REPORT

September 6, 2019

GEOTECHNICAL AND PAVEMENT INVESTIGATION AND DESIGN REPORT

SCHEDULE 'B' MUNICIPAL CLASS ENVIRONMENTAL ASSESSMENT, ALBION VAUGHAN ROAD AND KING STREET, TOWN OF CALEDON, REGIONAL MUNICIPALITY OF PEEL

Submitted to:

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by CIMA+ (CIMA) on behalf of The Regional Municipality of Peel (Region) to provide geotechnical and pavement engineering services in support of the Class Environmental Assessment (Schedule B) study for improvements to Albion Vaughan Road and King Street intersection, Town of Caledon (see Figure 1).

The purpose of the investigation was to determine the subsurface soil and groundwater conditions at the two stream crossing structures near the intersection, as well as, where the creek meanders and is in close proximity to Albion Vaughn Road by means of a limited number of boreholes and, based on our interpretation of the data, to provide preliminary engineering recommendations on the geotechnical aspects of design of the project. The investigation and reporting was carried out in general accordance with the scope of work provided in our Proposal No. P1664714, dated September 16, 2016. The scope of work was developed based on the requirements of the Request for Proposal outlined in The Regional Municipality of Peel's Request for Proposal (RFP 16-4390) dated August 30, 2016.

The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location.

This report should be read in conjunction with "Important Information and Limitations of This Report", in Appendix A, following the text of this report. The reader's attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Site Description

The study area is located at the intersection of Albion Vaughan Road and King Street East/King Road; east of the intersection King Road is within the Regional Municipality of York and south of King Road, Albion Vaughan Road is within the Town of Caledon and King Street West and Caledon-King Townline (north of the intersection) are within the Regional Municipality of Peel, Ontario. Relative to the intersection, the study limits extend approximately 80 m north and 90 m south along Albion Vaughn Road, and approximately 100 m west along King Street East and 80 m east along King Road (see Figure 1 for a site location plan).

The intersection is situated in a rural residential setting and is currently a two lane road with one lane in each direction. Within the study area Cold Creek (which is a tributary of the Humber River) crosses the Caledon-King Townline approximately 80 north of the intersection and then it meanders in a southerly direction and crosses King Road East approximately 40 m east of the intersection. Cold Creek continues meandering to the south and at about 120 m south of the intersection along the Albion Vaughan Road the creek is about 8 m to east of the existing road. In general, the topography in the site area slopes towards the intersection and within the study limits the ground surface of the various roads varies from about Elevation 215 m to 210 m.

2.2 Project Description

It is understood that as part of the Class Environmental Assessment (Schedule B) consideration is being given to intersection improvement works including widening at Albion Vaughan Road and King Street. It is understood that as an interim, consideration is being given to constructing right turn lanes, but the ultimate improvements may consist of widening the intersection to accommodate two lanes each way. This ultimate widening may require that the existing structures that cross over Cold Creek on the Caledon-King Townline and on King Street East will require extension. In the interim a retaining wall (toe wall) may be required to permit construction of the right turn lanes.

3.0 INVESTIGATION PROCEDURES

The field work for the geotechnical and pavement investigation at the intersection was carried out between May 23 and 29, 2017, during which time a total of six boreholes were advanced for the pavement investigation (designated as Borehole 17-02 to 17-05, 17-08 and 17-11) and five boreholes were advanced for the foundation investigation (designated as Borehole 17-01, 17-06, 17-07, 17-09, and 17-10) were advanced at the locations shown on Figure 1.

The field borehole investigation was carried out using a truck-mounted CME-55 drill rig supplied and operated by Atcost Soil Drilling Inc., of Gormley, Ontario. The boreholes were advanced through the overburden using 102 mm outer diameter continuous flight solid stem augers. Soil samples were obtained at intervals of 0.75 m and 1.5 n intervals of depth, using a nominal 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586-08)[1].

The pavement boreholes were advanced to depths of 1.5 m to 2.3 m below ground surface and the foundation boreholes were advanced to depths of between 6.7 m and 9.8 m below ground surface.

The groundwater conditions and water levels in the open boreholes were observed during the drilling operations. Monitoring wells were installed in Boreholes 17-01, 17-07, and 17-10, to permit monitoring of the water level at those locations. The monitoring wells consist of 50 mm diameter PVC pipe, with a slotted screen sealed at a selected depth within the boreholes. The borehole and annulus surrounding the well pipe above the screen sand pack was backfilled to the ground surface with bentonite, in accordance with Ontario Regulation 903, Wells (as amended). Monitoring well installation details and water level readings are presented on the Record of Borehole sheets in Appendix B. In the boreholes not instrumented with a monitoring well, the borehole was backfilled with bentonite upon completion in accordance with Ontario Regulation 903, Wells (as amended), and restored with asphalt at road surface. A single well response test was carried out each of the three monitoring wells on May 30 and 31, 2017. During this time it was observed that the groundwater was slightly flowing above the top of the pipe (at ground surface). The monitoring well was sealed with a j-plug cap and the next day we installed a datalogger and a packer above the datalogger in order to temporarily prevent the water from flowing. We returned to site on July 17, 2017 and retrieved the datalogger and the packers from the monitoring well. The boreholes were then immediately decommissioned by removing the monitoring well using a truck-mounted CME-55 drill rig supplied and operated by Geo-Environmental Drilling Ltd. of Milton, Ontario. The monitoring well was removed by overcoring overtop of the monitoring well and then the borehole was immediately grouted with a cement bentonite mixture in order to resist the artesian groundwater pressure.

The field work was observed by a member of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground utilities, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the

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[1] ASTM D1586-11 – Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.

samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to ASTM standards, as appropriate. Classification testing (water content determination, grain size distribution, and Atterberg limits) was carried out on selected soil samples.

The as-drilled borehole locations were obtained using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the horizontal direction. The ground surface elevations at the borehole locations were determined from the topographic drawing provided by CIMA. The borehole locations in UTM NAD 83 northing and easting coordinates, the ground surface elevations referenced to Geodetic datum, and the drilled depths are summarized in Table 1:

Table 1: Borehole Northing, Easting, Elevation and Depth

Borehole No.	Location (UTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
17-01	4,860,127.0	602,998.4	210.0	6.7
17-02	4,860,177.5	602,964.0	209.0	2.1
17-03	4,860,215.8	602,888.8	210.2	2.1
17-04	4,860,205.5	602,849.1	212.0	2.1
17-05	4,860,197.8	602,804.2	214.8	2.1
17-06	4,860,291.4	602,915.8	209.9	8.2
17-07	4,860,313.3	602,917.0	210.0	8.2
17-08	4,860,379.8	602,884.3	210.0	2.1
17-09	4,860,241.3	602,968.7	210.0	9.8
17-10	4,860,254.1	602,995.9	211.1	9.8
17-11	4,860,263.2	603,050.3	214.2	2.1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located in the South Slope physiographic region as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[2].

The South Slope physiographic region covers portions of the Regional Municipalities of Peel, York and Durham. A surficial till sheet, which generally follows the surface topography, is generally present throughout much of this area. The till is typically comprised of clayey silt to silty clay, with occasional silt to sand zones and is mapped in this area as the Halton Till.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the investigation and the results of the laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets provided in Appendix B. The results of the in situ field tests (i.e. SPT “N” values) as

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[2] Chapman, L.J. and Putnam, D,F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.

presented on the Record of Borehole sheets and in sub sections of Section 4.2 are uncorrected. The geotechnical laboratory testing plots are contained in Appendix B.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions. A description of the soil classification symbols presented in subsections in Sections 4.2, are detailed in Appendix B: Method of Soil Classification.

In general, the boreholes advanced encountered the pavement structure at ground surface, underlain by fill materials comprised of inter-layered deposits of loose to very dense sand and gravel to gravelly sand to silty sand, and silt and sand to stiff to hard clayey silt to sandy clayey silt. In Boreholes 17-03, 17-04, 17-06, 17-08 the fill material is underlain by a deposit of sand to sandy silt. In Boreholes 17-01, 17-06, 17-07, 17-09 and 17-10, advanced at the location of the proposed bridge extension and where the creek meanders close to Albion Vaughn Road, the fill material is underlain by a deposit of silt to clayey silt to silty clay.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Existing Pavement Structure

The existing pavement structure at each borehole is summarized in Table 2. The granular material is further described in the following sections.

Table 2: Existing Pavement Structure

Borehole	Asphalt Layer (mm)	Granular Layer (mm)
17-01	180	580
17-02	130	630
17-03	130	780
17-04	130	500
17-05	130	630
17-06	150	760
17-07	130	630
17-08	150	610
17-09	130	1,140*
17-10	180	610
17-11	130	630
Average	140	630

Note: * Not included in average thickness calculation

4.2.2 Fill – (CL/ML) Sandy Clayey Silt to (SW/GW) Sand and Gravel

Underlying the asphalt in all boreholes advanced at this site was variable fill material consisting of cohesive and granular material was encountered below the asphalt in all boreholes. The depth of and elevation of the surface of the fill layer, thickness and base elevation of the unit, as encountered in the boreholes is summarized in Table 3 below.

Table 3: Summary of Fill Surface Depths and Elevations, Deposit Thickness and Deposit Base Elevation

Borehole No.	Depth to Surface of Layer (m)	Fill Surface Elevation (m)	Fill Thickness (m)	Fill Base Elevation (m)
17-01	0.18	209.9	3.9	205.9
17-02	0.13	208.9	Greater than 2.1	Below 206.9
17-03	0.13	210.1	0.8	209.3
17-04	0.13	211.9	0.9	211.0
17-05	0.13	214.7	Greater than 2.1	Below 212.7
17-06	0.15	209.7	1.4	208.3
17-07	0.13	209.9	2.9	207.0
17-08	0.15	209.9	1.4	208.5
17-09	0.13	209.9	2.1	207.8
17-10	0.18	210.9	1.3	209.6
17-11	0.13	214.1	0.6	213.4

The cohesive layers consist of sandy clayey silt to silty clay. The granular fill layers vary from sand and gravel to silt and sand. Organics (rootlets and wood fragments) were noted to be present within the fill in Boreholes 17-01, 17-02, 17-04, 17-05, 17-07, 17-09 and 17-10. Boreholes 17-02 and 17-05 were terminated within the fill layers at depths of about 2.1 m (Elevation 206.9 m and 212.7 m, respectively) below ground surface.

The SPT “N”-values measured within the granular fill layers range from 4 blows to 77 blows per 0.3 m of penetration, indicating a loose to very dense relative density. The SPT “N”-values measured within the cohesive fill range from 4 blows to 32 blows per 0.3 m of penetration, suggesting a stiff to hard consistency.

The results of grain size distribution testing completed on two samples of cohesive fill are shown on Figure B1 in Appendix B. The sandy clayey silt to silty clay material presented in Figure B1 had low to moderate frost susceptibility. Atterberg limits testing carried out on one sample of the cohesive fill measured liquid limit of about 23 per cent, plastic limit of about 14 per cent and plasticity index of about 9 per cent. The test results, which are plotted on a plasticity chart on Figure B2, indicate that the material is classified as a silty clay of low plasticity.

The results of grain size distribution testing completed on two samples of sand and silt fill are shown on Figure B3 in Appendix B. The results of grain size distribution testing completed on two samples of gravelly sand to sand and gravel fill are shown on Figure B4 in Appendix B. Atterberg limits testing carried out on one sample of the silt and sand fill measured liquid limit of about 23 per cent, plastic limit of about 19 per cent and plasticity index of about 4 per cent. The test results, which are plotted on a plasticity chart on Figure B5, indicate that the material is classified as a silty sand of low plasticity.

The natural water contents measured on thirteen samples of the granular fill layers ranged from 3 per cent to 19 per cent. The natural water contents measured on samples of the cohesive fill layers ranged from 14 per cent to 21 per cent.

4.2.3 (SW/GW) Sandy Gravel to (SW) Silty Sand

Deposits of sandy gravel to silty sand were encountered underlying fill material in Boreholes 17-04, 17-06, 17-08 and in Borehole 17-11, underlying the silty clay deposit. The depth of and elevation of the surface of the granular layer, thickness and base elevation of the deposit, as encountered in the boreholes is summarized Table 4 below.

Table 4: Summary of Sandy Gravel to Silty Sand Surface Depths and Elevations, Deposit Thickness and Deposit Base Elevation

Borehole No.	Depth to Surface of Layer (m)	Surface Elevation of Layer (m)	Layer Thickness (m)	Base Elevation of Layer (m)
17-04	1.1	211.0	Greater than 1.1	Below 209.9
17-06	1.5	208.3	2.2	206.1
17-08	1.5	208.5	Greater than 0.6	Below 207.9
17-11	2.1	212.1	Greater than 0.05	Below 212.15

Boreholes 17-04 and 17-08 were terminated within the sand to silty sand deposit at a depth of about 2.1 m (Elevation 209.9 m and 207.9, respectively) below ground surface. Borehole 17-11 was terminated at a depth of 2.1 m (Elevation 212.1 m) below ground surface after penetrating 0.05 m into the deposit.

The results of grain size distribution testing completed on one sample of sandy gravel is shown on Figure B6 in Appendix B.

The natural water content measured on samples of the silty sand to sand range from about 9 per cent to 28 per cent.

4.2.4 (ML) Sandy Silt to Silt

A deposit of sandy silt to silt were encountered in Boreholes 17-01, 17-03 and 17-07. The depth of and elevation of the surface of the sandy silt to silt deposit, thickness and base elevation of the deposit, as encountered in the boreholes is summarized in Table 5 below.

Table 5: Summary of Sandy Silt to Silt Surface Depths and Elevations, Deposit Thickness and Deposit Base Elevation

Borehole No.	Depth to Surface of Deposit (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)
17-01	4.1	205.9	Greater than 2.6	Below 203.3
17-03	0.9	209.3	Greater than 1.2	Below 208.1
17-07	3.1	207.0	Greater than 5.2	Below 201.8

Borehole 17-01 was terminated within the silt deposit at a depth of 6.7 m (Elevation 203.3 m) below ground surface. Borehole 17-03 was terminated within the sandy silt deposit at a depth of 2.1 m (Elevation 208.1 m) below ground surface after penetrating 1.2 m into the deposit. Borehole 17-07 was terminated within the silt deposit at a depth of 8.2 m (Elevation 201.8 m) below ground surface after penetrating 5.2 m into the deposit.

The SPT “N”-values measured within the sandy silt to silt deposit range from 11 blows to 36 blows per 0.3 m of penetration, indicating a compact to dense relative density.

The results of grain size distribution testing completed on one sample of sandy silt is shown on Figure B7 in Appendix B.

Atterberg limits testing carried out on two samples of the silt both measured a liquid limit of about 20 per cent, a plastic limit of 18 per cent and a plasticity index of 2 per cent. The test results, which are plotted on a plasticity chart on Figure B8, indicate that the material tested is classified as a silt with slight plasticity.

The natural water content measured on samples of the sandy silt to silt range from about 20 per cent to 27 per cent.

4.2.5 (CL/ML) Clayey Silt to (CL) Silty Clay

Underlying the sandy gravel in Borehole 17-06 and underlying the fill in Boreholes 17-09 to 17-11 a deposit of clayey silt to silty clay was encountered. The depth of and elevation of the surface of the clayey silt to silty clay deposit, thickness and base elevation of the deposit, as encountered in the boreholes is summarized in Table 6 below.

Table 6: Summary of Clayey Silt to Silty Clay Surface Depths and Elevations, Deposit Thickness and Deposit Base Elevation

Borehole No.	Depth to Surface of Deposit (m)	Deposit Surface Elevation (m)	Deposit Thickness (m)	Deposit Base Elevation (m)
17-06	3.8	206.0	Greater than 4.4	Below 201.6
17-09	2.2	207.8	Greater than 7.5	Below 200.3
17-10	1.5	209.6	Greater than 8.2	Below 201.4
17-11	0.8	213.4	1.3	212.1

Borehole 17-06 terminated within the clayey silt deposit at a depth of 8.2 m (Elevation 201.6 m) below ground surface. Borehole 17-09 and 17-10 terminated within the silty clay deposit at depths of 9.8 m (Elevation 200.3 m and 201.4 m, respectively) below ground surface. The silty clay encountered in Borehole 17-11 contains sand inclusions.

The SPT “N”-values measured within the clayey silt to silty clay range from 6 to 69 blows per 0.3 m of penetration, suggesting a firm to hard consistency.

The results of grain size distribution testing completed on two samples of silty clay are shown on Figure B9 in Appendix B. Atterberg limits testing carried out on four samples of silty clay measured liquid limits range from about 24 per cent to 31 per cent, plastic limits ranging from 16 per cent to 20 per cent and a plasticity indices from about 8 per cent to 11 per cent. The test results, which are plotted on a plasticity chart on Figure B10, indicate that the material tested is classified as a silty clay of low to medium plasticity.

The natural water content measured on samples of the clayey silt to silty clay range from about 15 per cent to 30 per cent.

4.2.6 Corrosivity Testing Results

Two soil samples were submitted for analysis of parameters used to assess the potential corrosivity of the site soils to steel and concrete. The detailed test results are presented in Appendix C and summarized in Table 7.

Table 7: Results of Corrosivity Testing

Parameter	BH17-6 SA4	BH17-10 SA10
pH	7.72	7.92
Resistivity (ohm-cm)	980	1600
Conductivity (umho/cm)	1020	614
Chlorides (ug/g)	460	270
Soluble Sulphate (ug/g)	140	21

4.3 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist to wet. The shallow boreholes advanced for pavement evaluation purposes were dry upon completion of drilling. In Boreholes 17-01, 17-06 and 17-07 the measured water level varied between depths of about 2.5 m and 3.1 m (between Elevations 207.4 m and 206.9 m) upon completion of drilling. In Boreholes 17-09 and 17-10 the measured water level was encountered at depths of about 6.9 m and 7.8 m (Elevations 203.1 m and 203.3 m) upon completion of drilling, respectively.

A monitoring well was installed in Boreholes 17-01, 17-07 and 17-10 to permit monitoring of the groundwater levels at the site. The details of the groundwater levels observed in the open boreholes and standpipe piezometer installed in Boreholes 17-01, 17-07 and 17-10 are summarized on the Records for Borehole sheets in Appendix B of this report. A summary of the measured groundwater levels (depth, elevation and date) in the monitoring wells is presented the Table 8 below:

Table 8: Summary of Water Levels in the Monitoring Well

Borehole	Ground Surface Elevation (m)	Material Sealed Into	Depth to Water level (m)	Groundwater Elevation (m)	Date of Water Level Measurement
17-01	210.0	Fill - Sand and Gravel / Silt	2.59	207.4	May 17, 2017
			2.5	207.5	May 30, 2017
			2.59	207.4	July 17, 2017
17-07	210.0	Silt	- 0.87[1]	210.87	May 31, 2017
			- 0.87	210.87	July 17, 2017
17-10	211.1	Silty Clay	- 0.14	211.24	May 30, 2017

			- 0.14	211.24	July 17, 2017

Notes:

1. Negative depth indicates that the groundwater level is above ground surface.

It is noted that the groundwater level measured in Boreholes 17-07 and 17-10 is above ground surface; therefore, the groundwater silt / silty clay deposit is artesian. The groundwater level is expected to fluctuate seasonally and in response to changes in precipitation and snow melt, and is expected to be higher wet periods of the year.

4.4 Pavement - Visual Condition Inspection

A member of Golder’s Pavement and Materials Engineering Group carried out a visual condition inspection of the existing pavement at the intersection in April 2017. The North leg of the intersection has a rural cross section with gravel shoulders and ditches for drainage. The South, East and West sections have curbs and catch basins for drainage with the exception of one portion of the East section that has a gravel shoulder and ditching. The pavement is generally in good condition. The following types, severities and densities of surface distresses were observed:

- Few, slight to moderate alligator cracking;
- Few, moderate meandering cracking;
- Few, moderate pavement edge cracking;
- Intermittent, moderate opening of construction joints; and
- Few, slight transverse cracks.

Photo 1 through Photo 4 show examples of the condition of the existing pavement.

Photo 1: Pavement in good condition on North leg of intersection

Photo 2: Alligator cracking on South leg of intersection

Photo 3: Longitudinal construction joint opening on West leg of intersection

Photo 4: Pavement edge cracking on East leg of intersection

4.5 Falling Weight Deflectometer

The Falling Weight Deflectometer (FWD) load/deflection testing was carried out on each through lane at each leg of the intersection of King Road and Albion Vaughan Road. The road name for each leg of the intersection was as follows:

- North Leg – Caledon Townline South;
- South Leg – Albion Vaughan Road;
- East Leg – King Road; and
- West Leg – King Street East.

For Caledon Townline South and Albion Vaughan Road, the testing was carried out in both the northbound and southbound directions. For King Road and King Street East, the testing was carried out in both the eastbound and westbound directions. The limits for the testing on each road are shown in Photo 5. For each tested lane, the testing was carried out at 25 m intervals, and the test points in adjacent lanes were staggered by 12.5 m.

Photo 5: Limits for FWD Testing

The FWD load/deflection testing was carried out on the above mentioned pavement section on September 1, 2017. Testing was performed by a calibrated FWD unit owned by Golder. During the FWD testing an impulse load similar in magnitude and duration to a moving truck wheel load was applied to a loading plate sitting on the pavement surface. The response of the pavement (resulting pavement deflection) to the applied load was measured using eight (8) seismic transducers (geophones) spaced at predetermined intervals from the centre of the loading plate (0, 200, 300, 500, 600, 900, 1200 and 1500 mm). From these deflection readings the deflection basin at a particular location was determined. At each test location three selected load impulses of about 40 kN, 55 kN and 70 kN were applied to the pavement and deflections were measured for each load pulse.

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides preliminary foundation and pavement engineering design recommendations for the proposed extension of two bridges over Cold Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

5.2 Foundations for Bridge Structures

5.2.1 Consequence and Site Understanding Classification

It is understood that the bridge extensions are to be designed in accordance with the CHBDC (2014). In accordance with Section 6.5 of the CHBDC and its Commentary, the proposed bridges and their foundation systems are considered to be classified as having a “typical consequence level” associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at these locations in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a “typical” degree of site and prediction model understanding. Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ and Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.2 of the CHBDC have been used for design.

5.2.2 Bridge along Caledon King Townline Road South

The existing single span bridge over Cold Creek along Caledon King Townline Road South (north of the intersection) consists of a single span bridge structure founded on shallow foundations. Based on the General Arrangement Drawing No. S6-1, Project 04-01, provided by CIMA, dated April 2004 (see Appendix D), the bridge is about 25 m wide. The bridge is supported on shallow foundations and the top of the footings are at Elevation 206.35 m, with the underside of the footings are at approximately Elevation 205.35 m (assuming a 1 m thick footing).

It is understood that the bridge may be widened in the future to accommodate two lanes in each direction.

5.2.2.1 Shallow Foundations

5.2.2.1.1 Founding Elevation and Frost Protection Requirements

For support of the abutments and associated wingwalls for the widened portion of the bridge structure, spread/strip footings should be founded below any existing organics and fill material on the clayey silt to silt deposit. It is assumed that the new footings will be founded to match the existing footing founding level of approximately

Elevation 205.35 m. Strip or spread footings should be founded at a minimum depth of 1.5 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)*.

5.2.2.1.2 Factored Geotechnical Resistance

The groundwater level in the silt to clayey silt was measured as above ground surface and is therefore artesian. Prior to excavation for the strip footings the groundwater table must be lowered to 2 m below the founding elevations provided in Section 5.2.2.1.1, otherwise, depending on the thickness of the cohesive deposit there is the potential for basal failure. Strip footings placed on the properly prepared native clayey silt to silt subgrade at the founding elevations provided above should be designed on a factored ultimate geotechnical resistance of 300 kPa and a factored serviceability geotechnical resistance (for 25 mm of settlement) of 250 kPa.

The geotechnical resistances provided above are given for loads will that be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Table 10.2 in CFEM (2006). The base of each footing excavation should be cleaned of loose / softened material. It is recommended to minimize construction traffic on the footing subgrade as the silt to clayey silt is susceptible to disturbance. It is recommended that the founding level for the footings be inspected by geotechnical personnel immediately prior to pouring concrete to confirm the adequacy of the foundation conditions for the above noted geotechnical resistances. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thickness of 20 MPa compressive strength concrete) be placed on the subgrade within three hours to protect the integrity of the bearing stratum.

These preliminary geotechnical resistances will have to be re-evaluated during detailed design, subject to additional borehole and groundwater information within the footprint of shallow foundation elements, if adopted.

5.2.2.1.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC. For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the generally very stiff to hard clayey silt to silt soils, the coefficient of friction, tan δ or tan ϕ', can be taken as follows:

- Cast-in-place footing to clayey silt to silt deposits: tan ϕ' = 0.47

5.2.3 Bridge along King Road

The existing bridge over Cold Creek along King Road (east of the intersection) consists of a three span bridge structure founded on driven steel-H-piles. Based on the design drawings provided by CIMA, dated March 2002, the bridge has a centre span of 20.2 m and the abutment to pier span is 13.1 m. The drawing indicates that the bridge structure is founded on steel H-piles consisting of HP 310x110 piles driven into dense till; no additional information is available on the foundation elements.

It is understood that the bridge may be widened in the future to accommodate two lanes in each direction.

5.2.3.1 Shallow Foundations

5.2.3.1.1 Founding Elevation and Frost Protection Requirements

For support of the abutments and associated wingwalls for the widened portion of the bridge structure, spread/strip footings should be founded below any existing organics and fill material on the silty clay deposit at about Elevation 205 m.

Strip or spread footings should be founded at a minimum depth of 1.5 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario)*. If adequate soil cover cannot be provided for the footing, rigid Styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

5.2.3.1.2 Factored Geotechnical Resistance

The groundwater level in the silt clay was measured as above ground surface and is therefore artesian. Prior to excavation for the strip footings the groundwater table must be lowered to 2 m below the founding elevations provided in Section 5.2.3.1.1, otherwise, depending on the thickness of the cohesive deposit there is the potential for basal failure. Strip footings placed on the properly prepared native silty clay subgrade at the founding elevations provided above should be designed on a factored ultimate geotechnical resistance of 350 kPa and a factored serviceability geotechnical resistance (for 25 mm of settlement) of 300 kPa.

The geotechnical resistances provided above are given for loads will that be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Table 10.2 in CFEM (2006). The base of each footing excavation should be cleaned of loose / softened material. It is recommended to minimize construction traffic on the footing subgrade as the silty clay is susceptible to disturbance. It is recommended that the founding level for the footings be inspected by geotechnical personnel immediately prior to pouring concrete to confirm the adequacy of the foundation conditions for the above noted geotechnical resistances. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thickness of 20 MPa compressive strength concrete) be placed on the subgrade within three hours to protect the integrity of the bearing stratum.

These preliminary geotechnical resistances will have to be re-evaluated during detail design, subject to additional (deeper) borehole and groundwater information within the footprint of shallow foundation elements, if adopted.

5.2.3.1.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC. For cast-in-place concrete footings constructed on a concrete working slab that is cast on top of the generally very stiff to hard silty clay, the coefficient of friction, tan δ or tan ϕ’, can be taken as follows:

- Cast-in-place footing to silty clay deposits: tan ϕ’ = 0.47

5.2.4 Widened Approach Embankments

5.2.4.1 Subgrade Preparation and Embankment Construction

Widened approach embankments constructed of suitable earth fill or granular fill and up to about 4 m high will be required. Prior to placement of fill material for construction of the embankment the existing topsoil and organics must be removed. For satisfactory performance of the approach embankments, the existing organic material should be removed within the footprint of the widened approach.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS.PROV 804 (Seed and Cover).

5.2.4.2 Approach Embankment Stability and Settlement

Based on observations at the time of Golder's 2017 field investigation, the existing embankment side slopes at both structures have performed satisfactorily, with no visual evidence of instability or settlement. Given that the native soils are predominantly comprised of very stiff to hard cohesive soil deposits at this site, stability issues are not anticipated within the limits of the widened approach embankments.

5.3 Assessment of Slope at Cold Creek Meander Adjacent to Albion Vaughan Road

At about 100 m south of the intersection at the site Cold Creek meanders and is within about 4 m of the edge of the pavement. The geomorphology study completed by Golder indicates that the expected future meandering is in a southerly direction. Borehole 17-01 was advanced in the vicinity of the where the meander is close to the road and encountered predominately granular fill material to a depth of 4.1 m below ground surface. The fill is underlain by dense silt and the groundwater level was measured at 2.6 m depth (Elevation 207.4 m).

Based on observations at the time of Golder's 2017 field investigation, the existing side slope in the area of the meander has performed satisfactorily, with no visual evidence of instability or settlement.

Table 9 presents the parameters used in the slope stability analyses for the slope at the meander adjacent to Albion Vaughan Road, based on field and laboratory test data as well as accepted correlations:

Table 9: Slope Stability Analysis Parameters – Albion Vaughan Road at the Cold Creek meander

Soil Deposit	Short-term (Undrained) Analysis			Long-term (Drained) Analysis		
	Bulk Unit Weight (kN/m^3)	**Effective Friction Angle Φ'**	**Undrained Shear Strength (kPa)**	**Unit Weight (kN/m^3)**	**Effective Friction Angle Φ'**	**Cohesion (kPa)**
Existing Fill	21	28°	--	21	28°	0
Dense silt	21	-	--	21	35°	0

The analysis results indicate that the existing 4 m high slope have a factor of safety greater than or equal 1.33 (short term case) to 1.54 (long term case) against global instability. Example static global stability results for both short-term (undrained) and long-term (drained) conditions are provided on Figures 3 and 4. It is noted that this should be re-assessed at detailed design and the analysis should be carried out based on the current conditions at the time of the assessment.

5.4 Seismic Design

5.4.1 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the borehole investigations. Based on the SPT 'N' values measured in the upper 30 m of the soil layers, below founding level, the site may be classified as Site Class C in accordance with Table 4.1 of the CHBDC, in the absence of any geophysical testing.

5.4.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC (2014), the peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.032	0.021	0.085
PGV (m/s)	0.027	0.043	0.069
Sa (0.2) (g)	0.055	0.085	0.138
Sa (0.5) (g)	0.037	0.055	0.086
Sa (1.0) (g)	0.021	0.032	0.049
Sa (2.0) (g)	0.01	0.016	0.025
Sa (5.0) (g)	0.0022	0.0037	0.0061
Sa (10.0) (g)	0.001	0.0016	0.0027

5.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and on any associated wingwalls will depend on the type and method of placement of the backfill material, the nature of the soils/embankment fill behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II should be used as backfill behind the abutment walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Granular 'B' Type III can be used if the excavation is dry. Backfill should be placed in a maximum of 200 mm loose lift thickness and nominally compacted. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.

- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall (Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the base of the walls (Figure C6.20(b) of the *Commentary* to the CHBDC).

5.5.1 Static Lateral Earth Pressures for Design

The following recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions.

- For restrained walls, the pressures are based on the soil strata adjacent to the culvert and the following parameters (unfactored) may be used assuming the use of earth fill or existing native materials:

Material	Earth Fill or Existing Native Materials
Soil Unit Weight:	20 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o	 0.33 0.50

- For unrestrained walls, the pressures are based on using engineered granular fill behind the walls and the following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II	Granular B Type III
Soil Unit Weight:	22 kN/m^3	21 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_p	 0.27 0.43 3.7	 0.27 0.43 3.7	 0.33 0.50 3.0

If the abutment does not allow lateral yielding, at-rest earth pressures should be assumed for the foundation design. If the abutment allows for lateral yielding, active earth pressures should be used in the foundation design. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC (2014).

5.5.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given in Section 6.4.1, above, plus the earthquake-induced dynamic earth pressure.

- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.

- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (restrained and unrestrained walls) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the walls is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

- Seismic Active Pressure Coefficients, K_{AE}

Wall Type	Design Earthquake	Site PGA	K_{AE} for Granular A	K_{AE} for Granular B Type II	K_{AE} for Earth Fill or Existing Native Materials
Yielding Wall	**2,475-Yr**	0.085	0.27	0.27	0.33
Non-Yielding Wall	**2,475-Yr**	0.085	0.30	0.30	0.36

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution).

5.6 Construction Considerations

The following sections identify future construction considerations that may impact the future design and construction.

5.6.1 Open-Cut Excavation

The construction of new footings and wingwalls for both bridge structures will extend to about Elevation 205 m, and will require excavations up to about 6 m below Caledon King Townline Road South and King Road and will be made through the existing embankment fill. The existing fill material and native loose silty sand and loose to dense sandy gravel, compact silt, hard clayey silt and firm to hard silty clay deposits are classified as Type 3 soils, according to the Occupational Health and Safety Act (OHSA) and, as such, temporary open-cut excavations above the groundwater level should be made with side slopes no steeper than 1H:1V. Prior to carrying out excavations for foundations the groundwater level must be lowered to 2 m below the base of the excavation.

All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

5.6.2 Temporary Protection Systems

Temporary protection systems may be required along Caledon King Townline Road South and King Road to facilitate the construction of the shallow foundations. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the Contractor.

5.6.3 Control of Groundwater

The standpipe piezometer installed at the Caledon King Townline Road South bridge site indicated that the water level measured in the silt deposit is above about 0.9 m above ground surface (Elevation 210.9 m), and the standpipe piezometer installed at the King Road bridge site indicated that the water level measured in the silty clay deposit is above about 0.1 m above ground surface (Elevation 211.2 m). Therefore, dewatering will be required prior to carrying out excavations for the shallow foundation construction at both sites.

Creek/ditch flows will need to be diverted or piped away from the excavation areas during the foundation construction period, or a cofferdam used to separate the foundation excavations and forming/pouring operations from the creek channel. Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade.

If Granular ‘A’ is used for bedding and backfill, placement and compaction should be carried out in dry conditions and this would likely be achieved by diverting/piping the existing creek and surface water away from the excavation. If wet conditions exist (due to precipitation for example), Granular ‘B’ Type II could be used for bedding and backfill.

5.6.4 Protection of Subgrade

The native deposits that will be exposed within the excavations at the proposed bridge sites will be susceptible to disturbance from construction traffic and/or precipitation and ponded water. To limit the effects of this disturbance, a concrete working slab should be placed on the subgrade within four hours after preparation, inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28-day compressive strength of 20 MPa.

