Sir John A. Macdonald Parkway Bridge O/P Lebreton (SN016470)

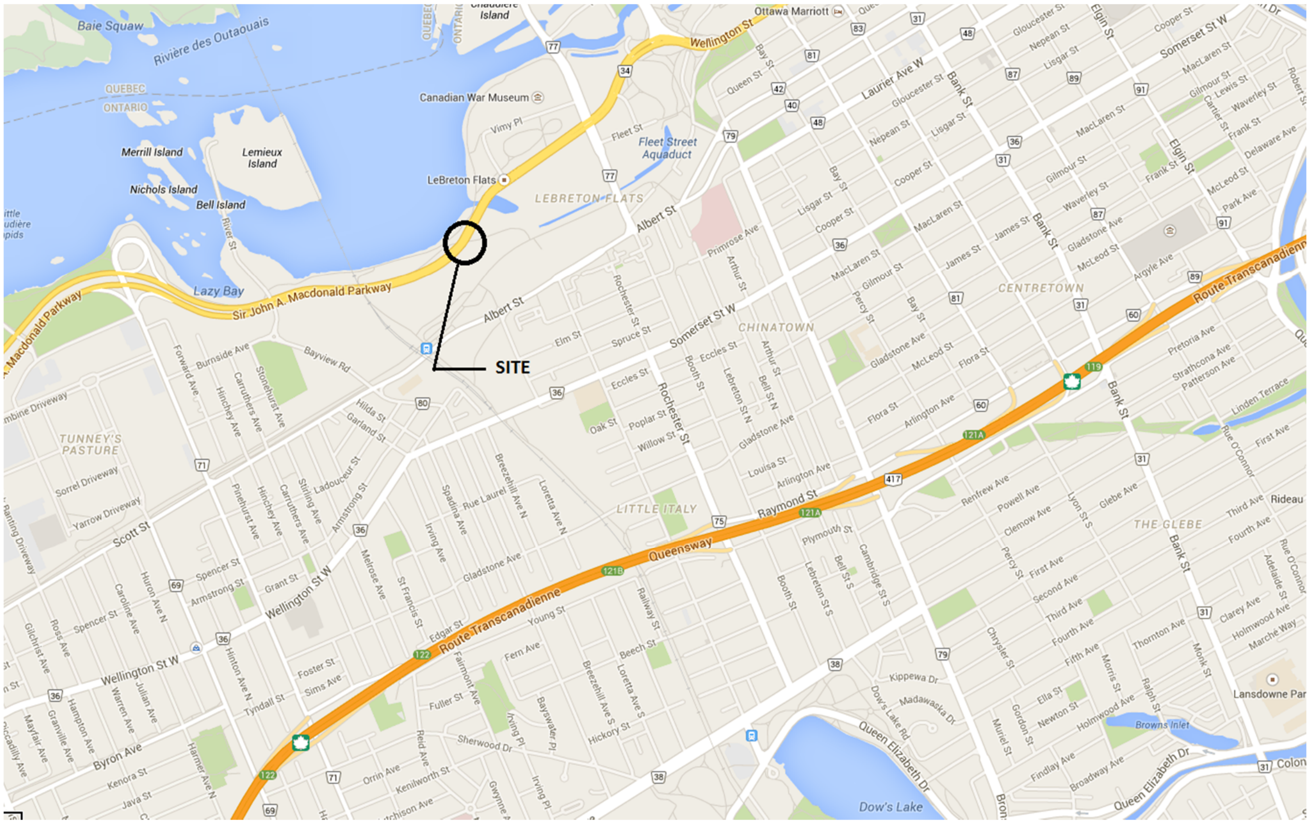
MH NO. 2140670
Contract No. ISD14-7114

STRUCTURE NO. 016470

STRUCTURE EVALUATION REPORT

January 2015

MORRISON HERSHFIELD


SITE
Rivière des Outaouais
Baie Squaw
QUEBEC
ONTARIO
Merrill Island
Lemieux Island
Nichols Island
Bell Island
Lazy Bay
Sir John A. Macdonald Parkway
Canadian War Museum
LeBreton Flats
LEBRETON FLATS
Fleet Street Aquaduct
Ottawa Marriott
Wellington St
Albert St
Somerset St W
Laurier Ave W
Bayview Rd
Scott St
Wellington St W
TUNNEY'S PASTURE
CHINATOWN
LITTLE ITALY
CENTRETOWN
THE GLEBE
Queensway
Route Transcanadienne
Raymond St
Rochester St
Bank St
Elgin St
Gladstone Ave
Queen Elizabeth Dr
Dow's Lake
Browns Inlet
Lansdowne Par

Key Plan

Appendix A – Bridge Design Drawings
Appendix B – Site Photos
Appendix C – Vehicle Load Information
Appendix D – Structural Analysis Calculations
Appendix E – Geophysical and Asphalt Coring Investigation Reports

2. SITE INFORMATION

Structure Name	Sir John A. Macdonald Parkway Bridge O/P Lebreton		
Site Number	SN016470		
Highway Above	Sir John A. Macdonald parkway	Below	Lebreton St./Bike Path
Type of Structure	Post-tensioned Concrete Structure		
Number of Spans	2	Span Lengths (m)	16.31, 20.88
Overall Structure Width	24.08	Year Built	1967
Direction of Structure	West to East		
Party Members	Joseph Ostrowski		
	Hui Liu		
	SPL Consultants Limited		
Dates of Inspection	October 24 2014; November 24, 2014 and January 15,16, 2015		
Year Last Rehabilitated:	2006		

3. INTRODUCTION

3.1 Background

Morrison Hershfield Limited was retained by the City of Ottawa to carry out the Structural Evaluation of the Sir John A. Macdonald Parkway (SJAM) Bridge O/P Lebreton (SN016470).

As a result of Ottawa Light Rail Transit's (OLRT'S) west portal and track alignment, there is a plan to route empty OC Transpo buses onto the SJAM Parkway, between Parkdale Avenue and Preston Street extension, for a period of 3 years. This report evaluates the capacity of the SJAM Parkway Bridge O/P Lebreton (SN016470) to carry OC Transpo buses and provides recommendations for necessary remedial work.

3.2 General Description and History of Structure

The SJAM Overpass (SN016470) is located on the SJAM Parkway, about 0.72 km east of Slidell Street. Built in 1967, the structure is a two span, post-tensioned concrete frame with an overall span of 37.19 m and the overall width of 24.08 m. The bridge is curved and skewed 45° to the highway alignment. Minor rehabilitations were carried out in 1980-1981, 1984, 1987, and 2006. A major deficiency is the superstructure deformation due to the settlement of falsework during initial construction, resulting in sag of more than 200 mm in the north span; no documentation is available of the method of compensation for the road profile. Deficient railings have been temporarily corrected with concrete jersey barriers. Trucks and buses are restricted to travel over this section of SJAM Parkway.

4. METHODOLOGY OF INSPECTION/ANALYSIS

The inspection and assessment of the structure was carried out in accordance with the 2008 Ontario Structure Inspection Manual (OSIM). In addition, coring and geophysical investigations were undertaken to determine hidden parameters related to the sag in the superstructure.

The visual inspection was conducted by Hui Liu, P.Eng. on October 24, 2014 under the direction of Joseph Ostrowski, P.Eng. Coring investigations to determine the asphalt thickness in the sag region of the bridge and Geophysical investigations using Ground Penetrating Radar (GPR) to verify the position and depth of the post-tensioning tendons were carried out on November 24, 2014 by SPL Consultants. Supplementary coring investigations for additional information regarding the deck and asphalt thickness in the sag region were performed on January 15 and 16, 2015

The structural evaluation was in accordance with CAN/CSA-S6-06, Canadian Highway Bridge Design Code (CHBDC). The analysis was based on the original construction drawings, supplemented by site observations and measurements.

5. SUMMARY OF SIGNIFICANT STRUCTURAL FINDINGS

The structure is generally in fair condition, no significant deterioration was noted other than the sag of the bridge deck which was due to sagging falsework during construction. This section summarizes the most significant findings of the visual inspection. Detailed descriptions are provided in the individual component subsections. Site photographs of the structure and components are included in Appendix B.

5.1 Concrete Bridge Deck

The bridge deck is a 2-span post-tensioned, solid concrete slab. The bridge deck cantilevers, poured after the main deck, do not contain post-tensioning tendons. Raised concrete aprons or sidewalks on either side of the bridge and the raised concrete median consist of precast, post-tensioned concrete panels.

The sag deformation in the north span is greatest on the east side of the bridge (photo 1). The visible components generally follow the sag; however the extent of any padding or overlay is not documented.

The bridge deck is generally in good condition, but the cantilevers exhibit previous patch repairs and deterioration (cracks, minor spalls) indicative of corrosion of embedded reinforcing steel as a result of splash and spray from the roadway (Photos 3,4,5,15).

5.2 Abutments

Both abutments are generally in good condition. Minor spalling at the southeast corner of the bridge was noted. Evidence of water leakage through the expansion joints includes staining and rusted shoe plates (Photo 7).

5.3 Wingwalls

The exposed parts of wingwalls are in good condition.

5.4 Bearings

The elastomeric bearings and steel plates are generally in fair to poor condition. Horizontal cracks (Photo 7, 8) were noted on several elastomeric bearings and the steel bearing shoe plates are severely corroded. Significant differential deformations between the bearings were noted.

5.5 Piers

The two reinforced concrete legs of the pier are in good condition. The exposed concrete surfaces appear to have been coated with concrete sealer (Photo 3). No significant deficiencies were noted.

5.6 Concrete Sidewalks and Median

The post-tensioned precast concrete sidewalks and median are in fair condition. Light scaling and narrow to wide cracks were noted on the surface of sidewalks and median, light spalling was also noted at several locations on the sidewalk (Photo 9, 13). Grout in the installation holes was broken off at some locations.

5.7 Curbs

The sidewalks and median curbs are generally in fair to poor condition. Localized abrasion, spalling, cracks and corroded reinforcement were noted (Photo 10) .

5.8 Railings

The steel HSS section railing and post traffic barrier system on the bridge are generally in fair condition. Severe corrosion was noted at the ends of railings (Photo 11); the portions over the bridge deck were well maintained. Missing anchor bolts were noted at two posts. The existing railing system does not meet current CHBDC requirements.

5.9 Barriers

Temporary Concrete Barriers (TCBs) were installed on both sides of the bridge immediately behind the curb face and in front of the steel railings. The TCBs are not anchored to the deck but are offset approximately 1.2 m from the edge of the deck. The TCBs are generally in good condition, no instability issue was noted.

5.10 Asphalt

The asphalt wearing surfaces in the west bound lane and east bound lane are in fair to poor condition. Large cracks and asphalt raveling coincide with the expansion joints at either end of the bridge deck. Severe longitudinal cracks and light to medium transverse cracks were noted near the approach slabs.

5.11 Foundations

The foundations were not accessible during the time of inspection. No visible evidence of geotechnical instability was observed.

5.12 Embankments

Both the north and south embankments are in good condition, the embankments have been well protected with grouted laid stone. Some loose stones (about 2.5 m^2 area) were observed at the northwest corner of the embankment.

5.13 Approaches

No significant findings were noted on the approach slabs.

5.14 Expansion Joints

The paved over expansion joints at both abutments are continuous across the bridge. The expansion joint assemblies are not visible. There are no concrete end dams. The asphalt pavement is distressed at the joints (parallel cracking, ravelings) and there is evidence of water leakage through the joints.

6. STRUCTURAL EVALUATIONS

The structural evaluation for the SJAM Parkway Bridge O/P Lebreton (SN016470) was carried out based on current condition of the structure. Details of the structural evaluation are provided in Appendix D.

6.1 Reference Material

The following information was obtained and used in carrying out the analysis:

1. Original design drawings, M. M. Dillon & Company Limited Consulting Engineers, February 1966.
2. Condition Inspection Report, 23 June 1994.
3. ORP Ramp E2000 Layout Survey.
4. Rehabilitation Drawings, Genivar, March 2006
5. Canadian Highway Bridge Design Code, CAN/CSA-S6-06
6. Structural Manual, MTO

6.2 Load Carrying Capacity

6.2.1 Method of Evaluation

The bridge is 2-span skewed (45 degrees) post-tensioned concrete structure. The primary post-tensioning tendons are perpendicular to the abutments and secondary post-tensioning tendons are at right angles to the primary tendons. The south span is 13.7 m long and the north span 23.47 m (measured along the centerline of the median). The south span is a rigid frame while the north span slides on bearings at the north abutment. The deck varies in thickness and the overall width is 24.1 m (measured perpendicular to the centre line of the median).

In accordance with CHBDC, only ultimate limit states are considered for the bridge evaluation. Due to the skew and curvature, the simplified method of analysis is not valid for this bridge configuration. Therefore, the structure was modeled in three dimensions and analyzed with SAP2000 finite element analysis program.

6.2.2 The Effect of Superstructure Settlement

Although it is known that the falsework shifted during construction, resulting in a 'sag' of more than 200 mm in the bridge superstructure, 'as- built' construction records are not available. Consequently, there is uncertainty regarding any additional dead load due to profile corrections (asphalt padding or concrete overlay), the effective deck thickness and most critically, the location of the post-tensioning tendons.

Asphalt cores at 13 locations in the sag region revealed asphalt thickness varying from 92 mm to 140 mm in the westbound lanes and 118 mm to 170 mm (2 lifts asphalt paving plus waterproofing system) in the eastbound lanes. The asphalt thickness in the south span (without sag) is around 90 mm (the original design thickness was 76 mm).

Full depth (through deck) cores at two locations revealed increased concrete deck thickness in the sag region, approximately 40 mm thicker than the original design thickness in the eastbound lanes.

While the local effects of increased asphalt thickness (additional dead load) are countered by the increased deck thickness (additional moment capacity), the global effects of the increased dead load (due to asphalt padding and thicker deck) include larger negative bending moments over the pier.

Ground Penetrating Radar scans in the sag region of the bridge identified post-tensioning ducts at 90 mm from the soffit of the bridge deck at approximately 600 mm spacing. These findings are consistent with the dimensions shown on the design drawings, providing confirmation that the relative position of the post tensioning tendons was not compromised. Details of the investigation are provided in Appendix F.

6.2.3 Loading

For the purpose of this evaluation, it is assumed that the deck dimensions are as shown on the design drawings, except that additional dead load (170 mm asphalt thickness) is considered in the sag region.

The ULS1 load combination was evaluated considering for the following 4 cases:

Case 1: Unladen buses in curb lanes only
Case 2: Unladen buses in all 4 lanes
Case 3: Fully loaded buses in curb lanes only
Case 4: Fully loaded buses in all 4 lanes.

The bus configurations and loadings (provided by OC Transpo) follow:

Type A: New Flyer INVERO
Two axles, distance of axles 7.17m, maximum axle weight 88.63kN, total unladen weight of vehicle 133.44kN.

Type B: New Flyer Articulated D60LFR
Three axles, distance of outmost axles 13.48m, maximum axle weight 101.80kN, total unladen weight of vehicle 202.68kN.

Type C: Orion VII Hybrid
Two axles, distance of axles 7.22m, maximum axle weight 97.61kN, total unladen weight of vehicle 142.7kN.

Type D: Alexander Dennis Double Decker ENVIRO 500
Three axles, distance of outmost axles 8.0m, maximum axle weight 78.1kN, total unladen weight of vehicle 178.5kN. The Type D bus is the most critical one among the 4 types of vehicles.

Additional data regarding the buses is provided in Appendix C.

