METROLINX

Tunnels and Underground Structures Interim Standard

MX-TUS-STD-001

Revision 0
Date: November 2022

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Preface

This is the first edition of the *Metrolinx Tunnels and Underground Structures Interim Standard*. The standard reflects current standards and alignment with current technologies existing on the market today. This document ensures that the requirements communicated to the Metrolinx vendor community are current, and forward looking, to allow construction projects to have recent technology when they are commissioned.

The following standard has been prepared using the process detailed below:

1. A search was conducted to identify representative sources of information.
2. These sources included; local sources - Toronto Transit Commission (TTC), Metrolinx and Infrastructure Ontario (IO); output specifications from recent Metrolinx Subway Program projects (Ontario Line, Scarborough Subway Extension, and Eglinton Crosstown West Extension); construction records from transit projects in Toronto (Sheppard Subway, Toronto-York Spadina Subway Extension, and Eglinton Crosstown Light Rail Transit); transit agencies within North America and limited International transit agencies; American Public Transit Association (APTA), American Association of State Highway and Transportation Officials (AASHTO), British Tunnelling Association (BTS), Federal Highway Administration (FHWA),and the International Tunnelling Association (ITA).
3. For each section of the standards, comparisons were made of the listed sources and, in particular, the ongoing contracts and current procurements. A compilation was prepared, including prospective inclusions from the above sources of information. The compilation included the suggested basis for each of the sections of the standards.
4. Draft sections were prepared for the standards.
5. Frequent progress presentations were made to the Metrolinx Tunnels and Underground Structures - E & AM Team. These presentations were followed by progress submittals of the standards. The Metrolinx team provided formal comments on the submitted documents. Responses to these comments were provided with adjustments to the standards, as required.
6. An independent peer review was undertaken, with comments provided. The comments were addressed and revisions made, as appropriate.
7. A presentation was made to representatives from the Technical Advisors (TA) for the Metrolinx Subway Program, and the Metrolinx Tunnels and Underground Structures E & AM Team. The attendees undertook further review, with comments provided by the Metrolinx Team, and by representatives from the Ontario Line and Yonge North Subway Extension TA teams.
8. The comments considered more significant were addressed, with other comments assigned to be addressed in later phases of the standards preparation.
9. Space holders have been included within the document where further inputs are required.

The Metrolinx Tunnels and Underground Structures Interim Standard is available for external users to download via the Metrolinx Public Website at http://www.gosite.ca/engineering_public/

Suggestions for revision or improvements can be sent to the Metrolinx Engineering and Asset Management, Attention: Director of Tunnels and Underground Structures (TUS) who shall introduce the proposed changes to Metrolinx Tunnels and Underground Structures team. The Director of TUS ultimately authorizes the changes. A description of the proposed change shall be included along with information on the background of the application and any other useful rationale or justification. Proposals for revisions or improvements shall also include your name, company affiliation (if applicable), e-mail address, and phone number.

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1. Scope

1.1 Overview

1.1.1 This standard sets out the Metrolinx requirements for the design and construction of Metrolinx Transit Tunnels and Underground Structures as it pertains to subway and LRT projects.

1.2 Purpose

1.2.1 The key objective of this standard is to develop and maintain the Metrolinx requirements and criteria for the design and construction of Tunnels and Underground Structures.

1.2.2 The Contracted Party shall perform all Work in accordance with the requirements of this standard.

2. Definitions, interpretation, and reference documents

2.1 Definitions

2.1.1 Capitalized terms used in this standard shall have the meanings prescribed in Table 1.

Table 1 List of definitions

Term	Definition
"As-Built Survey"	field activities and drawings prepared by the Contractor to reflect the installed, constructed, or commissioned conditions of the Project in a format and with content and details that Contracting Authority, acting reasonably, considers appropriate.
"Asset(s)"	any physical or tangible item that has potential or actual value to Contracting Authority (excluding Intellectual Property, inventory to be sold, human resources, and financial instruments), as well as IT Systems and Software.
"**Asset data and information standards**"	the set of standards defined, maintained, and applied to Metrolinx Contracting Authority Asset Information.
"**Asset Information**"	" means the combined set of data (physical asset data, location and spatial links, work management, performance, condition, and cost data, etc.) and documents (drawings, manuals, etc.) required to support the management of Metrolinx Contracting Authority Assets over the whole Life Cycle. Asset Information is monitored and controlled in Metrolinx Contracting Authority Asset Information Systems in accordance with Metrolinx Contracting Authority Asset data and information standards.
"**Baseline**"	the readings obtained from all monitoring instruments on a continuous basis for a definite period prior to commencing construction activities related to the tunnel for the purpose of establishing a baseline set of monitoring instrument readings against which the limits of movements will be measured.
"**Bored Tunnel**"	the portion or portions of the Work to be constructed using the Tunnel Boring Machine.

Term	Definition
"**Combined Lining**"	means that initial shotcrete lining is part of the SEM structures' permanent load carrying System and contributes to all applicable load cases, except the groundwater load which is fully carried by the Final Lining. Since the Initial Lining is considered to be a part of the permanent structure, it therefore shall meet all the quality and durability requirements for a permanent structure.
"**Construction Tolerances**"	allowable deviation from specified value.
"**Cross Passage**"	a Safety walkway connecting two twin tunnels.
"**Cross-Over**"	two turnouts connecting two generally parallel adjacent tracks, which allow rail vehicles to cross from one track to another.
"Cut-and-Cover Structures"	structures constructed by cut-and-cover methods, including: (i) the use of a temporary excavation support systems to excavate to a planned base elevation followed by permanent structure construction, backfilling, and surface restoration; and (ii) the use of temporary excavation support systems such as soldier-piles and lagging, secant or tangent piles (caissons), concrete diaphragm walls (slurry walls), sheet piles, and soil-nail wall.
"**Design Life**"	a period during which a structure is intended to perform its design function for the intended use(s).
"**Drainage**"	means (a) the process of water being removed from an area naturally or by artificial means, (b) the manner by which water is removed from an area, or (c) the infrastructure that conveys water from an area.
"Emergency Exit Building"	exits at track level complete with stairways to an exit building at grade.
"Final Lining"	means permanent lining in SEM structures that is constructed over the initial shotcrete lining and waterproofing system and is considered to resist the long-term loads, including groundwater load, throughout the required Design Life of the structure. Final Lining can be constructed using rebar or fibre reinforced cast-in-place concrete or shotcrete.

Term	Definition
"Geotechnical Instrumentation and Monitoring Plan"	or "GIMP" means a comprehensive report prepared by the Contractor indicating the location and type of instruments proposed as part of the GIMP to monitor the ground, adjacent infrastructure, and utility movements during the construction activities.
"Guideway"	means the part of a rapid transit system on which the Trackwork is located.
"Headwall"	SOE structures delineating part of the end walls of open excavations along the tunnel Alignment, including those for future open excavations that the TBM will mine through after the SOE structures are installed.
"Heat Release Rate"	the rate at which a fire releases energy, measured in Watts.
"Initial Lining"	is a reinforced shotcrete layer of minimum thickness as indicated on the drawings for the support of excavation areas. The shotcrete lining is reinforced with synthetic or steel fiber, steel reinforcement and lattice girders, where indicated on the drawings. The initial lining is placed after excavation on shotcrete sealed perimeter surfaces to provide initial support, acting alone or in a combination with other reinforcement or additional measures.
"Initial Support"	are measures required to be implemented to construct the SEM structures under reasonably stable ground conditions. Initial Support includes support elements installed prior to excavation of the ground, such as rebar spiling and pipe umbrella, as well as the fibre or steel reinforced initial shotcrete lining implemented for support of excavation.
"Pre-cast Concrete Tunnel Lining"	means reinforced pre-cast concrete segments, connected together in the tunnel to form continuous rings, in turn connected together to create a tunnel lining, used for permanent support of the Bored Tunnel.
"Project Zone of Influence"	the width and magnitude of the predicted settlement trough associated with the construction of tunnel to a point at which the estimated settlement is no greater than 5 mm.
"Right-of-Way"	means Right-of-Way as defined by the governing Municipality. May be used interchangeably with "Municipal Right-of-Way".

Term	Definition
"SEM Structure (s)"	means tunnels and/or caverns constructed: (i) using mechanized or hand excavation to excavate a sequence of headings that form a fraction of the final tunnel face area; (ii) with prior continuous support of ground around the entire perimeter above the tunnel spring line (upper tunnel half and within face of all tunnel headings); and (iii) with incremental support of the ground using steel ribs, lattice girders, or other structural support, as required, along with sprayed concrete (shotcrete).
"Setback"	means distance on a lot measured at a right angle from the lot line to the nearest main wall of a building or structure.
"Shaft" or "**Shafts**"	means supported open excavations required for the construction of the Works with the SOE retaining ground on all sides of the excavations. They may include permanent structures within the SOE limits per Works scope definition.
"Support of Excavation"	means earth retaining structural system (typically temporary) to allow the removal of soil with vertical or near vertical cuts (or excavation profiles that would be otherwise unstable).
"Temporary Structure(s)"	means a structure with a Design Life of not more than 10 years.
"Tunnel Boring Machine"	means the tunneling machine used by the Contractor in the performance of the Works in relation to the Bored Tunnel.
"Tunnel Invert"	means the lowest section of a tunnel. On a circular configuration, it is approximately the bottom 90 degree of the arc of the tunnel.

* This table will be further developed in the upcoming revisions

2.2 Abbreviations

2.2.1 The abbreviations used in this standard shall have the meaning prescribed in Table 2.

Table 2 List of abbreviations

Abbreviation	Definition
3-D	means 3-Dimensional.
AASHTO	means American Association of State Highway and Transportation Officials.
ACI	means American Concrete Institute.
ASCE	means American Society of Civil Engineers.
ASTM	means American Society for Testing and Materials.
BIM	means Building Information Modelling.
CAN/CSA	means Canadian Standard Association.
CIAR	means Construction Impact Assessment Report.
CIAR-1	means Construction Impact Assessment Report – Level 1.
CIAR-2	means Construction Impact Assessment Report – Level 2.
“DM” or **“TTC-DM”**	means Toronto Transit Commission (TTC) Design Manual.
“DSVR”	means Design Standard Variance Request.
“EA”	means Environmental Assessment.
“EAS”	means Existing Adjacent Structure.
“EAU”	means Existing Adjacent Utilities.
“EEB”	means Emergency Exit Building or Emergency Exit Button.
“EGRESS”	means Emergency Global Rescue, Escape & Survival System.
“EW”	means Early Works.
“GDMR”	means Geotechnical Design Methodology Report.
“GDR”	means Geotechnical Data Report.
“GIMP”	means Geotechnical Instrumentation and Monitoring Plan.
“GIR”	means Geotechnical Interpretive Report.
“GIS”	means Geographic Information System.
“GUI”	means Graphical User Interface.
“HRR”	means Heat Release Rate.
“IFC”	means Issued for Construction.
“LRT”	means Light Rail Transit.
“NBC”	means National Building Code of Canada.
“OBC”	means Ontario Building Code.
“OHSA”	means Occupational Health and Safety Act.
“OPSS”	means Ontario Provincial Standard Specification.
“PCTL”	means Precast Concrete Tunnel Linings.

Abbreviation	Definition
"RCD"	means Reference Concept Design.
"RFP"	means Request for Proposals.
"ROW"	means Right of Way.
"SEM"	means Sequential Excavation Method.
"SOE"	means Support of Excavation.
"STD"	means Station.
"TBM"	means Tunnel Boring Machine.
"ToR"	means Top of Rail.
"TTC"	means Toronto Transit Commission.
"ZOI"	Means Zone of Influence.

* This table will be further developed in the upcoming revisions

2.3 Interpretation

2.3.1 This standard shall be interpreted according to the following provisions, unless the context requires a different meaning:

2.3.2 unless the context specifically states otherwise, all obligations included herein are the responsibility of the Contracted Party to undertake;

2.3.3 wherever used herein the plural includes the singular, the singular includes the plural, and each of the masculine, feminine and neutral genders include all other genders;

2.3.4 references to persons shall include their successors and permitted assigns. References to a public organization shall include their successors and assigns, and if a public organization ceases to exist or ceases to perform its functions without a successor or assign, references to such public organization shall be deemed to include a reference to any public organization or any organization or entity which has taken over either or both the functions and responsibilities of such public organization; and

2.3.5 references containing terms such as "includes" and "including", whether or not used with the words "without limitation" or "but not limited to", shall not be deemed limited by the specific enumeration of items but shall, in all cases, be deemed to be without limitation and construed and interpreted to mean "includes without limitation" and "including without limitation".

2.4 Codes, standards and reference documents

2.4.1 All systems, equipment and materials required for Work relating to this standard, shall be provided in accordance with the most current edition of applicable federal, provincial, municipal, and industry codes, standards, and guidelines including the following:

2.5 Standards Deviation Process

2.5.1 All permanent and temporary systems shall be designed and constructed to the same specification, quantity and quality outlined in Metrolinx Standards, unless requirements are otherwise outlined in this standard; or a deviation is approved by Metrolinx through the Standard Deviation Process.

3. General

3.1 Alignment (placeholder)

3.2 Right of Way/Property (placeholder)

3.3 Clearances (placeholder)

3.4 Control Survey (placeholder)

3.5 Design Life

3.5.1 The target system design life is its intended useful life. Individual subsystems or components may be renewed, replaced, or upgraded during the system design life. Verification methods as to how to predict and meet design life targets shall be submitted to Metrolinx for review.

