APPENDIX D – STRUCTURAL REPORT

AECOM
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

IBI GROUP

SCARBOROUGH MALVERN LIGHT RAIL TRANSIT ENVIRONMENTAL ASSESSMENT – BRIDGE REVIEW

Draft

Prepared by:

AECOM Canada Ltd.
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

Date:

FEBRUARY 2009

Statement of Qualifications and Limitations

The attached Report (the "Report") has been prepared by Totten Sims Hubicki Associates (1997) Limited doing business as AECOM ("AECOM") for the benefit of Ministry of transportation, Central Region ("Client") in accordance with the agreement between AECOM and Client (the "Agreement").

The information, data, recommendations and conclusions contained in the Report:

- are subject to the budgetary, time and other constraints and limitations in the Agreement and the qualifications contained in the Report (the "Limitations")
- represent AECOM's professional judgement in light of the Limitations and industry standards for the preparation of similar reports
- may be based on information provided to AECOM which has not been independently verified
- have not been updated
- must be read as a whole and sections thereof should not be read out of such context
- were prepared for the specific purposes described in the Report and the Agreement and must not be used for any other purpose whatsoever

Unless expressly stated to the contrary in the Report or the Agreement, AECOM:

- shall not be responsible for any events or circumstances that may have occurred since the date on which the Report was prepared or for any inaccuracies contained in information that was provided to AECOM
- makes no guarantees or warranties whatsoever, whether express or implied, with respect to the Report or any part thereof, other than that the Report represents AECOM's professional judgement as described above
- shall not be deemed to have represented that the Report or any part thereof is exhaustive or applicable to any specific use other than that described in the Report and the Agreement

Except as required by law or otherwise agreed by AECOM and Client, the Report:

- is to be treated as confidential
- may not be used or relied upon by third parties

Except as described above, AECOM denies any liability in respect of the Report or parts thereof and shall not be responsible for any damages arising from use of the Report or parts thereof.

This Disclaimer is attached to and forms part of the Report.

THIS DOCUMENTATION IS SUPPLIED TO MTO-Central Region BY AECOM AND CONSTITUTES CONFIDENTIAL TRADE SECRETS, OR COMMERCIAL, FINANCIAL, SCIENTIFIC, OR TECHNICAL INFORMATION. THIS DOCUMENTATION IS SUBMITTED TO MTO-Central Region IN CONFIDENCE. IT HAS SIGNIFICANT ECONOMIC VALUE TO AECOM AND ITS DISCLOSURE, WITHOUT THE EXPRESS CONSENT OF AECOM, COULD REASONABLY BE EXPECTED TO LEAD TO SIGNIFICANT AND UNDUE FINANCIAL AND OTHER HARM TO AECOM, INCLUDING HARM TO AECOM'S COMPETITIVE AND NEGOTIATING POSITIONS, AND UNDUE FINANCIAL GAIN TO ONE OR MORE THIRD PARTIES.

AECOM
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

February 18, 2009 Project Number: 109344

Mr. Harold Sich
Associate
IBI Group
230 Richmond Street, 5th Floor
Toronto, Ontario
M5V 1V6

Dear Harold:

Re: SCARBOROUGH MALVERT LIGHT RAPID TRANSIT ENVIRONMENTAL ASSESSMENT – BRIDGE REVIEW

We are enclosing herewith two (2) copies of our Bridge Review report as noted above.

Please advise if we could be of further assistance in the above regards.

Sincerely,
Totten Sims Hubicki Associates (1997) Limited doing business as AECOM

David LeBlanc, M.Eng., P.Eng.
Head, Structures Department
david.leblanc@aecom.com

DL:smb
Encl.
cc: File

Executive Summary

AECOM was retained by IBI Group to investigate and confirm the feasibility of implementing a Light Rapid Transit (LRT) right-of-way (ROW) on the existing bridges located along the preferred alignment for Scarborough Malvern line, specifically addressing the structural adequacy of the structure, as well as long term maintenance and operational requirements. The intent is upon confirmation of the feasibility of the LRT ROW implementation on the structure, to obtain approval from federal, municipal and railway authorities during the environmental assessment phase in order to move forward with the project. It is recognized that that there are various design and contractual arrangements to be addressed in the subsequent project phases, and the TTC is committed to working with the authorities on these issues.

The preferred alignment is from Eglinton-Kennedy Station to Kingston Road; Kingston Road-Eglinton Avenue to Morningside Avenue; Morningside Avenue-Kingston Road to Sheppard Avenue; Neilson Road-Sheppard Avenue to Malvern Town. This alignment involves 4 overpass structures and one Subway structure as listed below:

- Eglinton Avenue – CNR Overpass
- Eglinton Avenue East – CNR Subway
- Kingston Road – CNR Overpass
- Morningside Avenue over Highland Creek
- Morningside Avenue over Highway 401

An assessment of the existing overpass structures have been carried out to determine if it can accommodate the proposed Scarborough - Malvern LRT designated ROW, including two lanes of traffic in each direction. The findings indicate that the new LRT ROW and two traffic lanes can be accommodated on all the existing structures without a need for deck widening, with the exception of the Morningside Avenue over Highland creek structure, which will require widening or a new structure

A detailed structural evaluation for the Highway 401 – Morningside Avenue Underpass structure was prepared for review by the Ministry of Transportation, Ontario as they have jurisdiction over this structure. The results of the evaluation indicate that it is feasible to accommodate the proposed LRT ROW on the structure, without a need for deck widening. The girders could be strengthened to accommodate the additional load from a conventional concrete track bed, or alternatively a light weight track bed could be considered for the LRT.

A detailed structural evaluation was also undertaken to investigate effects of additional loads due to LRT and its accessories for Eglinton Avenue – CNR Overpass indicate that it is feasible to accommodate the proposed LRT right-of-way on the Eglinton Avenue – CNR Overhead structure, without a need for deck widening. The structure has adequate capacity to accommodate LRT loads in conjunction with the use of a light weight material for the track bed.

Summary of General Structural Findings

1. Impact of LRT loading and geometry

In general the overpass structures have sufficient deck width to accommodate the proposed LRT tracks with little modification, with the exception of the Morningside Avenue bridge over Highland Creek as discussed below. The weight of the proposed LRT vehicle is slightly less than standard CHBDC vehicle loading, and the existing bridges will have adequate capacity to support the vehicular load due to the LRT vehicle. Strengthening of the bridges may however be required due to additional loads from the trackwork, overhead poles, rail breakage forces, and other items required to accommodate the LRT trackwork.

2. Track Bed Infill for Overpass Structures

   The additional surcharge due to a concrete infill slab for the LRT track may necessitate strengthening of the existing structures. The increase in moment due to superimposed track bed dead load and LRT live load over the CL625-Ont live load ranges from 40 to 70% and the increase in shear force ranges from 16 to 50%. It may be feasible to use a light weight polymer infill for the trackbed, which may reduce or eliminate the need to strengthen the bridges. Another alternative would be to fix the rails directly to the concrete deck.

3. Expansion joints at structure locations

   Expansion joints will preferable be located at the two ends of the structure, providing the grade at the joint location is generally flat. The effects of structure movement on the continuous welded rail, and rail breakage effects, will need to be accounted for during the detailed design process.

4. Longitudinal slope of road way

   The maximum allowable slope permitted for the new LRT vehicle is 5%. The longitudinal slope of 5.2% at Eglinton Avenue Overhead – CNR marginally exceeds this limit.

5. Eglinton Avenue - CNR Subway (at Bellamy)

   The available vertical clearance at this structure is 4.65 m, less than the preferred vertical clearance of 4.7 m. The reduced vertical clearance will require TTC approval. Alternatively, lowering of the track bed at the structure location could be considered, however it is not a preferred alternative due to proximity of the footing to the top of the road. The additional surcharge due to live load and impact effects on the existing footings will need to be further reviewed during future studies.

6. Morningside Avenue over Highway 401

A detailed structural evaluation for the Highway 401 – Morningside Avenue Underpass structure was prepared for review by the Ministry of Transportation, Ontario as they have jurisdiction over this structure. The results of the evaluation indicate that it is feasible to accommodate the proposed LRT ROW on the structure, without a need for deck widening. The girders could be strengthened to accommodate the additional load from a conventional concrete track bed, or alternatively a light weight track bed could be considered for the LRT.

5. Eglinton Avenue - CNR Overhead (at Kennedy)

A detailed structural evaluation was also undertaken to investigate effects of additional loads due to LRT and its accessories for Eglinton Avenue – CNR Overpass indicate that it is feasible to accommodate the proposed LRT right-of-way on the Eglinton Avenue – CNR Overhead structure, without a need for deck widening. The structure has adequate capacity to accommodate LRT loads in conjunction with the use of a light weight material for the track bed.

Table of Contents

List of Figures

Error! No table of figures entries found.

Appendices

1. INTRODUCTION

AECOM was retained by IBI Group to investigate and confirm the feasibility of implementing a Light Rapid Transit (LRT) right-of-way (ROW) on the existing bridges located along the preferred alignment for Scarborough Malvern line, specifically addressing the structural adequacy of the structure, as well as long term maintenance and operational requirements. The intent is upon confirmation of the feasibility of the LRT ROW implementation on the structure, to obtain approval from federal, municipal and railway authorities during the environmental assessment phase in order to move forward with the project. It is recognized that that there are various design and contractual arrangements to be addressed in the subsequent project phases, and the TTC is committed to working with the authorities on these issues.

The preferred alignment (see Figure – 1) is from Eglinton-Kennedy Station to Kingston Road; Kingston Road-Eglinton Avenue to Morningside Avenue; Morningside Avenue-Kingston Road to Sheppard Avenue; Neilson Road-Sheppard Avenue to Malvern Town. This alignment involves 4 overpass structures and one Subway structure as listed below:

- Eglinton Avenue – CNR Overpass
- Eglinton Avenue East – CNR Subway
- Kingston Road – CNR Overpass
- Morningside Avenue over Highland Creek
- Morningside Avenue over Highway 401

Figure 1 – Preferred Alignment and Structure Locations

2. EXISTING CONDITIONS

The General Arrangement drawing of the existing structures are presented in Appendix A.

A site visit was undertaken by AECOM to confirm the existing structural arrangements and photographs were taken which are presented in Appendix B for records.

List of photographs:

<u>Eglinton Avenue East – CNR subway (at Bellamy)</u>

Picture 1: East Elevation
Picture 2: West Elevation
Picture 3: East Approach
Picture 4: West approach

<u>Kingston Road – CNR Overhead</u>

Picture 5: Looking North at the structure
Picture 6: South approach
Picture 7: Looking South at the structure
Picture 8: North approach

<u>Morningside Avenue over Highland Creek</u>

Picture 9: East Elevation
Picture 10: West Elevation
Picture 11: Looking South at structure
Picture 12: Northapproach
Picture 13: Looking North at structure
Picture 14: Southapproach

<u>Morningside Avenue over Highway 401</u>

Picture 15: East Elevation
Picture 16: West Elevation
Picture 17: Looking South at structure
Picture 18: Northapproach
Picture 19: Looking North at structure
Picture 20: Southapproach

3. STRUCTURE GEOMETRY

The proposed general arrangement drawings are presented in Appendix C and are summarized in Table 1 (on following page)

1. Eglinton Avenue – CNR Overhead (Uxbridge Subdivision, Mile. 59.40)

- Width of the bridge deck from gutter to gutter is 24.383m, including a 1.220m median, which is a part of the bridge deck structure.
- There are 3 lanes in each direction.
- The maximum longitudinal slope of the bridge is 5.2%, is more than assumed maximum slope of 5 % for the new LRT Vehicle.

The existing bridge can accommodate the required horizontal clearance for the 2 lanes of traffic eachway and the new LRT designated right-of-way configuration without widening.

2. Eglinton Avenue - CNR Subway at Bellamy (Oshawa Subdivision, Mile. 323.18)

- The width of the roadway under the bridge from gutter to gutter is approximately 31.700m, including a 2.438m median. There is 1.524m wide pier column located within the median.
- There are 3 lanes in each direction.
- The maximum longitudinal slope of Eglinton Avenue below the bridge structure is 5%, which satisfies the assumed maximum slope of 5% for the new LRT Vehicle.

The existing bridge can accommodate the required horizontal clearance for the 2 lanes of traffic each way and the new LRT designated right-of-way configuration.

3. Kingston Road – CNR Overhead (Kingston Subdivision, Mile. 321.45)

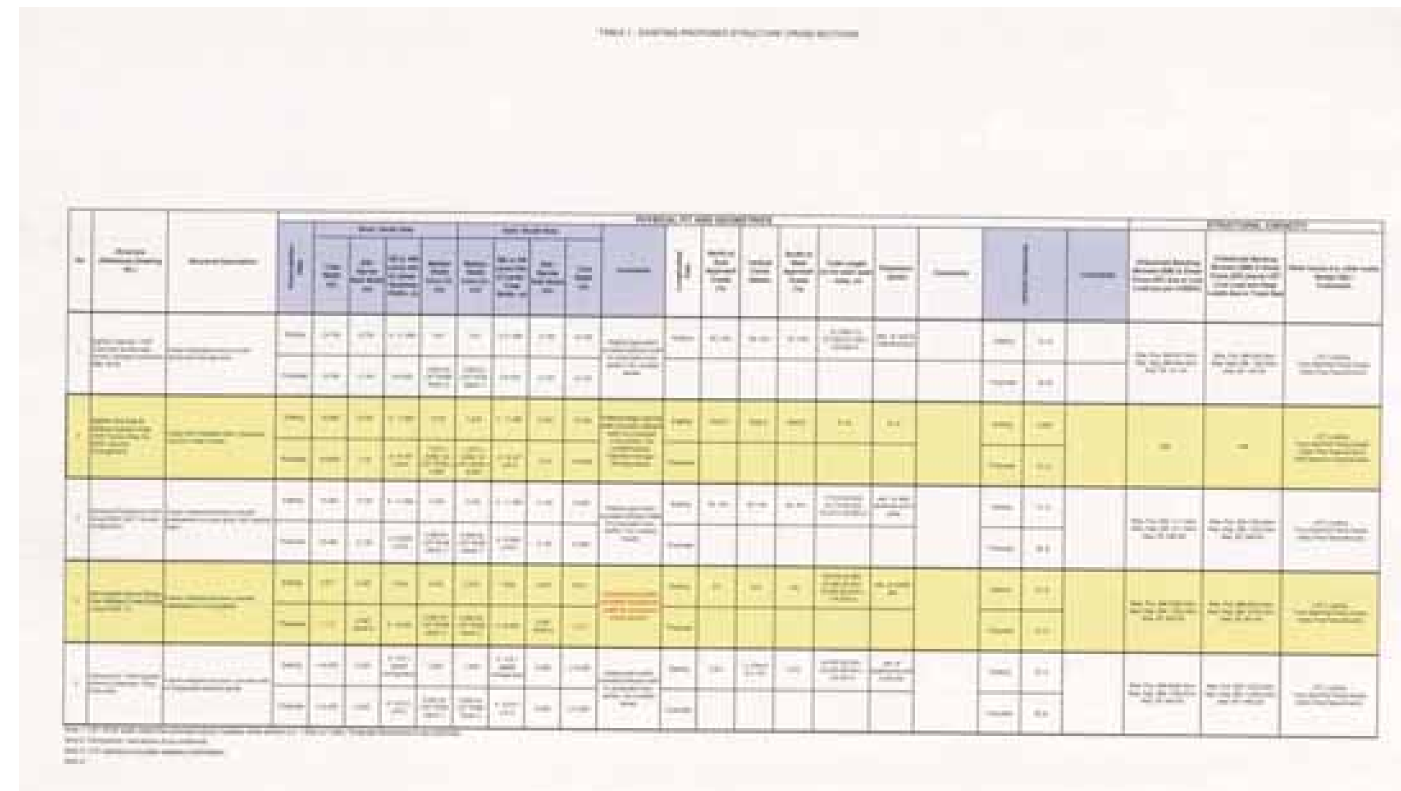
- Width of the bridge deck from gutter to gutter is 24.690m including a 1.530m median.
- There are 3 lanes in each direction.
- The maximum longitudinal slope of the bridge is 4.9%, which satisfies the assumed maximum slope of 5% for the new LRT Vehicle.

The existing bridge can accommodate the required horizontal clearance for the 2 lanes of traffic eachway and the new LRT designated right-of-way configuration.

4. Morningside Avenue over Highland Creek

- Width of the bridge deck from gutter to gutter is 15.240m. There is no median.
- There are 2 lanes in each direction.
- The maximum longitudinal slope of the bridge is 5%, which satisfies the assumed maximum slope of 5% for the new LRT Vehicle.

The existing bridge cannot accommodate the required horizontal clearance for the 2 lanes of traffic each way and the new LRT designated right-of-way configuration. The structure is needed to be widened and a east side widening is preferred for as the west side has more environmental constraints



The existing structure is a 6 span 173.75m (approx.) long structure constructed in 1963. The superstructure consists of 19.508 wide cast-in-place (CIP) reinforced concrete deck composite with with 9 precast prestressed concrete girder (CPCI type IV). The superstructure is supported on conventional CIP reinforced concrete piers and abutments. Structure is provided with expansion joints at Pier 3 and North abutment. At present the existing structure carries 2 lanes of north bound traffic and 2 lanes of south bound traffic without median. Raised concrete sidewalks are provided on both the east and west side of the structure.

For the structure widening following 5 alternatives were considered:

- Option 1A LRT right of way (ROW) at the median with east side structure widening.
- Option 1B LRT ROW at the median with east side structure widening and with bicycle lanes
- Option 2A LRT ROW on the east side with east side structure widening
- Option 2B LRT ROW on the east side with east side structure widening and with bicyle lanes
- Option 2C Separate dedicated structure for the LRT ROW and structure widening to accommodate the bicycle lanes.

Due to the environmental concerns on the west side widening on this side was not considered.

In our cost comparisons we have not included the cost for track bed construction as it is common to all options. The differences in the cost of approach road works are to be taken in to consideration while deciding on the final choice of the structure.

The General Arrangement drawing for the various options are provided in Appendix C

OPTION 1A: LRT ROW at median and with bicycle lanes

For this alternative, the existing structures would be widened by 7.492m to accommodate the proposed LRT Right of way at the median. This will require additional girders and widening of superstructure, substructure and foundations to support the widened superstructure. Structural system will be similar to the existing structure.

Construction will be carried out in 2 stages:

- Stage 1: Construct the structure widening while maintaining 2 lanes of traffic in both north and south direction on the original structure.
- Stage 2: Divert the north bound traffic to the new widened structure and construct LRT track bed.

This will involve extensive traffic staging works and roadway protection works for the construction of abutments.

The estimated cost for this alternative is $5.40 million.

OPTION 1B: LRT ROW at median and without bicycle lanes

This alternative is similar to Option 1A except for reduced area of deck widening due to the elimination of bicycle lanes.

The estimated cost for this alternative is $4.55 million.

OPTION 2A: LRT ROW on the east side and with bicycle lanes

For this alternative, the existing structures would be widened by 11.312m to accommodate the proposed LRT Right of way on the east side. This will require additional girders and widening of superstructure, substructure and foundations to support the widened superstructure. Structural system will be similar to the existing structure.

Construction will be carried out in a single stage, while maintaining 2 lanes of traffic in both north and south direction on the original structure.

This will involve minimum traffic staging works and roadway protection works for the construction of abutments.

The estimated cost for this alternative is $6.25 million.

OPTION 2B: LRT ROW on the east side and without bicycle lanes

This alternative is similar to Option 2A except for reduced area of deck widening due to the elimination of bicycle lanes.

The cost estimate for this alternative is $5.40 million.

OPTION 2C: LRT ROW on the east side on separate structure and with addition of bicycle lanes on the original structure

The new structure in this option can be constructed without affecting the existing traffic conditions significantly. Minimal traffic staging works and roadway protection works will be required during construction for widening the deck due to addition of bicycle lanes.

The cost estimate for this alternative is $6.45 million.

Recommendations

The least cost alternative is Option 1B, with a value of $4.55 million, which consists of widening the structure on east side with the LRT ROW in the median. This alternative, however does not allow for bicycle lanes.

If the bicycle lanes are required the least cost alternative will be Option 1A, which consists of widening the structure on east side with the LRT ROW in the median.

5. Morningside Avenue over Highway 401

- The width of the roadway under the bridge from gutter to gutter is approximately 31.000m There is no median.
- There are 2 lanes in each direction and also 1 South bound merging lane.
- The maximum longitudinal slope of the bridge structure is 3.5%, which satisfies the assumed maximum slope of 5% for the new LRT Vehicle.

The existing bridge can accommodate the required horizontal clearance for the 2 lanes of traffic eachway and the new LRT designated right-of-way configuration.

4. STRUCTURAL ASSESSMENT

A structural assessment has been carried out to determine if the existing bridges can accommodate the proposed LRT loading.

The design loads that the existing structure has been evaluated include the following:

Dead Loads:
The dead loads due to deck, sidewalk, parapet walls with handrails, asphalt wearing surface, and light poles.

Live Loads:
The original design live loads were based on AASHTO HS 25 load. While investigating the structure for the suitability of carrying the LRT vehicle, we have also investigated for the requirements of the current Canadian Highway Bridge Design Code (CHBDC) CAN/CSA – S6-06 and CL-625-ONT Truck load of 625 kN.

Other Loads:
Other loads that need to be considered in the design of the structure include secondary loads due to post-tensioning, thermal, wind, braking etc. as specified in the code.

A structural assessment of the existing bridges has been carried for the following load conditions:

- AASHTO HS25 Truck (where original structure has been designed for this load)
- CHBDC CAN/CSA - S6-06 CL-625-ONT Truck
- Proposed LRT Live load and additional loads due to conventional trackbed & accessories
- Proposed LRT Live load and additional loads due to lightweight trackbed & accessories

The results of the structural evaluation are summarized in Table 2 (at the end of this Section). The table indicates the increase in bending moment and shear forces due to LRT and track bed loads over the current design live loads under service load limit states (SLS). The loads acting on substructure and foundation are expected to

increase in the range of 5 to 10%, similar to increase in support reactions/ shear forces in the deck, if a conventional concrete trackbed is adopted. It is unlikely that strengthening of the foundations will be required for this additional load, however, underpinning methods are available to strengthen the capacity of existing abutment and pier footings, if necessary.

The results of the structural evaluation indicates that if a light-weight polymer infill with a unit weight in the order of 2 to 4 kN/m3 is provided for the track bed , strengthening of superstructure and substructure will not be required. It should be noted that, the TTC are investigating this technique for several bridges in the City of Toronto for the Transit City program.

A further option which could be considered would be to fix the rails directly to the concrete deck and eliminate the trackbed, in which case the structure is subjected to loads similar to that of the existing structure. However there are numerous maintenance and durability issues associated with fixing the rails directly to the deck, which could compromise the long term life of the structure, and this alternative is not recommended for further consideration.

Project : SCARBOROUGH MALVERN LRT EA

Project No. : 42-71256
Date : August 6, 2008

Table 2:Bridge Loading Analysis

	Max. Design Forces due to				Min. Design Forces due to			
	CL-625-ONT TRUCK	CL-625-ONT Lane	LRT	Track Bed (Add. DL)	CL-625-ONT TRUCK	CL-625-ONT Lane	LRT	Track Bed (Add. DL)
Structure 1	Eglinton Avenue East - CNR Overpass							
SLS BM (in kNm)	546.48	474.83	393.73	204.83	-425.92	-443.35	-368.96	-376.89
SLS SF (in kN)	131.89	116.30	110.86	77.29	-156.32	-129.61	-120.64	-77.07
Moment Ratio [1]	1.10				1.69			
Shear Ratio [2]	1.43				1.32			
Structure 3	Kingston Road - CNR Overpass							
SLS BM (in kNm)	1110.74	929.47	732.08	300.43	-810.74	-856.05	-824.77	-468.11
SLS SF (in kN)	335.94	283.53	267.54	125.11	-368.62	-302.73	-272.33	-125.84
Moment Ratio [1]	0.95				1.42			
Shear Ratio [2]	1.16				1.08			
Structure 4	Morningside Avenue Bridge over Highland Creek							
SLS BM (in kNm)	2026.22	1744.26	1460.72	777.41	-1294.43	-1322.65	-1106.43	-1042.35
SLS SF (in kN)	380.47	338.89	339.79	197.34	-381.58	-324.01	-308.94	-171.29
Moment Ratio [1]	1.10				1.62			
Shear Ratio [2]	1.41				1.26			
Structure 5	Morningside Avenue over Highway 401							
SLS BM (in kNm)	8469.00	8139.89	7327.99	3214.13	-6177.77	-7398.37	-5705.50	-4860.42
SLS SF (in kN)	967.80	980.00	914.69	551.03	-990.28	-993.62	-924.71	-536.37
Moment Ratio [1]	1.24				1.43			
Shear Ratio [2]	1.50				1.47			

Note :
1 . Moment Ratio = Design Moment (BM) due to LRT+Track Bed loading / Design Moment (BM) due to CL-625-ONT loading
2 . Shear Ratio = Design Shear Force (SF) due to LRT+Track Bed loading / Design Shear Force (SF) due to CL-625-ONT loading

Design Force Comparision 06AUG08.xls

printed 20/02/2009

Table 2

5. MISCELLANEOUS STRUCTURAL DETAILS

There are a number of details associated with the LRT ROW which will require modification of the existing structure, and which will need to be detailed during the design phases of the project. A preliminary assessment of the impact of the LRT ROW on the structure has been carried out, and the following items will need to be addressed:

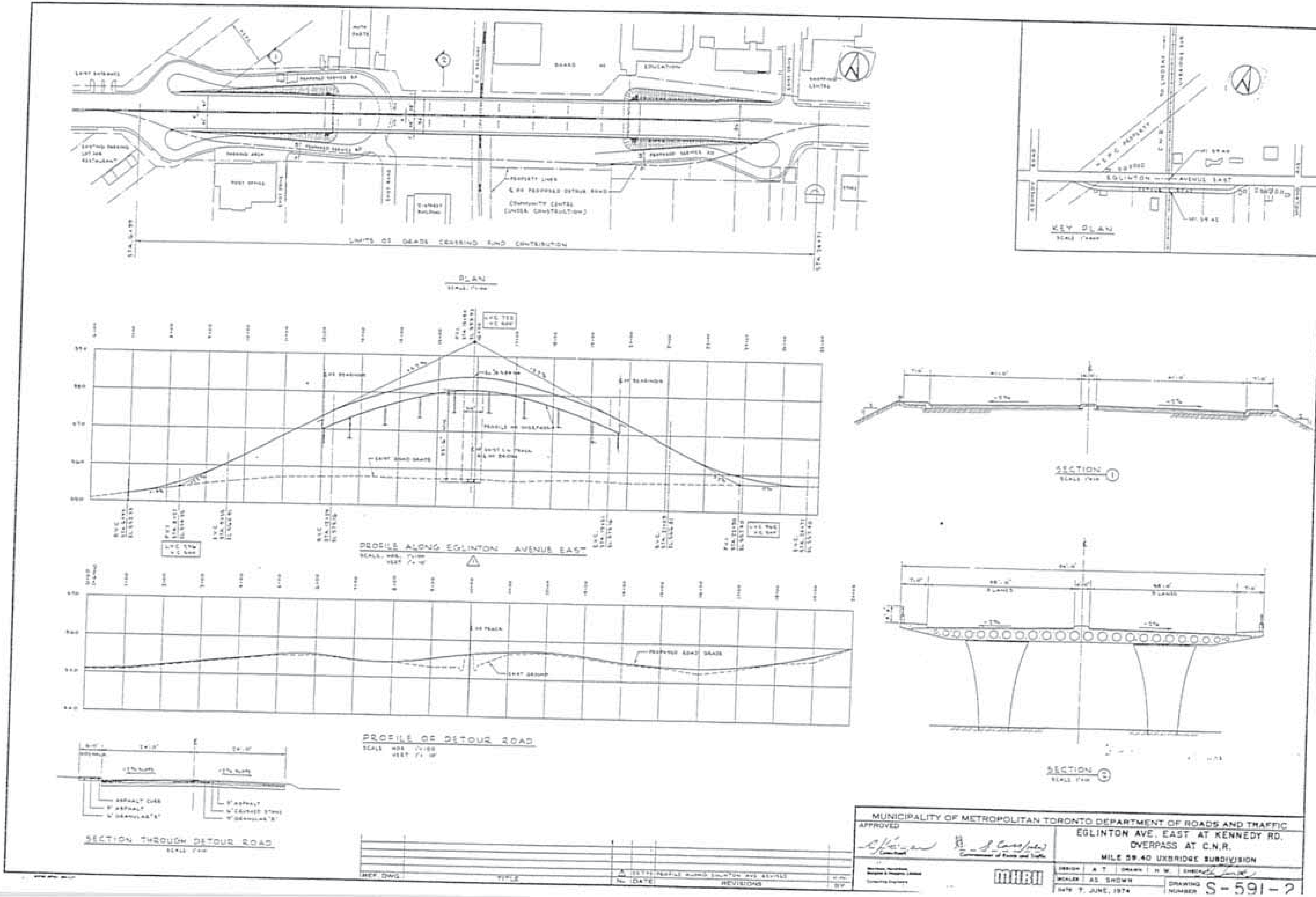
- Poles will be required on the deck to provide overhead power for the LRT. The forces due to poles supporting the catenaries will induce primarily localized effects. Pedestals and connections to deck slab will need to be provided and detailed appropriately.
- Expansion joints will need to be provided to minimize the effect of movement of the structure on the continuous welded rail. Expansion can be accommodated through combinations of rail anchors and bolted joints allowing for limited movements or special proprietary rail expansion joints.
- Protection of the structures and its components from corrosion due to stray currents should be provided by appropriate method of grounding or coating of reinforcement or insulating with a membrane below the trackbed.
- Proper detailing of waterproofing and paving where it abuts the LRT trackbed will be required to maintain the long term durability of the deck.
- The existing structure does not have deck drains. As the existing roadway is on a symmetrical crest curve, deck drains could be provided if necessary on the bridge structure, to provide adequate drainage of the LRT right-of-way.

The above identified miscellaneous structural details can be addressed with standard techniques that have been adopted elsewhere, and will be fully investigated during the preliminary and detail design phases of the project. The TTC is committed to working with City of Toronto and other authorities on these issues.

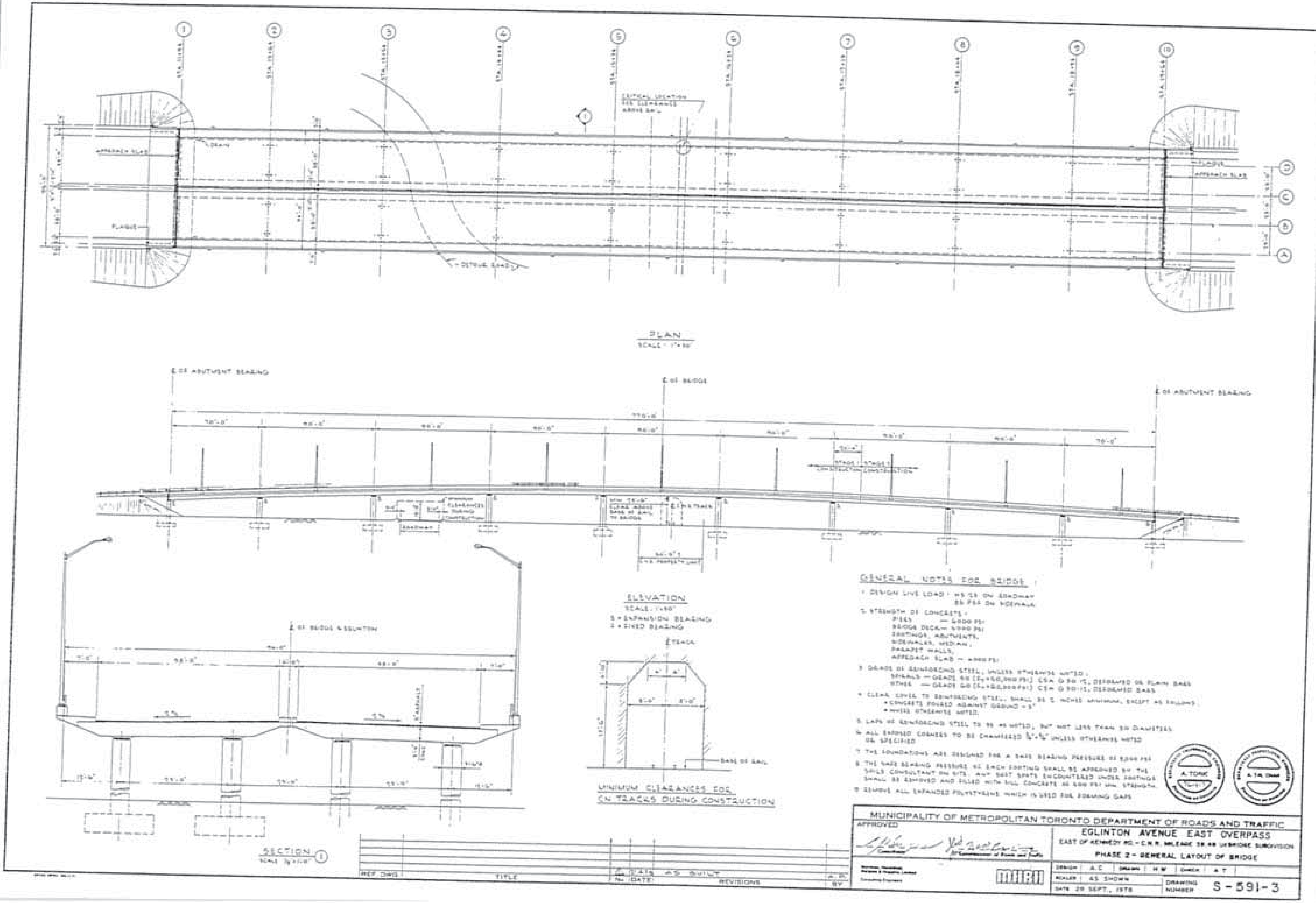
Long term maintenance and rehabilitation of the bridge deck and the LRT trackbed will be somewhat complicated by the LRT right-of-way. There are a number of alternatives available, with the simplest being that a temporary closure of the LRT ROW will be required during major rehabilitative works on the bridge, which extend for 4 to 6 months in duration, and local bus service be utilized. Alternatives and details will be developed in subsequent project phases.