6.2.4 Results of Evaluation

The factored moments and resistances at critical locations for all the load cases considered are presented below:

ULS Case 1		Critical Structural Element: Midspan of North Span		Critical Structural Element:Deck Over Pier	
Unladen Buses (In Curb Lanes Only)		M_f^+ (kNm/m)	(M_r^+/M_f^+)	M_f^- (kNm/m)	(M_r^-/M_f^-)
DL+ SDL + Prestress Secondary. Moments		760	1.00	2290	1.10
DL + SDL + Prestress Secondary Moment + LL	New Flyer INVERO (Type A)	860	0.88	2571	0.98
	Articulated D60LFR (Type B)	872	0.87	2650	0.95
	Orion VII Hybrid (Type C)	868	0.88	2588	0.97
	Double Decker ENVIRO 500 (Type D)	879	0.86	2650	0.95
Resistance (M_r)		760		2517	

Table 1: Unladen Buses In Curb Lanes Only

ULS Case 2		Critical Structural Element: Midspan of North Span		Critical Structural Element:Deck Over Pier	
Unladen Buses (In 4 Lanes)		M_f^+ (kNm/m)	(M_r^+/M_f^+)	M_f^- (kNm/m)	(M_r^-/M_f^-)
DL+ SDL + Prestress Secondary. Moment		760	1.00	2290	1.10
DL + SDL + Prestress Secondary Moment + LL	New Flyer INVERO (Type A)	881	0.86	2643	0.95
	Articulated D60LFR (Type B)	881	0.86	2732	0.92
	Orion VII Hybrid (Type C)	891	0.85	2664	0.94
	Double Decker ENVIRO 500 (Type D)	911	0.83	2747	0.92
Resistance (M_r)		760		2517	

Table 2: Unladen Buses In 4 Lanes

ULS Case 3		Critical Structural Element: Midspan of North Span		Critical Structural Element:Deck Over Pier	
Fully Loaded Buses (In Curb Lanes Only)		M_f^+ (kNm/m)	(M_r^+/M_f^+)	M_f^- (kNm/m)	(M_r^-/M_f^-)
DL+ SDL + Prestress Secondary. Moment		760	1.00	2285	1.10
DL + SDL + Prestress Secondary Moment + LL	New Flyer INVERO (Type A)	896	0.85	2671	0.94
	Articulated D60LFR (Type B)	897	0.85	2804	0.90
	Orion VII Hybrid (Type C)	896	0.85	2672	0.94
	Double Decker ENVIRO 500 (Type D)	914	0.83	2759	0.91
Resistance (M_r)		760		2517	

Table 3: Fully Loaded Buses In Curb Lanes Only

ULS Case 4		Critical Structural Element: Midspan of North Span		Critical Structural Element:Deck Over Pier	
Fully Loaded Buses (In 4 Lanes)		M_f^+ (kNm/m)	(M_r^+/M_f^+)	M_f^- (kNm/m)	(M_r^-/M_f^-)
DL+ SDL + Prestress Secondary. Moment		760	1.00	2290	1.10
DL + SDL + Prestress Secondary Moment + LL	New Flyer INVERO (Type A)	927	0.82	2775	0.91
	Articulated D60LFR (Type B)	933	0.81	2927	0.86
	Orion VII Hybrid (Type C)	927	0.82	2775	0.91
	Double Decker ENVIRO 500 (Type D)	956	0.79	2889	0.87
Resistance (M_r)		760		2517	

Table 4: Fully Loaded Buses In 4 Lanes

The results indicate that the structure is overstressed for all bus loading cases. The overstress occurs in both the positive moment region (mid-span) of the north east quadrant and the negative moment region over the pier.

6.3 Traffic Barrier Evaluation

The traffic barriers on the bridge consist of a permanent HSS railing system augmented by temporary concrete barriers (TCBs) located 1.2m from either edge of the bridge.

The required level of protection, assuming an AADT of approximately 26,000 (based on turning movement counts at Vimy Place Intersection), is PL-2.

The existing railing configuration does not match any of the current crash-tested barrier requirements required by the CHBDC. However the existing TCB, located more than 1.0m (deflection distance) from the edge, provides adequate traffic protection.

7. RECOMMENDATIONS

7.1 Remedial Work Needed To Accommodate OC Transpo Buses

Shoring is recommended to prop the north span of the bridge. Temporary shoring could be implemented without major disruption and without permanently altering the bridge. Upon completion of the adjacent LRT construction, the installation would be removed. The shoring system has a commercial salvage that could be recovered. The proposed shoring layout is shown on the drawing General Arrangement in Appendix A.

7.2 General Rehabilitation to Extend the Bridge Service Life

1. Remove existing TCBs, railing system, sidewalks and bridge deck cantilevers. Reconstruct conrete sidewalk and bridge cantilever to accommodate new PL2 steel railing.

2. Replace elastomeric bearings and steel plates.

3. Localized concrete patch repair.

4. Replace expansion joint assemblies with strip seal type assemblies. Although eliminating the expansion joints by means of semi-integral abutment conversion may be feasible, it is not recommended due to performance complications associated with the high skew angle.

8. CLOSURE

We trust that this report is sufficient for your immediate requirements. Please contact us if there are any questions or concerns regarding the evaluation or recommendations contained herein.

Sincerely,

Hui Liu

Hui Liu, P.Eng
Structural Engineer
Morrison Hershfield Limited

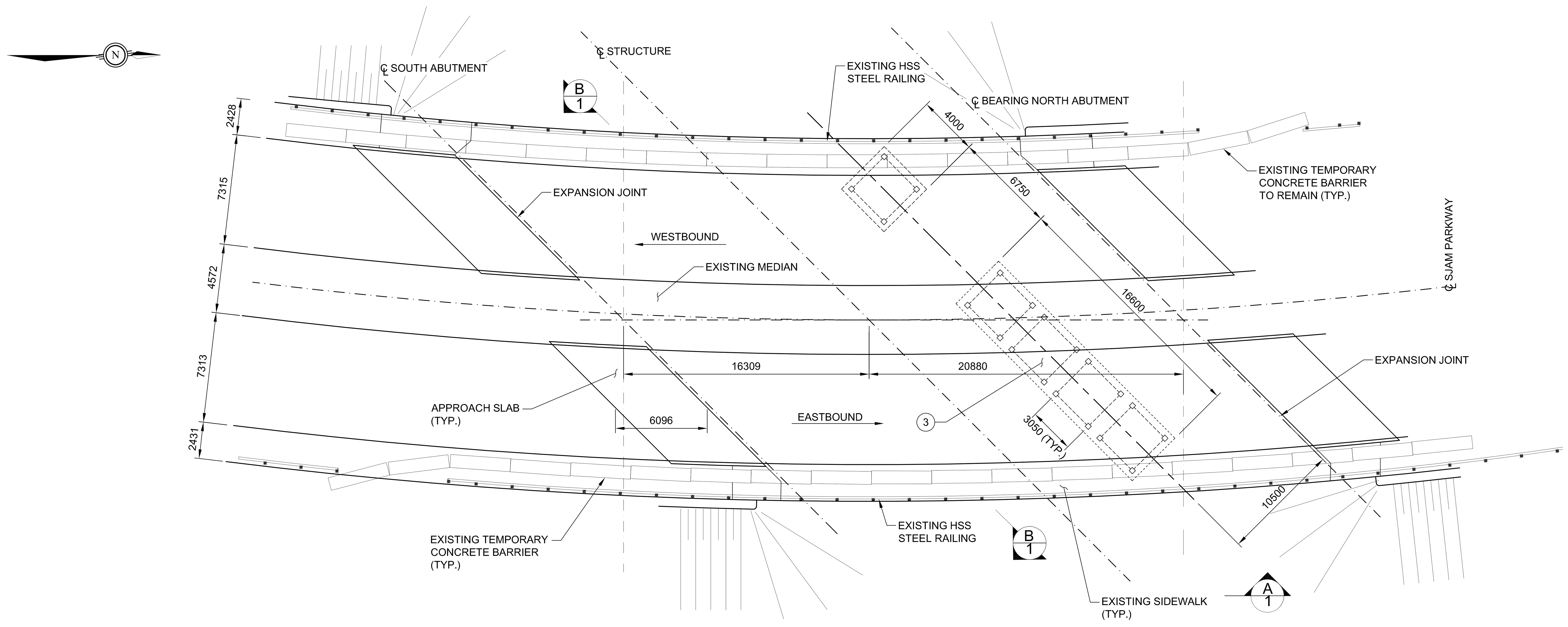
J Ostrowski

Joe Ostrowski, P.Eng.
Project Manager
Morrison Hershfield Limited

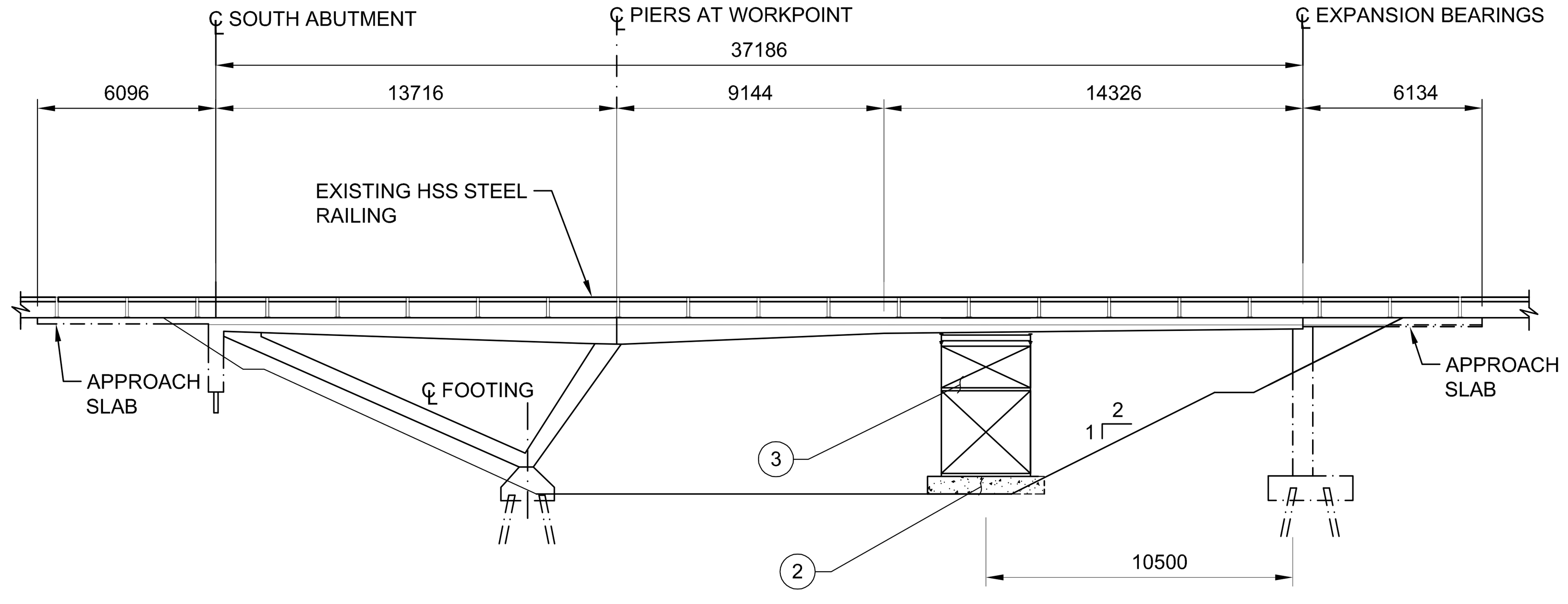
APPENDIX A

Bridge Design Drawings

DRAWING FRAME: 790mm x 534mm City of Ottawa 2008



PLAN
1:200



ELEVATION (A/1)
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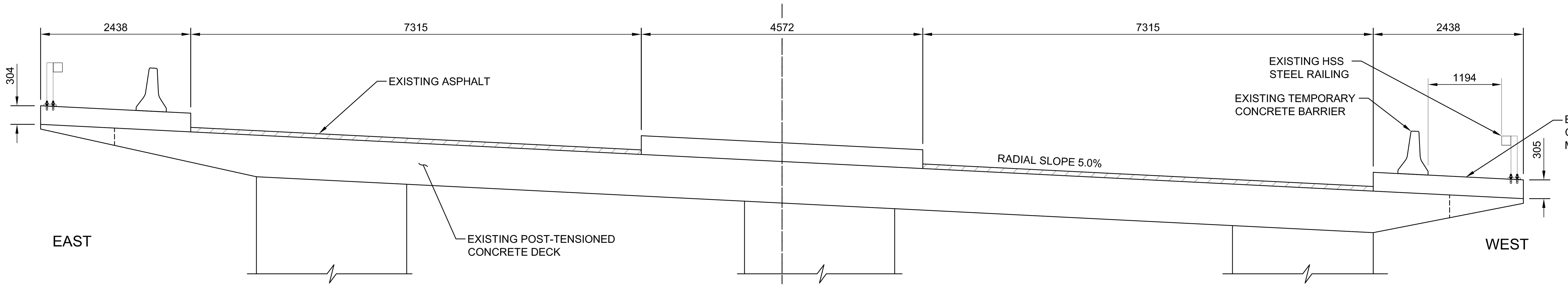


SECTION (B/1)
1:50

SCOPE OF WORK INCLUDES BUT IS NOT LIMITED TO:

1. TRAFFIC CONTROL
2. CONSTRUCT FOUNDATION PADS FOR SHORING TOWERS
3. SUPPLY AND ERECT HIGH-LOAD SHORING TOWERS
4. REMOVE SHORING TOWERS AND CONCRETE FOUNDATION PADS AFTER COMPLETION OF ADJACENT LRT CONTRACT. RESTORE SITE TO ORIGINAL OR BETTER CONDITION

SIR JOHN A. MACDONALD PARKWAY
BRIDGE O/P LEBRETON
(SN016470)

Ottawa

GENERAL ARRANGEMENT

Contract No. ISD14-7114 | Dwg. No. 1
Sheet 1 of 1

Asset No. ______
Asset Group ______

MORRISON HERSHFIELD

Des.	HL	Chk'd.	JBE
Dwn.	GK	Chk'd.	HL
Utility Circ. No.		Index No.	

Const. Inspector

Scale: AS NOTED

NOTE: The location of utilities is approximate only, the exact location should be determined by consulting the municipal authorities and utility companies concerned. The contractor shall prove the location of utilities and shall be responsible for adequate protection from damage.

REVISIONS

No.	Description	By	Date (dd/mm/yy)

NOTES:

1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH ALL OTHER APPLICABLE CONTRACT DRAWINGS. INFORMATION SHOWN ON THIS DRAWING HAS BEEN EXTRACTED FROM THE ORIGINAL DESIGN DRAWINGS (PROJECT 5877 - 20 - 4) AND SUPPLEMENTED BY FIELD INVESTIGATIONS.

CONSTRUCTION NOTES:

1. THE CONTRACTOR SHALL VERIFY ALL EXISTING CONDITIONS, RELEVANT DIMENSIONS, AND ELEVATIONS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE COMMENCING WORK.
2. CONTRACTOR TO COORDINATE WITH ANY AFFECTED UTILITY COMPANIES FOR RELOCATION.
3. CONTRACTOR TO SUPPORT ALL UTILITIES WITH A METHOD APPROVED BY THE GOVERNING UTILITY.
4. ALL REMOVED MATERIALS TO BE DISPOSED OF OFF SITE, IN ACCORDANCE WITH MUNICIPAL AND PROVINCIAL GUIDELINES.