3.5.2 Unless specified elsewhere, the following system component minimum target design life ranges (Table 3) shall apply:

Table 3 System Component Minimum Target Design Life

Element	Design Life (years)
Structures and tunnels	100
Major structural components	100
Buildings (per CSA 478-95 guideline)	50-80
Elevated structures and bridges	75 (see Note 1)
Permanent retaining walls and earth structures	75 (see Note 1)
Trackwork slab structures	50
Trackwork	30 (see Note 2)
Underground services (pipelines, etc.)	40-50
Minor structural components (replaceable)	30-50
Electric / electronic equipment / cabling	30 (see Note 3)
Train control signal equipment	30 (see Note 3)

Element	Design Life (years)
Communications equipment	30 (see Note 3)
Traffic signals	30 (see Note 3)
Mechanical equipment	30
Elevators and escalators	30
Ventilation equipment	30
Traction power and distribution equipment	30
Excavation support systems (to be handed over)	10
Note 1: Compatible with the Canadian Highway Bridge Design Code. Note 2: With subcomponent renewal and ongoing inspection and maintenance Note 3: Hardware/software shall be upgraded at the end of useful product life or when declared obsolete by the manufacturer, whichever occurs first.	

Modified from Metrolinx Light Rail Transit Design Manual (March 2016).

3.6 Building Information Modelling (BIM) Requirements

3.6.1 BIM Data Collection Process

3.6.1.1 The production of a BIM based project record system is a complex process requiring the coordination of data collection by multiple parties (see Figure 1). The project records shall include those gathered during construction and those accumulated during the operation and maintenance of the facilities. The data shall be collected and organized in accordance with the Metrolinx Asset Data and Information Standard.

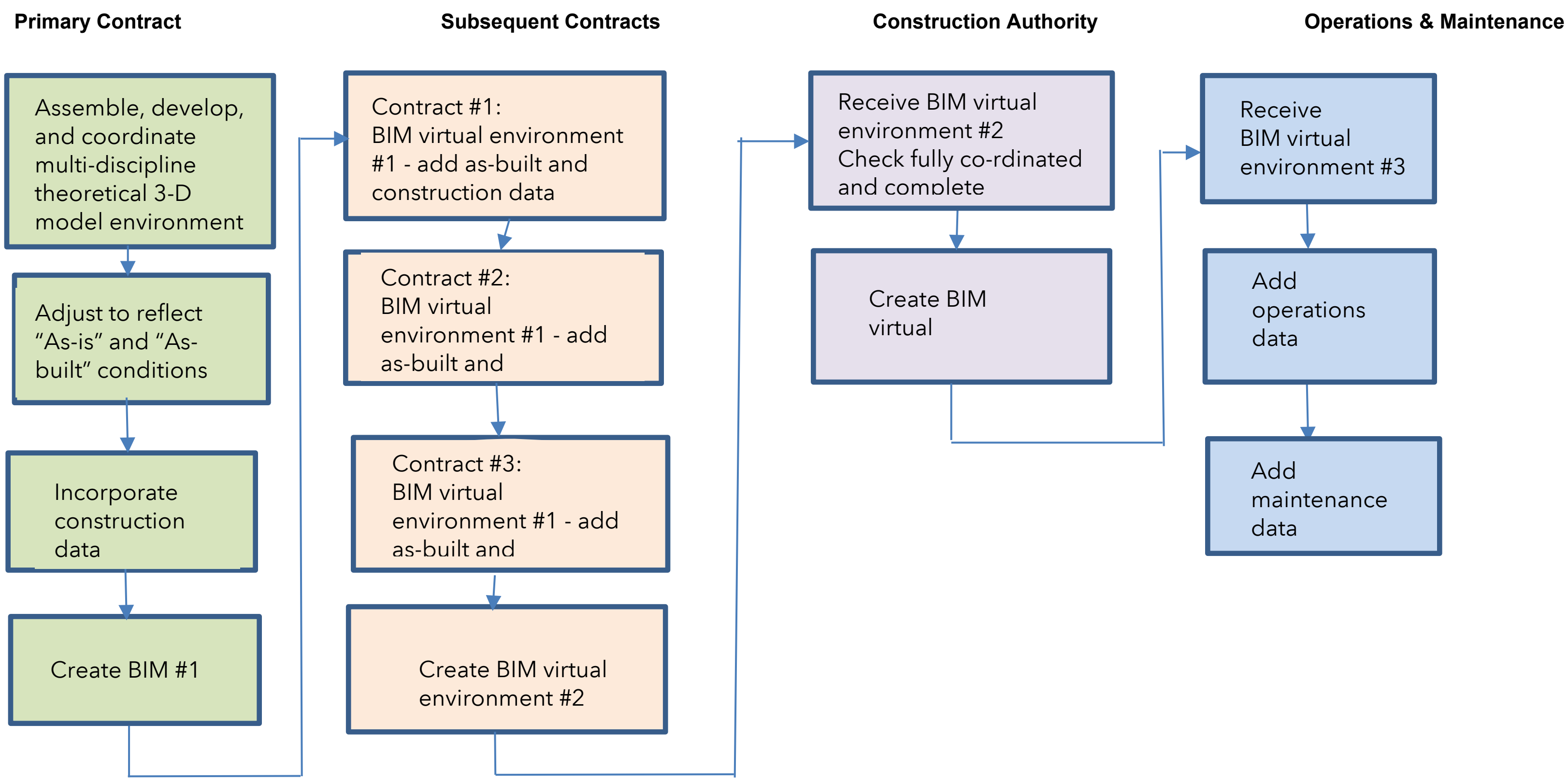


Figure 1: BIM Data Collection Process

(Modified from Metrolinx - Ontario Line Project Agreement Schedule 15 - Appendix X; Figure 1-1)

3.6.1.2 Throughout these transfers it is important that a Common Data Environment (CDE) is maintained. The following Table 4 provides the Contracting Authority CDE Technologies.

Table 4 Contracting Authority CDE Technologies

Technology	Function
Microsoft SharePoint	Storage, review and archiving of all project documentation
Autodesk BIM 360	Storage and management of CAD product source files
ProjectWise	Storage, management, and review of CAD product source files
Aconex	Storage, review and archiving of all project documentation
Bluebeam	Collaborative drawing review
Enterprise Maintenance Management System	Enterprise Maintenance Management System
ESRI GIS	Enterprise Geographic Information System
OpenText	Electronic Document and Records Management System. Storage, review and archiving of all project documentation
Unifier	Content Management System

(From Metrolinx - Ontario Line Project Agreement Schedule 15 - Appendix W; Figure 5-1).

3.6.1.3 The Operations and Maintenance use of the BIM system shall be addressed in 7: “State of Good Repair - Tunnels and Structures”, specifically Section 7.4.1 “BIM Model”.

3.6.2 3-D CADD Modelling of As-Built Condition

3.6.2.1 This section addresses the following item:

a) The production of the 3-D model environment of the as-built location of the physical structures and other elements of the construction project.

3.6.2.2 The creation of the 3-D model environment shall commence with the production of the design 3-D model environment during the multi-discipline design process to reflect the theoretical dimensions and location of the structural elements and related features.

a) LiDAR scanning of the internal surfaces of the structure (and external surfaces where accessible), shall be used, after post-processing, to correct the design theoretical 3-D model environment to reflect the as-built location of the structures. The LiDAR scans shall be supplemented with physical surveys to relate the scans to geospatial references and where LiDAR scanning is not practicable, such as areas hidden by equipment. The scans shall also locate cross passages and ancillary tunnels along the alignment and shall survey these structures to include their locations in the 3-D model environment. The level of development of the 3-D model environment for the transit structures shall be LOD 500 (Reference: BIM Forum - Level of Development (LOD) - Specification Part 1 - Guide and Commentary, December 2021).

3.6.2.3 LiDAR scanning shall also be used to locate and reproduce surface features within 50 metres (m) each side of the reference line along the alignment and to include these into the 3-D model environment. The level of development of the 3-D model environment for surface features shall be LOD 300.

3.6.2.4 The model shall include the as-built locations of Support of Excavation (SOE) elements, such as survey data for secant piles collected by a SONICaliper or equivalent instrument for surveying drill holes.

3.6.2.5 The As-Built 3-D model environment should also contain any existing underground foundations/structures (e.g., piles) from adjacent buildings within 50m of the alignment. This should be in a site context model.

3.6.2.6 The model shall include the incorporation of any underground and above ground utilities that have been relocated and any surveyed locations of existing utilities at maintenance holes or where they have been exposed.

3.6.2.7 The standard shall address the use of the adjusted model to incorporate visualizations of elements from other disciplines, such as trackwork, electrical/mechanical, and systems.

3.6.3 Data that shall be Attached to the BIM Model

3.6.3.1 This section addresses the following items:

a) The records that shall be attached to the model for data from the manufacturing and construction of the structures.

b) The records that shall be attached to the model for product data and data from installation of elements from other disciplines, such as trackwork, electrical/mechanical, systems.

3.6.3.2 For the underground structures, the minimum data lists (for item 1 above) are shown in Sections 3.6.4, 3.6.5, 3.6.7 and 3.6.8. These lists shall form the basis for the standard data lists.

3.6.3.3 For surface structures, equivalent data lists shall be developed.

3.6.3.4 BIM development shall include the attachment of product and installation data from other disciplines, such as trackwork, electrical/mechanical and systems.

3.6.3.5 It is important that sufficient development and time is given to defining/assessing empty placeholders for assigned data and meta data fields that are required for each of these sections. This is to facilitate, (using Digital Site apps/technologies via handheld devices) their capture in a near real-time as possible during the construction process and not have to be reverse-fitted or post-processed after the activity.

3.6.4 Minimum Data List for Tunnel Boring Machine (TBM) Driven Tunnels.

3.6.4.1 Quality records and lining manufacturing process details including, as applicable:

a) Design development records, concrete strength, and permeability test results, rebar mill and weld test results, steel fibre type and quantity, beam test results, plastic fire protection fibre type and quantity included, and repairs undertaken.

b) Results of fire tests on PCTL.

c) Manufacturing activities start and finish dates, for each ring, including casting, curing process, outside storage and delivery to site.

d) Non-Conformance Reports (NCRs) and their resolution.

3.6.4.2 Tunnel excavation and lining process for each tunnel, including:

a) Ground conditions excavated based on tested samples from spoil discharge.

b) Ground treatment type and extent.

c) Construction records including lining annulus grouting records, soil conditioners used with type and quantity, foam injection ratio (FIR) and foam expansion ratio (FER), quantity of bentonite injected around TBM shield and contact grouting records.

d) Damage detected on rings, segment rejected or assigned for repair.

e) Visible leak locations with associated repair records and water ingress data with overall volumes related to specified limits.

f) Ground and building movement data, related to defined review and alert levels.

g) Construction activities start and finish dates including launch of TBM, shifts length and number worked per day, days worked per week, stoppages (including cause and resolution), installation of first lining ring (chainage, start and end time, and date installation of each subsequent ring (chainage, start and end time and date), mining start and end time for each ring.

h) Note: These could be defined through the 4-D model environment - planned and actual along with other data fields.

3.6.5 Minimum Data List for Sequential Excavation Method (SEM) Driven Tunnels

3.6.6 The list is based on soft ground SEM tunnels but is applicable to rock SEM tunnels.

3.6.6.1 Quality records, including:

a) Shotcrete mix proportions, accelerator used, type and quantity, slump, air content and density, 24-hour and 28-day compressive strength required values and test results during construction.

b) Shotcrete field test panel results including coring results, nozzlemen proficiency test results.

c) Reinforcing used in initial and final linings, steel rebar, mesh, or fibre. Lattice girders used or not.

d) Membrane type, single or double, thickness, use of protective fleece, compartmentalization, waterstop type used to create compartments, grouting tube locations.

e) NCRs and their resolution.

3.6.6.2 Dewatering records including:

a) Well types, spacing and installation dates.

b) Groundwater table lowering per day.

c) Date of required groundwater lowering achievement.

d) Steady state dewatering pumping rate.

3.6.6.3 Excavation and lining design records including:

a) Segmentation of excavation face, with cross section drawing. Face areas of each heading, bench, or invert excavation. Order of excavation of headings.

b) Additional support measures - spiling; grouted canopy tubes (barrel vault) arch with pipe sizes, lengths and spacings; earth wedge for face support, pocket excavation; face shotcrete, ground treatment type and extent.

c) Round lengths for each heading.

d) Initial shotcrete lining thickness and installation stages.

e) Final concrete or shotcrete lining thickness and installation method.

3.6.6.4 Excavation and lining construction records including:

a) Excavation start-dates and daily advance on each heading. Excavation completion dates for each heading.

b) Ground conditions encountered in each heading, by chainage.

c) LiDAR records of excavation surface, initial lining surface and lining thickness for each heading, including temporary walls.

d) Installed additional support measures.

e) Completion date for initial lining installation including removal of temporary walls.

f) Start date for membrane installation and daily advance. Date of completion of fully tested membrane.

g) Start and completion of installation of invert for final lining for first pour with start and end chainages. Equivalent records for subsequent pours.

h) Start and completion of formwork installation for arch of final lining for first pour with start and end chainages. Completion of concrete placement. Equivalent records for subsequent pours.

i) If shotcrete is used for final lining, start and completion of section with start and end chainages.