Appendix A

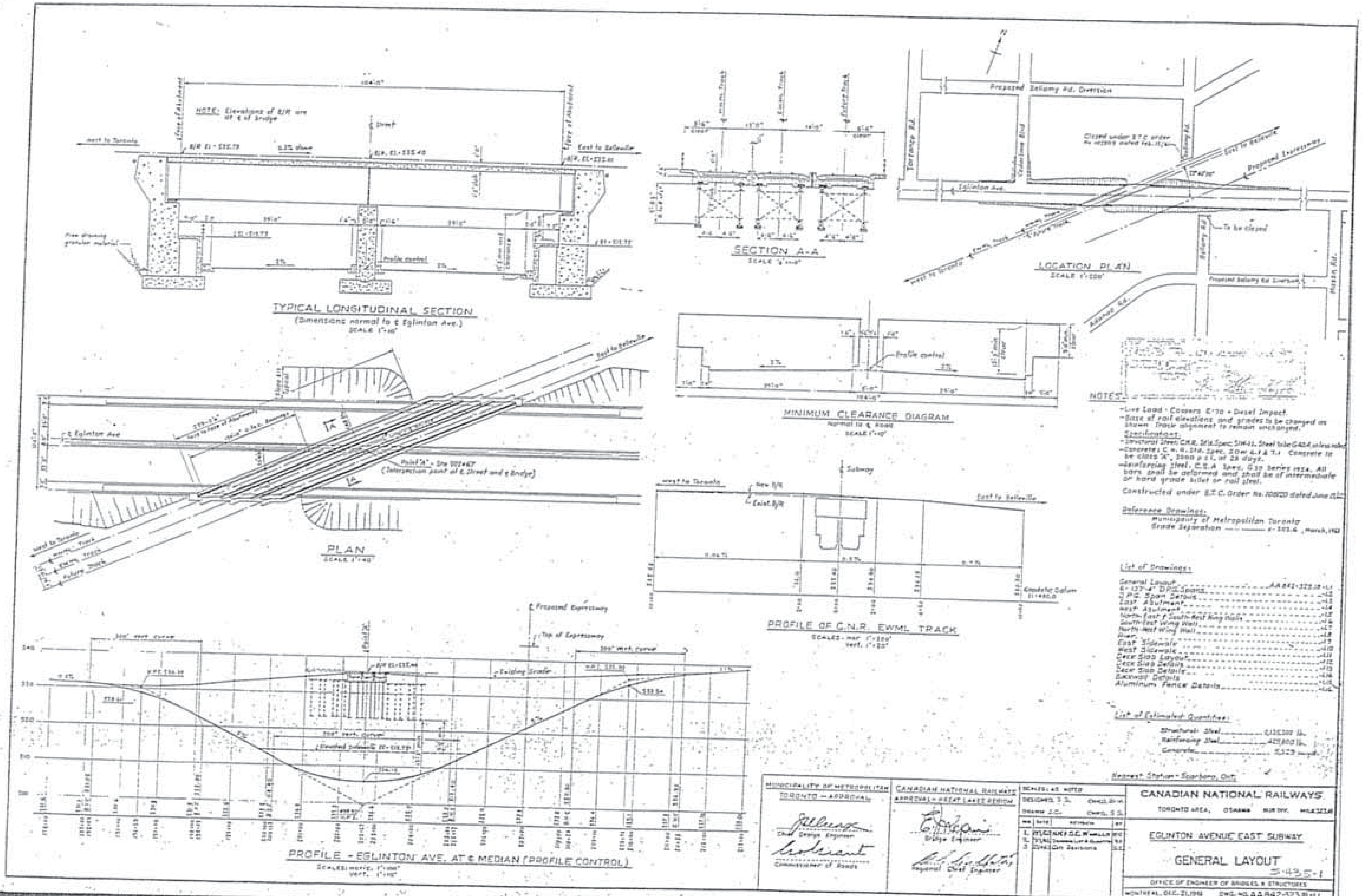
General Arrangement Drawings – Existing Structure



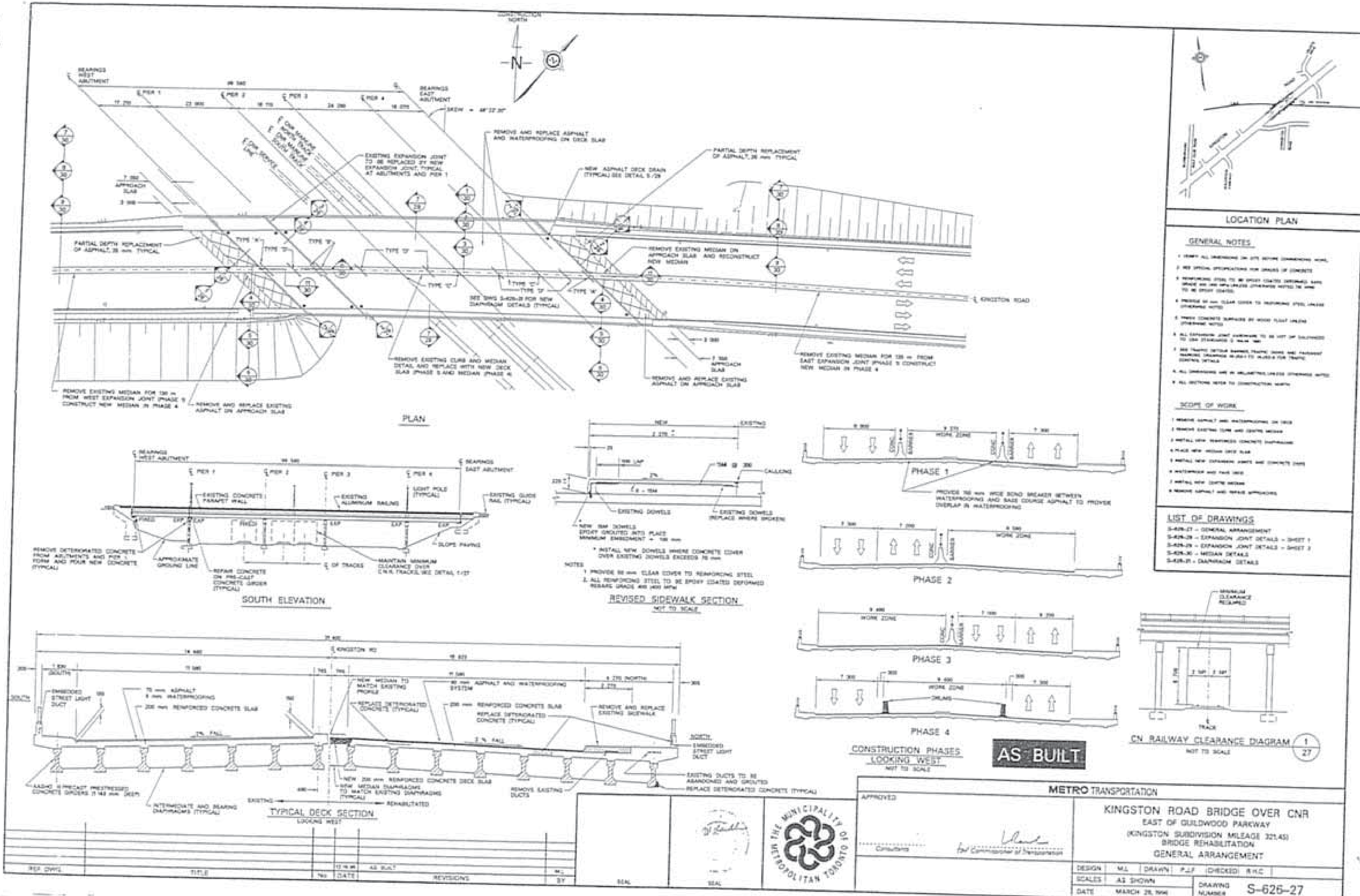
PLAN
KEY PLAN
EGLINTON AVENUE EAST
LIMITS OF GRADE CROSSING FUND CONTRIBUTION
PROFILE ALONG EGLINTON AVENUE EAST
PROFILE OF DETOUR ROAD
SECTION THROUGH DETOUR ROAD
SECTION 1
SECTION 2
MUNICIPALITY OF METROPOLITAN TORONTO DEPARTMENT OF ROADS AND TRAFFIC
APPROVED
EGLINTON AVE. EAST AT KENNEDY RD.
OVERPASS AT C.N.R.
MILE 59.40 UXBRIDGE SUBDIVISION
DESIGN A.T.
DRAWN H.W.
SCALES AS SHOWN
DATE 7. JUNE, 1974
DRAWING NUMBER S-591-2
REF. DWG.
TITLE
No. DATE
REVISIONS


PLAN
SCALE 1"=30'
CRITICAL LOCATION SEE CLEARANCE ABOVE RAIL
APPROACH SLAB
DRAIN
PLAQUE
DETOUR ROAD
℄ OF ABUTMENT BEARING
℄ OF BRIDGE
770'-0"
STAGE I CONSTRUCTION
STAGE II CONSTRUCTION
MINIMUM CLEARANCES DURING CONSTRUCTION
ROADWAY
℄ C.N.R. TRACK
C.N.R. PROPERTY LIMIT
ELEVATION
SCALE 1"=30'
E = EXPANSION BEARING
F = FIXED BEARING
℄ OF BRIDGE & EGLINTON
℄ TRACK
BASE OF RAIL
MINIMUM CLEARANCES FOR CN TRACKS DURING CONSTRUCTION
SECTION 1
SCALE 1/8"=1'-0"
GENERAL NOTES FOR BRIDGE
1 DESIGN LIVE LOAD: HS 25 ON ROADWAY
85 PSF ON SIDEWALK
2 STRENGTH OF CONCRETE:
PIERS — 6000 PSI
BRIDGE DECK — 5000 PSI
FOOTINGS, ABUTMENTS,
SIDEWALKS, MEDIAN,
PARAPET WALLS,
APPROACH SLAB — 4000 PSI
3 GRADE OF REINFORCING STEEL, UNLESS OTHERWISE NOTED:
SPIRALS — GRADE 40 (Fy=40,000 PSI) CSA G30.15, DEFORMED OR PLAIN BARS
OTHER — GRADE 60 (Fy=60,000 PSI) CSA G30.12, DEFORMED BARS
4 CLEAR COVER TO REINFORCING STEEL SHALL BE 2 INCHES MINIMUM, EXCEPT AS FOLLOWS:
• CONCRETE POURED AGAINST GROUND = 3"
• UNLESS OTHERWISE NOTED.
5 LAPS OF REINFORCING STEEL TO BE AS NOTED, BUT NOT LESS THAN 30 DIAMETERS
6 ALL EXPOSED CORNERS TO BE CHAMFERED ¾"×¾" UNLESS OTHERWISE NOTED OR SPECIFIED
7 THE FOUNDATIONS ARE DESIGNED FOR A SAFE BEARING PRESSURE OF 8,000 PSF
8 THE SAFE BEARING PRESSURE OF EACH FOOTING SHALL BE APPROVED BY THE SOILS CONSULTANT ON SITE. ANY SOFT SPOTS ENCOUNTERED UNDER FOOTINGS SHALL BE REMOVED AND FILLED WITH FILL CONCRETE OF 2000 PSI MIN. STRENGTH.
9 REMOVE ALL EXPANDED POLYSTYRENE WHICH IS USED FOR FORMING GAPS
A. TONK
MUNICIPALITY OF METROPOLITAN TORONTO DEPARTMENT OF ROADS AND TRAFFIC
APPROVED
EGLINTON AVENUE EAST OVERPASS
EAST OF KENNEDY RD. – C.N.R. MILEAGE 39.48 UXBRIDGE SUBDIVISION
PHASE 2 – GENERAL LAYOUT OF BRIDGE
DESIGN A.C.
DRAWN H.W.
CHECK A.T.
SCALES AS SHOWN
DATE 29 SEPT., 1978
DRAWING NUMBER S-591-3
REF. DWG.
TITLE
AS BUILT
No. DATE
REVISIONS
BY

5

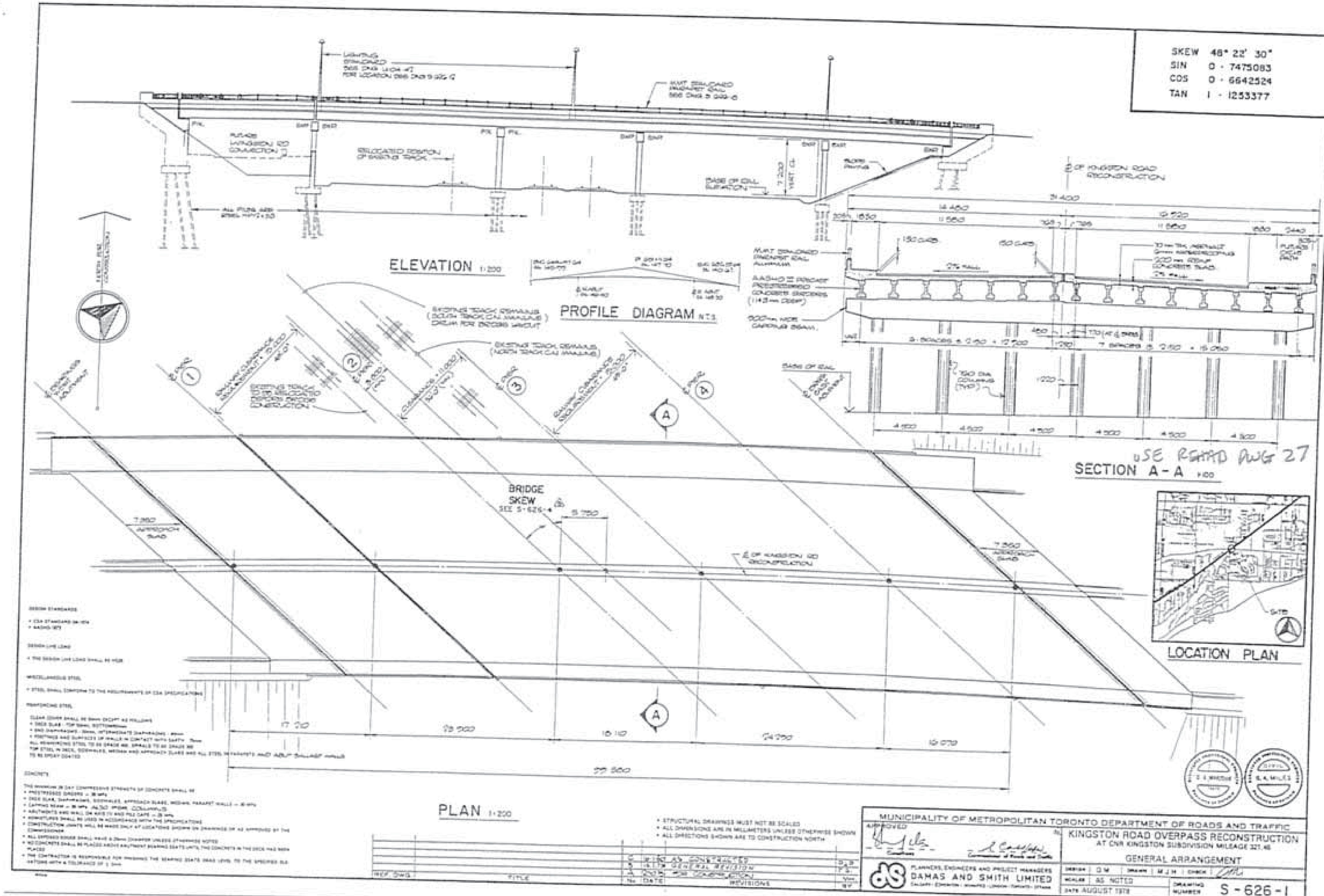

TYPICAL LONGITUDINAL SECTION
(Dimensions normal to ℄ Eglinton Ave.)
SCALE 1"=10'
NOTE: Elevations of B/R are at ℄ of Bridge
West to Toronto
East to Belleville
Free draining granular material
Profile control
SECTION A-A
LOCATION PLAN
SCALE 1"=200'
Proposed Bellamy Rd. Diversion
Torrance Rd.
Eglinton Ave.
Closed under B.T.C. order
To be closed
Proposed Expressway
Proposed Bellamy Rd. Diversion
Bellamy Rd.
Midland Ave.
West to Toronto
Future Track
PLAN
SCALE 1"=40'
℄ Eglinton Ave
Point "A" - Sta 322+67
(Intersection point of ℄ Street and ℄ Bridge)
East to Belleville
West to Toronto
EWML Track
Future Track
MINIMUM CLEARANCE DIAGRAM
Normal to ℄ Road
SCALE 1"=10'
Profile control
℄ Subway
New B/R
Exist. B/R
East to Belleville
Geodetic Datum
PROFILE OF C.N.R. EWML TRACK
SCALES - Hor. 1"=200'
Vert. 1"=20'
℄ Proposed Expressway
Top of Expressway
Point "A"
Existing Grade
vert. curve
PROFILE - EGLINTON AVE. AT ℄ MEDIAN (PROFILE CONTROL)
SCALES: Horiz. 1"=100'
Vert. 1"=10'
NOTES:
- Live Load - Coopers E-70 + Diesel Impact.
- Base of rail elevations and grades to be changed as shown. Track alignment to remain unchanged.
Specifications:
- Structural Steel: C.N.R. Std. Spec. S1W-41. Steel to be G40.4 unless noted.
- Concrete: C.N.R. Std. Spec. 20W 6.1 & 7.1. Concrete to be class "A", 3000 p.s.i. at 28 days.
- Reinforcing Steel: C.S.A. Spec. G30 Series 1954. All bars shall be deformed and shall be of intermediate or hard grade billet or rail steel.
Constructed under B.T.C. Order No. 108120 dated June
Reference Drawings:
Municipality of Metropolitan Toronto
Grade Separation
List of Drawings:
General Layout — AA 842-323.18-L1
6-127'-4" D.P.G. Spans — -L2
D.P.G. Span Details — -L3
East Abutment — -L4
West Abutment — -L5
North-East & South-West Wing Walls — -L6
South-East Wing Wall — -L7
North-West Wing Wall — -L8
Pier — -L9
East Sidewalk — -L9
West Sidewalk — -L10
Deck Slab Layout — -L11
Deck Slab Details — -L12
Deck Slab Details — -L13
Backwall Details — -L14
Aluminum Fence Details — -L15
List of Estimated Quantities:
Structural Steel
Reinforcing Steel
Concrete
Nearest Station - Scarboro, Ont.
MUNICIPALITY OF METROPOLITAN TORONTO - APPROVAL
Chief Design Engineer
Commissioner of Roads
CANADIAN NATIONAL RAILWAYS APPROVAL - GREAT LAKES REGION
Bridge Engineer
Regional Chief Engineer
SCALES: AS NOTED
CANADIAN NATIONAL RAILWAYS
TORONTO AREA, OSHAWA SUB DIV.
EGLINTON AVENUE EAST SUBWAY
GENERAL LAYOUT
5-435-1
OFFICE OF ENGINEER OF BRIDGES & STRUCTURES

4

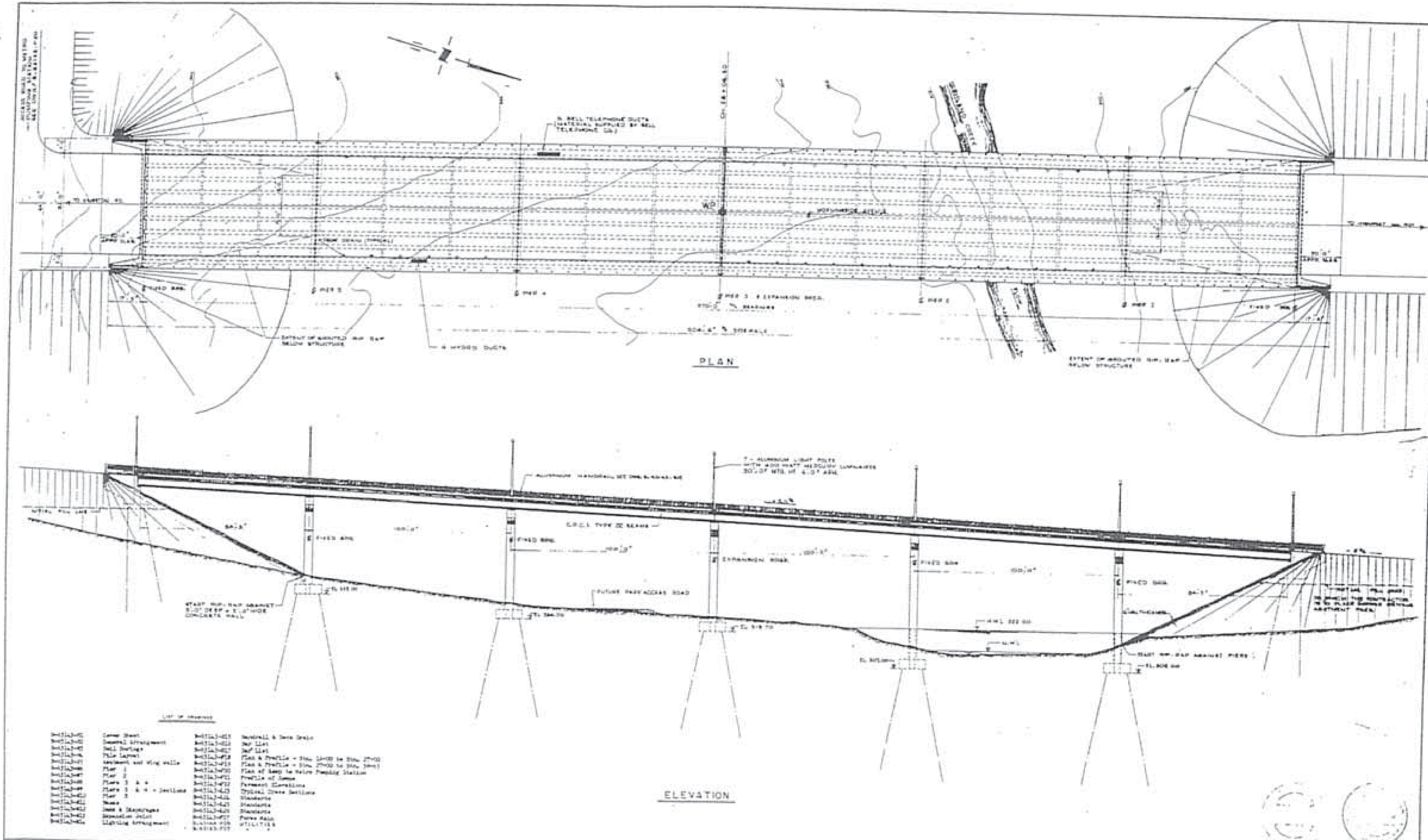


PLAN
SOUTH ELEVATION
REVISED SIDEWALK SECTION
NOT TO SCALE
TYPICAL DECK SECTION
LOOKING WEST
PHASE 1
PHASE 2
PHASE 3
PHASE 4
CONSTRUCTION PHASES
LOOKING WEST
NOT TO SCALE
AS BUILT
CN RAILWAY CLEARANCE DIAGRAM
NOT TO SCALE
1
27
LOCATION PLAN
GENERAL NOTES
SCOPE OF WORK
LIST OF DRAWINGS
KINGSTON ROAD
METRO TRANSPORTATION
APPROVED
Consultants
For Commissioner of Transportation
KINGSTON ROAD BRIDGE OVER CNR
EAST OF GUILDWOOD PARKWAY
(KINGSTON SUBDIVISION MILEAGE 321.45)
BRIDGE REHABILITATION
GENERAL ARRANGEMENT
DESIGN
M.L
DRAWN
P.J.F
(CHECKED)
R.H.C
SCALES
AS SHOWN
DATE
MARCH 28, 1996
DRAWING NUMBER
S-626-27
THE MUNICIPALITY OF METROPOLITAN TORONTO
SEAL
REF. DWG.
TITLE
DATE
REVISIONS
BY

3


SKEW 48° 22' 30"
SIN 0 · 7475083
COS 0 · 6642524
TAN 1 · 1253377
ELEVATION 1:200
PROFILE DIAGRAM N.T.S.
SECTION A - A 1:100
USE REHAB DWG 27
BRIDGE SKEW SEE S-626-4
LOCATION PLAN
SITE
PLAN 1:200
DESIGN STANDARDS
DESIGN LIVE LOAD
MISCELLANEOUS STEEL
REINFORCING STEEL
CONCRETE
MUNICIPALITY OF METROPOLITAN TORONTO DEPARTMENT OF ROADS AND TRAFFIC
APPROVED
KINGSTON ROAD OVERPASS RECONSTRUCTION
AT CNR KINGSTON SUBDIVISION MILEAGE 321.46
GENERAL ARRANGEMENT
PLANNERS, ENGINEERS AND PROJECT MANAGERS
DAMAS AND SMITH LIMITED
DESIGN D M
DRAWN M J H
CHECK
SCALE AS NOTED
DATE AUGUST 1978
DRAWING NUMBER S - 626 - 1
AS CONSTRUCTED
GENERAL REVISION
FOR CONSTRUCTION
REF. DWG
TITLE
No. DATE REVISIONS BY

3



NOTES

1. All concrete shall attain a minimum strength of 3,000 psi at 28 days, unless shown otherwise.
2. All concrete shall be air entrained in accordance with specifications.
3. All exposed concrete edges shall have a 3/4" x 3/4" chamfer unless shown otherwise.
4. Reinforcement to have cover as given below: Deck Slab = 1½"; Footings = 3"; Abutments, Piers & Retaining Walls = 2".
5. All construction joints to be approved by the Engineer.
6. The Township of Scarborough does not guarantee the accuracy of the soil report or the abridged version shown on these plans.
7. The General Contractor shall be responsible for ensuring that the final deck elevations conform with the elevations shown. He shall also be responsible for finishing the bridge seats level to a tolerance of plus or minus 1/8".
8. Finish shall be Class 1 except where shown. The underside of the bridge deck being Class 2.
9. For Prestressed Concrete Notes, see beam drawings.

APPROVED COMMISSIONER OF WORKS

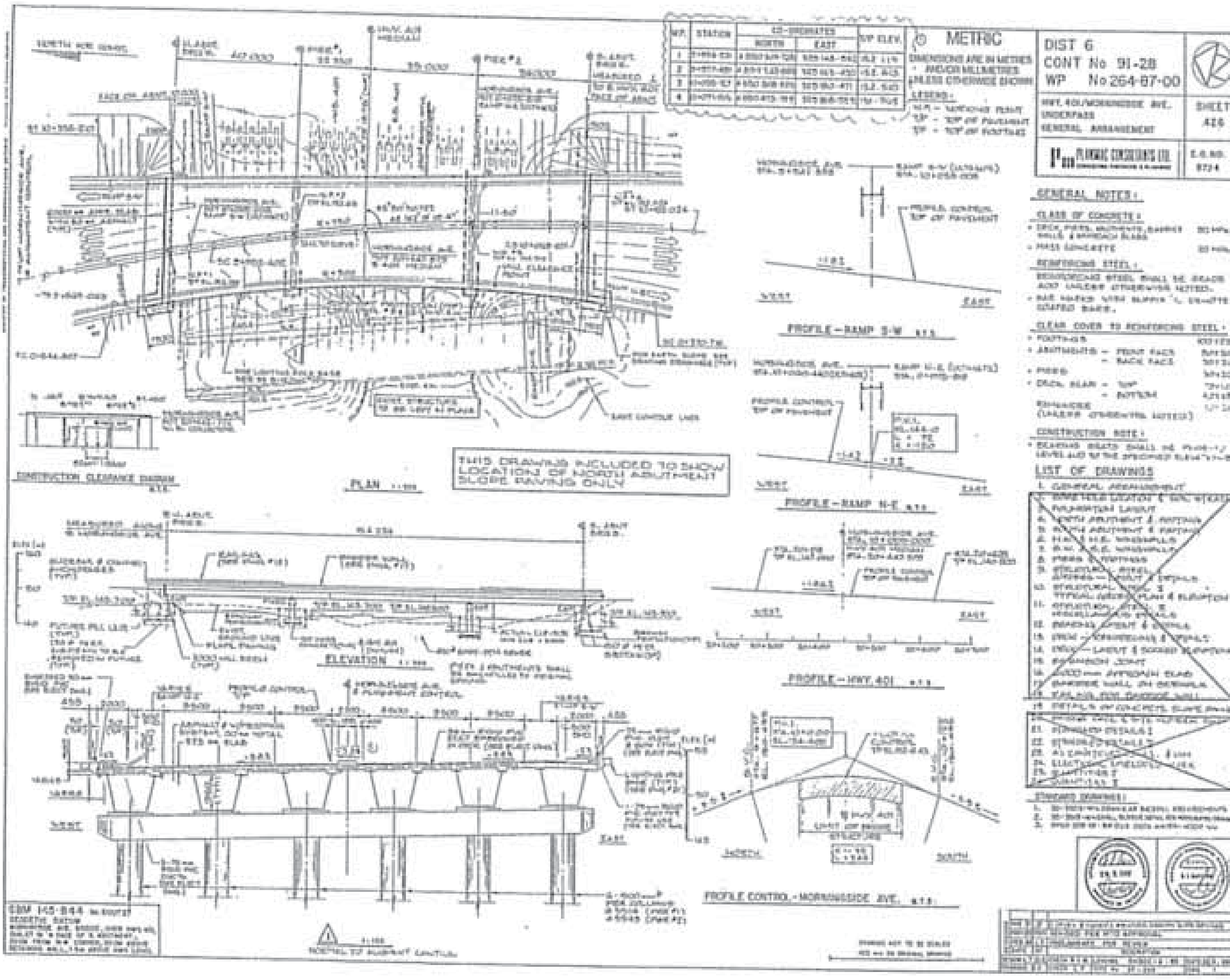
D.H.O. APPROVAL NO. WT 64-45

TOWNSHIP OF SCARBOROUGH
ENGINEERING DEPARTMENT

MORNINGSIDE AVENUE
BRIDGE OVER HIGHLAND CREEK
GENERAL ARRANGEMENT

PROCTOR & REDFERN
CONSULTING ENGINEERS TORONTO

S-656-1



METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN
DIST 6
CONT No 91-28
WP No 264-87-00
HWY. 401/MORNINGSIDE AVE. UNDERPASS
GENERAL ARRANGEMENT
SHEET 426
GENERAL NOTES:
CLASS OF CONCRETE:
REINFORCING STEEL:
CLEAR COVER TO REINFORCING STEEL:
CONSTRUCTION NOTE:
LIST OF DRAWINGS
STANDARD DRAWINGS:
PROFILE - RAMP S-W
PROFILE - RAMP N-E
PROFILE - HWY. 401
PROFILE CONTROL - MORNINGSIDE AVE.
THIS DRAWING INCLUDED TO SHOW LOCATION OF NORTH ABUTMENT SLOPE PAVING ONLY
PLAN
ELEVATION
CONSTRUCTION CLEARANCE DIAGRAM
WEST
EAST
NORTH
SOUTH

1

Appendix B

Photographs – Existing Structures

SITE PHOTOGRAPHS
EGLINTON AVENUE EAST – CNR SUBWAY (AT BELLAMY)