LEGEND

ASPHALT

NEW CONCRETE

APPENDIX B

Site Photos

Photo 1: East elevation of the bridge

Photo 2: Top of bridge

Photo 3: Bridge deck soffit and fascia

Photo 4: Soffit of bridge deck

Photo 5: Corrosion of reinforcement in cantilever

Photo 6: Overbuilt patches in the bridge deck

Photo 7: leaking water and rusted steel plates

Photo 8: Horizontal cracks in elastomeric in bearing

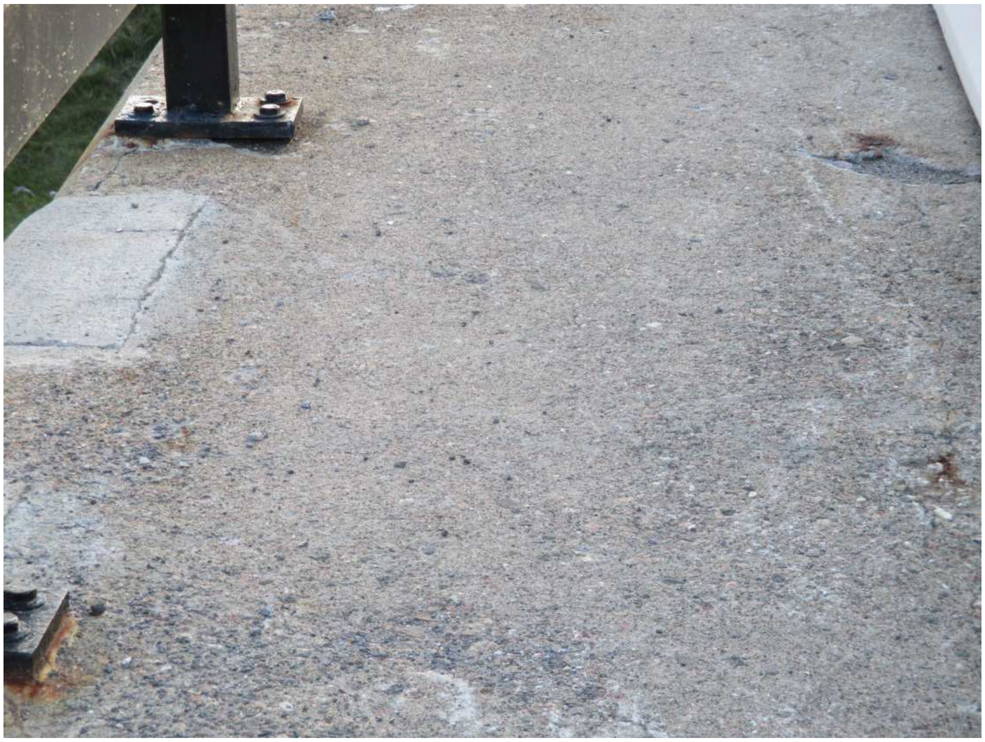


Photo 9: Sidewalks

Photo 10: Spalling in curb

Photo 11: Corrosion and missing connection in railing

Photo 12: Severe cracks in asphalt at expansion joint

Photo 13: Cracks in median

Photo 14: Expansion joint

Photo 15: Loose stones in embankment

Photo 16: West wingwall

APPENDIX C

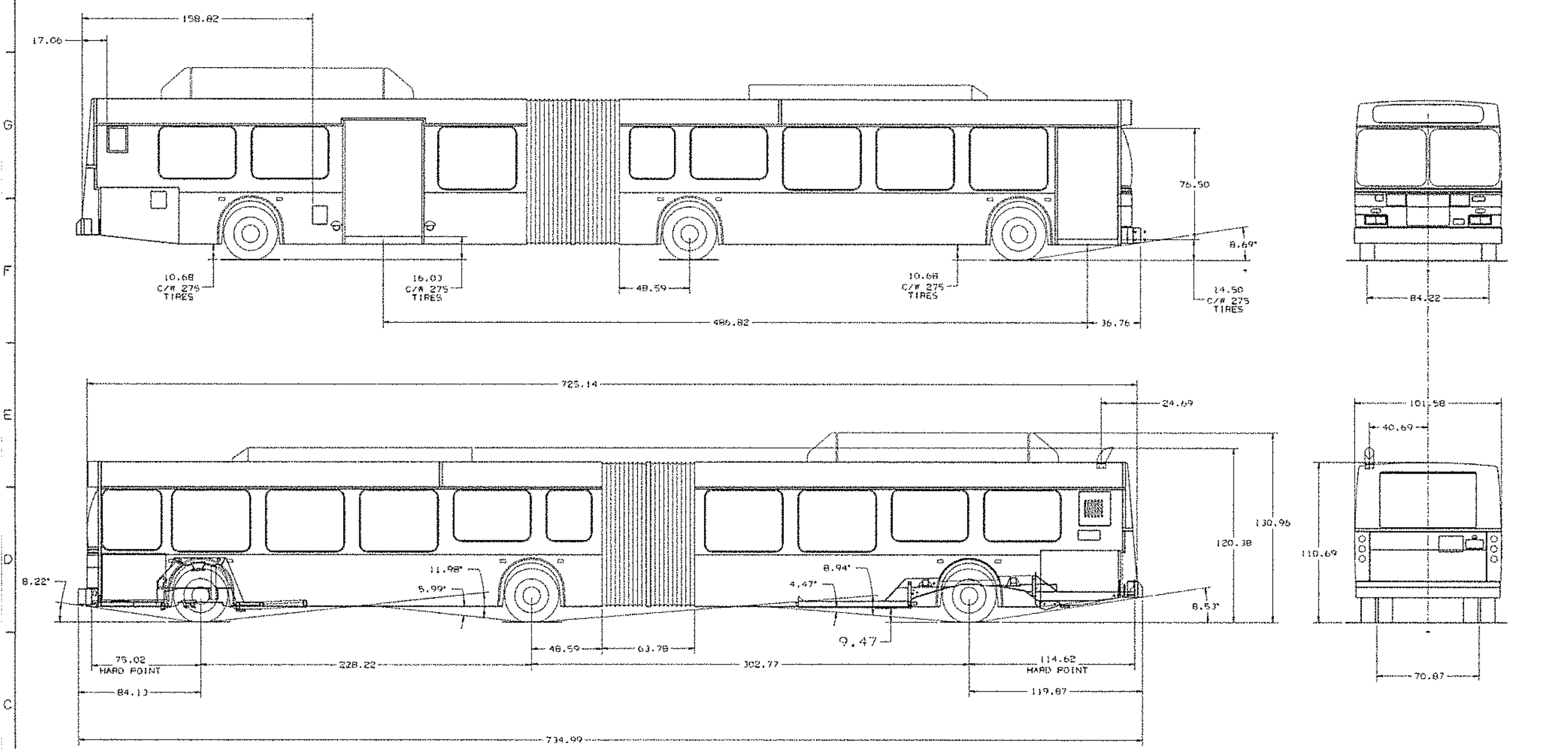
Vehicle Load Information

OC Transpo Transit Fleet - Bus Axle Loads

Bus Type	New Flyer INVERO	New Flyer Articulated D60LFR	Orion VII Hybrid	Alexander Dennis Double Decker ENVIRO 500
platform	40 - ft	60 - ft	40 - ft	40 - ft
front axle weight	9880 lb (4481 Kg)	9220 lb (4182 Kg)	9940 lb (4509 Kg)	11023 lb (5000 Kg)
# of wheels-front	2	2	2	2
centre axle weight	N/A	13020 lb (5906 Kg)	N/A	17218 lb (7810 Kg)
# of wheels-centre	N/A	4	N/A	4
rear axle weight	19540 lb (8863 Kg)	22440 lb (10180 Kg)	21520 lb (9761 Kg)	11111 lb (5040 Kg)
# of wheels-rear	4	4	4	2
unladen weight	29420 lb (13344 Kg)	44680 lb (20268 Kg)	31460 lb (14270 Kg)	39352 lb (17850 Kg)
front axle GAWR	14780 lb (6700 Kg)	14770 (6700 Kg)	6704 Kg	7100 Kg
centre axle GAWR	N/A	24250 lb (11000 Kg)	N/A	10000 Kg
rear axle GAWR	27760 lb (12590 Kg)	27760 lb (12590 Kg)	12592 Kg	7100 Kg
number of buses	326	359	177	75
Notes	GAWR as per the OEM stamp.	GAWR as per the OEM stamp.	GAWR as per the OEM stamp.	1) GAWR as per the OEM stamp. 2) Centre axle is drive axle and rear axle is auxiliary axle.

THIS DRAWING AND ITS SUBJECT MATTER ARE DISCLOSED IN CONFIDENCE, MUST BE RETURNED UPON REQUEST AND SHALL NOT BE DISCLOSED TO OTHERS WITHOUT WRITTEN CONSENT OF NEW FLYER INDUSTRIES LIMITED

DRAWING Nº DE60LFB



ADD 0.62 ±.25 INCHES TO ALL VERTICAL DIMENSIONS
AND .65° TO ALL ANGLES
WHEN USING 305 TIRES

SYM	REVISION	DRAWN	CHK'D	ENG'R	RELEASE	DATE

NOTICE: THIS DRAWING IS A FACSIMILE OF THE ELECTRONIC MASTER. ALL REVISIONS MUST BE INCORPORATED ELECTRONICALLY. MANUAL REVISION IS NOT ALLOWED.

MATERIAL
WEIGHT
TREATMENT

DO NOT SCALE
THD ANGLE

TITLE DE61LFB

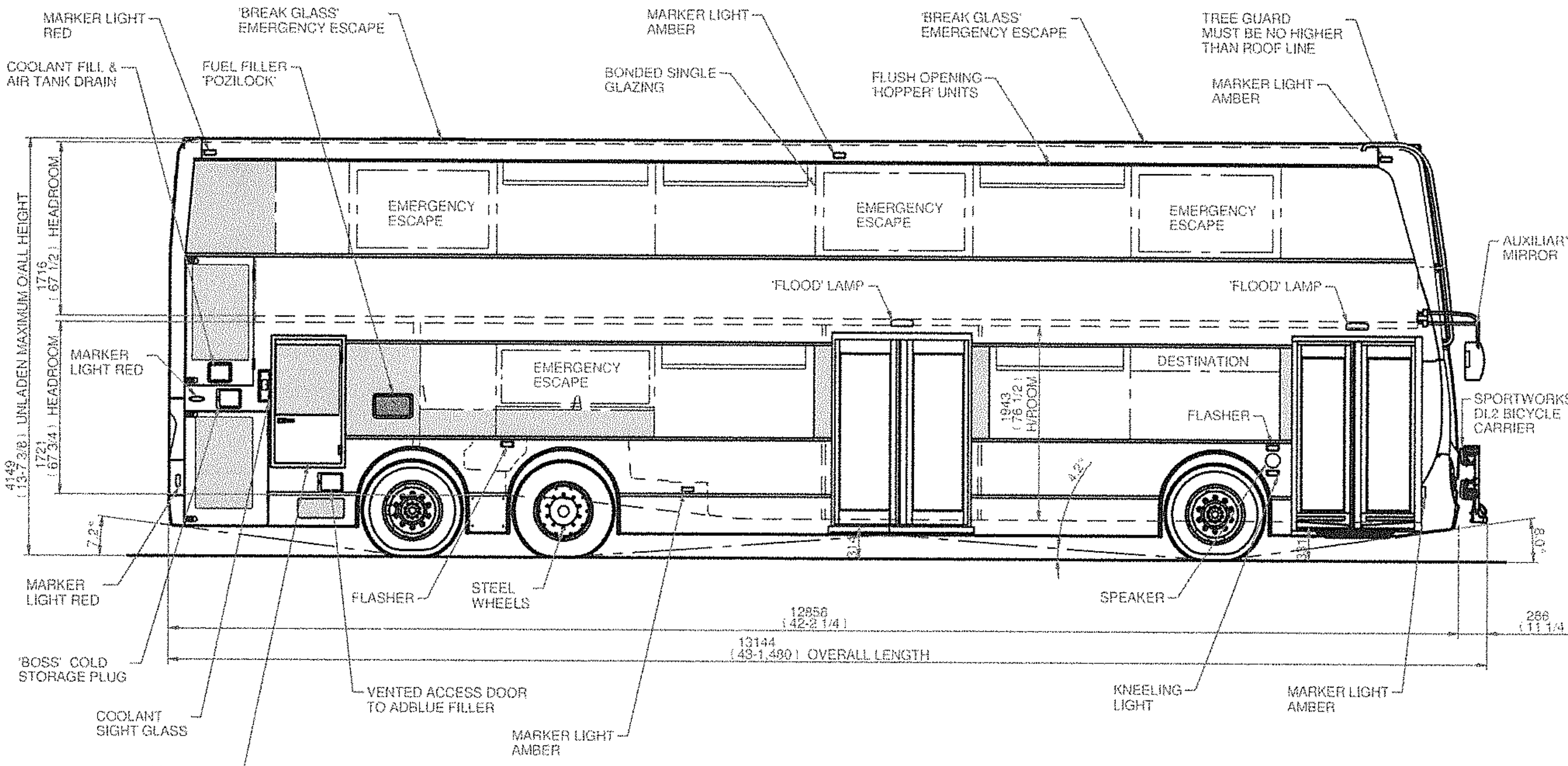
New Flyer

PART Nº DE61LFB

SCALE 1 : 25

SHEET 1 OF 1

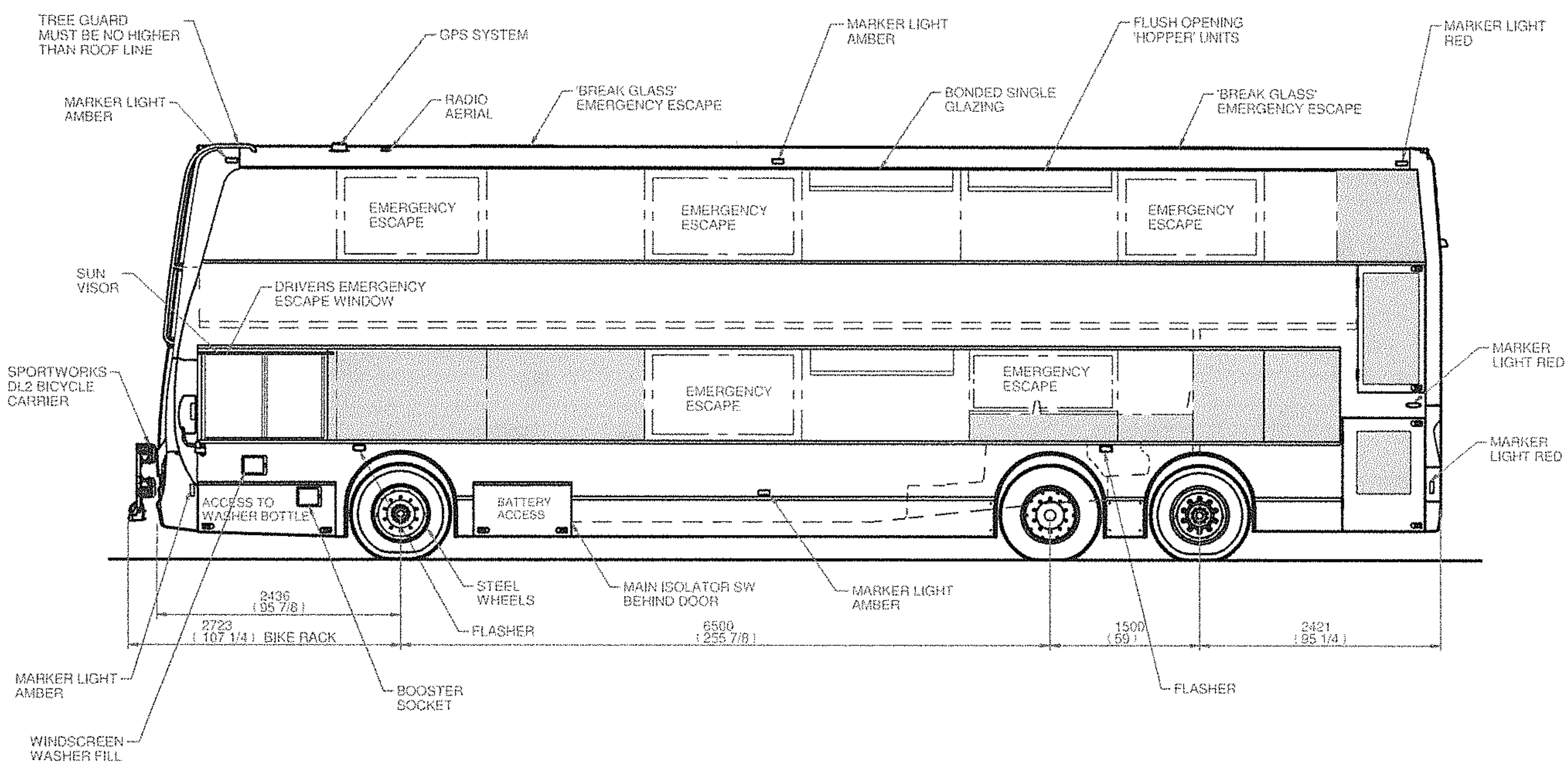
REPORT ALL ERRORS TO ENG. DEPT.