Note: These could be defined through the 4-D model environment - planned and actual along with other data fields.

3.6.6.5 Repair records, as applicable:

a) Leaks found, with location and date repairs conducted. Repair method and material quantities used. Use of membrane grouting tubes. Visual records and other data to provide evidence of the effectiveness of the repair.

b) Damage found, with location and date repairs conducted. Visual records and other data to provide evidence of the effectiveness of the repair.

Note: These could be defined through the 4-D model environment - planned and actual along with other data fields.

3.6.7 Minimum Data List for Cut and Cover Tunnels, Stations and Associated SOE

3.6.7.1 Quality records including:

a) Steel sections mill tests, rebar mill tests, weld test results, concrete test results, strength, slump and air entrainment, concrete mix data, plastic fibre type and quantity for fire protection, if applicable.

b) Tie back stressing records for SOE.

c) NCRs and their resolution.

3.6.7.2 Excavation and structure construction records including:

a) Ground conditions encountered.

b) Ground treatment type and extent.

c) Construction activities start and finish dates, including utility relocations, temporary and permanent road realignment and associated activities, traffic staging, SOE installation stages, excavation and the forming, rebar placement and concrete placement of base slabs, walls, and roof slabs.

d) Waterproofing type and start and finish dates.

e) Leak locations, leak volume test results, crack locations and sizes, associated repairs and repair results with start and finish dates.

f) Record Drawings, including SOE details.

g) SOE performance during excavation and build-back construction, including horizontal deflection records, tie back tensioning records, strut strain gauge records and ground vertical settlement adjacent to the excavation data.

Note: These could be defined through the 4-D model environment - planned and actual along with other data fields.

3.6.8 Minimum List for Cross Passages and Other Ancillary Tunnels

3.6.8.1 Quality records including:

a) Steel sections mill tests, rebar mill tests, weld test results, concrete test results, strength, slump and air entrainment, concrete mix data, if applicable.

b) Shotcrete mix proportions, accelerator used, type and quantity, slump, air content and density, 24-hour and 28-day compressive strength required values and test results during construction.

c) Shotcrete field test panel results including coring results, nozzlemen proficiency test results.

d) Reinforcing used, steel rebar, mesh, or fibre. Lattice girders used or not.

e) Membrane type, single or double, thickness, use of protective fleece, compartmentalization, waterstop type used to create compartments, grouting tube locations.

f) Groutable tubes and hydrophilic gaskets used in joint with running tunnel lining.

3.6.8.2 Dewatering records including:

a) Well type, spacing and installation date.

b) Date of start of pumping.

c) Groundwater table lowering per day.

d) Date of required groundwater lowering achievement.

e) Steady state dewatering pumping rate.

3.6.8.3 Excavation and lining design records including:

a) Ground conditions encountered.

b) Ground treatment type and extent.

c) Excavation and initial lining, type, and installation methods.

d) Waterproofing methods and details.

e) Final lining, type, and installation method.

3.6.8.4 Excavation and lining construction records including:

a) Start and finish dates for installation of temporary propping of running tunnel lining, for cutting of the required opening in the running tunnel lining, for excavation of the tunnel, for installation of the initial lining, for installation of the waterproofing membrane for final lining invert and arch installation, for the completion of the cross passage, removal of temporary propping and restoration of the groundwater to the preconstruction level.

b) Note: These could be defined through the 4-D model environment - planned and actual along with other data fields.

3.6.8.5 Repair records, as applicable:

a) Leaks identified, with location and date repairs conducted. Repair method and material quantities used. Use of membrane or joint grouting tubes. Visual records and other data to provide evidence of the effectiveness of the repairs.

b) Damage found, with location and date repairs conducted. Visual records and other data to provide evidence of the effectiveness of the repairs.

Note: These could be defined through the 4-D model environment - planned and actual along with other data fields.

3.7 Water Ingress Limits

3.7.1 Watertightness Criteria

3.7.1.1 Table 5 presents the watertightness criteria that shall be achieved. These are adapted from Haack's Watertightness Criteria (1991). It shall be noted that it is not acceptable to combine sections into an aggregated length to meet the 100 m criterion on average - any 100 m length, as selected by Metrolinx, shall meet the criterion.

Table 5 Watertightness Criteria (based on Haack's Watertightness Criteria, 1991)

Tightness Class	Moisture Characteristics	Intended Use	Definition	Permissible Daily Leakage Water Quantity (L/m2), Given a Reference Length of: 10 m	100 m
1	Completely dry	Transit Stations	The wall of the lining shall be so tight that no moisture patches are detectable on the inside	0.02	0.01
2	Substantially dry	Running Tunnels and Cross Passages for Transit Facilities	The wall of the lining shall be so tight that only slight, isolated patches of moisture can be detected on the inside (e.g., as a result of discolouration). After touching such slightly moist patches with a dry hand, no traces of water should be detectable on it. If a piece of blotting paper	0.1 (Not applicable to Cross Passages)	0.05

Tightness Class	Moisture Characteristics	Intended Use	Definition	Permissible Daily Leakage Water Quantity (L/m2), Given a Reference Length of: 10 m	100 m
			or newspaper is placed upon a patch, it shall on no account become discoloured as result of absorbing moisture.		
3	Capillary wetting	Supported Excavations for Transit Facilities to be handed over, Emergency Exit Buildings and Ventilation Shafts	The wall of the excavation shall be so tight that only isolated, locally restricted patches of moisture occur. Restricted patches of moisture reveal that the wall is wet, leading to a discolouration of a piece of blotting paper or newspaper if placed upon it - but no trickling water is evident.	0.2	0.1

3.7.1.2 In addition to the requirements of Table 5, water seepage shall not drip or flow onto track, walkways, escalators, electrical services, mechanical equipment, signaling, lighting, communication or controls equipment, and there shall be no more than one drip per minute from any single location into any underground structure.

3.7.1.3 These ingress criteria shall be applied to all future Metrolinx structures. The extent of the transit station structures shall be defined on station footprint drawings providing the locations of the interfaces with other structures. The extent of the cross passages shall be defined on cross passage footprint drawings that shall show that the joint between the cross passage and the running tunnel shall be included in the cross-passage footprint.

3.8 Fire Resiliency Requirements

3.8.1 Applicability

3.8.1.1 The following requirements shall apply to any structures or portions of structures that may be exposed, in close enough proximity to cause concrete spalling damage, to a transit vehicle fire within the transit facilities.

3.8.2 Design Fire Intensity Values

3.8.2.1 The Heat Release Rate curves for the various Metrolinx operational vehicle types shall be included in this section. These curves; shall be based on vehicle fire criteria provided by the manufacturers and analysis thereof, similar to that presented in "Transit Expansion - Light Rail Transit Project - Station Fire Computational Fluid Dynamics (CFD) Criteria Report, 14/11/2012, Rev 1". The appropriate curve shall be used as input to CFD modelling to predict the fire development over time and to calculate the maximum temperature at the surface of the tunnel lining or structural element. This analysis would be similar to that presented in "HMM 270798-03 Tunnel Fire Loading Evaluation November 9, 2012".

3.8.3 Effects of Fire on the Structure

3.8.3.1 Coupled Finite Element or Finite Difference analysis shall be used to assess the effect of the predicted maximum temperature at the surface of the tunnel lining or structural element.

3.8.3.2 The models shall assess the action of water vapour pressure causing explosive spalling of the concrete surfaces.

3.8.3.3 The design may recognize that the depth of spalling can be reduced by the addition of polypropylene fibres to the concrete mix, if this approach is employed. Research, (Sullivan,2001 and Khoury, 2008), indicates that the melting of these fibres during a fire produces relief routes for pressurized water vapour, (ITAtech 2016).

3.8.3.4 The depth of spalling and the resulting loss of cross-section shall be defined. For tunnel linings, the required strength of the structural element shall be shown to be maintained after this loss of cross-section. For other structure types, such as cut and cover box structures, the required strength of the structural element shall be shown to be maintained after the loss of cross-section and any damage that may have occurred to the reinforcement within the cross-section due to loss of concrete cover.

3.8.3.5 In all structure types, the structural element shall not experience structural failure after the fire exposure and shall be able to be repaired to restore its remaining service life to that required by design.

3.8.4 Verification by Physical Fire Testing

3.8.4.1 For tunnel linings, the predicted depth of spalling shall be demonstrated not to be exceeded, by physical fire testing.

3.8.4.2 These fire tests shall be conducted in accordance with the "Efectis Fire Testing Procedure for Concrete Tunnel Linings and Other Tunnel Components, September 2020", where it relates to uninsulated concrete tunnel linings.

3.8.4.3 The fire test sample shall be a complete lining segment reflecting the concrete mix, reinforcement and curing used in the manufacturing production runs.

3.8.4.4 It shall be noted that the test requirements include the application of a load that creates hoop stresses in the tunnel segment during the test to reflect the in-practice applied load in the lining.

3.8.4.5 For other structure types, similar tests shall be conducted on representative samples of the same concrete mix, reinforcement and curing as the wall or roof slab, under consideration, of minimum area of 5.0 m^2 x (wall or slab thickness). These samples shall not have to be loaded to reflect in-practice loading.

4. Underground Structures

4.1 Loads

4.1.1 Normal Load Applications

4.1.1.1 The load factors shall be based on Table 6. The unbalanced load case shall be described, and a diagram added.

Table 6 Load Factors

Load	Maximum Load Factor	Minimum Load Factor
Dead Load		
Factory produced components, including trackwork	1.25	0.95
Concrete structure, wood and non-structural	1.25	0.90
components	1.25	0.80
Ties and ballast	1.4	0.80
Negative skin friction on piles		
Vertical Earth Load γ = 22 kN/m3 (minimum)	1.5	0.80
If cover ≤ 3.0 m	1.4	0.80
If cover > 3.0 m		
Earth Pressure and Porewater Pressure		
Horizontal passive earth pressure 1	1.4	0.60
Horizontal at-rest or active earth pressure	1.4	0.85
Minimum horizontal earth pressure case 2	N/A	0.50
Horizontal water pressure	1.25	0.85
Uplift water pressure 3	1.3	0.8
Surcharge	1.5	0
Prestress		
Secondary prestress effects	1.05	0.95
Prestress	1.00	1.00
Live Loads		
Transit vehicle loads	1.5	0
Platform and concourse	1.5	0
Buoyancy Permanent load with factor of 1.0 and vertical water pressure with a factor of 1.1		
Notes: 1 When considered a load 2 Post-construction loading case 3 For design of components for water pressure		

4.1.2 Earthquake Loads

4.1.2.1 Earthquake loads shall be added to the load factor table.

4.1.2.2 When earthquake loads are included, the unbalanced load combinations do not need to be considered unless the conditions inducing the unbalanced loads exist permanently or for more than 75% of the expected service life of the underground structure.

4.1.3 Time Dependent Loads

4.1.3.1 The Georgian Bay Shale is known to have swelling properties in the presence of certain conditions. These conditions shall be described, and the resulting loads quantified when mitigating measures are absent. More development is required for this section.

4.1.4 Numerical Ground/Structure Interaction Modelling

4.1.4.1 Where a numerical ground/structure interaction model is used, the following analysis methodology shall be employed:

a) Model runs shall be executed with K_o values at the upper and lower bounds defined in the geotechnical design report (two runs).

b) Model runs shall be executed with water levels at the upper and lower bounds defined in the geotechnical design report (two runs).

c) A factor of 1.4 shall be applied to the results for calculation of structural loading effects. Any loads that would have a load factor different from 1.4 would have their input loads adjusted to allow a factor of 1.4 to be used for the model results (e.g., live load inputs would be multiplied by $\frac{1.5}{1.4}$ to reflect a load factor of 1.5).

4.1.5 Allowance for Future Developments

4.1.5.1 Where the transit facility shall be located within a street right-of-way, in the absence of project and site-specific development and overbuild loading diagrams, a prototypical future adjacent development is to be allowed for in the structural design of the facility.

4.1.5.2 This development shall be assumed to have the following features:

a) Located where the transit facility is closest to the street property line along the alignment.

b) Excavation depth of 15 m.

c) Excavation to extend to the street property line.

d) Excavation to extend to 30 m away from the property line.

e) Excavation to extend 30 m in the direction parallel to the property line.

f) Excavation to be supported by 1.0 m diameter (dia.) Secant piles and tiebacks at 3.0 m vertical spacing. Tiebacks shall be located to avoid conflict with transit facility.

g) Building to be 30 storeys high, for office use, and to occupy the full footprint of the excavation.

h) A diagram shall be added to show the details described above.

i) Analysis shall include, initially, the effect of the excavation and, finally, the effect of the building loads.

4.1.5.3 This prototypical development shall be in addition to any known approved development, that shall require site specific analysis and consideration of its substructure, along the alignment.

4.2 Tunnels Excavated by TBM

4.2.1 TBM Requirements

4.2.1.1 Soft Ground TBMs

a) Soft ground TBM's shall be either Earth Pressure Balance (EPB) TBMs or Slurry TBMs and shall have the following features:

1) The ability to handle and install a precast concrete segmental tunnel lining and to derive forward propulsion by hydraulic rams acting on the installed lining.