Picture 1 – East Elevation

Photo 2 – West Elevation

SITE PHOTOGRAPHS
EGLINTON AVENUE EAST – CNR SUBWAY (AT BELLAMY)

Picture 3 – East Approach

Photo 4 – West Approach

SITE PHOTOGRAPHS
KINGSTON ROAD – CNR OVERPASS

Picture 5 – Looking North at Structure

Picture 6 – South Approach

Picture 7 – Looking South at Structure

Picture 8 – North Approach

Photo 9 – East Elevation

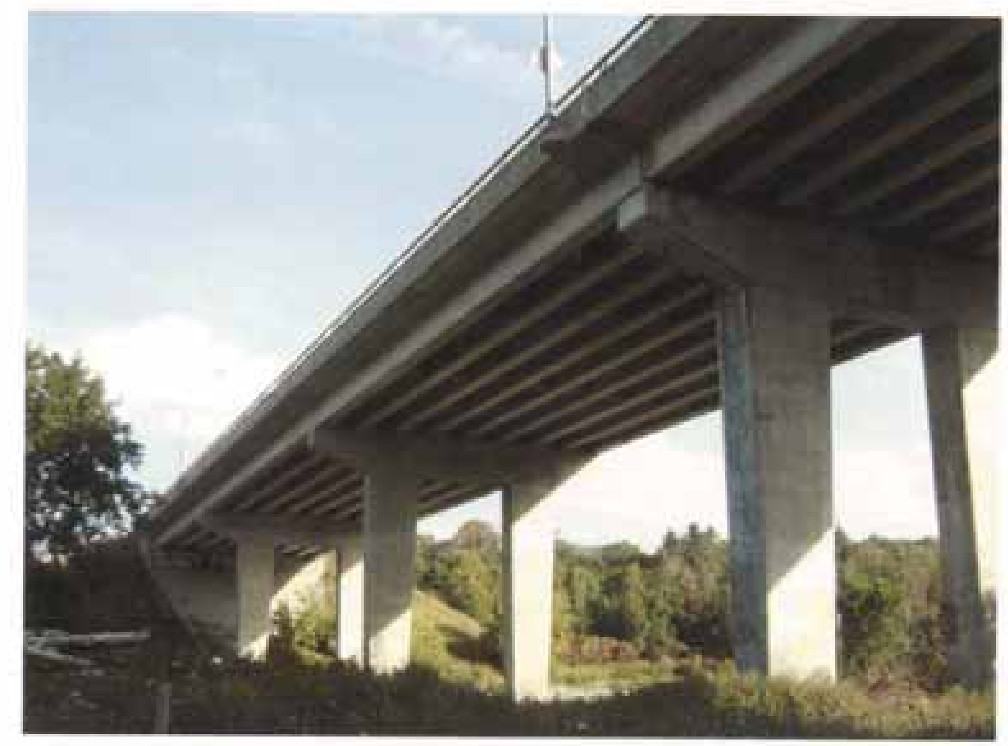


Photo 10 – West Elevation

Picture 11 – Looking South at Structure

Picture 12 – North Approach

Picture 13 – Looking North at Structure

Picture 14 – South Approach

Photo 15 – East Elevation

Paint 16 – West Elevation

Photo 17 – Looking South at Structure

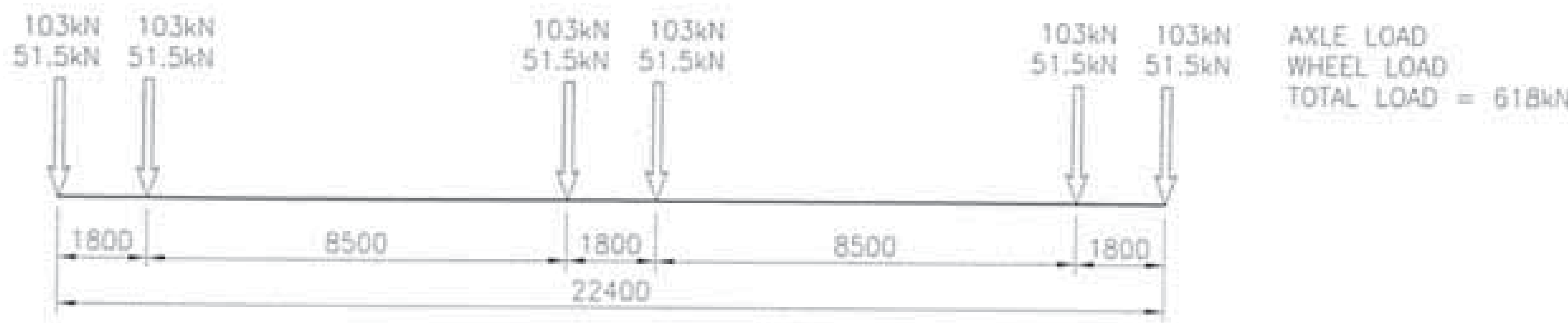
Photo 18 – North Approach

Picture 19 – Looking North at Structure

Picture 20 – South Approach

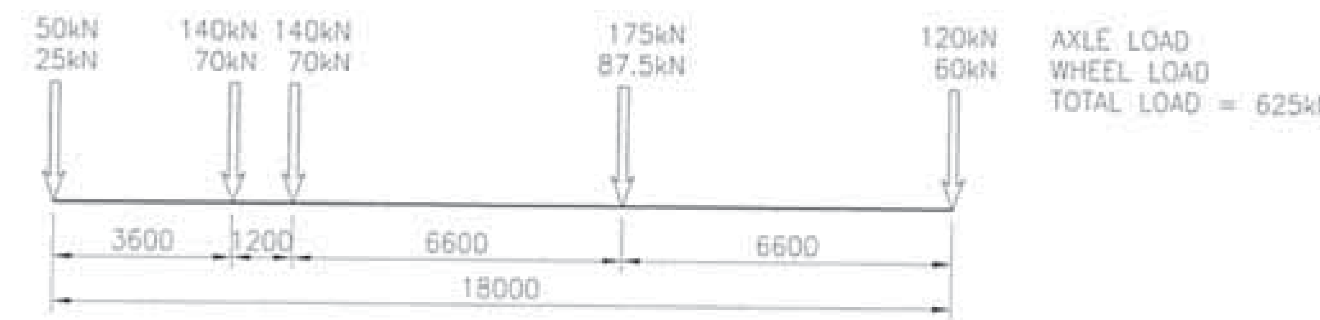
Appendix C

Proposed General Arrangement Drawings



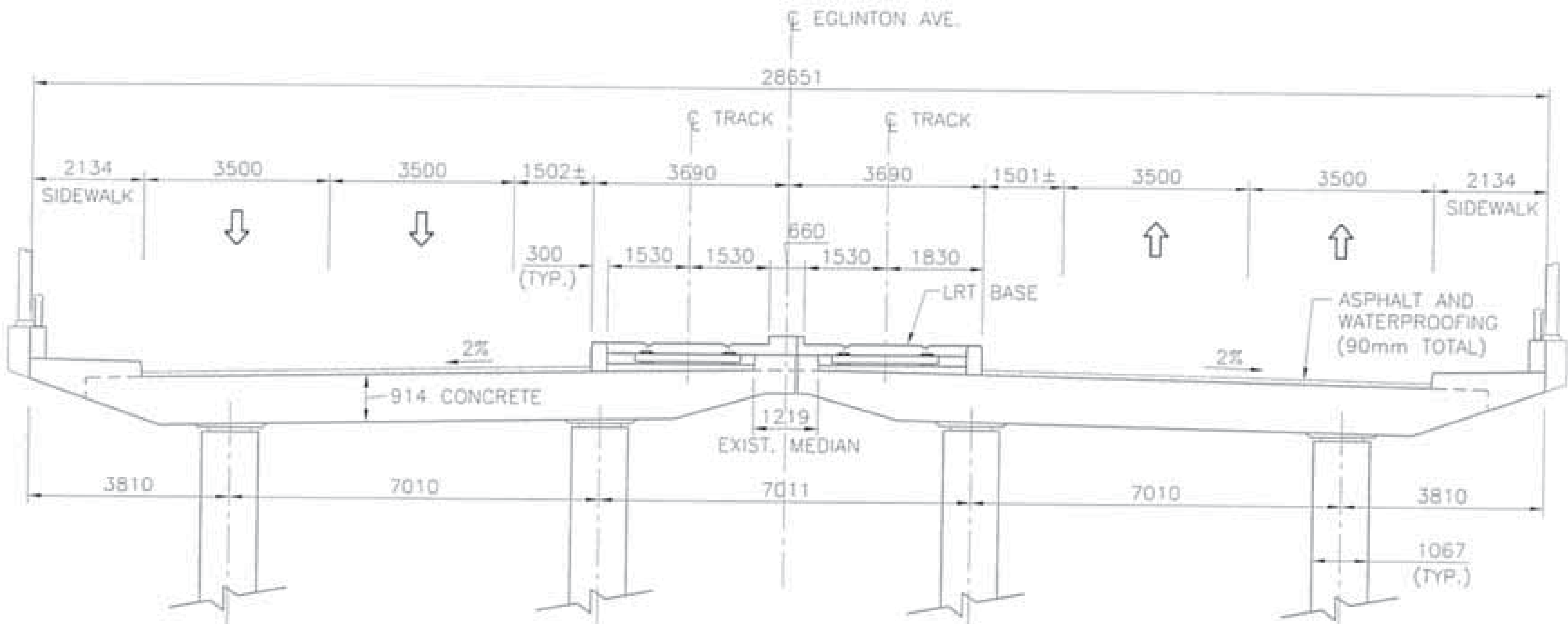
ASSUMED LOADS OF THE NEW LRT VEHICLE

N.T.S.



CL-625-ON TRUCK LIVE LOADING (AS PER CHBDC CAN /CSA-S6-06)

N.T.S.



STRUCTURE No. 1 - EGLINTON AVENUE C.N.R. OVERPASS

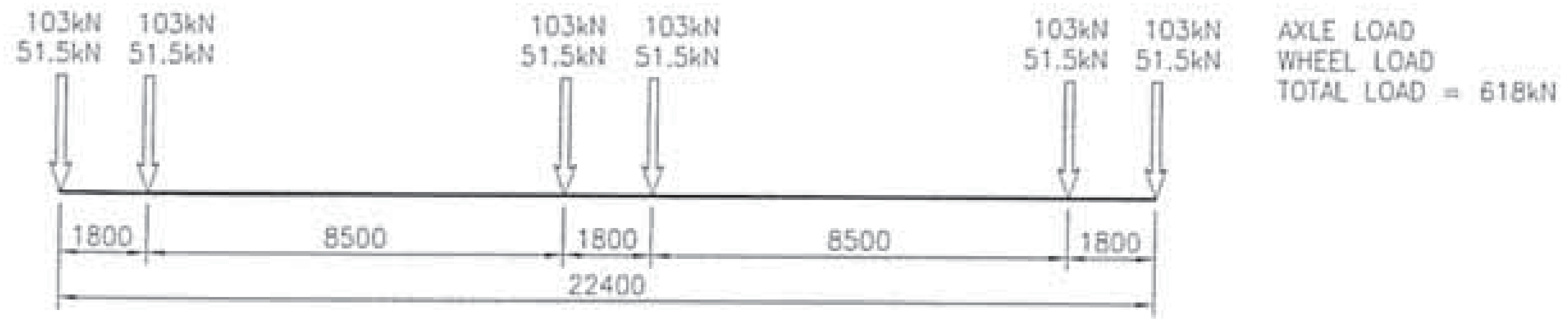
1:100 0 1 2 3 4m

Totten Sims Hubicki Associates (1997) Ltd.

PROJECT: SCARBOROUGH MALVERN LRT EA

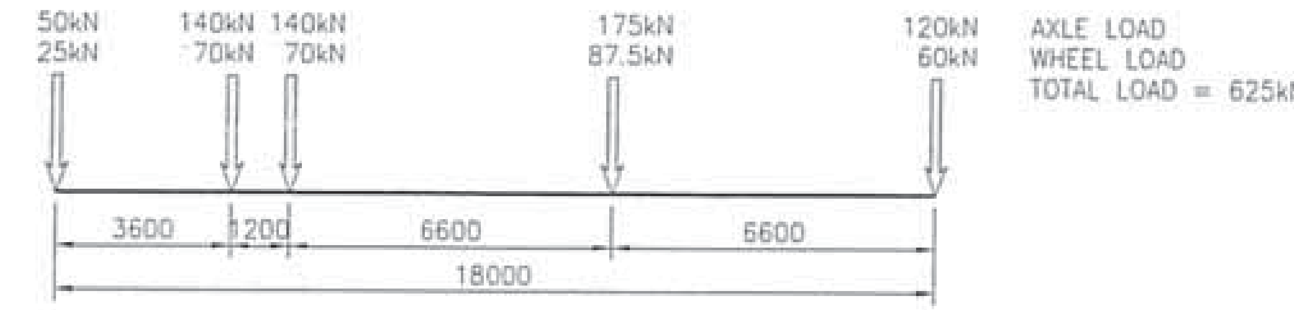
DRAWING: EGLINTON AVENUE EAST - CNR OVERPASS
CONCEPTUAL BRIDGE CROSS SECTION

1



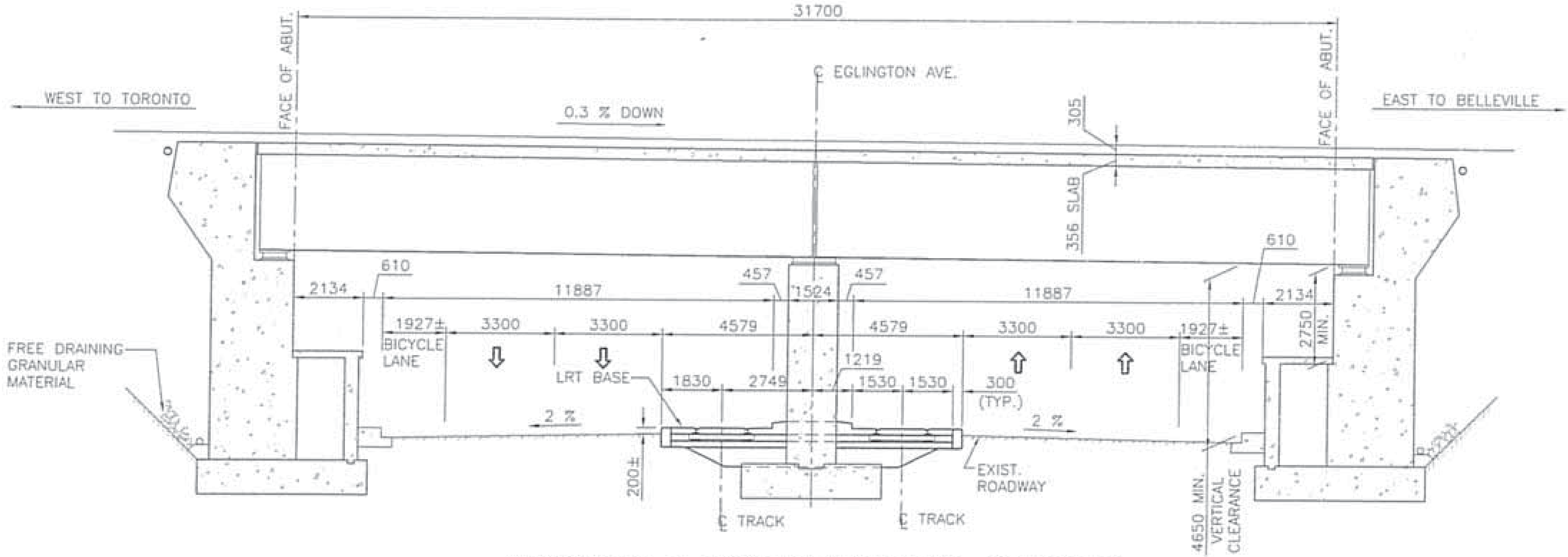
ASSUMED LOADS OF THE NEW LRT VEHICLE

N.T.S.



CL-625-ON TRUCK LIVE LOADING (AS PER CHBDC CAN /CSA-S6-06)

N.T.S.



STRUCTURE No. 2 - EGLINTON AVENUE EAST - CNR SUBWAY

0 1 2 3 4 5m

1:125

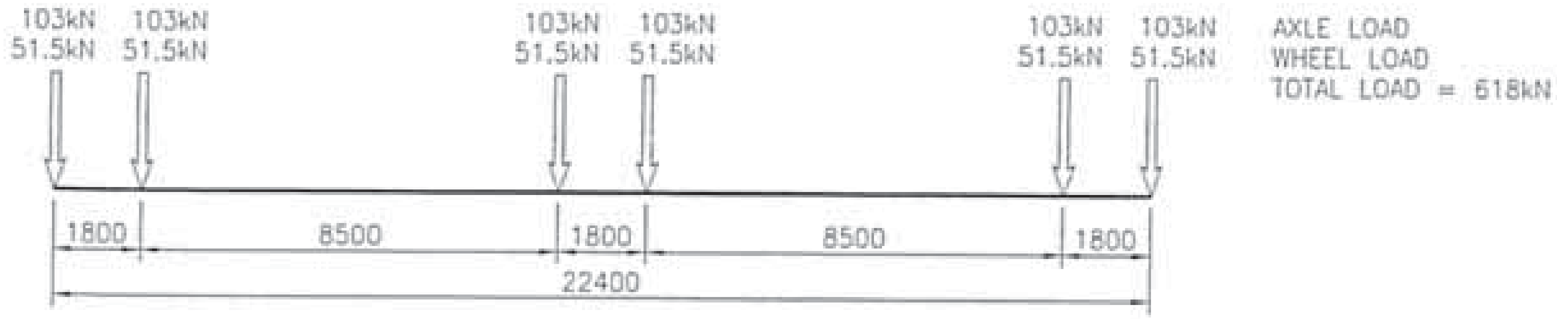
Totten Sims Hubicki Associates

TSH

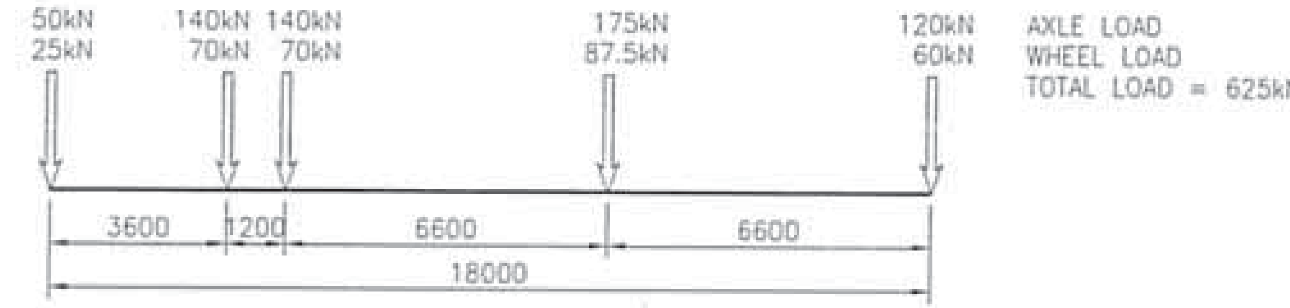
SCARBOROUGH MALVERN LRT EA

EGLINTON AVENUE EAST - CNR SUBWAY
CONCEPTUAL BRIDGE CROSS SECTION

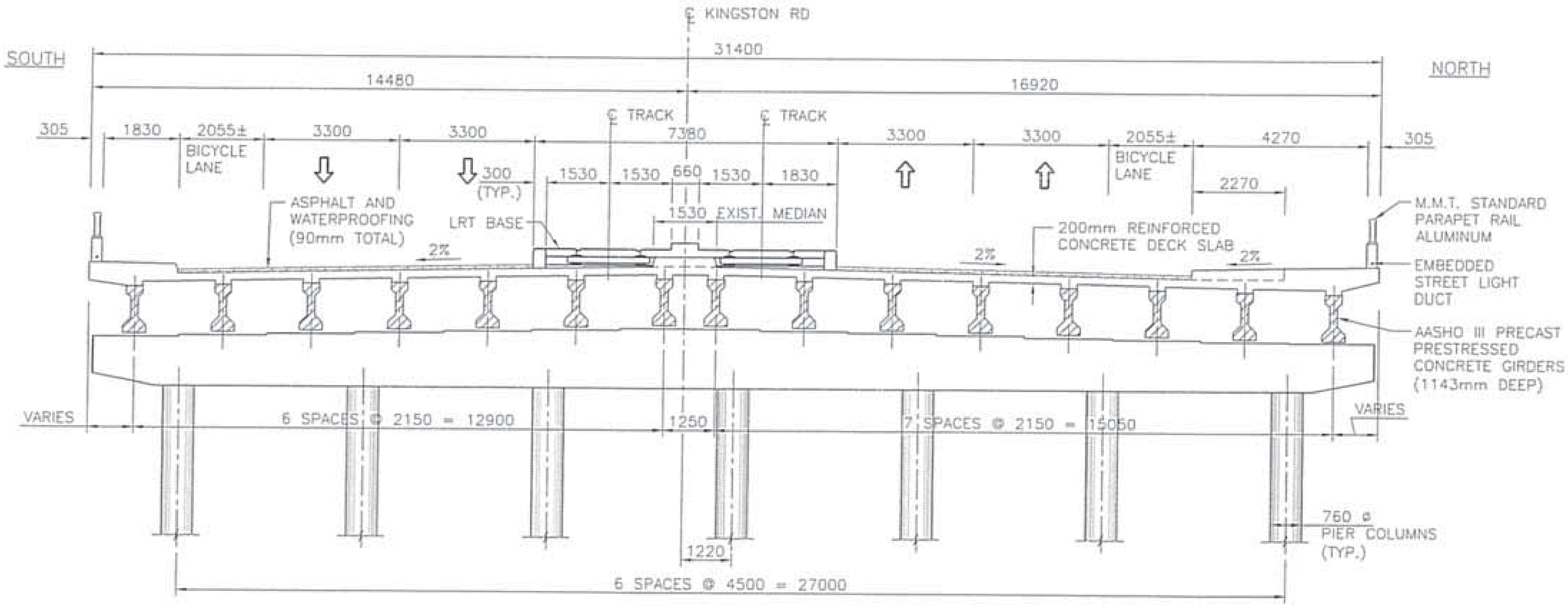
2



ASSUMED LOADS OF THE NEW LRT VEHICLE
N.T.S.



CL-625-ON TRUCK LIVE LOADING (AS PER CHBDC CAN /CSA-S6-06)
N.T.S.



STRUCTURE No. 3 - KINGSTON ROAD BRIDGE OVER CNR

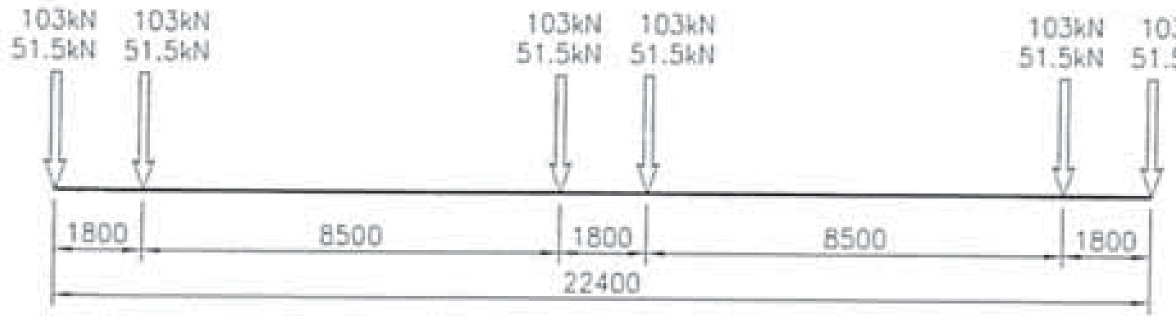
1:100 0 1 2 3 4m

TSH

SCARBOROUGH MALVERN LRT EA

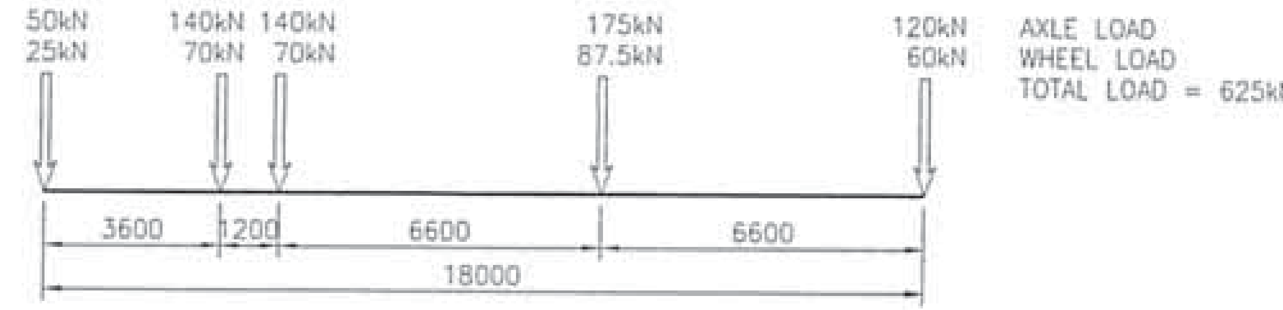
KINGSTON ROAD - CNR OVERPASS
CONCEPTUAL BRIDGE CROSS SECTION

3



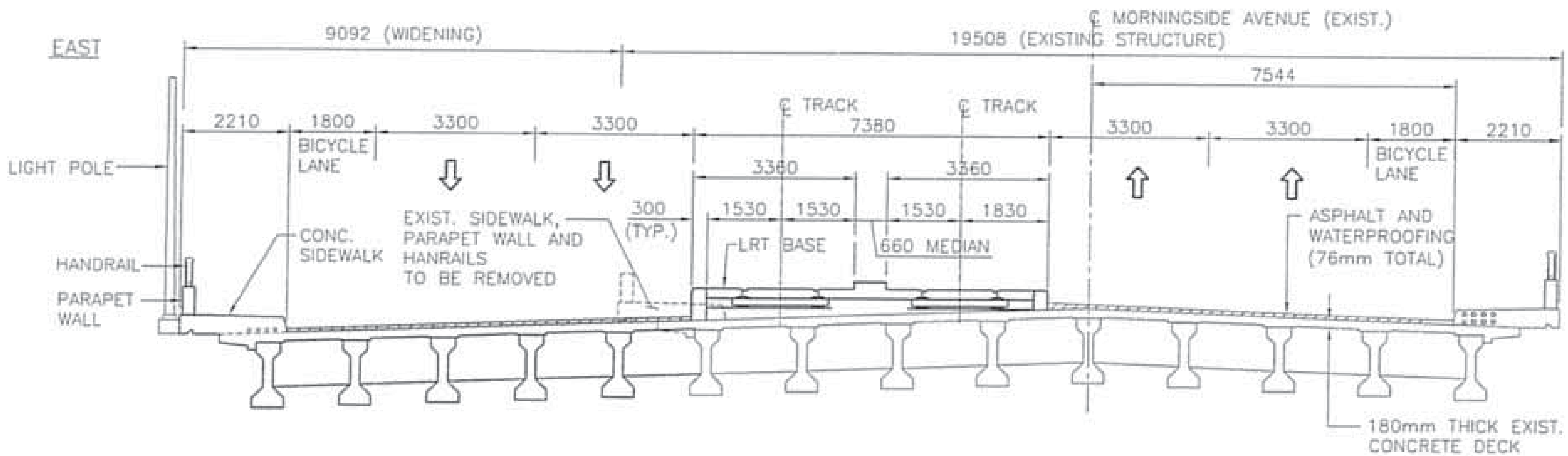
ASSUMED LOADS OF THE NEW LRT VEHICLE

N.T.S.



CL-625-ON TRUCK LIVE LOADING (AS PER CHBDC CAN /CSA-S6-06)

N.T.S.



STRUCTURE No. 4 - MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK

OPTION 1A - LRT AT MEDIAN (WITH BICYCLE LANES)

1:100 0 1 2 3 4m

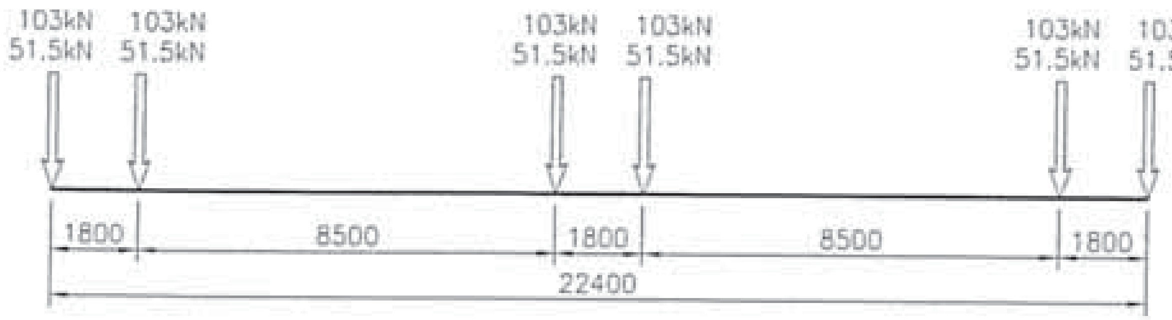
Totten Sims Hubicki Associates (1981) Ltd.

TSH

SCARBOROUGH MALVERN LRT EA

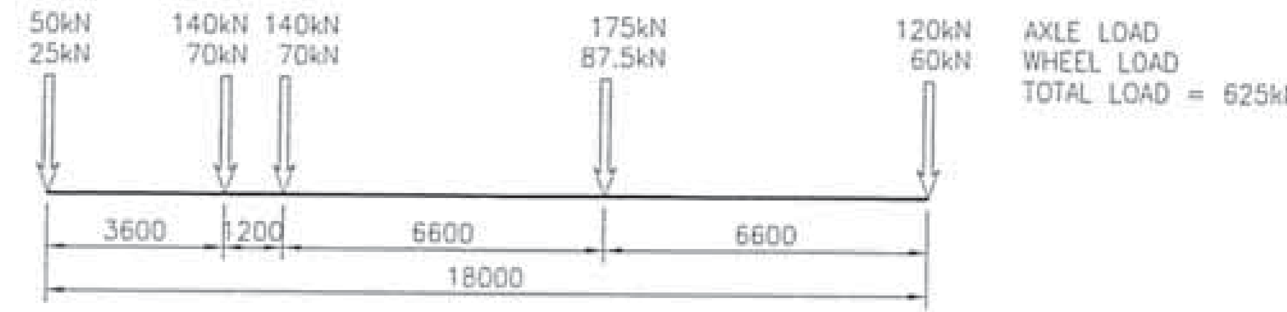
MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK
CONCEPTUAL BRIDGE CROSS SECTION
OPTION 1A - LRT AT MEDIAN (WITH BICYCLE LANES)

4-1A



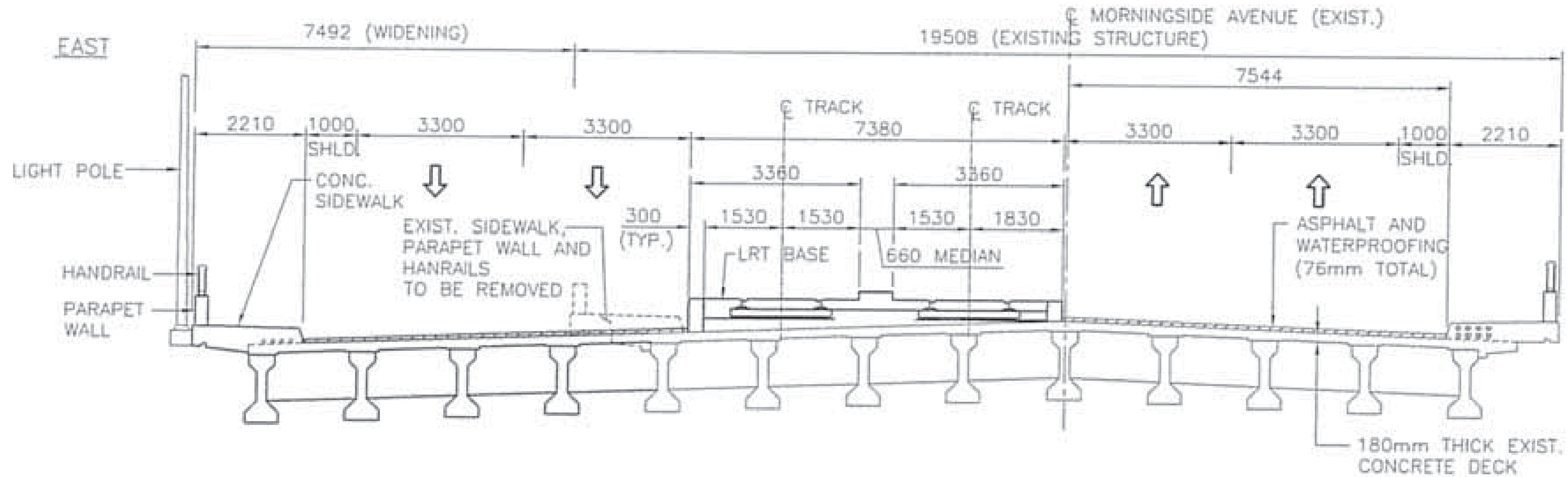
ASSUMED LOADS OF THE NEW LRT VEHICLE

N.T.S.