RIGHTHAND ELEVATION

STEP WILL REDUCE TO 250 - MID STEP WHEN FULLY KNELT

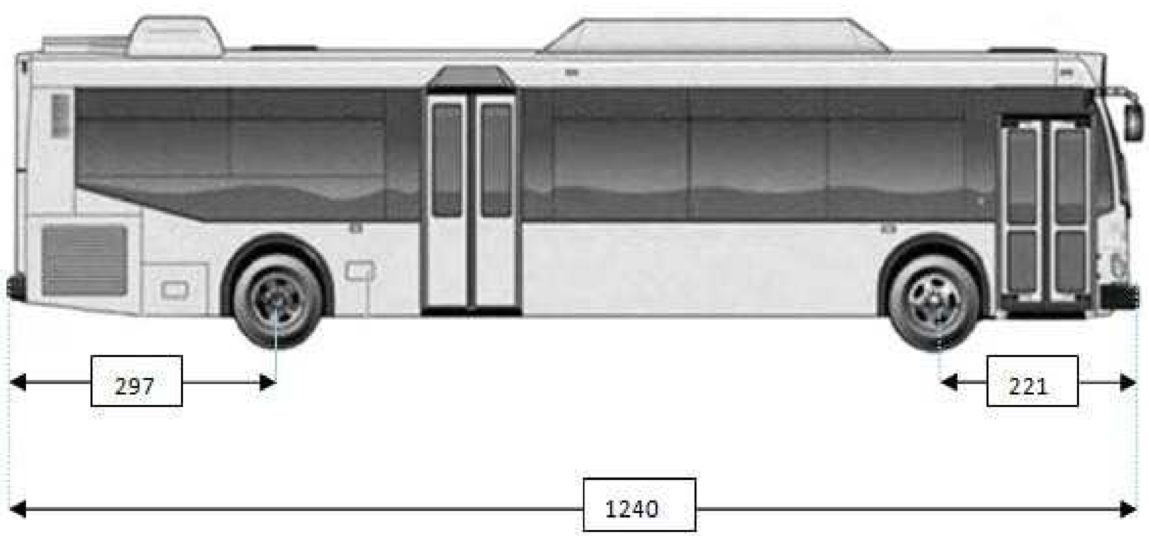
ALEXANDER DENNIS Form No EN-059	Rev	Date	Description	Name	Title : ENVIRO 500 12.8 METRE LOW HEIGHT LHD BODY DENNIS E500 CHASSIS - CUMMINS EPA10 ENGINE, COMPLETE WITH ALLISON B500R TRANSMISSION REF : 'OC TRANSPO' OTTAWA	Drawn : Alex. D Date : 27-01-12 Enq : 2011/0406/EDD	Seat Type : American Seating Vision Seat Alexander Dennis Ltd 91 Glasgow Road Falkirk FK1 4JB T- 01324 621672 F- 01324 621746	Iss No : 00 Page No : 1 Drg No : B503GA

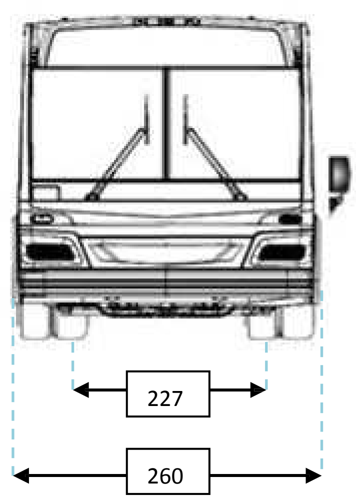


LEFTHAND ELEVATION

	Rev	Date	Description	Name				
ALEXANDER DENNIS					Title : ENVIRO 500 12.8 METRE LOW HEIGHT LHD BODY DENNIS E500 CHASSIS - CUMMINS EPA10 ENGINE, COMPLETE WITH ALLISON B500R TRANSMISSION REF : 'OC TRANSPO' OTTAWA	Drawn : Alex. D Date : 27-01-12 Enq : 2011/0406/EDD	Seat Type : American Seating Vision Seat Alexander Dennis Ltd 91 Glasgow Road Falkirk FK1 4JB T- 01324 621672 F- 01324 621746	Iss No : 00 Page No : 2 Drg No : B503GA
Form No EN-059								

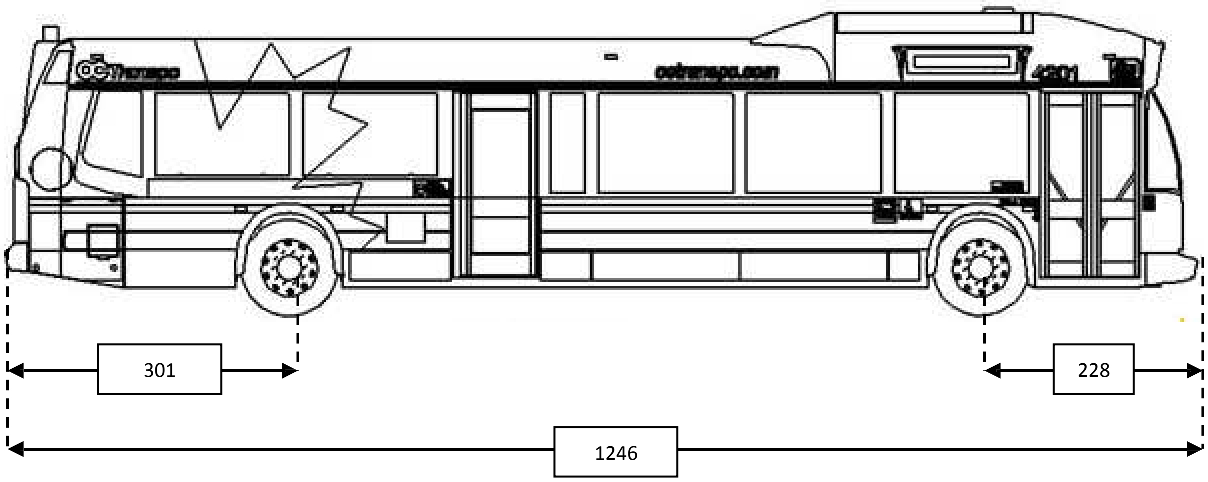
Orion VII Diesel-Electric Hybrid Bus

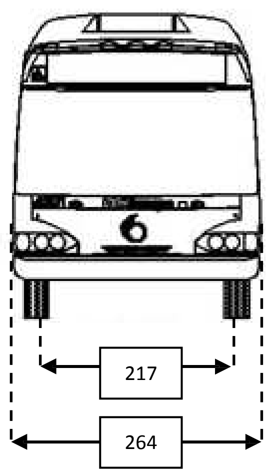




Dimensions are in cm

New Flyer INVERO





Dimensions are in cm

APPENDIX D

Structural Analysis Calculations

Bridge Evaluation

Project:	2140670	**Site No. :**	SN16470
Designer:	BL	**Date:**	Jan 23/ 15

1. Bridge Description (Rehabilitation)

Type of structure		Post tension bridge, built in 1966	
Length of Span (2 span)		13.716	23.470
Total Width		24.079	m
Width of Driving Lanes		14.630	m
Slab Thickness	min.	381	mm
Asphalt thickness	max.	170	mm
The Angle of Skew		45	°

2. Design Reference CHBDC Charpter 5. 8 & 14

3. Materials

Concrete	fc' =	34.5	Mpa
	Unit Weight =	34.5	kN/m3
	Mass Density, g_c =	2450	kg/m3
	Ec =	26951.480	Mpa
Reinforcing Steel	fy =	275	Mpa
	Ep =	200000	Mpa
Tendon	fpu =	1620	Mpa

4. Loads Statement S2, E2, INSP2, for Normal Traffic, β= 3.25

	Calculation	Load (KN/m)	Load Factor	DLA
from Deck			1.1	
from sidewalk(one side)		13.4	1.2	
from Barrier(one)		12.0	1.2	
from Asphalt		9.7	1.4	
from median		12.5	1.2	
from Double Deck Bus			1.6	0.3

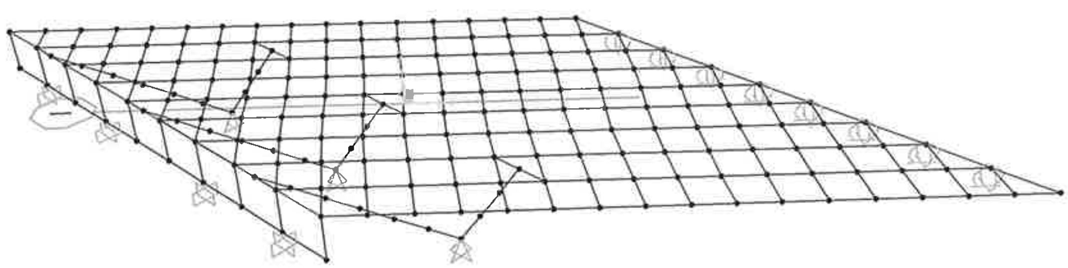
5. Summary of Forces

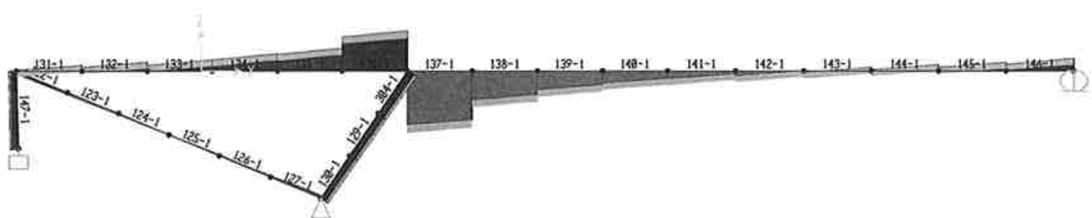
Superstructure

	Span L = 13.716m		Span L = 23.47m		
	V_{max} (KN)	M^{+}_{max} (KN)	V_{max} (KN)	M^{-}_{max} (KN)	M^{+}_{max} (KN)
Comb-ULS (2 lane Unladen Double Decker Bus)	578	566	900	2650	879
Resistance	1229	1222	1132	2517	760
Resistance /Facotored Load	2.1	2.2	1.3	0.95	0.86

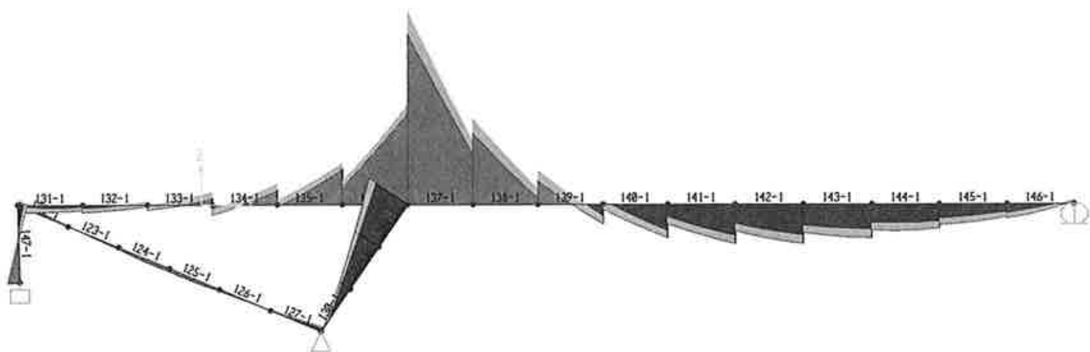
Pier (2 Lane)

	Pier Top	
	V_{max} (KN)	M_{max} (KN)
Comb-ULS (Double Decker Bus)	206	950
Resistance	1614	1308
Resistance /Facotored Load	7.8	1.4





SAP2000 v16.0.1 - File:SN16470 grillage model-perpendicular-extra asphalt-move pier - Shear Force 2-2 Diagram (COM



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127-1

PROJECT		SECTION	South Midspan
TITLE		DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Box or I Section Under Bending

Description:This spreadsheet shows the design of Box or I Section Girder subjected to Flexure

Data Input

Basic Data

Top Flange Width	b	=			1000.00	mm
Web Width	bw	=			1000.00	mm
Top Flange Thickness	hf	=			0.00	mm
Total Height	h	=			687.00	mm
Concrete Strength	fc'	=			34.48	MPa
Steel Rebar Strength	fy	=			275.00	MPa
Prestress Steel Strength	fpu	=			1620.00	MPa
Bottom Rebar Diameter	D	=			0.00	mm
Top Rebar Diameter	D'	=			0.00	mm
Prestress Cable Diametre	Dp	=			15.00	
Bottom Rebar Area	As	=			0.00	mm2
Top Rebar Area	As'	=			0.00	mm2
Prestress Steel Area	Ap	=			2730.31	mm2
Concrete Cover	t	=			38.10	mm
Concrete Cover for Tendons	tp	=			254.00	mm
Concrete Performance Factor	ϕc	=			0.75	
Steel Performance Factor	ϕs	=			0.90	
Prestress Strand P. Factor	ϕp	=			0.95	
Prestress Steel Type	Type	=			low	
	kp	=	IF(Type="low",0.3,IF(Type="re", 0.4,IF(Type="deformed",0.5)))	=	0.30	

Calculation

	α1	=	0.85-0.0015*fc'	=	0.80	
	β1	=	0.97-0.0025*fc'	=	0.88	
	ds	=	h-t-D/2	=	648.90	mm
	ds'	=	t+D'/2	=	38.10	mm
	dp	=	h-tp-Dp/2	=	425.50	mm
	de	=	IF(Ap>0,dp,ds)	=	425.50	
Value of c / dp	ratio	=	(ϕp*Ap*fpu+ϕs*As*fy-ϕs*As'*fy-α1*ϕc*fc'*hf*(b-bw))/(α1*ϕc*β1*fc'*bw*dp+ϕp*kp*Ap*fpu)	=	0.47	
	fps	=	fpu*(1-kp*ratio)	=	1393.67	MPa
Neutral axis location	a	=	(ϕs*As*fy+ϕp*Ap*fps-ϕs*As'*fy-α1*ϕc*fc'*hf*(b-bw))/(α1*ϕc*fc'*bw)	=	175.13	mm
	M1	=	(ϕs*As*fy*(ds-a/2)+ϕp*Ap*fps*(dp-a/2)-ϕs*As'*fy*(ds'-a/2)-α1*ϕc*fc'*hf*(b-bw)*(hf/2-a/2))/1000000	=	1221.59	KNm
	M2	=	(ϕs*As*fy*(ds-a/2)+ϕp*Ap*fps*(dp-a/2)-ϕs*As'*fy*(ds'-a/2))/1000000	=	1221.59	
	M3	=	(0.3*α1*ϕc*fc'*bw*de^2+α1*ϕc*fc'*(b-bw)*hf*(de-hf/2)+ϕs*fy*As'*(de-ds'))/1000000	=	1121.10	KNm
Moment Resistance	Mr	=	IF(ratio<0.5,IF(a<hf, M1,M2),M3)	=	1221.59	KNm
		=		=		