2) The ability to pressurize the excavation face and the excavation chamber with pressure levels sufficient to balance the calculated and factored maximum soil and hydrostatic pressures for the tunnel drive, including allowance for pressure fluctuations.

3) The ability to inject fast-gelling grout through the tail shell of the TBM filling the annulus, formed between the extrados of the tunnel lining and the cut surface of the excavation, as the machine moves forward.

4) The ability to maintain tunnel alignment within the required parameters in all ground conditions including transition zones having a mixed face of soft ground and rock.

5) The ability to record machine data in near real time and to receive ground movement monitoring data, when delivered. "Near real time" means a time delay not exceeding the time needed for automatic data acquisition, electronic continuous transfer of data and automatic data processing for visualization.

b) If Earth Pressure Balance TBM:

1) The ability to inject soil conditioning agents ahead of the cutterhead and into the excavation chamber to produce a pressure-resisting paste of the excavated material.

2) The ability to dissipate the pressures through the length of the screw conveyor allowing the discharge at atmospheric pressure on to an open conveyor.

3) The ability to inject bentonite under pressure into the annulus around the TBM shield formed between the extrados of the shield and the cut surface of the excavation.

4) The ability to measure muck weight and volume on the open conveyor.

c) If Slurry TBM:

1) The ability to convey all excavated material to a separation plant using fluid bentonite as a transport medium.

2) The ability to separate excavated material from the transport medium at the separation plant.

3) The ability to measure muck discharge rate with flow meters and monitoring of the separation plant discharge.

4.2.1.2 Rock TBMs

a) For rock TBMs to be used within the Greater Toronto Area (GTA), these shall be shielded TBMs similar to those used for soft ground and shall be used in combination with a segmental tunnel lining.

b) The cutting heads shall be configured with a larger proportion of cutting discs compared with ripper tools than the configurations for soft ground TBMs.

c) The cutterhead configuration shall allow for the potential for mixed face conditions at rock/soil or soil/rock transitions and at buried valleys and faults.

d) The machines shall have an ability to apply pressure to the excavation but shall be generally operated in the open non-pressurized mode when in a full face of rock.

e) The point at which the machine shall be switched from pressurized to non-pressurized operation, or vice versa, shall be determined from the local conditions at the transition, on a case-by-case basis.

f) Contract documents will provide minimum probe hole drilling lengths and probe hole positions according to the subsurface conditions.

g) In absence of project specific requirements when non-pressurized face TBM is used:

1) Alternate probe holes positioned between 1 to 2 o’clock and between 10 to 11 o’clock shall be carried out;

2) At least one probe hole shall be maintained 30 m ahead of the TBM current face with a 10 m overlap from the previous drilled probe hole;

3) When a TBM bore trails an adjacent previously completed TBM bore or trails a TBM bore that is at least 50 m ahead, only probe holes on the far side of the new TBM bore shall be caried out.

h) The rock TBMs shall have the same ability as soft ground TBMs to handle and install the segmental lining, and to fill the annular space around the lining with grout.

i) The TBMs shall have the ability to inject dust suppression and abrasion control conditioners into the excavation chamber.

4.2.2 Precast Concrete Tunnel Linings (PCTL)

4.2.2.1 General

a) The precast concrete tunnel lining is a prefabricated support system that is designed to carry loads from construction handling, from the ground and from groundwater, that is erected as a ring within the TBM tail shield and that serves as the initial and final lining for the tunnel.

4.2.2.2 Ring Length and Segmentation

a) There is no recognized standard for ring length for PCTL, but the following observations are from recent precedents.

1) There is a general trend to employ longer ring lengths due to resulting improvements in productivities. Since these decisions are driven by handling, transportation and installation issues they are mostly made by the tunnel contractor.

2) The ratio of internal diameter to ring length varies with tunnel diameter as shown below:

A. Twin tunnel subway/Light Rail Transit (LRT) size tunnels ratio = 3.0 - 4.0

B. Intermediate size tunnels (SSE) ratio = 5.0 - 6.0

C. Large tunnels (Alaskan Way) ratio = 6.0 - 8.0

b) Similarly, there is no recognized standard for segmentation for PCTL. The following are recent trends:

1) In North America, the Universal Ring has become a popular format.This has all radial joints at an angle to the tunnel alignment and has a key segment and a counter-key segment that are wedge shaped. Recent variants on this have the key segment smaller than the counter-key segment. This configuration is considered to offer advantages in maintaining gasket position.

2) Elsewhere, the Rectangular Ring is more used. This has most of the radial joints parallel to the tunnel alignment except for those adjacent to the wedge-shaped key. This arrangement relies on gasket lubrication to avoid dragging of the gasket during segment installation. This ring configuration provides a somewhat more flexible ring than the Universal Ring.

c) Lining rings shall be tapered in length as the sole means to allow the negotiation of curves along the tunnel alignment and to allow alignment corrections during tunnelling.

4.2.2.3 Concrete Properties

a) The concrete property requirements shall be influenced by the choice of reinforcement type.

b) If steel rebar is used, there shall be emphasis on the reduction of micro-cracking during the manufacturing and handling processes and that the concrete is to be highly impermeable and resistant to chloride diffusion to achieve a long design life.

c) If steel fibre is used, chloride corrosion shall not be an issue due to the lack of connectivity between the fibres. However, as steel fibre reinforced segments are more fragile than steel rebar reinforced segments then ductility of the element shall be important.

d) All PCTL shall be manufactured using high performance concrete. The PCTL shall go through a steam curing process to accelerate maturation and allow early stripping from the moulds. Concrete compressive strengths shall be a minimum of 50 MPa at 28-days.

4.2.2.4 Reinforcement

a) Reinforcement for PCTL shall be either steel rebar or steel fibres. Recent projects have used steel fibre.

b) Steel rebar generally needs to be welded at intersections due to inadequate room for rebar development lengths.

c) To improve ductility for concrete segments, the ultimate post-cracking flexural resistance of the lining segments in the hoop direction shall be no less than 1.2 times the initial cracking flexural capacity in the same direction.

d) Mono-filament polypropylene fibres shall be added to the concrete mix, at a minimum rate of 1.5 kg/m^3 of concrete, to provide improved resistance to spalling under fire conditions.

4.2.2.5 Waterproofing Gaskets

a) Ethylene Propylene Diene Monomer (EPDM) gaskets shall be used within the joints between the segments forming the tunnel lining.

b) Gaskets in the installed lining shall provide the required water tightness for the required design life.

c) The gaskets are typically located near the extrados of the segments. The number of gaskets shall be determined by the designer recognizing that the water tightness criteria shall be achieved.

d) The gaskets shall be embedded-type gaskets where the gaskets have attached tangs that are embedded into the concrete of the segments.

e) The gaskets shall have internal open cells within their cross-sections that are continuous around the gasket corners to allow the gaskets to compress consistently along their length.

f) The gaskets groove depth shall allow the gaskets to achieve the required design and performance criteria over a range of anticipated gap widths and allowable tolerances as they relate to gasket seating offset and segment ring out of roundness.

4.2.2.6 Bolts and Dowels

a) Proprietary plastic dowels shall be used to provide the connection between the lining rings.

b) Connections between the segments forming the rings shall be made using straight bolts installed on a diagonal through the joint.

c) The bolts shall be threaded into embedded sockets on one segment through a preformed bolt hole on the adjacent segment.

d) Bolts shall be left in place during and after construction.

4.2.2.7 Other Hardware and Features

a) Plastic guide rods shall be used in the radial joints of the lining. The rods shall be located within formed grooves in the joint face and shall be held in one of the faces by embedded attachment inserts.

b) Shear cone recesses shall be formed in the intrados of the segment for the vacuum segment erector. If required, a grouting insert shall be included in the centre of the segments.

c) If rebar is used, dimples shall be formed on the intrados to indicate rebar-free locations for drilling of attachment devices.

d) To allow tracking of the segments, a RFID (radio frequency identification) tracking tag shall be embedded in each segment. The RFID tags shall be programmed with manufacturing data, casting times and dates etc., related to the segment.

4.2.2.8 PCTL Production and Handling

a) For large scale PCTL production, the segments shall be manufactured using a carousel that is a moving production line where the segment moulds are delivered to different workstations for mould preparation, reinforcement installation (if used), concrete placement and compaction, finishing of the extrados, travel through the steam curing facility, and to the demoulding location.

b) The segments shall be moved either to secondary curing or to the storage location.

c) The segments shall be transported to the storage facility at the tunnel site. For each of these processes, the segments shall be handled with care to avoid damage.

d) By using test rings at the manufacturing plant and, their test assembly, it shall be demonstrated that the lining segments can be assembled to meet the required tolerances.

4.2.2.9 Repair Procedures

a) Concrete segments with major damage that, in the opinion of Metrolinx, impairs structural integrity or performance shall be rejected.

b) Segments that show excessive crazing, damage or defects shall be recorded and investigated to determine the cause.

c) After completion of the investigation, the segments shall be rejected or accepted following repair.

d) Repairs shall generally occur at the segment manufacturing facility. Circumferential and radial edges of the segments shall be stone rubbed to remove sharp edges.

e) When damage occurs during transportation or during erection of the lining, the damage shall be immediately deemed repairable or non-repairable.

f) If non-repairable, the segment involved shall be removed and rejected from use in construction.

g) Repairable segments shall be repaired prior to handover of the tunnel.

h) Repair of damage shall be performed as described in the following Table 7 and Table 8.

Table 7 Defect Remedies

Damage/Defect	Description	Location	Extent	Remedy
Non-structural patching	Blowholes and air voids	All locations	Diameter >10 mm or depth >5 mm	Repair Procedure 1
Non-structural cosmetic	Chipping and spalling	All locations	Area <40 mm x 40 mm or depth <15 mm	Repair Procedure 1
Non-structural cosmetic	Chipping and spalling	All locations	Area ≥40 mm x 40 mm or depth ≥15 mm	Repair Procedure 2
Surface irregularities	Local protrusions	Unformed surfaces	>3 mm high	Stone rubbed or ground
Surface irregularities	Local protrusions	Formed surfaces	>1 mm high	Stone rubbed, check mould
Localized surface cracking and crazing		All locations	Cracks <0.2 mm wide	No repair
Structural crack		Radial joint faces	All cracks	Review for approval of repair procedure
Structural crack		Circumferential joint faces	Cracks ≤0.05 mm wide	No repair

Damage/Defect	Description	Location	Extent	Remedy
Structural crack		Circumferential joint faces	Cracks >0.05 mm and ≤2.0 mm wide not crossing gasket groove	Repair Procedure 3
Structural crack		All locations	Cracks >2.0 mm wide	Reject segment
Broken segment	Structural crack	Through segment	Full depth	Reject segment
Honeycombing		Circumferential and radial joint faces		Reject Segment
Honeycombing		Other locations		Repair Procedure 2
Structural damage	Damage exposing rebar	All locations except radial joint faces		Review for approval of repair procedure

Table developed through multiple construction contracts in the GTA and through reviews by contractors and pre-casters.

Table 8 Repair Procedures

Repair Procedure 1	
Material	Sikadur 31 Epoxy Mortar or approved alternative. Mix one-to-one ratio by volume with oven dried silica sand.
Procedure	Repair area shall be dry and clean. Place material into void and work material by trowel or spatula. Cure mortar at minimum temperature of 5°C.
Repair Procedure 2	
Material	MasterEmaco S-488 CI or approved alternative.
Procedure	Sawcut all edges of repair area to a depth of 15 mm. Break back to sound concrete and clean area. Soak the repair area to dampen for a period of two hours. Fill repair area with patching compound, Finish with steel trowel. Dampen and keep the repair area moist for a period of seven days.
Repair Procedure 3	
Material	Sikadur Injection Gel or approved alternative
Procedure	Clean injection site, avoiding contaminating the crack. Mix two components. Using putty knife or spatula, force material into the crack to seal it. Wipe excess resin from surface. After four hours of curing, finish area with a rub stone or grinder.

Table developed through multiple construction contracts in the GTA and through reviews by contractors and pre-casters.

4.2.3 Tunnels by TBM in Soft Ground and Mixed Face

4.2.3.1 General

a) Metrolinx shall determine the required internal diameter of the tunnels, for twin or single bore arrangement, based on transit vehicle performance, track form, alignment allowances, clearances, and design and construction requirements.

b) Alignment control shall be presented as a requirement to avoid an encroachment of the lining inside a circle of radius 100 mm less than that of the PCTL, with the circle centred on the theoretical alignment. The 100 mm requirement is for twin tunnel sized tunnels. For larger tunnels (e.g., SSE), this shall be increased to 125 mm.

c) Pressurized face TBMs shall be required for transit tunnels. To date, within the GTA, this requirement has consistently been met by Earth Pressure Balance TBMs in preference to Slurry TBMs.

d) Break-out and break-in arrangements at launch shafts and removal shafts shall be planned with attention given to potential reduced cover and resulting ground treatment requirements, to the initiation of face pressure at launch and to the maintenance of longitudinal compression on the lining during break-through.