CL-625-ON TRUCK LIVE LOADING (AS PER CHBDC CAN /CSA-S6-06)

N.T.S.



STRUCTURE No. 4 - MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK

OPTION 1B - LRT AT MEDIAN (WITHOUT BICYCLE LANES)

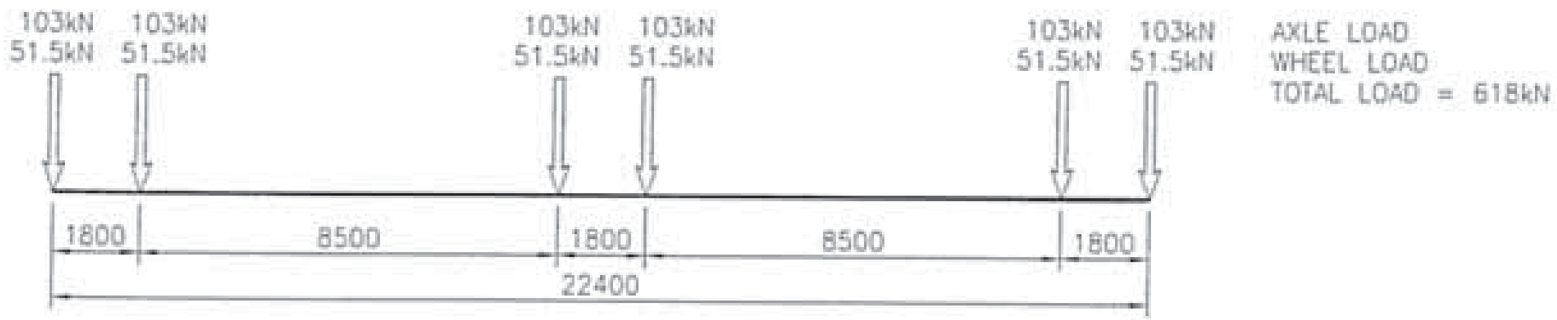
1:100 — 0 1 2 3 4m

TSH

SCARBOROUGH MALVERN LRT EA

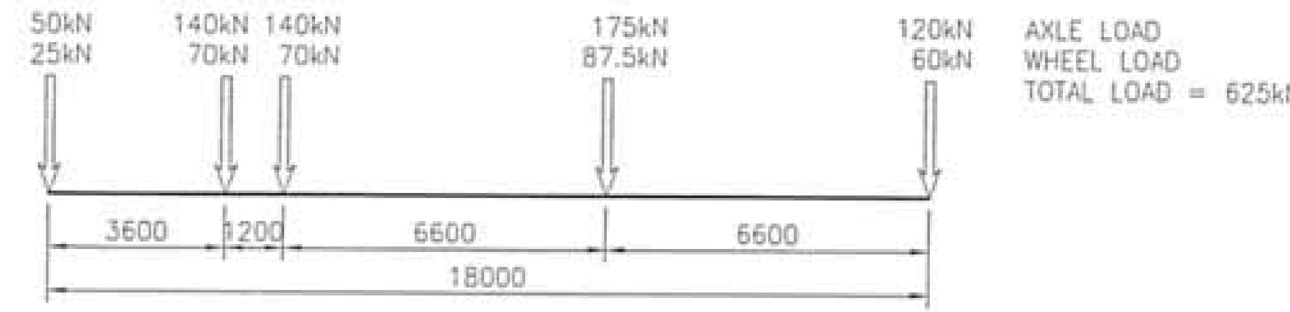
MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK
CONCEPTUAL BRIDGE CROSS SECTION
OPTION 1B - LRT AT MEDIAN (WITHOUT BICYCLE LANES)

4-1B



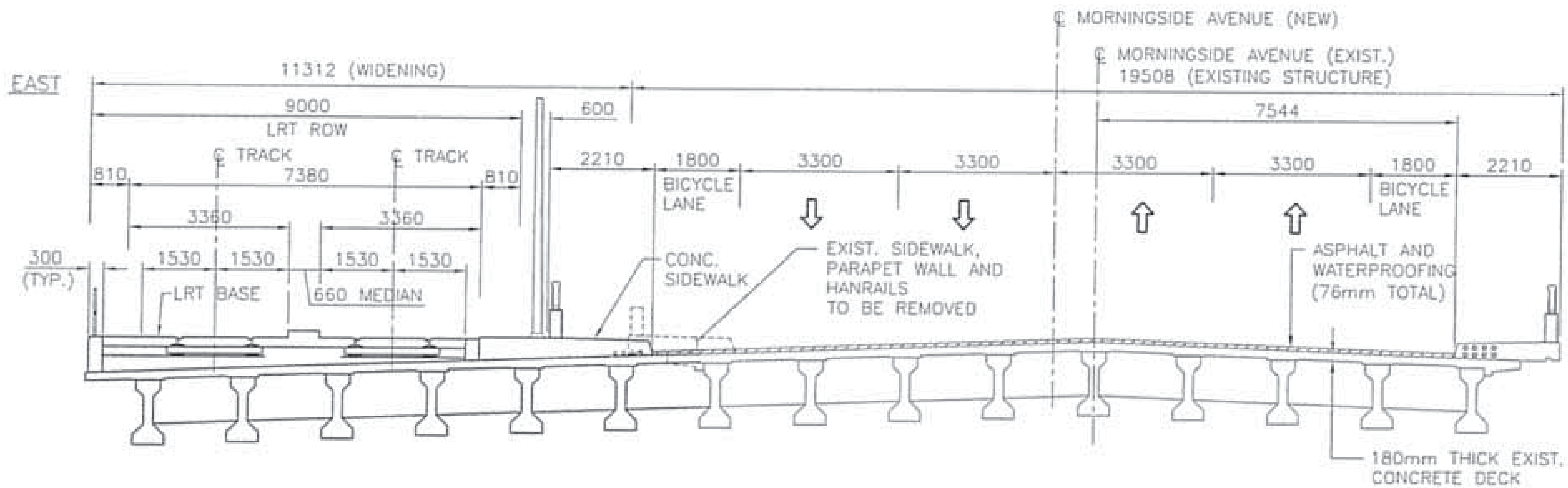
ASSUMED LOADS OF THE NEW LRT VEHICLE

N.T.S.



CL-625-ON TRUCK LIVE LOADING (AS PER CHBDC CAN /CSA-S6-06)

N.T.S.



STRUCTURE No. 4 - MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK

OPTION 2A - LRT AT EAST SIDE (WITH BICYCLE LANES)

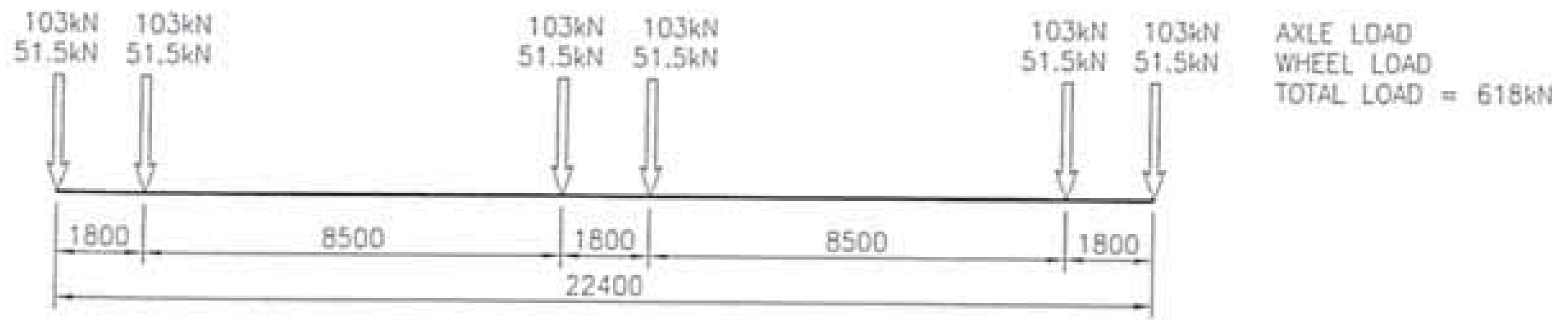
1:100 0 1 2 3 4m

Totten Sims Hubicki Associates (1997) Ltd.
Whitby, Ontario, Canada
Email: tsh@tsh.ca www.tsh.ca

SCARBOROUGH MALVERN LRT EA

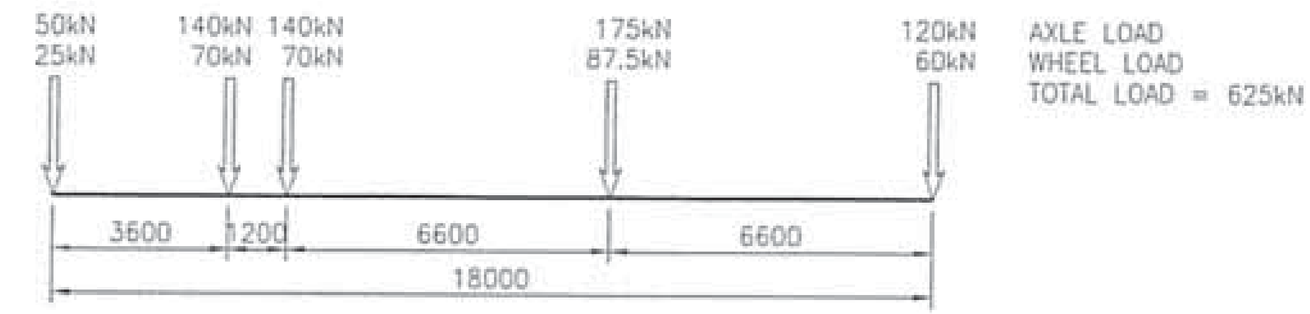
MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK
CONCEPTUAL BRIDGE CROSS SECTION
OPTION 2A - LRT AT EAST SIDE (WITH BICYCLE LANES)

4-2A



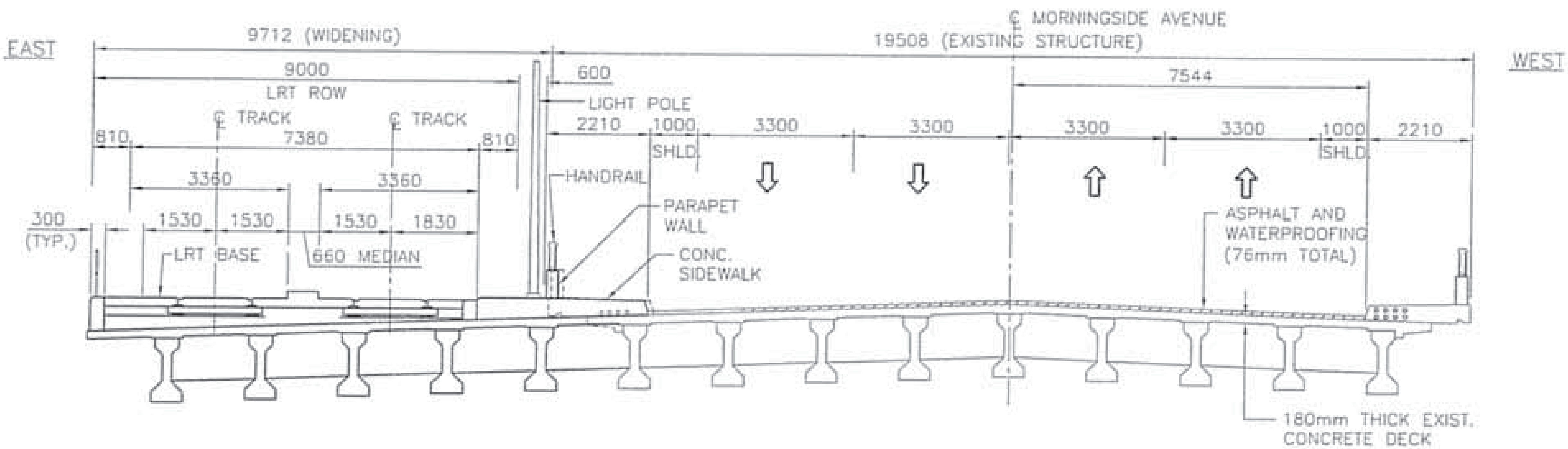
ASSUMED LOADS OF THE NEW LRT VEHICLE

N.T.S.



CL-625-ON TRUCK LIVE LOADING (AS PER CHBDC CAN /CSA-S6-06)

N.T.S.



STRUCTURE No. 4 - MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK

OPTION 2B - LRT AT EAST SIDE (WITHOUT BICYCLE LANES)

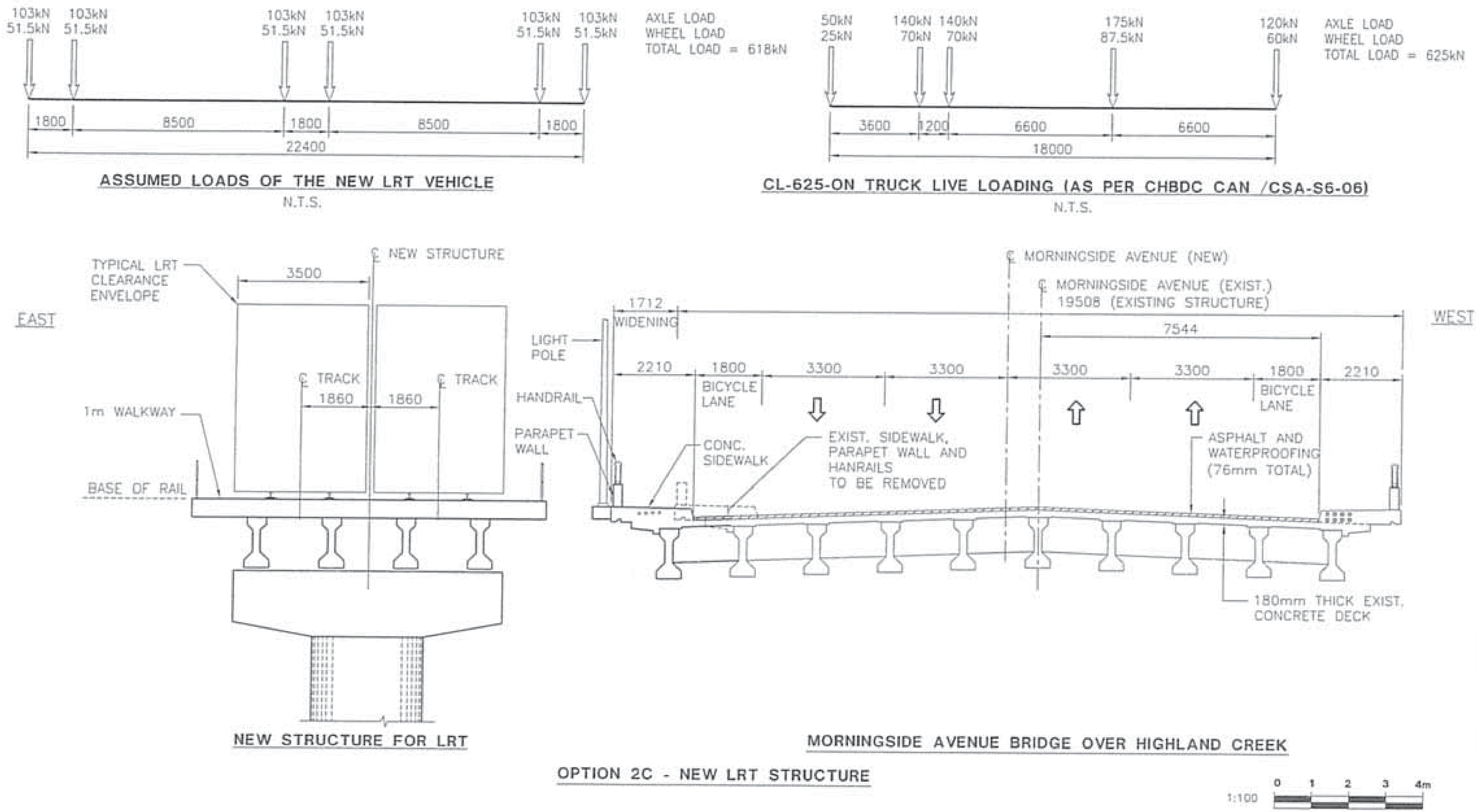
1:100 — 0 1 2 3 4m

Totten Sims Hubicki Associates (1997) Ltd.

SCARBOROUGH MALVERN LRT EA

MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK
CONCEPTUAL BRIDGE CROSS SECTION
OPTION 2B - LRT AT EAST SIDE (WITHOUT BICYCLE LANES)

4-2B



Totten Sims Hubicki Associates (1997) Ltd.

TSH

SCARBOROUGH MALVERN LRT EA

MORNINGSIDE AVENUE BRIDGE OVER HIGHLAND CREEK
CONCEPTUAL BRIDGE CROSS SECTION
OPTION 2C - NEW LRT STRUCTURE

4-2C

AECOM

Appendix D

Structural Assessment of Highway 401 - Morningside Avenue Underpass

AECOM
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

IBI GROUP

STRUCTURAL ASSESSMENT OF THE HIGHWAY 401 – MORNINGSIDE AVENUE UNDERPASS STRUCTURE FOR LIGHT RAPID TRANSIT

FINAL

Prepared by:

Totten Sims Hubicki Associates (1997) Limited **doing business as AECOM**
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

Date:

December 2008

AECOM
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

December 12, 2008 Project Number: 42-71256

Mr. Harold Sich
Associate
IBI Group
230 Richmond Street, 5th Floor
Toronto, Ontario
M5V 1V6

Dear Harold:

Re: STRUCTURAL ASSESSMENT FOR HIGHWAY 401 - MORNINGSIDE AVENUE UNDERPASS STRUCTURE FOR LIGHT RAPID TRANSIT

We are enclosing herewith two (2) copies of our structural assessment report as noted above.

Please advise if we could be of further assistance in the above regards.

Sincerely,
Totten Sims Hubicki Associates (1997) Limited doing business as AECOM

David LeBlanc, M.Eng., P.Eng.
Head, Structures Department
david.leblanc@aecom.com

DL:smb
Encl.
cc: File

Signature Page

Report Prepared By:	Report Reviewed By:
LICENSED PROFESSIONAL ENGINEER S. N. BALASUNDARAM Dec/08 PROVINCE OF ONTARIO	LICENSED PROFESSIONAL ENGINEER N. D. LeBLANC 15/12/08 PROVINCE OF ONTARIO
Selva Balasundaram, P.Eng., Senior Structural Engineer	David LeBlanc, M.Eng., P.Eng., Head, Structural Department

Executive Summary

AECOM was retained by IBI Group to investigate and confirm the feasibility of implementing a Light Rapid Transit (LRT) right-of-way (ROW) on the Morningside Avenue structure over Highway 401, specifically addressing the structural adequacy of the underpass structure, as well as long term maintenance and operational requirements. The intent is upon confirmation of the feasibility of the LRT ROW implementation on the structure, to obtain approval from the MTO during the environmental assessment phase in order to move forward with the project. It is recognized that that there are various design and contractual arrangements to be addressed in the subsequent project phases, and the TTC is committed to working with the MTO on these issues.

It is our understanding that this structure was designed to accommodate an ultimate 6-lane configuration for the Morningside Avenue, and in the interim provide tapers on the south approach to pick-up and lose a lane, and on the north approach have the additional lane develop and end at the intersection as a ramp lane and must right turn lane respectively.

An assessment of the existing Highway 401 – Morningside Avenue underpass structure has been carried out to determine if it can accommodate the proposed Scarborough - Malvern LRT designated ROW, including two lanes of traffic in each direction. The findings indicate that the new LRT ROW and two traffic lanes can be accommodated on the existing structure without a need for deck widening.

A detailed structural evaluation was also undertaken to investigate effects of additional loads due to LRT and its accessories. The comparison between the CL-625-ONT truck load and the assumed new LRT vehicle load shows that these loads are almost same. There is additional load on the bridge due to the weight of the LRT trackbed. It is observed that a conventional reinforced concrete track bed will require strengthening of the steel girders. The strengthening is required in the negative bending moment zones over the piers which could be undertaken by strengthening the compression flanges with additional plates. Alternatively if the trackbed load is reduced by use of light weight materials, the structure will be subjected to load effects similar or less than that due to current CHBDC loading conditions.

There are a number of operational and maintenance features which will need to be accommodated for the new LRT, including the provision of poles on the deck to power the trains, modifications to the waterproofing and paving on the deck to accommodate the track bed, provision for drainage, provision for expansions joints in the continuous rail. These considerations have been identified, and a number of standard techniques that have been adopted elsewhere are available for investigation during the preliminary and detail design phases of the project.

Our findings indicate that it is feasible to accommodate the proposed LRT right-of-way on the Highway 401 - Morningside Avenue underpass structure, without a need for deck widening. The girders could be strengthened to accommodate the additional load from a conventional concrete bed, or alternatively a light weight material track bed could be considered.

Table of Contents

List of Figures

Appendices

1. LOCATION

The underpass structure is located along Morningside Avenue where it intersects with Highway 401 as shown on the following Key Plan.

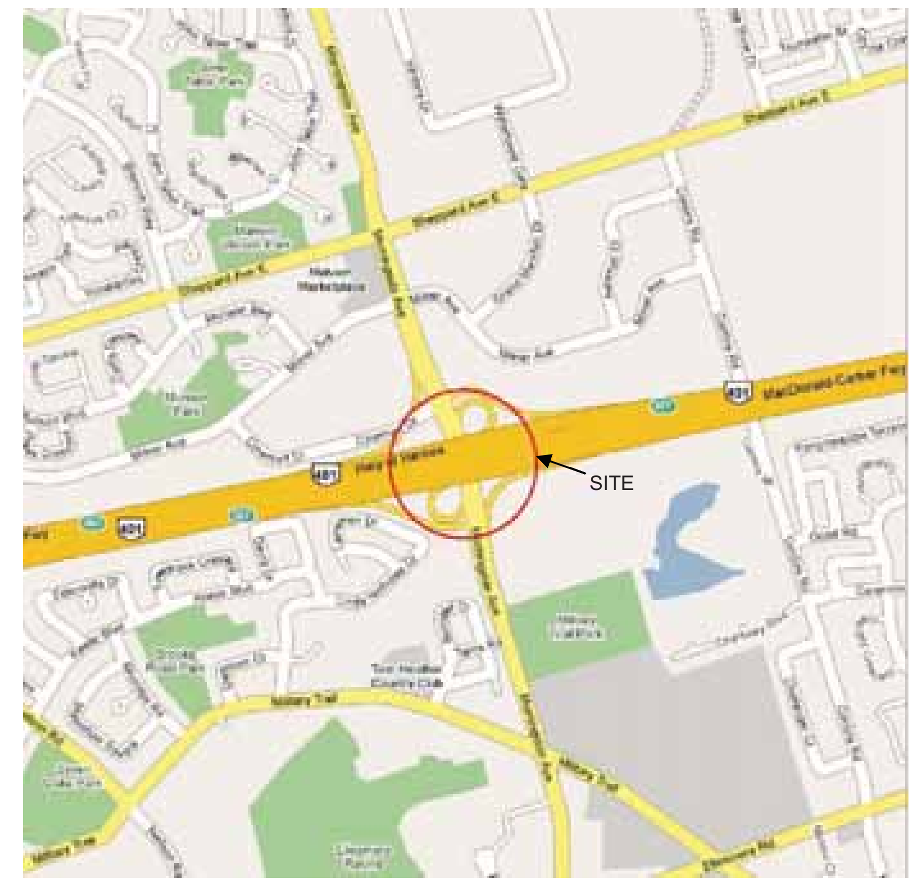


Figure 1. Key Plan

2. EXISTING STRUCTURE

The existing structure, which was constructed in about 1988, is a 133.35 m long three span (40.0m, 57.35m, 36.0m) structural steel trapezoidal box girder bridge with a 225 mm thick cast-in-place concrete deck and 90mm thick waterproofing and asphalt wearing surface. The bridge superstructure is supported on cast-in-place reinforced concrete abutments and piers founded on footings and constructed at normal to the road alignment.

The General Arrangement drawing of the existing structure is provided in Appendix A.

3. EXISTING CROSS SECTION

The cross section of the existing structure consists of the following:

East Barrier wall	0.450 m
Sidewalk	2.000 m
Ramp Lane	Varies
Traffic Lanes	3 x 3.500m
Median	2.000 m
Traffic Lanes	3 x 3.500m
Ramp Lane	Varies
Sidewalk	2.000 m
West Barrier wall	0.450 m

4. STRUCTURE GEOMETRY

It is our understanding that the Highway 401 – Morningside Underpass structure was originally designed to accommodate an ultimate 6-lane configuration for Morningside Avenue, and in the interim provided tapers on the south approach to pick-up and lose a lane, and on the north approach have the additional lane develop and end at the intersection as a ramp lane and must right turn lane respectively.

A preliminary assessment of the existing bridge geometry has been carried out, indicating the following:

- The width of the roadway at the bridge from gutter to gutter is approximately 31.0 m, which includes a 1.20 m wide median. The bridge can accommodate the required horizontal clearance for the 2 lanes of traffic in each and the new LRT designated right-of-way configuration. It is feasible to implement the LRT right-of-way geometrically.
- There are 2 lanes and an auxiliary lane in each direction (as previously mentioned, the additional lane in each direction was constructed to be consistent with the then planned ultimate 6-lane cross-section for Morningside Avenue.

- The maximum longitudinal slope of the bridge structure is 3.5%, which satisfies the assumed maximum slope of 5% for the new LRT Vehicle.

A preliminary general arrangement drawing showing the proposed LRT configuration on the Morningside Underpass structure is provided in Appendix B.

5. STRUCTURAL ASSESSMENT

The design loads that the existing structure has been designed to include the following:

Dead Loads:
The dead loads due to girder, deck, sidewalk, barrier walls, asphalt wearing surface, and light poles

Live Loads:
The original design live loads were based on Ontario Highway Bridge Design Code, 1983. While investigating the structure for the suitability of carrying the LRT vehicle, we have considered the requirements of the current Canadian Highway Bridge Design Code (CHBDC) CAN/CSA – S6-06. It is noted that the Gross load due to OHBD Truck was 700 kN, compared to the CHBDC CL-625-ONT Truck load of 625 kN. While the overall truck load has decreased in the most recent code, the new live load factor is higher than that specified in the OHBDC 1983.

Other Loads:
Other loads that need to be considered in the design of the structure include thermal, wind, braking etc. as specified in the code.

A structural assessment of the existing bridge has been carried for the following load conditions:

- CHBDC CAN/CSA - S6-06 CL-625-ONT Truck
- Proposed LRT Live load and additional loads due to conventional trackbed & accessories
- Proposed LRT Live load and additional loads due to lightweight trackbed & accessories

The results of the structural evaluation are summarized in Appendix C, and indicate that the superstructure will require strengthening if a conventional concrete trackbed is provided for the LRT. The results indicate the ultimate limit state (ULS) moments under LRT loading with a conventional concrete trackbed increase by approximately 15% in comparison with CHBDC loading for which the existing structures have been designed. The extent of overloading for the structure is summarized in Table – 1. The loads acting on substructure and foundation would be expected to increase by significantly less then this amount, in the range of 5 to 10%, if conventional concrete trackbed is adopted. It is unlikely that strengthening of the foundations will be required for this additional load, however, underpinning methods are available to strengthen the capacity of existing abutment and pier footings, if necessary.

The results of the structural evaluation indicates that if a light-weight polymer infill with a unit weight in the order of 2 to 4 kN/m3 is provided for the trackbed, strengthening of superstructure and substructure strengthening will not be required. It should be noted that, the TTC are investigating this technique for several bridges in the City of Toronto for the Transit City program.

A further option which could be considered would be to fix the rails directly to the concrete deck and eliminate the trackbed, in which case the structure is subjected to loads similar to that of the existing structure. However there are numerous maintenance and durability issues associated with fixing the rails directly to the deck, which could compromise the long term life of the structure, and this alternative is not recommended for further consideration.

6. MISCELLANEOUS STRUCTURAL DETAILS

There are a number of details associated with the LRT ROW which will require modification of the existing structure, and which will need to be detailed during the design phases of the project. A preliminary assessment of the impact of the LRT ROW on the structure has been carried out, and the following items will need to be addressed:

- Poles will be required on the deck to provide overhead power for the LRT. The forces due to poles supporting the catenaries and light poles will induce primarily localized effects. Pedestals and connections to deck slab will need to be provided and detailed appropriately.
- Expansion joints will need to be provided to minimize the effect of movement of the structure on the continuous welded rail. Expansion can be accommodated through combinations of rail anchors and bolted joints allowing for limited movements or special proprietary rail expansion joints.
- Ensure protection of structures and components from corrosion due to stray currents by appropriate method of grounding or coating reinforcements or insulating with a membrane below the trackbed.
- Proper detailing of waterproofing and paving where it abuts the LRT trackbed will be required to maintain the long term durability of the deck.
- As the existing roadway is on a symmetrical crest curve and the structure is approximately 134.2m long this structure does not require deck drains. However adequate drainage of the LRT right-of-way drainage will need to be addressed.

Long term maintenance and rehabilitation of the bridge deck and the LRT trackbed will be somewhat complicated by the LRT right-of-way. There are a number of alternatives available, with the simplest being that a temporary closure of the LRT ROW will be required during major rehabilitative works on the bridge, which extend for 4 to 6 months in duration, and local bus service be utilized. Alternatives and details will be developed in subsequent project phases.

The above identified miscellaneous structural details can be addressed with standard techniques that have been adopted elsewhere, and will be fully investigated during the preliminary and detail design phases of the project. The TTC is committed to working with the MTO on these issues.

- The maximum longitudinal slope of the bridge structure is 3.5%, which satisfies the assumed maximum slope of 5% for the new LRT Vehicle.

A preliminary general arrangement drawing showing the proposed LRT configuration on the Morningside Underpass structure is provided in Appendix B.

5. STRUCTURAL ASSESSMENT

The design loads that the existing structure has been designed to include the following:

Dead Loads:
The dead loads due to girder, deck, sidewalk, barrier walls, asphalt wearing surface, and light poles

Live Loads:
The original design live loads were based on Ontario Highway Bridge Design Code, 1983. While investigating the structure for the suitability of carrying the LRT vehicle, we have considered the requirements of the current Canadian Highway Bridge Design Code (CHBDC) CAN/CSA – S6-06. It is noted that the Gross load due to OHBD Truck was 700 kN, compared to the CHBDC CL-625-ONT Truck load of 625 kN. While the overall truck load has decreased in the most recent code, the new live load factor is higher than that specified in the OHBDC 1983.

Other Loads:
Other loads that need to be considered in the design of the structure include thermal, wind, braking etc. as specified in the code.

A structural assessment of the existing bridge has been carried for the following load conditions:

- CHBDC CAN/CSA - S6-06 CL-625-ONT Truck
- Proposed LRT Live load and additional loads due to conventional trackbed & accessories
- Proposed LRT Live load and additional loads due to lightweight trackbed & accessories

The results of the structural evaluation are summarized in Appendix C, and indicate that the superstructure will require strengthening if a conventional concrete trackbed is provided for the LRT. The results indicate the ultimate limit state (ULS) moments under LRT loading with a conventional concrete trackbed increase by approximately 15% in comparison with CHBDC loading for which the existing structures have been designed. The extent of overloading for the structure is summarized in Table – 1. The loads acting on substructure and foundation would be expected to increase by significantly less then this amount, in the range of 5 to 10%, if conventional concrete trackbed is adopted. It is unlikely that strengthening of the foundations will be required for this additional load, however, underpinning methods are available to strengthen the capacity of existing abutment and pier footings, if necessary.