5.7 Analytical Laboratory Testing

5.7.1 Corrosivity Testing

The results of an analytical test on two samples of soil from Borehole 17-06 and 17-10 are presented in Section 4.2.6 and in Appendix C. The analytical test results were compared to CSA A23.1 Table 3 *("Additional requirements for concrete subjected to sulphate attack")* for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the two soil samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

The analytical test results of the soil samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The resistivity measured indicates the corrosion potential to be severe. Based on the results of the samples tested, and given that the structure is located under/adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a “C” type exposure class as defined by CSA A23.1 Table 1.

It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

5.7.2 Geo-Environmental (Analytical) Testing

This section of the report provides preliminary management considerations for excavated soils that may be re-used on-site or disposed offsite during the construction of the proposed widening of the bridge structures. Samples of soils obtained from the site were submitted for chemical analyses to provide background information for assessment of the chemical quality of the soils at the site.

Phase I and II Environmental Site Assessments are outside the terms of reference for this Project and have not been carried out by Golder. Therefore, the results of the chemical testing described in this section of the report should not be construed as indicating that chemical impacts to the Site soils, beyond those described herein, do not exist at the Site.

5.7.2.1 Applicable Regulations and Guidance

Soil quality were evaluated relative to the generic site condition standards for a non-potable groundwater condition defined by O.Reg. 153/04 and presented in the Ministry of the Environment and Climate Change (MOECC)'s "*Soil, Ground Water and Sediment Standards for Use under Part XV.1 of the Environmental Protection Act*" dated April 15, 2011. To assess whether the soil quality in the tested areas of the Site may need to be addressed through soil management programs, the analytical results were compared to the MOE 2011 Table 1 Standards: Full Depth Generic Site Condition Standards in Background Condition for All Property Uses and for All Textured Soil (MOECC 2011 Table 1 Standards); and MOECC Table 3 Standards: Full Depth Generic Site Condition Standards in a Non Potable Ground Water Condition for Industrial, Commercial and Community Property Use for Medium to Fine Textured Soil (MOECC 2011 Table 3 Standards).

5.7.2.1.1 On-site reuse

To evaluate the suitability of the excavated materials to be re-used as backfill material on site, the Table 3 Soil Standards for "Full Depth Generic Site Condition Standards" in a Non-Potable Ground Water Condition for Industrial/ Commercial/Community Property Use are generally considered appropriate. Where excavated materials are to be placed below 1.5 m below ground surface, the Table 5 Soil Standards for "Stratified Site Condition Standards in a Non-Potable Ground Water Condition" may be considered appropriate.

5.7.2.1.2 Off-site Transfer

O.Reg.153/04 Record of Site Condition Properties

To evaluate the suitability of the excavated materials for unrestricted transferred to another site, the Table 1 Soil Standards for "Full Depth Background Site Condition Standards" for Residential / Parkland / Institutional/Industrial/Commercial Property Use were referenced for comparing soil sample analytical results. It should be noted that the applicable importation standard at a property subject to the submission and filing of a Record of Site Condition is defined by the Qualified Person acting on behalf of the receiving site and may correspond to one of the published standards listed in the MOECC document "Soil, Groundwater and Sediment Standards for Use under Part XV.1 of the EPA, April 15, 2011", or to a risk-based standard established by the Qualified Person representing the receiving site.

Non-Record of Site Condition Properties

In accordance with the MOECC guidance on "Management of Excess Soil – A Guide for Best Management Practices" issued in January 2014, beneficial reuse of excess soil at sites not subject to the requirements of O.Reg. 153/04 Records of Site Condition is based on the principle of imposing "no adverse effect". The importation requirements are to be established by a Qualified Person representing the receiving site.

5.7.2.2 Results of Testing

Four select soil samples were submitted to Maxxam Analytics (Maxxam) of Mississauga, Ontario, a Standards Council of Canada (SCC) accredited laboratory. The samples were submitted for analysis of metals and inorganics to assess the environmental quality of fill material encountered at the Site. The selected fill soil samples were collected from depths of between 0.9 m and 2.9 m below ground surface and selected native soil samples were collected from depths of between 6.1 m and 9.7 m below ground surface.

Soil analytical test results are presented in Table C1 in Appendix C. Laboratory Certificates of Analysis are also presented in Appendix C. A summary of the analytical results compared to the aforementioned standards are summarized below:

- Exceedances of the MOECC 2011 Table 1 and 3 Standards for Conductivity and Sodium Adsorption Ratio (SAR) were reported for fill soil samples from Boreholes 17-01 and 17-03.
- The results meet the MOECC 2011 Table 5.

The elevated inorganics contamination (Conductivity and SAR impact) in soil is believed to be attributed to the application of road salt and limited to the surficial or near surface soils.

5.7.2.3 Soil Management Conclusions

The following section presents preliminary soil management strategies based on the analytical results of the limited environmental assessment program. It should be noted that during construction the contractor will be required to carry out additional chemical testing prior to develop a comprehensive soil management program.

The limited soil sampling program conducted at the site of the future widening of the structures on King Road and Caledon King Townline indicated that the quality of all soil parameters met the Table 1 Standards and Table 3 Standards, with the exception of SAR and conductivity at the locations sampled.

While unrestricted off-site transfer is not applicable due to the identified exceedances of the Table 1 Standard for SAR and conductivity, beneficial re-use of excess fill at either a Record of Site Condition property or another property may be feasible if the soil quality meets the requirements for importation to that property as defined by the Qualified Person (QP) representing the receiving site. Written approval is required from the Qualified Person (QP) of receiving site.

While the soil samples collected and analysed as part of this program exceeded Table 3 Standards, the exceedances were limited to conductivity and SAR. As such, the analytical results were also compared to the Table 5 Standards for stratified site conditions, to assess potential re-use on-site below 1.5 m. As the standards for SAR and conductivity are based on direct contact with terrestrial plants and soil invertebrates, they therefore apply only to the upper 1.5 m of soil below ground surface. Therefore, on the basis of the limited sampling program undertaken, excess soil from the site may be suitable for on-site re-use at depths greater than 1.5 m below ground surface.

5.7.3 Limitations of the Environmental Investigation

The environmental interpretation of the analytical results presented herein is intended to provide a generalized assessment of the environmental conditions of the site within limited portions and areas of the proposed widening of the existing structures. The soil chemistry was assessed according to the chemical analysis results for a limited number of parameters and samples. The nature and extent of environmental chemistry between the sampling points can vary in terms of the conditions encountered at the locations where the analyzed samples were taken.

The findings are based on conditions as they were observed at the time of the investigation, and to a large degree, on interpretation of data obtained from boreholes and selected soil samples. No assurance can be provided with respect to the potential changed physical and/or chemical characteristics of the soil between or beyond the tested locations or the effects of subsequent activities on site. With respect to regulatory compliance issues, regulatory statutes and the interpretation of regulatory statute are also subject to change over time.

No matter how thorough an investigation may be, findings derived from the sampling and testing are limited and Golder cannot know or state for an absolute fact that areas of the property, or neighboring properties, or portions thereof, are unaffected by contaminants. The property owner bears risk that such contaminants may be present on, or may migrate to or off the property after the study is complete.

5.8 Falling Weight Deflectometer Analysis

5.8.1 Normalized Deflection and Pavement Surface Modulus

The measured deflections were normalized to represent a standard wheel load of 40 kN and a standard temperature of 21°C. In addition to normalizing the measured deflections, the analysis of the FWD data also involved determination of the pavement surface modulus. Pavement surface modulus is determined using the normalized deflection measured by the geophone located at the centre of the loading plate. Pavement surface modulus is an indication of the overall load bearing/support characteristics of the entire pavement structure. The detailed analysis results are provided in APPENDIX E.

Table 10 shows a summary of the normalized deflections and pavement surface modulus for the tested lanes of the intersection section. The typical pavement surface modulus for a medium to heavy traffic asphalt pavement in relatively good condition is between 800 and 1,200 MPa. All the legs of the intersection, in each direction, had pavement surface modulus values that were either below the above noted typical range, or at the low end of this range. The King Street East leg of the intersection had the lowest pavement surface modulus values, and therefore has the lowest pavement structural capacity.

Table 10: Summary of Normalized Deflection and Pavement Surface Modulus

Road	Lane	Normalized Deflection (mm)		Pavement Surface Modulus (MPa)	
		Mean	Standard Deviation	Mean	Standard Deviation
King Street East	Eastbound	0.29	0.05	526	107
	Westbound	0.27	0.07	572	121
King Road	Eastbound	0.19	0.05	826	174
	Westbound	0.18	0.03	851	155
Albion Vaughan Road	Northbound	0.19	0.05	798	184
	Southbound	0.23	0.07	705	198
Caledon Townline South	Northbound	0.23	0.03	661	98
	Southbound	0.23	0.05	670	129

Based on the mean normalized deflection and the standard deviation, the corrected spring static deflection for each of the sections was determined. The maximum allowable deflection for each tested lane was determined using The Asphalt Institute Manual Series No. 17 (MS-17) and using the anticipated future traffic loading of 10,500,000 ESALs.

Table 11 shows a summary of the corrected spring static deflection and the maximum allowable deflection for each section and also summarizes the structural condition of the pavement.

Table 11: Summary of Corrected Spring Static Deflection and Maximum Allowable Deflection

Road	Lane	Correction Spring Static Deflection (mm)	Maximum Allowable Deflection (mm)	Structural Adequacy
King Street East	Eastbound	0.95	0.51	Deficient
	Westbound	0.97	0.51	Deficient
King Road	Eastbound	0.72	0.51	Deficient
	Westbound	0.57	0.51	Deficient
Albion Vaughan Road	Northbound	0.70	0.51	Deficient
	Southbound	0.89	0.51	Deficient
Caledon Townline South	Northbound	0.68	0.51	Deficient
	Southbound	0.76	0.51	Deficient

If the corrected spring static deflection is found to be lower or equal to the maximum allowable deflection, this indicates that the existing pavement is structurally adequate for the anticipated traffic loading. Conversely, a corrected static spring deflection higher than the maximum allowable deflection is indicates that the pavement requires structural improvement. With the exception of the southbound lane of Caledon Townline South, all other lanes had corrected spring static deflection values that were greater than the maximum allowable deflection, and therefore the pavement in these lanes is structurally deficient to accommodate the anticipated future traffic loading.

5.8.2 Backcalculated Subgrade Modulus and Effective Pavement Modulus

If the layer thicknesses are known, the deflection basins that are measured during the FWD testing, can be subsequently used to backcalculate the modulus of each of the pavement layers at each test point. For the purpose of the analysis, all the asphalt layers of the pavement structure were combined. Additionally, the base and subbase layers (granular materials) were also combined for the backcalculation and a combined modulus for the granular materials was obtained. The detailed analysis results are provided in Appendix C.

Backcalculation of layer moduli is very sensitive to the input layer thickness and if the layer thickness at a particular test location are not representative of the actual thicknesses at that location the backcalculated moduli can be inaccurate. Very low modulus values for the asphalt concrete layers could indicate that the asphalt is either cracked and/or delaminated in that location, although this can be verified during visual condition inspection, and/or that the layer thickness is questionable. Similarly, very high modulus values could indicate that the asphalt thickness input for the backcalculation is questionable.

Table 12 shows a summary of the backcalculated pavement layer moduli for each of the tested lanes at the intersection. Typically the modulus of hot-mix asphalt (HMA) in relatively good condition is between 2,500 and

5,000 MPa. The asphalt modulus values for all the lanes was within this typical range, although the lower end of the range. Our quick visual inspection indicated that the HMA surface exhibits some cracking and this is likely the cause of the somewhat lower modulus values. Typical granular layer modulus values are between 200 and 700 MPa.

Table 12: Summary of FWD Backcalculation Analysis Results

Road	Lane	Asphalt Modulus (MPa)			Granular Modulus (MPa)			Subgrade Modulus (MPa)		
		Mean	Standard Deviation	30th*	Mean	Standard Deviation	30th*	Mean	Standard Deviation	30th*
King Street East	EB	5,275	2,650	3,495	280	81	237	82	7	79
	WB	8,672	3,559	5,827	250	71	228	70	6	68
King Road	EB	7,426	1,667	6,807	517	195	516	95	13	90
	WB	7,930	2,887	5,775	551	204	430	89	33	76
Albion Vaughan Road	NB	6,754	3,023	4,356	446	129	340	111	14	101
	SB	8,438	4,243	5,865	318	157	199	96	18	89
Caledon Townline South	NB	4,387	1,084	3,854	366	140	272	92	13	85
	SB	4,343	2,367	2,726	410	118	356	96	18	84

* Percentile

5.9 Pavement Design Analysis

The results from the field investigation and laboratory testing were utilized by Golder to carry out a pavement design analysis to determine a suitable pavement rehabilitation design for the intersection.

5.9.1 Traffic Analysis

CIMA provided to Golder the anticipated future traffic volumes that the pavement within the intersection would be required to accommodate. The anticipated traffic information was utilized by Golder to calculate the design traffic loading for a 20 year period. Traffic data was provided for each leg of the intersection. The direction with the largest amount of traffic was used in this analysis (Westbound King Road). Table 13 outlines the traffic parameters that were used to calculate the design Equivalent Single Axle Loads (ESALs) to be accommodated within the intersection.

Table 13: Traffic Parameters Westbound King Road

Parameter	Value
2018 Average Annual Daily Traffic (AADT) (1 Way)	8,474
2031 AADT	11,544
Traffic Growth Rate	2.4 %
Truck Percentage	7.4%
Truck Factor (Assumed by Golder)	1.8
Lane Factor	1
Design Life	20 years

Based on the above parameters the design ESALs was calculated to be 10,500,000 over a 20 year design life for the intersection of Albion Vaughan Road and King Road.

It is understood that the intersection alignment maybe adjusted to two thru lanes in each direction in the future. If this occurs, then a lane distribution of 0.9 would be applicable. The design ESALs for a 20 year design life would be 9,400,000.

5.9.2 Pavement Structural Design

The pavement design analysis was carried out using the methodology outlined in the American Association of State Highway and Transportation Officials (AASHTO) 1993 “Guide for Design of Pavement Structures”. The parameters that were used for the design analysis are provided in Table 14.

Table 14: Parameters for Structural Design Analysis

Parameters		Value
Design ESALs (1 thru lane)		10,500,000
Design ESALs (2 thru lanes)		9,400,000
Reliability		90%
Standard Deviation		0.45
Subgrade Resilient Modulus		30 MPa
Initial Serviceability		4.5
Terminal Serviceability		2.5
Structural Coefficient	New Hot Mix Asphalt (HMA)	0.44
	Stabilized HMA and Existing Granular Blend with Expanded Foamed Asphalt	0.25
	Pulverized HMA and Existing Granular Blend	0.14
	Existing Granular Material	0.08
Drainage Coefficient	New Hot Mix Asphalt (HMA)	1.0
	Stabilized HMA and Existing Granular Blend with Expanded Foamed Asphalt	1.0
	Pulverized HMA and Existing Granular Blend	0.9
	Existing Granular Material	0.9

Based on the above parameter values, the required structural number for the rehabilitated pavement to accommodate future traffic loading was calculated to be 144 mm when one thru lane exists in each direction. The proposed rehabilitation design alternative considered the future traffic loading, existing drainage, existing pavement structure and constructability.

Golder developed the following rehabilitation pavement structural design, which would include a grade raise of 90 mm:

- Pulverize existing HMA and blend one to one with underlying granular (average of 140 mm of each);
- Remove 50 mm of blended material;
- Stabilize 150 mm of blended HMA and existing granular material with expanded asphalt;

- Place 50 mm lower binder course asphalt;
- Place 50 mm upper binder course asphalt; and
- Place 40 mm surface course asphalt.

Based on the parameter values in Table 14, the required structural number for the rehabilitated pavement to accommodate future traffic loading was calculated to be 142 mm when two thru lanes exists in each direction. The proposed rehabilitation design alternative considered the future traffic loading, existing drainage, existing pavement structure and constructability. The following pavement rehabilitation and widening structural design would involve a grade raise of 120 mm.

- Pulverize existing HMA and blend one to one with underlying granular (average of 140 mm of each);
- Stabilize 150 mm of blended HMA and existing granular material with expanded asphalt;
- Place 70 mm binder course asphalt; and
- Place 50 mm surface course asphalt.

When carrying out full depth reclamation with expanded asphalt stabilization, the stabilized layer does not need to remain closed to traffic as it cures. It can be opened to traffic shortly after the compaction of the stabilized layer is completed. The only requirement for curing of the stabilized layer is that prior to placement of the HMA layers, the stabilized layer needs to cure for 2 days; however, during this curing period the stabilized layer can be opened to traffic, and any surficial damage to this layer can be corrected during the placement of the binder course HMA layer.

Golder also considered the use of Hot In-Place Recycling (HIR) as a potential alternative for the pavement rehabilitation; however, at this time there is only a single contractor is Ontario that can carry out this treatment. This particular Contractor only has one highway train in Ontario, and no smaller recycling train that is in the province. Implementation of this strategy would be very expensive and likely significantly more costly than full depth reclamation. Additionally, with HIR the potential for reflective cracking is still present due to the fact that only the top 50 mm is recycled. With the extent of cracking observe on the subject pavement, and the amount of structural improvement that is required, full depth reclamation is considered to be the more suitable option from a technical perspective.

5.10 Recommendations for Detailed Design

During detailed design stage once the proposed widening limits are confirmed it is recommended that boreholes or surficial test holes be advanced to obtain information on the surficial materials, i.e., potential organics that may be encountered adjacent to the existing footings for widening of both structures. In addition, artesian groundwater conditions were encountered in the clayey silt to silty clay deposit. It is most likely that there is an underlying granular deposit is present below the cohesive deposit; during detailed design it is recommended that the boreholes be advanced to determine the thickness of the cohesive deposit and it is recommended that a monitoring well be installed in the lower granular deposit (if present) to measure and confirm the artesian groundwater level.

6.0 CLOSURE

This Preliminary geotechnical report was prepared by Ms. Sandra McGaghran, M.Eng., P.Eng., a senior geotechnical engineer and an Associate with Golder. The pavement inspection and recommendations in this report was prepared by Ms. Vimy Henderson, P.Eng., a pavement engineer with Golder. The pavement recommendations were reviewed by Ludomir Uzarowski, P.Eng. a Principal with Golder.

GOLDER ASSOCIATES LTD.

S.M. McGAGHRAN

Sept 6/19

L. UZAROWSKI

Sep 6/19

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Senior Geotechnical Engineer, Associate	Principal - Pavement and Materials Engineering

NK/VH/SMM/LU/rb

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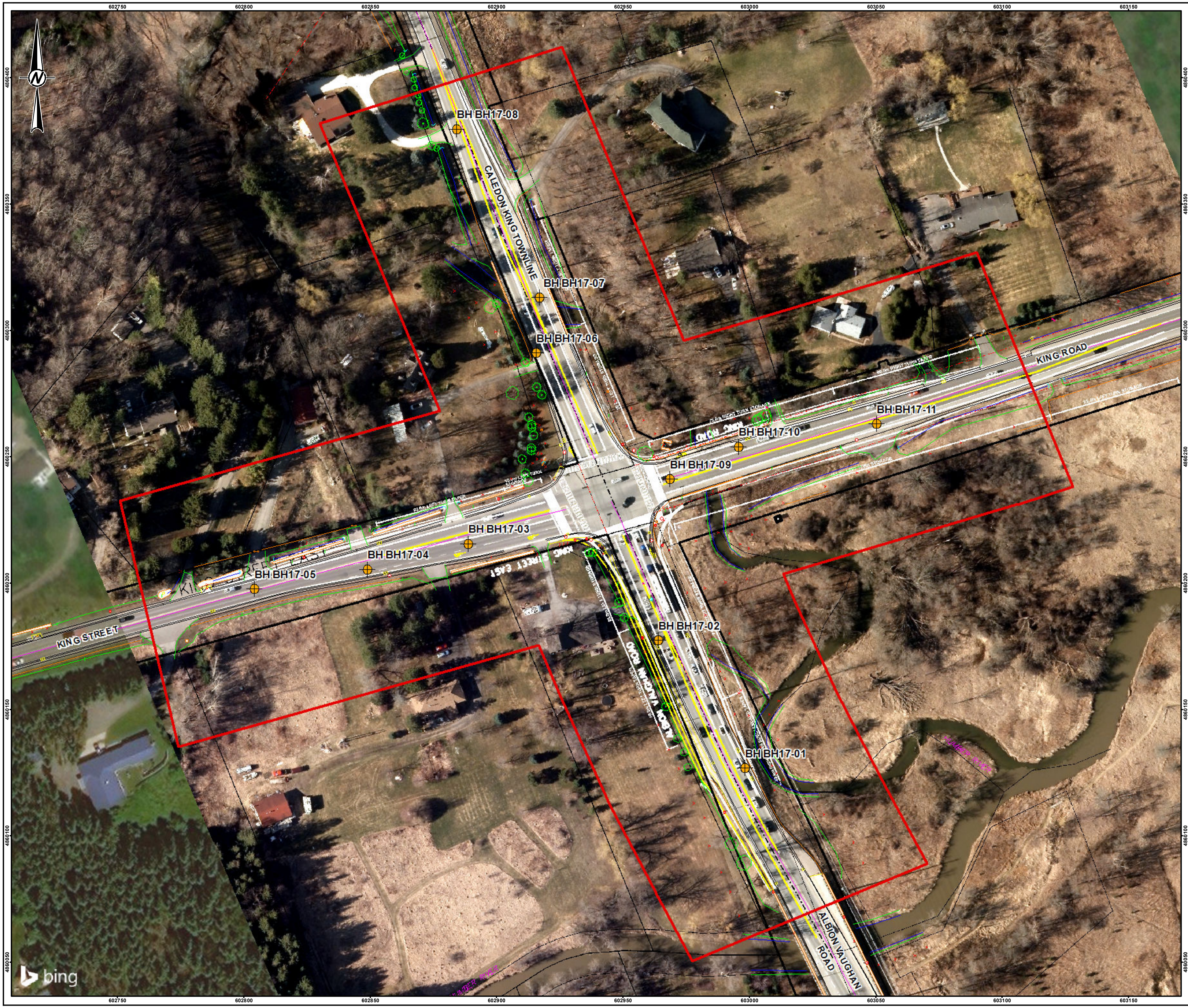
REFERENCES

Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual (CFEM), 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6-06. 2014. CSA Special Publication, S6.1 06. Canadian Standard Association

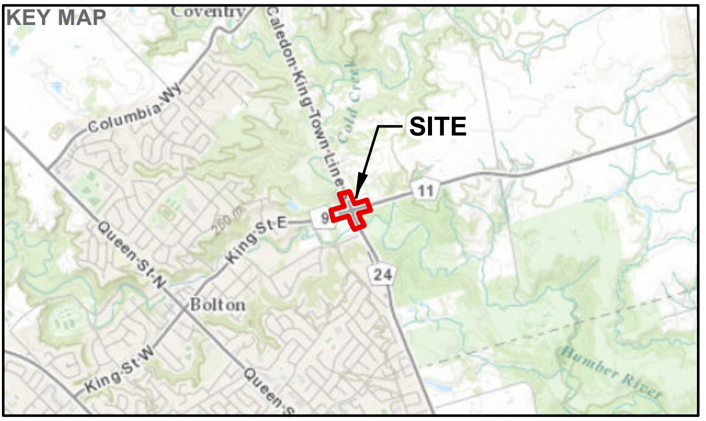
US Navy. 1986. NAVFAC Design Manual DM 7.02: Soil Mechanics, Foundation and Earth Structures. U.S. Navy. Alexandria, Virginia.



LEGEND

⊕ BOREHOLE LOCATION

KEY MAP



0 25 50

1:1,500 Metres

REFERENCE(S)

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IMAGERY - PROVIDED BY CIMA; BING SOURCES: ESRI, HERE, DELORME, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), SWISSTOPO, MAPMYINDIA, © OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
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CLIENT
REGION OF PEEL

PROJECT
ENVIRONMENTAL ASSESSMENT ALBION-VAUGHAN ROAD AND KING STREET INTERSECTION

TITLE
BOREHOLE LOCATION PLAN - 4 LANE WIDENING

CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-01
DESIGNED	JR
PREPARED	JB
REVIEWED	AJ/VH
APPROVED	SMM

PROJECT NO.	CONTROL	REV.	FIGURE
1664714			1

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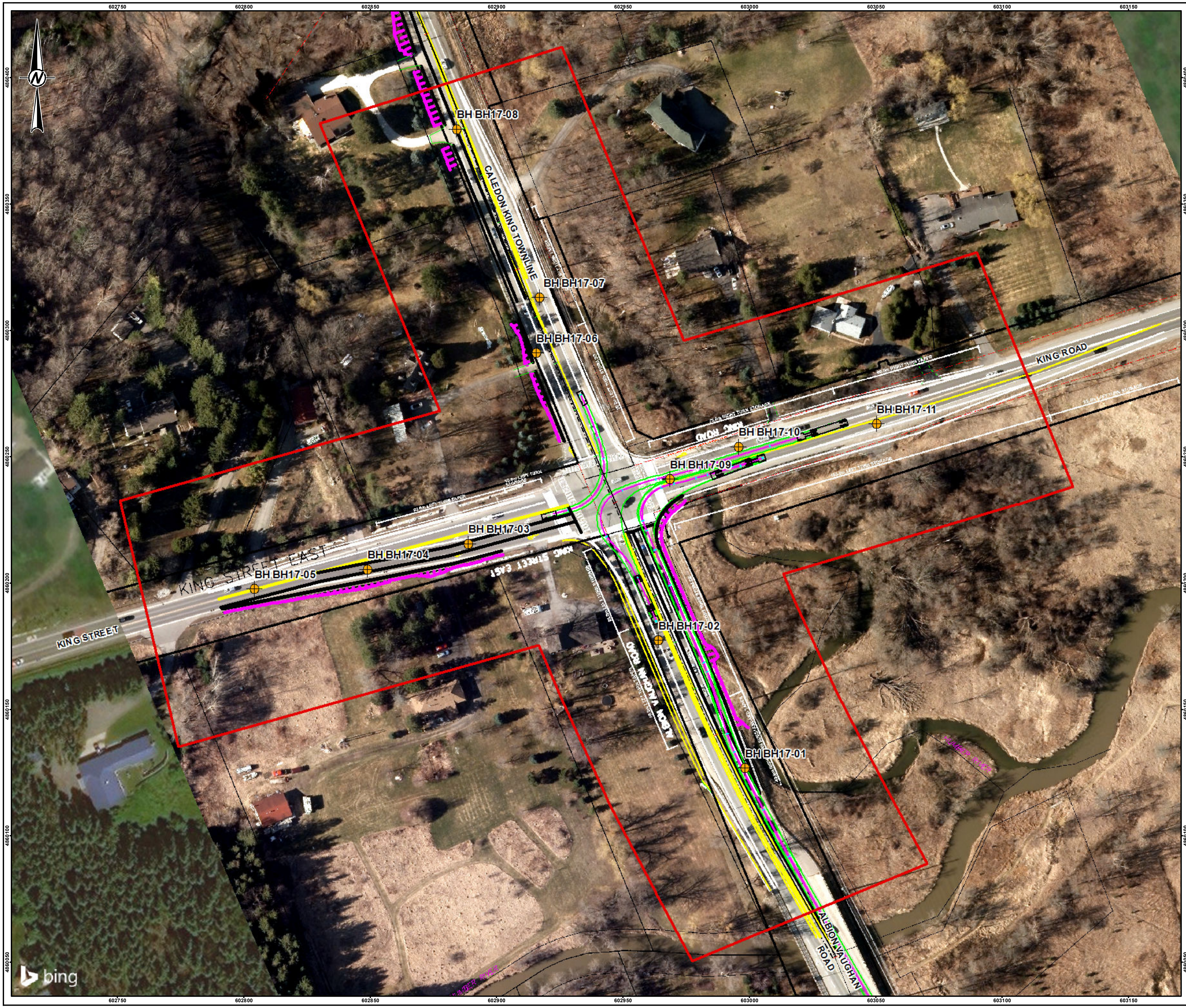
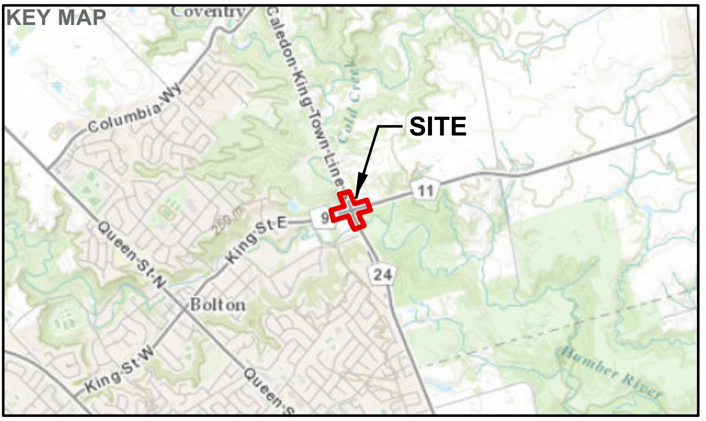


LEGEND

⊕ BOREHOLE LOCATION



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REFERENCE(S)

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BASE 4-LANING MICROSTATION DRAWING PROVIDED BY CIMA ON OCTOBER 2, 2017
IMAGERY - PROVIDED BY CIMA; BING SOURCES: ESRI, HERE, DELORME, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), SWISSTOPO, MAPMYINDIA, © OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY
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CLIENT
REGION OF PEEL

PROJECT
ENVIRONMENTAL ASSESSMENT ALBION-VAUGHAN ROAD AND KING STREET INTERSECTION

TITLE
BOREHOLE LOCATION PLAN - ULTIMATE FOUR LANE WIDENING

CONSULTANT

Golder Associates

YYYY-MM-DD	2017-11-01
DESIGNED	JR
PREPARED	JB
REVIEWED	AJ/VH
APPROVED	SMM

PROJECT NO.	CONTROL	REV.	FIGURE
1664714			2

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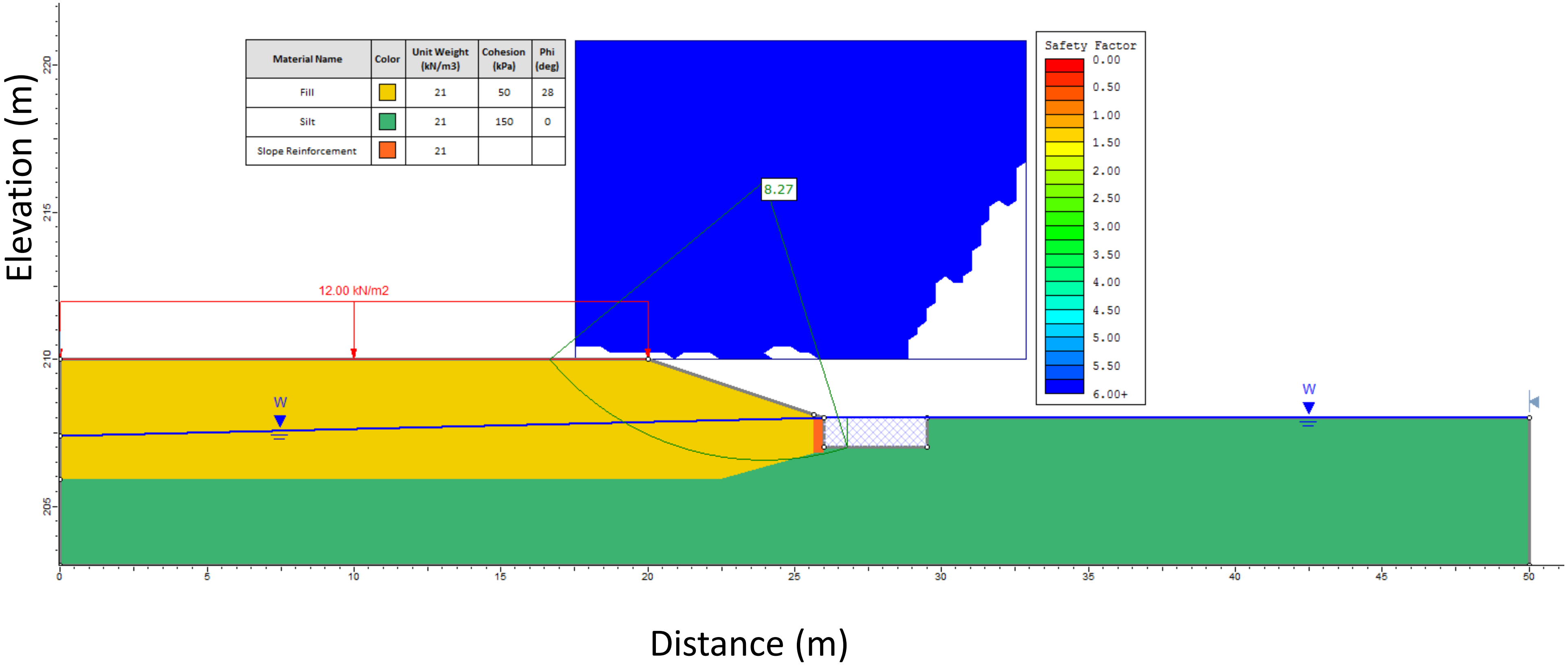


Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill	🟨	21	50	28
Silt	🟩	21	150	0
Slope Reinforcement	🟧	21		

CLIENT
CIMA+

PROJECT
SCHEDULE ‘B’ MUNICIPAL CLASS ENVIRONMENTAL ASSESSMENT, ALBION VAUGHAN ROAD AND KING STREET, TOWN OF CALEDON, REGIONAL MUSICALITY OF PEEL

CONSULTANT

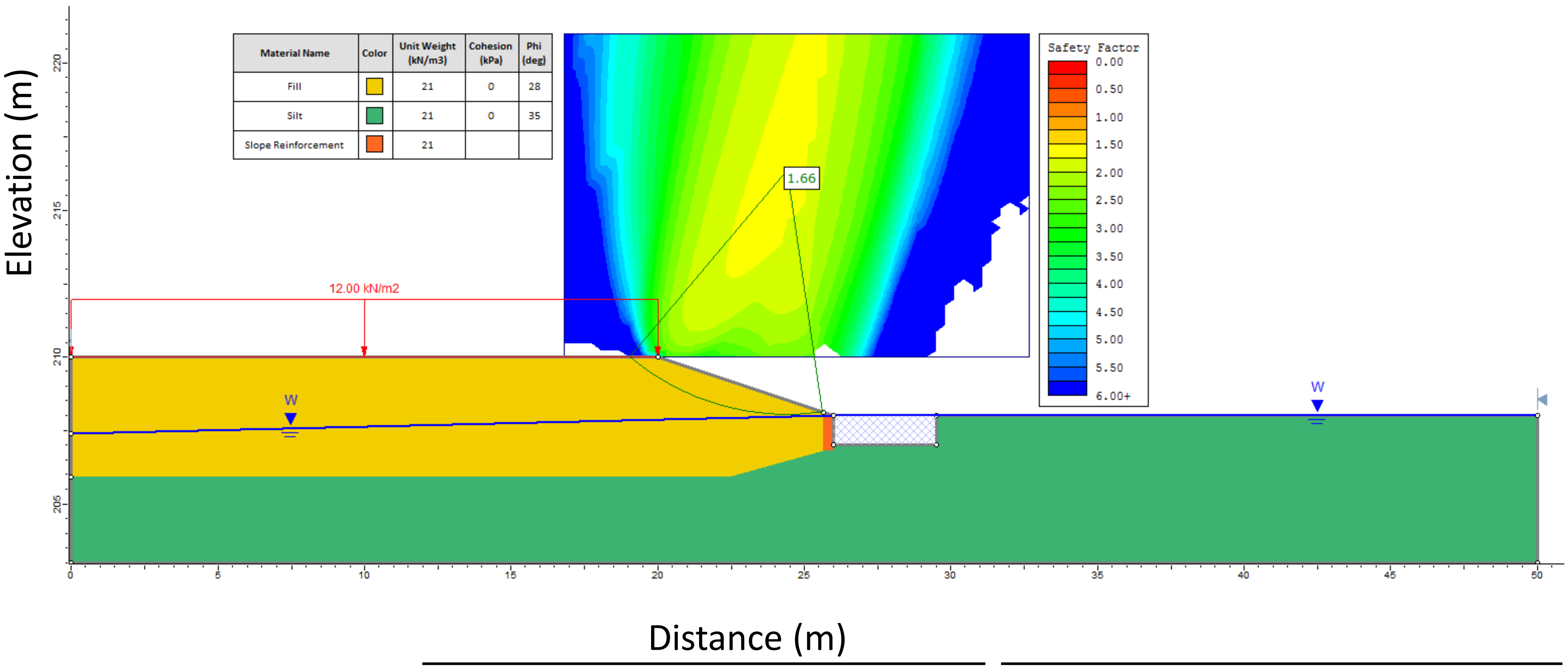
Golder Associates

YYYY-MM-DD	2017-11-03
PREPARED	NK
DESIGN	NK
REVIEW	SMM
APPROVED	SMM

TITLE
STATIC GLOBAL STABILITY - ALBION VAUGHAN AT COLD CREEK MEANDER
SHORT-TERM (UNDRAINED) ANALYSIS

1664714

3



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Fill	🟨	21	0	28
Silt	🟩	21	0	35
Slope Reinforcement	🟧	21		

CLIENT
CIMA+

PROJECT
SCHEDULE ‘B’ MUNICIPAL CLASS ENVIRONMENTAL ASSESSMENT, ALBION VAUGHAN ROAD AND KING STREET, TOWN OF CALEDON, REGIONAL MUSICALITY OF PEEL

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-11-03
PREPARED	NK
DESIGN	NK
REVIEW	SMM
APPROVED	SMM

TITLE
STATIC GLOBAL STABILITY - ALBION VAUGHAN AT COLD CREEK MEANDER LONG-TERM (DRAINED) ANALYSIS

PROJECT No.
1664714

4

APPENDIX A

Important Information and Limitations

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

APPENDIX B

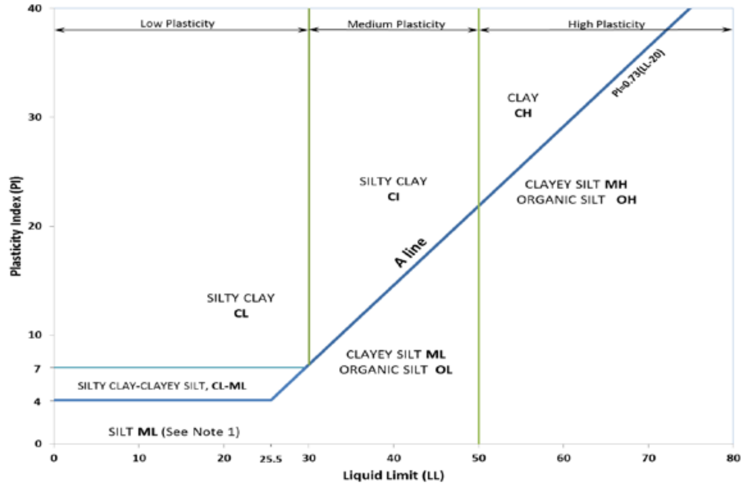
Record of Borehole Sheets and Laboratory Test Results

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.

Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.

For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.

For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.

A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL, SAND and CLAY)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.).

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
GS	Grab Sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 - 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
2. Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-01

SHEET 1 OF 1

LOCATION: N 4860127.01; E 602998.35

BORING DATE: May 24, 2017

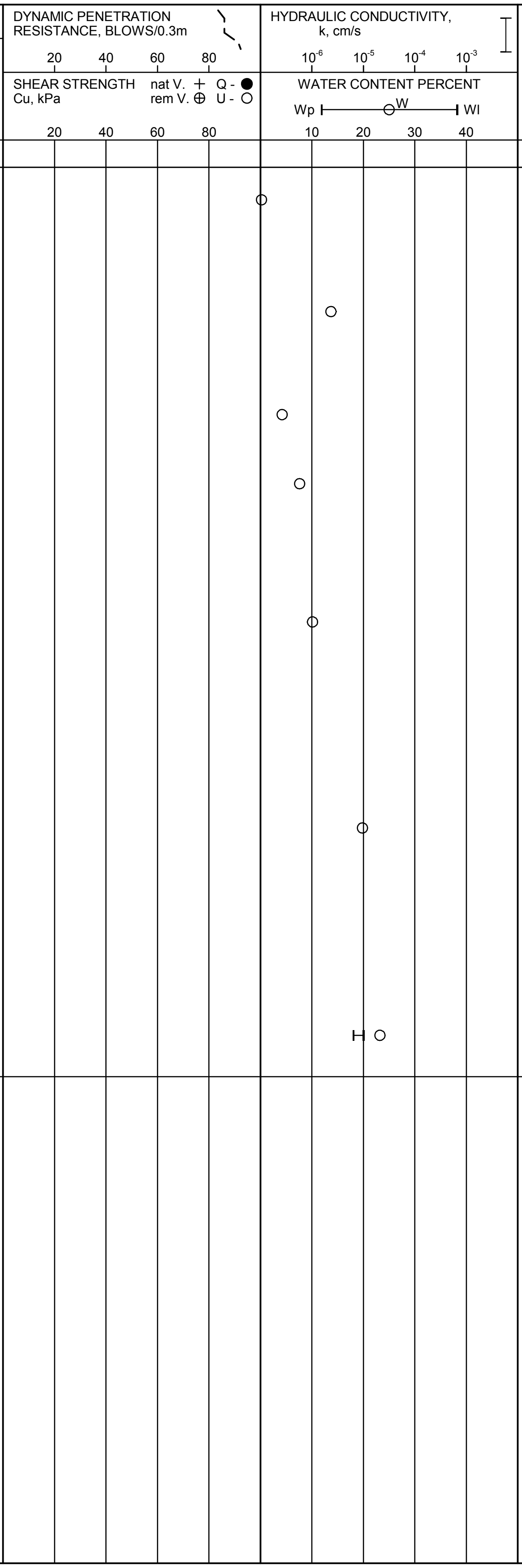
DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

BORING METHOD: Power Auger, 102 mm O.D. Solid Stem Augers

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	210.03						
ASPHALT	0.00						
FILL - (SW/GW) SAND and GRAVEL; brown, with asphalt fragments; non-cohesive, moist, very dense	209.85 0.18	1	SS	50/0			
FILL - (ML) CLAYEY SILT, some sand, trace gravel, trace organics; brown and grey mottled; cohesive, w<PL, stiff to very stiff	209.27 0.76	2	SS	15			Bentonite Seal
FILL - (SW/GW) SAND and GRAVEL, trace fines; brown; non-cohesive, moist to wet, very dense	208.51 1.52	3 4A	SS	72			
FILL - (ML/SW) SILT and SAND, trace fines; grey; non-cohesive, wet, loose	207.59 2.44	4B	SS	4	MH	0 44 51 5	∇
FILL - (SW/GW) SAND and GRAVEL; grey; non-cohesive, wet, compact	207.06 2.97	5	SS	11			Screen and Sand Filter
(ML) SILT with slight plasticity, some clay; grey; non - cohesive, moist, dense	205.92 4.11	6 7	SS SS	36 35	 MH	 0 0 91 9	Bentonite Seal
END OF BOREHOLE	203.32 6.71						



Notes:

1. Groundwater encountered at a depth of 2.7 m (Elev. 207.3 m) below ground surface upon completion of drilling.

2. Groundwater level measurements in piezometer:

Date	Depth (m)	Elev. (m)
05/30/17	2.5	207.5
07/17/17	2.6	207.4

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AJ
CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-02

SHEET 1 OF 1

LOCATION: N 4860177.53; E 602963.99

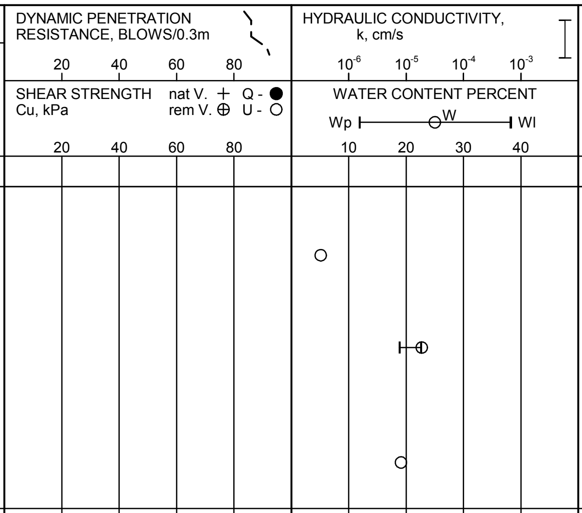
BORING DATE: May 23, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		GROUND SURFACE		209.05					
0	Power Auger 102 mm O.D. Solid Stem Augers	ASPHALT	■	0.00					
		FILL - (SW/GW) SAND and GRAVEL; brown; non-cohesive, moist, dense	▨	0.13	1	SS	30		
1		FILL - (SM) SILTY SAND of slight plasticity, trace plastic fines, trace gravel, some organics, sand pockets; dark grey; non-cohesive, moist, loose	▨	208.29 0.76	2	SS	6	MH	2 57 36 5
		FILL - (SW) gravelly SAND; grey; non-cohesive, wet, loose	▨	207.53 1.52	3	SS	9		
2		END OF BOREHOLE Note: 1. Borehole dry upon completion of drilling.		206.92 2.13					



DEPTH SCALE

1 : 50

GOLDER

LOGGED: AJ

CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-03

SHEET 1 OF 1

LOCATION: N 4860215.85; E 602888.77

BORING DATE: May 23, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		GROUND SURFACE	210.23					
0	Power Auger 102 mm O.D. Solid Stem Augers	ASPHALT	0.00					
		FILL - (SW) gravelly SAND, some fines, trace clay; brown; non-cohesive, moist, very dense	0.13	1	SS	54	MH	30 57 11 2
			209.32	2A				
1		(ML) Sandy SILT, some plastic fines, trace gravel; dark grey; non-cohesive, moist, dense to compact	0.91	2B	SS	32		
				3	SS	11	MH	0 21 71 8
2			208.10					
		END OF BOREHOLE	2.13					
		Note:						
		1. Borehole dry upon completion of drilling.						
3								
4								
5								
6								
7								
8								
9								
10								

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AJ

CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-04

SHEET 1 OF 1

LOCATION: N 4860205.54; E 602849.09

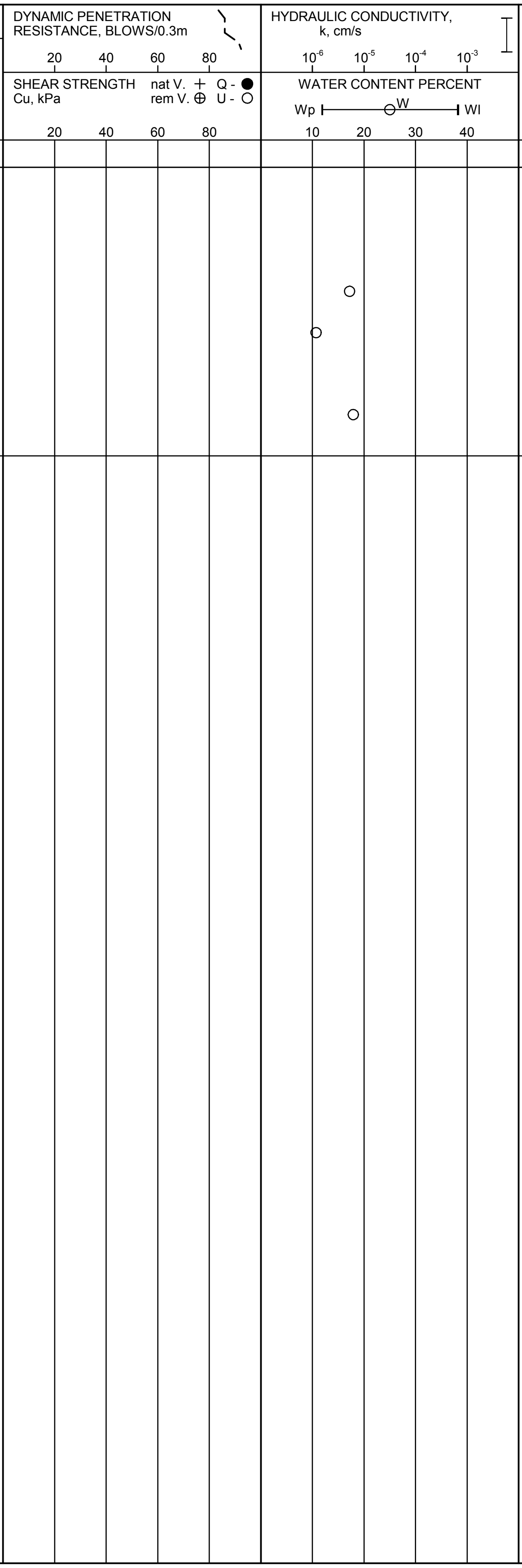
BORING DATE: May 23, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		GROUND SURFACE		212.04					
0	Power Auger 102 mm O.D. Solid Stem Augers	ASPHALT		0.00					
		FILL - (SW/GW) SAND and GRAVEL; brown; non-cohesive, moist, compact		0.13	1A	SS	25		
		FILL - (ML) Sandy CLAYEY SILT, trace gravel, trace rootlets; dark brown; w<PL, hard		211.41 0.63	1B				
1					2A	SS	32	MH	2 22 59 17
		(SW) SAND, some fines; brown; non-cohesive, moist to wet, dense to compact		210.97 1.07	2B				
					3	SS	12		
2		END OF BOREHOLE Note: 1. Borehole dry upon completion of drilling		209.91 2.13					
3									
4									
5									
6									
7									
8									
9									
10									



DEPTH SCALE

1 : 50

LOGGED: AJ

CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-05

SHEET 1 OF 1

LOCATION: N 4860197.85; E 602804.16

BORING DATE: May 23, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION; GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		GROUND SURFACE		214.83							
0	Power Auger 102 mm O.D. Solid Stem Augers	ASPHALT	■	0.00							
		FILL - (SW) gravelly SAND; brown; non-cohesive, moist, dense	▨	0.13	1	SS	32		○		
1		FILL - (CL) SILTY CLAY, some sand, trace gravel; mottled brown and grey; cohesive, w~PL, firm to stiff	▨	214.07 0.76	2	SS	8		⊢○⊣	MH	1 22 60 17
2		- Rootlets and wood fragments encountered at a depth of 1.5 m	▨		3	SS	10				
		END OF BOREHOLE		212.70 2.13							
		Note: 1. Borehole dry upon completion of drilling									
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AJ

CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-06

SHEET 1 OF 1

LOCATION: N 4860291.35; E 602915.83

BORING DATE: May 23, 26 and 29, 2017

DATUM: Geodetic

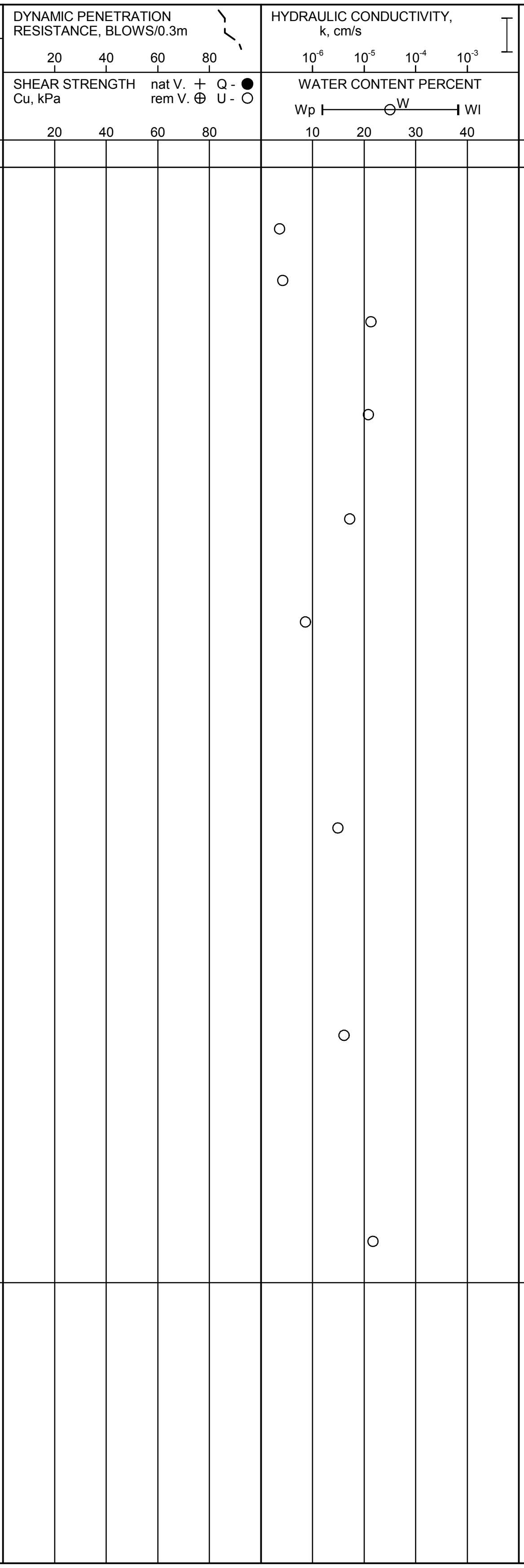
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

BORING METHOD: Power Auger, 102 mm O.D. Solid Stem Augers

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	GROUND SURFACE	209.86					
0	ASPHALT	0.00					
	FILL - (SW/GW) SAND and GRAVEL, some fines; brown; non-cohesive, moist, very dense	0.15 208.95	1	SS	65		
			2A				
1	FILL - (ML) sandy CLAYEY SILT, plastic fines; grey; mottled; cohesive, w<PL, stiff	0.91 208.34	2B	SS	10		
2	(SM) SILTY SAND, some plastic fines; brown; mottled; non-cohesive, wet, loose	1.52 207.57	3	SS	6		
	(SW-GW) Sandy GRAVEL, some plastic fines, trace clay; brown to grey; non-cohesive, wet, loose to very dense	2.29	4	SS	5		
3		206.05	5	SS	77	MH	54 33 11 2
4	(ML) CLAYEY SILT, some to trace sand; grey; cohesive, w<PL to w>PL, hard	3.81					
5			6	SS	42		
6			7	SS	50		
8		201.63	8	SS	69		
	END OF BOREHOLE	8.23					

PIEZOMETER OR STANDPIPE INSTALLATION: ∇



Note:

1. Groundwater encountered at a depth of 2.5 m (Elev. 207.4 m) below ground surface upon completion of drilling.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AJ

CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-07

SHEET 1 OF 1

LOCATION: N 4860313.30; E 602916.97

BORING DATE: May 26, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

BORING METHOD: Power Auger, 102 mm O.D. Solid Stem Augers

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	210.00
ASPHALT	0.00
FILL - (SW/GW) SAND and GRAVEL; brown; non-cohesive, moist, compact	0.13
FILL - (ML) CLAYEY SILT, some gravel, trace organics (rootlets and wood fragments); dark grey; cohesive, w<PL to w>PL, stiff to firm	209.24 0.76
(ML) SILT of slight plasticity, some clay, trace gravel; grey; non-cohesive, wet, compact	206.95 3.05
END OF BOREHOLE	201.77 8.23

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	29		
2	SS	9		
3	SS	4		
4	SS	4		
5	SS	13		
6	SS	16		
7	SS	21	MH	1 0 87 12
8	SS	13		

PIEZOMETER OR STANDPIPE INSTALLATION:
- Sand
- Bentonite Seal
- Screen and Sand Filter

Notes:

1. Groundwater encountered at a depth of 3.1 m (Elev. 206.9 m) below ground surface upon completion of drilling.

2. Water level in standpipe piezometer measured 0.9 m above ground surface (Elev. 210.9 m) on July 17, 2017.

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AJ
CHECKED: SMM

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PROJECT: 1664714

RECORD OF BOREHOLE: BH17-08

SHEET 1 OF 1

LOCATION: N 4860379.79; E 602884.28

BORING DATE: May 23, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○: 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl: 10 20 30 40

BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	GROUND SURFACE	210.02					
Power Auger 102 mm O.D. Solid Stem Augers	ASPHALT	0.00					
	FILL - (SW/GW) SAND and GRAVEL, some fines; brown; non-cohesive, moist, dense	0.15	1A	SS	39	MH	48 41 9 2
	FILL - (SM) SILTY SAND; brown; non-cohesive, moist, dense	209.31 0.76	1B				
	FILL - (ML) sandy CLAYEY SILT, trace gravel; mottled brown; cohesive, w~PL, stiff		2	SS	12		
	(SW) SAND, some fines, trace organics; mottled brown; non-cohesive, moist to wet, compact	208.50 1.52	3	SS	12		
	END OF BOREHOLE	207.89 2.13					

Note:

1. Borehole dry upon completion of drilling.

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

PIEZOMETER OR STANDPIPE INSTALLATION

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DEPTH SCALE

1 : 50

GOLDER

LOGGED: AJ

CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-09

SHEET 1 OF 2

LOCATION: N 4860241.29; E 602968.72

BORING DATE: May 26, 2017

DATUM: Geodetic

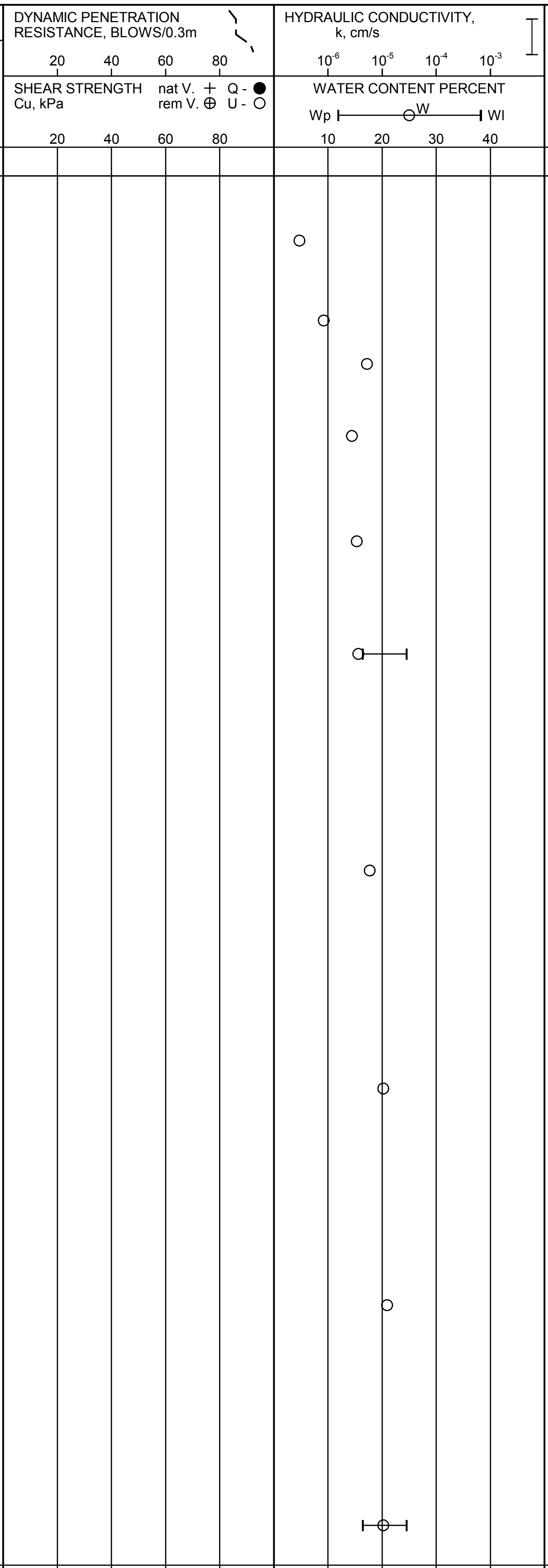
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

BORING METHOD: Power Auger, 102 mm O.D. Solid Stem Augers

SOIL PROFILE DESCRIPTION	ELEV. (m)	DEPTH (m)
GROUND SURFACE	210.03	
ASPHALT		0.00
FILL - (SW) gravelly SAND, some fines, trace rootlets; brown; non-cohesive, moist, very dense to compact		0.13
FILL - (ML) CLAYEY SILT, some sand, trace organics (rootlets and wood fragments); grey; cohesive, w<PL, hard	208.76	1.27
(CL) SILTY CLAY, trace sand, trace gravel; brown to grey; cohesive, w<PL to w> PL at a depth of 7.6 m, hard	207.80	2.23
END OF BOREHOLE	200.28	9.75

SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	53		
2A	SS	16		
2B				
3	SS	32		
4	SS	37		
5	SS	32	MH	0 2 75 23
6	SS	44		
7	SS	21		
8	SS	23		
9	SS	20		



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DEPTH SCALE
1 : 50

LOGGED: AJ
CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-09

SHEET 2 OF 2

LOCATION: N 4860241.29; E 602968.72

BORING DATE: May 26, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION GRAIN SIZE DISTRIBUTION (%)
		--- CONTINUED FROM PREVIOUS PAGE ---									GR SA SI CL
10		Note: 1. Groundwater encountered at a depth of 6.9 m (Elev. 203.1 m) below ground surface upon completion of drilling.									
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AJ

CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-10

SHEET 1 OF 2

LOCATION: N 4860254.14; E 602995.86

BORING DATE: May 24, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

BORING METHOD: Power Auger, 102 mm O.D. Solid Stem Augers

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	PIEZOMETER OR STANDPIPE INSTALLATION
GROUND SURFACE	211.11							
ASPHALT	0.00							
FILL - (SW/GW) SAND and GRAVEL; brown; non-cohesive, moist, dense	210.93 0.18	1	SS	37				Bentonite Seal
FILL - (ML) CLAYEY SILT, some sand, trace gravel, trace organics; grey; cohesive, w<PL, stiff	210.32 0.79	2	SS	12				
(CL) SILTY CLAY, trace sand at 3.1 m; mottled brown and grey; cohesive, w<PL to w~PL, firm to hard	209.59 1.52	3	SS	6				
		4	SS	22				
		5	SS	35				
		6	SS	29				
		7	SS	33				
		8	SS	23	Wp ⊢—○ W	MH	0 0 67 33	
		9	SS	17				Screen and Sand Filter
END OF BOREHOLE	201.36 9.75							

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DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 10 20 30 40

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE
1 : 50

GOLDER

LOGGED: AJ
CHECKED: SMM

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PROJECT: 1664714

RECORD OF BOREHOLE: BH17-10

SHEET 2 OF 2

LOCATION: N 4860254.14; E 602995.86

BORING DATE: May 24, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION GRAIN SIZE DISTRIBUTION (%)
		--- CONTINUED FROM PREVIOUS PAGE ---									GR SA SI CL
10		Notes: 1. Groundwater encountered at a depth of 7.8 m (Elev. 203.3 m) below ground surface upon completion of drilling. 2. Water level in stand pipe piezometer measured at a depth of 0.1 m above ground surface (Elev. 211.2 m) on July 17, 2017.									
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: AJ

CHECKED: SMM

PROJECT: 1664714

RECORD OF BOREHOLE: BH17-11

SHEET 1 OF 1

LOCATION: N 4860263.16; E 603050.30

BORING DATE: May 23, 2017

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 55 Truck Mounted Drill Rig

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 10 20 30 40	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION; GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		GROUND SURFACE		214.19							
0	Power Auger 102 mm O.D. Solid Stem Augers	ASPHALT		0.00							
		FILL - (SW/GW) SAND and GRAVEL; brown; non-cohesive, moist, dense		0.13	1	SS	46				
1		(CI) SILTY CLAY, sand pockets; brown; cohesive, w~PL to w<PL, stiff		213.43 0.76	2	SS	9				
2					3A	SS	13				
		(SW) SAND,some fines; brown; non-cohesive, wet, compact		212.11 2.13	3B						
		END OF BOREHOLE Note: 1. Borehole dry upon completion of drilling.									
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

GOLDER

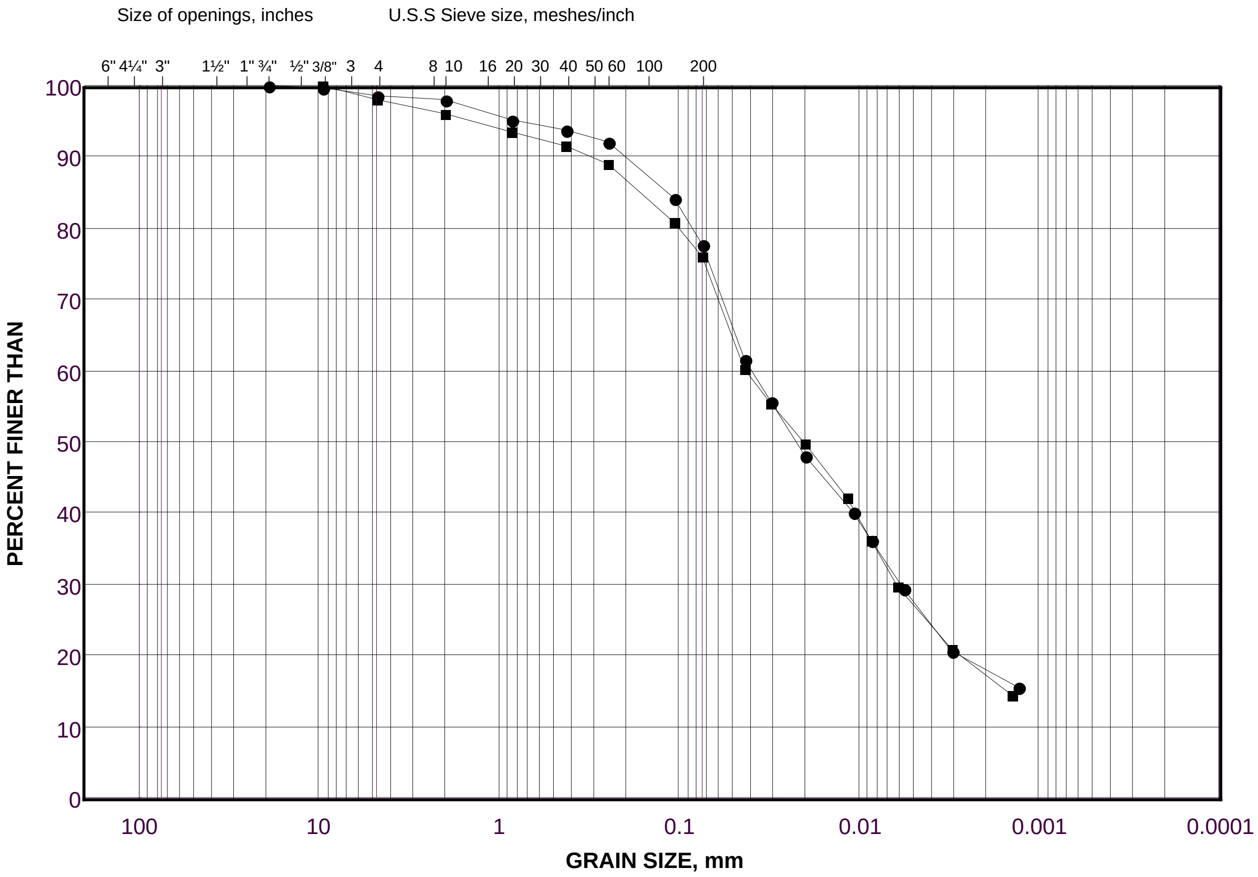
LOGGED: AJ

CHECKED: SMM

GRAIN SIZE DISTRIBUTION

Fill - (ML) Sandy Clayey Silt to (CL) Silty Clay

FIGURE B1



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	**GRAVEL SIZE**		**SAND SIZE**			**FINE GRAINED**