4.2.3.2 Excavation

a) The TBMs used to excavate the tunnel shall be operated in pressurized mode throughout the length of the tunnel. Required face pressures shall be calculated by use of the methodology described in "Recommendation for Face Support Pressure Calculations for Shield Tunnelling in Soft Ground" DAUB German Tunnelling Committee (ITA-AITES) October 2016.

b) For EPB TBMs, soil conditioning agents shall be injected ahead of the cutterhead and into the excavation chamber to allow the excavated material to form a paste. This paste shall be able to transmit pressure to the excavation face and shall be able to dissipate this pressure through the length of the screw conveyor allowing discharge of excavated material at atmospheric pressure.

c) For Slurry TBMs, the face pressure shall be provided by the bentonite slurry conveying medium containing the excavated material, with pressure control provided by a compressed air bubble acting on the slurry. The conveying medium shall be pumped to the separation plant where the excavated material shall be extracted, and the slurry returned to the TBM. Some of the clays and cohesive tills, in the GTA, may present challenges for separation from the bentonite slurry.

d) For EPB TBMs, the annular space between the shield and the excavation cut line shall be filled with pressurized bentonite slurry to prevent inward ground movement towards the shield. For Slurry TBMs this annular space is filled as part of the face pressurization.

e) For both TBM types, the annular space between the tunnel lining and the excavation cut line shall be filled by pumping grout, through the tail shield around the tail seals, into the annulus. The grouting pressure shall be calculated as a minimum of hydrostatic pressure plus ground pressure plus 0.5 Bar. The grout shall be two-component, with the inert component pumped separately from the accelerator component, and with mixing achieved at the two grout pipe nozzles. The mixed grout shall attain an early strength of at least 0.1 MPa after 30 minutes.

f) For both TBM types, the face pressure shall be maintained at all times. This shall include weekend stoppages/shift exchanges and associated stoppages, and maintenance stoppages including those requiring an intervention where personnel are required to enter the excavation chamber. The pressure requirement for interventions may be able to be waived if the TBM face is fully contained within a pre-installed headwall or equivalent ground treatment, such as jet grouting.

g) For both TBM types, the metering of excavation quantities shall be required. For EPB TBMs this shall be achieved with at least two conveyor belt weighers and a muck volume profiler. For Slurry TBMs this would be achieved by metering quantities generated at the separation plant. For both machine types, the measurements shall be used to detect trends and thereby identify where over-mining is occurring.

h) The TBMs shall be able to deliver machine data to the overall data collection system that is monitoring ground and building movements. This integration shall happen in near real-time and shall be available to the shift engineer in the tunnel.

i) During EPB TBM tunnelling, soil samples shall be collected from the conveyor belt for each 10 m of tunnel advance of sufficient volume to allow grain size analysis to be conducted. These grain size curves shall be used, if required, to identify ground conditions at the tunnel face along the alignment. For Slurry TBMs a similar exercise shall be required at the separation plant to identify production rate of various grain sized materials.

4.2.3.3 Lining Installation

a) On removal from the on-site storage, during periods of ambient temperatures below 0°C, the lining segments shall pass through a heating facility to raise the segment temperature as required to avoid the freezing of the annulus grout on contact with the lining. The segments shall then be delivered to the tunnel level and to the TBM. The segments shall be transported to the segment erector on the TBM where they shall be picked up by the vacuum erector and placed into position in the lining ring.

b) The lining shall be installed so that the maximum lips between installed segments shall not exceed 5 mm. Other variations of adjacent segment joint surfaces shall be recorded as deficiencies are; gaps between segments (radially and circumferentially), jutting of gaskets, and ring rolls.

c) Any alignment corrections shall be made at the maximum rate of 3 mm per metre of tunnel length. Every 10th ring shall be checked for alignment conformance during tunnelling. The alignment conformance survey (of every 10th ring) shall be conducted at 200 m intervals of advance of the tunnel.

d) All dowels between the rings and bolts between the segments shall be installed. All bolts shall remain in place during and after construction completion. Guide rods shall be installed in the radial joints. Gaskets shall be lubricated during the installation process.

e) The size and quantity of cracks in the tunnel lining shall not exceed that required to ensure compliance with the required design life for the lining.

4.2.3.4 Secondary Waterproofing

a) Leaks that are found in the lining shall be addressed by appropriated grouting methods and tools to ensure that the water ingress limits are not exceeded for the design life of the lining.

4.3 Tunnels Excavated by Sequential Excavation Method (SEM)

4.3.1 Tunnels by SEM in Soft Ground and Mixed Face

4.3.1.1 General

a) The Sequential Excavation Method (SEM) or New Austrian Tunnelling Method (NATM) was developed for rock tunnels in Austria by L. von Rabcevicz, and L. Muller between 1957 and 1965. It uses the geological stress within the surrounding rock mass to stabilize the tunnel itself. It relies on the use of early application of shotcrete to the exposed rock to prevent loosening and excessive rock deformation. It requires the measurement of excavation deformations to guide the selection of relatively flexible support systems. Quickly closing the invert and creating a load bearing lining ring is important in maintaining excavation stability. More recently, this tunnelling method has been applied to soft ground tunnelling, using smaller excavation increments, with additional pre-support systems frequently employed and with each drift requiring timely closure of its initial lining ring.

b) The application of the Sequential Excavation Method (SEM) to soft ground in the GTA has had limited precedence, restricted to the Toronto-York Spadina Subway Extension (triple-drive SEM tunnels for a double-ended pocket track north of Finch Station), Eglinton Crosstown Light Rail Transit (ECLRT) and Highways 401/409 projects. On the ECLRT project SEM was applied on three stations from the total of 14 underground stations. These were Laird, Oakwood and Avenue Stations, where the geotechnical conditions were deemed to be suitable for the method. This highlights the importance of careful analysis of geotechnical conditions prior to the use of this technique. Further precedence can be found on the Highway 401/409 Tunnel project for Metrolinx, but this tunnel was special application of SEM where the ground was supported by an arch of large steel pipes prior to the tunnel excavation, allowing a much larger segmentation of the excavation face than in typical SEM in soft ground.

c) The excavations created on the ECLRT project were large, with overall excavated widths of 19.4 m for the platform tunnel, 15.5 m for the crosscut and excavated heights of 15.6 m for the platform tunnel, 18.0 m for the crosscut. The crosscut is the tunnel from the off-street shafts to access the platform tunnel headings.

d) The excavations for the Highway 401/409 Tunnel were also large, with overall excavated widths of 9.9 m for each of the two tunnels and excavated height of 11.4 m, not including the 0.8 m diameter pipes used for the pre-support arch. This project featured a very shallow cover, above the pre-support, of less than 3 m to the 21 lanes of the highways passed under by the tunnels.

4.3.1.2 Excavation and Initial Support

a) The excavation support system shall be based on a two-pass lining arrangement with the initial lining and final lining separated by a waterproofing system.

b) Prior to any excavation work by SEM in soft ground, dewatering shall be instituted from the surface (or from preinstalled TBM tunnels where they exist, through station sites). The ground water level shall be lowered to below the invert of the lowest excavation. Other ground treatment methods shall be used where dewatering is not practicable.

c) The tunnel headings for SEM in soft ground or mixed face shall be divided into drifts. For small access or ventilation tunnels, the division shall likely be into a top heading and a bench. For larger cross section tunnels, such as station platform tunnels, the divisions shall include two side drifts, that shall be subdivided into top heading, bench and invert, and a central drift that shall be subdivided into top heading, bench and invert. For intermediate-sized tunnels it may be possible to select a main heading with a single side drift, both divided into top heading, bench and invert. The selection of the multiple drift arrangement is usually based on experience. However, with recent development of Finite Element or Finite Difference 3-D analysis methods it is now possible to test and confirm or refine these selections. These analysis methods shall be used to ensure the stability of the excavation and that ground movements are controlled within the required limits.

d) Excavation shall start in the top headings. If required, the headings shall be advanced under the protection of spiling, un-grouted or grouted, or grouted pipe umbrella. The grouted pipe umbrella shall be installed as an arch, either in a single or double row to form a Barrel Vault or Canopy Tube Arch. In particularly problematic ground conditions, horizontal jet grouting shall be employed. The canopy tubes shall be installed at a shallow angle (approximately 5 degrees) with the heading alignment, with a typical reset at 10 m to 15 m intervals. This shall create a saw-toothed excavation surface, in the longitudinal direction, that later shall be filled with shotcrete.

e) In some cases, it may be necessary to widen the bottom of the walls of the top heading (forming “elephant’s feet”) to increase the bearing capacity of the lining.

f) Face support may be required by the incorporation of an earth wedge in front of the face, or by spraying shotcrete on the face, or face bolting. The need for face support may be reduced by the use of pocket (small area) excavation of the face.

g) The excavation round length shall vary with each type of heading but shall approximate to the following typical lengths: for top headings: 1.0 m initial, 1.5 m sustained; and for bench and invert: 2.0 m initial, 3.0 m sustained.

h) Shotcrete shall be applied to the excavation surface, initially as a thin flashcrete sealing and smoothing layer, followed by two applications of shotcrete to make up the full initial lining thickness. This shall be either steel mesh or steel fibre reinforced. The shotcrete surface shall be smooth enough to be suitable to receive the geotextile and membrane waterproofing system. Lattice girders shall be included for shape control or omitted if the lining profile is provided by survey means.

i) Field test panels, at least 1.5 m x 1.5 m in area x (representative thickness to applied thickness), shall be produced in vertical and overhead shotcreting positions. Nozzlemen shall be certified in accordance with ACI standards. The shotcrete thickness shall be demonstrated by the use of LiDAR scanning of the excavated surface and the completed shotcrete surface or by the use of Geotechnical Monitoring LiDAR or equivalent.

j) Monitoring of ground and lining movements shall be important to detect whether these are continuing to develop or have stabilized. Multi-point extensometers shall be installed above the excavations. Survey targets shall be embedded in the shotcrete lining to monitor movement of the lining. Other geotechnical instruments and building movement monitoring systems shall be installed as required.

4.3.1.3 Waterproofing

a) A membrane shall be installed between the initial and final tunnel lining. The membrane shall be polyvinyl chloride (PVC), or flexible polyolefin (FPO) based polymeric sheet. This shall be installed in single or double layers. The membrane and its appurtenances shall be certified by the manufacturer that no components can leach out during the design life and deleteriously affect durability and resistance to the passage of groundwater. The membrane thickness shall be in accordance with the manufacturer’s recommendation based on location-specific groundwater, gas and contaminant conditions and on required water tightness criteria. Sprayed-on membranes shall not be used.

b) The membrane shall be protected from damage by a geotextile fleece and shall be resistant to physical or mechanical degradation during handling. The fleece shall be attached to the shotcrete by PVC rondels as recommended by the manufacturer. The membrane shall be safe to apply, non-toxic and fire retardant. Joints in the membrane shall be overlapped, and heat welded with parallel welds to create a chamber for pressure testing of the joint.

c) The membrane shall be divided into compartments by water barriers, heat welded to the membrane. The compartments shall be >20 m^2 and <200 m^2 in area. The compartments shall be equipped with grout injection tubes and ports at spacings recommended by the manufacturer. The number of injection tubes shall be sufficient to allow grouting of the full extent of the compartment. Perforated injection tubes shall also be installed along the water barriers.

4.3.1.4 Final Lining

a) The final linings shall be placed by conventional means, likely using hydraulic steel-faced forms custom fabricated for the project. Concrete shall be placed by pumping, with a slickline located in the top of the form. Reinforcement shall be steel rebar or steel fibres.

b) In some instances, it may be possible to install the final lining by means of shotcrete. This shall only be acceptable, where the as-sprayed surface finish is permitted, likely in areas not visible to the public. Only wet-mix shotcrete shall be used in the final lining. The shotcrete materials shall provide sufficient strength and durability to meet the required design life. In addition to the test panels, described above for the initial lining, mock-up panels at least 2.5 m x 2.5 m x (the representative final lining thickness) shall be produced including reinforcement, the waterproof membrane and the surface finish to demonstrate lining quality. At least five mock-up panels shall be produced for overhead spray position and five for vertical position. The evaluation of the mock-up panels shall demonstrate constructability, integrity and physical properties of the as-shot lining.

c) In areas where the final lining shall be potentially exposed damage from a train fire, mono-filament polypropylene fibres shall be included in the concrete mix.

4.3.2 Tunnels by SEM in Rock

4.3.2.1 General

a) The application of the SEM to in rock in the GTA has had limited precedence, restricted to the Billy Bishop Toronto City Airport (BBTCA) Pedestrian Tunnel. However, this project was not a true application of SEM since the project used a pre-installation of interlocking concrete-filled tunnels to form an arch to provide support prior to excavating the tunnels. This avoided the need to segment the excavation face, beyond heading and bench, and to provide rock support, beyond rock bolts and a nominal shotcrete layer.

b) The excavation for the BBTCA Pedestrian Tunnel is quite large, with an excavated width of 10.2 m and an excavated height of 7.6 m, (not including the 1.8 m concrete filled tunnels above). The excavation was in Georgian Bay Shale and the tunnel is located below the Western Channel section of Lake Ontario on the waterfront of the City of Toronto.

c) Challenges for the application of SEM in Georgian Bay Shale in the GTA include high in situ horizontal stresses, time dependent deformation, presence of bedding joints, weak bedding planes and subvertical joints, slabbing, spalling, overbreak, exposed surface degradation and a requirement for probing ahead to detect the presence of sudden changes in rock quality or of alluvium-filled and water-logged buried valleys.