The results of the structural evaluation indicates that if a light-weight polymer infill with a unit weight in the order of 2 to 4 kN/m3 is provided for the trackbed, strengthening of superstructure and substructure strengthening will not

Table - 1

Maximum (Minimum) Moment Forces of the Load Combinations:

Positive moment, Unit: KN.m

Location	Load Name	ULS1	ULS2	ULS3		Max. ULS	Mmax/Mr
Location No. 2	ON-625	19559.6	19000.2	19000.2		19559.6	49.7%
	LRT+Deck	21506.48	21069.62	21069.62		21506.5	54.7%
	LRT+LWF	17815.9	17379.1	17379.1		17815.9	45.3%
	Mr					39335.0	

Negative Moment, Unit: KN.m

Location	Load Name	ULS1	ULS2	ULS3		Max. ULS	Mmax/Mr
Location No. 3	ON-625	-28199.7	-28675.8	-28675.8		-28675.8	93.5%
	LRT+Deck	-36004.32	-36512.83	-36512.83		-36512.8	119.0%
	LRT+LWF	-28353.9	-28862.4	-28862.4		-28862.4	94.1%
	Mr					-30683.0	

Positive moment, Unit: KN.m

Location	Load Name	ULS1	ULS2	ULS3		Max. ULS	Mmax/Mr
Location No. 4	ON-625	25585.6	24896.4	24896.4		25585.6	70.4%
	LRT+Deck	31328.67	30753.11	30753.11		31328.7	86.2%
	LRT+LWF	24301.3	23725.7	23725.7		24301.3	66.8%
	Mr					36361.0	

Maximum (Minimum) Shear Forces of the Load Combinations:

Force Unit: KN

Location	Load Name	ULS1	ULS2	ULS3		Max. ULS	Vmax/Vr
Location No. 1	ON-625	2502.5	2427.4	2427.4		2502.5	51.0%
	LRT+Deck	2952.344	2883.7	2883.7		2952.3	60.2%
	LRT+LWF	2440.4	834.0	834.0		2440.4	49.7%
	Vr					4907.0	

Force Unit: KN

Location	Load Name	ULS1	ULS2	ULS3		Max. ULS	Vmax/Vr
Location No. 3	ON-625	3957.8	3870.9	3870.9		3957.8	51.0%
	LRT+Deck	5008.011	4924.612	4924.612		5008.0	64.6%
	LRT+LWF	3992.0	3908.6	3908.6		3992.0	51.5%
	Vr					7755.4	

Note: For locations of critical moments and shears, see analysis model in Appendix - C.

be required. To be noted, the TTC are investigating this technique for several bridges in the City of Toronto for the Transit City program.

A further option which could be considered would be to fix the rails directly to the concrete deck, in which case the structure is subjected to loads similar to that of the existing structure. However there are numerous maintenance and durability issues associated with fixing the rails directly to the deck, which could compromise the long term life of the structure, and this alternative is not recommended for further consideration.

6. MISCELLANEOUS STRUCTURAL DETAILS

There are a number of details associated with the LRT ROW which will require modification of the existing structure, and which will need to be detailed during the design phases of the project. A preliminary assessment of the impact of the LRT ROW on the structure has been carried out, and the following items will need to be addressed:

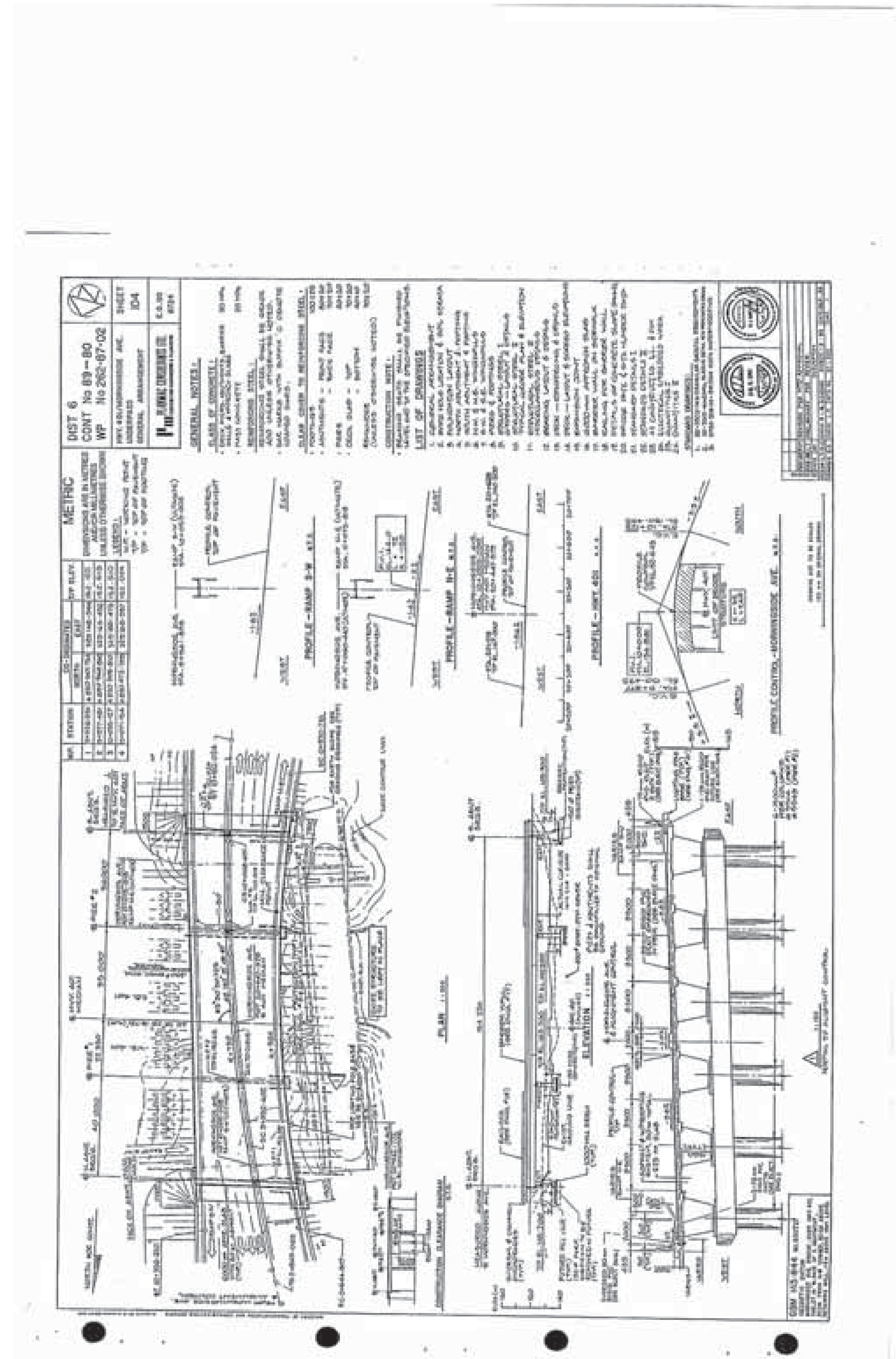
- Poles will be required on the deck to provide overhead power for the LRT. The forces due to poles supporting the catenaries and light poles will induce primarily localized effects and pedestals and connections to deck slab will need to be provided and detailed appropriately.
- Expansion joints will need to be provided to minimize the effect of movement of the structure on continuous welded rail. Expansion can be accommodated through combinations of rail anchors and bolted joints allowing for limited movements or special proprietary rail expansion joints.
- Ensure protection of structures and components from corrosion due to stray currents by appropriate method of grounding or coating reinforcements or insulating with a membrane below the trackbed.
- Proper detailing of waterproofing and paving where it abuts the LRT trackbed will be required to maintain the long term durability of the deck.
- As the existing roadway is on a symmetrical crest curve and the structure is approximately 134.2m long this structure does not require deck drains. However adequate drainage of the LRT right-of-way drainage will need to be addressed.

Long term maintenance and rehabilitation of the bridge deck and the LRT trackbed will be somewhat complicated by the LRT right-of-way. There are a number of alternatives available, with the simplest being that a temporary closure of the LRT ROW will be required during major rehabilitative works on the bridge, which extend for 4 to 6 months in duration, and local bus service be utilized. Alternatives and details will be developed in subsequent project phases.

The above identified miscellaneous structural details can be addressed with standard techniques that have been adopted elsewhere, and will be fully investigated during the preliminary and detail design phases of the project. The TTC is committed to working with the MTO on these issues.

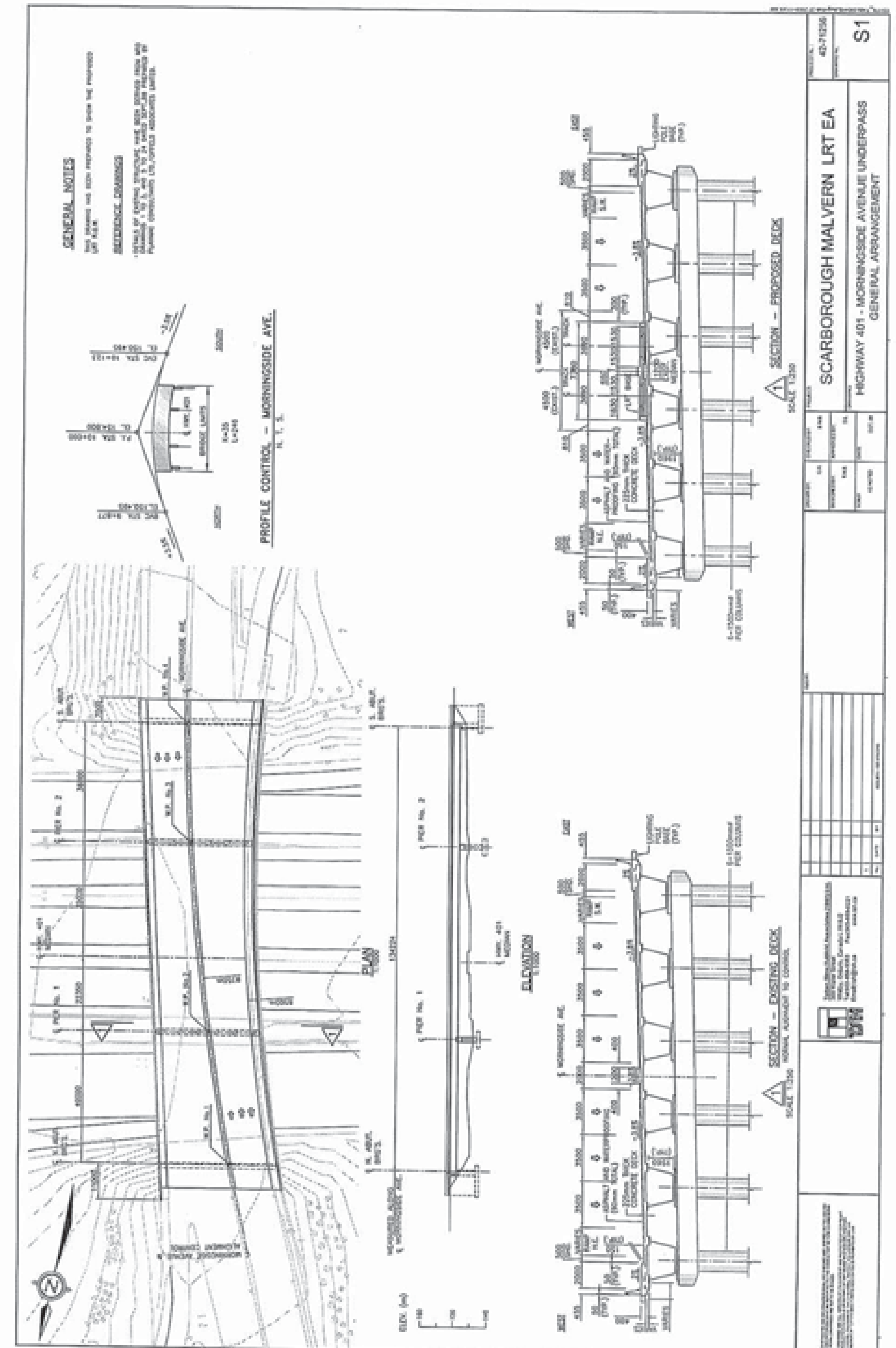
Appendix A

General Arrangement Drawing – Existing Structure



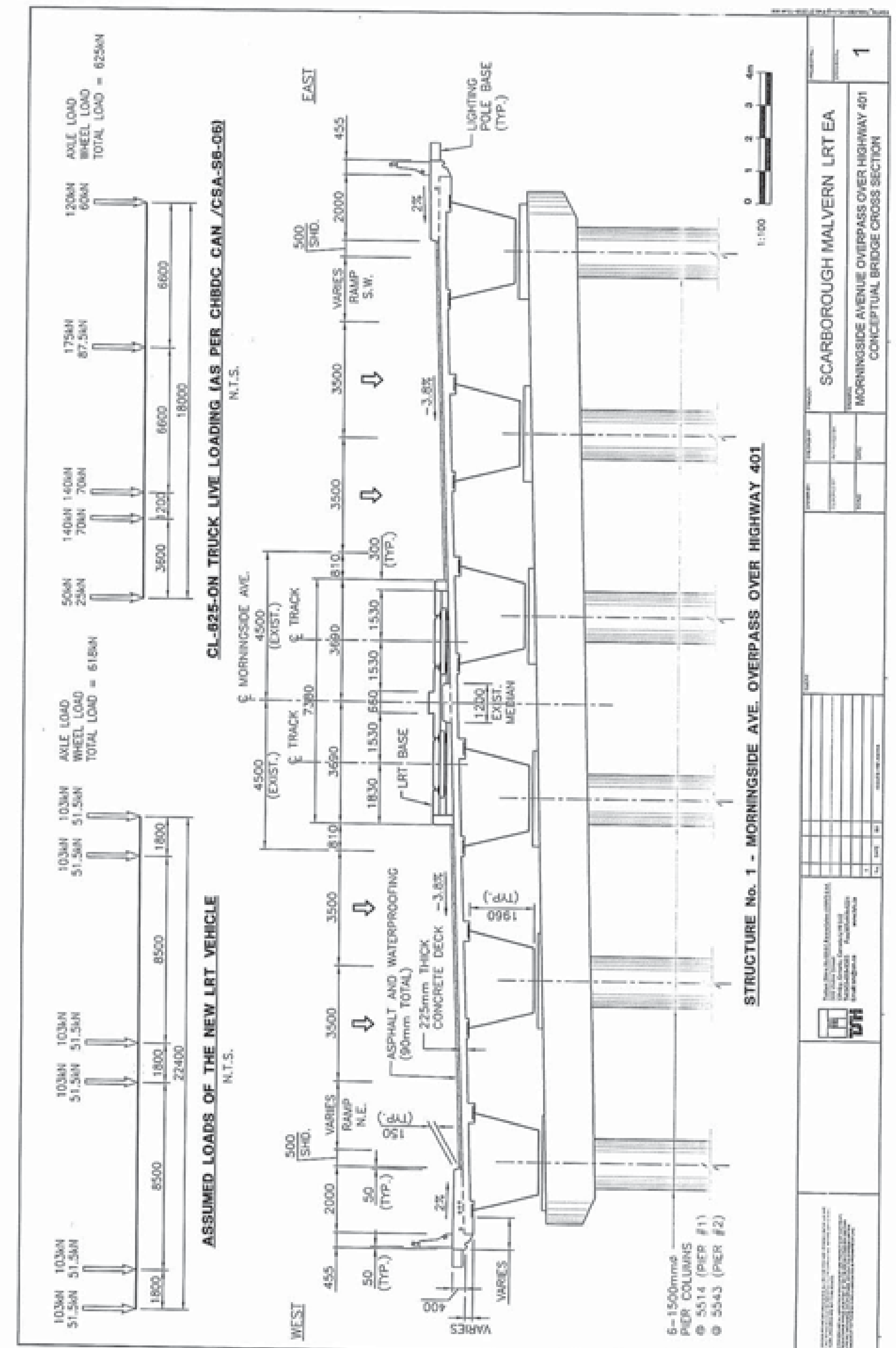
Appendix B

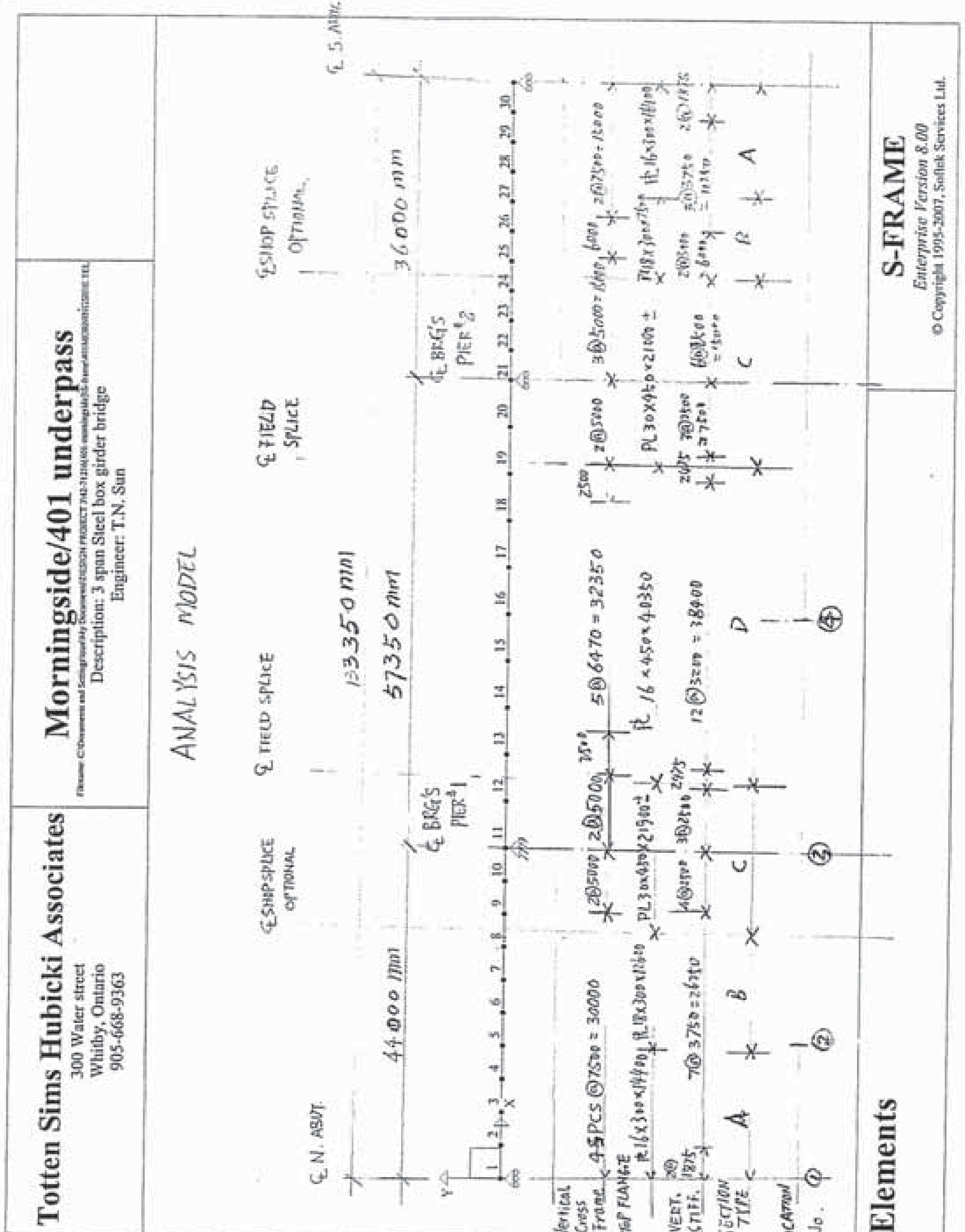
General Arrangement Drawing – Proposed Deck Cross Section with LRT Tracks



Appendix C

Details of Structural Evaluation





TSH engineers architects planners

Project: Steel box girder bridge inspection

Page

Date 06-Oct-08

Project No. 42-71256

Structure: 401-Morningside underpass

Cal. by TNS

Check by

1) General Information

Structure Type : Slab On Girder
Spans : 3
Length of Longest Span : 57350 mm
Width of Span : 37425 mm
Hwy Class : A
Deck Thickness : 225 mm
Asphalt & Waterproofing : 90 mm
Girder Material : Steel
Girder Type : Box
Girder Spacing (S): 6150 mm
Girders: 6

Overhang (right) : 1850 mm
Overhang (left) : 1825 mm
Sidewalk Width : 2000 mm
Sidewalks : 2
Barrier Wall / Parapet Wall Width : 455 mm   # of barriers : 2
Area : 0.2300 m^2
Total # Girders 6
Top width of girder : 3000

2) Specified Material Properties

Deck Slab Concrete
f'c : 30 MPa
Unit Weight : 24.0 kN/m^3
2400.0 kg/m^3
Structural Steel
Fy : 350 MPa
Unit Weight : 77.0 kN/m^3

Prestressing Steel
fpu : N/A
Area of 1 strand : N/A

Reinforcing Steel
fy : 400 MPa
Unit Weight : 77 kN/m^3

Asphlat 23.5 kN/m^3

3) Moduli of Elasticity

Slab
Ec = 24870 MPa   CL - 8-4.1.7

Girders
Eg = 200000 MPa
Gg = 77000 MPa

n = Modular Ratio = Eg/Ec = 8.04

4) Effective Slab Width CL 5.8.2

Lpos = 0.6 L = 34410 mm

Interior1 :	Exterior1:
B = 1575.00 mm	B = 1825.00 mm
L/B = 21.85	18.85
> 15, therefore Be = B	> 15, therefore Be = B
Be = 1575 mm	1825 mm
Therefore, Be1 = **1825 mm**	

Interior2 :	Exterior2:
B = 1575.00 mm	B = 1825.00 mm
L/B = 21.85	18.85
> 15, therefore Be = B	> 15, therefore Be = B
Be = 1575 mm	1825 mm
Therefore, Be2 = **1825 mm**	

Flange width (external girder) = **6400 mm** for section property calculations

Flange width (internal girder) = **6650 mm** for section property calculations

Live Load Distribution Factors (Section 5 of CHBDC) - Interior Girder

CL 5.7.1.2.2

Span 1 = 40.00 m

Span 2 = 57.35 m

Span 3 = 36.00 m

Fm = S*N/(F*(1+μ*Cf/100) **>= 1.05**

for n <=4,	F, Cf	from table A5.7.1.2.1
for n >4,	$F=F_4/2.8$	F_4 = F for n = 4

We = 37425 mm

 # design design lanes = n = 8 table 3.8.2

We = Wc/n = 4.68 m

μ = min((We - 3.3)/0.6, 1) = 1.0

girder spacing S = 6.150 m

over hang = 1.850 m Factor = 1.00

D_{VE} = 1.055 m

Positive Moment

Span 1 - SLS & ULS

fig. A5.1(a) L = 32.00 m

n = 8

N = # girders = 6

table 3.8.4.2 MLRF = RL = 0.550

$\alpha = n/N^*R_L$ = **0.733**

F = F4*n*RL/2.8

β = 5.63

	Exterior	Interior
read F4	10.562 m	10.562 m
F =	16.598 m	16.598 m
Cf =	4.75 %	4.75 %
F(1+μCf/100) = Dd =	17.386 m	17.386 m
Fm =SN/Dd =	2.122 m	2.122 m
Dist Factor = α.Fm =	**1.56**	**1.56**

Support 1 - SLS & ULS

fig. A5.1(a) L = 19.47 m

n = 8

N = # girders = 6

table 3.8.4.2 MLRF = RL = 0.550

$\alpha = n/N^*R_L$ = **0.733**

F = F4*n*RL/2.8

β = 5.63

	Exterior	Interior
read F4	10.562 m	10.562 m
F =	16.598 m	16.598 m
Cf =	4.75 %	4.75 %
F(1+μCf/100) = Dd =	17.386 m	17.386 m
Fm =SN/Dd =	2.122 m	2.122 m
Dist Factor = α.Fm =	**1.56**	**1.56**

Span 2 - SLS & ULS

fig. A5.1(a) L = 45.88 m

n = 8

table 3.8.4.2 MLRF = RL = 0.550

N = # girders = 6.000

$\alpha = n/N^*R_L$ = **0.733**

F = F4*n*RL/2.8

β = 4.36

	Exterior	Interior
read F4	11.448 m	11.448 m
F =	17.990 m	17.990 m
Cf =	7.28 %	7.28 %
F(1+μCf/100) = Dd =	19.300 m	19.300 m
Fm =SN/Dd =	1.912 m	1.912 m
Dist Factor = α.Fm =	**1.40**	**1.40**

Shear for ULS & SLS

n = 8

MLRF = RL = 0.550

N = 6

$\alpha = n/N^*R_L$ = **0.733**

F or F4 = 11.2 CL. 5.7.1.5

F reduction factor = 1.00 CL 5.7.1.4.1

Fv = S*N/F

F = 17.600 m Table 5.7.1.4.1

Fv = 2.097

Dist Factor = α Fv = 1.54

4) Effective Slab Width CL 5.8.2

Lpos = 0.6 L = 34410 mm

Interior1 :	Exterior1:
B = 1575.00 mm	B = 1825.00 mm
L/B = 21.85	18.85
> 15, therefore Be = B	> 15, therefore Be = B
Be = 1575 mm	1825 mm
Therefore, Be1 = **1825 mm**	

Interior2 ;	Exterior2:
B = 1575.00 mm	B = 1825.00 mm
L/B = 21.85	18.85
> 15, therefore Be = B	> 15, therefore Be = B
Be = 1575 mm	1825 mm
Therefore, Be2 = **1825 mm**	

Flange width (external girder) = 6400 mm for section property calculations
Flange width (internal girder) = 6650 mm for section property calculations

Live Load Distribution Factors (Section 5 of CHBDC) - Interior Girder
CL 5.7.1.2.2

Span 1 = 36.00 m
Span 2 = 57.35 m
Span 3 = 40.00 m

Fm = S*N/(F*(1+μ*Cf/100) >= 1.05

for n <=4, F, Cf from table A5.7.1.2.1
for n >4, $F=F_4/2.8$ F_4 = F for n = 4

Wc = 37425 mm
 # design design lanes = n = 8 table 3.8.2
We = Wc/n = 4.68 m
μ = min((We - 3.3)/0.6, 1) = 1.0
girder spacing S = 6.150 m
over hang = 1.850 m Factor = 1.00
D_{VE} = 1.055 m

Positive Moment

Span 1 - SLS & ULS

fig. A5.1(a) L = 28.80 m
n = 8
N = # girders = 6
table 3.8.4.2 MLRF = RL = 0.550
α =n/N*R_L = **0.733**
F = F4*n*RL/2.8
β = 6.25

	Exterior	Interior
read F4	10.125 m	10.125 m
F =	15.910 m	15.910 m
Cf =	3.50 %	3.50 %
F(1+μCf/100) = Dd =	16.467 m	16.467 m
Fm =SN/Dd =	2.241 m	2.241 m
Dist Factor = α.Fm =	**1.64**	**1.64**

TSH engineers architects planners
Project: Steel box girder bridge inspection

Structure: 401-Morningside underpass
Cal. by TNS

Support 1 - SLS & ULS

fig. A5.1(a) L = 18.67 m
n = 8
N = # girders = 6
table 3.8.4.2 MLRF = RL = 0.550
α =n/N*R_L = **0.733**
F = F4*n*RL/2.8
β = 6.25

	Exterior	Interior
read F4	10.125 m	10.125 m
F =	15.910 m	15.910 m
Cf =	3.50 %	3.50 %
F(1+μCf/100) = Dd =	16.467 m	16.467 m
Fm =SN/Dd =	2.241 m	2.241 m
Dist Factor = α.Fm =	**1.64**	**1.64**

Span 2 - SLS & ULS

fig. A5.1(a) L = 45.88 m
n = 8
table 3.8.4.2 MLRF = RL = 0.550
N = # girders = 6.000
α =n/N*R_L = **0.733**
F = F4*n*RL/2.8
β = 3.92

	Exterior	Interior
read F4	11.753 m	11.753 m
F =	18.470 m	18.470 m
Cf =	8.15 %	8.15 %
F(1+μCf/100) = Dd =	19.976 m	19.976 m
Fm =SN/Dd =	1.847 m	1.847 m
Dist Factor = α.Fm =	**1.35**	**1.35**

Shear for ULS & SLS

n = 8
MLRF = RL = 0.550
N = 6
α =n/N*R_L = **0.733**
F or F4 = 11.2 CL. 5.7.1.5
F reduction factor = 1.00 CL 5.7.1.4.1
Fv = S*N/F
F = 17.600 m Table 5.7.1.4.1
Fv = 2.097

Dist Factor = α Fv = 1.54

Distribution Factors

Final Result

	SLS/ULS		
	+ve Momen	-ve Momen	Shear
Span 1	1.556	1.556	1.538
span 2	1.402	1.402	1.538
Span 3	1.643	1.643	1.538

ESTIMATED ***PER GIRDER* LOADS**

1) Girder Selfweight Composite — will be considered separately for the following spreadsheet

2) CIP Deck Slab

	Span1: 32.7 kN/m	Span 2: 29.2 kN/m	Span 3: 33.2 kN/m
t =	225 mm	225 mm	225 mm
S_{girder} =	6050 mm	5400 mm	6150 mm
γ =	24.0 kN/m3	24.0 kN/m3	24.0 kN/m3

3) Waterproofing & Asphalt — 13.0 kN/m
(Load Case 1 in S-Frame)

t = 90 mm
S_{girder} = 6150 mm
γ = 23.5 kN/m3

4) Barrier Wall — 5.3 kN/m
(Load Case 1 in S-Frame)

A = 0.7 m2
γ = 24.0 kN/m3
girders = 6

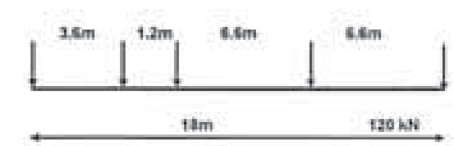
5) Thermal Gradient (vertical Expansion/Contraction)
(Load Case 6&8 in S-Frame)

positive gradient: 10.0 degrees
negative gradient: -5.0 degrees

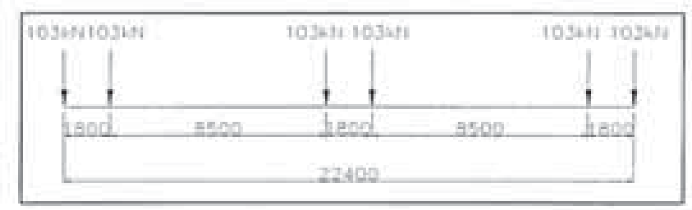
Type of structure = C — CL 3.9.3
Depth of Structure = 2.0 m

		Gradient	
Condition (+ve) =	Summer	10	figure 3.9.4.1
Condition (-ve) =	Winter	-5	figure 3.9.4.1

6) Live Load Appendix A3.4 — CL-625-ONT
(Load Case 20 in S-Frame)



7) Live Load Appendix A3.4 — TTC LRT



8) Extra LRT deck load

Extra Concrete Deck Weight

Width=	7380 mm	1) Weight=	79.7 kN/m
Depth(Ave.)=	450 mm	(varies from 300 to 600mm)	
Deensity=	24 kN/m^3		

Deduct the Asphalt weight:

Width=	7380 mm	2) Weight=	15.6 kN/m
Depth=	90 mm		
Deensity=	23.5 kn/m^3		
Extra LRT deck load=		[1) - 2)]/2=	32.0 kN/m

If light weight fill is used

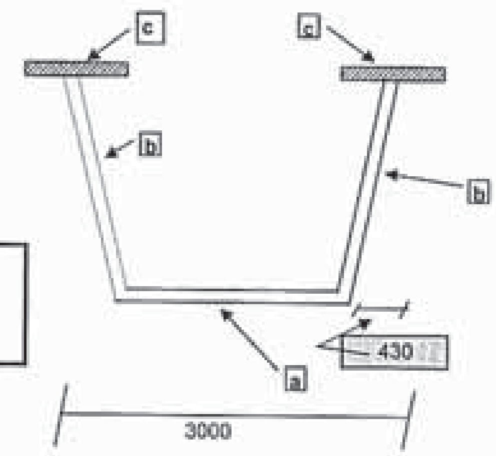
1) Weight=		16.6 kN/m
Deensity=	5	kN/m^3
2) Weight=		15.6 kN/m
3) Curb w.=		4.3 kN/m
[1) +2) - 3)]/2=		2.7 kN/m

File Name: Loads-401-Deck450
Sheet Name: Loading

TSH
Morningside/Highway 401 Underpass
NAKED GIRDER SECTION PROPERTIES

Section C (midspan)

Project No.	42-71256	**Page**	
		Date	15-Oct-08
Cal. by	T.N. Sun	**Check by**	



Total depth of girder = 2008 mm

a:			
width =	2180	mm	
thickness =	18	mm	
area =	39240	mm^2	
Yc =	9	mm	measured to bottom of girder
d =	853.8	mm	
Ic * 10^6 =	1.1	mm^4	

Total Area =	*128665.19*	mm^2
Yg =	*862.80*	mm
Total Moment of Inertia *10^6 =	*85522.05*	mm^4

b:			
width =	14	mm	
vert. thickness =	1960	mm	
inclined width =	14.3	mm	
inclined length =	2006.6		
area =	28092.59689	mm^2	
Yc =	998	mm	measured to bottom of girder
d =	135.2	mm	
θ =	0.22	rad	
Ic * 10^6 =	8993.4	mm^4	

* Note: Red designates that the parameter is calculated by spreadsheet.

c:			
width =	450	mm	
thickness =	30	mm	
area =	13500	mm^2	
Yc =	1993	mm	measured to bottom of girder
d =	1130.2	mm	
Ic * 10^6 =	1.0	mm^4	

Long. Stiff. for bottom flange:

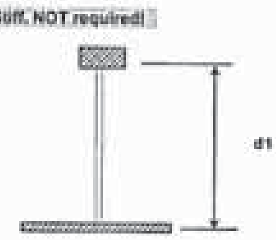
No. of stiff. =	3		
Section Type =	WT125x16.5		
Area =	2080	mm^2	
Ic * 106 =	2.85	mm^4	
height =	129	mm	
ycg of T =	22.7	mm	
Yc =	124.3	mm	measured to bottom of girder
d =	738.5		

Naked Girder Moment Resistance
Section C (+ve Moment Region)

Table 10-8.2.1

compression flange (b/t)	7.5	web (h/w)	143.3	
525 / SQRT (Fy)	28.1	1700 / SQRT (Fy)	90.9	
670 / SQRT (Fy)	35.8	1900 / SQRT (Fy)	101.6	
	Class 2		Class 4	
		3150 / SQRT (Fy)	168.4	Web Long. Stiff. NOT required!

