluation – Draft

OUR FILE: 18-7650

BACKGROUND

In April, 2018, Dillon was retained by the Region to prepare a Schedule "C" Environmental Assessment (EA) and Detailed Design, under the Municipal Class Environmental Assessment (EA) process (Class EA), for Casablanca Boulevard extending from North Service Road to Main Street in Grimsby, Ontario. **Figure** illustrates the study area.

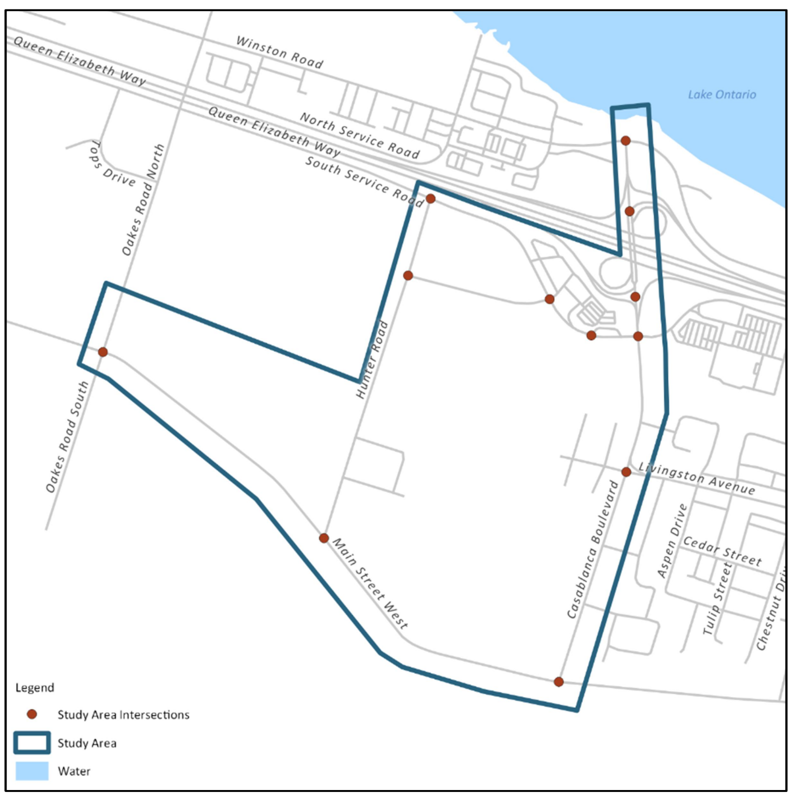

FIGURE 1: STUDY AREA

As part of this study, Dillon undertook a Detailed Transportation Assessment to identify the transportation infrastructure requirements to address the area's growth and implementation of a GO Station to be located in the southwest corner of South Service Road and Casablanca Boulevard. Various interchange options were analyzed as part of the assessment for the Casablanca at the QEW Interchange. It was determined that improvements to the interchange are required to meet the future growth.

One of the interchange options is a conversion to a Diverging Diamond Interchange (DDI) to improve capacity, safety, and active transportation. This memo summarizes the results of a structural evaluation of the Casablanca Boulevard at QEW Underpass, to determine if the existing structure has sufficient capacity to carry the additional loads resulting from the DDI configuration.

EXISTING STRUCTURE

The Q.E.W. Casablanca Boulevard Underpass, constructed in 1968, is a four span concrete slab on precast AASHO girder bridge. The structure carries four lanes of traffic over the QEW and is 104 m long with span lengths of 26.5 m, 25.3 m, 25.3 m and 26.5 m. The bridge deck is skewed 22.1 degrees. The existing deck cross section is shown below in **Figure 2**.

The superstructure is supported on three piers each consisting of a pier cap on six columns founded on individual spread footings. The pier caps are inverted tee bents supporting the dapped ends of the precast girders. The abutments are founded on spread footings.

Since the original bridge construction, the following rehabilitations have been completed:

- In approximately 1991 the work included: expansion joint replacement, raised median modification, west parapet wall modification, construction of a new west sidewalk, concrete patch repairs, waterproofing and asphalt paving of the deck.

- In 2014 the work included: abutment bearing seat reconstruction, new abutment bearings, new parapet walls, new semi-integral abutment retrofit including new approach slabs, waterproofing and asphalt paving of the deck, encasement of pier columns and slope paving repairs.