4.3.2.2 Excavation and Initial Support

a) The excavation support system shall be based on a two-pass lining arrangement with the initial lining and final lining separated by a watertight membrane.

b) Dewatering is generally not required for SEM tunnels in rock unless the rock is very fractured or weathered. However, in the GTA, significant water inflows shall be managed to avoid degradation of the shale bedrock into slurry, particularly in the invert.

c) The tunnel headings for SEM in rock shall be divided into drifts. However, these drifts generally shall be larger than those for soft ground. For small access or ventilation tunnels, the division shall likely be into a top heading and a bench, or full face. For larger cross section tunnels, such as station platform tunnels, the divisions shall likely include a sidewall drift, that shall be subdivided into top heading, bench and invert, and a main drift that shall be subdivided into top heading, bench and invert. The selection of the multiple drift arrangement shall be based on experience, supplemented by Finite Element or Finite Difference analysis to ensure stability of the excavation and that ground movements are controlled within required limits.

d) Excavation shall start in the top headings. If required, the headings shall be advanced under the protection of spiling, un-grouted or grouted. Wall support shall be provided by rock bolts and shotcrete.

e) In some cases, it may be necessary to widen the bottom of the walls of the top heading (forming “elephant’s feet”) to increase the bearing capacity of the lining.

f) Face support may be required by the use of rock bolts or spraying of shotcrete on the face. The excavation round length shall vary with each type of heading and shall be determined by rock standup times and strength.

g) Shotcrete shall be applied to the excavation surface, initially as a 50 mm thick flashcrete sealing layer, followed by one or two applications of shotcrete to make up the full initial lining thickness. This shall be either steel mesh or steel fibre reinforced. The shotcrete surface shall be smooth enough to be suitable to receive the geotextile and membrane waterproofing system. Lattice girders shall be included for shape control or omitted if the lining profile is provided by survey means.

h) Field test panels, at least 1.5 m x 1.5 m in size x representative to applied thickness, shall be produced in vertical and overhead shotcreting positions. Nozzlemen shall be certified in accordance with ACI standards. The shotcrete thickness shall be demonstrated by the use of LiDAR scanning of the excavated surface and the completed shotcrete surface or by the use of Geotechnical Monitoring LiDAR or equivalent.

i) Monitoring of ground and lining movements shall be important to detect whether these are continuing to develop or have stabilized. Multi-point extensometers shall be installed above the excavations. Survey targets shall be embedded in the shotcrete lining to monitor movement of the lining. Other geotechnical instruments and building movement monitoring systems shall be installed as required.

4.3.2.3 Waterproofing

a) A membrane shall be installed between the initial and final tunnel lining. The membrane shall be PVC or FPO based polymeric sheet. This shall be installed in single or double layers. The membrane and its appurtenances shall be certified by the manufacturer that no components can leach out during the design life and deleteriously affect durability and resistance to the passage of groundwater. The membrane thickness shall be in accordance with the manufacturer's recommendation based on location-specific groundwater, gas and contaminant conditions and on required water tightness criteria. Sprayed-on membranes shall not be used.

b) The membrane shall be protected from damage by a geotextile fleece and shall be resistant to physical or mechanical degradation during handling. The fleece shall be attached to the shotcrete by PVC rondels as recommended by the manufacturer. The membrane shall be safe to apply, non-toxic and fire retardant. Joints in the membrane shall be overlapped, and heat welded with parallel welds to create a chamber for pressure testing of the joint.

c) The membrane shall be divided into compartments by water barriers, heat welded to the membrane. The compartments shall be $>20\ m^2$ and $<200\ m^2$ in area. The compartments shall be equipped with grout injection tubes and ports at spacings recommended by the manufacturer. The number of injection tubes shall be sufficient to allow grouting of the full extent of the compartment. Perforated injection tubes shall also be installed along the water barriers.

4.3.2.4 Final Lining

a) The final linings shall be placed by conventional means, likely using hydraulic steel-faced forms custom fabricated for the project. Concrete shall be placed by pumping, with a slickline located in the top of the form. Reinforcement shall be steel rebar or steel fibres.

b) In some instances, it may be possible to install the final lining by means of shotcrete. This shall only be acceptable, where the as-sprayed surface finish is permitted, likely in areas not visible to the public. Only wet-mix shotcrete shall be used in the final lining. The shotcrete materials shall provide sufficient strength and durability to meet the required design life. In addition to the test panels, described above for the initial lining, mock-up panels at least 2.5 m x 2.5 m x the representative final lining thickness shall be produced including reinforcement, the waterproof membrane and the surface finish to demonstrate lining quality. At least five mock-up panels shall be produced for overhead spray position and five for vertical position. The evaluation of the mock-up panels shall demonstrate constructability, integrity and physical properties of the as-shot lining.

c) In areas where the final lining shall be potentially exposed damage from a train fire, mono-filament polypropylene fibres shall be included in the concrete mix.

d) The design of the initial and final linings shall consider the time dependent deformations of the Georgian Bay Shale, if applicable.

4.4 Tunnels Excavated by Other Methods

4.4.1 Tunnels by Other Methods in Soft Ground or Mixed Face

4.4.1.1 General

a) This section addresses smaller tunnels that are part of transit facilities and shall be constructed by a variety of methods. These tunnels include cross-passages, between twin tunnels, ventilation and access tunnels associated with stations, and tunnels to connect to Emergency Exit Buildings (EEBs).

4.4.1.2 Excavation and Initial Support

a) The excavation support system shall be based on a two-pass lining arrangement with the initial lining and final lining separated by a watertight membrane.

b) Prior to any excavation work, dewatering shall be instituted from the surface. The ground water level shall be lowered to below the invert of the lowest excavation. This is particularly important in the context of the running tunnels having been constructed with a pressurized TBM, with no problems, but these smaller tunnels being excavated in the absence of a pressurized face. Other ground treatment methods shall be used where dewatering is not practicable.

c) There are several options for the initial linings of these smaller tunnels. In the past, for cross passages, steel liner plates have been used supported by steel ribs. The excavation has been staged with an initial smaller pilot tunnel installed and supported by liner plates, followed by an enlargement tunnel to the required diameter excavation, again supported by liner plates. More recently, methods similar to SEM, have been used with shotcrete as the initial lining material, usually installed in a heading and bench excavation configuration. The selection of the multiple drift arrangement shall be based on experience, supplemented by Finite Element or Finite Difference analysis to ensure stability of the excavation and that ground movements are controlled within required limits. If required, the heading shall be advanced under the protection of spiling, un-grouted or grouted.

d) These methodologies are acceptable and shall be selected based on geotechnical conditions and constructor preference and familiarity. In all cases the initial lining shall be completed with an internal surface that is suitable for the installation of a waterproofing membrane. The constructor shall propose other methodologies to Metrolinx if these are considered preferred.

4.4.1.3 Waterproofing

a) A membrane shall be installed between the initial and final tunnel lining. The membrane shall be PVC or FPO based polymeric sheet. This shall be installed in single or double layers. The membrane and its appurtenances shall be certified by the manufacturer that no components can leach out during the design life and deleteriously affect durability and resistance to the passage of groundwater. The membrane thickness shall be in accordance with the manufacturer’s recommendation based on location-specific groundwater, gas and contaminant conditions and on required water tightness criteria. Sprayed-on membranes shall not be used.

b) The membrane shall be protected from damage by a geotextile fleece and shall be resistant to physical or mechanical degradation during handling. The fleece shall be attached to the initial lining by PVC rondels as recommended by the manufacturer. The membrane shall be safe to apply, non-toxic and fire retardant. Joints in the membrane shall be overlapped, and heat welded with parallel welds to create a chamber for pressure testing of the joint.

c) Most of the subject tunnels shall be too small to need the membrane to be compartmentalized. However, if large enough, the following shall apply. The membrane shall be divided into compartments by water stops, heat welded to the membrane. The compartments shall be >50 m^2 and <100 m^2 in area. The compartments, even if only a single compartment, shall be equipped with grout injection tubes and ports at spacings recommended by the manufacturer. Perforated injection tubes shall also be installed along the water stops.

d) Most of these tunnels shall involve a joint between the small tunnel and the running tunnel. This involves an interface between a cast-in-place lining and a precast concrete lining making this a particularly difficult joint to waterproof. Typical measures have included the inclusion of a perforated groutable tube within the joint, hydrophilic gaskets within the joint, and clamping of the membrane around the entrance door frame, or a combination of all of these.

4.4.1.4 Final Lining

a) The final linings shall be placed by conventional means, likely using plywood-faced forms custom fabricated for the project. Concrete shall be placed by pumping, with a slickline located in the top of the form. Reinforcement shall be steel rebar or steel fibres.

b) In some instances, it may be possible to install the final lining by means of trowelled shotcrete. This shall only be acceptable if the trowelled surface is reasonably equivalent, in the opinion of Metrolinx, to that produced by conventional formwork. Only wet-mix shotcrete shall be used in the final lining. The shotcrete materials shall provide sufficient strength and durability to meet the required design life.

4.4.2 Tunnels by Other Methods in Rock

4.4.2.1 General

a) This section addresses smaller tunnels that are part of transit facilities. These tunnels include cross passages, between twin tunnels, ventilation and access tunnels associated with stations, and tunnels to connect to EEBs.

4.4.2.2 Excavation and Initial Support

a) The excavation support system for these smaller tunnels shall be based on a two-pass lining arrangement with the initial lining and final lining separated by a watertight membrane.

b) Dewatering is generally not required for smaller tunnels in rock unless the rock is very fractured or weathered. However, in the GTA, significant water inflows shall be managed to avoid degradation of the shale bedrock into slurry, particularly in the invert.

c) The tunnel headings for smaller tunnels in rock likely shall be divided into top heading and bench or shall be excavated full face. The selection of the multiple drift or single drift arrangement shall be based on experience, supplemented by Finite Element or Finite Difference analysis to ensure stability of the excavation and that ground movements are controlled within required limits. If required, the headings shall be advanced under the protection of spiling, un-grouted or grouted. Wall support shall be provided by rock bolts and shotcrete.

d) Face support may be required by the use of rock bolts or spraying of shotcrete on the face. The excavation round length shall vary with each type of heading and shall be determined by rock standup times and strength.

e) Shotcrete shall be applied to the excavation surface, initially as a 50 mm thick flashcrete sealing layer, followed by one or two applications of shotcrete to make up the full initial lining thickness. This shall be either steel mesh or steel fibre reinforced. The shotcrete surface shall be smooth enough to be suitable to receive the geotextile and membrane waterproofing system. Lattice girders shall be included for shape control or omitted if the lining profile is provided by survey means.

f) Field test panels, at least 1.5 m x 1.5 m in size x representative to applied thickness, shall be produced in vertical and overhead shotcreting positions. Nozzlemen shall be certified in accordance with ACI standards. The shotcrete thickness shall be demonstrated by the use of LiDAR scanning of the excavated surface and the completed shotcrete surface or by the use of Geotechnical Monitoring LiDAR or equivalent.

4.4.2.3 Waterproofing

a) A membrane shall be installed between the initial and final tunnel lining. The membrane shall be PVC or FPO based polymeric sheet. This shall be installed in single or double layers. The membrane and its appurtenances shall be certified by the manufacturer that no components can leach out during the design life and deleteriously affect durability and resistance to the passage of groundwater. The membrane thickness shall be in accordance with the manufacturer's recommendation based on location-specific groundwater, gas and contaminant conditions and on required water tightness criteria. Sprayed-on membranes shall not be used.

b) The membrane shall be protected from damage by a geotextile fleece and shall be resistant to physical or mechanical degradation during handling. The fleece shall be attached to the initial lining by PVC rondels as recommended by the manufacturer. The membrane shall be safe to apply, non-toxic and fire retardant. Joints in the membrane shall be overlapped, and heat welded with parallel welds to create a chamber for pressure testing of the joint.

c) Most of the subject tunnels shall be too small to need the membrane to be compartmentalized. However, if large enough, the following shall apply. The membrane shall be divided into compartments by water stops, heat welded to the membrane. The compartments shall be >50 m^2 and <100 m^2 in area. The compartments, even if only a single compartment, shall be equipped with grout injection tubes and ports at spacings recommended by the manufacturer. Perforated injection tubes shall also be installed along the water stops.

d) Most of these tunnels shall involve a joint between the small tunnel and the running tunnel. This involves an interface between a cast-in-place lining and a precast concrete lining making this a particularly difficult joint to waterproof. Typical measures have included the inclusion of a perforated groutable tube within the joint, hydrophilic gaskets within the joint, and clamping of the membrane around the entrance door frame, or a combination of all of these.

4.4.2.4 Final Lining

a) The final linings shall be placed by conventional means, likely using plywood-faced forms custom fabricated for the project. Concrete shall be placed by pumping, with a slickline located in the top of the form.

Reinforcement shall be steel rebar or steel fibres.

b) In some instances, it may be possible to install the final lining by means of trowelled shotcrete. This shall only be acceptable if the trowelled surface is reasonably equivalent, in the opinion of Metrolinx, to that produced by conventional formwork. Only wet-mix shotcrete shall be used in the final lining. The shotcrete materials shall provide sufficient strength and durability to meet the required design life.