LEGEND

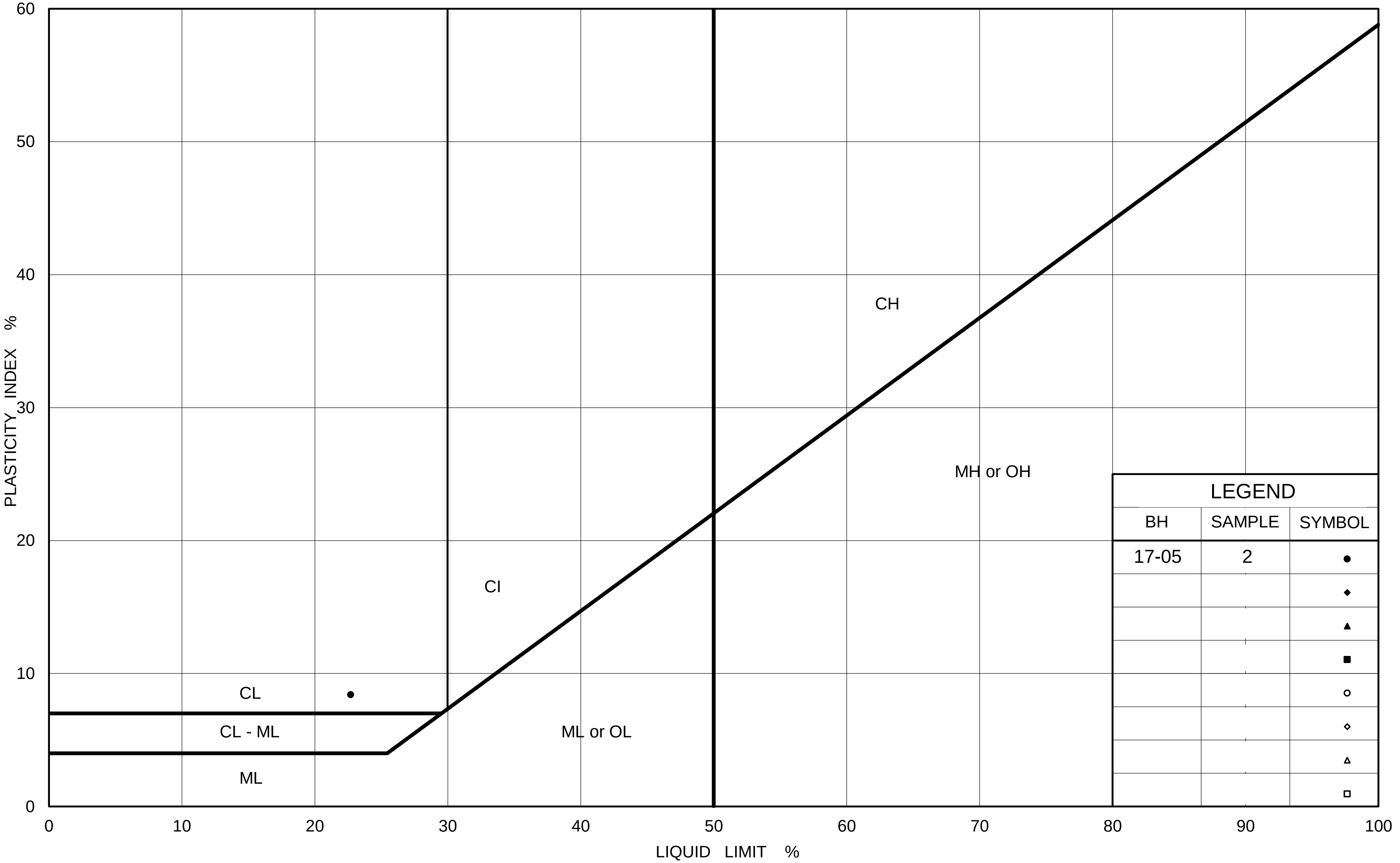
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-05	2	213.7
■	17-04	2A	210.9

Project Number: 1664714

Checked By: ____________________

Golder Associates

Date: 16-Oct-17



PLASTICITY CHART
Fill - (CL) Silty Clay

Figure No. B2

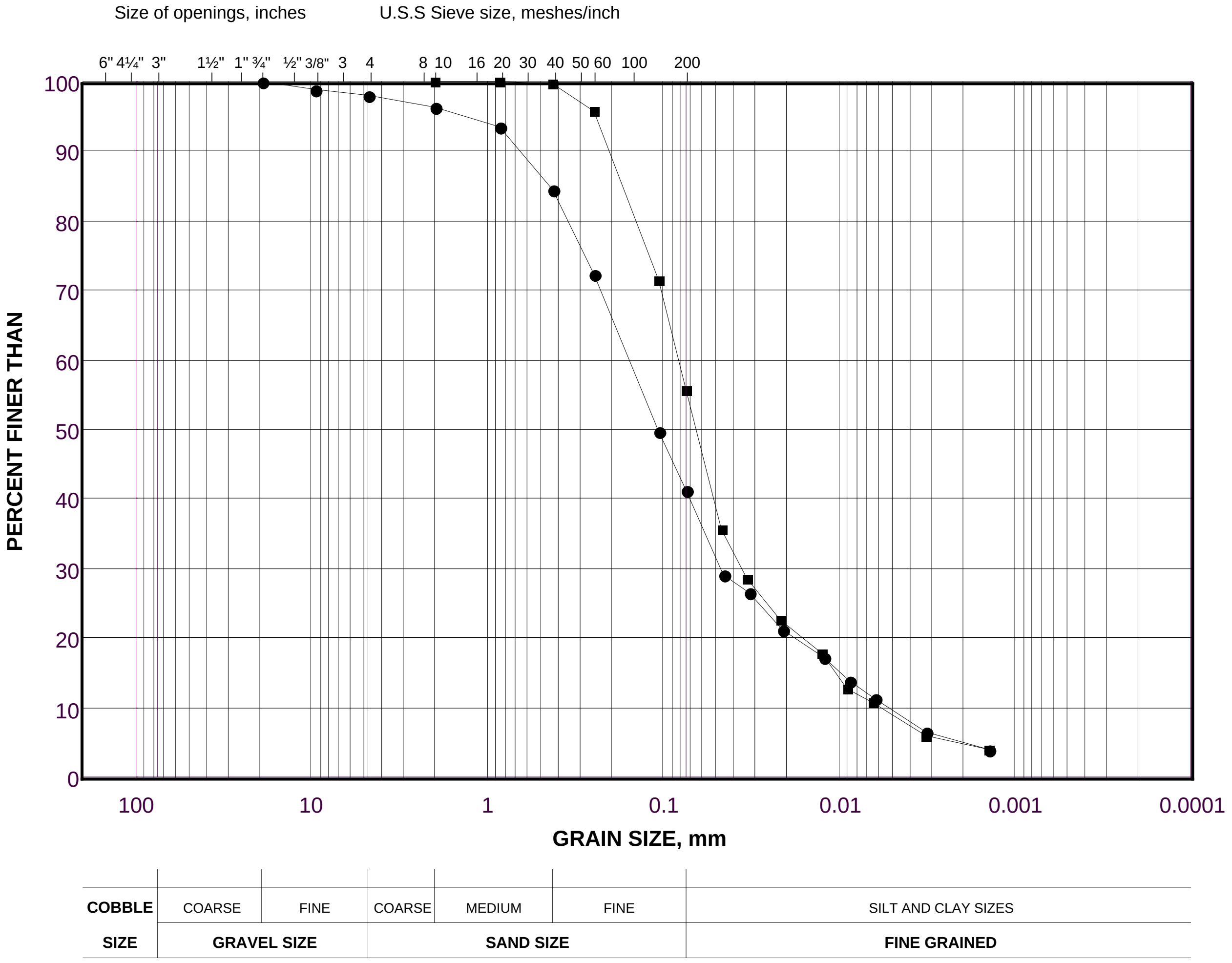
Project No. 1664714

Checked By:

GRAIN SIZE DISTRIBUTION

(ML/SW) Silt and Sand

FIGURE B3



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-02	2	207.9
■	17-01	4B	207.4

Project Number: 1664714

Checked By: ______________________

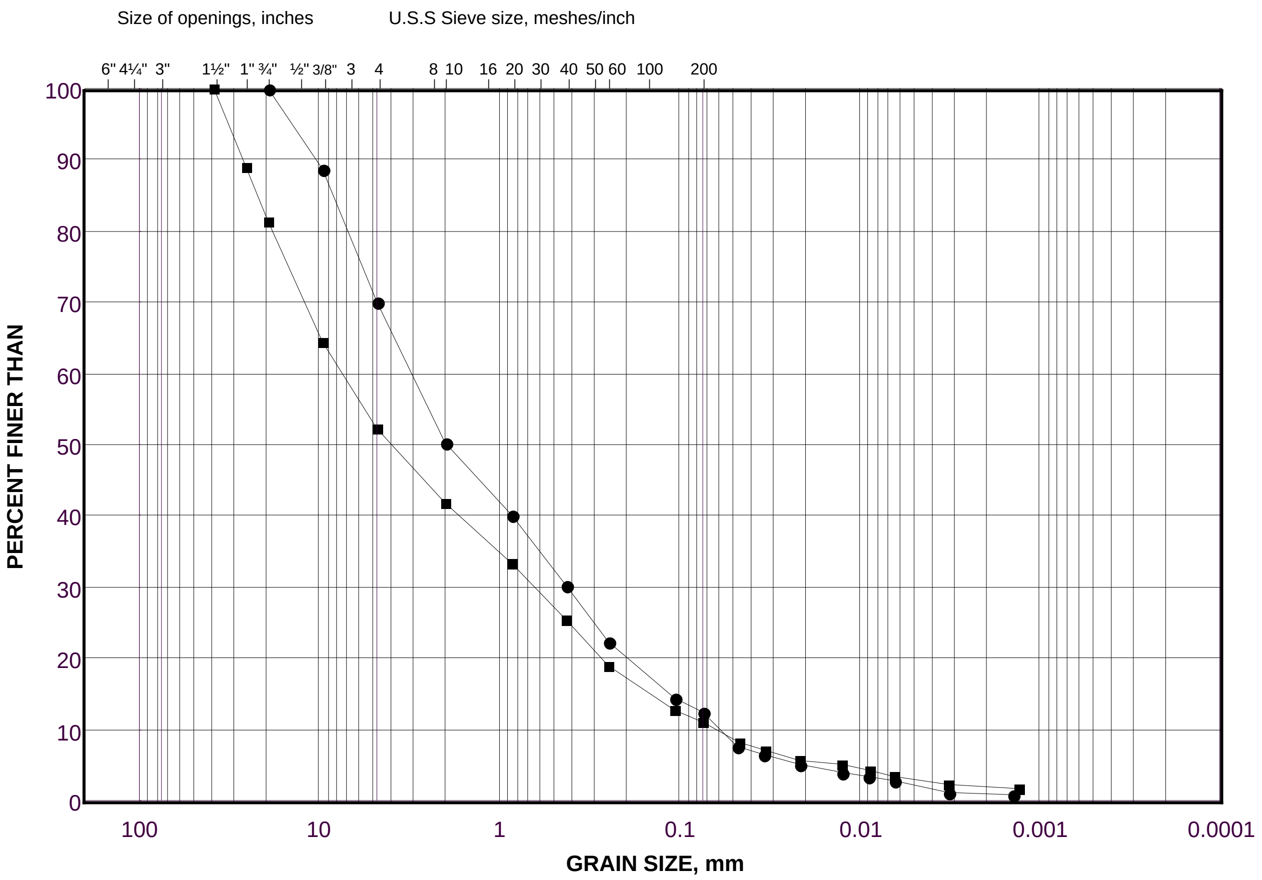
Golder Associates

Date: 16-Oct-17

GRAIN SIZE DISTRIBUTION

Fill - (SW/GW) Sand and Gravel to (SW) Gravelly Sand

FIGURE B4



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	**GRAVEL SIZE**		**SAND SIZE**			**FINE GRAINED**

LEGEND

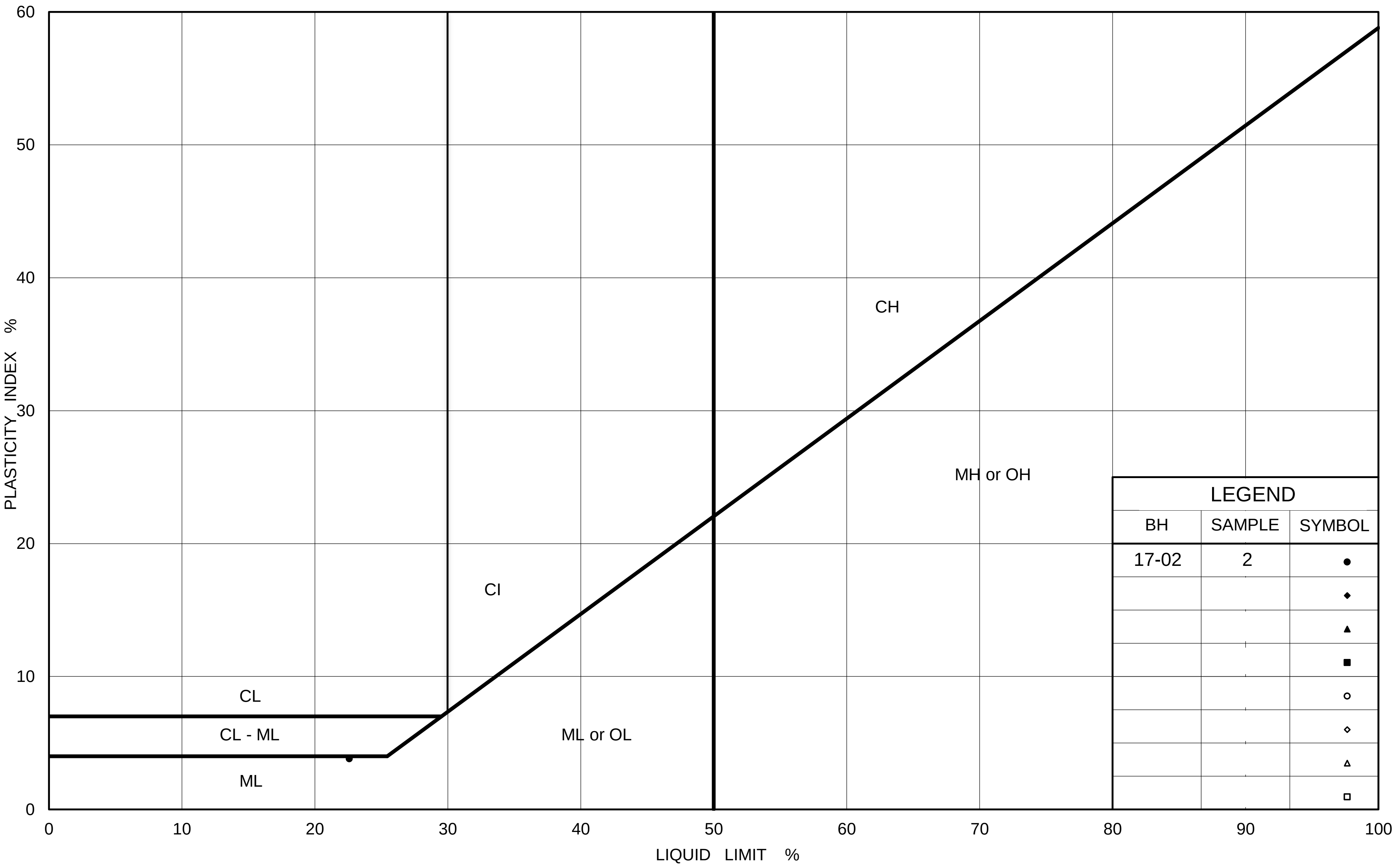
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-03	1	209.7
■	17-08	1A	209.6

Project Number: 1664714

Checked By: ____________________

Golder Associates

Date: 16-Oct-17



PLASTICITY CHART

(SM) Silty Sand of Slight Plasticity

Figure No. B5

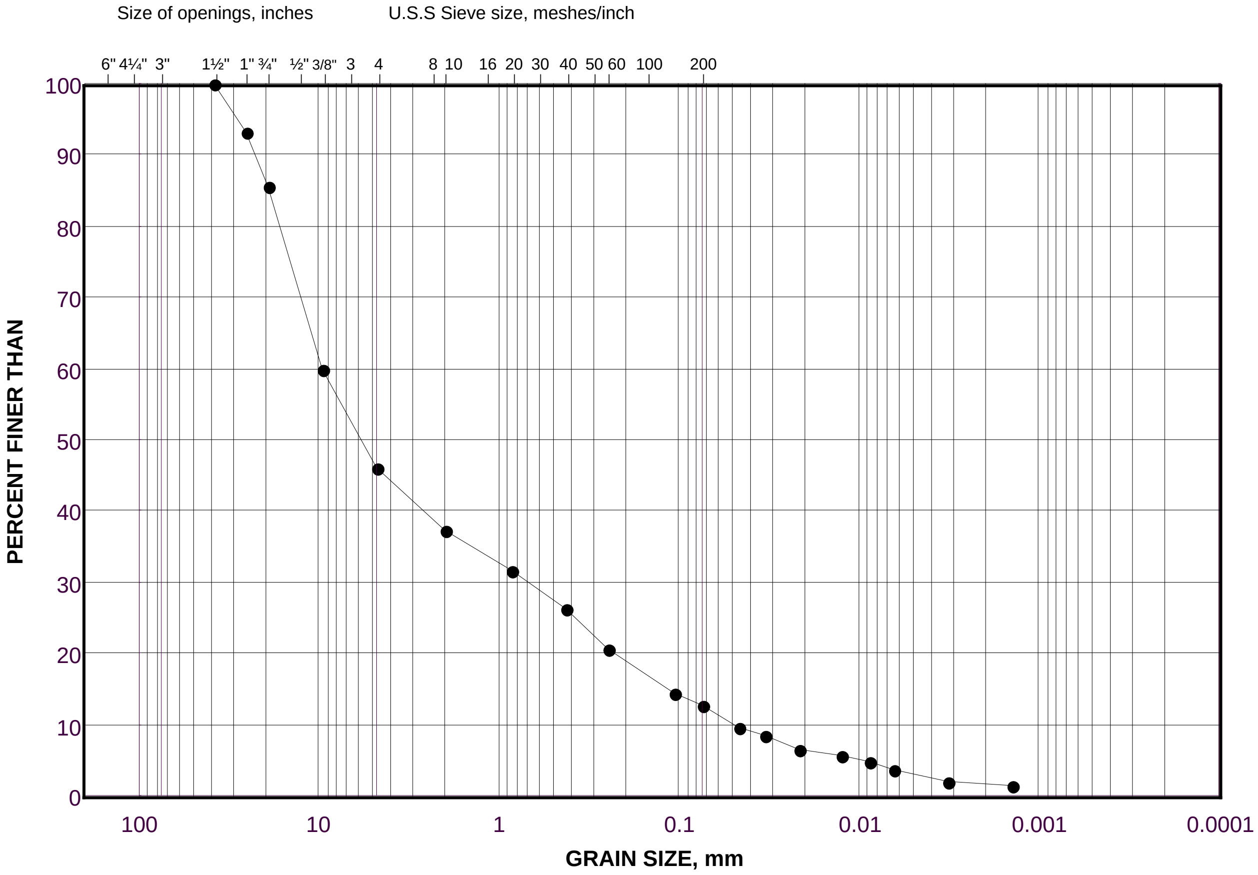
Project No. 1664714

Checked By:

GRAIN SIZE DISTRIBUTION

(SW-GW) Sandy Gravel

FIGURE B6



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	**GRAVEL SIZE**		**SAND SIZE**			**FINE GRAINED**

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-06	5	206.6

Project Number: 1664714

Checked By: ______________________

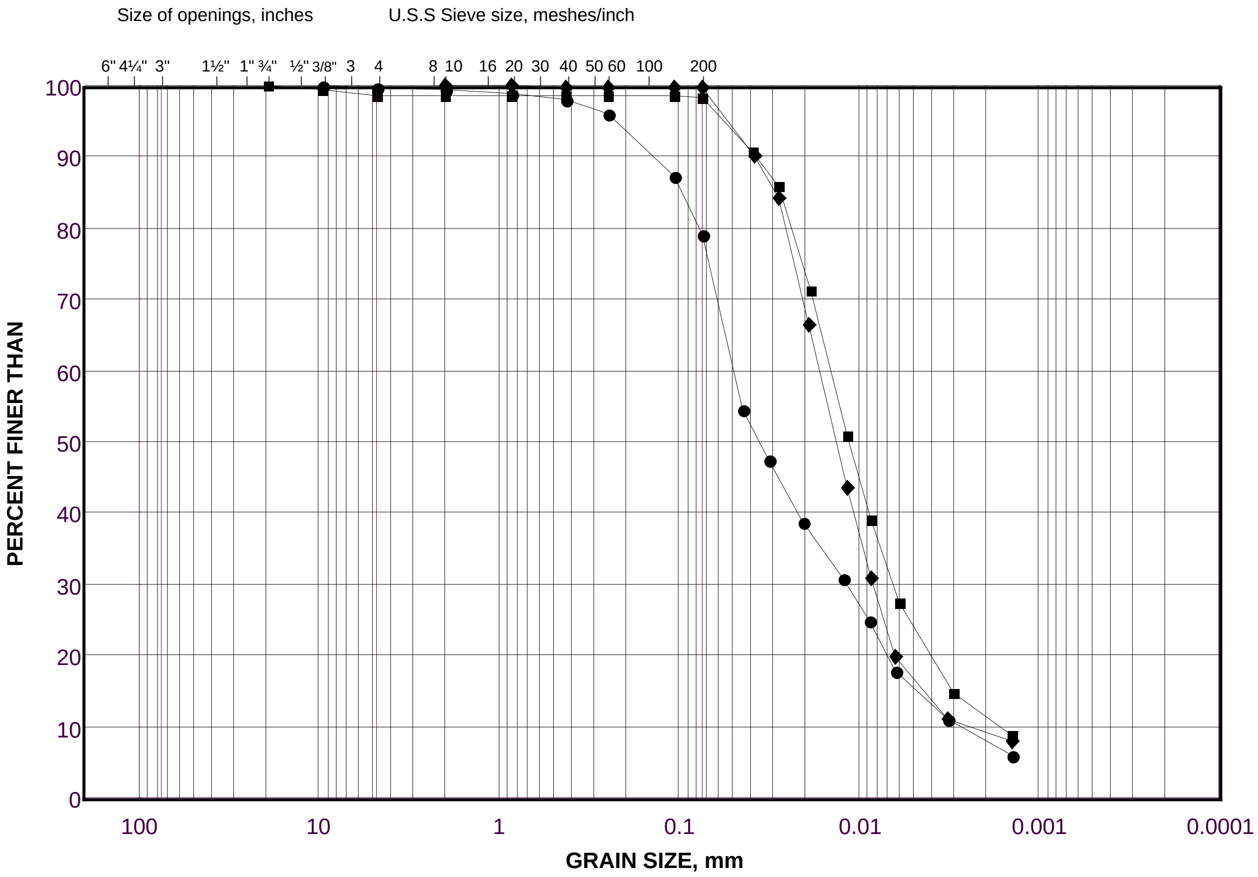
Golder Associates

Date: 16-Oct-17

GRAIN SIZE DISTRIBUTION

(ML) Sandy Silt to Silt

FIGURE B7



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

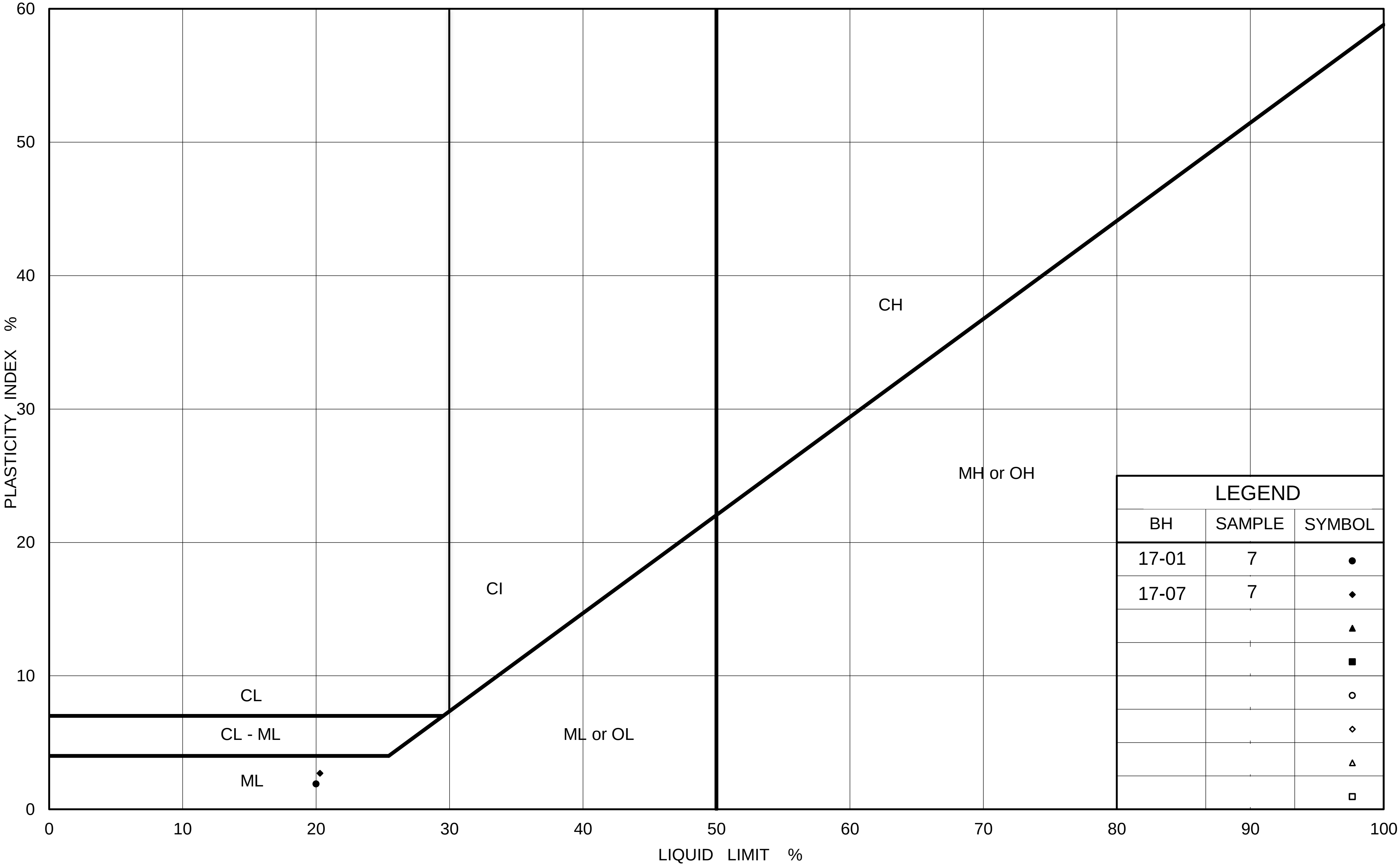
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-03	3	208.4
■	17-07	7	203.6
◆	17-01	7	203.6

Project Number: 1664714

Checked By: ____________________

Golder Associates

Date: 16-Oct-17



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PLASTICITY CHART
(ML) Silt

Figure No. B8

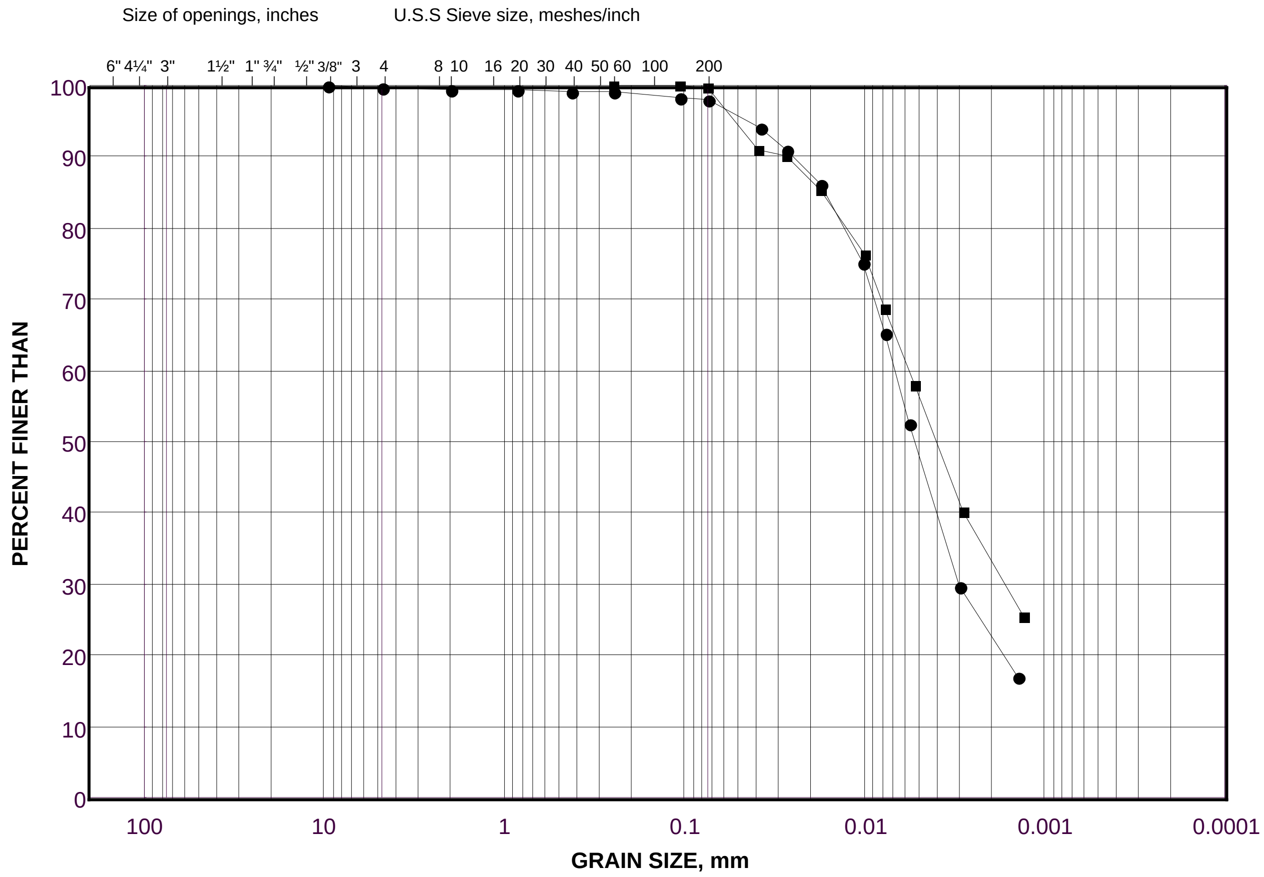
Project No. 1664714

Checked By:

GRAIN SIZE DISTRIBUTION

(CL) Silty Clay

FIGURE B9



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

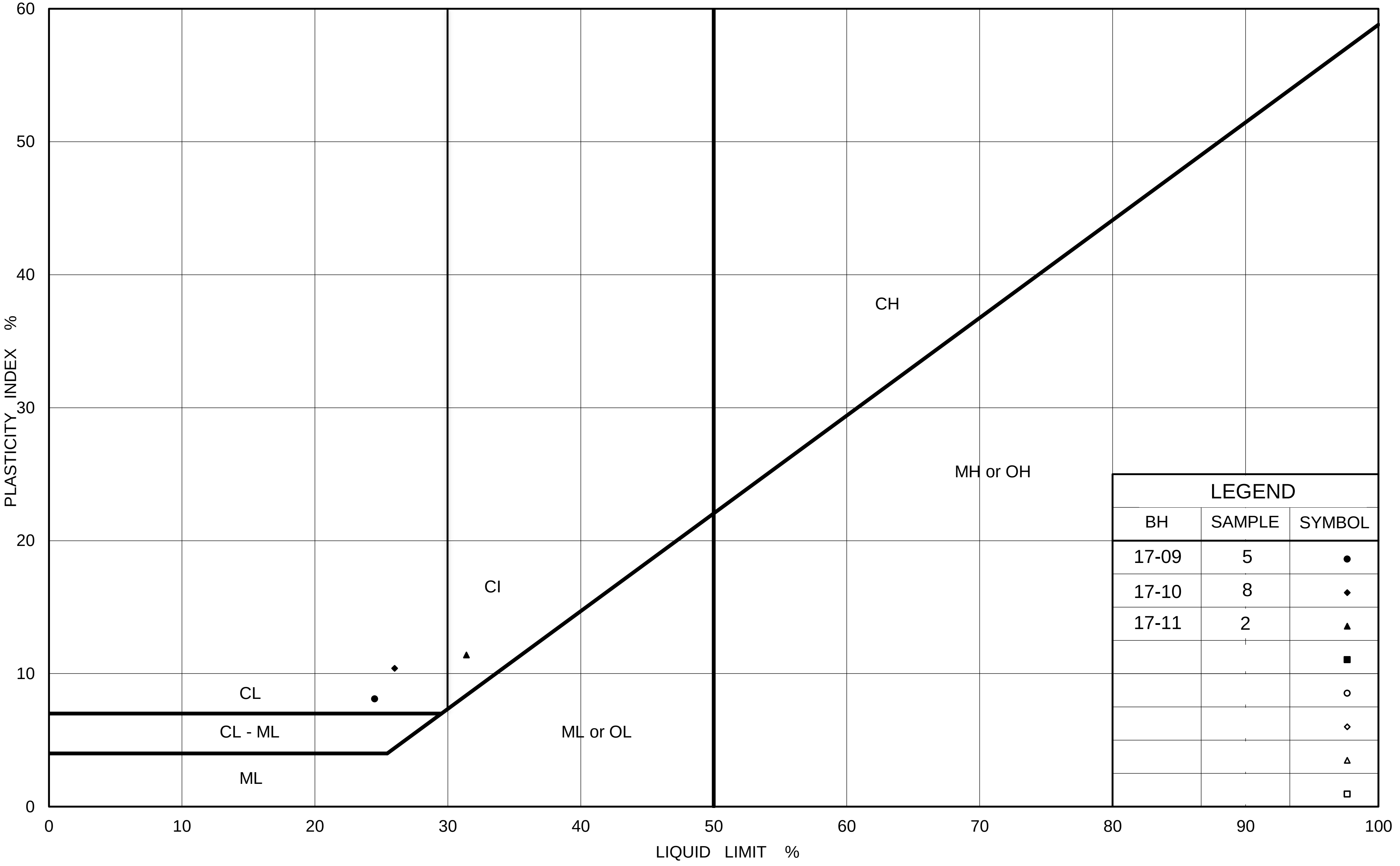
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-09	5	206.7
■	17-10	8	203.2

Project Number: 1664714

Checked By: ______________________

Golder Associates

Date: 16-Oct-17



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PLASTICITY CHART
(CL/CI) Silty Clay

Figure No. B10

Project No. 1664714

Checked By:

APPENDIX C

Chemical Test Results

Appendix C
Table C1
Soil Analytical Results - Metals and Inorganics

			Location	BH17-1 SA4B	BH17-3 SA2B	BH17-7 SA7	BH17-10 SA9
			Sample Date	24-May-2017	23-May-2017	26-May-2017	24-May-2017
			Sample Depth	2.4 - 2.9 m	0.9 - 1.4 m	6.1 - 6.7 m	9.1 - 9.7 m
Parameter	**MOE Table 1 Standards**	**MOE Table 3 Standards**	**Unit**				
Metals and Inorganics							
Antimony	1.3	50	µg/g	<0.20	<0.20	<0.20	<0.20
Arsenic	18	18	µg/g	1.5	3.5	1.8	2.5
Barium	220	670	µg/g	77	43	58	53
Beryllium	2.5	10	µg/g	0.28	0.39	0.31	0.36
Boron	36	120	µg/g	0.72	0.19	0.18	0.18
Cadmium	1.2	1.9	µg/g	<0.10	0.12	<0.10	<0.10
Chromium	70	160	µg/g	11	17	14	15
Cobalt	21	100	µg/g	4.4	7.6	5.5	6.6
Copper	92	300	µg/g	9.4	25	12	16
Lead	120	120	µg/g	4.9	25	4.7	5.7
Mercury	0.27	20	µg/g	<0.050	<0.050	<0.050	<0.050
Molybdenum	2	40	µg/g	<0.50	<0.50	<0.50	<0.50
Nickel	82	340	µg/g	8.5	16	11	14
Selenium	1.5	5.5	µg/g	<0.50	<0.50	<0.50	<0.50
Silver	0.5	50	µg/g	<0.20	<0.20	<0.20	<0.20
Thallium	1	3.3	µg/g	0.051	0.1	0.087	0.097
Uranium	2.5	33	µg/g	0.35	0.54	0.55	0.62
Vanadium	86	86	µg/g	18	24	23	23
Zinc	290	340	µg/g	27	44	29	34
Hexavalent Chromium	0.66	10	µg/g	<0.2	<0.2	<0.2	<0.2
Boron, Hot Water Soluble	NV	2	µg/g	<5.0	5.9	6.9	7.4
Conductivity	0.57	1.4	ms/cm	**1.9**	**1.6**	0.25	0.22
Cyanide (free)	0.051	0.051	µg/g	0.02	<0.01	<0.01	<0.01
Sodium Adsorption Ratio	2.4	12	-	**12**	**6.8**	**0.4**	0.35
pH	5 - 9 (5 - 11)	5 - 9 (5 - 11)	pH units	7.28	7.91	7.88	7.84

NOTES

m = depth in metre below ground surface

ppm = parts per million

µg/g = micrograms per gram

\- = parameter not anaylzed

5 - 9 (5 - 11) = standards for pH: 5 to 9 for surface soil, defined as soil from ground surface and 1.5 mbgs; 5 to 11 for subsurface soil, defined as soil from depths deeper than 1.5 mbgs

NV = No value

< = concentration not detected above the RDL

MOE Table 1 Standards = MOE Table 1 Standards - Ministry of the Environment Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act, Table 1: Full Depth Generic Site Condition Standards in Background Condition for Residential/Parkland/Institutional /Industrial/Commercial/ Community Property Use and for All Textured Soil (April 15, 2011)

MOE Table 3 Standards = MOE Table 3 Standards - Ministry of the Environment Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act, Table 3: Full Depth Generic Site Condition Standards in a Non-Potable Ground Water Condition for Industrial/Commercial/Community Property Use and for Medium and Fine Textured Soil (April 15, 2011)

3.6 = parameter exceeds the MOE Table 1 Soil Standards

1.7 = parameter exceeds the MOE Table 3 Soil Standards

Your Project #: 1664714
Site Location: CALDON EA
Your C.O.C. #: 272044-23-01

Attention:Amelia Jewison

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/06/19
Report #: R4548075
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7C2145
Received: 2017/06/13, 11:56

Sample Matrix: Soil
Samples Received: 6

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Hot Water Extractable Boron	4	2017/06/16	2017/06/16	CAM SOP-00408	R153 Ana. Prot. 2011
Chloride (20:1 extract)	2	N/A	2017/06/16	CAM SOP-00463	EPA 325.2 m
Free (WAD) Cyanide	4	2017/06/15	2017/06/16	CAM SOP-00457	OMOE E3015 m
Conductivity	2	N/A	2017/06/16	CAM SOP-00414	OMOE E3530 v1 m
Conductivity	4	2017/06/16	2017/06/16	CAM SOP-00414	OMOE E3530 v1 m
Hexavalent Chromium in Soil by IC (1)	4	2017/06/15	2017/06/16	CAM SOP-00436	EPA 3060/7199 m
Strong Acid Leachable Metals by ICPMS	4	2017/06/16	2017/06/16	CAM SOP-00447	EPA 6020B m
Moisture	4	N/A	2017/06/15	CAM SOP-00445	Carter 2nd ed 51.2 m
pH CaCl2 EXTRACT	6	2017/06/15	2017/06/15	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2017/06/13	2017/06/16	CAM SOP-00414	SM 22 2510 m
Sodium Adsorption Ratio (SAR)	2	N/A	2017/06/16	CAM SOP-00102	EPA 6010C
Sodium Adsorption Ratio (SAR)	2	N/A	2017/06/19	CAM SOP-00102	EPA 6010C
SAR - ICP Metals	4	2017/06/16	2017/06/16	CAM SOP-00408	EPA 6010D m
Sulphate (20:1 Extract)	2	N/A	2017/06/16	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.