Check

Applied Moment	Mf	=		=	565.88	KNm
Check	check	=	IF(Mf<Mr,"OK","NG")	=	OK	

PROJECT		SECTION	North Midspan
TITLE	Type A Cables	DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Box or I Section Under Bending

Description:This spreadsheet shows the design of Box or I Section Girder subjected to Flexure

Data Input

Basic Data

Top Flange Width	b	=			1000.00	mm
Web Width	bw	=			1000.00	mm
Top Flange Thickness	hf	=			0.00	mm
Total Height	h	=			495.30	mm
Concrete Strength	fc'	=			34.48	MPa
Steel Rebar Strength	fy	=			275.00	MPa
Prestress Steel Strength	fpu	=			1620.00	MPa
Bottom Rebar Diameter	D	=			0.00	mm
Top Rebar Diameter	D'	=			0.00	mm
Prestress Cable Diametre	Dp	=			15.00	
Bottom Rebar Area	As	=			0.00	mm2
Top Rebar Area	As'	=			0.00	mm2
Prestress Steel Area	Ap	=			2730.31	mm2
Concrete Cover	t	=			38.10	mm
Concrete Cover for Tendons	tp	=			98.43	mm
Concrete Performance Factor	φc	=			0.75	
Steel Performance Factor	φs	=			0.90	
Prestress Strand P. Factor	φp	=			0.95	
Prestress Steel Type	Type	=			low	
	kp	=	IF(Type="low",0.3,IF(Type="re", 0.4,IF(Type="deformed",0.5)))	=	0.30	

Calculation

	α1	=	0.85-0.0015*fc'	=	0.80	
	β1	=	0.97-0.0025*fc'	=	0.88	
	ds	=	h-t-D/2	=	457.20	mm
	ds'	=	t+D'/2	=	38.10	mm
	dp	=	h-tp-Dp/2	=	389.38	mm
	de	=	IF(Ap>0,dp,ds)	=	389.38	
Value of c / dp	ratio	=	(φp*Ap*fpu+φs*As*fy-φs*As'*fy-α1*φc*fc'*hf*(b-bw))/(α1*φc*β1*fc'*bw*dp+φp*kp*Ap*fpu)	=	0.50	
	fps	=	fpu*(1-kp*ratio)	=	1375.83	MPa
Neutral axis location	a	=	(φs*As*fy+φp*Ap*fps-φs*As'*fy-α1*φc*fc'*hf*(b-bw))/(α1*φc*fc'*bw)	=	172.89	mm
	M1	=	(φs*As*fy*(ds-a/2)+φp*Ap*fps*(dp-a/2)-φs*As'*fy*(ds'-a/2)-α1*φc*fc'*hf*(b-bw)*(hf/2-a/2))/1000000	=	1081.04	KNm
	M2	=	(φs*As*fy*(ds-a/2)+φp*Ap*fps*(dp-a/2)-φs*As'*fy*(ds'-a/2))/1000000	=	1081.04	
	M3	=	(0.3*α1*φc*fc'*bw*de^2+α1*φc*fc'*(b-bw)*hf*(de-hf/2)+φs*fy*As'*(de-ds'))/1000000	=	938.82	KNm
Moment Resistance	Mr	=	IF(ratio<0.5,IF(a<hf, M1,M2),M3)	=	938.82	KNm
		=		=		

Check

Applied Moment	Mf	=		=	879.00	KNm
Check	check	=	IF(Mf<Mr,"OK","NG")	=	OK	

PROJECT		SECTION	North Midspan
TITLE	Type C Cables	DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Box or I Section Under Bending

Description:This spreadsheet shows the design of Box or I Section Girder subjected to Flexure

Data Input

Basic Data

Top Flange Width	b	=			1000.00	mm
Web Width	bw	=			1000.00	mm
Top Flange Thickness	hf	=			0.00	mm
Total Height	h	=			495.30	mm
Concrete Strength	fc'	=			34.48	MPa
Steel Rebar Strength	fy	=			275.00	MPa
Prestress Steel Strength	fpu	=			999.74	MPa
Bottom Rebar Diameter	D	=			0.00	mm
Top Rebar Diameter	D'	=			0.00	mm
Prestress Cable Diametre	Dp	=			34.93	
Bottom Rebar Area	As	=			0.00	mm2
Top Rebar Area	As'	=			0.00	mm2
Prestress Steel Area	Ap	=			527.44	mm2
Concrete Cover	t	=			38.10	mm
Concrete Cover for Tendons	tp	=			187.33	mm
Concrete Performance Factor	φc	=			0.75	
Steel Performance Factor	φs	=			0.90	
Prestress Strand P. Factor	φp	=			0.95	
Prestress Steel Type	Type	=			low	
	kp	=	IF(Type="low",0.3,IF(Type="re", 0.4,IF(Type="deformed",0.5)))	=	0.30	

Calculation

	α1	=	0.85-0.0015*fc'	=	0.80	
	β1	=	0.97-0.0025*fc'	=	0.88	
	ds	=	h-t-D/2	=	457.20	mm
	ds'	=	t+D'/2	=	38.10	mm
	dp	=	h-tp-Dp/2	=	290.51	mm
	de	=	IF(Ap>0,dp,ds)	=	290.51	
Value of c / dp	ratio	=	(φp*Ap*fpu+φs*As*fy-φs*As'*fy-α1*φc*fc'*hf*(b-bw))/(α1*φc*β1*fc'*bw*dp+φp*kp*Ap*fpu)	=	0.09	
	fps	=	fpu*(1-kp*ratio)	=	972.17	MPa
Neutral axis location	a	=	(φs*As*fy+φp*Ap*fps-φs*As'*fy-α1*φc*fc'*hf*(b-bw))/(α1*φc*fc'*bw)	=	23.60	mm
	M1	=	(φs*As*fy*(ds-a/2)+φp*Ap*fps*(dp-a/2)-φs*As'*fy*(ds'-a/2)-α1*φc*fc'*hf*(b-bw)*(hf/2-a/2))/1000000	=	135.77	KNm
	M2	=	(φs*As*fy*(ds-a/2)+φp*Ap*fps*(dp-a/2)-φs*As'*fy*(ds'-a/2))/1000000	=	135.77	
	M3	=	(0.3*α1*φc*fc'*bw*de^2+α1*φc*fc'*(b-bw)*hf*(de-hf/2)+φs*fy*As'*(de-ds'))/1000000	=	522.61	KNm
Moment Resistance	Mr	=	IF(ratio<0.5,IF(a<hf, M1,M2),M3)	=	135.77	KNm
		=		=		

Check

Applied Moment	Mf	=		=	879.00	KNm
Check	check	=	IF(Mf<Mr,"OK","NG")	=	NG	

Deck Longitudinal Moment Capacity @ Midspan

$M_r^+ = (939 \times \text{COS } 45^\circ + 136 \times \text{COS } 45^\circ) = 760$ kN.m

PROJECT		SECTION	Deck Over Pier
TITLE	Type A Cables	DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Box or I Section Under Bending

Description:This spreadsheet shows the design of Box or I Section Girder subjected to Flexure

Data Input

Basic Data

Top Flange Width	b	=			1000.00	mm
Web Width	bw	=			1000.00	mm
Top Flange Thickness	hf	=			0.00	mm
Total Height	h	=			908.05	mm
Concrete Strength	fc'	=			34.48	MPa
Steel Rebar Strength	fy	=			275.00	MPa
Prestress Steel Strength	fpu	=			1620.00	MPa
Bottom Rebar Diameter	D	=			0.00	mm
Top Rebar Diameter	D'	=			0.00	mm
Prestress Cable Diametre	Dp	=			15.00	
Bottom Rebar Area	As	=			0.00	mm2
Top Rebar Area	As'	=			0.00	mm2
Prestress Steel Area	Ap	=			2730.31	mm2
Concrete Cover	t	=			38.10	mm
Concrete Cover for Tendor	tp	=			92.08	mm
Concrete Performance Fac	ϕc	=			0.75	
Steel Performance Factor	ϕs	=			0.90	
Prestress Strand P. Factor	ϕp	=			0.95	
Prestress Steel Type	Type	=			low	
	kp	=	IF(Type="low",0.3,IF(Type="re", 0.4,IF(Type="deformed",0.5)))	=	0.30	

Calculation

	α1	=	0.85-0.0015*fc'	=	0.80	
	β1	=	0.97-0.0025*fc'	=	0.88	
	ds	=	h-t-D/2	=	869.95	mm
	ds'	=	t+D'/2	=	38.10	mm
	dp	=	h-tp-Dp/2	=	808.48	mm
	de	=	IF(Ap>0,dp,ds)	=	808.48	
Value of c / dp	ratio	=	(ϕp*Ap*fpu+ϕs*As*fy-ϕs*As'*fy-α1*ϕc*fc'*hf*(b-bw))/(α1*ϕc*β1*fc'*bw*dp+ϕp*kp*Ap*fpu)	=	0.26	
	fps	=	fpu*(1-kp*ratio)	=	1492.44	MPa
Neutral axis location	a	=	(ϕs*As*fy+ϕp*Ap*fps-ϕs*As'*fy-α1*ϕc*fc'*hf*(b-bw))/(α1*ϕc*fc'*bw)	=	187.55	mm
	M1	=	(ϕs*As*fy*(ds-a/2)+ϕp*Ap*fps*(dp-a/2)-ϕs*As'*fy*(ds'-a/2)-α1*ϕc*fc'*hf*(b-bw)*(hf/2-a/2))/1000000	=	2766.67	KNm
	M2	=	(ϕs*As*fy*(ds-a/2)+ϕp*Ap*fps*(dp-a/2)-ϕs*As'*fy*(ds'-a/2))/1000000	=	2766.67	
	M3	=	(0.3*α1*ϕc*fc'*bw*de^2+α1*ϕc*fc'*(b-bw)*hf*(de-hf/2)+ϕs*fy*As'*(de-ds'))/1000000	=	4047.43	KNm
Moment Resistance	Mr	=	IF(ratio<0.5,IF(a<hf, M1,M2),M3)	=	2766.67	KNm
		=		=		

Check

Applied Moment	Mf	=		=	2650.00	KNm
Check	check	=	IF(Mf<Mr,"OK","NG")	=	OK	

PROJECT		SECTION	Deck Over Pier
TITLE	Type B cables	DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Box or I Section Under Bending

Description:This spreadsheet shows the design of Box or I Section Girder subjected to Flexure

Data Input

Basic Data

Top Flange Width	b	=			1000.00	mm
Web Width	bw	=			1000.00	mm
Top Flange Thickness	hf	=			0.00	mm
Total Height	h	=			908.05	mm
Concrete Strength	fc'	=			34.48	MPa
Steel Rebar Strength	fy	=			275.00	MPa
Prestress Steel Strength	fpu	=			1620.00	MPa
Bottom Rebar Diameter	D	=			0.00	mm
Top Rebar Diameter	D'	=			0.00	mm
Prestress Cable Diametre	Dp	=			7.01	
Bottom Rebar Area	As	=			0.00	mm2
Top Rebar Area	As'	=			0.00	mm2
Prestress Steel Area	Ap	=			868.01	mm2
Concrete Cover	t	=			38.10	mm
Concrete Cover for Tendons	tp	=			260.35	mm
Concrete Performance Factor	φc	=			0.75	
Steel Performance Factor	φs	=			0.90	
Prestress Strand P. Factor	φp	=			0.95	
Prestress Steel Type	Type	=			low	
	kp	=	IF(Type="low",0.3,IF(Type="re", 0.4,IF(Type="deformed",0.5)))	=	0.30	

Calculation

	α1	=	0.85-0.0015*fc'	=	0.80	
	β1	=	0.97-0.0025*fc'	=	0.88	
	ds	=	h-t-D/2	=	869.95	mm
	ds'	=	t+D'/2	=	38.10	mm
	dp	=	h-tp-Dp/2	=	644.19	mm
	de	=	IF(Ap>0,dp,ds)	=	644.19	
Value of c / dp	ratio	=	(φp*Ap*fpu+φs*As*fy-φs*As'*fy-α1*φc*fc'*hf*(b-bw))/(α1*φc*β1*fc'*bw*dp+φp*kp*Ap*fpu)	=	0.11	
	fps	=	fpu*(1-kp*ratio)	=	1566.58	MPa
Neutral axis location	a	=	(φs*As*fy+φp*Ap*fps-φs*As'*fy-α1*φc*fc'*hf*(b-bw))/(α1*φc*fc'*bw)	=	62.59	mm
	M1	=	(φs*As*fy*(ds-a/2)+φp*Ap*fps*(dp-a/2)-φs*As'*fy*(ds'-a/2)-α1*φc*fc'*hf*(b-bw)*(hf/2-a/2))/1000000	=	791.75	KNm
	M2	=	(φs*As*fy*(ds-a/2)+φp*Ap*fps*(dp-a/2)-φs*As'*fy*(ds'-a/2))/1000000	=	791.75	
	M3	=	(0.3*α1*φc*fc'*bw*de^2+α1*φc*fc'*(b-bw)*hf*(de-hf/2)+φs*fy*As'*(de-ds'))/1000000	=	2569.69	KNm
Moment Resistance	Mr	=	IF(ratio<0.5,IF(a<hf, M1,M2),M3)	=	791.75	KNm
		=		=		

Check

Applied Moment	Mf	=		=	2650.00	KNm
Check	check	=	IF(Mf<Mr,"OK","NG")	=	NG	

Deck Longitudinal Moment Capacity @ Pier

$M_r^- = (2767 \times \mathrm{COS}\ 45^\circ + 792 \times \mathrm{COS}\ 45^\circ) = 2517$ kN.m

PROJECT	2140670	SECTION	Vr in deck at pier
TITLE		DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Shear Capacities

DESCRIPTION

This page is to caluclate shear resistance of PC girder.