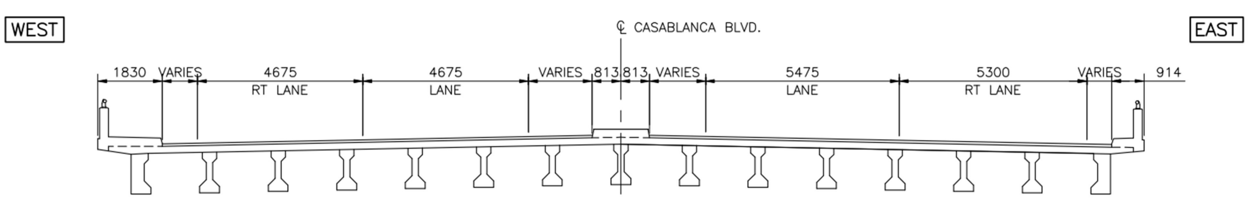

FIGURE 2: EXISTING DECK SECTION

DIVERGING DIAMOND INTERCHANGE PROPOSED OPTION

The following structural modifications were assumed for the conversion to a DDI:

- Removal of the existing raised median and construction of a 4.5 m wide raised multi-use pathway at the centre of the bridge
- New concrete parapet walls and railings at multi-use pathway (MUP)
- Removal of the existing west sidewalk and reconstruction to a maintenance curb configuration
- Modification of semi-integral abutments retrofits and approach slabs at west side.

See **Figure 3** below, showing the proposed bridge cross-section.

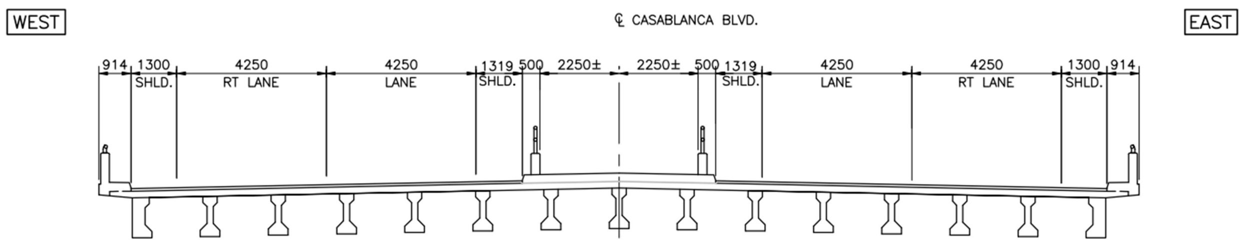

FIGURE 3: PROPOSED SECTION – DIVERGING DIAMOND INTERCHANGE

STRUCTURAL EVALUATION

A structural evaluation was completed in accordance with the Canadian Highway Bridge Design Code (CHBDC). The structure was evaluated based on the proposed cross-section dimensions and the original and rehabilitation drawings. Due to completion of the recent rehabilitation, an inspection of the bridge was not completed and the nominal as-designed section properties were used for the components evaluated.

EVALUATION METHOD

The bridge structure was evaluated at the Ultimate Limit State using four lanes of traffic and load factors corresponding to Sections 3 and 14 of the CHBDC. A grillage model was created using STAAD Pro v8i software to complete the structural analysis reflecting the proposed configuration.

The evaluation considered pedestrian only, traffic only and pedestrian and traffic combined. In accordance with Section 14 of the CHBDC, the maximum pedestrian load on the MUP does not need to be considered in conjunction with the maximum traffic loads for a bridge evaluation if there is a low likelihood that the design pedestrian loads will occur coincidentally with maximum traffic loading; however at the request of MTO this load combination was included for information considering the additional pedestrian capacity provided by the MUP.

Material Properties

The material properties used in the analysis were based on the information shown on the original drawings and are summarized in **Table 1**. The reinforcing steel grade is unknown. In accordance with Table 14.2 of the MTO exceptions to the CHBDC, a yield strength of 275 MPa was assumed.

TABLE 1: MATERIAL PROPERTIES

Item		Value
f_y	Reinforcing Steel	275 MPa
f_{pu}	Pre-stressing Strands (Low-Relax.)	1860 MPa (41,300 lbs)
E_s	Reinforcing Steel	200 000 MPa
f'_c	Remainder of Concrete	27.6 MPa (4000 psi)
f'_c	Centre Spans Concrete Girders	34.5 MPa (5000 psi)
f'_c	End Spans Concrete Girders	41.4 MPa (6000 psi)

SUPERSTRUCTURE EVALUATION RESULTS

Bending moment and shear capacities were determined in accordance with Section 8 of the CHBDC for the girder and concrete deck. The Ultimate Limit State (ULS) demand/capacity ratios at worst case locations on the structure, are presented in **Tables 2** and **3**, as factored loads divided by factored resistances. Therefore, values less than or equal to 1.0 indicate that the structure satisfies the minimum requirements of the CHBDC.