Your Project #: 1664714
Site Location: CALDON EA
Your C.O.C. #: 272044-23-01

Attention:Amelia Jewison

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/06/19
Report #: R4548075
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7C2145
Received: 2017/06/13, 11:56

Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
(1) Soils are reported on a dry weight basis unless otherwise specified.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Total Cover Pages : 2

RESULTS OF ANALYSES OF SOIL

Maxxam ID		ENZ122		ENZ123		ENZ124		ENZ125		
Sampling Date		2017/05/24		2017/05/23		2017/05/26		2017/05/24		
COC Number		272044-23-01		272044-23-01		272044-23-01		272044-23-01		
	UNITS	**BH17-1 SA4B**	**QC Batch**	**BH17-3 SA2B**	**QC Batch**	**BH17-7 SA7**	**QC Batch**	**BH17-10 SA9**	**RDL**	**QC Batch**
Calculated Parameters										
Sodium Adsorption Ratio	N/A	12	5026642	6.8	5026642	0.40	5026642	0.35		5026642
Inorganics										
Conductivity	mS/cm	1.9	5030077	1.6	5030150	0.25	5030077	0.22	0.002	5031349
Moisture	%	22	5029658	13	5029658	20	5029658	17	1.0	5029658
Available (CaCl2) pH	pH	7.28	5029470	7.91	5029470	7.88	5029470	7.84		5029470
WAD Cyanide (Free)	ug/g	0.02	5029506	<0.01	5029506	<0.01	5029506	<0.01	0.01	5029506
Metals										
Soluble Calcium (Ca)	mg/L	46.1	5030069	161	5030149	20.6	5030069	23.1	0.5	5031342
Soluble Magnesium (Mg)	mg/L	5.0	5030069	5.5	5030149	16.3	5030069	11.9	0.5	5031342
Soluble Sodium (Na)	mg/L	324	5030069	322	5030149	10	5030069	8	5	5031342

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Maxxam ID		ENZ126	ENZ126	ENZ127		
Sampling Date		2017/05/23	2017/05/23	2017/05/24		
COC Number		272044-23-01	272044-23-01	272044-23-01		
	UNITS	**BH17-6 SA4**	**BH17-6 SA4 Lab-Dup**	**BH17-10 SA10**	**RDL**	**QC Batch**
Calculated Parameters						
Resistivity	ohm-cm	980		1600		5027714
Inorganics						
Soluble (20:1) Chloride (Cl)	ug/g	460		270	20	5030086
Conductivity	umho/cm	1020	1010	614	2	5031550
Available (CaCl2) pH	pH	7.72		7.92		5029470
Soluble (20:1) Sulphate (SO4)	ug/g	140	130	21	20	5030090

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

ELEMENTS BY ATOMIC SPECTROSCOPY (SOIL)

Maxxam ID		ENZ122	ENZ123	ENZ124	ENZ125		
Sampling Date		2017/05/24	2017/05/23	2017/05/26	2017/05/24		
COC Number		272044-23-01	272044-23-01	272044-23-01	272044-23-01		
	UNITS	**BH17-1 SA4B**	**BH17-3 SA2B**	**BH17-7 SA7**	**BH17-10 SA9**	**RDL**	**QC Batch**
Inorganics							
Chromium (VI)	ug/g	<0.2	<0.2	<0.2	<0.2	0.2	5029540
Metals							
Hot Water Ext. Boron (B)	ug/g	0.72	0.19	0.18	0.18	0.050	5031111
Acid Extractable Antimony (Sb)	ug/g	<0.20	<0.20	<0.20	<0.20	0.20	5031413
Acid Extractable Arsenic (As)	ug/g	1.5	3.5	1.8	2.5	1.0	5031413
Acid Extractable Barium (Ba)	ug/g	77	43	58	53	0.50	5031413
Acid Extractable Beryllium (Be)	ug/g	0.28	0.39	0.31	0.36	0.20	5031413
Acid Extractable Boron (B)	ug/g	<5.0	5.9	6.9	7.4	5.0	5031413
Acid Extractable Cadmium (Cd)	ug/g	<0.10	0.12	<0.10	<0.10	0.10	5031413
Acid Extractable Chromium (Cr)	ug/g	11	17	14	15	1.0	5031413
Acid Extractable Cobalt (Co)	ug/g	4.4	7.6	5.5	6.6	0.10	5031413
Acid Extractable Copper (Cu)	ug/g	9.4	25	12	16	0.50	5031413
Acid Extractable Lead (Pb)	ug/g	4.9	25	4.7	5.7	1.0	5031413
Acid Extractable Molybdenum (Mo)	ug/g	<0.50	<0.50	<0.50	<0.50	0.50	5031413
Acid Extractable Nickel (Ni)	ug/g	8.5	16	11	14	0.50	5031413
Acid Extractable Selenium (Se)	ug/g	<0.50	<0.50	<0.50	<0.50	0.50	5031413
Acid Extractable Silver (Ag)	ug/g	<0.20	<0.20	<0.20	<0.20	0.20	5031413
Acid Extractable Thallium (Tl)	ug/g	0.051	0.10	0.087	0.097	0.050	5031413
Acid Extractable Uranium (U)	ug/g	0.35	0.54	0.55	0.62	0.050	5031413
Acid Extractable Vanadium (V)	ug/g	18	24	23	23	5.0	5031413
Acid Extractable Zinc (Zn)	ug/g	27	44	29	34	5.0	5031413
Acid Extractable Mercury (Hg)	ug/g	<0.050	<0.050	<0.050	<0.050	0.050	5031413

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

TEST SUMMARY

Maxxam ID: ENZ122
Sample ID: BH17-1 SA4B
Matrix: Soil

Collected: 2017/05/24
Shipped:
Received: 2017/06/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Hot Water Extractable Boron	ICP	5031111	2017/06/16	2017/06/16	Jolly John
Free (WAD) Cyanide	TECH	5029506	2017/06/15	2017/06/16	Louise Harding
Conductivity	AT	5030077	2017/06/16	2017/06/16	Neil Dassanayake
Hexavalent Chromium in Soil by IC	IC/SPEC	5029540	2017/06/15	2017/06/16	Manoj Kumar Gera
Strong Acid Leachable Metals by ICPMS	ICP/MS	5031413	2017/06/16	2017/06/16	Viviana Canzonieri
Moisture	BAL	5029658	N/A	2017/06/15	Valentina Kaftani
pH CaCl2 EXTRACT	AT	5029470	2017/06/15	2017/06/15	Tahir Anwar
Sodium Adsorption Ratio (SAR)	CALC/MET	5026642	N/A	2017/06/19	Automated Statchk
SAR - ICP Metals	ICP	5030069	2017/06/16	2017/06/16	Jolly John

Maxxam ID: ENZ123
Sample ID: BH17-3 SA2B
Matrix: Soil

Collected: 2017/05/23
Shipped:
Received: 2017/06/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Hot Water Extractable Boron	ICP	5031111	2017/06/16	2017/06/16	Jolly John
Free (WAD) Cyanide	TECH	5029506	2017/06/15	2017/06/16	Louise Harding
Conductivity	AT	5030150	2017/06/16	2017/06/16	Neil Dassanayake
Hexavalent Chromium in Soil by IC	IC/SPEC	5029540	2017/06/15	2017/06/16	Manoj Kumar Gera
Strong Acid Leachable Metals by ICPMS	ICP/MS	5031413	2017/06/16	2017/06/16	Viviana Canzonieri
Moisture	BAL	5029658	N/A	2017/06/15	Valentina Kaftani
pH CaCl2 EXTRACT	AT	5029470	2017/06/15	2017/06/15	Tahir Anwar
Sodium Adsorption Ratio (SAR)	CALC/MET	5026642	N/A	2017/06/16	Automated Statchk
SAR - ICP Metals	ICP	5030149	2017/06/16	2017/06/16	Jolly John

Maxxam ID: ENZ124
Sample ID: BH17-7 SA7
Matrix: Soil

Collected: 2017/05/26
Shipped:
Received: 2017/06/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Hot Water Extractable Boron	ICP	5031111	2017/06/16	2017/06/16	Jolly John
Free (WAD) Cyanide	TECH	5029506	2017/06/15	2017/06/16	Louise Harding
Conductivity	AT	5030077	2017/06/16	2017/06/16	Neil Dassanayake
Hexavalent Chromium in Soil by IC	IC/SPEC	5029540	2017/06/15	2017/06/16	Manoj Kumar Gera
Strong Acid Leachable Metals by ICPMS	ICP/MS	5031413	2017/06/16	2017/06/16	Viviana Canzonieri
Moisture	BAL	5029658	N/A	2017/06/15	Valentina Kaftani
pH CaCl2 EXTRACT	AT	5029470	2017/06/15	2017/06/15	Tahir Anwar
Sodium Adsorption Ratio (SAR)	CALC/MET	5026642	N/A	2017/06/19	Automated Statchk
SAR - ICP Metals	ICP	5030069	2017/06/16	2017/06/16	Jolly John

TEST SUMMARY

Maxxam ID: ENZ125
Sample ID: BH17-10 SA9
Matrix: Soil

Collected: 2017/05/24
Shipped:
Received: 2017/06/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Hot Water Extractable Boron	ICP	5031111	2017/06/16	2017/06/16	Jolly John
Free (WAD) Cyanide	TECH	5029506	2017/06/15	2017/06/16	Louise Harding
Conductivity	AT	5031349	2017/06/16	2017/06/16	Neil Dassanayake
Hexavalent Chromium in Soil by IC	IC/SPEC	5029540	2017/06/15	2017/06/16	Manoj Kumar Gera
Strong Acid Leachable Metals by ICPMS	ICP/MS	5031413	2017/06/16	2017/06/16	Viviana Canzonieri
Moisture	BAL	5029658	N/A	2017/06/15	Valentina Kaftani
pH CaCl2 EXTRACT	AT	5029470	2017/06/15	2017/06/15	Tahir Anwar
Sodium Adsorption Ratio (SAR)	CALC/MET	5026642	N/A	2017/06/16	Automated Statchk
SAR - ICP Metals	ICP	5031342	2017/06/16	2017/06/16	Jolly John

Maxxam ID: ENZ126
Sample ID: BH17-6 SA4
Matrix: Soil

Collected: 2017/05/23
Shipped:
Received: 2017/06/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5030086	N/A	2017/06/16	Alina Dobreanu
Conductivity	AT	5031550	N/A	2017/06/16	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5029470	2017/06/15	2017/06/15	Tahir Anwar
Resistivity of Soil		5027714	2017/06/16	2017/06/16	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5030090	N/A	2017/06/16	Alina Dobreanu

Maxxam ID: ENZ126 Dup
Sample ID: BH17-6 SA4
Matrix: Soil

Collected: 2017/05/23
Shipped:
Received: 2017/06/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	5031550	N/A	2017/06/16	Neil Dassanayake
Sulphate (20:1 Extract)	KONE/EC	5030090	N/A	2017/06/16	Alina Dobreanu

Maxxam ID: ENZ127
Sample ID: BH17-10 SA10
Matrix: Soil

Collected: 2017/05/24
Shipped:
Received: 2017/06/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5030086	N/A	2017/06/16	Alina Dobreanu
Conductivity	AT	5031550	N/A	2017/06/16	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5029470	2017/06/15	2017/06/15	Tahir Anwar
Resistivity of Soil		5027714	2017/06/16	2017/06/16	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5030090	N/A	2017/06/16	Alina Dobreanu

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	2.0°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**
5029470	Available (CaCl2) pH	2017/06/15			99	97 - 103			0.46	N/A
5029506	WAD Cyanide (Free)	2017/06/16	104	75 - 125	105	80 - 120	<0.01	ug/g	17	35
5029540	Chromium (VI)	2017/06/16	61 (1)	75 - 125	91	80 - 120	<0.2	ug/g	NC	35
5029658	Moisture	2017/06/15							NC	20
5030069	Soluble Calcium (Ca)	2017/06/16			100	80 - 120	<0.5	mg/L	6.9	30
5030069	Soluble Magnesium (Mg)	2017/06/16			101	80 - 120	<0.5	mg/L	6.2	30
5030069	Soluble Sodium (Na)	2017/06/16			98	80 - 120	<5	mg/L	7.2	30
5030077	Conductivity	2017/06/16			98	90 - 110	<0.002	mS/cm	5.8	10
5030086	Soluble (20:1) Chloride (Cl)	2017/06/16	108	70 - 130	106	70 - 130	<20	ug/g	NC	35
5030090	Soluble (20:1) Sulphate (SO4)	2017/06/16	NC	70 - 130	104	70 - 130	<20	ug/g	4.4	35
5030149	Soluble Calcium (Ca)	2017/06/16			93	80 - 120	<0.5	mg/L	2.0	30
5030149	Soluble Magnesium (Mg)	2017/06/16			97	80 - 120	<0.5	mg/L	8.8	30
5030149	Soluble Sodium (Na)	2017/06/16			97	80 - 120	<5	mg/L	NC	30
5030150	Conductivity	2017/06/16			99	90 - 110	<0.002	mS/cm	5.3	10
5031111	Hot Water Ext. Boron (B)	2017/06/16	100	75 - 125	97	75 - 125	<0.050	ug/g	0.52	40
5031342	Soluble Calcium (Ca)	2017/06/16			95	80 - 120	<0.5	mg/L	15	30
5031342	Soluble Magnesium (Mg)	2017/06/16			100	80 - 120	<0.5	mg/L	NC	30
5031342	Soluble Sodium (Na)	2017/06/16			98	80 - 120	<5	mg/L	1.7	30
5031349	Conductivity	2017/06/16			100	90 - 110	<0.002	mS/cm	0.30	10
5031413	Acid Extractable Antimony (Sb)	2017/06/16	101	75 - 125	105	80 - 120	<0.20	ug/g	NC	30
5031413	Acid Extractable Arsenic (As)	2017/06/16	100	75 - 125	102	80 - 120	<1.0	ug/g	7.0	30
5031413	Acid Extractable Barium (Ba)	2017/06/16	112	75 - 125	101	80 - 120	<0.50	ug/g	11	30
5031413	Acid Extractable Beryllium (Be)	2017/06/16	98	75 - 125	93	80 - 120	<0.20	ug/g	NC	30
5031413	Acid Extractable Boron (B)	2017/06/16	99	75 - 125	92	80 - 120	<5.0	ug/g	2.8	30
5031413	Acid Extractable Cadmium (Cd)	2017/06/16	104	75 - 125	104	80 - 120	<0.10	ug/g	NC	30
5031413	Acid Extractable Chromium (Cr)	2017/06/16	107	75 - 125	102	80 - 120	<1.0	ug/g	7.8	30
5031413	Acid Extractable Cobalt (Co)	2017/06/16	101	75 - 125	99	80 - 120	<0.10	ug/g	0.048	30
5031413	Acid Extractable Copper (Cu)	2017/06/16	101	75 - 125	99	80 - 120	<0.50	ug/g	0.35	30
5031413	Acid Extractable Lead (Pb)	2017/06/16	NC	75 - 125	101	80 - 120	<1.0	ug/g	15	30
5031413	Acid Extractable Mercury (Hg)	2017/06/16	96	75 - 125	95	80 - 120	<0.050	ug/g	23	30
5031413	Acid Extractable Molybdenum (Mo)	2017/06/16	105	75 - 125	103	80 - 120	<0.50	ug/g	NC	30

QUALITY ASSURANCE REPORT(CONT'D)

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	**Parameter**	**Date**	**% Recovery**	**QC Limits**	**% Recovery**	**QC Limits**	**Value**	**UNITS**	**Value (%)**	**QC Limits**
5031413	Acid Extractable Nickel (Ni)	2017/06/16	100	75 - 125	99	80 - 120	<0.50	ug/g	3.3	30
5031413	Acid Extractable Selenium (Se)	2017/06/16	103	75 - 125	101	80 - 120	<0.50	ug/g	NC	30
5031413	Acid Extractable Silver (Ag)	2017/06/16	102	75 - 125	102	80 - 120	<0.20	ug/g	NC	30
5031413	Acid Extractable Thallium (Tl)	2017/06/16	95	75 - 125	100	80 - 120	<0.050	ug/g	NC	30
5031413	Acid Extractable Uranium (U)	2017/06/16	97	75 - 125	99	80 - 120	<0.050	ug/g	26	30
5031413	Acid Extractable Vanadium (V)	2017/06/16	101	75 - 125	100	80 - 120	<5.0	ug/g	0.19	30
5031413	Acid Extractable Zinc (Zn)	2017/06/16	NC	75 - 125	108	80 - 120	<5.0	ug/g	1.2	30
5031550	Conductivity	2017/06/16			99	90 - 110	<2	umho/cm	1.0	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

(1) The matrix spike recovery was below the lower control limit. This may be due in part to the reducing environment of the sample. The matrix spike was reanalyzed to confirm result.

Maxxam Job #: B7C2145
Report Date: 2017/06/19

Golder Associates Ltd
Client Project #: 1664714
Site Location: CALDON EA
Sampler Initials: AJ

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

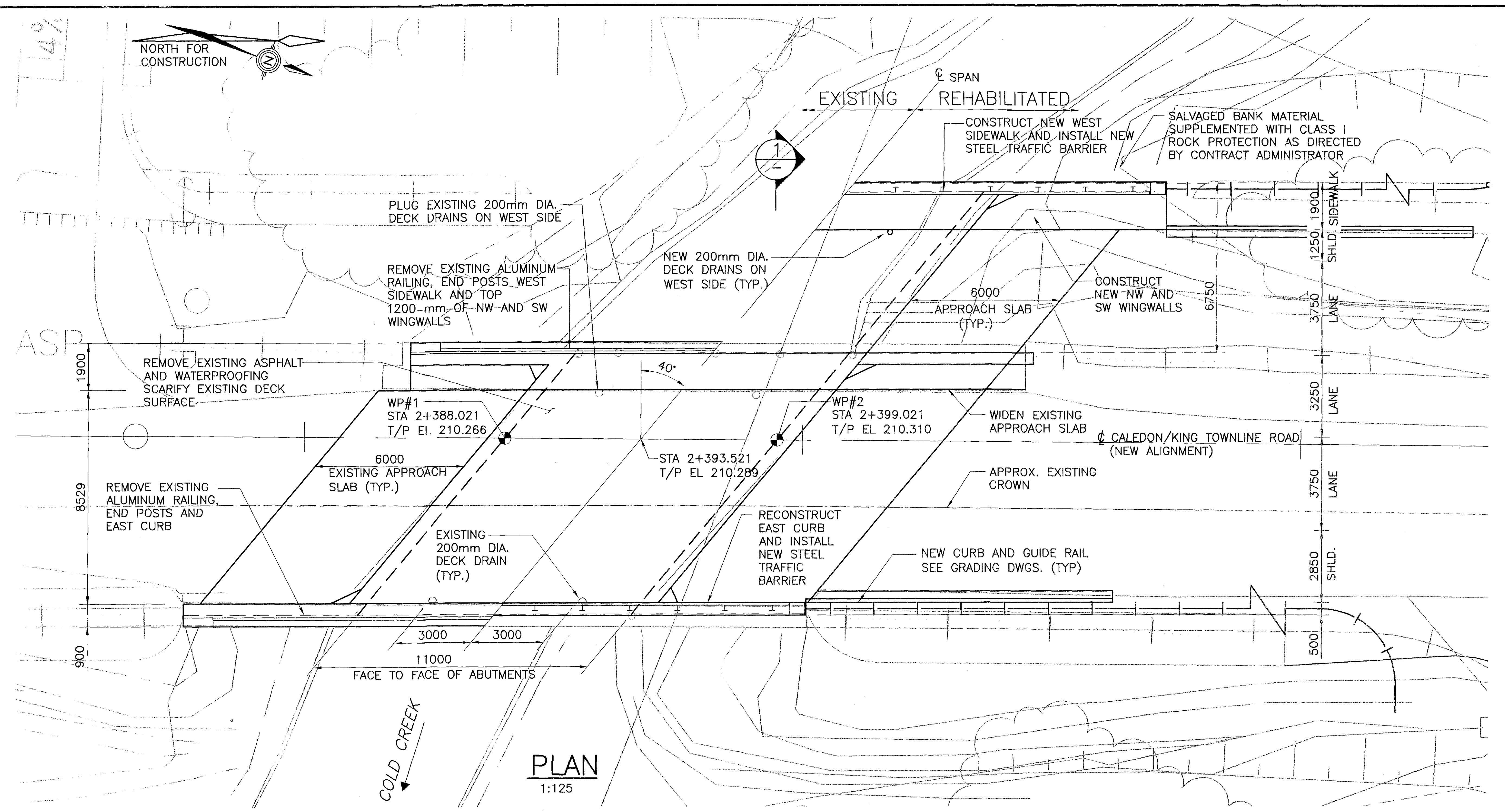
Cristina Carriere

Cristina Carriere, Scientific Services

APPENDIX D

Existing General Arrangement Drawings

- Project No. 04-01, Drawing No. S6 and
- Project No. 02-07, Drawing No. S1.



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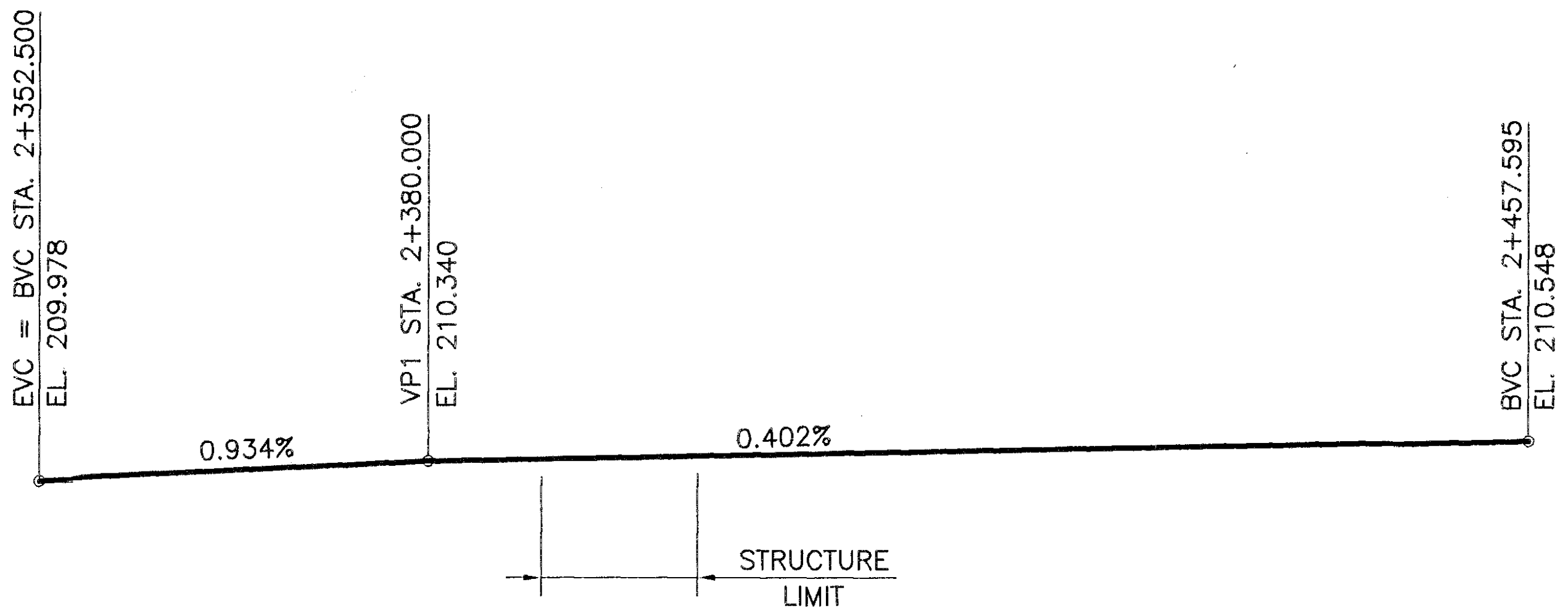
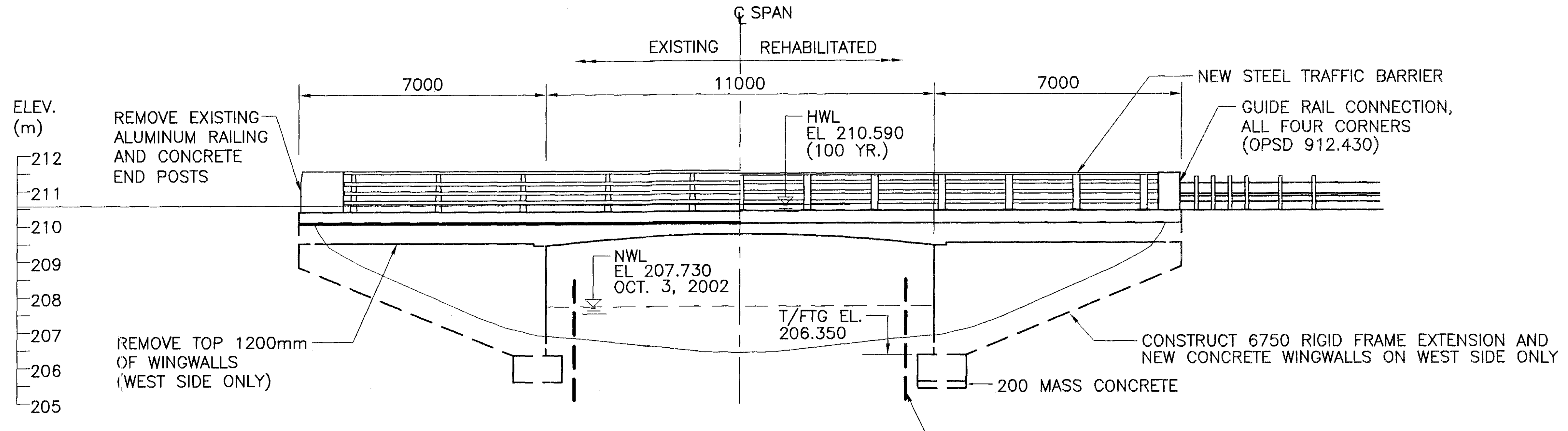


PROFILE CALEDON-KING TOWNLINE ROAD

(NEW ALIGNMENT)

N.T.S.