Input

Basic Data

Applied Shear Force	Vf	=			900.0	KN		
Applied Axial Force	Nf	=			-1720.0	KN		
Appied Moment	Mf	=			2400.0	KNm		
Web Width	bv	=		=	1,000	mm		
Total Height	h	=		=	908	mm		
Concrete Strength	fc'	=		=	34.48	MPa		
Steel Rebar Strength	fy	=		=	275.00	MPa		
Prestress Steel Strength	fpu	=			1620.00	MPa		
Jacking force	fj	=		=	1296.00	MPa		
Elastic Modular of Rebar	Es	=			200000	MPa		
Elastic Modular of Prestress Steel	Ep	=			200000	MPa		
Longitudinal Rebar Area	As	=			0	mm2		
Transverse Rebar Area	Av1	=		=	0	mm2		
Inclinating Transverse Rebar Area	Av2	=			0	mm2		
Inclintinf Prestress Area	Ap	=			2730.31	(type A)	868	(type B)
Total Prestress Steel Area	Apt	=		=	2730.31	mm2	868	mm2
Prestress Tedon Diametre	dp	=			0.00	mm		
Transverse Rebar Diametre	dt	=			10			
cover	t	=			40			
Inclination of Rebar	α	=		=	0.78539816	rad		
Inclination of Prestress Steel	αα	=		=	0.083	rad (type A)	0.350	(type B)
Transverse Rebar Spacing	s	=			300	mm		
Rebar Vertical Spacing	sz	=		=	771.8	mm		
Concrete Performance Factor	φc	=		=	0.75			
Steel Performance Factor	φs	=		=	0.9			
Prestress Strand P. Factor	φp	=		=	0.95			

Calculations

Effective Prestress Stress	fse	=	from drawing	=	972.0	MPa
Concrete Crack Strength	fcr	=	0.4*SQRT(fc')	=	2.3	MPa
Depression Stress	fpo	=	0.7*fpu	=	1134.0	MPa
Effective Shear Depth	dv	=	IF(Apt=0, 0.9*(h-t-dt/2),0.72*h)	=	653.8	mm
Area of Tendon on Tension Side	Aps	=		=	2730.3	mm2
Resistance of Tendon	Vp	=	Ap*fse*SIN(αα)	=	509384.4	N
Shear Stress Ratio	ratio	=	(Vf*1000-φp*Vp)/(bv*dv*φc*fc')	=	0.0246	
Longitudinal Strain	εx	=	(0.5*Nf*1000+Vf*1000-Vp+Mf*1000000/dv-Aps*fpo)/(Es*As+Ep*Aps)/2	=	9.64E-05	
Clause 8.9.3.7	β	=	(0.4/(1+1500*εx))*(1300/(1000+0.85*sz))	=	0.274	
	θ	=	(29+7000*εx)*(0.88+0.85sz/2500)*pi()/180	=	0.592	
Resistance of Concrete	Vc	=	2.5*β*φc*fcr*bv*dv	=	789793.1	N
Resistance of Steel Rebar	Vs1	=	φs*fy*Av1*dv/TAN(θ)/s	=	0.0	N
	Vs2	=	φs*fy*Av2*dv*(ATAN(θ)+ATAN(α))*SIN(α)/s	=	0	N
Total Shear Resistance	Vr	=	(Vc+Vs1+Vs2+φp*Vp)/1000	=	1132.0	KN

Summary

check	check	=	IF(Vf<Vr,"OK","NG")	=	**OK**

PROJECT	2140670	SECTION	Vr in deck at pier
TITLE		DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Shear Capacities

DESCRIPTION

This page is to caluclate shear resistance of PC girder.

Input

Basic Data

Applied Shear Force	Vf	=			578.0	KN	
Applied Axial Force	Nf	=			-1720.0	KN	
Appied Moment	Mf	=			2400.0	KNm	
Web Width	bv	=		=	1,000	mm	
Total Height	h	=		=	879	mm	
Concrete Strength	fc'	=		=	34.48	MPa	
Steel Rebar Strength	fy	=		=	275.00	MPa	
Prestress Steel Strength	fpu	=			1620.00	MPa	
Jacking force	fj	=		=	0.00	MPa	
Elastic Modular of Rebar	Es	=			200000	MPa	
Elastic Modular of Prestress Steel	Ep	=			200000	MPa	
Longitudinal Rebar Area	As	=			0	mm2	
Transverse Rebar Area	Av1	=		=	0	mm2	
Inclinating Transverse Rebar Area	Av2	=			0	mm2	
Inclintinf Prestress Area	Ap	=			2730.31	(type A)	868 (type B)
Total Prestress Steel Area	Apt	=		=	2730.31	mm2	868 mm2
Prestress Tedon Diametre	dp	=			0.00	mm	
Transverse Rebar Diametre	dt	=			10		
cover	t	=			40		
Inclination of Rebar	α	=		=	0.785398163	rad	
Inclination of Prestress Steel	αα	=		=	0.083	rad (type A)	0.350 (type B)
Transverse Rebar Spacing	s	=			300	mm	
Rebar Vertical Spacing	sz	=		=	747.2	mm	
Concrete Performance Factor	φc	=		=	0.75		
Steel Performance Factor	φs	=		=	0.9		
Prestress Strand P. Factor	φp	=		=	0.95		

Calculations

Effective Prestress Stress	fse	=	from drawing	=	972.0	MPa
Concrete Crack Strength	fcr	=	0.4*SQRT(fc')	=	2.3	MPa
Depression Stress	fpo	=	0.7*fpu	=	1134.0	MPa
Effective Shear Depth	dv	=	IF(Apt=0, 0.9*(h-t-dt/2),0.72*h)	=	632.9	mm
Area of Tendon on Tension Side	Aps	=		=	2730.3	mm2
Resistance of Tendon	Vp	=	Ap*fse*SIN(αα)	=	509384.4	N
Shear Stress Ratio	ratio	=	(Vf*1000-φp*Vp)/(bv*dv*φc*fc')	=	0.0057	
Longitudinal Strain	εx	=	(0.5*Nf*1000+Vf*1000-Vp+Mf*1000000/dv-Aps*fpo)/(Es*As+Ep*Aps)/2	=	0.00E+00	
Clause 8.9.3.7	β	=	(0.4/(1+1500*εx))*(1300/(1000+0.85*sz))	=	0.318	
	θ	=	(29+7000*εx)*(0.88+0.85sz/2500)*pi()/180	=	0.574	
Resistance of Concrete	Vc	=	2.5*β*φc*fcr*bv*dv	=	886338.8	N
Resistance of Steel Rebar	Vs1	=	φs*fy*Av1*dv/TAN(θ)/s	=	0.0	N
	Vs2	=	φs*fy*Av2*dv*(ATAN(θ)+ATAN(α))*SIN(α)/s	=	0	N
Total Shear Resistance	Vr	=	(Vc+Vs1+Vs2+φp*Vp)/1000	=	1228.6	KN

Summary

check	check	=	IF(Vf<Vr,"OK","NG")	=	OK

PROJECT		SECTION	Pier top
TITLE		DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Box or I Section Under Bending

Description:This spreadsheet shows the design of Box or I Section Girder subjected to Flexure

Data Input

Basic Data

Top Flange Width	b	=			1000.00	mm
Web Width	bw	=			1000.00	mm
Top Flange Thickness	hf	=			0.00	mm
Total Height	h	=			700.00	mm
Concrete Strength	fc'	=			34.48	MPa
Steel Rebar Strength	fy	=			275.00	MPa
Prestress Steel Strength	fpu	=			1620.00	MPa
Bottom Rebar Diameter	D	=			57.00	mm
Top Rebar Diameter	D'	=			57.00	mm
Prestress Cable Diametre	Dp	=			15.00	
Bottom Rebar Area	As	=			9765.08	mm2
Top Rebar Area	As'	=			9765.08	mm2
Prestress Steel Area	Ap	=			0.00	mm2
Concrete Cover	t	=			50.80	mm
Concrete Cover for Tendons	tp	=			-115.98	mm
Concrete Performance Factor	φc	=			0.75	
Steel Performance Factor	φs	=			0.90	
Prestress Strand P. Factor	φp	=			0.95	
Prestress Steel Type	Type	=			low	
	kp	=	IF(Type="low",0.3,IF(Type="re", 0.4,IF(Type="deformed",0.5)))	=	0.30	

Calculation

	α1	=	0.85-0.0015*fc'	=	0.80	
	β1	=	0.97-0.0025*fc'	=	0.88	
	ds	=	h-t-D/2	=	620.70	mm
	ds'	=	t+D'/2	=	79.30	mm
	dp	=	h-tp-Dp/2	=	808.48	mm
	de	=	IF(Ap>0,dp,ds)	=	620.70	
Value of c / dp	ratio	=	(φp*Ap*fpu+φs*As*fy-φs*As'*fy-α1*φc*fc'*hf*(b-bw))/(α1*φc*β1*fc'*bw*dp+φp*kp*Ap*fpu)	=	0.00	
	fps	=	fpu*(1-kp*ratio)	=	1620.00	MPa
Neutral axis location	a	=	(φs*As*fy+φp*Ap*fps-φs*As'*fy-α1*φc*fc'*hf*(b-bw))/(α1*φc*fc'*bw)	=	0	mm
	M1	=	(φs*As*fy*(ds-a/2)+φp*Ap*fps*(dp-a/2)-φs*As'*fy*(ds'-a/2)-α1*φc*fc'*hf*(b-bw)*(hf/2-a/2))/1000000	=	1308.49	KNm
	M2	=	(φs*As*fy*(ds-a/2)+φp*Ap*fps*(dp-a/2)-φs*As'*fy*(ds'-a/2))/1000000	=	1308.49	
	M3	=	(0.3*α1*φc*fc'*bw*de^2+α1*φc*fc'*(b-bw)*hf*(de-hf/2)+φs*fy*As'*(de-ds'))/1000000	=	3694.15	KNm
Moment Resistance	Mr	=	IF(ratio<0.5,IF(a<hf, M1,M2),M3)	=	1308.49	KNm
		=		=		

Check

Applied Moment	Mf	=		=	950.00	KNm
Check	check	=	IF(Mf<Mr,"OK","NG")	=	OK	

PROJECT	2140670	SECTION	Vr @pier top
TITLE		DATE	1/23/2015
FILE	PC Design.xls	TIME	11:57 AM

Shear Capacities

DESCRIPTION

This page is to caluclate shear resistance of PC girder.

Input

Basic Data

Applied Shear Force	Vf	=			206.0	KN
Applied Axial Force	Nf	=			0.0	KN
Appied Moment	Mf	=			950.0	KNm
Web Width	bv	=		=	1,000	mm
Total Height	h	=		=	700	mm
Concrete Strength	fc'	=		=	34.48	MPa
Steel Rebar Strength	fy	=		=	275	MPa
Prestress Steel Strength	fpu	=			0	MPa
Jacking force	fj	=		=	0	MPa
Elastic Modular of Rebar	Es	=			200000	MPa
Elastic Modular of Prestress Steel	Ep	=			200000	MPa
Longitudinal Rebar Area	As	=			9765	mm2
Transverse Rebar Area	Av1	=		=	1806	mm2
Inclinating Transverse Rebar Area	Av2	=			0	mm2
Inclintinf Prestress Area	Ap	=			0	
Total Prestress Steel Area	Apt	=		=	0	mm2
Prestress Tedon Diametre	dp	=			0	mm
Transverse Rebar Diametre	dt	=			10	
cover	t	=			50.8	
Inclination of Rebar	α	=		=	0.78539816	rad
Inclination of Prestress Steel	αα	=		=	0.000	rad
Transverse Rebar Spacing	s	=			300	mm
Rebar Vertical Spacing	sz	=		=	780.0	mm
Concrete Performance Factor	φc	=		=	0.75	
Steel Performance Factor	φs	=		=	0.9	
Prestress Strand P. Factor	φp	=		=	0.95	

Calculations

Effective Prestress Stress	fse	=	from drawing	=	0.0	MPa
Concrete Crack Strength	fcr	=	0.4*SQRT(fc')	=	2.3	MPa
Depression Stress	fpo	=	0.7*fpu	=	0.0	MPa
Effective Shear Depth	dv	=	IF(Apt=0, 0.9*(h-t-dt/2),0.72*h)	=	579.8	mm
Area of Tendon on Tension Side	Aps	=		=	0.0	mm2
Resistance of Tendon	Vp	=	Ap*fse*SIN(αα)	=	0.0	N
Shear Stress Ratio	ratio	=	(Vf*1000-φp*Vp)/(bv*dv*φc*fc')	=	0.0137	
Longitudinal Strain	εx	=	(0.5*Nf*1000+Vf*1000-Vp+Mf*1000000/dv-Aps*fpo)/(Es*As+Ep*Aps)/2	=	4.72E-04	
Clause 8.9.3.7	β	=	(0.4/(1+1500*εx))*(1300/(1000+0.85*sz))	=	0.183	
	θ	=	(29+7000*εx)*(0.88+0.85sz/2500)*pi()/180	=	0.646	
Resistance of Concrete	Vc	=	2.5*β*φc*fcr*bv*dv	=	467316.8	N
Resistance of Steel Rebar	Vs1	=	φs*fy*Av1*dv/TAN(θ)/s	=	1146508.5	N
	Vs2	=	φs*fy*Av2*dv*(ATAN(θ)+ATAN(α))*SIN(α)/s	=	0	N
Total Shear Resistance	Vr	=	(Vc+Vs1+Vs2+φp*Vp)/1000	=	1613.8	KN

Summary

check check = IF(Vf<Vr,"OK","NG") = **OK**

APPENDIX E

Geophysical and Asphalt Coring Investigation Reports

Project: 10001116

January 20, 2015

Morrison Hershfield Limited
2440 Don Reid Drive
Ottawa, ON
K1H 1E1

Attention: Mr. Joe Ostrowski, P.Eng.

Re: Geophysical and Asphalt Coring Investigation – SJAM Parkway Bridge

Dear Mr. Ostrowski:

SPL Consultants Limited (SPL) was retained by Morrison Hershfield Limited (MH) complete asphalt and concrete coring as well as a geophysical survey at the Sir John A. MacDonald Parkway Bridge O/P Lebreton (SN016470).

This letter summarizes the work carried out, and provides comments related to the investigations completed for the project.

1.0 Project and Site Description

SN0164470 is a post-tensioned concrete frame bridge, constructed in 1967. A sag of up to 200 mm has been identified in the bridge, which has been attributed to possible sagging of the original falsework during construction. As part of the structural assessment of the bridge (being completed by MH) it is understood that because of this sag the location of the longitudinal post-tensioning cables (identified as Type "A" Cables in the original design drawing No. E-10 prepared in 1966) is uncertain, particularly in the portion of the deck where they are expected to be relatively close to the underside of the bridge.

In addition, deflections in the surface of the bridge deck may have been corrected through the placement of additional asphalt or concrete over the bridge deck, resulting additional dead load on the bridge.

2.0 Investigation Procedures

The investigations completed by SPL as part of this assignment included asphalt and concrete coring and a geophysical survey.

2.1 Asphalt and Concrete Coring

A total of fifteen cores were obtained, thirteen cores taken to determine the thickness of the asphalt and two cores were advanced through the full depth of the bridge deck. Cores were obtained using portable coring equipment, obtained on November 24, 2014 by SPL's field engineer and on January 15th and 16th 2015 by Capital Cutting and Coring Ltd. of Ottawa, ON. The locations of the cores, selected by Morrison Hershfield Ltd., as well as a summary of the findings and photographs of the cores are included as an attachment.

The asphalt and concrete thicknesses encountered at the coring locations are shown in Drawing No. 2.

2.2 Geophysical Survey

A geophysical survey of portions of the bridge was also completed as part of this assignment. The geophysical survey included a Ground Penetrating Radar (GPR) survey and was completed by Geophysics GPR International Inc. of Mississauga ON. A copy of the geophysical report including typical sections across the bridge is included as an attachment.

The scan was completed on the bridge deck as well as portions of the underside of the bridge (see Figures 1 and 2 in the attachment). Typical sections taken at various locations on the bridge are shown in Figures 3 through 6 of the attachment.

3.0 Discussion

The following discussion is provided related to the "as-found" SJAM bridge configuration is provided based on the asphalt and concrete coring and geophysical survey completed as part of this assignment.

3.1 Asphalt Thickness

Consideration had been given to the possibility that the "sag" in the bridge structure had been previously corrected by the placement of additional asphalt (in which case the asphalt would be thicker in the "sag" area than other areas).

The majority of the asphalt cores were obtained in the "sag" area near the east end of the bridge. The results of the asphalt coring in this area suggests the asphalt ranges from 125 mm to 170 mm (only CH14-9 encountered less asphalt at 95 mm). Asphalt cores taken outside this "sag" area (CH14-1, 14-2 and 14-3 encountered asphalt thicknesses of 90 mm to 95 mm.

The GPR survey carried out in the general area of the "sag" identified sections of asphalt ranging from 90 mm to 150 mm in one survey line (see Figure 3 of the attachment) and ranging from 110 mm to more than 200 mm with an average of about 160 mm in another (see Figure 5 of the attachment). Asphalt coring at this location (as well as the general area) showed the asphalt thickness to be between 145 m and 165 mm, suggesting the GPR results slightly over-estimate the thickness in this area.