4.5 Face Loss Percentage and Settlement Trough Width for Mined Tunnels

4.5.1 Settlement Prediction Methodology

Figure 2 presents the idealized surface settlement half curve, transverse to the tunnel alignment, with the complete profile being symmetrical about the tunnel centreline. This was developed by Schmidt B. in 1969. This curve is the basis for the empirical method of calculating settlements due to tunnelling. The curve is defined by the following base equations for vertical displacement:

$$S = S_{max} \times \exp(-y^2/2i^2)$$

$$V_s = (2\pi)^{0.5} \times i \times S_{max}\text{, or } V_s = 2.5 \times i \times S_{max}$$

$$i = K \times z_o$$

$$\text{Trough width} = 6 \times i$$

where, S= vertical displacement.(mm)

S_{max} = maximum vertical displacement at the tunnel centreline (mm)

V_s = volume of the settlement trough (per unit advance) i.e., area of trough (mm^2)

y = transverse distance from the tunnel centreline (mm)

K = trough width parameter

z_o = depth, surface to tunnel axis level (mm)

a) K, the trough width parameter, varies according to the ground conditions, but can be assumed to be 0.5 in cohesive deposits and 0.4 in cohesionless soils.

4.5.1.2 The face loss percentage is a required initial input parameter to allow the calculation of predictions of surface settlement and building movements resulting from tunnelling. The face loss percentage is the area of the settlement trough, for a two-dimensional slice, expressed as a percentage of the tunnel excavation face area. This parameter is established from data recorded during previous similar projects and is a function of construction methods and geotechnical conditions. The use of the loss percentage is shown in the reorganized equation below:

$$S_{max} = (D^2 \times \pi/4 \times f) / (100 \times 2.5 \times i)$$

where, D = excavated diameter of tunnel (mm)

f = face loss percentage of tunnel excavated area (%)

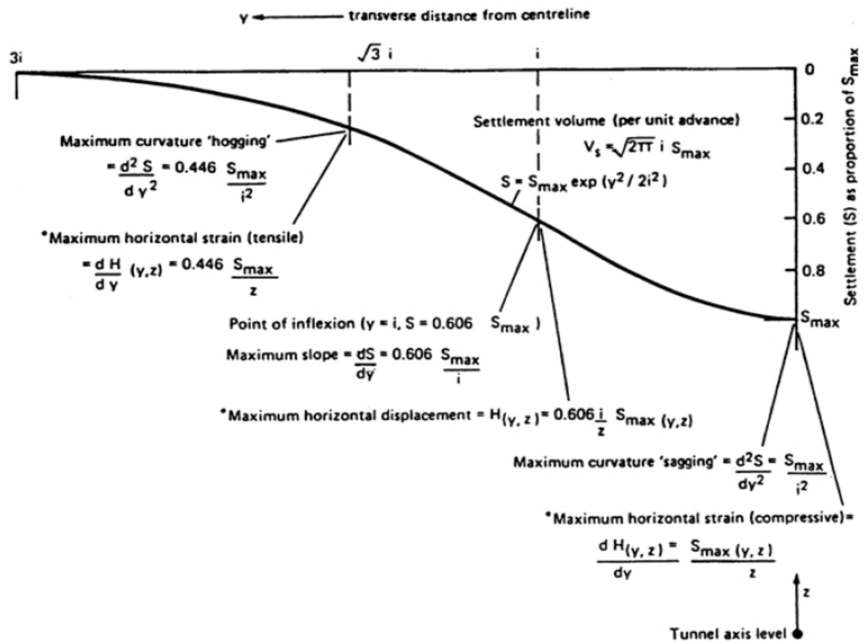


Figure 2 Idealized Transverse Settlement Curve (from Lake, Rankin and Hawley)

4.5.2 Zone of Influence

4.5.2.1 The Zone of Influence (ZOL) shall be defined as the limits of the settlement trough. The width of the settlement trough is usually defined as 6 x i. The limit of the zone of influence can also be defined by the 1 mm settlement contour.

4.5.3 Historical Performance

4.5.3.1 Table 9 presents the available records of face loss percentages that have been achieved on transit tunnels in the GTA since the introduction of pressurized face TBMs.

4.5.3.2 It should be noted that the face losses presented show a marked improvement from those shown in earlier reference documents. For instance, O'Reilly and New (1982), and Lake, Rankin and Hawley (1992) both refer to pressurized face machines in granular material below the water table achieving 1 - 10% face loss, the FHWA Technical Manual for Design and Construction of Road Tunnels (2009) describes face losses from 0.5% to 1.0% for usual practice in slow ravelling or squeezing ground with higher values for more difficult ground or worse practices, and the Singapore Land Transport Authority (LTA) Civil Design Criteria (2019) refers to several projects with EPB TBMs that achieved 1% to 3% volume losses.

4.5.3.3 This improvement is assisted somewhat by the tunnel-favourable geotechnical conditions that exist in many parts of the GTA. It is also clear from the data that settlement control performance has continued to improve since the introduction of EPB TBM's on the Sheppard Subway project, with face losses now consistently below 0.4%, and generally in the 0.1% to 0.3% range.

Table 9 GTA Historical Performance for Face Loss Percentages

Year	Project	Tunnel Drive	Geotechnical Conditions	Average Face Loss (%)
1997-1999	Sheppard Subway Twin Tunnels 5.2 m ID EPB TBMs	West Drive - North Tunnel	Hard Till	0.3
			Granular	0.8
			Mixed Soils	0.7
		West Drive - South Tunnel	Hard Till	0.2
			Granular	0.3
			Mixed Soils	0.6
		East Drive - North Tunnel	Soft Till	0.3
			Fill	1.6
		East Drive - South Tunnel	Soft Till	0.2
			Fill	1.2
2011-2013	TYSSE North Twin Tunnels 5.4 m ID EPB TBMs	North Bound - LS1 to ES1	Lower Till	0.18
			Upper Granular	0.10
			Mixed Soils	0.33
		South Bound - LS1 to ES1	Lower Till	0.17
			Upper Granular	0.06
			Mixed Soils	0.08
		North Bound - LS2 to ES2	Lower Till	0.18
		South Bound - LS2 to ES2	Lower Till	0.18
		North Bound - LS3 to ES3	Lower Till	0.38
			Mixed Soils	0.27
		South Bound - LS3 to ES3	Lower Till	0.36
			Mixed Soils	0.21
2011-2013	TYSSE South Twin Tunnels 5.4 m ID EPB TBMs	North Bound - LS4 to ES4	Upper Till	0.4
			Lower Till	0.2
			Mixed Soils	0.2
		South Bound - LS4 to ES4	Upper Till	0.3
			Lower Till	0.3
			Mixed Soils	0.2
		North Bound - LS5 to ES5	Upper Till	0.3
			Mixed Soils	0.2
			Fill	0.3
		South Bound - LS5 to ES5	Upper Till	0.2
			Mixed Soils	0.2
			Fill	0.3
2013-2015	ECLRT (West)	Drives 1 & 2 - LS1 to ES1	Learning Curve (Clay)	0.40

Year	Project	Tunnel Drive	Geotechnical Conditions	Average Face Loss (%)
	Twin Tunnels 5.75 m ID EPB TBMs		Clay	0.29
			Sand	0.27
			Sand with Silt Till Cap	0.13
			Mixed Soils with Clay Cap	0.03
			Sand with Clay Cap	0.08
			Mixed Soils with Clay Cap	0.02
		Drives 3 & 4 - LS2 to ES2	Clay	0.09
			Mixed Soils with Clay Cap	0.10
			Sand with Fill Cap	0.42
			Sand with Clay Cap	0.11
			Sand	0.12

From Close Out Reports for Sheppard Subway, TYSSE and Eglinton Crosstown LRT tunnel projects.

4.5.4 Influences on Settlement

4.5.4.1 It important to consider other critical factors such advancing technology in TBMs, ground control success with bentonite injection around the front shield, and the role of tunnel diameter when considering face loss percentages.

4.5.4.2 To avoid bias, it should be note that the historic compiled face loss data are based on older technologies and more conservative approaches. Users should make sure new data, new technology and methods resulting in new achievements are considered in making decisions about face loss percentages.

4.5.4.3 It should be note that with the pressurized face TBMs and advancing technologies, the focus has shifted from “expected face loss percentages” to monitoring ground behaviour at the source, prevention of ground loss by actively maintaining a pressurized envelope environment from the face to tail as well as near real-time monitoring of TBM sensors and other geotechnical instruments to achieve ground and groundwater control objectives.

4.5.4.4 For context to Table 9 , the following provides general details of the methodologies employed on the referenced projects:

a) Sheppard Subway - Second project using EPB technology in Canada. Annulus grouting through the segments. Soil conditioning generally by trial and error, probably overdosed since spoil was very wet and created disposal challenges. Rectangular segments were used - ring length 1.4 m. Lining ID 5.2 m.

b) TYSSE - EPB TBMs. Annulus grouting through the tail shield. Injection of bentonite around shield was used for tunnelling under Schulich Building. More mature application of soil conditioning - no issues with spoil disposal. Universal Ring used with trapezoidal segments - ring length 1.5 m. Lining ID 5.4 m.

c) Eglinton Crosstown LRT - EPB TBMs. Annulus grouting through the tail shield. Some injection of bentonite around shield. Mature application of soil conditioning - no issues with spoil disposal. Universal Ring used with trapezoidal segments - ring length 1.5 m. Lining ID 5.75 m.

4.5.4.5 These improvements have allowed such projects as the Alaskan Way Replacement Tunnel, that incorporated all these technologies, as well as a large tunnel diameter, to achieve average face losses of less than 0.1%.

4.6 Tunnels Excavated by Cut and Cover

4.6.1 Tunnels by Cut and Cover, or Shafts, in Soft Ground

4.6.1.1 General

a) Cut and cover is usually selected as the construction methodology for transit facilities in soft ground for the following reasons:

1) The alignment is too shallow to allow tunnelling methods.

2) The running tunnel(s) are within a TBM launch or receiving shaft.

3) The running tunnel(s) are within a portal forming a transition from underground to surface or elevated alignment.

4) The transit facility is a station structure where SEM tunnelling methods cannot be used.

5) The transit facility is a structure that accesses the surface, such as an EEB or ventilation shaft.

4.6.1.2 Excavation Support

a) Historically, Support of Excavation (SOE), for cut and cover excavations for transit facilities, has been supplied by steel piles and timber lagging, combined with generalized dewatering. More recently, this methodology has been replaced by secant piling, that has become the standard for transit facilities in the GTA. Transit facilities typically require long narrow excavations and therefore propping is provided by struts across the excavation in preference to tiebacks. However, in situations such as TBM launch or receiving shafts, where an unimpeded internal space is required, tiebacks shall be used.

b) The installation of secant piles shall require the temporary or permanent relocation of utilities that would be impacted by the secant pile wall. In many instances, the SOE shall be required to support street decking at the surface to restore surface road capacity. The design of the SOE shall include for the effects of frost action, creating increased lateral loads, and for the development of falling ice hazards.

c) An alternative to secant piles, is a diaphragm wall system. This consists of the excavation of the wall as a deep trench supported by bentonite slurry. The excavated material shall be separated through a separation plant similar to that required for a slurry TBM. A rebar cage is lowered into the trench. Concrete is tremied into the trench, displacing the bentonite slurry, thereby forming a reinforced concrete wall. Diaphragm walls are rarely used in transit facilities in the GTA, except for the streetcar tunnel on Bay Street south of Union Station. Diaphragm walls are not economical, being more than twice the cost of secant piles, unless the walls form part of the permanent structure.

4.6.1.3 Excavation

a) Excavation is undertaken in stages with vertical steps determined by the vertical spacing of the propping. The first excavation step shall usually involve the installation of deck beams, for support of decking and partially acting as struts, and excavation around utilities that are supported in place. Struts or tiebacks shall then be installed, and similar excavation/propping steps shall be undertaken until the base of the excavation is reached.

b) Strutted excavations tend to negate the use of an excavation ramp, with excavated materials removed by cranes hoisting muck boxes.

4.6.1.4 Structure

a) Generally, cut and cover structures in soft ground are constructed starting from the base slab and working upwards, incrementally completing walls and slabs. The structures are typically poured against the SOE, acting as external formwork, with internal plywood-faced formwork. Finally, the structures are backfilled, and the ground surface restored. This process is referred to as “bottom-up” and is the methodology typically employed for transit facilities in the GTA.

b) The “top-down” construction process, although having limited use in the GTA, has seen significant application in Europe and elsewhere in the world. This process is generally used in combination with diaphragm walls. The process consists of the placement of the top slab supported on unexcavated ground. This slab is structurally tied into the diaphragm walls and includes a temporary opening for removal of excavated material. The ground to the soffit level of the next lower slab is then excavated. The slab placement and excavation sequence is then repeated until the base slab is completed. Internal supports for vertical loads, if required, are provided by piles installed prior to the structure placement. This process avoids the need for SOE and excavation propping, with both functions provided by the permanent structure. The negatives of the process are the need for complex connections, to transfer shears and bending moments, between the slabs and diaphragm walls, and the reliance on the diaphragm walls and their connections with the top and base slabs for the waterproofing of the structure.

c) In both “bottom-up” and “top-down” processes the structures shall be reinforced with rebar, but in areas where the final lining shall be potentially exposed damage from a train fire, mono-filament polypropylene fibres shall be included in the concrete mix.