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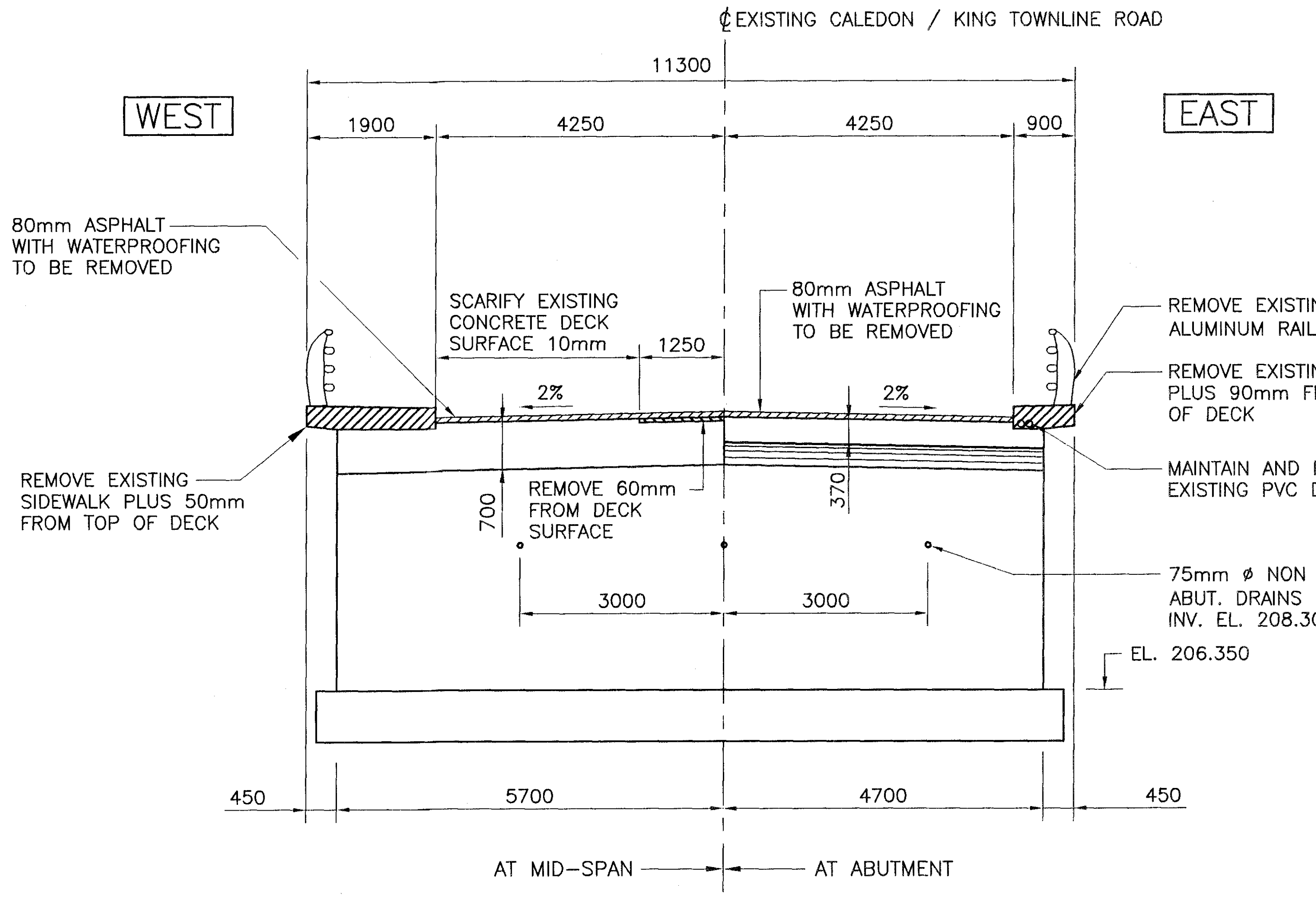

SECTION ① - EXISTING
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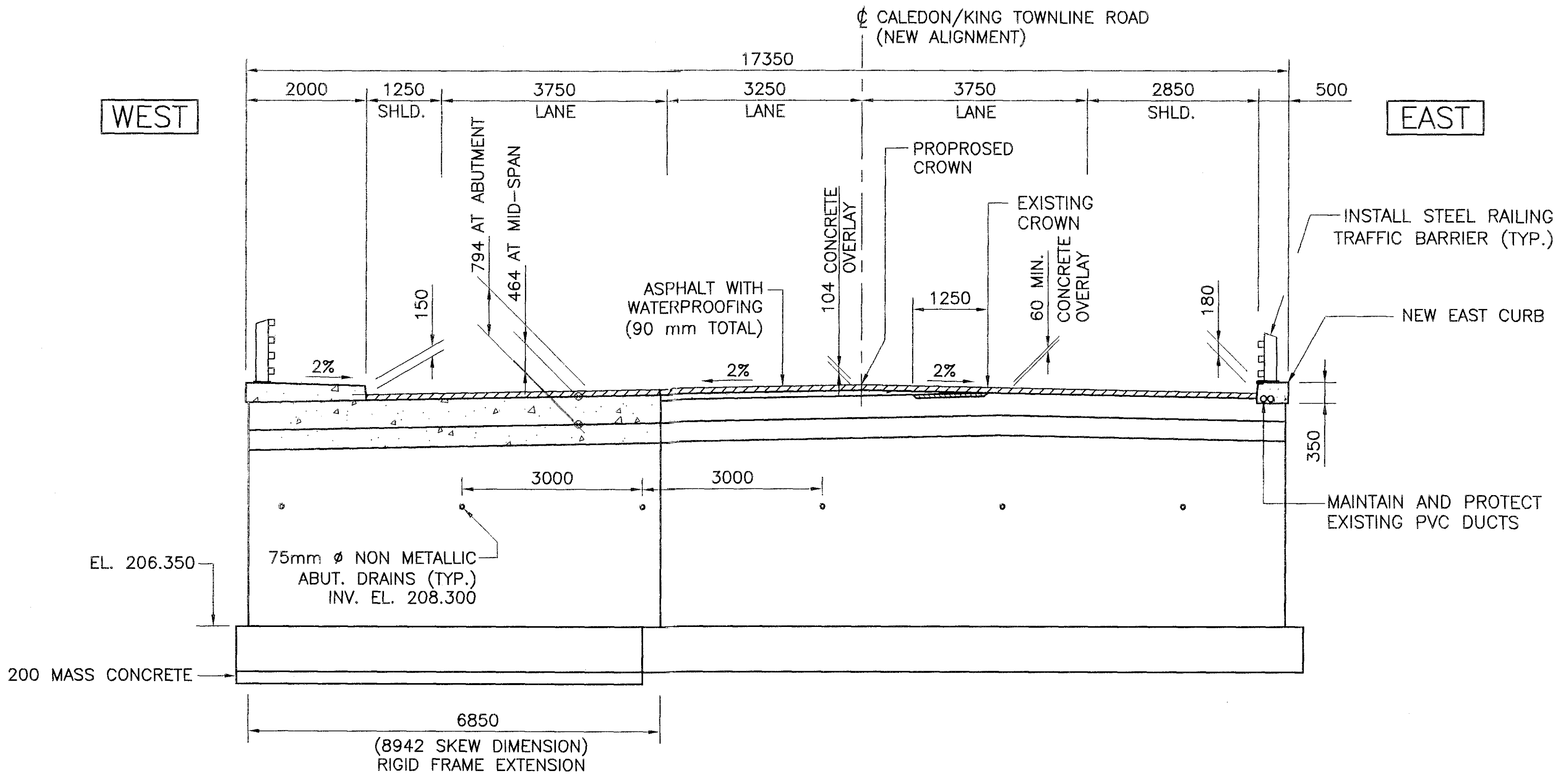


SECTION ① - PROPOSED
1:75

CONSTRUCTION RECORD DRAWING
CONTRACTOR FERMAR PAVING LIMITED
WORK COMMENCED JULY 2004
WORK COMPLETED JULY 2006

These record drawings have been prepared based on information submitted, in part, by others. While this information is believed to be reliable, MRC is not responsible for its accuracy, or for errors or omissions that may have been incorporated into this document as a result.

GENERAL NOTES:

1. ALL WORK SHALL BE UNDERTAKEN IN ACCORDANCE WITH ONTARIO PROVINCIAL STANDARD SPECIFICATIONS AND THE SPECIAL PROVISIONS OF THE CONTRACT
2. CLASS CONCRETE: 30 MPa
   – MASS CONCRETE : 20 MPa

   REINFORCING STEEL SHALL BE GRADE 400.
   BARS MARKED WITH PREFIX 'C' DENOTE COATED BARS.
3. CLEAR COVER TO REINFORCING STEEL

	70 ± 20 mm
DECK – TOP..	50 ± 10 mm
– BOTTOM..	100 ± 25 mm
4. BOTTOM OF FOOTING..................................	70 ± 20 mm
REMAINDER UNLESS NOTED OTHERWISE.........	

5. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE EXISTING WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE ENGINEER PRIOR TO PROCEEDING WITH THE WORK.
6. DESIGN CODE AND LOADING CHBDC–2000, CLASS A. TRAFFIC RAILING MEETS PERFORMANCE LEVEL 1 CRITERIA.
7. UNLESS SHOWN OTHERWISE, TENSION LAP LENGTHS SHALL BE CLASS B.
8. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12–1 AND 12–2, UNLESS INDICATED OTHERWISE.
9. SAWCUTS AT LIMITS OF REMOVAL SHALL BE 25mm DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL, WHICHEVER IS LESS (UNLESS OTHERWISE NOTED.)
10. EPOXY GROUT FOR DOWELS SHALL BE FROM THE MTO'S DESIGNATED SOURCES MANUAL AND INSTALLED IN ACCORDANCE WITH THE MANUFACTURER'S INSTRUCTIONS.
11. CALEDON–KING TOWNLINE ROAD TO BE CLOSED DURING CONSTRUCTION.
12. NO BACKFILL SHALL BE PLACED BEHIND ABUTMENT UNTIL DECK CONCRETE HAS REACH 25MPa STRENGTH.
13. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 0.5m

LIST OF DRAWINGS:

S6–1	GENERAL ARRANGEMENT
S6–2	REMOVALS
S6–3	FOUNDATIONS
S6–4	NEW CONSTRUCTION DETAILS
S6–5	WINGWALLS
S6–6	RAILING DETAILS
S6–7	RAILING ON SIDEWALK
S6–8	6000 mm APPROACH SLAB

NO.	BY	DATE	REVISION	CONS. CHECKED	TOWN APPR'D

APPROVED FOR CONSTRUCTION

THIS APPROVAL CONSTITUTES A GENERAL REVIEW AND DOES NOT CERTIFY DIMENSIONAL ACCURACY.

THIS APPROVAL IS SUBJECT TO THE FURTHER CERTIFICATION OF THE "AS CONSTRUCTED" WORKS BY A REGISTERED PROFESSIONAL ENGINEER OF THE PROVINCE OF ONTARIO.

DATE :______________ APPROVED BY:______________________

Town Engineer

BENCH MARK:

S.D. LEITCH
JUNE 23/09

K. ALBAZI
June 23/09

PROJECT NAME
CALEDON-KING TOWNLINE SOUTH RECONSTRUCTION
BRIDGE WIDENING AT STA. 2+393.521

CONSULTANT
McCORMICK RANKIN
CORPORATION

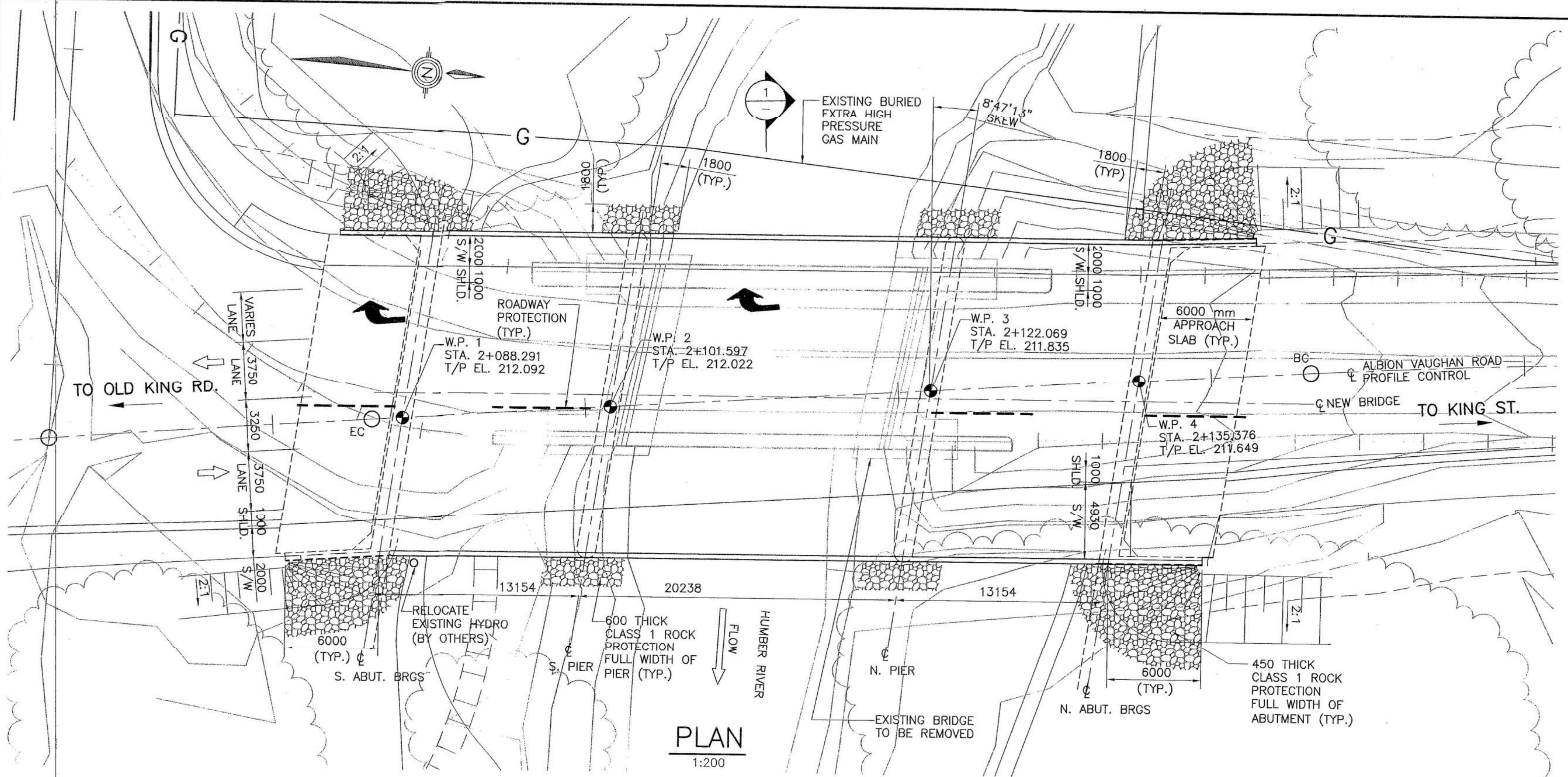
REGION OF PEEL

THE CORPORATION OF THE
TOWN OF CALEDON

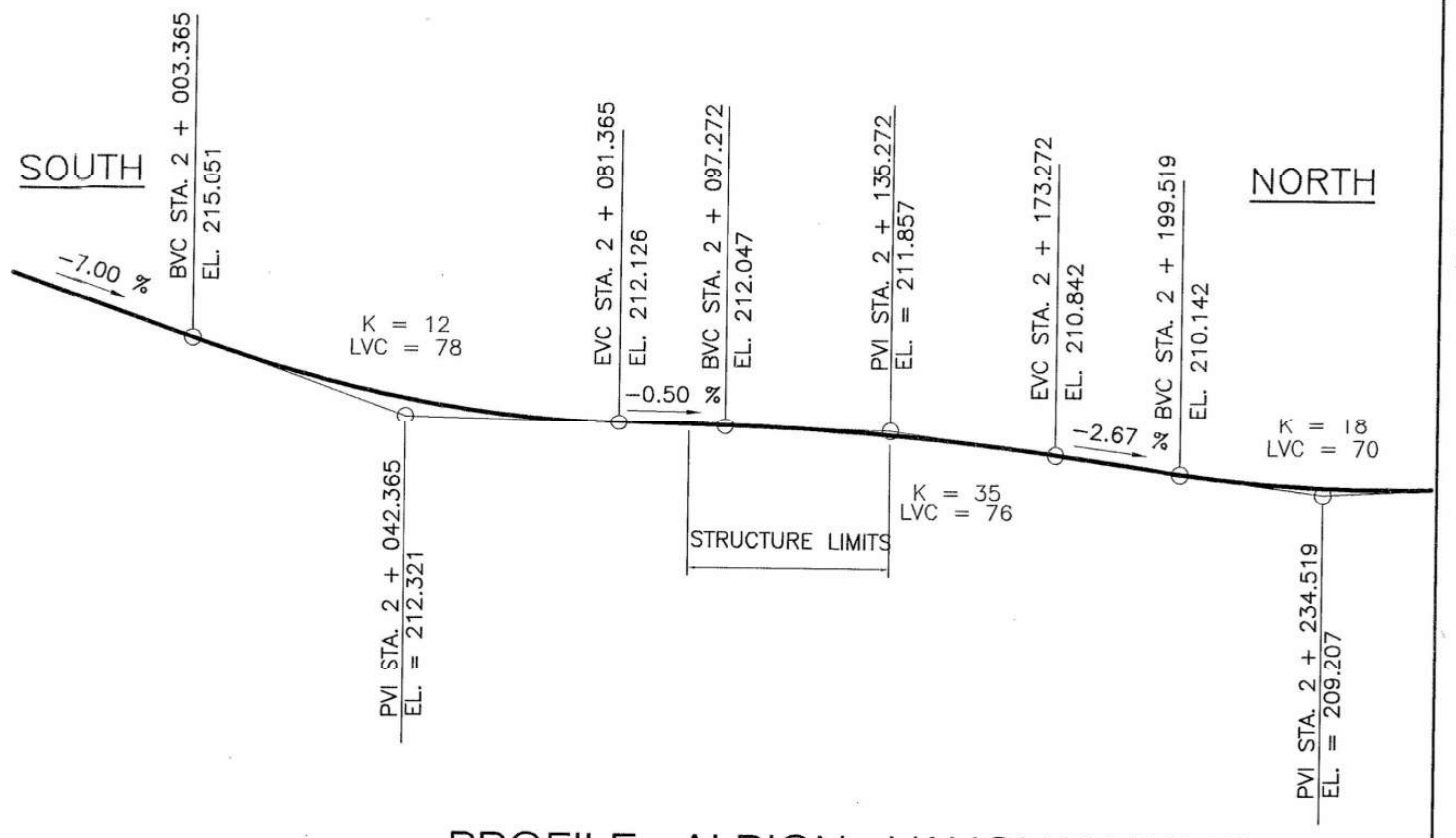
GENERAL ARRANGEMENT

SCALE: AS SHOWN		PROJECT No. 04–01
DESIGNED BY: SL	DRAWN BY: PRJ	DRAWNING No.
CHECKED BY: KA	DATE: APRIL, 2004	S6–

FILE LOCATION–S:\4355\306\
DRAWING NAME: S4355–306–01GA.DWG
DRAWN BY: DANI V.
MODIFIED: 2004/05/070:14:17
DATE PLOTTED: 2008/06/209:25:18



PLAN
1:200



PROFILE - ALBION - VAUGHAN ROAD
N.T.S.

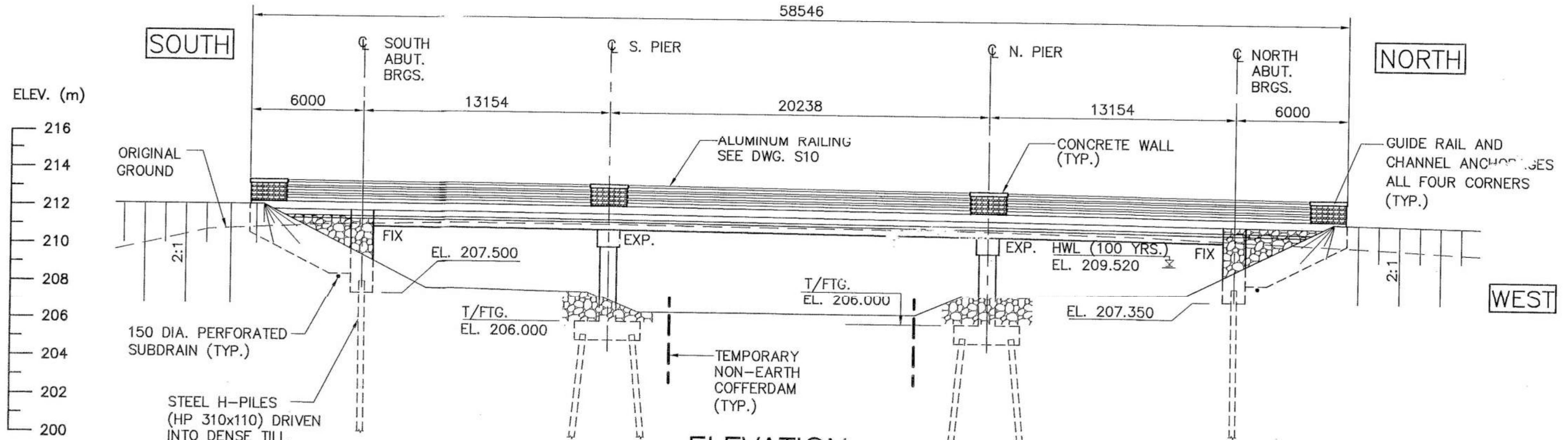
CONSTRUCTION RECORD DRAWING

CONTRACTOR: FERMAR PAVING LIMITED

WORK COMMENCED: JULY, 2002

WORK COMPLETED: JULY, 2004

These record drawings have been prepared based on information submitted, in part, by others. While this information is believed to be reliable, MRC is not responsible for its accuracy, or for errors or omissions that may have been incorporated into this document as a result.



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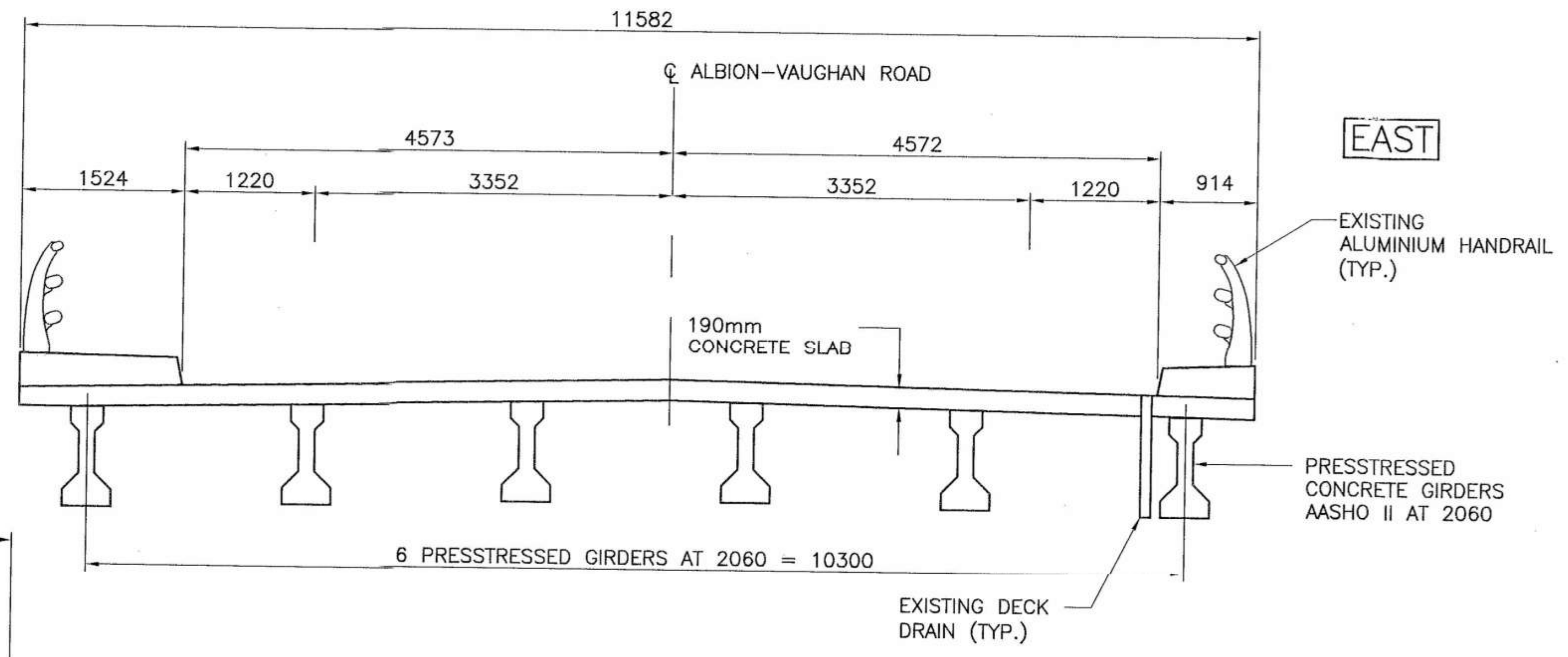

SECTION - EXISTING
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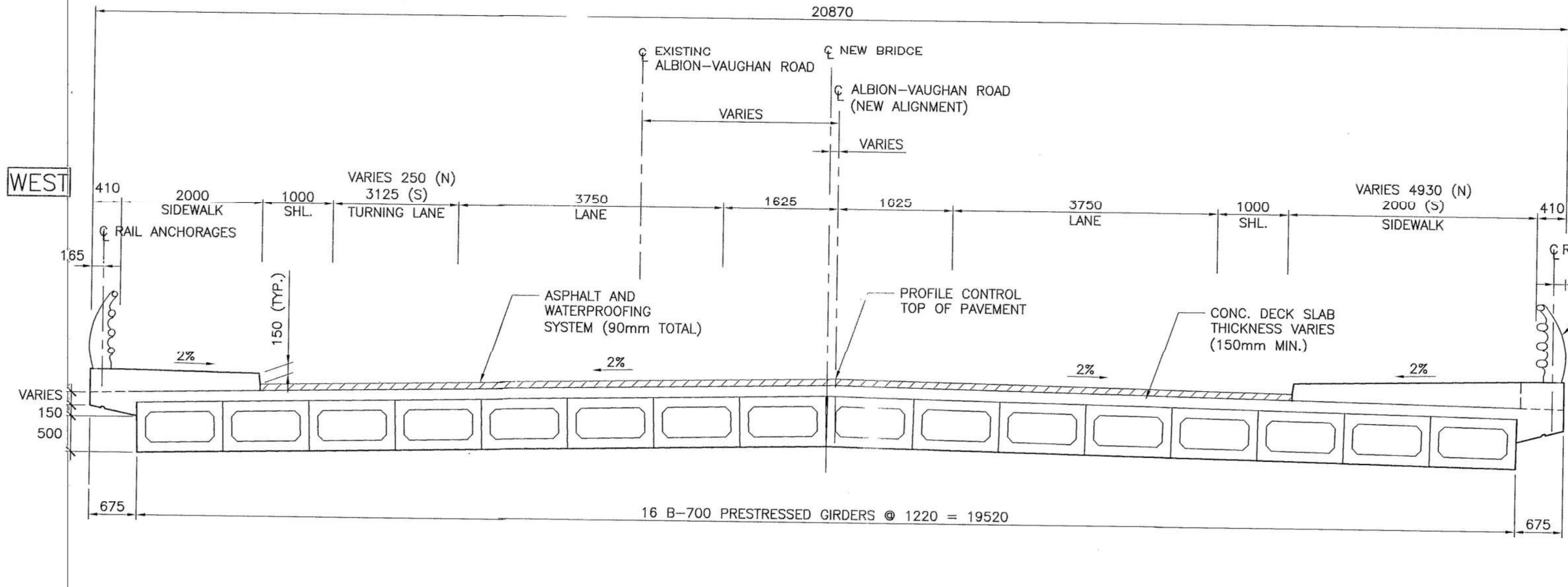


SECTION - NEW
1:50

GENERAL NOTES

1. CLASS OF CONCRETE

•PRECAST GIRDERS	40 MPa
DECK (INCLUDING SIDEWALKS AND TOPS OF ABUTMENTS AND WINGWALLS)	50 MPa (HPC)
PARAPET WALLS	50 MPa (HPC)
•REMAINDER UNLESS OTHERWISE NOTED	30 MPa

2. CLEAR COVER TO REINFORCING STEEL:

•FOOTINGS		100 ± 25
•BOTTOM OF ABUTMENTS		100 ± 25
•DECK	TOP	70 ± 20
	BOTTOM	40 ± 10
•REMAINDER UNLESS OTHERWISE NOTED		70 ± 20

3. REINFORCING STEEL:

- REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED.
- BAR MARKS WITH PREFIX 'C' DENOTE EPOXY COATED BARS.
- UNLESS SHOWN OTHERWISE, TENSION LAP LENGTHS NOT INDICATED ON THE CONTRACT DRAWINGS SHALL BE CLASS B.
- HOOKS AND BENDS FOR REINFORCING STEEL SHALL BE DETAILED ACCORDING TO OHBDC-91. UNLESS SHOWN OTHERWISE, THE FOLLOWING SHALL APPLY:
  a) STANDARD HOOKS WITH MINIMUM BEND DIAMETERS SHALL BE USED FOR STIRRUPS & TIES, ACCORDING TO CLAUSE 8-14.1.
  b) OTHER BARS SHALL HAVE STANDARD HOOKS WITH BEND DIAMETERS ACCORDING TO CLAUSE C8-14.1.

4. DESIGN CODE AND LOADING OHBDC 3rd EDITION. (RAILING EXCLUDED

5. ALL WORK SHALL CONFORM WITH ONTARIO PROVINCIAL STANDARD SPECIFICATIONS AND THE SPECIAL PROVISION OF THE CONTRACT.

CONSTRUCTION NOTES:

- THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
- NO BACKFILL SHALL BE PLACED BEHIND ABUTMENTS UNTIL DECK CONCRETE HAS REACHED 25MPa STRENGTH.
- BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 0.5m.
- TEMPORARY ROADWAY PROTECTION SHALL BE DESIGNED TO PERFORMANCE LEVEL 2 CRITERIA.
- ALL ELEVATIONS ARE TO GEODETIC DATUM.

APPLICABLE STANDARD DRAWINGS

OPSD-3501.00	GRANULAR BACKFILL REQUIREMENTS-INTEGRAL ABUTMENTS
OPSD-3906.02	BRIDGE DECK WATERPROOFING
OPSD-3906.03	BRIDGE DECK WATERPROOFING DETAILS
OPSD-4010.00	GUIDE RAIL AND CHANNEL ANCHORAGE

LIST OF ABBREVIATIONS

W.P. - DENOTES WORKING POINT
N.T.S. - DENOTES NOT TO SCALE
T/P - DENOTES TOP OF PAVEMENT
T/F - DENOTES TOP OF FOOTING
S/W - DENOTES SIDEWALK

NO.	BY	DATE	REVISION	CONS. CHECKED	TOWN APPR'D
1	SDL	01/03	TRCA APPROVAL		

APPROVED FOR CONSTRUCTION

THIS APPROVAL CONSTITUTES A GENERAL REVIEW AND DOES NOT CERTIFY DIMENSIONAL ACCURACY.

THIS APPROVAL IS SUBJECT TO THE FURTHER CERTIFICATION OF THE "AS CONSTRUCTED" WORKS BY A REGISTERED PROFESSIONAL ENGINEER OF THE PROVINCE OF ONTARIO.

DATE: ____________ APPROVED BY: ________________

H. MUNTZ, P.ENG.
Town Engineer

BENCH MARK:

S.D. LEITCH
JUNE 16/08

L. YU
JUNE 16/08

PROJECT NAME
ALBION - VAUGHAN ROAD
HUMBER RIVER BRIDGE REPLACEMENT

CONSULTANT
MRC
McCORMICK RANKIN
CORPORATION

REGION OF PEEL

THE CORPORATION OF THE
TOWN OF CALEDON

GENERAL ARRANGEMENT

SCALE: AS SHOWN		CONTRACT No. 02-07
DESIGNED BY: S.D.L.	DRAWN BY: A.M.	DRAWNING No. S1
CHECKED BY: L.Y.	DATE: MARCH, 2002	

LIST OF DRAWINGS

S1. GENERAL ARRANGEMENT
S2. CONSTRUCTION STAGING
S3. FOOTING LAYOUT AND PILE DATA
S4. NORTH ABUTMENT LAYOUT AND REINFORCING
S5. SOUTH ABUTMENT LAYOUT AND REINFORCING
S6. WINGWALL DETAILS
S7. PIERS DETAILS
S8. BOX GIRDER DETAILS
S9. DECK LAYOUT AND REINFORCING
S10. ALUMINUM RAIL DETAILS
S11. 6000mm APPROACH SLAB
S12. PILE DRIVING - STEAM AND DIESEL HAMMER
S13. ELECTRICAL EMBEDDED WORK

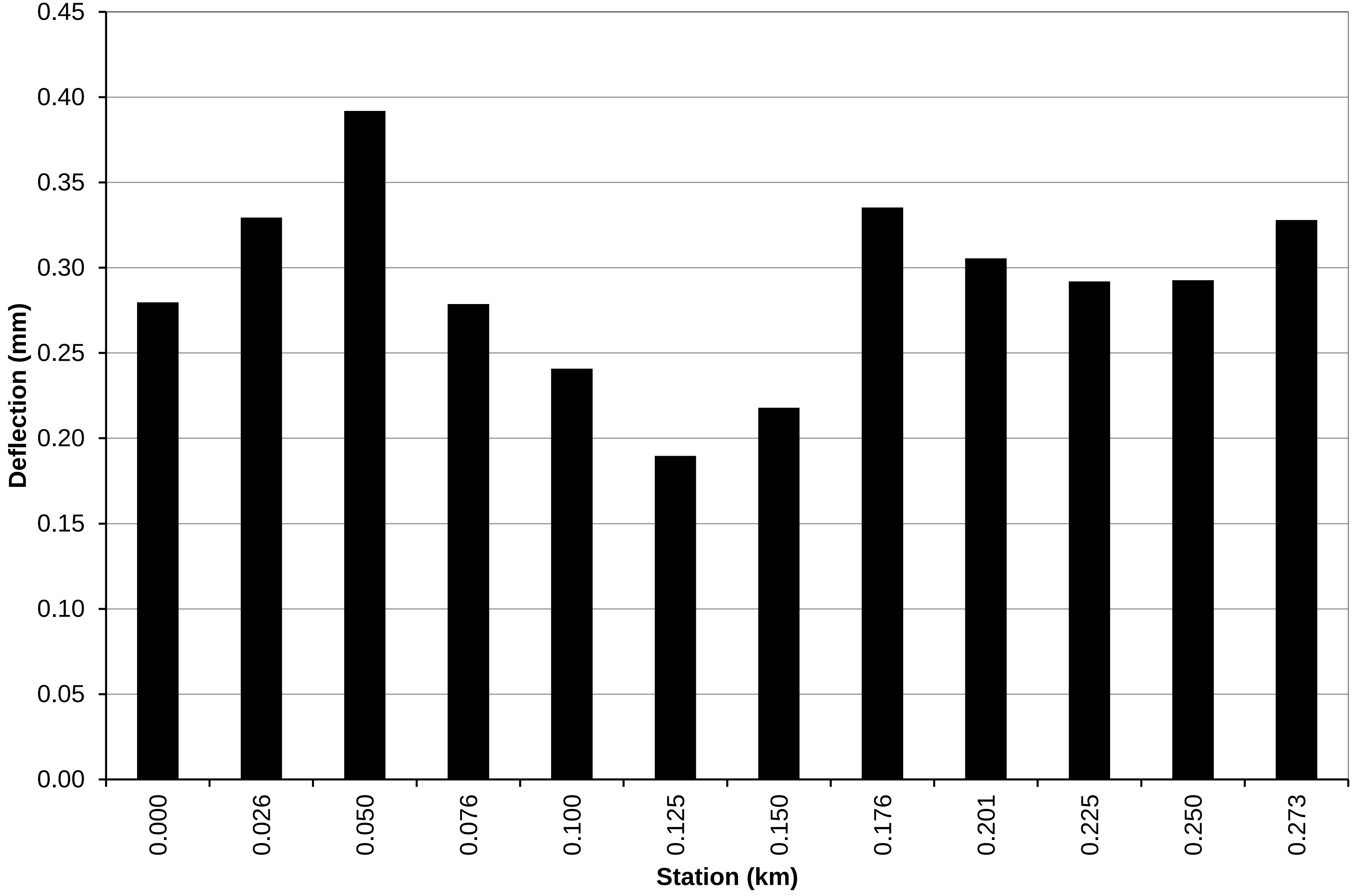
APPENDIX E

Falling Weight Deflectometer Data and Analysis

TABLE 1 - KING STREET EAST EASTBOUND LANE
SUMMARY OF FWD DEFLECTION RESULTS

STATION	NORMALIZED DEFLECTION	PAVEMENT SURFACE MODULUS
(km)	**(mm)**	**(MPa)**
0.000	0.28	525
0.026	0.33	446
0.050	0.39	375
0.076	0.28	527
0.100	0.24	610
0.125	0.19	775
0.150	0.22	674
0.176	0.34	438
0.201	0.31	481
0.225	0.29	503
0.250	0.29	502
0.273	0.33	448
Mean	0.29	526
Standard Deviation	0.05	107
Mean + 2SD	0.40	-
Static Deflection	0.63	-
Spring Deflection	0.95	-
Maximum Allowable Deflection	0.66	-