Simplified Half Section



Top Flange

width = 450.0 mm	Ycx = 1984.0 mm	
thickness = 30.0 mm	Ycy = 1090.0 mm	
Area = 13500.0 mm^2	dx = 1091.1 mm	
Ix = 1.01E+06 mm^4	dy = 0.0 mm	
Iy = 2.28E+08 mm^4		

Web

width = 1960.0 mm	Ycx = 989.0 mm	
thickness = 14.3 mm	Ycy = 1090.0 mm	
Area = 28092.6 mm^2	dx = 96.1 mm	
Ix = 8.99E+09 mm^4	dy = 0.0 mm	
Iy = 4.81E+05 mm^4		from bot/girder

Bot Flange

width = 2180.0 mm		CGx = 892.9	mm
1/2 thickness = 9.0 mm	Ycx = 4.5 mm	CGy = 1090	mm
Area = 19620.0 mm^2	Ycy = 1090.0 mm	Global Ix = 4.08E-02	m^4
Ix = 1.32E+05 mm^4	dx = 888.4 mm	Global Iy = 8.00E-03	m^4
Iy = 7.77E+09 mm^4	dy = 0.0 mm	J = 8.50346E-06	m^4

Total Area = 61213 mm^2

Es = 2.00E+08	KPa	Fy =	350000	KPa			
Gs = 7.70E+07	KPa	Bottom Sx =	0.0457	m^3		Bx =	-1.62
Lu = 5.0	m	Top Sx =	0.0369	m^3		B1 =	-28.69
ρ = 0.0285		ω2 =	1.0	(UDL)		B2 =	136.74
d1 = 1979.5	m						
Cw = 0.00086724	m^6	Mu =	41,044.06	KN-m			
φs= 0.95		My =	12,913.50	KN-m			
		0.67*My =	8,652.05	KN-m			
		φsMy =	12,267.83	KN-m			

Mr = 12267.83 KN-m

Identification of Web = Thin   therefore
Moment Resistance Has To be Reduced

Moment Reduction CL 10-9.4.4

Mr =	12267.83	KN-m		
Aw =	28093	mm^2	**Mr' = 10399.15 KN-m**	
Af =	13500	mm^2		
h =	1960.0	mm	Red'n Factor =	15.23 %
w =	14.3	mm		
Mf =	12267.83	KN-m		
φ =	0.95			
S =	0.03690	m^3		

Naked Girder Moment Resistance
Section C (-ve Moment Region)

Fy =	350	MPa			
compression flange (b/t)	242.2	web (h/w)	143.3		
525 / SQRT (Fy)	28.1	1700 / SQRT (Fy)	90.9		
670 / SQRT (Fy)	35.8	1900 / SQRT (Fy)	101.6		
Class 4 flange, Unacceptable I – Cl. 10.9.4, Bot Flange Need Stiffening			Class 4		
		3150 / SQRT (Fy)	168.4	Web Long. Stiff. NOT required!	

Stiffened Bottom Flanges Cl. 10-11.2.3

t = 18 mm
No. of stiff. = 3
b = 2180
bs = 545 mm
bs / t = 30

Properties of stiffener

Section Type	WT125x16.5		
area =	2080	mm^2	
d =	129	mm	
Ixx =	2.85	$\times 10^6\ mm^4$	
y =	22.7	mm	IF Tr.Stif. Used
I_s =	26	$\times 10^6\ mm^4$	a=2500 mm
I_s / bs =	48.4	$\times 10^4\ mm^3$	b=2180 mm
			K2=3.145
$\varepsilon = I_s/(b_s t^3)$ =	8.29		Is(min)=25.4 $\times 10^6\ mm^4$
$\varepsilon = 0.125 k1^2$ => k1 =	N/A	for n = 1	OK
$\varepsilon = 0.07 k1^3 n^4$ => k1 =	1.14	for n > 1	It(min)=15.3 $\times 10^6\ mm^4$
k1 =	1.14		It=27.0 $\times 10^6\ mm^4$
k2 =	3.15		OK

Class 3 flange, OK

255*sqrt(k1/Fy) = 24.2
550*sqrt(k1/Fy) = 52.1
cs =[550sqrt(k2)-(bs/t)*sqrt(Fy)] / (295*sqrt(k2)) = 0.782

ϕ_s = 0.95
Fcr = 341.3 MPa

ϕFcr = 324.2 MPa

ϕ_s = 0.95
Fy = 350 MPa
Bottom Sx = 0.0991 m^3 I/y_{bot}
Top Sx = 0.0747 m^3 I/y_{top}

Top Flange - Tension

$Mr = \phi_s * Fy * S_{top}$ = 24830.7 kN.m for full girder

Bottom Flange - Compression

$Mr = \phi_s * Fcr * S_{bot}$ = 32135.0 kN.m for full girder

Girder C Composite Section Properties

f'c = 30 Mpa
Ec = 27386.128 Mpa
Es = 200000 Mpa
n = 7.30

	3n	1n
Eff. flange width Be =	6150.0 mm	
Eq. Steel Slab width Beq =	280.7 mm	842.1 mm
Slab thickness =	225.0 mm	
Hounch Depth =	0.0 mm	
Eq. Steel Slab thick. teq =	30.8 mm	

Total Depth of Composite Girder = 2233.0 mm

Positive Moment Region

3n

	Area mm^2	Yc (from bottom of girder) mm	d mm	Ic * 10e6 mm^4
Eq Steel Slab	63159.26	2120.5	843.59	266.4531
Girder	128665.19	862.80	414.10	85522.05
Sums	191824.45			85788.5
		S_{bot} (m^3)	S_{top} (m^3)	
Composite Yg =	*1276.9 mm*	0.1197	0.1566	
Composite Ic *10e6 =	*152799.6 mm^4*			

Positive Moment Region

1n

	Area mm^2	Yc (from bottom of girder) mm	d mm	Ic * 10e6 mm^4
Eq Steel Slab	189477.77	2120.5	508.65	799.3594
Girder	128665.19	862.80	749.05	85522.05
Sums	318142.97			86321.41
		S_{bot} (m^3)	S_{top} (m^3)	
Composite Yg =	*1611.9 mm*	0.1288	0.3341	
Composite Ic *10e6 =	*207534.5 mm^4*			

Negative Moment Region concrete in tension neglected

	Area mm^2	Yc (from bottom of girder) mm	d mm	Ic * 10e6 mm^4	
Top Rebars	14192.31	2133	1109.10	0.154416	Assumed 20M @ 130 spc.
Bot Rebars	4730.77	2078	1054.10	0.077208	Assumed 15M @ 260 spc.
Closed Box Eq. Steel Slab	0.00	0.00	0.00	0.00	
Girder	128665.19	862.80	161.10	85522.05	
Sums	147588.27			85522.28	
		S_{bot} (m^3)	S_{top} (m^3)		
Composite Yg =	*1023.9 mm*	0.1080	0.0923		
Composite Ic *10e6 =	*111575.9 mm^4*				

Composite Girder Moment Resistance - Section C
Positive Moment Region

b_e	6150 mm	f'c =	30 MPa
t_{slab} =	225 mm	Ec =	24870 MPa
$A'_{s-top\ only}$	14192 mm^2	Es =	200000 MPa
t_c	30 mm	n =	8.04
d_b	2008 mm	Fy =	350 MPa
COV_{bay} = d's =	78 mm	fy =	400 MPa
t_{bf}	18 mm	ϕc =	0.75
A_s^{girder}	128665 mm^2	ϕs =	0.95
t_{haunch} =	0.0 mm	ϕr =	0.85

CL 10-10.5.2

1) Assumed a = 225 mm   *******keep changing 'a' until C1 = C2, but a <= tc*******

C1 = Cc+Cs = Total compression force
Cc = 0.85*ϕc*f'c*be*a = 26464 kN   Cc =compression force in concrete in slab
e_c = 1117 mm

Cs = ϕr*A's$_{top}$*fy   Cs = compression force in steel reinforcement in slab
Cs = 4825 kN
e_s = 1152 mm

C1 = 31290 kN

C2 = ϕs*As*Fy = Total Tension Force
C2 = 42781 kN   Compression < Tension, a = tc, Mr = Cc*ec+Cs*es+C"e'
NA is in the steel girder !

Moment Resistance

NA is in the Steel Girder, NOT APPLICABLE !
Mr = N/A for a full girder

2) Assumed a = 244.2 mm   ************************ Start with 'a' > tc ************************
tolerance = 0.00

C = Cc+Cs+C' = Total compression force
Cc = 0.85*ϕc*f'c*be*tc = 26464 kN   compression in concrete slab
e_c = 1434 mm

Cs = ϕr*A's$_{top}$*fy   compression in steel reinforcement
Cs = 4825 kN
e_s = 1469 mm

C1 = Cc + Cs = 31290 kN

C' = 0.5(ϕs*As*Fy - C1)   compression force in STEEL GIRDER
C' = 5746 kN
Y_{tc} = 19 mm   depth of compression block for steel section
e' = 1337 mm

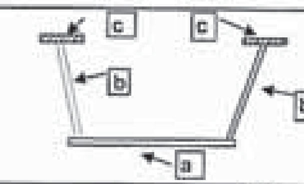
C = 37035.390 kN

T = ϕs*As^{tens}*Fy = Total Tension Force (from tension area in steel girder)
T = 37035.390 kN
Compression = Tension, a =
NA is in the flange of the Steel Girder !

Prepared By: Taining Sun   Morningside/Highway 401   10/15/2008 2:24 PM

Calculating Centroid of Tension Structural Steel

NA Location from top girder = 19 mm
$t_{top-flange}$ = 11 mm
h_{web} = 1960.0 mm
$t_{bot-flange}$ = 18 mm
Depth of section of girder in tension = 1988.8 mm

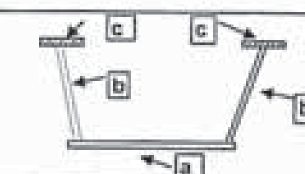


a:		b:		c:	
width =	2180.0 mm	width =	14.0 mm	width =	450.0 mm
thickness =	18.0 mm	vert. thickness =	1960.0 mm	thickness =	10.8 mm
area =	39240.0 mm^2	inclined width =	14.3 mm	area =	4859.7 mm^2
Yc (from bot of girder) =	9.0 mm	inclined length =	1960.0 mm	Yc =	1983.4 mm
d =	677.6 mm	area =	28092.6 mm^2	d =	1296.8 mm
Ic * 10^6 =	1.1 mm^4	Yc =	998.0 mm	Ic * 10^6 =	0.0 mm^4
		d =	311.4 mm		
		θ =	0.22 rad		
		Ic * 10^6 =	8993.4 mm^4		

Tension Area of Girder = 111384.6 mm^2   As^{tens}
Yg (from top of section) = 1302.2 mm
Yg (from bot of section) = 686.6 mm
Ix *10^6 = 59779.5 mm^4

Calculating Centroid of Compression Structural Steel

NA Location from top girder = 19 mm
$t_{top-flange}$ = 19.2 mm
h_{web} = 0.0 mm
$t_{bot-flange}$ = 0 mm
Depth of section of girder in compression = 19.2 mm



a:		b:		c:	
width =	2180.0 mm	width =	14.0 mm	width =	450.0 mm
thickness =	0.0 mm	vert. thickness =	0.0 mm	thickness =	19.2 mm
area =	0.0 mm^2	inclined width =	14.0 mm	area =	8640.3 mm^2
Yc (from bot of girder) =	0.0 mm	inclined length =	0.0 mm	Yc =	9.6 mm
d =	35.3 mm	area =	0.0 mm^2	d =	-25.7 mm
Ic * 10^6 =	0.0 mm^4	Yc =	0.0 mm	Ic * 10^6 =	0.3 mm^4
		d =	-35.3 mm		
		θ =	0.00 rad		
		Ic * 10^6 =	0.0 mm^4		

Compr'n Area of Girder = 23520.8 mm^2   A^{comp}
Yg (from top of section) = -18.1 mm
Yg (from bot of section) = 35.3 mm
Ix *10^6 = 51.9500 mm^4

SECTION C (AT PIERS)

Moment Resistance

Mr = Cc*ec+Cs*es+C' *e' *******note that Cc is based on a = tc (from part 1)

Mr = 52719 kN.m for full a girder

Composite Girder Moment Resistance

Negative Moment Region

CL 10-10.5.3

For negative moment region you can use the same flexural resistance as the naked girder alone (concrete in tension (in slab) has cracked and is neglected)

S_{bot} = 0.1090 m^3

S_{top} = 0.0923 m^3

Mr (for a full girder) = 35328 kN.m Bottom Flange - Compression

30683 kN.m Top Flange - Tension

Mf= 28676 kN.m Mf/Mr= 93%

Shear Design - CL 10-9.5.1

Transverse Intermediate Stiffeners CL 10-9.7

3- Design - End Panel

Vf_{DL} = 0.00 kN

Vf_{SDL} = 0.00 kN

Vf_{LL} = 1978.90 kN

Vf = 1978.9 kN Vf/Vr= 158%

Provide Transverse Web Stiffeners !!

Detemining Spacing 'a' of Stiffeners

SECTION 1

a = 2350 mm 'a' is OK

a/h = 1.17 Kv = 8.26

SQRT(Kv/Fy) = 0.1536

X1 = 502*SQRT(Kv/Fy) = 77.1

X2 = 621*SQRT(Kv/Fy) = 95.4

	Fcr [MPa]	Ft [MPa]	Fs [MPa]
h/w <= X1 =	108.8	0.0	108.8
h/w <= X2 =	108.8	52.5	161.2
h/w > X2 =	72.3	73.0	145.3
		CL 10-9.5.1	

Therefore,

Fs = 145.3 MPa

ϕs = 0.95

Aw = 28093 mm^2 shear area of one web

Vr = 3877.7 kN factored shear resistance

Vf = 1978.9 kN

Web Stiffeners spacing is OK Vf/Vr= 51%

Design of Transverse Intermediate Stiffeners CL 10-9.7.2

(b/t)max = 200/sqrt(Fy) = 10.69

a = 2350 mm

w = 14 mm

h/a = 0.9

j = 0.50

min I =aw³j = 3.22×10^6 mm⁴

Vf/Vr = 0.51

C = 0.644

D = 2.4 single plate stiffeners

Y = Fy/Fys = 1

min As = -420.1 mm² < 0, web can develope compressive resistance

b_{stiff}^{min} = 117 mm CL 10-9.7.4

t_{stiff} = 18 mm

b_{stiff} = 210 mm ***OK***

b/t = 11.67 ***NOT OK***

As = 3780.0 mm^2 ***OK***

I = 61.31 $\times 10^6$ mm⁴ ***OK***

Table - 1

Maximum (Minimum) Moment Forces of the Load Combinations:

Positive moment, Unit: KN.m

	Load Name	ULS1	ULS2	ULS3		Max. ULS	Mmax/Mr
Location No. 2	ON-625	19559.6	19000.2	19000.2		19559.6	49.7%
	LRT+Deck	21506.48	21069.62	21069.62		21506.5	54.7%
	LRT+LWF	17815.9	17379.1	17379.1		17815.9	45.3%
	Mr					39335.0	

Negative Moment, Unit: KN.m

	Load Name	ULS1	ULS2	ULS3		Max. ULS	Mmax/Mr
Location No. 3	ON-625	-28199.7	-28675.8	-28675.8		-28675.8	93.5%
	LRT+Deck	-36004.32	-36512.83	-36512.83		-36512.8	119.0%
	LRT+LWF	-28353.9	-28862.4	-28862.4		-28862.4	94.1%
	Mr					-30683.0	

Positive moment, Unit: KN.m

	Load Name	ULS1	ULS2	ULS3		Max. ULS	Mmax/Mr
Location No. 4	ON-625	25585.6	24896.4	24896.4		25585.6	70.4%
	LRT+Deck	31328.67	30753.11	30753.11		31328.7	86.2%
	LRT+LWF	24301.3	23725.7	23725.7		24301.3	66.8%
	Mr					36361.0	

Maximum (Minimum) Shear Forces of the Load Combinations:

Force Unit: KN

	Load Name	ULS1	ULS2	ULS3		Max. ULS	Vmax/Vr
Location No. 1	ON-625	2502.5	2427.4	2427.4		2502.5	51.0%
	LRT+Deck	2952.344	2883.7	2883.7		2952.3	60.2%
	LRT+LWF	2440.4	834.0	834.0		2440.4	49.7%
	Vr					4907.0	

Force Unit: KN

	Load Name	ULS1	ULS2	ULS3		Max. ULS	Vmax/Vr
Location No. 3	ON-625	3957.8	3870.9	3870.9		3957.8	51.0%
	LRT+Deck	5008.011	4924.612	4924.612		5008.0	64.6%
	LRT+LWF	3992.0	3908.6	3908.6		3992.0	51.5%
	Vr					7755.4	

Note: For locations of critical moments and shears, see analysis model in Appendix - C.

Appendix E

Structural Assessment of Eglinton Avenue – CNR Overhead Structure (Uxbridge Subdivision Mile 59.40)

AECOM
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

IBI GROUP

STRUCTURAL ASSESSMENT OF EGLINTON AVENUE – CNR OVERHEAD STRUCTURE UXBRIDGE SUBDIVISION MILE 59.40 FOR LIGHT RAPID TRANSIT

Draft

Prepared by:

Totten Sims Hubicki Associates (1997) Limited **doing business as AECOM**
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

Date:

December 2008

Statement of Qualifications and Limitations

The attached Report (the "Report") has been prepared by Totten Sims Hubicki Associates (1997) Limited doing business as AECOM ("AECOM") for the benefit of Ministry of transportation, Central Region ("Client") in accordance with the agreement between AECOM and Client (the "Agreement").

The information, data, recommendations and conclusions contained in the Report:

- are subject to the budgetary, time and other constraints and limitations in the Agreement and the qualifications contained in the Report (the "Limitations")
- represent AECOM's professional judgement in light of the Limitations and industry standards for the preparation of similar reports
- may be based on information provided to AECOM which has not been independently verified
- have not been updated
- must be read as a whole and sections thereof should not be read out of such context
- were prepared for the specific purposes described in the Report and the Agreement and must not be used for any other purpose whatsoever

Unless expressly stated to the contrary in the Report or the Agreement, AECOM:

- shall not be responsible for any events or circumstances that may have occurred since the date on which the Report was prepared or for any inaccuracies contained in information that was provided to AECOM
- makes no guarantees or warranties whatsoever, whether express or implied, with respect to the Report or any part thereof, other than that the Report represents AECOM's professional judgement as described above
- shall not be deemed to have represented that the Report or any part thereof is exhaustive or applicable to any specific use other than that described in the Report and the Agreement

Except as required by law or otherwise agreed by AECOM and Client, the Report:

- is to be treated as confidential
- may not be used or relied upon by third parties

Except as described above, AECOM denies any liability in respect of the Report or parts thereof and shall not be responsible for any damages arising from use of the Report or parts thereof.

This Disclaimer is attached to and forms part of the Report.

THIS DOCUMENTATION IS SUPPLIED TO MTO-Central Region BY AECOM AND CONSTITUTES CONFIDENTIAL TRADE SECRETS, OR COMMERCIAL, FINANCIAL, SCIENTIFIC, OR TECHNICAL INFORMATION. THIS DOCUMENTATION IS SUBMITTED TO MTO-Central Region IN CONFIDENCE. IT HAS SIGNIFICANT ECONOMIC VALUE TO AECOM AND ITS DISCLOSURE, WITHOUT THE EXPRESS CONSENT OF AECOM, COULD REASONABLY BE EXPECTED TO LEAD TO SIGNIFICANT AND UNDUE FINANCIAL AND OTHER HARM TO AECOM, INCLUDING HARM TO AECOM'S COMPETITIVE AND NEGOTIATING POSITIONS, AND UNDUE FINANCIAL GAIN TO ONE OR MORE THIRD PARTIES.

AECOM
300 Water Street, Whitby, ON, Canada L1N 9J2
T 905.668.9363 F 905.668.0221 www.aecom.com

December 15, 2008 Project Number: 42-71256

Mr. Harold Sich
Associate
IBI Group
230 Richmond Street, 5th Floor
Toronto, Ontario
M5V 1V6

Dear Harold:

Re: STRUCTURAL ASSESSMENT OF EGLINTON AVENUE – CNR OVERHEAD STRUCTURE UXBRIDGE SUBDIVISION MILE 59.40 FOR LIGHT RAPID TRANSIT

We are enclosing herewith two (2) copies of our structural assessment report as noted above.

Please advise if we could be of further assistance in the above regards.

Sincerely,
Totten Sims Hubicki Associates (1997) Limited doing business as AECOM

David LeBlanc, M.Eng., P.Eng.
Head, Structures Department
david.leblanc@aecom.com

DL:smb
Encl.
cc: File

Revision Log

Revision #	Revised By	Date	Issue / Revision Description

Signature Page

Report Prepared By:	Report Reviewed By:
Selva Balasundaram, P.Eng., Senior Structural Engineer	David LeBlanc, M.Eng., P.Eng., Head, Structural Department

AECOM

Executive Summary

AECOM was retained by IBI Group to investigate and confirm the feasibility of implementing a Light Rapid Transit (LRT) right-of-way (ROW) on the Eglinton Avenue – CNR Overhead structure, Uxbridge Subdivision Mile 59.40, at Kennedy Road, specifically addressing the structural adequacy of the overhead structure, as well as long term maintenance and operational requirements. The intent is upon confirmation of the feasibility of the LRT ROW implementation on the structure, to obtain approval from City of Toronto and Canadian National Railway during the environmental assessment phase in order to move forward with the project. It is recognized that that there are various design and contractual arrangements to be addressed in the subsequent project phases, and the TTC is committed to working with the authorities on these issues.

Existing structure is a 234.70m (approx.) long 9 span (21.34m + 7 x 27.43m + 21.34m) post-tensioned concrete twin structures constructed in 1979. At present the existing structure carries 3 lanes of east bound lanes and 3 lanes of west bound lanes with a raised concrete median and sidewalks at both north and south side of the structure. The existing structure has been rehabilitated in 1998.

An assessment of the existing Eglinton Avenue – CNR Overhead structure has been carried out to determine if it can accommodate the proposed Scarborough - Malvern LRT designated ROW, including two lanes of traffic in each direction. The findings indicate that the new LRT ROW and two traffic lanes can be accommodated on the existing structure without a need for deck widening.

A detailed structural evaluation was also undertaken to investigate effects of additional loads due to LRT and its accessories. Existing structure has been designed for AASHTO HS 25 live load. Our evaluation indicates that the structure is overstressed under service limit state (SLS) for both CL-625-ONT truck load and LRT live loads with conventional reinforced concrete track bed. Structural capacity is adequate under ultimate limit states (ULS) for both CL-625-ONT truck load and LRT live loads. Alternatively if the trackbed load is reduced by use of light weight materials, the structure will be subjected to load effects within the permissible limits at both SLS and ULS limit states.

There are a number of operational and maintenance features which will need to be accommodated for the new LRT, including the provision of poles on the deck to power the trains, modifications to the waterproofing and paving on the deck to accommodate the track bed, provision for drainage, provision for expansions joints in the continuous rail. These considerations have been identified, and a number of standard techniques that have been adopted elsewhere are available for investigation during the preliminary and detail design phases of the project.

Our findings indicate that it is feasible to accommodate the proposed LRT right-of-way on the Eglinton Avenue – CNR Overhead structure, without a need for deck widening. The structure has adequate capacity to withstand LRT loads in conjunction with the use of a light weight material for the track bed.

Table of Contents

List of Figures

Appendices

A. General Arrangement Drawing – Existing Structure
B. General Arrangement Drawing - Proposed Deck Cross Section with LRT Tracks
C. Details of Structural Evaluation

1. LOCATION

The overhead structure is located at the intersection of Eglinton Avenue East and CN Rail tracks, at mileage 59.40 Uxbridge Subdivision, near Kennedy Road as shown on the following Key Plan. Eglinton Avenue East is under the jurisdiction of City of Toronto.

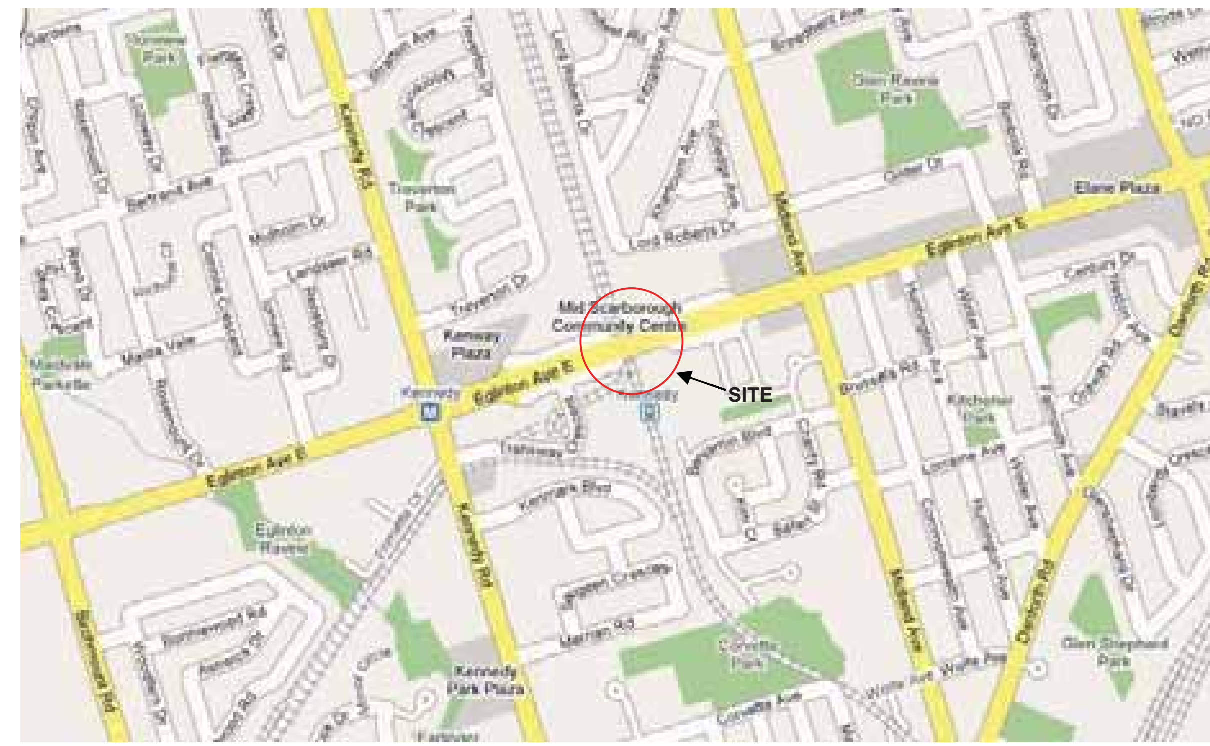


Figure 1. Key Plan

2. EXISTING STRUCTURE

The existing structure, which was constructed in about 1979, is a 234.69 m long nine span (21.33m, 7 x 27.43m, 21.33m) post-tensioned concrete structure with 90mm thick waterproofing and asphalt wearing surface. The bridge superstructure is supported on cast-in-place reinforced concrete abutments and piers founded on footings and constructed at normal to the road alignment.

The General Arrangement drawing of the existing structure and post-tensioning details are presented in Appendix A.

3. EXISTING CROSS SECTION

The cross section of the existing structure consists of the following:

North parapet wall	0.300 m
Sidewalk	1.834 m
Side clearance	1.082 m
Traffic Lanes	3 x 3.500m
Median	1.200 m
Traffic Lanes	3 x 3.500m
Side clearance	1.082 m
Sidewalk	1.834 m
West Barrier wall	0.300 m

4. STRUCTURE GEOMETRY

A preliminary assessment of the existing bridge geometry has been carried out, indicating the following:

- The width of the roadway at the bridge from gutter to gutter is approximately 24.40 m, which includes a 1.20 m wide median. The bridge can accommodate the required horizontal clearance for the 2 lanes of traffic in each and the new LRT designated right-of-way configuration. It is feasible to implement the LRT right-of-way geometrically.
- The maximum longitudinal slope of the bridge structure is 5.2%, which is marginally more than the assumed maximum slope of 5% for the new LRT Vehicle.

A preliminary general arrangement drawing showing the proposed LRT configuration on the Morningside Underpass structure is provided in Appendix B.

5. STRUCTURAL ASSESSMENT

The design loads that the existing structure has been designed to include the following:

Dead Loads:
The dead loads due to deck, sidewalk, parapet walls with handrails, asphalt wearing surface, and light poles

Live Loads:
The original design live loads were based on AASHTO HS 25 load. While investigating the structure for the suitability of carrying the LRT vehicle, we have also investigated for the requirements of the current Canadian Highway Bridge Design Code (CHBDC) CAN/CSA – S6-06 and CL-625-ONT Truck load of 625 kN.

Other Loads:
Other loads that need to be considered in the design of the structure include secondary loads due to post-tensioning, thermal, wind, braking etc. as specified in the code.

A structural assessment of the existing bridge has been carried for the following load conditions:

- AASHTO HS25 Truck
- CHBDC CAN/CSA - S6-06 CL-625-ONT Truck
- Proposed LRT Live load and additional loads due to conventional trackbed & accessories
- Proposed LRT Live load and additional loads due to lightweight trackbed & accessories

The results of the structural evaluation are summarized in Appendix C, and indicate that the superstructure is overstressed under service limit states (SLS) by about 3% for CHBDC CL-625-ONT Truck load and about 18% for LRT loads with conventional reinforced concrete trackbed. The structure has adequate capacity under ultimate limit states (ULS) for the various loading conditions considered. The loads acting on substructure and foundation are expected to increase in the range of 5 to 10%, similar to increase in support reactions/ shear forces in the deck, if conventional concrete trackbed is adopted. It is unlikely that strengthening of the foundations will be required for this additional load, however, underpinning methods are available to strengthen the capacity of existing abutment and pier footings, if necessary.

The results of the structural evaluation indicates that if a light-weight polymer infill with a unit weight in the order of 2 to 4 kN/m3 is provided for the trackbed, strengthening of superstructure and substructure strengthening will not be required. It should be noted that, the TTC are investigating this technique for several bridges in the City of Toronto for the Transit City program.

A further option which could be considered would be to fix the rails directly to the concrete deck and eliminate the trackbed, in which case the structure is subjected to loads similar to that of the existing structure. However there are numerous maintenance and durability issues associated with fixing the rails directly to the deck, which could compromise the long term life of the structure, and this alternative is not recommended for further consideration.