TABLE 2: ULS DEMAND/CAPACITY RATIO – BENDING MOMENT

Span	Location	Section 3			Section 14		
		Ped	Traffic	Combined	Ped	Traffic	Combined
		+ve/-ve	+ve/-ve	+ve/-ve	+ve/-ve	+ve/-ve	+ve/-ve
End Spans	X/L = 0.0 (Abutment)	---	---	---	---	---	---
	X/L = 0.5 (Mid-Span)	0.49	0.55	0.68	0.44	0.50	0.60
	X/L = 1.0 (Exterior Pier)	0.75	0.99	**1.27**	0.65	0.85	**1.09**
Centre Spans	X/L = 0.0 (Exterior Pier)	0.75	1.00	**1.28**	0.65	0.86	**1.09**
	X/L = 0.5 (Mid-Span)	0.50	0.53	0.67	0.45	0.48	0.60
	X/L = 1.0 (Centre Pier)	0.71	0.97	**1.27**	0.61	0.83	**1.08**

TABLE 3: ULS DEMAND/CAPACITY RATIO – SHEAR FORCES

Span	Location	Section 3			Section 14		
		Ped	Traffic	Combined	Ped	Traffic	Combined
End Spans	X/L = 0.0 (Abutment)	0.36	0.46	0.54	0.34	0.42	0.49
	X/L = 0.5 (Mid-Span)	0.29	0.66	0.71	0.21	0.59	0.63
	X/L = 1.0 (Exterior Pier)	0.70	0.90	**1.10**	0.45	0.78	0.95
Centre Spans	X/L = 0.0 (Exterior Pier)	0.48	0.68	0.78	0.32	0.62	0.70
	X/L = 0.5 (Mid-Span)	0.24	0.56	0.60	0.14	0.49	0.54
	X/L = 1.0 (Centre Pier)	0.46	0.60	0.71	0.32	0.55	0.63

Note: Inspection Level – INSP 3 was assumed for determining the target reliability index, β for Section 14 loads for sake of comparison with CHBDC Section 3 results in **Tables 2** and **3**.

As indicated above, the existing superstructure bending moment and shear capacities were determined to be sufficient to carry the factored loads and meet the minimum requirements of the CHBDC for the proposed DDI option for either the pedestrian only or traffic only load cases for Section 3 and Section 14 load factors. To satisfy the pedestrian/traffic loading combination, structural strengthening would be required to increase the negative bending moment capacity and possibly the girder shear strength, at the piers. Other critical locations such as at stirrup transitions (not presented here) were reviewed and found to have adequate reserve capacity.

PIERS CAPS, COLUMNS AND FOOTINGS

The pier caps were evaluated in longitudinal and transverse directions and were determined to be adequate to carry the added loads. The pier columns and pier and abutment footings were also determined to be adequate for the added loads. All components were reviewed for the pedestrian/traffic load case and were found to be adequate under this load case.

FOUNDATIONS

A desktop study was completed by GeoPro Consulting who provided design bearing capacities. **Table 4** summarized the factored bearing capacities.

TABLE 4: FACTORED BEARING CAPACITIES

Location	Soil Type	Serviceability Limit State (kPa)	Ultimate Limit State (kPa)
Center and South Piers	Shale Bedrock	950	1400
North Pier	Weathered Shale	480	720
Abutments	Engineered Fill	300	600

Based on our evaluation, the applied loads will not exceed the bearing capacities listed in **Table 4** for the worst case of combined pedestrian and traffic loadings**.**

CONCLUSIONS

Based on our findings discussed above, the existing structure is deemed to have the capacity to carry the CHBDC required loads and loads result from the construction of the proposed cross section and lane configuration, detailed in **Figure 3**, with no strengthening required. This assumes that the maximum pedestrian loads do not occur coincidentally with maximum traffic loads, which is considered reasonable for this site.

DILLON CONSULTING LIMITED
LONDON, ONTARIO

____
Brad Schmidt, P.Eng
Structural Engineer

____
Hossam Bakr, EIT
Structural Designer