PAVEMENT FWD DEFLECTION
FIGURE 1-A - KING STREET EAST EASTBOUND LANE



PAVEMENT SURFACE MODULUS
FIGURE 1-B - KING STREET EAST EASTBOUND LANE

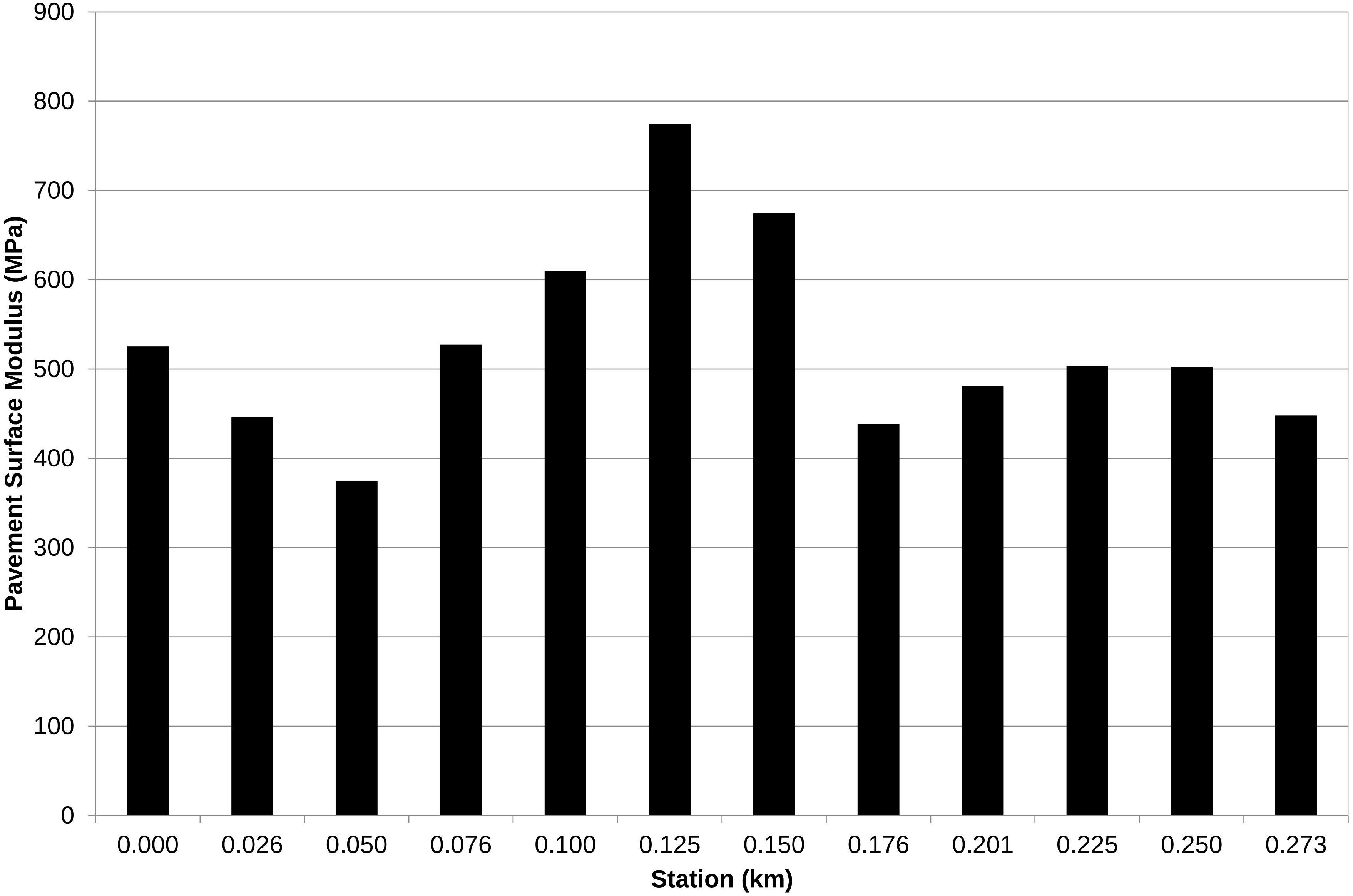
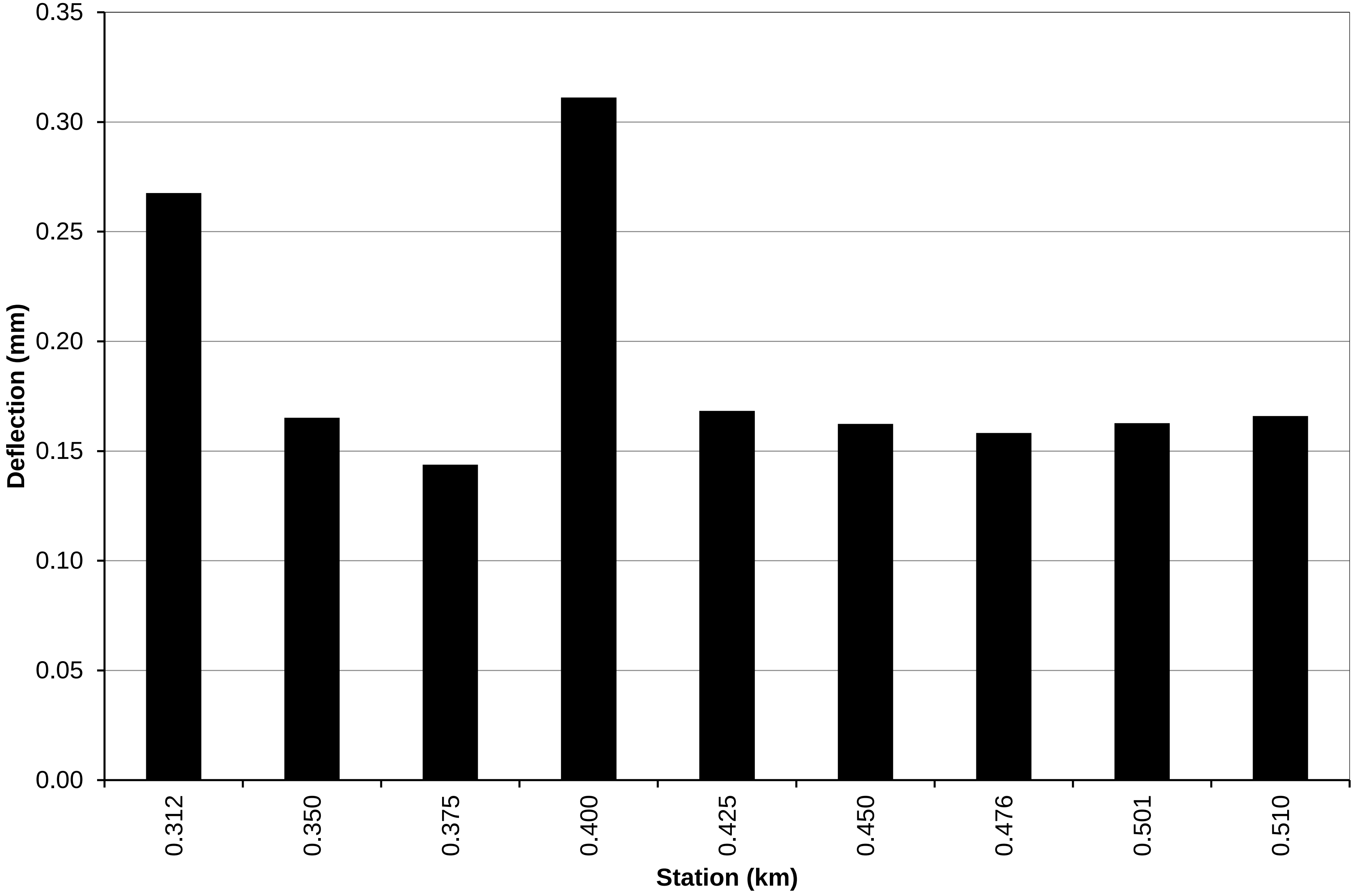


TABLE 2 - KING ROAD EASTBOUND LANE
SUMMARY OF FWD DEFLECTION RESULTS

STATION	NORMALIZED DEFLECTION	PAVEMENT SURFACE MODULUS
(km)	**(mm)**	**(MPa)**
0.312	0.27	549
0.350	0.17	890
0.375	0.14	1,022
0.400	0.31	472
0.425	0.17	873
0.450	0.16	905
0.476	0.16	929
0.501	0.16	903
0.510	0.17	886
Mean	0.19	826
Standard Deviation	0.05	174
Mean + 2SD	0.30	-
Static Deflection	0.48	-
Spring Deflection	0.72	-
Maximum Allowable Deflection	0.66	-

PAVEMENT FWD DEFLECTION
FIGURE 2-A - KING ROAD EASTBOUND LANE



PAVEMENT SURFACE MODULUS
FIGURE 2-B - KING ROAD EASTBOUND LANE

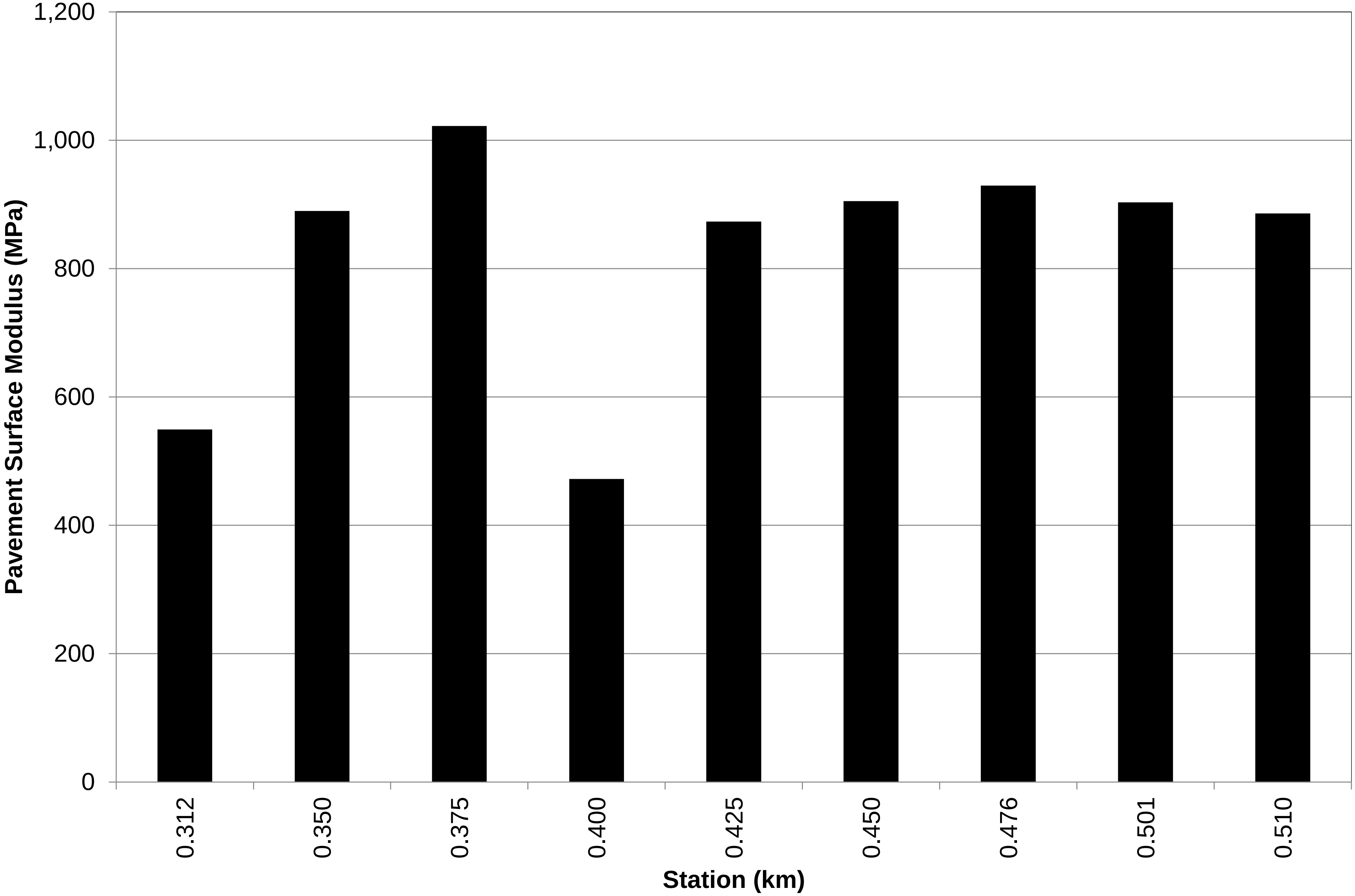
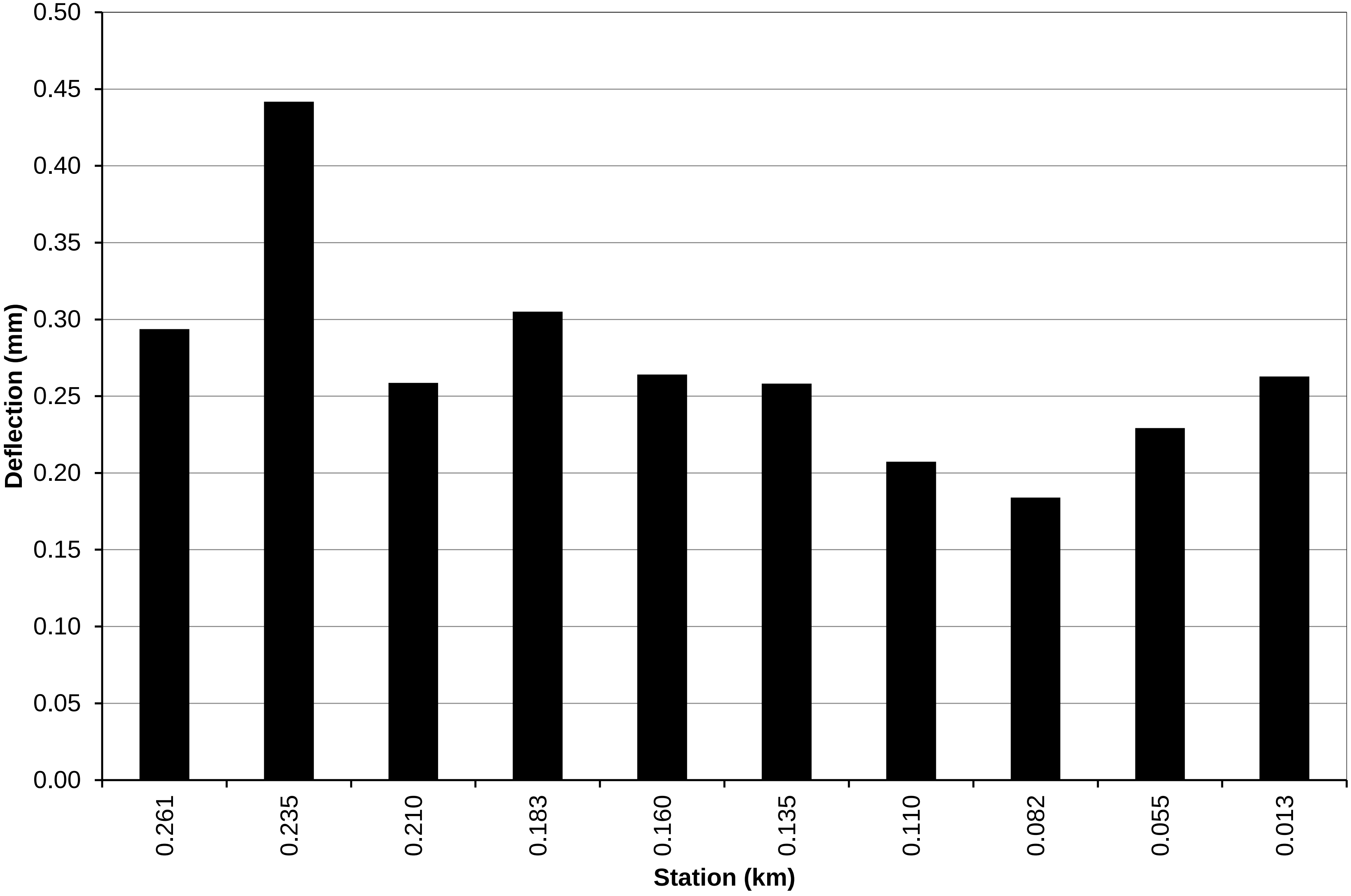


TABLE 3 - KING STREET EAST WESTBOUND LANE
SUMMARY OF FWD DEFLECTION RESULTS

STATION	NORMALIZED DEFLECTION	PAVEMENT SURFACE MODULUS
(km)	**(mm)**	**(MPa)**
0.261	0.29	500
0.235	0.44	333
0.210	0.26	568
0.183	0.31	482
0.160	0.26	557
0.135	0.26	569
0.110	0.21	709
0.082	0.18	799
0.055	0.23	641
0.013	0.26	559
Mean	0.27	572
Standard Deviation	0.07	121
Mean + 2SD	0.40	-
Static Deflection	0.65	-
Spring Deflection	0.97	-
Maximum Allowable Deflection	0.51	-

PAVEMENT FWD DEFLECTION
FIGURE 3-A - KING STREET EAST WESTBOUND LANE



PAVEMENT SURFACE MODULUS
FIGURE 3-B - KING STREET EAST WESTBOUND LANE

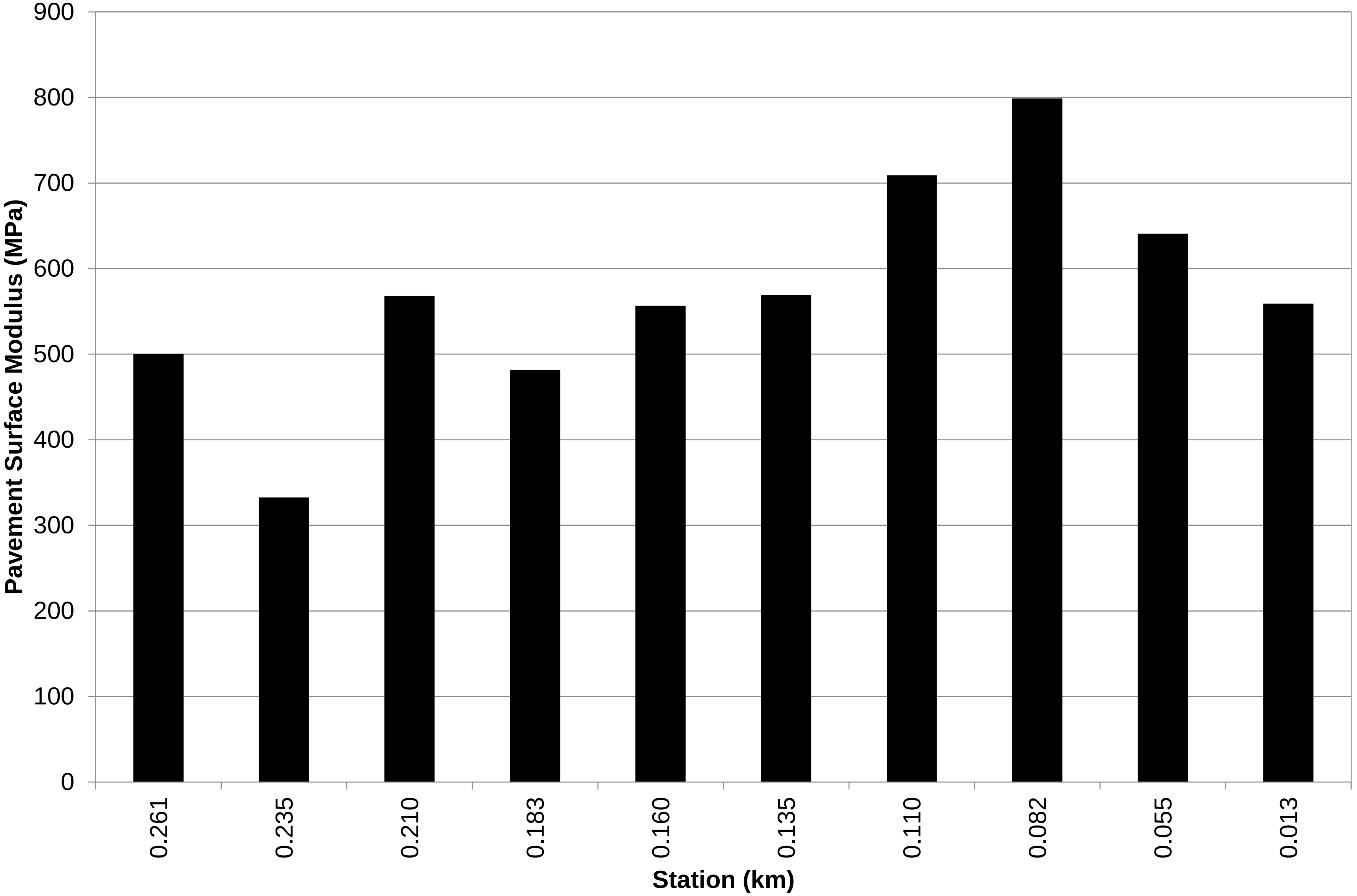
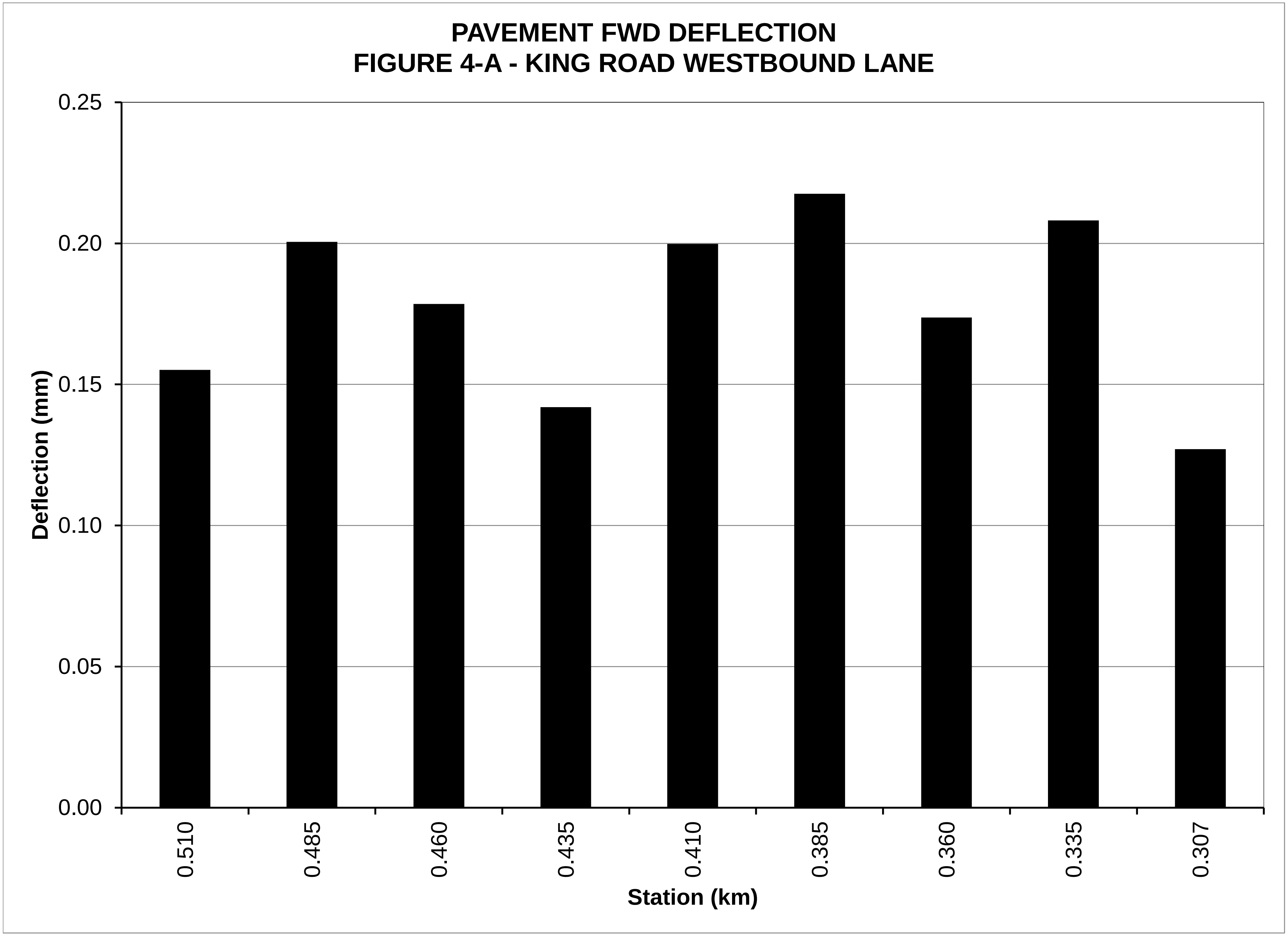


TABLE 4 - KING ROAD WESTBOUND LANE
SUMMARY OF FWD DEFLECTION RESULTS

STATION	NORMALIZED DEFLECTION	PAVEMENT SURFACE MODULUS
(km)	(mm)	(MPa)
0.510	0.16	947
0.485	0.20	733
0.460	0.18	823
0.435	0.14	1,035
0.410	0.20	735
0.385	0.22	675
0.360	0.17	846
0.335	0.21	706
0.307	0.13	1,157
Mean	0.18	851
Standard Deviation	0.03	155
Mean + 2SD	0.24	-
Static Deflection	0.38	-
Spring Deflection	0.57	-
Maximum Allowable Deflection	0.51	-


PAVEMENT FWD DEFLECTION
FIGURE 4-A - KING ROAD WESTBOUND LANE
Deflection (mm)
0.25
0.20
0.15
0.10
0.05
0.00
0.510
0.485
0.460
0.435
0.410
0.385
0.360
0.335
0.307
Station (km)

PAVEMENT SURFACE MODULUS
FIGURE 4-B - KING ROAD WESTBOUND LANE

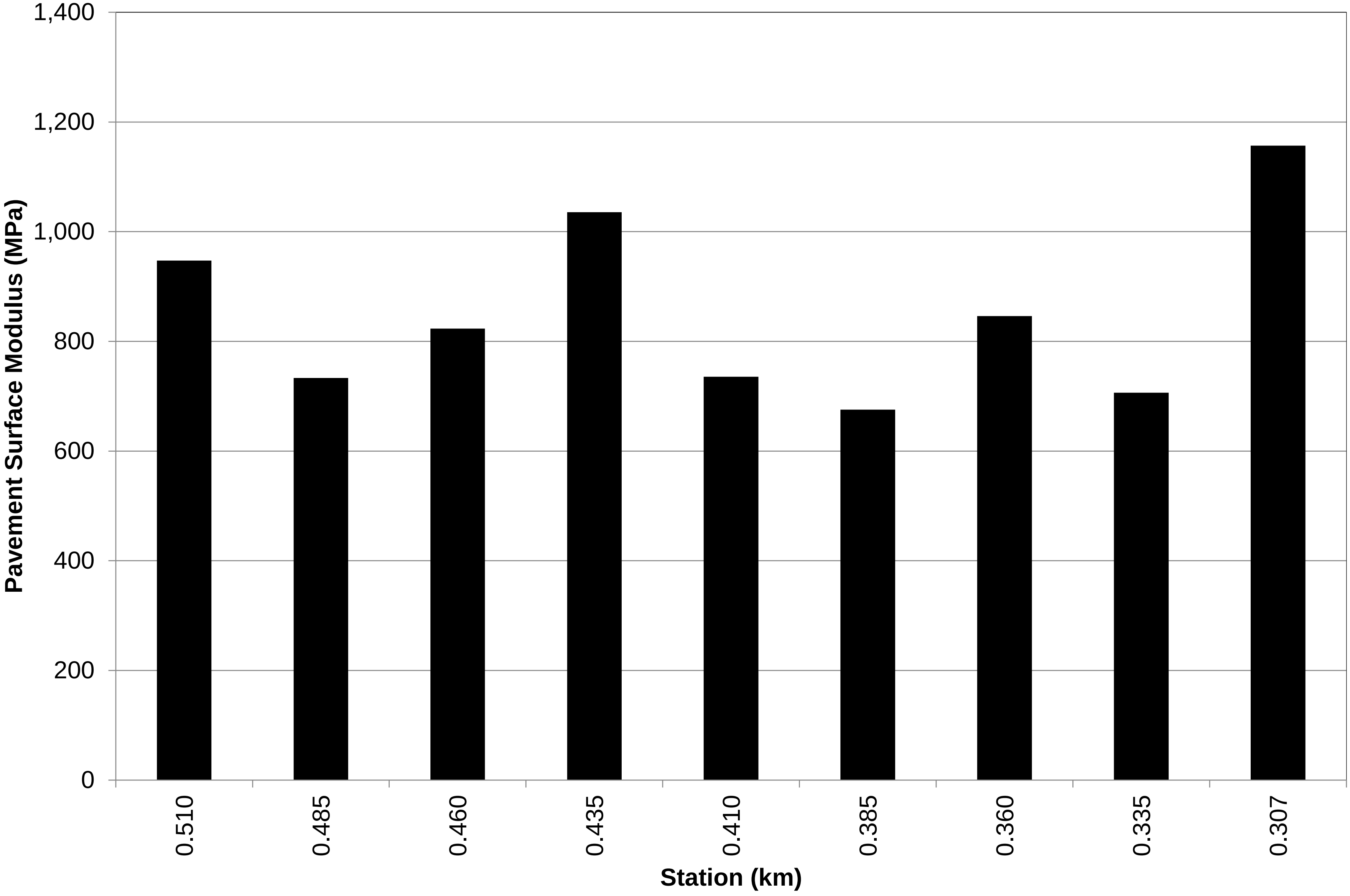
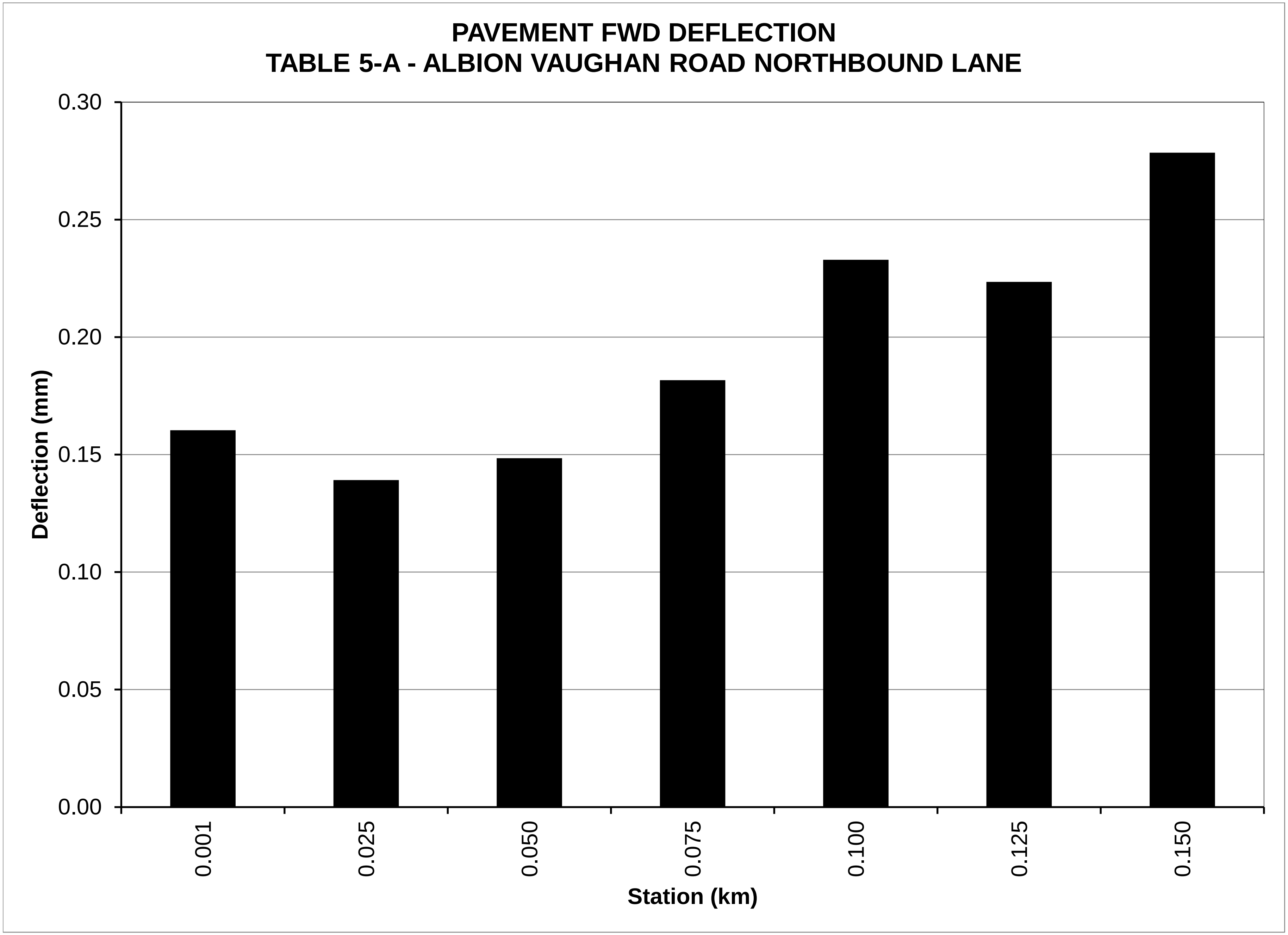
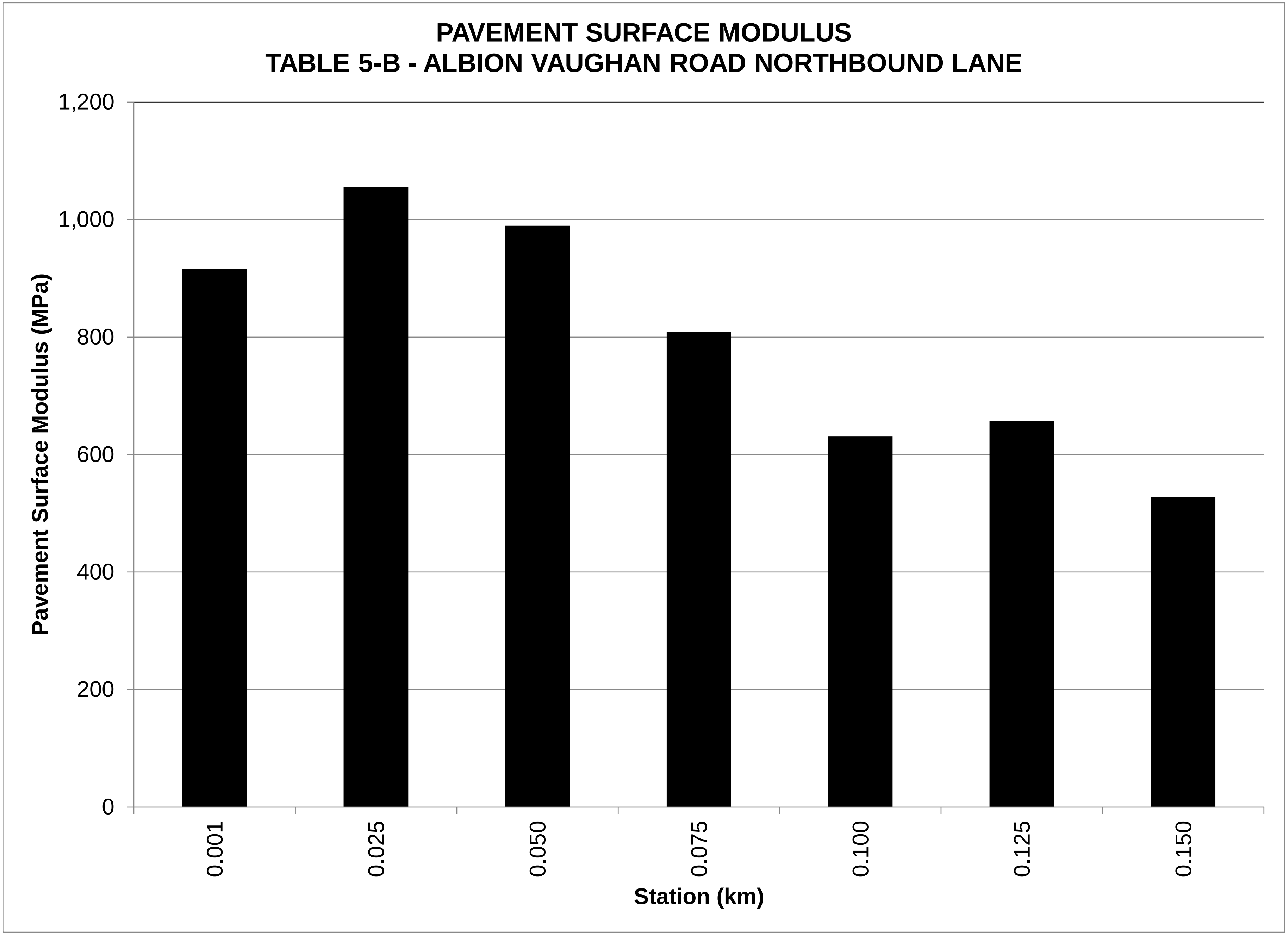