Based on the results of the coring and GPR surveys it appears that the asphalt thickness over most of the bridge is 90 mm to 100 mm with a thicker section in the eastern and (125 mm to 170 mm) in the “sag” area.

3.2 Concrete Thickness

Two cores (CH14-8 and CH14-7, located on the eastbound and westbound side of the bridge, respectively) were advanced through the full depth of the bridge. At CH14-7 on the north side of the median, the concrete deck was found to be 410 mm thick (not including the asphalt above). At CH14-8 on the south side of the bridge, the concrete deck was found to be 510 mm thick.

Cold joints were not identified in the cores obtained at the two locations.

3.3 Longitudinal Post-Tensioning Cables

There are several sets of post-tensioning cables included in the original bridge design. These cables include one set of longitudinal cables (“Type A” cables) and three sets of transverse cables (Type B, C and D cables). It is understood that the position of the longitudinal Type A cables is particularly important to the structural assessment of the bridge.

In some areas the Type A cables appear to be too low in the deck profile (where the deck is thicker) to identify from the top of the deck. Towards the east, however, where the deck is thinner the cables can be seen in some of the GPR scans from the surface. Figure 3 of the geophysical survey results shows an example of a section where the Type A cables can be identified and appear to be at approximately the correct location.

Scans were repeated from the underside of the deck at four locations. Areas 3 and 4, in particular, are in areas where the Type A cables are supposed to be at their lowest point. At both locations the cables were identified with the correct horizontal spacing and depth of cover. Figure 6 of the geophysical report shows an example of one of the areas (Area 3). Similar results were obtained in Area 4. Note that in Figure 6 of the geophysical report the “top” of the profile is the underside of the deck and the “depth” is actually depth from the surface into the structure (so in this case it is upwards).

Based on the geophysical surveys, in the areas where the Type A cables should be near the underside of the deck they appear at a horizontal spacing of approximately 600 mm with a depth of cover of approximately 90 mm. These locations are consistent with the profile indicated in the design drawings (for example Sheet E-10 of the 1966 design drawings).

4.0 Closure

We trust that this report provides the information you require at this time. Should you have any questions, or require any further information please feel free to contact the undersigned at your convenience.

SPL CONSULTANTS LIMITED

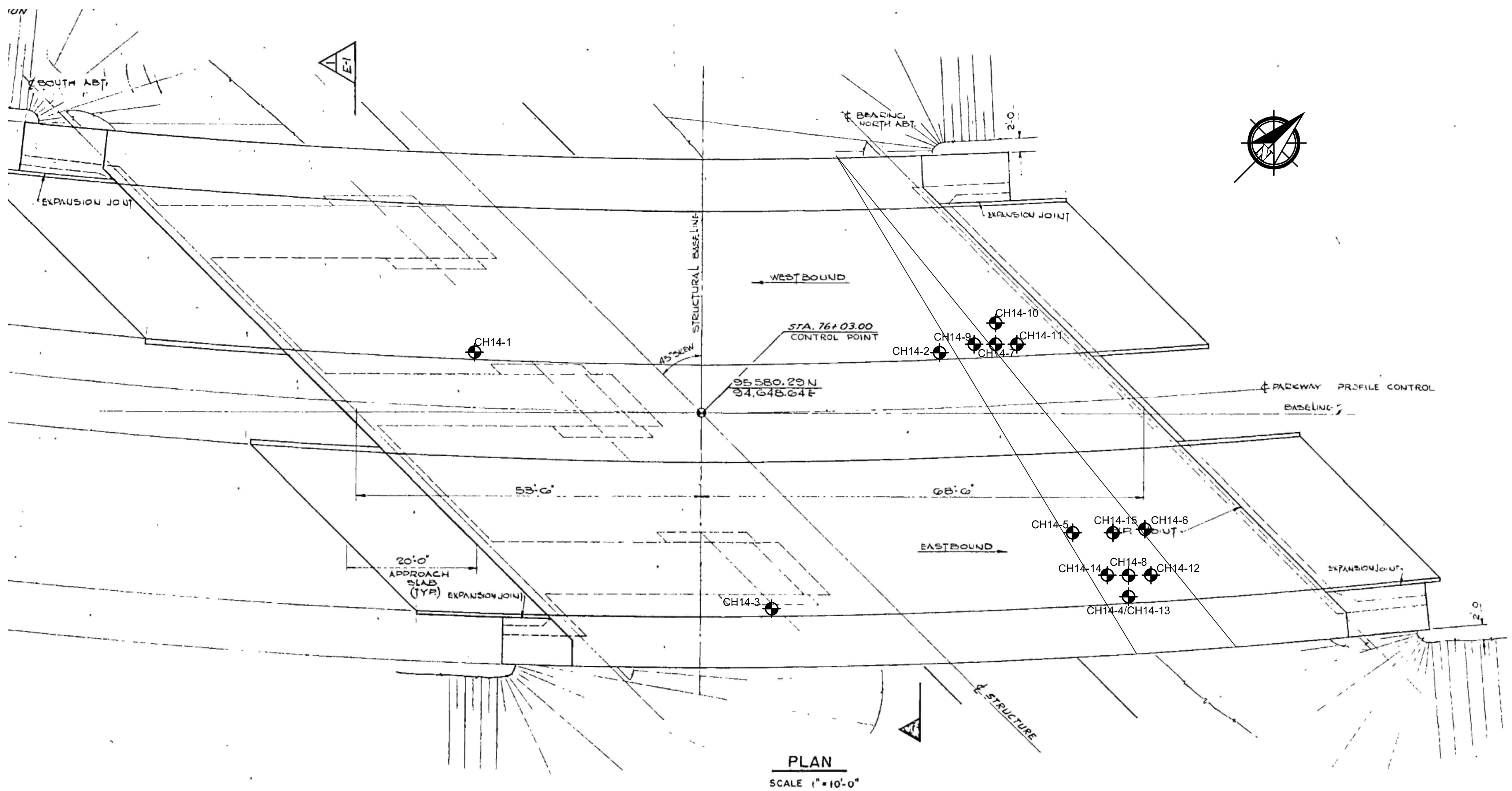
Chris Hendry, M.Eng., P.Eng.
Senior Geotechnical Engineer

Attachments:

Drawing No. 1 – Site Location Plan
Drawing No. 2 – Core Hole Location Plan and Results of Coring
Geophysical Survey Report
Limitations of This Report

DRAWINGS

Client:	Morrison Hershfield Limited			Title:	Site Location Plan
Project#:	10001116	DWG #:	1	Project:	Geotechnical Investigation Sir John A. MacDonald Parkway Bridge (SN016470)
Drawn:	DW	Approved:	CH		
Date:	November 2014	Scale:	N. T. S.		SPL Consultants Limited Geotechnical Environmental Materials Hydrogeology
Size:	Letter	Rev:	0		



PLAN

SCALE 1" = 10'-0"

Eastbound Lane				
CORE #	DISTANCE FROM EASTERN EXP. JOINT	DISTANCE FROM MEDIAN (EDGE/CURB)	ASPHALT	CONCRETE
4	9.3 m (W)	7.0 m (S)	118 mm	--
5	9.1 m (W)	3.8 m (S)	145 mm	--
6	5.5 m (W)	3.7 m (S)	165 mm	--
8	8.5 m (W)	5.9 m (S)	0 – 165 mm	165 mm – 675 mm
12	7.4 m (W)	5.9 m (S)	170 mm	--
13	9.4 m (W)	7.0 m (S)	170 mm	--
14	9.5 m (W)	6.0 m (S)	155 mm	--
15	7.2 m (W)	3.8 m (S)	160 mm	--

Eastbound Lane				
CORE #	DISTANCE FROM WESTERN EXP. JOINT	DISTANCE FROM MEDIAN (EDGE/CURB)	ASPHALT	CONCRETE
3	10.9 m (E)	6.9 m (S)	94 mm	--

Westbound Lane				
CORE #	DISTANCE FROM EXP. JOINT	DISTANCE FROM MEDIAN (EDGE/CURB)	ASPHALT	CONCRETE
2	7.0 m (W)	0.3 m (N)	92 mm	--
7	4.0 m (W)	0.6 m (N)	0 – 130 mm	130 mm – 540 mm
9	5.0 m (W)	0.6 m (N)	95 mm	--
10	3.0 m (W)	1.6 m (N)	125 mm	--
11	3.0 m (W)	0.6 m (N)	140 mm	--

Westbound Lane				
CORE #	DISTANCE FROM WESTERN EXP. JOINT	DISTANCE FROM MEDIAN (EDGE/CURB)	ASPHALT	CONCRETE
1	8.9 m (E)	0.3 m (N)	90 mm	--

Notes: Waterproofing membrane was encountered at the base of the asphalt layer in all cores with the exception of CH14–4

LEGEND

◆ Asphalt/Concrete Core

Client:	Morrison Hershfield Limited	Title:	Asphalt and Concrete Coring Location Plan
Project No.:	10001116	Drawing No.:	2
Drawn:	DW	Approved:	CH
Date:	January, 2015	Scale:	Not to Scale
Original	Tabloid	Rev:	0

Project: Geotechnical Investigation - Sir John A. MacDonald Parkway Bridge (SN016470)

SPL Consultants Limited
Geotechnical • Environmental • Materials • Hydrogeology

GEOPHYSICAL SURVEY REPORT

GEOPHYSICS GPR INTERNATIONAL INC.
6741 Columbus Road
Unit 14
Mississauga, Ontario
Canada L5T 2G9
Tel.: (905) 696-0656
Fax: (905) 696-0570
gprtor@gprtor.com
www.geophysicsgpr.com

December 15, 2014 Our File: T14674E

Chris Hendry, M.Eng. P.Eng.
Sr. Geotechnical Engineer
SPL Consultants Ltd.
146 Colonnade Road, Unit 17
Ottawa, Ontario
K2E 7Y1

RE: Mapping the structural features of a John A. MacDonald Pwky bridge.

Dear Mr. Hendry:

Geophysics GPR International Inc. was requested by SPL Consultants to perform a ground penetrating radar scan at the above locations. The purpose of the investigation was to verify the structural composition of the structure (see Figure 1). The survey was performed on November 24th and 25th 2014.

Georadar utilizes radar technology to obtain a near-continuous profile of the subsurface. The basic principle is to emit an electromagnetic impulse into the ground or concrete at a predetermined frequency rate (typically 10 to 80 scans/second). This pulse will travel through the sub-surface and reflect off boundaries of differing dielectric constants (contrasts of EM impedances). The reflected pulse returns to the surface and is recorded by a receiver and displayed in real-time as a cross-sectional image. Only by moving the antennas along a profile directly over the targets can the locations and depths be determined. Examples of radar reflecting boundaries included air/water (water table); water/earth (bathymetry); earth/metal, PVC, or concrete (pipe locating); and differing earth materials (stratigraphic profiles, including bedrock profiles).

There were two antenna sizes applied to the survey. The 400MHz antenna is a small to medium sized antenna that can generally obtain signal penetration depths in the order of 2 meters. The 1500MHz can obtain depths in the upper 0.5 meters. The 1500 MHz produced the only useful information.

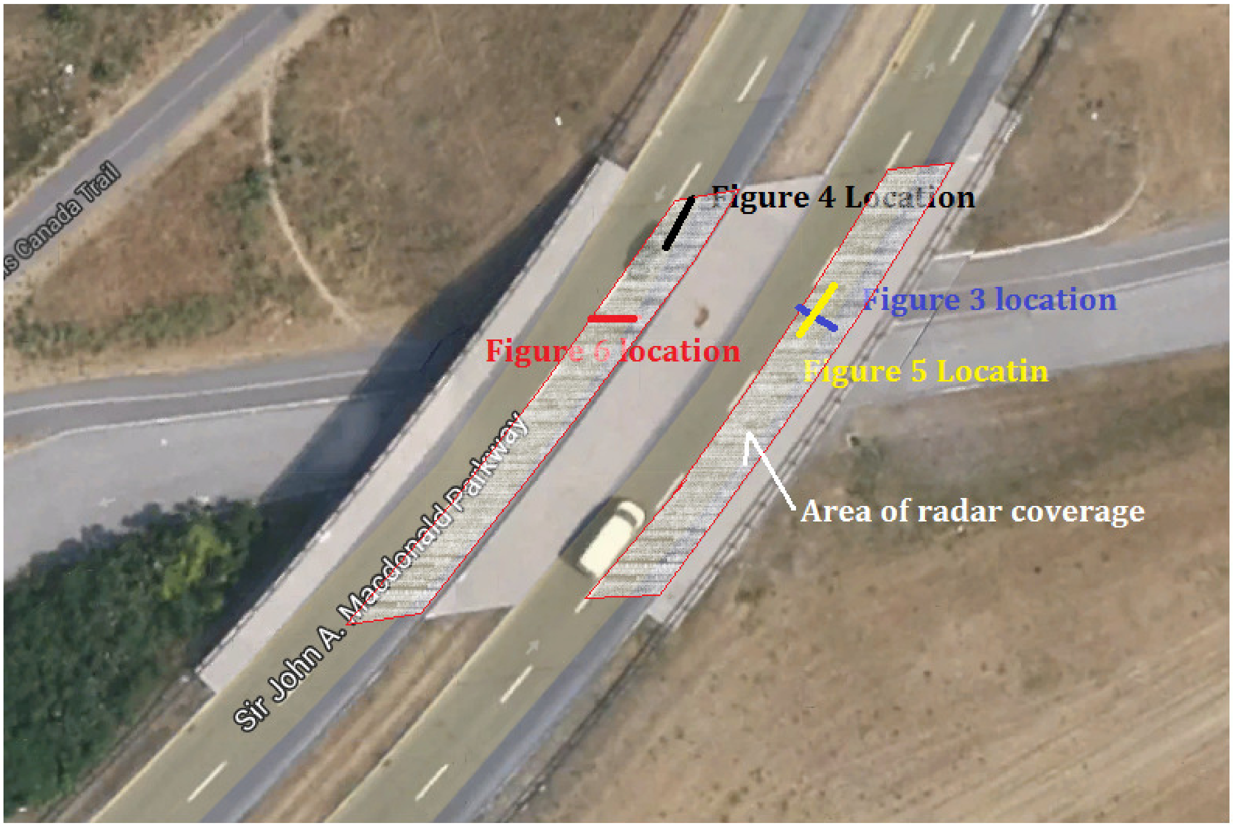


Figure 1: Site Location with Example Locations

Scans were performed on the deck surface and the underside with both antennas. The antenna with the most detail is the 1500MHz which high the highest detail in the upper 50 to 60 cm.

There is attached an example image in Figure 3 which shows several features. This example was collected perpendicular to the bridge over Lane 4 (Eastbound lane). The total length of the example is 3.5 meters

- Typcial asphalt thickness of 9 to 15cm. The average is closer to 12 cm.
- Type A cables that run the length of the bridge are very large in comparison to the reinforcing steel. This example was collected in the middle of the bridge so the depth is between 40 and 50 cm. The spacing between the Type A cables is between 60 and 70 cm (2 feet).

There is attached an example image in Figure 4 which shows transverse cables (likely Type C), asphalt and rebars. This example has been extracted from a one small portion of a profile collected parallel to Lane 2 (Westbound). There is some chainage distances relative to the eastern joint.

Figure 5 is a small portion of a longitudinal profile collected on Lane 4 (Eastbound). This shows some unusual asphalt thickness of 22cm. There is some chainage marks relative to the western joint. The asphalt starts at 11cm at the western end increases slowly to 22 and goes back up to 12cm. The average may be 16 to 17 cm for this lane. The Type C cables are indicated in blue.

Data was also collected at four locations using a cherry-picker lift on the underside of the bridge (Figure 2). Two of those locations were in the middle of the long span directly under the bike path. There was a scan under the westbound lanes and another under the eastbound lanes that produced identical results as shown in Figure 6. Figure 7 shows the regions where data was collected overlaying an image provided by the National Capital Commission. There is reinforcing steel that is only 2 or 3 cm deep and the Type A cables that are precisely 9cm deep or 3.5 inches.