6. MISCELLANEOUS STRUCTURAL DETAILS

There are a number of details associated with the LRT ROW which will require modification of the existing structure, and which will need to be detailed during the design phases of the project. A preliminary assessment of the impact of the LRT ROW on the structure has been carried out, and the following items will need to be addressed:

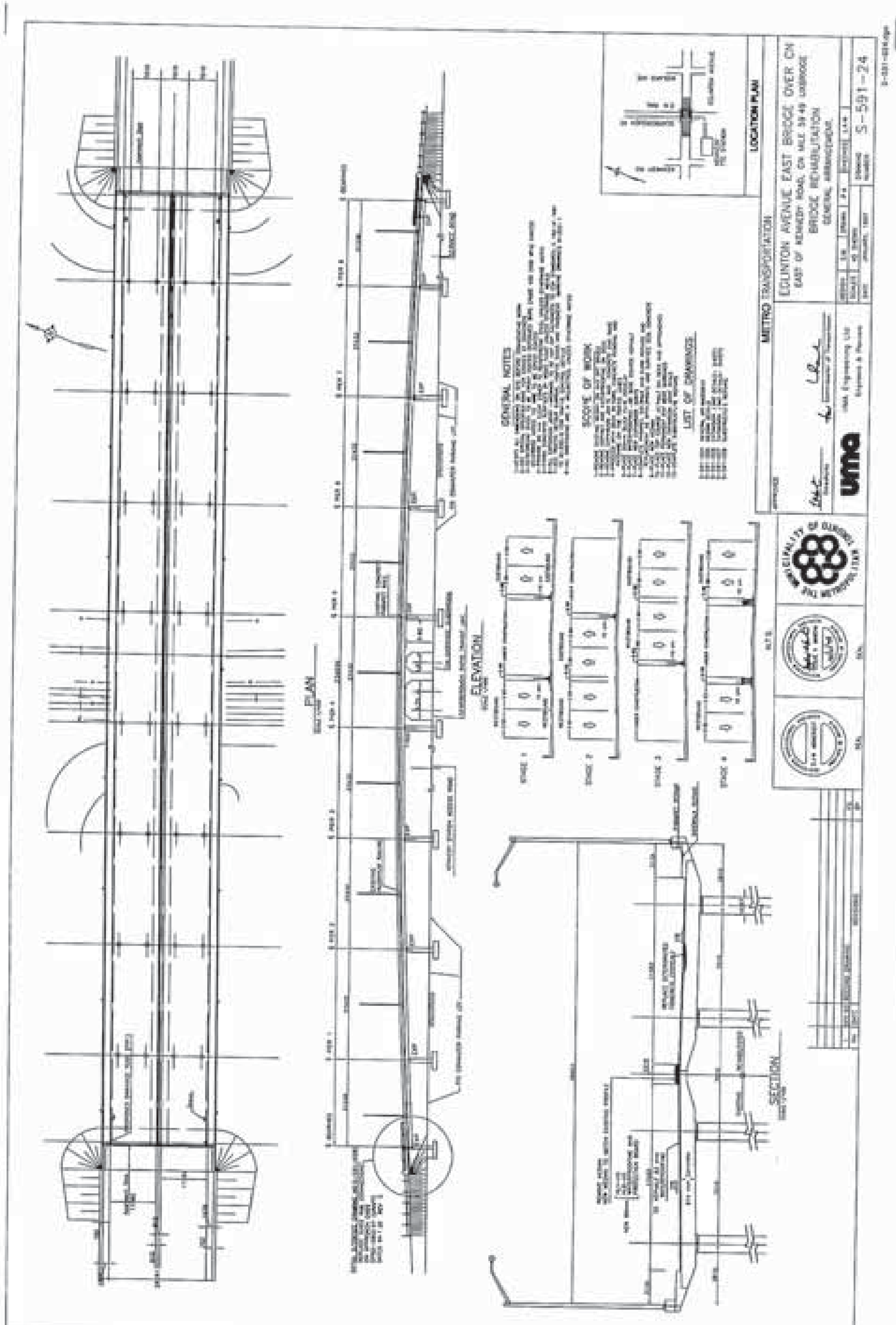
- Poles will be required on the deck to provide overhead power for the LRT. The forces due to poles supporting the catenaries and light poles will induce primarily localized effects. Pedestals and connections to deck slab will need to be provided and detailed appropriately.
- Expansion joints will need to be provided to minimize the effect of movement of the structure on the continuous welded rail. Expansion can be accommodated through combinations of rail anchors and bolted joints allowing for limited movements or special proprietary rail expansion joints.
- Ensure protection of structures and components from corrosion due to stray currents by appropriate method of grounding or coating reinforcements or insulating with a membrane below the trackbed.
- Proper detailing of waterproofing and paving where it abuts the LRT trackbed will be required to maintain the long term durability of the deck.
- Existing structure is not provided with any deck drains. As the existing roadway is on a symmetrical crest curve deck drains could be avoided on the bridge structure although the length of the structure approximately 234.70m is more than 120m. However adequate drainage of the LRT right-of-way drainage will need to be addressed.

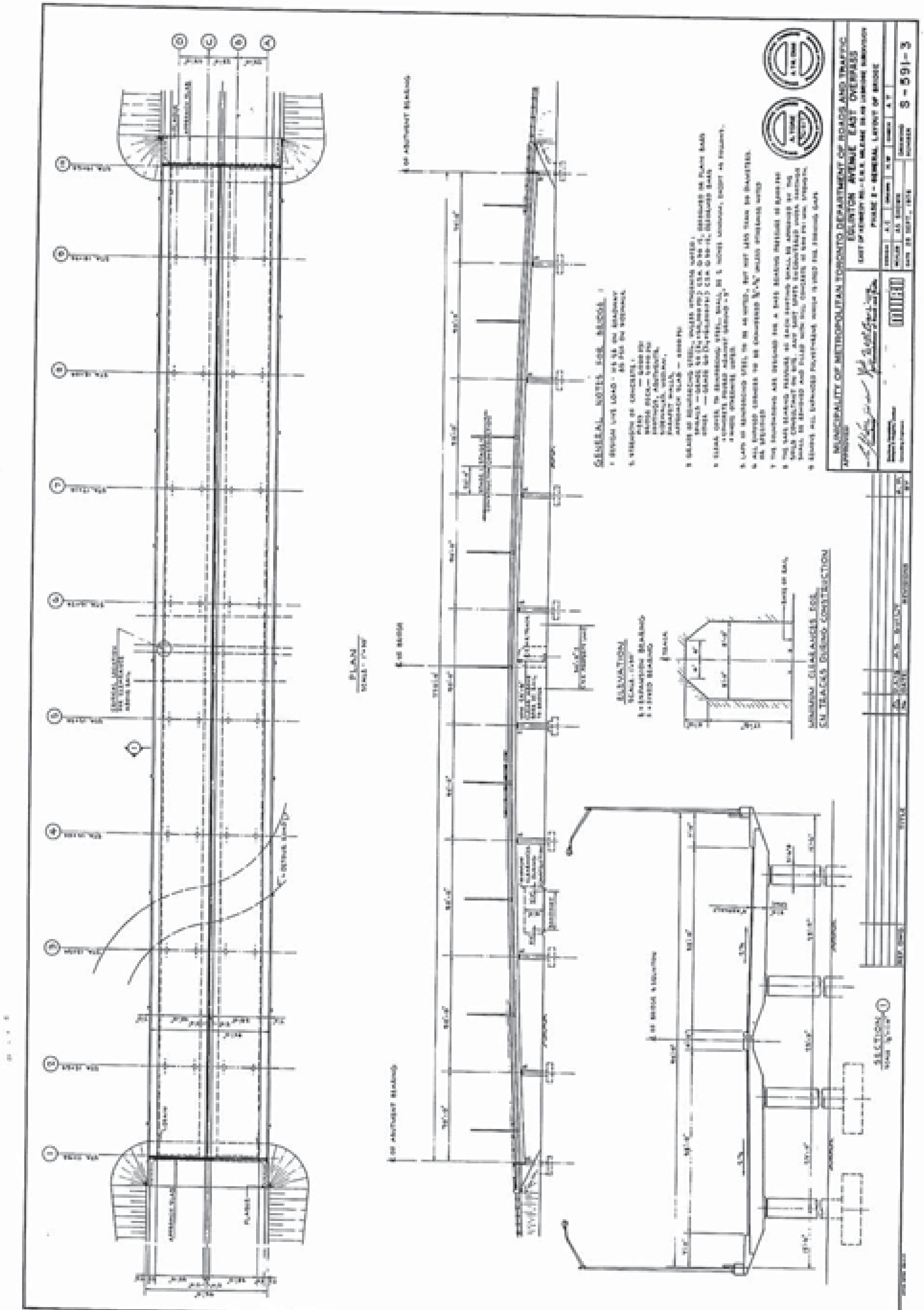
Long term maintenance and rehabilitation of the bridge deck and the LRT trackbed will be somewhat complicated by the LRT right-of-way. There are a number of alternatives available, with the simplest being that a temporary closure of the LRT ROW will be required during major rehabilitative works on the bridge, which extend for 4 to 6 months in duration, and local bus service be utilized. Alternatives and details will be developed in subsequent project phases.

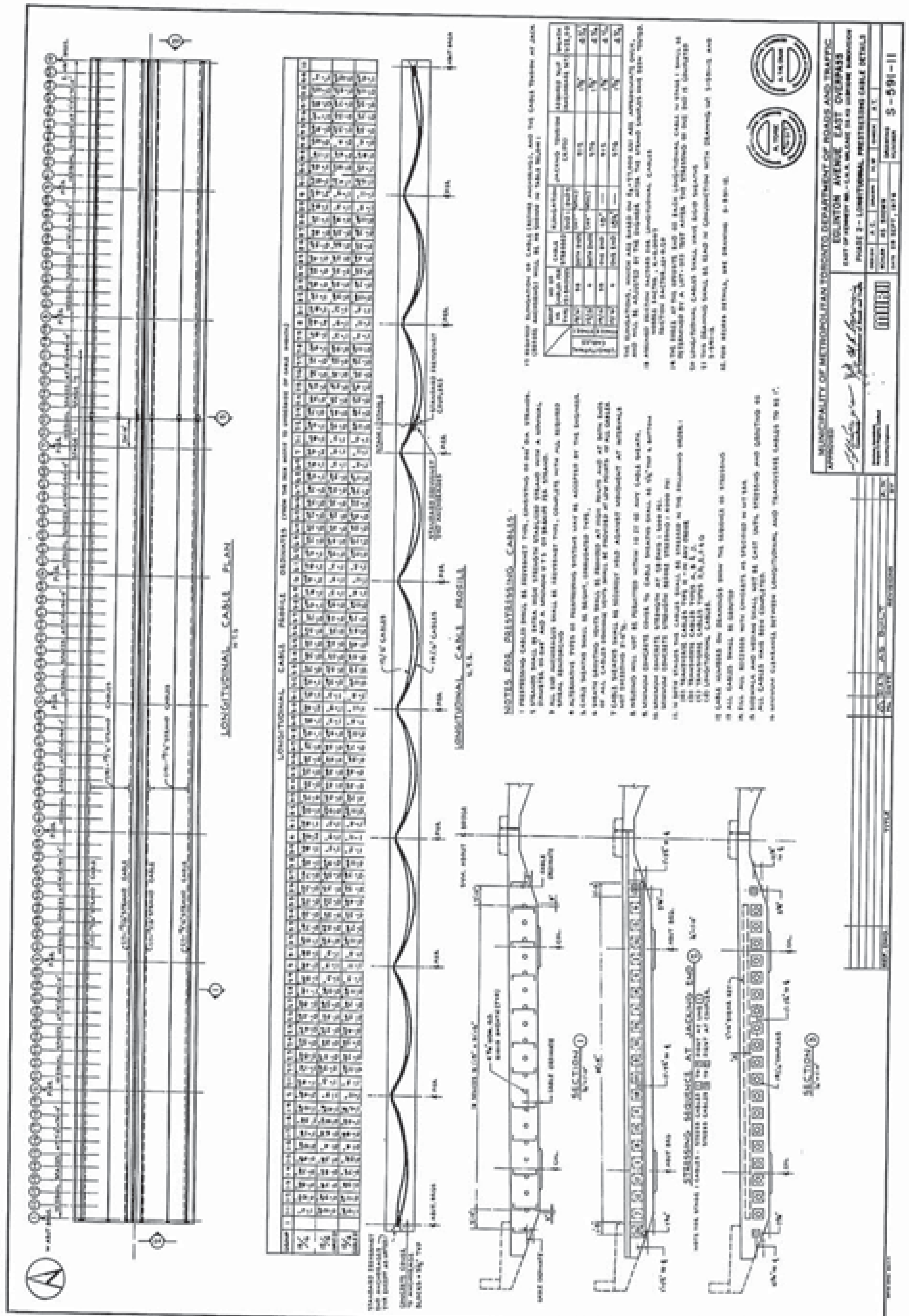
The above identified miscellaneous structural details can be addressed with standard techniques that have been adopted elsewhere, and will be fully investigated during the preliminary and detail design phases of the project. The TTC is committed to working with City of Toronto and other authorities MTO on these issues.

Appendix A

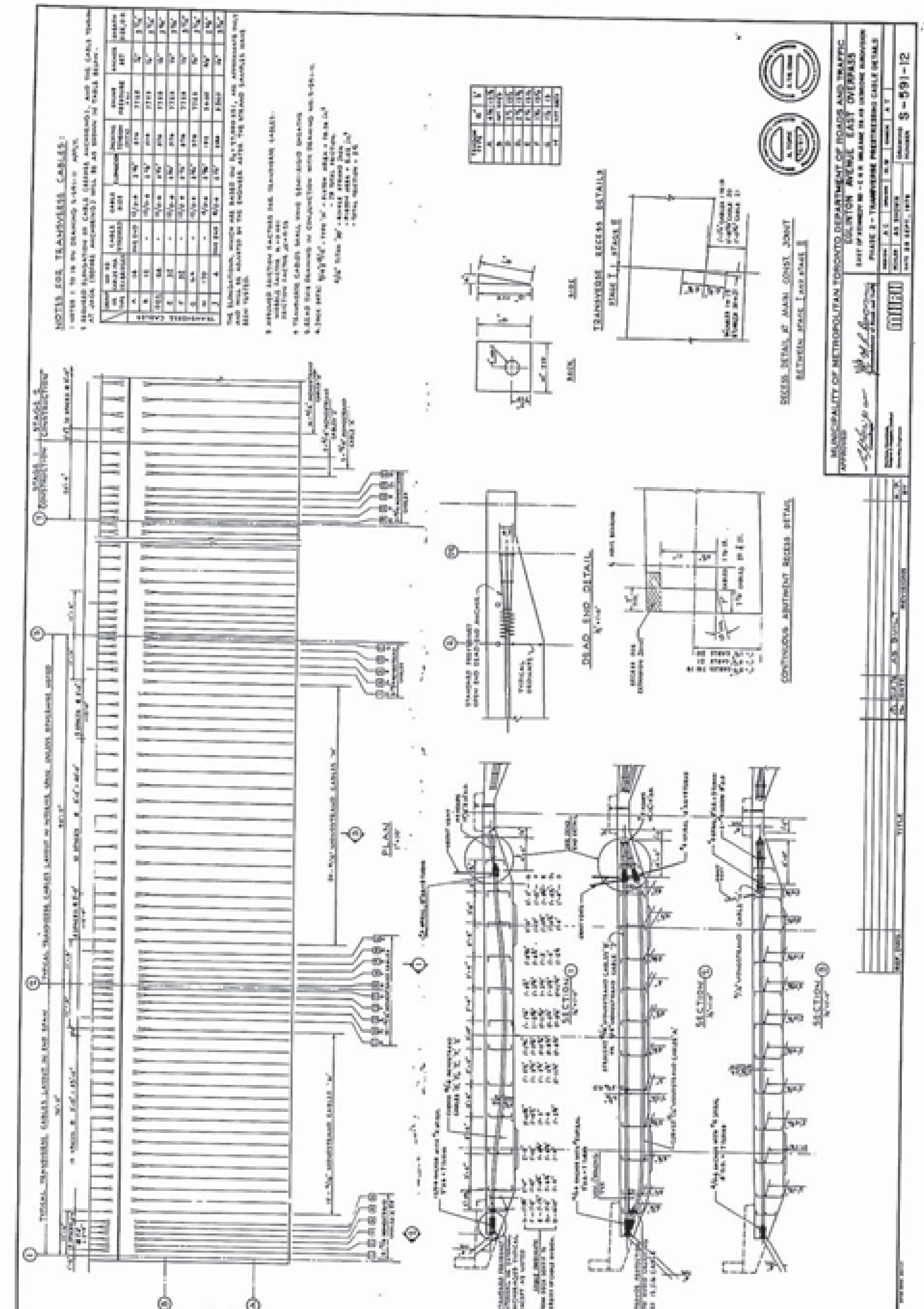
General Arrangement Drawing – Existing Structure


LOCATION PLAN
METRO TRANSPORTATION
EGLINTON AVENUE EAST BRIDGE OVER CN
BRIDGE REHABILITATION
GENERAL ARRANGEMENT
S-591-24
PLAN
ELEVATION
SECTION
GENERAL NOTES
SCOPE OF WORK
LIST OF DRAWINGS

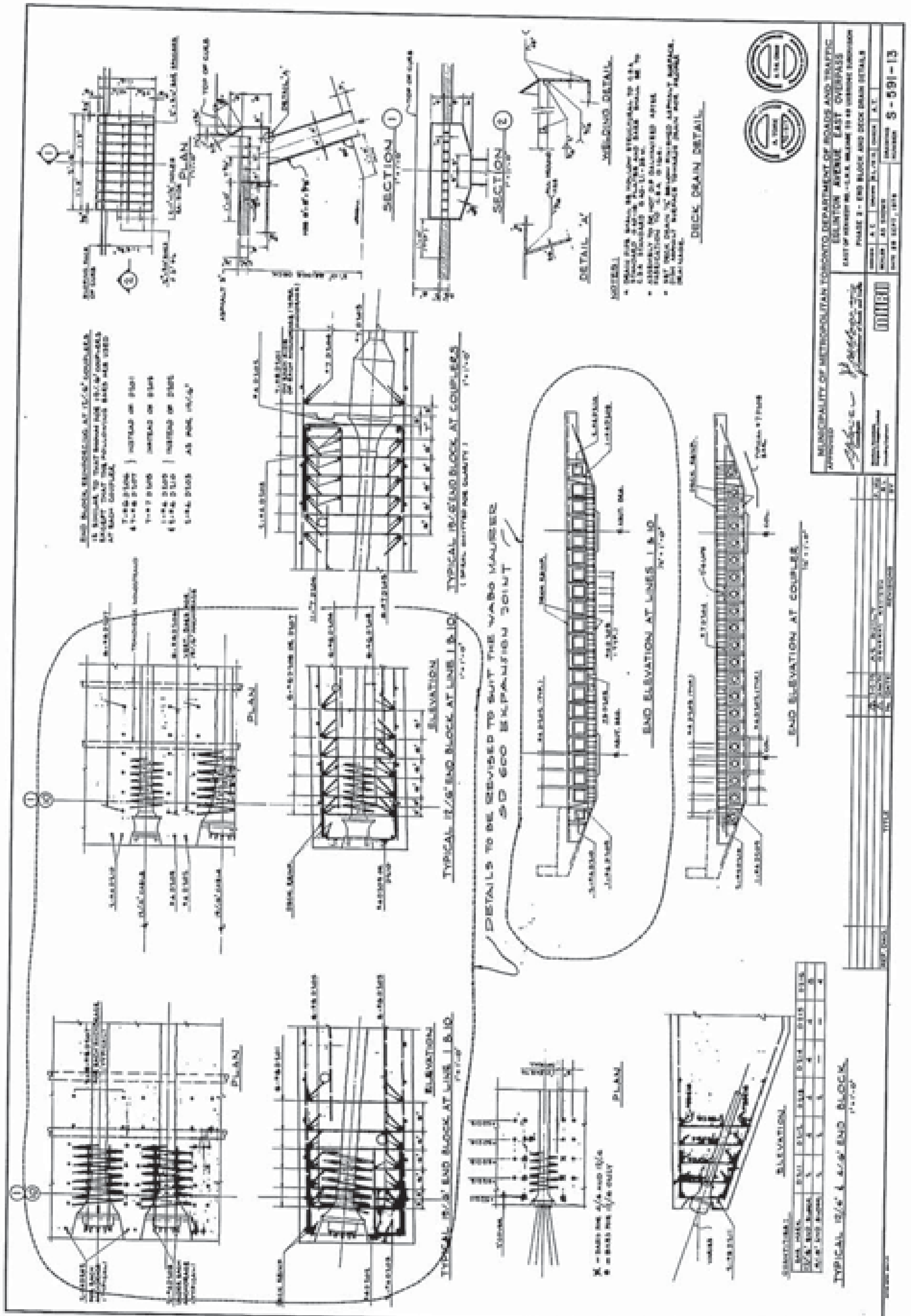

MUNICIPALITY OF METROPOLITAN TORONTO DEPARTMENT OF ROADS AND TRAFFIC
EGLINTON AVENUE EAST OVERPASS
S-591-3
PLAN
ELEVATION
GENERAL NOTES FOR BRIDGE
MINIMUM CLEARANCES FOR CN TRACKS DURING CONSTRUCTION



LONGITUDINAL CABLE PLAN
LONGITUDINAL CABLE PROFILE
NOTES FOR PRESTRESSING CABLES
MUNICIPALITY OF METROPOLITAN TORONTO DEPARTMENT OF ROADS AND TRAFFIC
EGLINTON AVENUE EAST OVERPASS
S-591-11

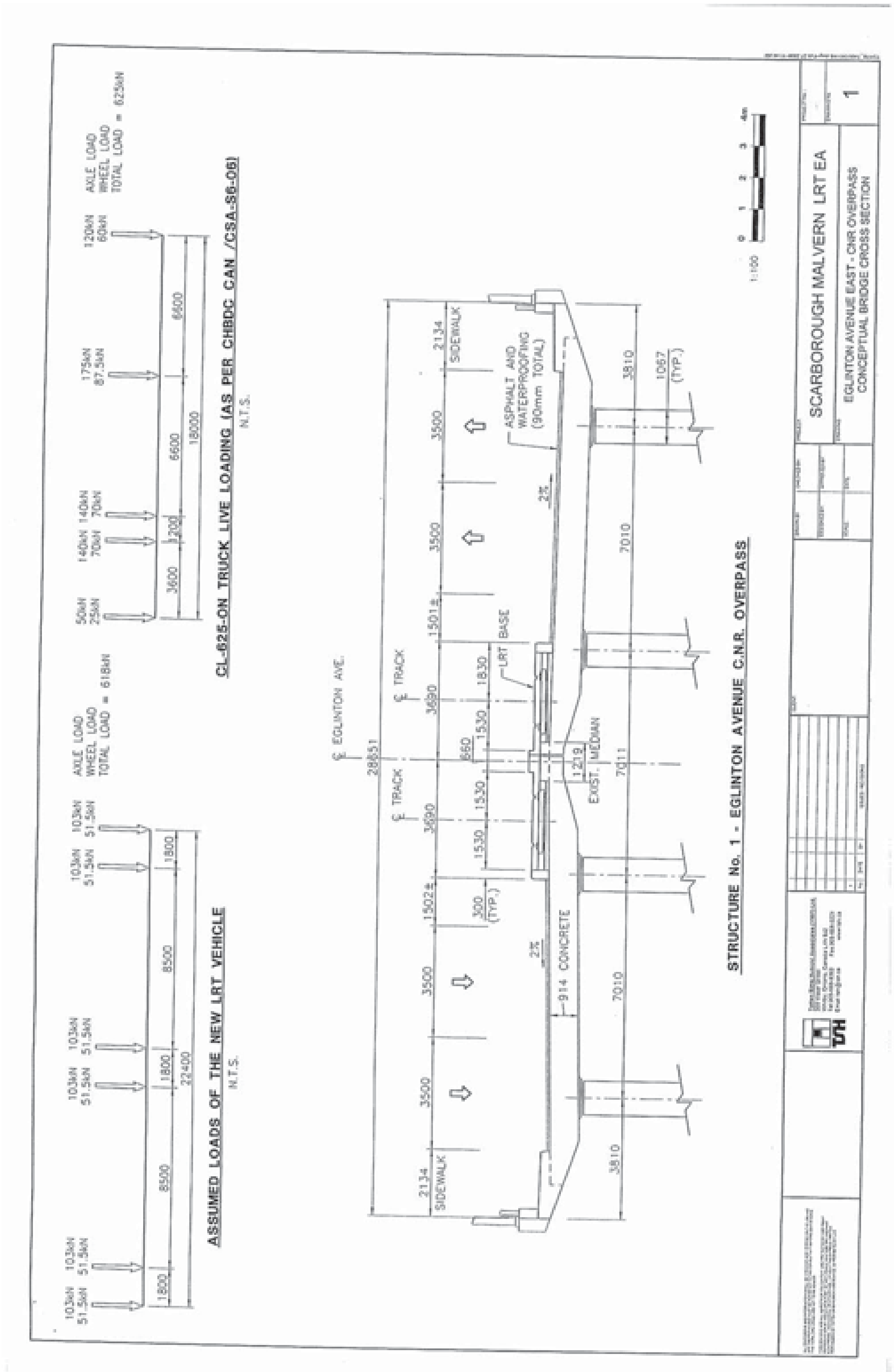


NOTES FOR TRANSVERSE CABLES
DEAD END DETAIL
TRANSVERSE RECESS DETAILS
PLAN
MUNICIPALITY OF METROPOLITAN TORONTO DEPARTMENT OF ROADS AND TRAFFIC
EGLINTON AVENUE EAST OVERPASS
S-591-12



Appendix B

General Arrangement Drawing – Proposed Deck Cross Section with LRT Tracks



AECOM

Appendix C

Details of Structural Evaluation

(file code)

Name of Bridge:	Eglinton Avenue-CNR Overhead Structure
Project # :	
Engineer:	TNS — Checked By:
Approved By:	
Company:	TSH
Date:	November 4, 2008
Sheet Title:	General Info and Assumptions According to CHBDC 2000

1) General Information

Structure Type :	Solid Slab
# Spans :	9
Length of Longest Span :	27432 mm
Width of Span :	14326 mm
Hwy Class :	A
Deck Thickness :	914 mm
Asphalt & Waterproofing :	76 mm
Girder Material :	Prestressed Concrete
Girder Type :	solid slab
Right Girder Spacing (B1):	1000 mm
# Girders:	6
Left Girder Spacing (B2):	1000 mm
# Girders:	6
Right Cantilevered Length :	1291 mm
Left Cantilevered Length :	1291 mm
Sidewalk Width :	0 mm
# Sidewalks :	2
Curb Width :	0 mm
# Curbs :	0
Barrier Wall / Parapet Wall Width:	0 mm
Area :	0.0000 mm^2
Total # Girders	12

Live Load Distribution Factors (Section 5 of CHBDC) - Interior Girder
CL 5.7.1.2.2

Span 1 = 21.34 m
Span 2 = 27.43 m
Span 3 = 27.43 m

$F_m = S*N/(F*(1+\mu*C_f/100)) >= 1.05$

for n <=4, F, Cf from table A5.7.1.2.1
for n >4, $F=F_4/2.8$ F_4 = F for n = 4

Wc = 14226 mm
 # design design lanes = n = 4 table 3.8.2
We = Wc/n = 3.56 m
μ = min((We - 3.3)/0.6, 1) = 0.4
girder spacing S = 1.000 m
over hang = 1.291 m FALSE
D_{VE} = 0.600 m

Positive Moment

Span 1 - SLS & ULS		
Coeff. = 0.80 fig. A5.1(a)	L = 17.07 m	
	n = 4	
	N = # girders = 12	
table 3.8.4.2	MLRF = RL = 0.700	
	α =n/N*R_L = **0.233**	
	F = read from table	
	Exterior	Interior
read F	10.331 m	10.331 m
F =	10.331 m	10.331 m
Cf =	14.24 %	14.24 %
F(1+μCf/100) = Dd =	10.960 m	10.960 m
Fm =SN/Dd =	**1.050 m**	**1.095 m**
Dist Factor = α.Fm =	**0.245**	**0.255**

Shear for ULS & SLS
n = 4
MLRF = RL = 0.700
N = 12
α =n/N*R_L = **0.233**
F or F4 = 10.51884377
F reduction factor = 1.00
Fv = S*N/F
F = 10.519 m
Fv = 1.141
Dist Factor = α Fv = 0.266

CL 5.7.1.4.1

Table 5.7.1.4.1

Interior Girder Design - DF

	SLS/ULS		
	+ve Moment	-ve Moment	Shear
Span 1	0.255	0.259	0.266
Support 1	0.274	0.267	0.266
Span2	0.259	0.245	0.266

Interior Girder Design - Fm, Fv

	SLS/ULS		
	+ve Moment	-ve Moment	Shear
Span 1	1.095	1.111	1.141
Support 1	1.175	1.144	1.141
Span2	1.108	1.050	1.141

Shear for ULS & SLS
n = 4
MLRF = RL = 0.700
N = 12
α =n/N*R_L = **0.233**
F or F4 = 10.51884377
F reduction factor = 1.00
Fv = S*N/F
F = 10.519 m
Fv = 1.141
Dist Factor = a Fv = 0.266

2

Support 1 - SLS & ULS		
Coeff. = 0.20 fig. A5.1(a)	L = 9.75 m	
	n = 4	
	N = # girders = 12	
table 3.8.4.2	MLRF = RL = 0.700	
	$\alpha = n/N*R_L$ = 0.233	
	F = read from table	
	Exterior	Interior
read F	7.921 m	9.899 m
F =	7.921 m	9.899 m
Cf =	7.44 %	7.44 %
$F(1+\mu Cf/100)$ = Dd =	8.172 m	10.214 m
Fm =SN/Dd =	1.050 m	1.175 m
Dist Factor = α.Fm =	0.245	0.274

Span 2 - SLS & ULS		
Coeff. = 0.80 fig. A5.1(a)	L = 21.95 m	
	n = 4	
table 3.8.4.2	MLRF = RL = 0.7	
	N = # girders = 12,000	
	$\alpha = n/N*R_L$ = 0.233	
	F = read from table	
	Exterior	Interior
read F	10.435 m	10.435 m
F =	10.435 m	10.435 m
Cf =	8.86 %	8.86 %
$F(1+\mu Cf/100)$ = Dd =	10.831 m	10.831 m
Fm =SN/Dd =	1.050 m	1.108 m
Dist Factor = α.Fm =	0.245	0.259

ESTIMATED ***PER GIRDER* LOADS**

1) Girder Selfweight Cast-in-place — will be considered separately for design of girder.

2) CIP Deck Slab — will be considered separately for design of girder.