TABLE 5 - ALBION VAUGHAN ROAD NORTHBOUND LANE
SUMMARY OF FWD DEFLECTION RESULTS

STATION	NORMALIZED DEFLECTION	PAVEMENT SURFACE MODULUS
(km)	(mm)	(MPa)
0.001	0.16	916
0.025	0.14	1,056
0.050	0.15	990
0.075	0.18	809
0.100	0.23	631
0.125	0.22	658
0.150	0.28	528
Mean	0.19	798
Standard Deviation	0.05	184
Mean + 2SD	0.29	-
Static Deflection	0.46	-
Spring Deflection	0.70	-
Maximum Allowable Deflection	0.58	-



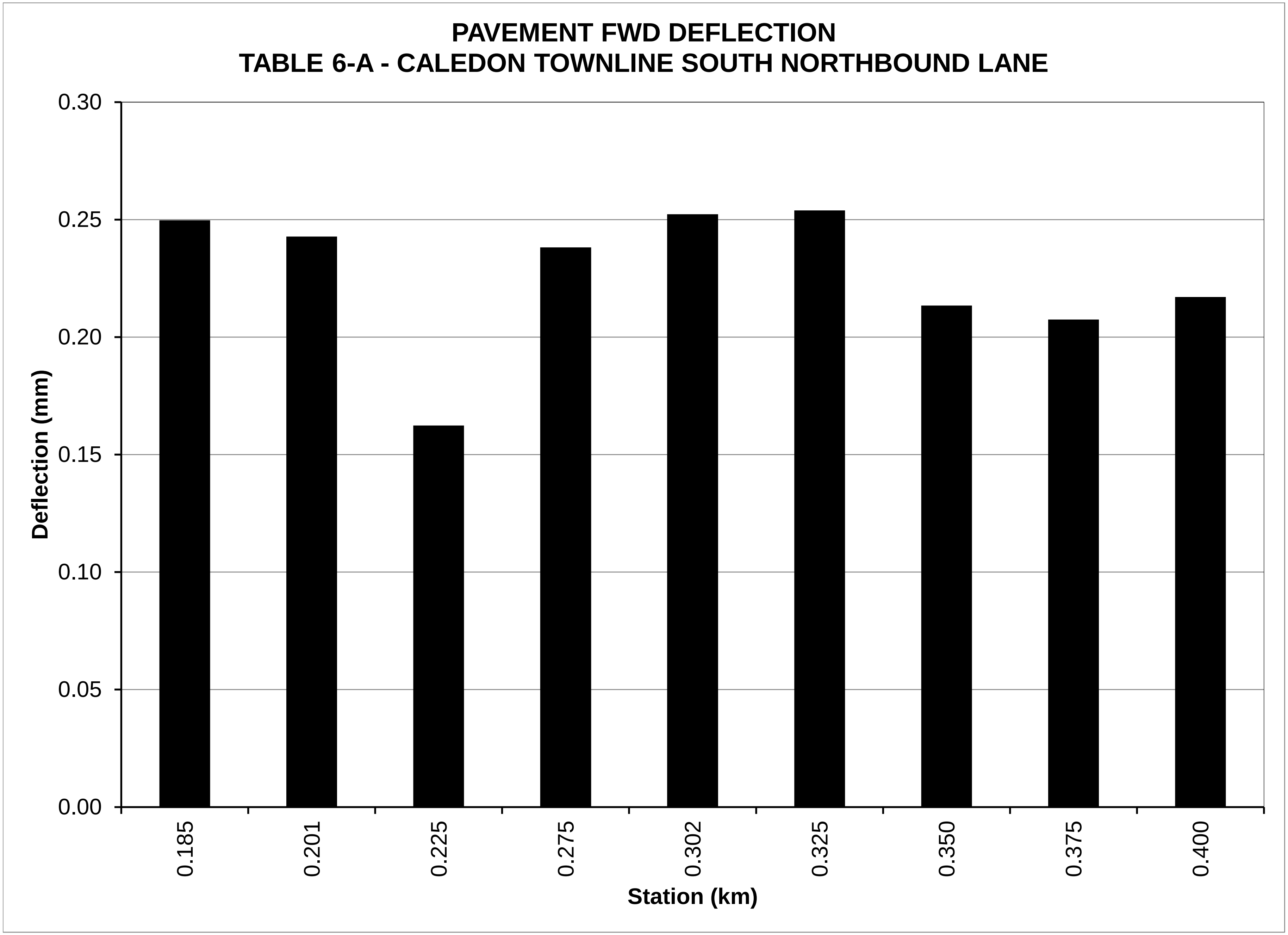
PAVEMENT FWD DEFLECTION
TABLE 5-A - ALBION VAUGHAN ROAD NORTHBOUND LANE



PAVEMENT SURFACE MODULUS
TABLE 5-B - ALBION VAUGHAN ROAD NORTHBOUND LANE

TABLE 6 - CALEDON TOWNLINE SOUTH NORTHBOUND LANE
SUMMARY OF FWD DEFLECTION RESULTS

STATION	NORMALIZED DEFLECTION	PAVEMENT SURFACE MODULUS
(km)	**(mm)**	**(MPa)**
0.185	0.25	589
0.201	0.24	605
0.225	0.16	905
0.275	0.24	617
0.302	0.25	583
0.325	0.25	579
0.350	0.21	689
0.375	0.21	708
0.400	0.22	677
Mean	0.23	661
Standard Deviation	0.03	98
Mean + 2SD	0.28	-
Static Deflection	0.45	-
Spring Deflection	0.68	-
Maximum Allowable Deflection	0.58	-


PAVEMENT FWD DEFLECTION
TABLE 6-A - CALEDON TOWNLINE SOUTH NORTHBOUND LANE
0.30
0.25
0.20
0.15
0.10
0.05
0.00
Deflection (mm)
0.185
0.201
0.225
0.275
0.302
0.325
0.350
0.375
0.400
Station (km)

PAVEMENT SURFACE MODULUS
TABLE 6-B - CALEDON TOWNLINE SOUTH NORTHBOUND LANE

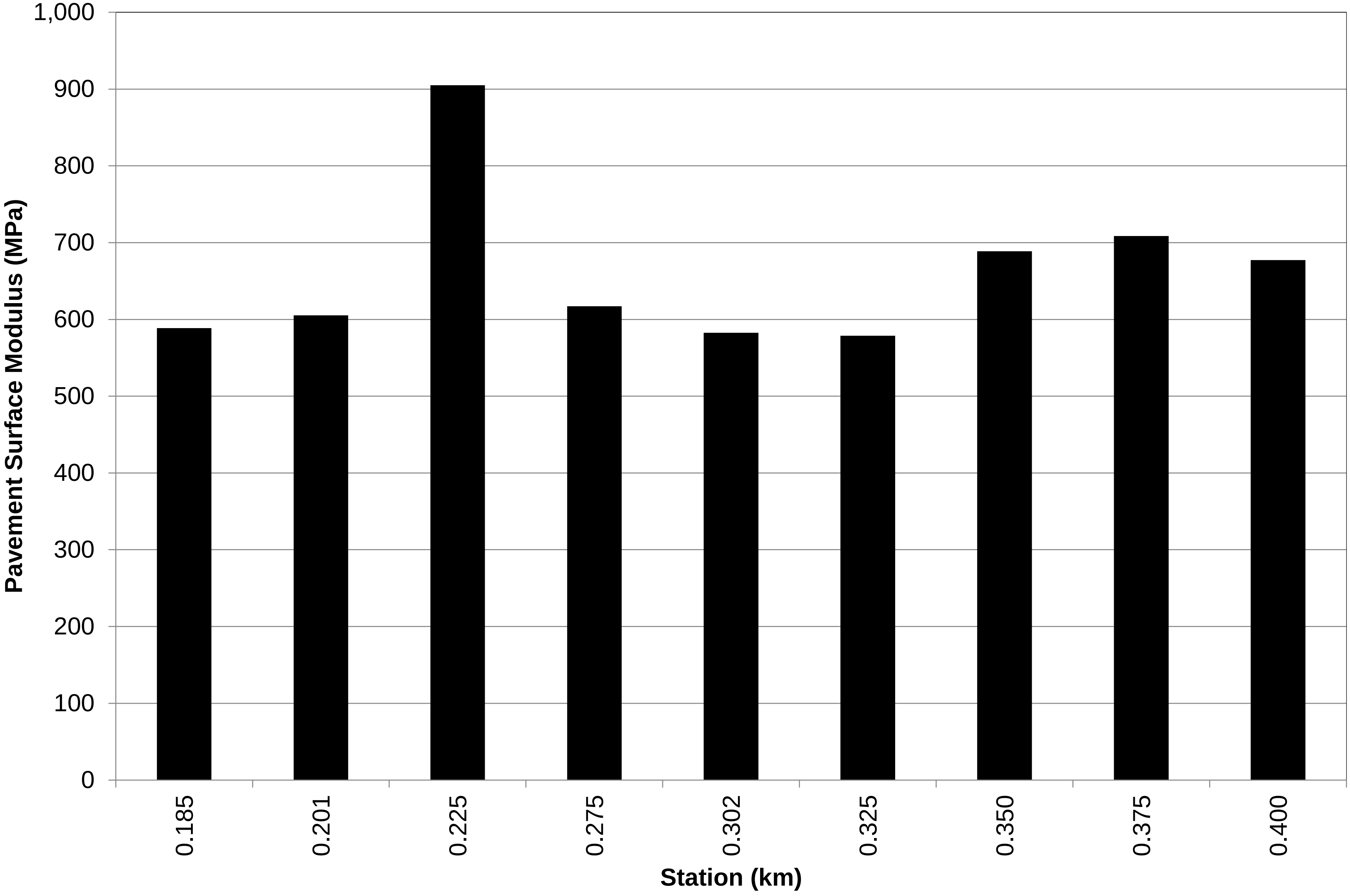
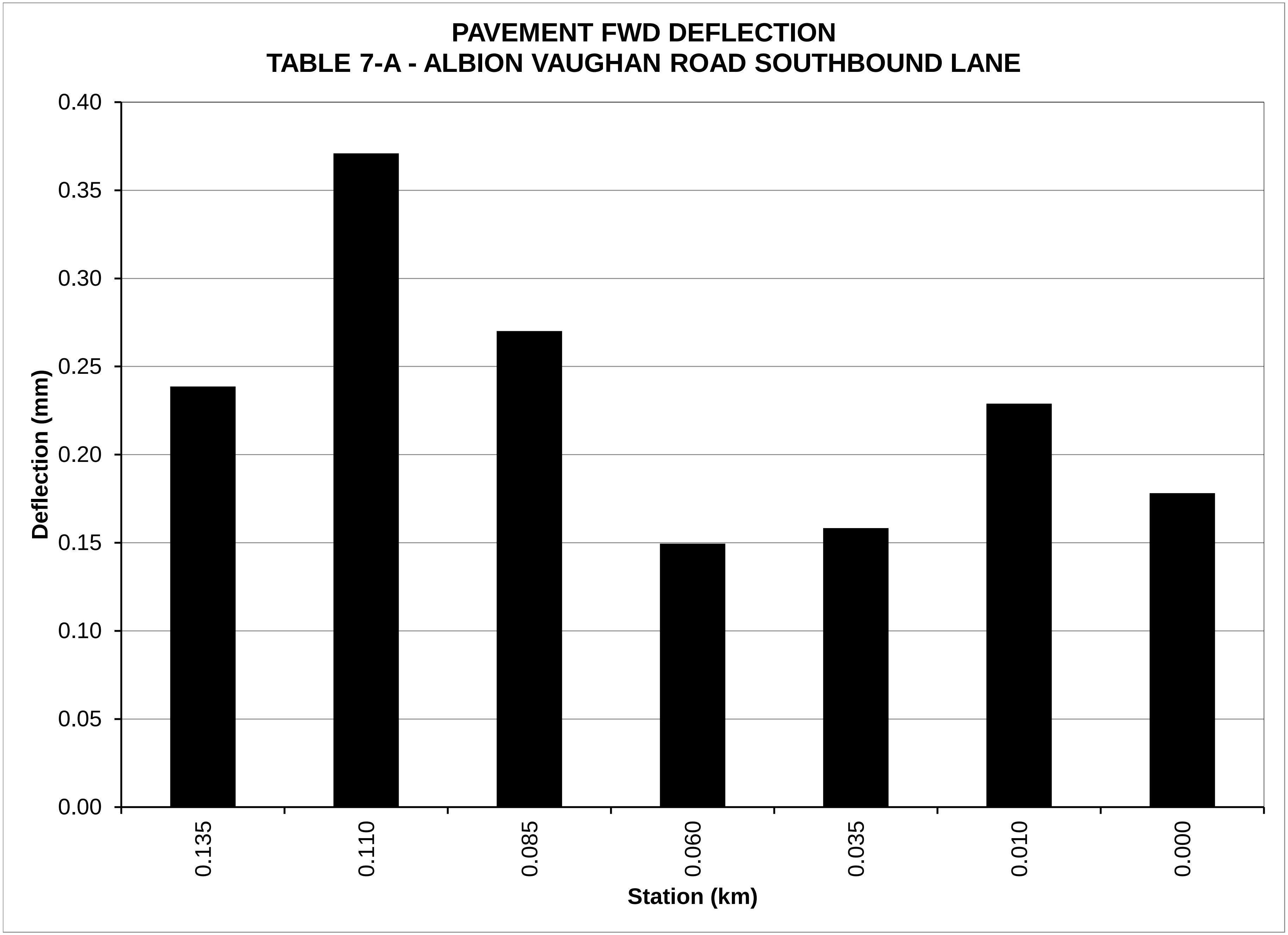


TABLE 7 - ALBION VAUGHAN ROAD SOUTHBOUND LANE
SUMMARY OF FWD DEFLECTION RESULTS

STATION	NORMALIZED DEFLECTION	PAVEMENT SURFACE MODULUS
(km)	(mm)	(MPa)
0.135	0.24	616
0.110	0.37	396
0.085	0.27	544
0.060	0.15	983
0.035	0.16	929
0.010	0.23	642
0.000	0.18	825
Mean	0.23	705
Standard Deviation	0.07	198
Mean + 2SD	0.37	-
Static Deflection	0.59	-
Spring Deflection	0.89	-
Maximum Allowable Deflection	0.87	-


PAVEMENT FWD DEFLECTION
TABLE 7-A - ALBION VAUGHAN ROAD SOUTHBOUND LANE
Deflection (mm)
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0.135
0.110
0.085
0.060
0.035
0.010
0.000
Station (km)

PAVEMENT SURFACE MODULUS
TABLE 7-B - ALBION VAUGHAN ROAD SOUTHBOUND LANE

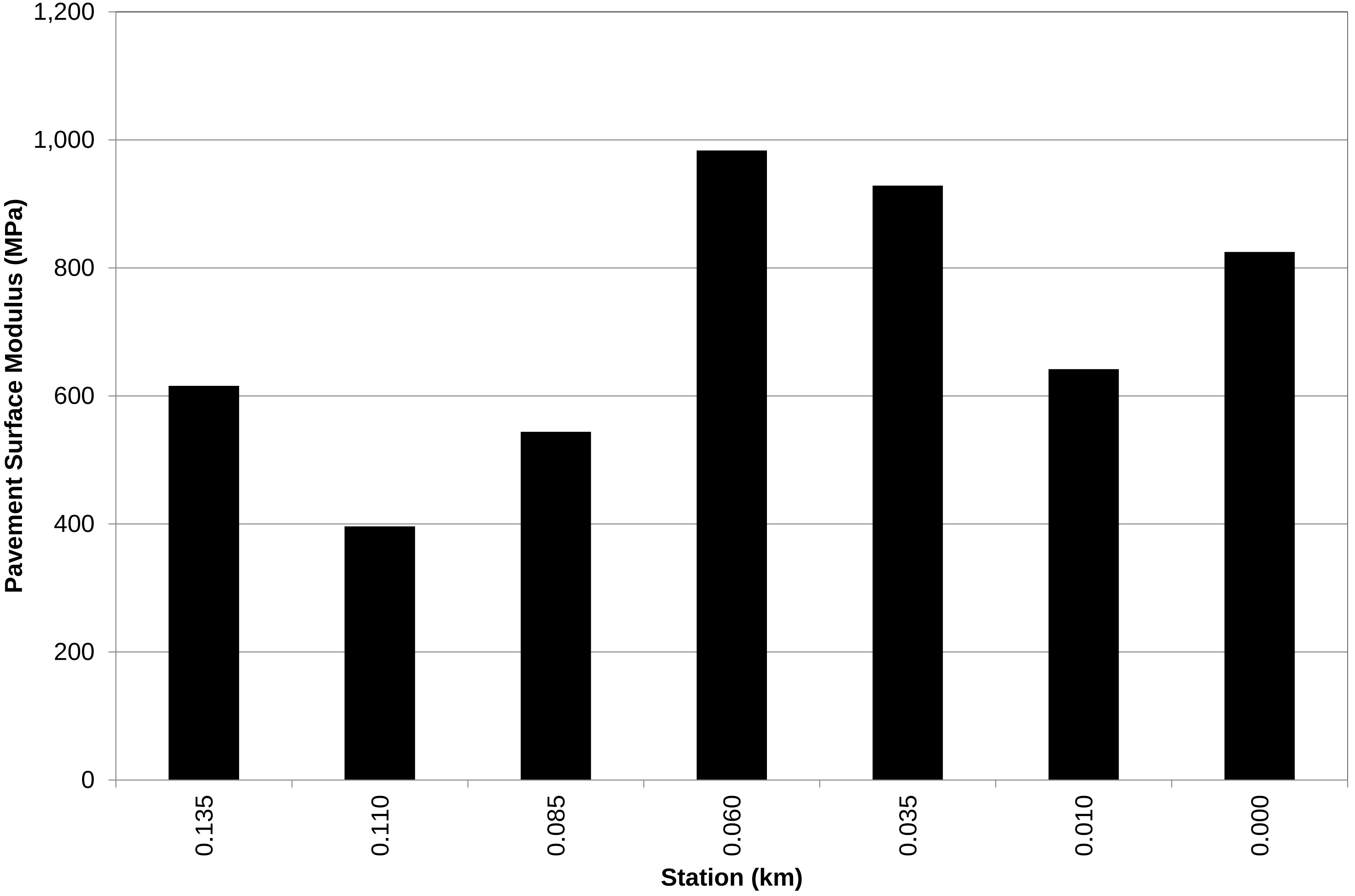
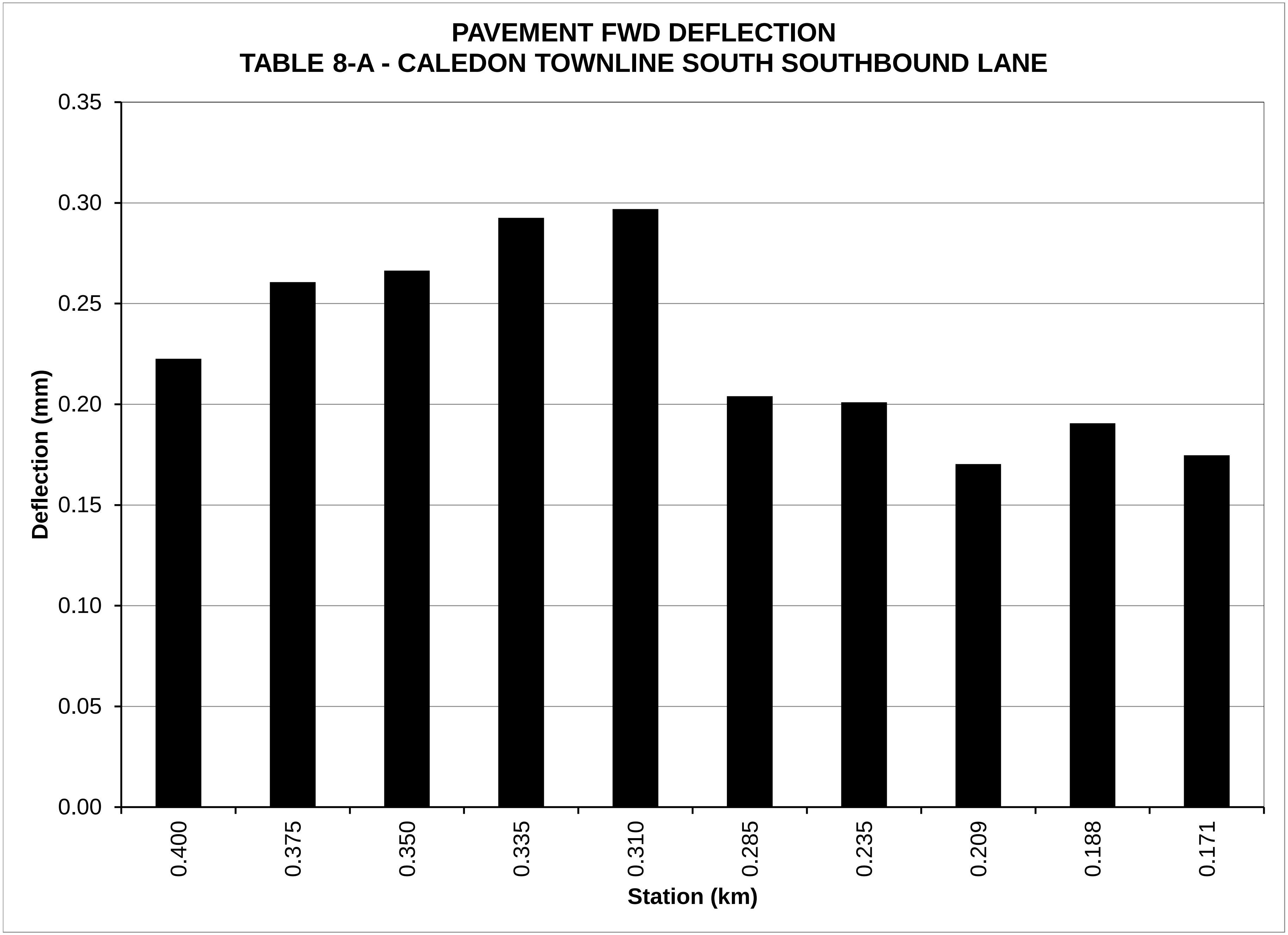
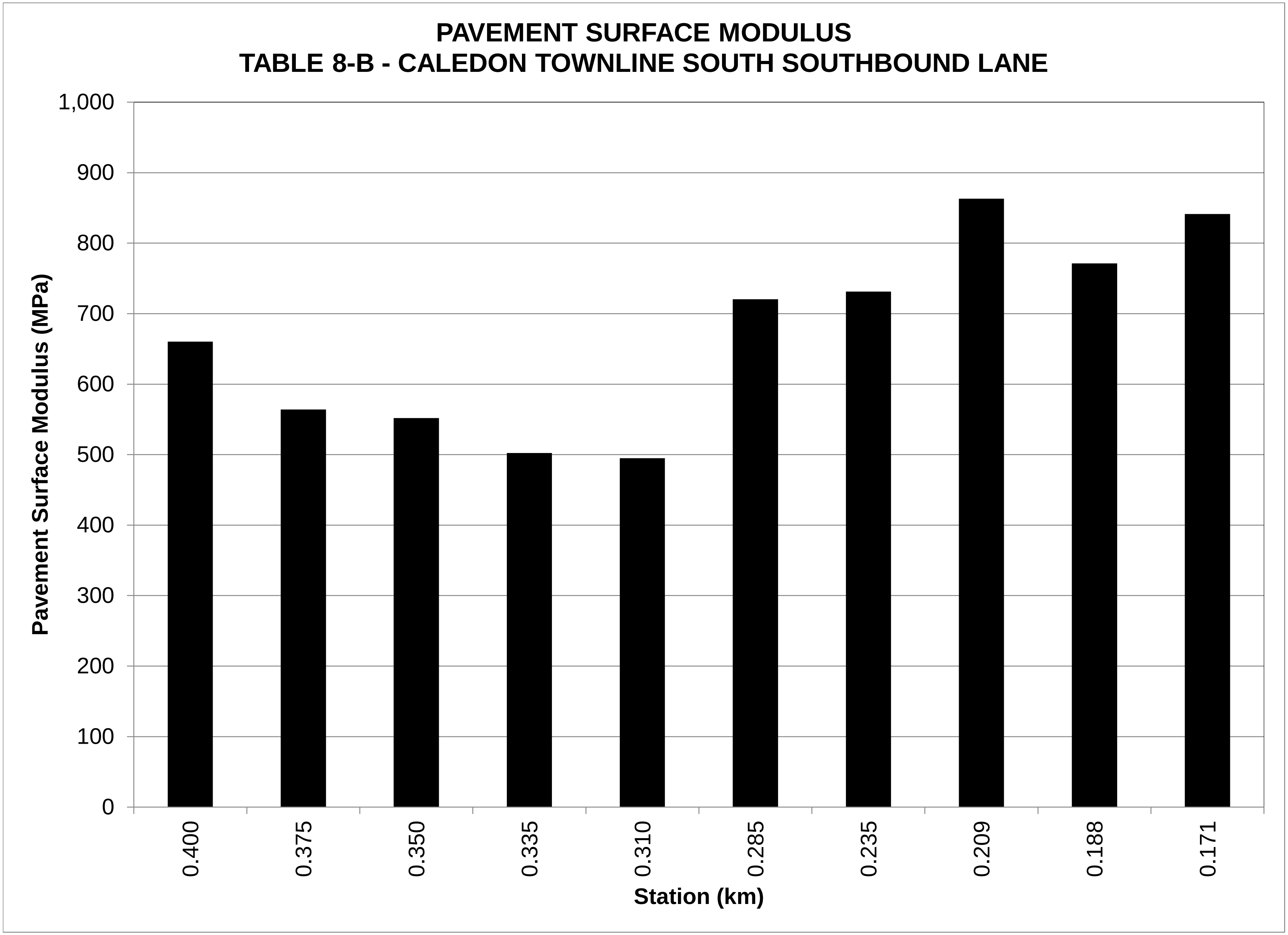


TABLE 8 - CALEDON TOWNLINE SOUTH SOUTHBOUND LANE
SUMMARY OF FWD DEFLECTION RESULTS

STATION	NORMALIZED DEFLECTION	PAVEMENT SURFACE MODULUS
(km)	**(mm)**	**(MPa)**
0.400	0.22	660
0.375	0.26	564
0.350	0.27	552
0.335	0.29	502
0.310	0.30	495
0.285	0.20	720
0.235	0.20	731
0.209	0.17	863
0.188	0.19	771
0.171	0.17	841
Mean	0.23	670
Standard Deviation	0.05	129
Mean + 2SD	0.32	-
Static Deflection	0.51	-
Spring Deflection	0.76	-
Maximum Allowable Deflection	0.87	-



PAVEMENT FWD DEFLECTION
TABLE 8-A - CALEDON TOWNLINE SOUTH SOUTHBOUND LANE



PAVEMENT SURFACE MODULUS
TABLE 8-B - CALEDON TOWNLINE SOUTH SOUTHBOUND LANE

TABLE 9 - KING STREET EAST EASTBOUND LANE BACKCALCULATED PAVEMENT LAYER MODULI

Station	Asphalt Modulus (MPa)	Granular Layer Modulus (MPa)	Subgrade Modulus (MPa)
0.000	5,218	294	73
0.026	5,567	204	80
0.050	5,931	110	70
0.076	7,180	209	83
0.100	5,951	326	76
0.125	12,033	348	86
0.150	6,132	402	85
0.176	2,653	248	79
0.201	2,148	354	79
0.225	3,415	321	94
0.250	3,682	309	84
0.273	3,385	233	90
Average	5,275	280	82
Standard Deviation	2,650	81	7
30th Percentile	3,495	237	79

TABLE 10 - KING ROAD WEST EASTBOUND LANE
BACKCALCULATED PAVEMENT LAYER MODULI

Station	Asphalt Modulus (MPa)	Granular Layer Modulus (MPa)	Subgrade Modulus (MPa)
0.312	9,313	227	84
0.350	9,067	591	106
0.375	7,620	626	105
0.400	7,020	174	70
0.425	7,635	515	94
0.450	9,292	628	107
0.476	4,417	770	107
0.501	5,808	606	88
0.510	6,665	517	94
Average	7,426	517	95
Standard Deviation	1,667	195	13
30th Percentile	6,807	516	90

TABLE 11 - KING STREET EAST WESTBOUND LANE
BACKCALCULATED PAVEMENT LAYER MODULI

Station	Asphalt Modulus (MPa)	Granular Layer Modulus (MPa)	Subgrade Modulus (MPa)
0.013	11,246	174	68
0.055	11,592	234	72
0.082	12,923	375	81
0.110	13,553	288	68
0.135	9,049	228	69
0.160	8,335	232	75
0.183	5,971	227	60
0.210	5,491	338	69
0.235	3,654	134	61
0.261	4,913	269	73
Average	8,672	250	70
Standard Deviation	3,559	71	6
30th Percentile	5,827	228	68

TABLE 12 - KING ROAD WEST WESTBOUND LANE
BACKCALCULATED PAVEMENT LAYER MODULI

Station	Asphalt Modulus (MPa)	Granular Layer Modulus (MPa)	Subgrade Modulus (MPa)
0.307	35,224	329	176
0.335	5,560	527	86
0.360	8,761	577	77
0.385	12,124	249	70
0.410	11,820	366	74
0.435	8,732	845	78
0.460	5,398	651	86
0.485	4,945	625	73
0.510	6,099	785	81
Average	7,930	551	89
Standard Deviation	2,887	204	33
30th Percentile	5,775	430	76

Excluded from analysis

TABLE 13 - ALBION - VAUGHAN NORTHBOUND
BACKCALCULATED PAVEMENT LAYER MODULI

Station	Asphalt Modulus (MPa)	Granular Layer Modulus (MPa)	Subgrade Modulus (MPa)
0.001	10,121	493	108
0.025	10,760	564	116
0.050	8,776	560	119
0.075	4,444	567	135
0.100	4,005	328	99
0.125	5,312	342	102
0.150	3,859	269	94
Average	6,754	446	111
Standard Deviation	3,023	129	14
30th Percentile	4,356	340	101

TABLE 14 - CALEDON TOWNLINE NORTHBOUND BACKCALCULATED PAVEMENT LAYER MODULI

Station	Asphalt Modulus (MPa)	Granular Layer Modulus (MPa)	Subgrade Modulus (MPa)
0.185	3,654	360	74
0.201	4,401	338	106
0.225	4,623	637	97
0.275	6,461	227	107
0.302	5,285	239	92
0.325	4,773	223	85
0.350	4,155	322	76
0.375	2,990	504	110
0.400	3,145	448	85
Average	4,387	366	92
Standard Deviation	1,084	140	13
30th Percentile	3,854	272	85

TABLE 15 - ALBION - VAUGHAN SOUTHBOUND
BACKCALCULATED PAVEMENT LAYER MODULI

Station	Asphalt Modulus (MPa)	Granular Layer Modulus (MPa)	Subgrade Modulus (MPa)
0.000	5,927	477	95
0.010	2,781	404	79
0.035	15,014	342	114
0.060	12,725	513	117
0.085	8,236	166	92
0.110	5,618	114	66
0.135	8,768	207	106
Average	8,438	318	96
Standard Deviation	4,243	157	18
30th Percentile	5,865	199	89

TABLE 16 - CALEDON TOWNLINE SOUTHBOUND BACKCALCULATED PAVEMENT LAYER MODULI

Station	Asphalt Modulus (MPa)	Granular Layer Modulus (MPa)	Subgrade Modulus (MPa)
0.171	9,789	413	96
0.188	3,798	487	118
0.209	4,416	606	127
0.235	7,117	366	116
0.285	3,470	500	94
0.310	2,576	307	85
0.335	4,386	198	83
0.350	2,790	332	79
0.375	2,535	381	74
0.400	2,552	508	92
Average	4,343	410	96
Standard Deviation	2,367	118	18
30th Percentile	2,726	356	84

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