I hope everything is to your satisfaction.

If you have any questions please do not hesitate to call.

Sincerely,

Milan Situm

Milan Situm, P.Geo.
Manager

PROFESSIONAL GEOSCIENTIST
MILAN SITUM
PRACTISING MEMBER
0237
ONTARIO

Figure 2: Scan Areas under the bridge

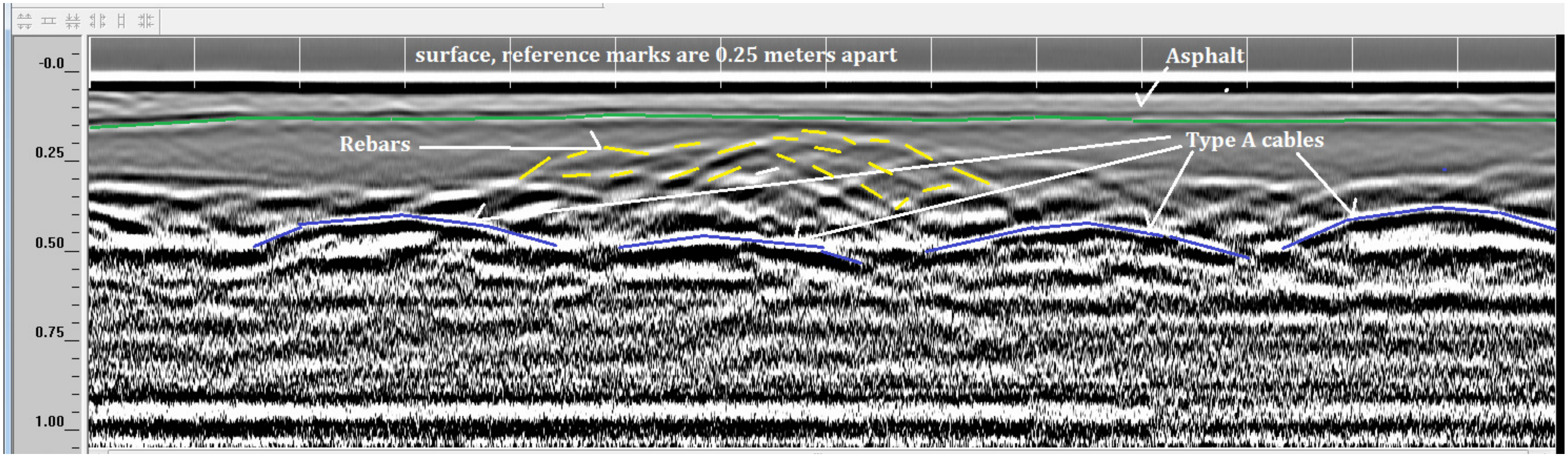


Figure 3: Example Radar image on the surface of the Sir John A. MacDonald Pkwy Bridge.

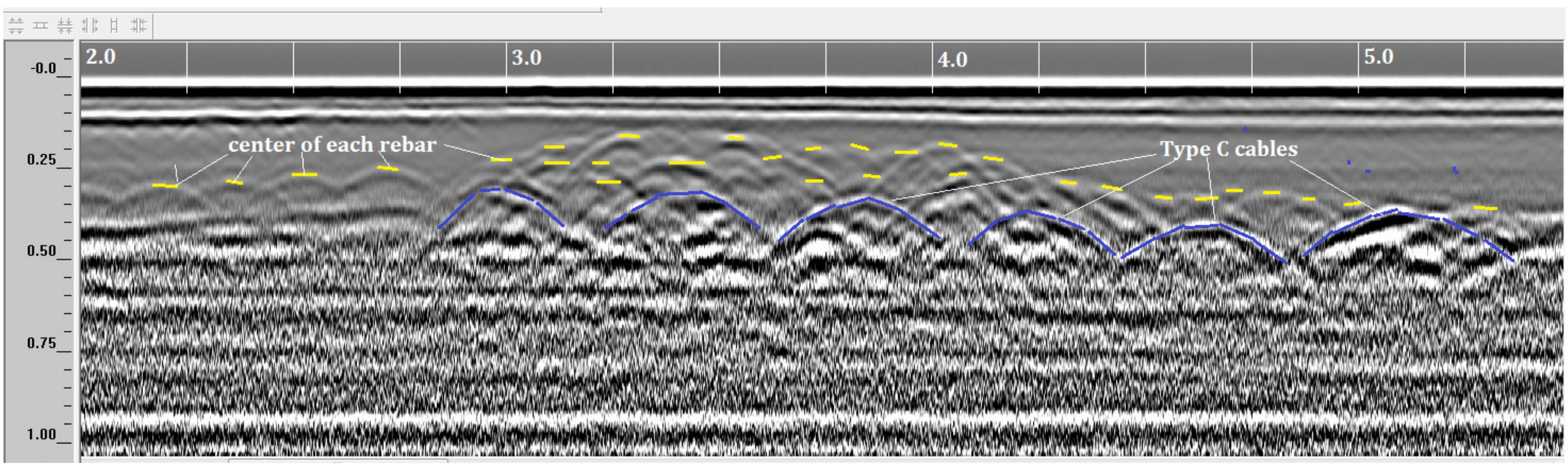


Figure 4: Example Image collected on one portion of Lane 2, highlighting Type C cables.

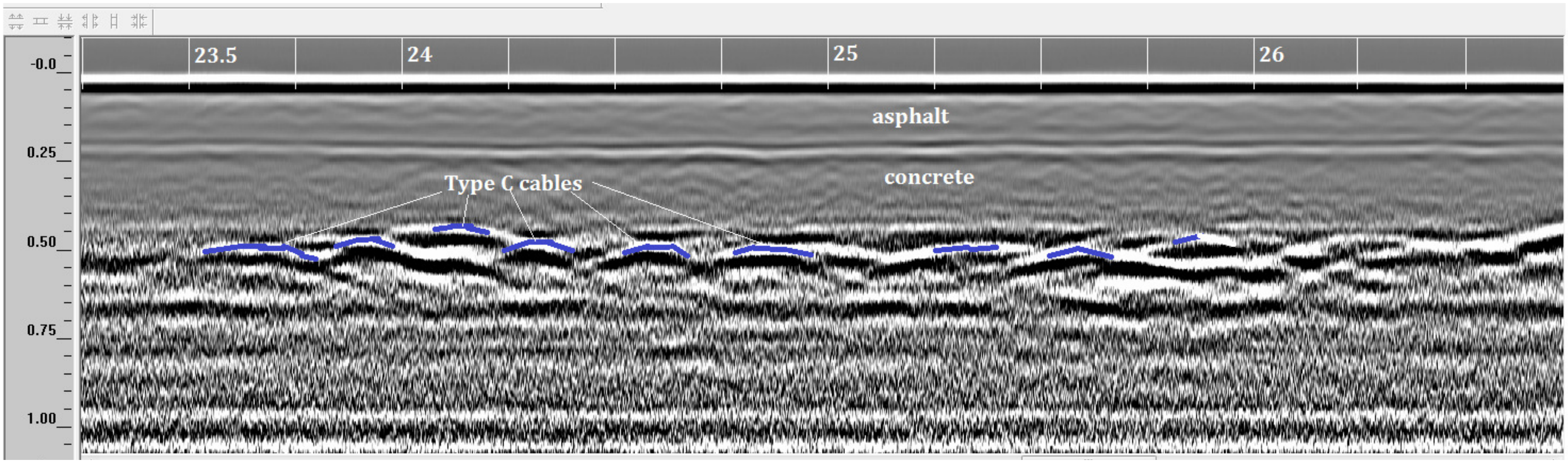


Figure 5: Example Image collected on one portion of Lane 4.

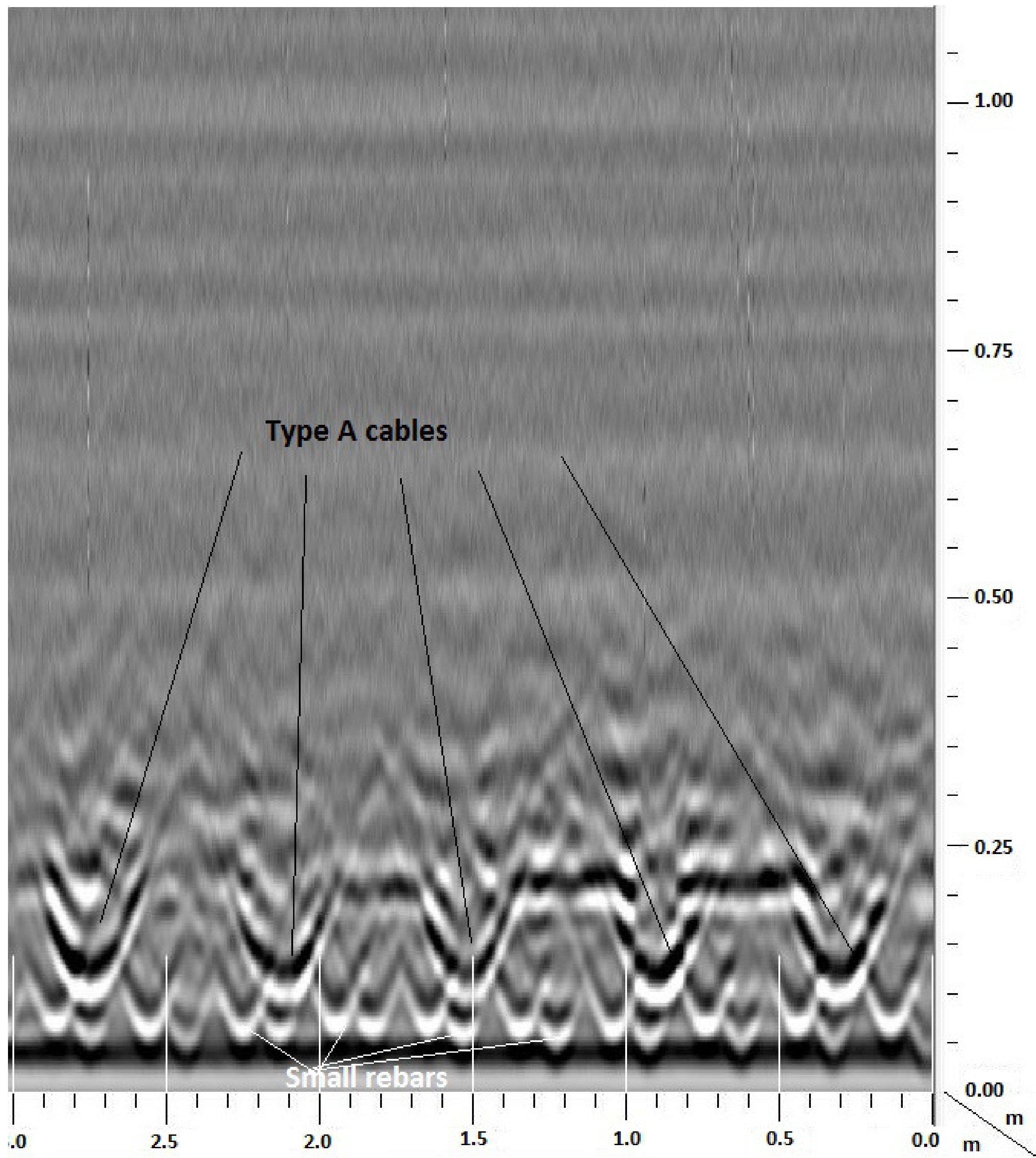


Figure 6: Example section collected under the westbound lanes. The Type A cables are precisely 9cm deep (3.5 inches) and 2 feet apart.

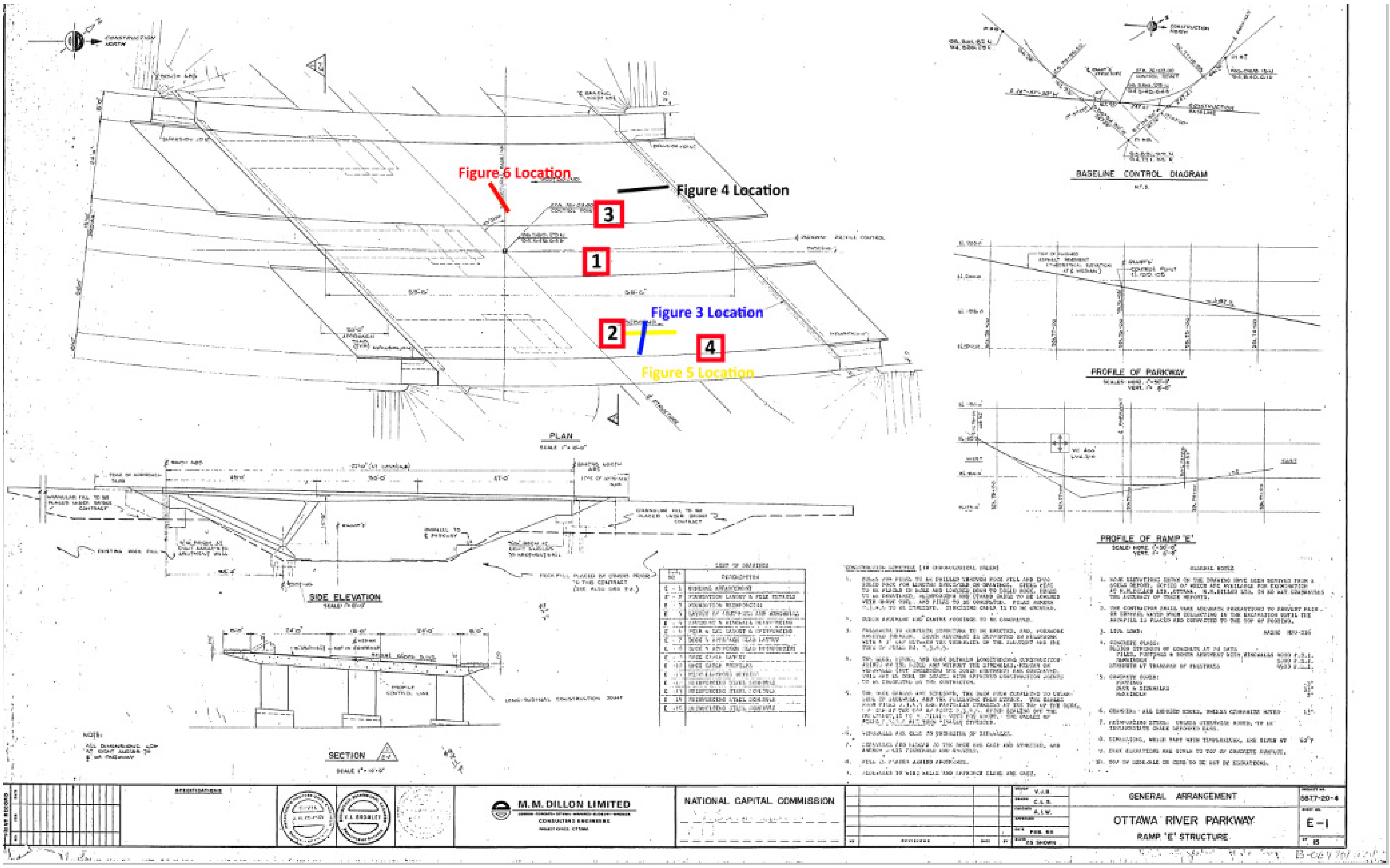


Figure 7: Data collected overlaying provided image

LIMITATIONS OF THIS REPORT

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to SPL Consultants Limited at the time of preparation. Unless otherwise agreed in writing by SPL Consultants Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. SPL Consultants Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.