3) Waterproofing & Asphalt — 20.74 kN/m
(Load Case 1 in S-Frame)

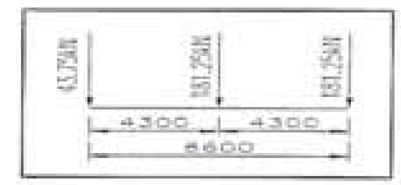
t = 76 mm
S_{girder} = 11582 mm
γ = 23.5 kN/m3

4) Barrier Wall — 30.6 kN/m
(Load Case 1 in S-Frame)

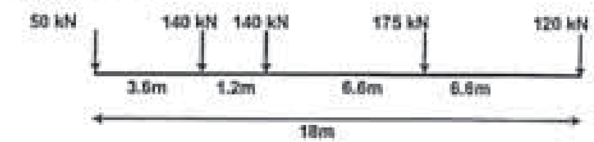
A = 0.6 m2
γ = 24.0 kN/m3
girders = 1

5) Live Load Appendix A3.4 — CL-625-ONT
(Load Case 23 in S-Frame)

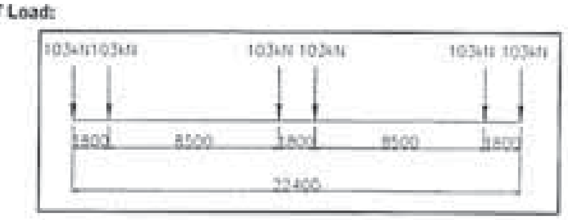
a) HS-25:



b) CHBDC CL-625-ONT:



c) TTC LRT Load:



6) Extra LRT deck load

Extra Concrete Deck Weight			
Width=	3690 mm	1) Weight=	26.6 kN/m
Depth=	300 mm		
Deensity=	24 kN/m^3		
Deduct the curb weight:			
Width=	609.6 mm	2) Weight=	4.5 kN/m
Depth=	304.8 mm		
Deensity=	24 kn/m^3		
Deduct the asphalt weight:			
Width=	3060.4 mm	3) Weight=	5.5 kN/m
Depth=	76.2 mm		
Deensity=	23.5 kn/m^3		
Extra LRT deck load=		[1) - 2) -3) =	16.6 kN/m

If light weight fill is used		
	1) Weight=	5.5 kN/m
Deensity=	5	kN/m^3
	2) Weight=	4.5 kN/m
	3) Weight=	5.5 kN/m
	4) Curb w.=	4.3 kN/m
	1) -2) - 3) +4) =	-0.1 kN/m

TSH Associates	Project:		Page	7
	Project No.		Date	9-Jan-07
Structure:	Cal. by	TNS	Check by	

POST - TENSIONED DECK (SOLID SLAB)
DESIGN FOR SERVICEABILITY LIMIT STATES - AS PER CHBDC CAN/CSA-S6-06

Important Note: Only data presented in black is to be input. Red coloured data means it is calculated by the spreadsheet

	span 1	span 2	span 3	span 4	span 5	span 6	span 7
Length of slab between BRG's (m):	21.336	27.432	27.432	27.432	27.432	27.43	27.432
Length of solid end portion from support (m):	21.336	27.432	27.432	27.432	27.432	27.43	27.432
Thickness of slab (m):	0.9144	0.9144	0.9144	0.9144	0.9144	0.914	0.9144
Eff. Width of Solid Slab (m):	12	12	12	12	12	12	12

Length of bridge = 234.696 m

1) MATERIAL PROPERTIES

a) prestressing steel

Strand Type	Low - Relaxation		Fpy/Fpu = 0.9
Ep		190,000	MPa
Size Designation	7 wire	15	mm
Grade		1860	MPa
Nominal Diameter		15.2	mm
Nominal Area		140.0	mm^2
# Strands / Tendon		19	strands
Number of Tendons		20.26	tendons
Area per Tendon		2660	mm^2

	span1	span2	span3	span4	span5	span6	span7
Dist from center of tendon's duct to bot (at sag)- mm	250	186.0	139.0	139.0	139.0	139.0	139.0
Eccentricity of Prestressing (Sag) - mm	228.4	292.4	339.4	339.4	339.4	339.4	339.4

Diameter of duct (outside)	112.7	mm
min. net vertical spacing of tendon:	0	mm
min. vertical tendon spacing(c/c)	112.7	mm
tendon vertical spacing at anchorage(c/c):	0	mm

b) Reinforcing Steel

Specified yield strength of rebar fy:	400	Mpa
Elastic modulus Es	200,000	Mpa
ϕ_s	0.9	

c) post-tensioned deck concrete properties

f'c,g		34.47379	MPa	28-day girder concrete compressive strength	
Ecg =		27780.4	MPa	young modulus of naked section	
fcr,g = 0.40*(f'c,g)^.5		2.35	MPa	all. girder tensile stress = craking strength	17.16.2.3
f'ci,g	75.0 % of f'c	25.9	MPa	@ transfer section concrete compressive strength	
Eci =		25106.1	MPa	youngs modulus of naked section @ transfer	
fcri = 0.4*(f'ci)^.5		2.03	MPa	all. transfer section tensile stress = craking strength @ tra	17.16.2.3

ϕc = 0.75

File Name :post-tent-ana-on625(Main Cal.)
Sheet Name :input

TSH Associates	Project:		Page	8
	Project No.		Date	9-Jan-07
Structure:	Cal. by	TNS	Check by	

POST - TENSIONED DECK (SOLID SLAB)
DESIGN FOR SERVICEABILITY LIMIT STATES - AS PER CHBDC CAN/CSA-S6-06

Important Note: Only data presented in black is to be input. Red coloured data means it is calculated by the spreadsheet

	span 1	span 2	span 3	span 4	span 5	span 6	span 7
Length of slab between BRG's (m):	21.336	27.432	27.432	27.432	27.432	27.43	27.432
Length of solid end portion from support (m):	21.336	27.432	27.432	27.432	27.432	27.43	27.432
Thickness of slab (m):	0.9144	0.9144	0.9144	0.9144	0.9144	0.914	0.9144
Eff. Width of Solid Slab (m):	12	12	12	12	12	12	12

Length of bridge = 234.696 m

2) STRESSES/FORCES IN PRESTRESSING STRAND

CL 8-7.1

		Stress	Force per Tendon		
fpu		1860 MPa	4948 kN	breaking strength	
fpy = 0.9*fpu	0.9	1674 MPa	4453 kN		
fsj = 0.80*fpu	0.8200	1525 MPa	4057 kN	@ jacking	CL 8-7.1
D(fs1)	PS loss @ transfer	317 MPa	844 kN	% = 21%	
fst = fsj - D(fs1)		1208 MPa	3214 kN	@ transfer	
fst, max = 0.70*fpu	0.70	1302 MPa	3463 kN		CL 8-7.1
fst < fst, max		O.K.			
D(fs) = D(fs1) + D(fs2)	total PS Loss	463 MPa	1232 kN	% = 30%	
fse = fsj - D(fs)		1062 MPa	2825 kN	@ after all losses	
fse, min = 0.45*fpu	0.45	837 MPa	2226 kN		CL 8-7.1
fse > fse, min		O.K.			

Total Tendon Force @ Transfer	65116.1	kN
Total Effective Tendon Force (after All Losses)	57252.2	kN

Concrete Stress Limitations CL 8.8.4.6

At transfer				
	compression: 0.60*f'ci	15.5132 Mpa	8.8.4.6.(a.1)	
	tension: 0.50*fcri	-1.02 Mpa	8.8.4.6.(a.2)	Rebar is not needed
At service:				
	compression 0.45f'c	15.51 Mpa	OHBDC 8.8.4.6.b	
	tension: fcr	-2.35 Mpa	No crack	

File Name :post-tent-ana-on625(Main Cal.)
Sheet Name :input

TSH Associates	Project:		Page	9
	Project No.		Date	9-Jan-07
Structure:	Cal. by	TNS	Check by	

POST - TENSIONED DECK (SOLID SLAB)
DESIGN FOR SERVICEABILITY LIMIT STATES - AS PER CHBDC CAN/CSA-S6-06

Important Note: Only data presented in black is to be input. Red coloured data means it is calculated by the spreadsheet

	span 1	span 2	span 3	span 4	span 5	span 6	span 7
Length of slab between BRG's (m):	21.336	27.432	27.432	27.432	27.432	27.43	27.432
Length of solid end portion from support (m):	21.336	27.432	27.432	27.432	27.432	27.43	27.432
Thickness of slab (m):	0.9144	0.9144	0.9144	0.9144	0.9144	0.914	0.9144
Eff. Width of Solid Slab (m):	12	12	12	12	12	12	12

Length of bridge = 234.696 m

3) GEOMETRY

a) post-tensioned deck

Ybg	478	mm	N.A. of section from bottom
Ag	11.077	m^2	section cross sectional area
Ig	0.745	m^4	section moment of intertia
radius of gyration rg	0.259	m	
Sbg	1.557	m^3	section modulus from bottom
Stg	1.709	m^3	section modulus from top

Converted section considering prestress tendons

n = Es / Ec 6.84

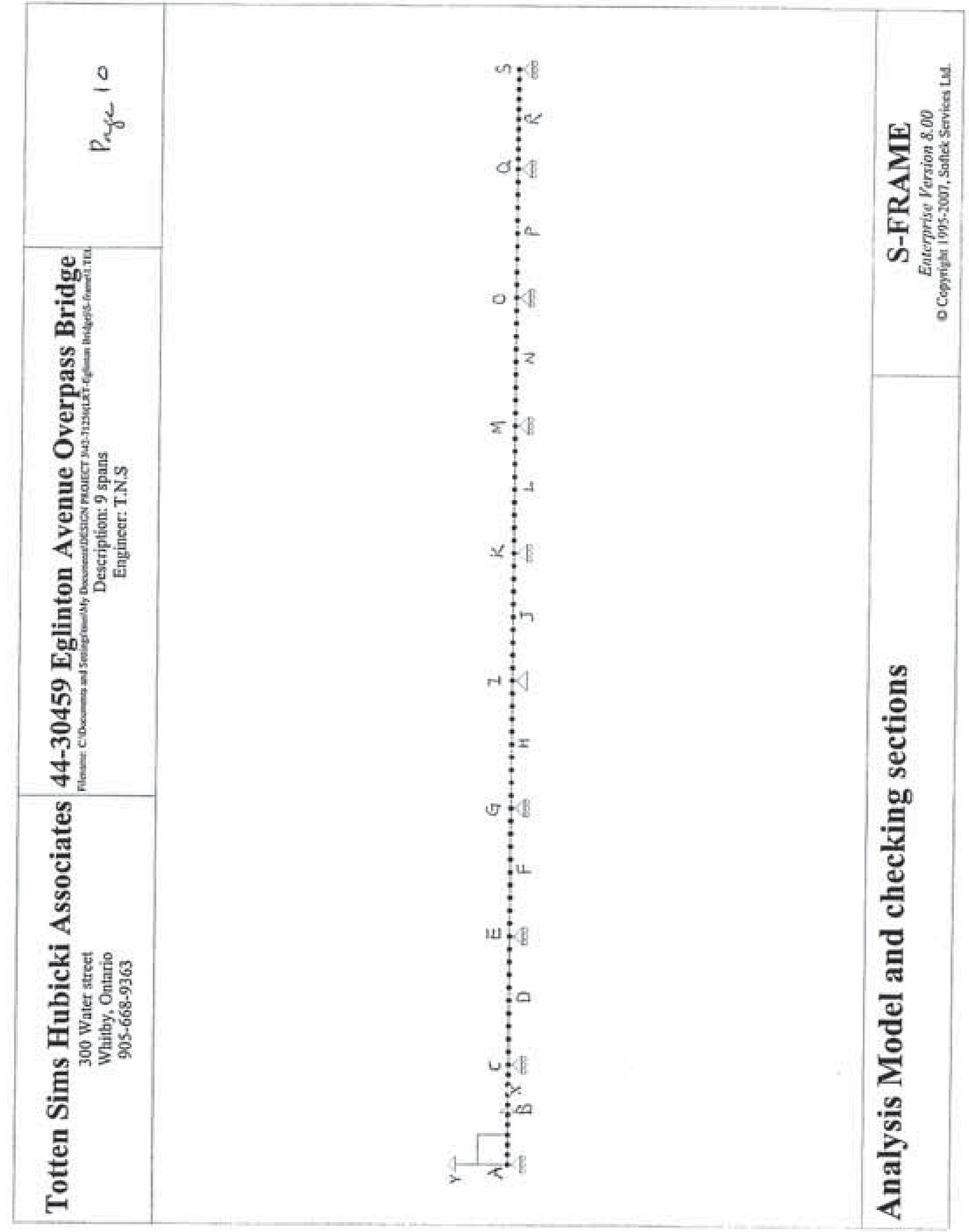
	A - m2	yci - m	A*yci	I - m4	d =yc-yci	A*d^2
concrete	11.077	0.4784	5.299237	0.745	0.0063104	0.0004
tendons	0.31474	0.250	0.078685	0	0.222	0.0155
Σ	11.39174		5.377922	0.745		0.016

yc =	0.472089553	m			
I =	0.760988348	m^4	S'bg	1.61196	m^3
A ' =	11.39174	m^2	S'tg	1.72048	m^3

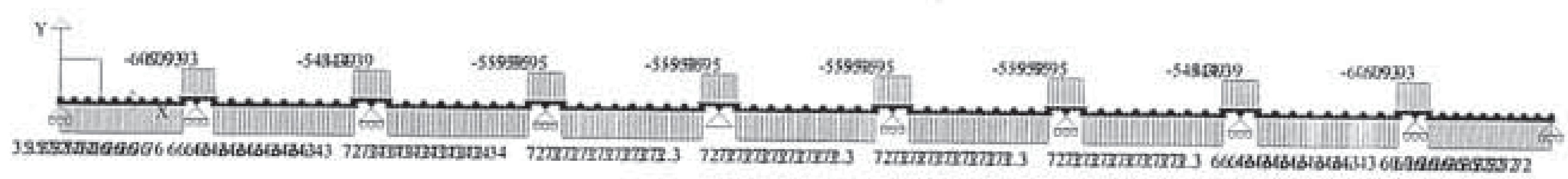
Calculation assumptions(optional)

Width for Flexual resistance & shear resistance calc.= 12 m
Width of total voids 0

File Name :post-tent-ana-on625(Main Cal.)
Sheet Name :Input



Totten Sims Hubicki Associates 300 Water street Whitby, Ontario 905-668-9363	**44-30459 Eglinton Avenue Overpass Bridge** Filename: C:\Documents and Settings\...\My Documents\DESIGN PROJECT 3142-71236(LRT-Eglinton Bridge)\S-frame\1.TEL Load Case 4: prestress Engineer: T.N.S	Page 11



Equivalent Prestree Force

S-FRAME
Enterprise Version 8.00

Page 12

Concrete Stress under the SLS status (1)

Load Case:		HS-25				
Span	Sec. No.	T. or B.	σ	[σ]	%	Status
1	2	3	4	5	4/5	7
Abut	A	Top	5.19	15.51	33.5%	OK
		Botton	4.85	15.51	31.3%	OK
1 Mid.	B	Top	8.28	15.51	53.4%	OK
		Botton	6.28	15.51	40.5%	OK
pier 1	C	Top	-1.92	-2.35	81.7%	OK
		Botton	13.55	15.51	87.4%	OK
2 Mid.	D	Top	7.84	15.51	50.5%	OK
		Botton	6.99	15.51	45.1%	OK
pier 2	E	Top	-0.77	-2.35	32.8%	OK
		Botton	12.27	15.51	79.1%	OK
3 Mid.	F	Top	6.70	15.51	43.2%	OK
		Botton	8.33	15.51	53.7%	OK
pier 3	G	Top	-0.10	-2.35	4.3%	OK
		Botton	11.54	15.51	74.4%	OK
4 Mid.	H	Top	7.04	15.51	45.4%	OK
		Botton	8.00	15.51	51.6%	OK
pier 4	I	Top	-0.90	-2.35	38.3%	OK
		Botton	12.48	15.51	80.5%	OK
5 Mid.	J	Top	6.92	15.51	44.6%	OK
		Botton	8.10	15.51	52.2%	OK
pier 5	K	Top	-0.90	-2.35	38.3%	OK
		Botton	12.48	15.51	80.5%	OK
6 Mid.	L	Top	7.03	15.51	45.3%	OK
		Botton	8.00	15.51	51.6%	OK
pier 6	M	Top	-0.10	-2.35	4.3%	OK
		Botton	11.54	15.51	74.4%	OK
7 Mid.	N	Top	6.70	15.51	43.2%	OK
		Botton	8.32	15.51	53.6%	OK
pier 7	O	Top	-0.78	-2.35	33.2%	OK
		Botton	12.27	15.51	79.1%	OK
8 Mid.	P	Top	7.84	15.51	50.5%	OK
		Botton	7.00	15.51	45.1%	OK
pier 8	Q	Top	-1.92	-2.35	81.7%	OK
		Botton	13.54	15.51	87.3%	OK
9 Mid.	R	Top	8.29	15.51	53.4%	OK
		Botton	6.27	15.51	40.4%	OK
Abut.	S	Top	5.19	15.51	33.5%	OK
		Botton	4.85	15.51	31.3%	OK
To the whole bridge		Max T.	8.92	15.51	57.5%	OK
		Min T.	-1.92	-2.35	81.7%	OK
		Max B.	13.55	15.51	87.4%	OK
		Min B.	0.73	15.51	4.7%	OK

Load Case:		ONT-825				
Span	Sec. No.	T. or B.	σ	[σ]	%	Status
1	2	3	4	5	4/5	7
Abut	A	Top	5.19	15.51	33.5%	OK
		Botton	4.85	15.51	31.3%	OK
1 Mid.	B	Top	8.60	15.51	55.4%	OK
		Botton	6.55	15.51	42.2%	OK
pier 1	C	Top	-2.43	-2.35	103.4%	Not OK
		Botton	14.10	15.51	90.9%	OK
2 Mid.	D	Top	8.27	15.51	53.3%	OK
		Botton	7.10	15.51	45.8%	OK
pier 2	E	Top	-1.26	-2.35	53.6%	OK
		Botton	12.79	15.51	82.5%	OK
3 Mid.	F	Top	7.12	15.51	45.9%	OK
		Botton	8.43	15.51	54.4%	OK
pier 3	G	Top	-0.58	-2.35	24.7%	OK
		Botton	12.07	15.51	77.8%	OK
4 Mid.	H	Top	7.46	15.51	48.1%	OK
		Botton	8.11	15.51	52.3%	OK
pier 4	I	Top	-1.19	-2.35	50.6%	OK
		Botton	12.77	15.51	82.3%	OK
5 Mid.	J	Top	7.35	15.51	47.4%	OK
		Botton	8.21	15.51	52.9%	OK
pier 5	K	Top	-1.19	-2.35	50.6%	OK
		Botton	12.77	15.51	82.3%	OK
6 Mid.	L	Top	7.46	15.51	48.1%	OK
		Botton	8.11	15.51	52.3%	OK
pier 6	M	Top	-0.59	-2.35	25.1%	OK
		Botton	12.07	15.51	77.8%	OK
7 Mid.	N	Top	7.13	15.51	46.0%	OK
		Botton	8.43	15.51	54.4%	OK
pier 7	O	Top	-1.26	-2.35	53.6%	OK
		Botton	12.80	15.51	82.5%	OK
8 Mid.	P	Top	8.27	15.51	53.3%	OK
		Botton	7.10	15.51	45.8%	OK
pier 8	Q	Top	-2.43	-2.35	103.4%	Not OK
		Botton	14.10	15.51	90.9%	OK
9 Mid.	R	Top	8.60	15.51	55.4%	OK
		Botton	6.55	15.51	42.2%	OK
Abut.	S	Top	5.19	15.51	33.5%	OK
		Botton	4.85	15.51	31.3%	OK
To the whole bridge		Max T.	9.16	15.51	59.1%	OK
		Min T.	-2.43	-2.35	103.4%	Not OK
		Max B.	14.10	15.51	90.9%	OK
		Min B.	0.47	15.51	3.0%	OK

Page 13

Concrete Stress under the SLS status (2)

Load Case:		LRT				
Span	Sec. No.	T. or B.	σ	[σ]	%	Status
1	2	3	4	5	4/5	7
Abut.	A	Top	5.19	15.51	33.5%	OK
		Botton	4.85	15.51	31.3%	OK
1 Mid.	B	Top	7.76	15.51	50.0%	OK
		Botton	6.21	15.51	40.0%	OK
pier 1	C	Top	-2.78	-2.35	118.3%	Not OK
		Botton	14.57	15.51	93.9%	OK
2 Mid.	D	Top	7.64	15.51	49.3%	OK
		Botton	6.95	15.51	44.8%	OK
pier 2	E	Top	-1.76	-2.35	74.9%	OK
		Botton	13.43	15.51	86.6%	OK
3 Mid.	F	Top	6.49	15.51	41.8%	OK
		Botton	8.28	15.51	53.4%	OK
pier 3	G	Top	-1.08	-2.35	46.0%	OK
		Botton	12.7	15.51	81.9%	OK
4 Mid.	H	Top	6.85	15.51	44.2%	OK
		Botton	7.95	15.51	51.3%	OK
pier 4	I	Top	-1.7	-2.35	72.3%	OK
		Botton	13.41	15.51	86.5%	OK
5 Mid.	J	Top	6.75	15.51	43.5%	OK
		Botton	8.05	15.51	51.9%	OK
pier 5	K	Top	-1.7	-2.35	72.3%	OK
		Botton	13.41	15.51	86.5%	OK
6 Mid.	L	Top	6.85	15.51	44.2%	OK
		Botton	7.95	15.51	51.3%	OK
pier 6	M	Top	-1.08	-2.35	46.0%	OK
		Botton	12.7	15.51	81.9%	OK
7 Mid.	N	Top	6.5	15.51	41.9%	OK
		Botton	8.27	15.51	53.3%	OK
pier 7	O	Top	-1.77	-2.35	75.3%	OK
		Botton	13.43	15.51	86.6%	OK
8 Mid.	P	Top	7.64	15.51	49.3%	OK
		Botton	6.95	15.51	44.8%	OK
pier 8	Q	Top	-2.78	-2.35	118.3%	Not OK
		Botton	14.57	15.51	93.9%	OK
9 Mid.	R	Top	7.77	15.51	50.1%	OK
		Botton	6.21	15.51	40.0%	OK
Abut.	S	Top	5.19	15.51	33.5%	OK
		Botton	4.85	15.51	31.3%	OK
To the whole bridge		Max T.	8.51	15.51	54.9%	OK
		Min T.	-2.78	-2.35	118.3%	Not OK
		Max B.	14.57	15.51	93.9%	OK
		Min B.	1.17	15.51	7.5%	OK

Load Case:		LRT - with LWF				
Span	Sec. No.	T. or B.	σ	[σ]	%	Status
1	2	3	4	5	4/5	7
Abut.	A	Top	5.19	15.51	33.5%	OK
		Botton	4.85	15.51	31.3%	OK
1 Mid.	B	Top	7.5	15.51	48.4%	OK
		Botton	6.21	15.51	40.0%	OK
pier 1	C	Top	-2.24	-2.35	95.3%	OK
		Botton	13.96	15.51	90.0%	OK
2 Mid.	D	Top	7.33	15.51	47.3%	OK
		Botton	6.95	15.51	44.8%	OK
pier 2	E	Top	-1.19	-2.35	50.6%	OK
		Botton	12.78	15.51	82.4%	OK
3 Mid.	F	Top	6.19	15.51	39.9%	OK
		Botton	8.28	15.51	53.4%	OK
pier 3	G	Top	-0.52	-2.35	22.1%	OK
		Botton	12.06	15.51	77.8%	OK
4 Mid.	H	Top	6.55	15.51	42.2%	OK
		Botton	7.95	15.51	51.3%	OK
pier 4	I	Top	-1.13	-2.35	48.1%	OK
		Botton	12.77	15.51	82.3%	OK
5 Mid.	J	Top	6.44	15.51	41.5%	OK
		Botton	8.05	15.51	51.9%	OK
pier 5	K	Top	-1.13	-2.35	48.1%	OK
		Botton	12.77	15.51	82.3%	OK
6 Mid.	L	Top	6.54	15.51	42.2%	OK
		Botton	7.95	15.51	51.3%	OK
pier 6	M	Top	-0.52	-2.35	22.1%	OK
		Botton	12.06	15.51	77.8%	OK
7 Mid.	N	Top	6.2	15.51	40.0%	OK
		Botton	8.27	15.51	53.3%	OK
pier 7	O	Top	-1.19	-2.35	50.6%	OK
		Botton	12.78	15.51	82.4%	OK
8 Mid.	P	Top	7.32	15.51	47.2%	OK
		Botton	6.95	15.51	44.8%	OK
pier 8	Q	Top	-2.23	-2.35	94.9%	OK
		Botton	13.96	15.51	90.0%	OK
9 Mid.	R	Top	7.51	15.51	48.4%	OK
		Botton	6.21	15.51	40.0%	OK
Abut.	S	Top	5.19	15.51	33.5%	OK
		Botton	4.85	15.51	31.3%	OK
To the whole bridge		Max T.	8.47	15.51	54.6%	OK
		Min T.	-2.24	-2.35	95.3%	OK
		Max B.	13.96	15.51	90.0%	OK
		Min B.	1.47	15.51	9.5%	OK

POST - TENSIONED DECK (SOLID SLAB)
DESIGN FOR SERVICEABILITY LIMIT STATES - AS PER CHBDC CAN/CSA-S6-06

Important Note: Only data presented in black is to be input. Red coloured data means it is calculated by the spreadsheet

	span 1	span 2	span 3
Length of slab between BRG's (m):	3.81	7.0104	3.7335
Length of solid end portion from support (m):	3.81	7.0104	3.7335
Thickness of slab (m):	0.9144	0.9144	0.9144
Eff. Width of Solid Slab (m):	5.283	5.283	5.283

Length of bridge = 14.554 m

SIGN CONVENTION		
Force		
	-ve	comp.
Moment (tens. side)		
	+ve	sagging
	-ve	hogging

1) MATERIAL PROPERTIES

a) prestressing steel

Strand Type	Low - Relaxation		Fpy/Fpu = 0.9
Ep		188,158	MPa
Size Designation	7 wire	15	mm
Grade		1860	MPa
Nominal Diameter		15.2	mm
Nominal Area		140.0	mm^2
# Strands / Tendon		12	strands
Number of Tendons		10.00	tendons
Area per Tendon		1680	mm^2

	span1	span2	span3
Dist from center of tendon's duct to bot (at sag)- mm	555.7	321.7	555.7
Eccentricity of Prestressing (Sag) - mm	98.5	135.5	98.5
Diameter of duct (outside)	87.3 mm		
min. net vertical spacing of tendon:	0 mm		
min. vertical tendon spacing(c/c)	87.3 mm		
tendon vertical spacing at anchorage(c/c):	0 mm		

b) Reinforcing Steel

Specified yield strength of rebar fy:	400	Mpa
Elastic modulus Es	200,000	Mpa
ϕ_s	0.9	

c) post-tensioned deck concrete properties

f'c,g		34.47379	MPa	28-day girder concrete compressive strength	
Ecg =		27780.4	MPa	young modulus of naked section	
fcr,g = 0.40*(f'c,g)^.5		2.35	MPa	all. girder tensile stress = craking strength	17.10.2.3
f'ci,g	75.0 % of f'c	25.9	MPa	@ transfer section concrete compressive strength	
Eci =		25106.1	MPa	youngs modulus of naked section @ transfer	
fcri = 0.4*(f'ci)^.5		2.03	MPa	all. transfer section tensile stress = craking strength @ transfer	

ϕc = 0.75 — 17.10.2.3

2) STRESSES/FORCES IN PRESTRESSING STRAND

CL 8-7.1

		Stress	Force per Tendon		
fpu		1860 MPa	3125 kN	breaking strength	
fpy = 0.9*fpu	0.9	1674 MPa	2812 kN		
fsj = 0.80*fpu	0.8200	1525 MPa	2562 kN	@ jacking	CL 8-7.1
D(fs1)	PS loss @ transfer	229 MPa	384 kN	% = 15%	
fst = fsj - D(fs1)		1297 MPa	2178 kN	@ transfer	
fst, max = 0.70*fpu	0.70	1302 MPa	2187 kN		CL 8-7.1
fst < fst, max		O.K.			
D(fs) = D(fs1) + D(fs2)	total PS Loss	345 MPa	579 kN	% = 23%	
fse = fsj - D(fs)		1180 MPa	1983 kN	@ after all losses	
fse, min = 0.45*fpu	0.45	837 MPa	1406 kN		CL 8-7.1
fse > fse, min		O.K.			

Total Tendon Force @ Transfer	21783.8	kN
Total Effective Tendon Force (after All Losses)	19828.6	kN

File Name :post-tent-ana-HS-25-trans
Sheet Name :input

TSH Associates	Project:		Page	15
	Project No.		Date	19-Nov-08
Structure:	Cal. by	TNS	Check by	

POST - TENSIONED DECK (SOLID SLAB)
DESIGN FOR SERVICEABILITY LIMIT STATES - AS PER CHBDC CAN/CSA-S6-06

Important Note: Only data presented in black is to be input. Red coloured data means it is calculated by the spreadsheet

	span 1	span 2	span 3
Length of slab between BRG's (m):	3.81	7.0104	3.7335
Length of solid end portion from support (m):	3.81	7.0104	3.7335
Thickness of slab (m):	0.9144	0.9144	0.9144
Eff. Width of Solid Slab (m):	5.283	5.283	5.283

Length of bridge = 14.554 m

SIGN CONVENTION
Force
-ve comp.
Moment (tens. side)
+ve sagging
-ve hogging

Concrete Stress Limitations CL 8.8.4.6

At transfer

compression:	0.60*f'ci	15.5132 Mpa	8.8.4.6.(a.1)	
tension:	0.50*fcri	-1.02 Mpa	8.8.4.6.(a.2)	Rebar is not needed

At service:

compression	0.45f'c	15.51 Mpa	OHBDC 8.8.4.6.b
tension:	fcr	-2.35 Mpa	No crack

3) GEOMETRY

a) post-tensioned deck

Ybg	457	mm	N.A. of section from bottom
Ag	4.831	m^2	section cross sectional area
Ig	0.337	m^4	section moment of intertia
radius of gyration rg	0.264	m	
Sbg	0.738	m^3	section modulus from bottom
Stg	0.738	m^3	section modulus from top

Converted section considering prestress tendons

n = Es / Ec 6.70

	A - m2	ycl - m	A*ycl	I - m4	d =yc-ycl	A*d^2
concrete	4.831	0.4572	2.20863	0.337	0.0019141	2E-05
tendons	0.09578	0.556	0.05322	0	0.097	0.0009
Σ	4.92655		2.26185	0.337		0.0009

yc =	0.459114108	m			
I =	0.337505664	m^4	S'bg	0.73512	m^3
A' =	4.92655	m^2	S'tg	0.74130	m^3

Calculation assumptions(optional)

Width for Flexual resistance & shear resistance calc.= 5.283 m
Width of total voids 0

File Name :post-tent-ana-HS-25-trans
Sheet Name :input

Page 16

TSH

Project: Eglinton Avenue-CNR Overhead Structure
Project #: 42-71256
Date: 9-Nov-08

Moment Resistances in ULS (Longitudinal)

Load cases	Ult. Mom. Capacity(Mr)	Max. middle span moment(+ve)			Ult. Mom. Capacity	Max. support moment (-ve)		
		Location	Mf	Mf/Mr		Location	Moment	Mf/Mr
	KN.m	Span	KN.m	%	KN.m	Pier	KN.m	%
HS-25	57865	4 & 6	30764	53%	-47473	1&8	-38702	82%
ONT-625	57865	4 & 6	32382	56%	-47473	1&8	-40680	86%
LRT	57865	4 & 6	29503	51%	-47473	1&8	-41375	87%
LRT-LWF	57865	4 & 6	28877	50%	-47473	1&8	-40181	85%

Shear Resistances in ULS

Load cases	Ult. Shear Capacity(Vr)	Maximum Shear Force		
		Location	Vf	Vf/Vr
	KN	Pier	KN	%
HS-25	12935	4 & 6	8178.6	63%
ONT-625	12935	4 & 6	8840.6	68%
LRT	12935	4 & 6	8397.2	65%
LRT-LWF	12935	4 & 6	8123.8	63%

TSH

Project: Eglinton Avenue-CNR Overhead Structure
Project #: 42-71256

Date: 9-Nov-08

Page 17

Allowable Compression Stress= 0.4 * f'c= 15.5 MPa
Allowable tension Stress= fcr = -2.3 Mpa

Transversal Stresses under SLS

Load cases	Allowable	Sections At Bearing point			Allowable	Sections at the mid span		
	Stress [σ]	σ	Mf/Mr	Status	**Stress [σ]**	Moment	Mf/Mr	Status
(Mf)	Mpa	Mpa	%		Mpa	Mpa	%	
HS-25	15.5 MPa	7.36 MPa	47%	OK	15.51 MPa	7.08 MPa	46%	OK
	-2.3 MPa	-1.24 MPa	53%	OK	-2.35 MPa	-0.96 MPa	41%	OK
ONT-625	15.5 MPa	7.42 MPa	48%	OK	15.51 MPa	7.7 MPa	49%	OK
	-2.3 MPa	-1.30 MPa	55%	OK	-2.35 MPa	-1.5 MPa	66%	OK
LRT	15.5 MPa	7.58 MPa	49%	OK	15.51 MPa	7.7 MPa	49%	OK
	-2.3 MPa	-1.46 MPa	62%	OK	-2.35 MPa	-1.5 MPa	66%	OK
LRT-LWF	15.5 MPa	6.56 MPa	42%	OK	15.51 MPa	7.7 MPa	49%	OK
	-2.3 MPa	-0.44 MPa	19%	OK	-2.35 MPa	-1.5 MPa	66%	OK

TSH

Project: Eglinton Avenue-CNR Overhead Structure
Project #: 42-71256

Date: 9-Nov-08

Page 18

Moment Resistances in ULS(Trans.)

Load cases	Ult. Mom.	Moment at Bearing Secction(-ve)			Ult. Mom.	Moment at Mid. Span(+ve)		
	Capacity(Mr)	**Mf**	**Mf/Mr**	**Status**	**Capacity**	**Moment**	**Mf/Mr**	**Status**
	KN.m	KN.m	%		KN.m	KN.m	%	
HS-25	-22535	-6297	28%	OK	22535	4255	19%	OK
ONT-625	-22535	-6574	29%	OK	22535	6621	29%	OK
LRT	-22535	-7610	34%	OK	22535	6621	29%	OK
LRT-LWF	-22535	-6575	29%	OK	22535	6621	29%	OK

Shear Resistances in ULS (Trans.)

Load cases	Ult. Shear	Maximum Shear Force		
	Capacity(Vr)	**Vf**	**Vf/Vr**	**Status**
	KN	KN	%	
HS-25	9811	4471	46%	OK
ONT-625	9811	5396	55%	OK
LRT	9811	6060	62%	OK
LRT-LWF	9811	5202	53%	OK