4.6.1.5 Waterproofing

a) For the “bottom-up” construction process, the structures shall be designed as waterproof structures with a waterproofing membrane placed on the mud slab and against the SOE.

b) As noted above, for the top-down construction process, the waterproofing shall be supplied by the diaphragm walls, and their connections with the top and bottom slabs. The installation of membranes for the top and bottom slabs shall be practicable but is not possible for the diaphragm walls.

4.6.2 Tunnels by Cut and Cover, or Shafts, in Rock

4.6.2.1 General

a) Cut and cover is usually selected as the construction methodology for transit facilities in rock for the following reasons:

1) The alignment is too shallow to allow tunnelling methods.

2) The running tunnel(s) are within a TBM launch or receiving shaft.

3) The running tunnel(s) are within a portal forming a transition from underground to surface or elevated alignment.

4) the transit facility is a station structure where SEM tunnelling methods cannot be used.

5) The transit facility is a structure that accesses the surface, such as an EEB or ventilation shaft.

b) In all cases in the GTA, cut and cover excavations in rock shall include an excavation in soft ground for the upper portion of the overall excavation. The SOE for this upper excavation shall be propped secant piles. The front face of the secant pile wall shall align with the rock excavation face or can be stepped back. The embedment for the secant piles shall be to competent rock.

4.6.2.2 Excavation Support

a) SOE for rock excavation, in the GTA, shall consist of a grid of rock bolts and a flash coat of shotcrete or other spalling protection layers. Specific rock wedges that have been identified shall have custom designed rock bolting to prevent wedge mobilization. A smoothing layer of shotcrete shall be required in areas that are to receive a waterproofing membrane. Insulation materials and water ingress control may be used to reduce ice build-up.

4.6.2.3 Excavation

a) Blasting for rock excavation shall not be permitted for transit facilities in the GTA. The Georgian Bay Shale shall be broken up with an excavator-mounted impact breaker and excavated with a typical bucket excavator. Water ingress shall be controlled since the shale will degrade to a slurry in the base of the excavation in the presence of water.

4.6.2.4 Structure

a) Generally, cut and cover structures in rock are constructed starting from the base slab and working upwards, incrementally completing walls and slabs. The structures are typically poured against the SOE, acting as external formwork, with internal plywood-faced formwork. This process is referred to as "bottom-up" and is the methodology typically employed for transit facilities in the GTA.

b) In both "bottom-up" and "top-down" processes the structures shall be reinforced with rebar, but in areas where the final lining shall be potentially exposed damage from a train fire, mono-filament polypropylene fibres shall be included in the concrete mix.

4.6.2.5 Waterproofing

a) For the "bottom-up" construction process, the structures shall be designed as waterproof structures with a waterproofing membrane placed on the mud slab and against the SOE.

4.7 Settlement Prediction for Cut and Cover Excavation

4.7.1 Canadian Foundation Engineering Manual

4.7.1.1 The Canadian Foundation Engineering Manual, 4th Edition (CFEM) presents approaches for estimating settlement adjacent to vertical excavations. Figure 3, Figure 4, and Figure 5 are excerpted from CFEM and present data collected by Clough and O'Rourke (1990).

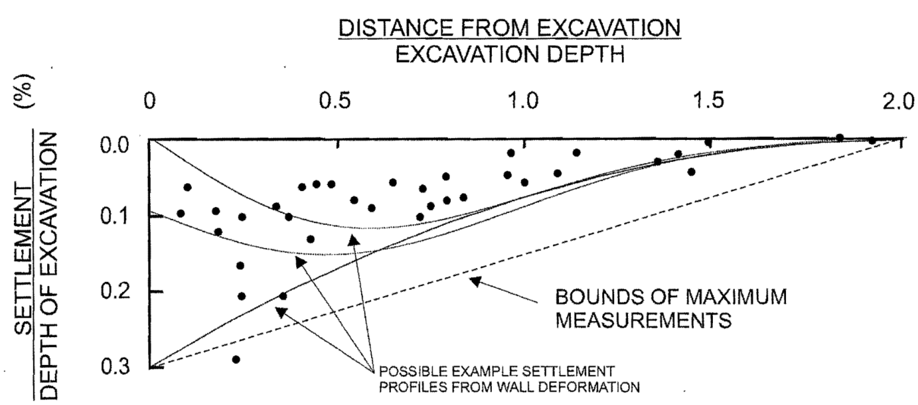


Figure 3 Settlements Adjacent to Excavations in Sand (from CFEM)

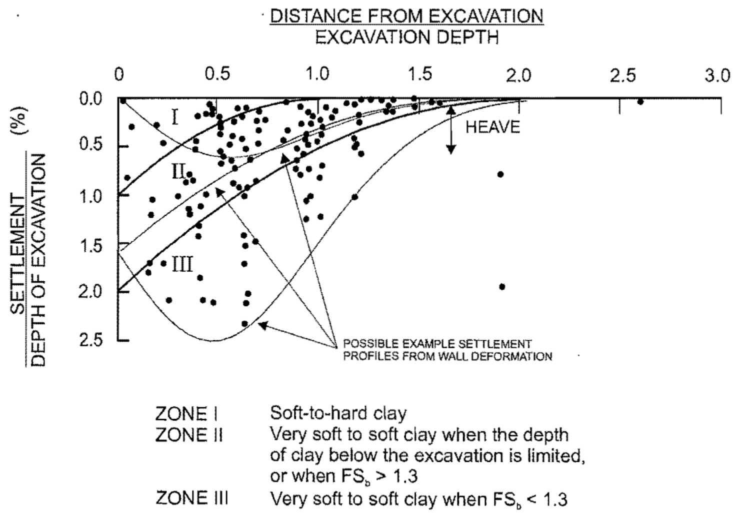


Figure 4 Settlements Adjacent to Excavations in Soft to Medium Stiff Clay (from CFEM)

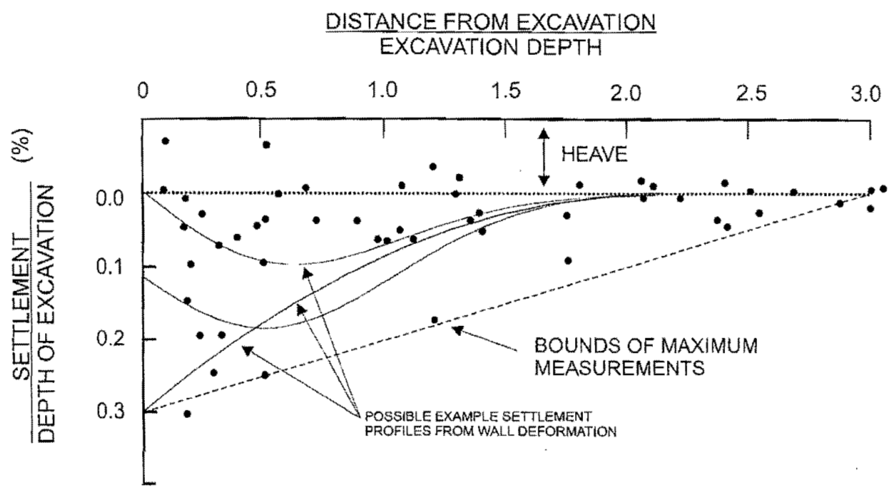


Figure 5 Settlements Adjacent to Excavations in Stiff Clay (from CFEM)

4.7.1.2 These data show the dramatic difference in settlements for sands and stiff clay compared with soft to medium stiff clay. For an example excavation of 20 m depth, the maximum settlements would be 20 mm - 60 mm in sands and stiff clays, compared with 120 mm - 400 mm in soft to medium stiff clays. Figure 6 excerpted from CFEM and attributed to Clough et al (1989), shows the influence of wall stiffness and factor of safety relating to base stability.

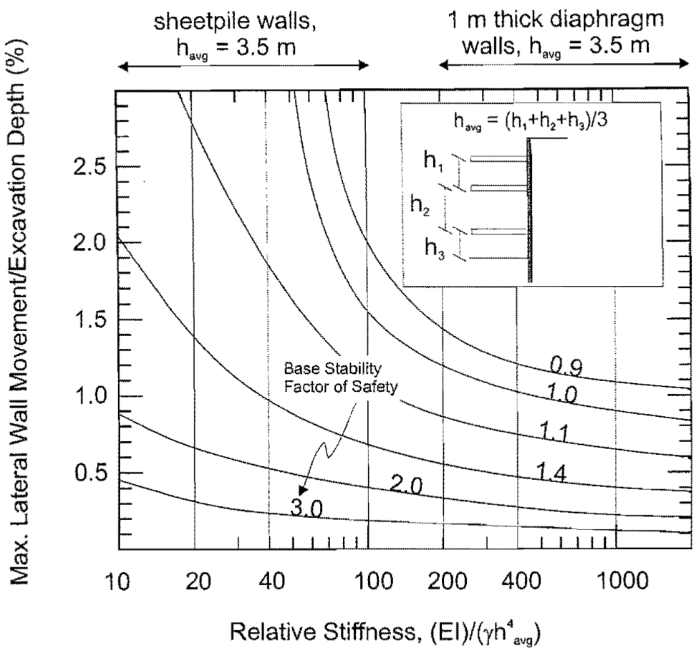


Figure 6 Relationship between Maximum Lateral Movement and Excavation Base Stability Factor of Safety (from CFEM)

4.7.1.3 It should be noted that the locally preferred SOE, of secant pile contiguous caisson walls with strut levels at about 3 m centres, would allow reference to the right-hand side of the diagram shown in Figure 6 providing a significant improvement over more flexible support systems.

4.7.1.4 The CFEM has summarized the findings as presented in Table 10.

Table 10 Settlement and Zone of Influence for Vertical Excavations

Soil Type	Settlement (% of Excavation Depth)	Extent of Zone of Influence (times Excavation Depth)
Granular Soils	0.2 - 0.3	2.0
Soft to Firm Clays	1.0 - 2.0	1.0 - 2.0
Stiff Clays or Hard Tills	0.1 - 0.3	2.0 - 3.0

Developed from CFEM.

4.7.1.5 The CFEM also presents equations for calculating the profiles of the settlement surfaces adjacent to excavations. Spandrel and concave settlement profiles shall be addressed. as applicable.

4.7.2 Historical Precedent

4.7.2.1 Historical precedent for settlement around excavations for transit facilities in the GTA, is claimed to indicate that the CFEM methodology appears to overestimate the settlements. However, there is no hard data available to support these assertions. It is noted that ground conditions in the GTA tend towards the stiffer clays, tills and granular soils and that the SOE systems have increased in stiffness since the development of the criteria.

4.7.2.2 Ongoing projects, particularly the TBM launch shaft on SSE, may offer opportunity to collect data on this subject.

4.8 Minimum Depth of Cover, Pillar Width and Buoyancy Effects

4.8.1 Minimum Cover for TBM Tunnels in Soil and Mixed Face Conditions

4.8.1.1 Since the introduction of pressurized face TBM's, the required dimension for minimum cover has reduced. This was demonstrated on the St Clair Tunnel where a 9.5 m excavated diameter (D) EPB TBM crossed under the 800 m wide river with less than 0.5 D of cover. This achievement should be caveated with the fact that there was little need for settlement control on the riverbed above the tunnel on the referenced project. Covers of 0.75 D have been used on several transit projects in the GTA.

4.8.1.2 There is no specified minimum cover for TBM tunnels in soil and mixed face. The required cover shall be determined by analysis of the geotechnical conditions, the settlement control required, the face pressure needs to maintain stability, and ground treatment to be applied at the location in question.

4.8.2 Minimum Cover for TBM Tunnels in Rock

4.8.2.1 Since transit projects in the GTA by TBMs, such as Advance Tunnels for Eglinton Crosstown West Extension (ATECWE), can have alignment profiles that are partially in rock, partially in soft ground and have transitions between these conditions, the minimum rock cover is, by profile design, zero. However, this is conditional on the TBMs being operated in the pressurized mode. The question is, therefore, at what depth of rock cover can the operating mode be changed from pressurized to unpressurized face.

4.8.2.2 There is no specified minimum cover for unpressurized TBM's in rock. The required cover shall be determined by analysis of the geotechnical conditions, the settlement control required, the face pressure needs to maintain stability, and ground treatment to be applied at the location in question. Particular attention shall be applied to the avoidance of overbreak in shallow tunnels and at soil/rock transitions. The Georgian Bay Shale, having generally thin horizontal bedding, has shown a tendency on previous tunnels to exhibit break out at the shoulders of the tunnel. This is a larger problem on unshielded TBMs but shall require attention on shielded machines as well.

4.8.3 Minimum Cover for SEM Tunnels in Soil and Mixed Face Conditions

4.8.3.1 The determination of minimum cover for SEM tunnels in soil and mixed face conditions is not possible since it is so conditional on multiple design choices and location specific conditions. The required cover shall be determined by analysis of the geotechnical conditions, the settlement control required, the face segmentation, pre-support to be installed above and in front of the excavation, the excavation advance length, initial support to be installed, and ground treatment to be applied at the location in question. This analysis shall require the application of complex Finite Element or Finite Difference ground/structure interaction modelling to determine the effects of the excavation increments.

4.8.4 Minimum Cover for SEM Tunnels in Rock

4.8.4.1 As for TBM tunnels in rock, SEM tunnels can have alignment profiles that are partially in rock, partially in soft ground and have transitions between these conditions, and therefore the minimum rock cover is, by profile design, zero. Within these transitions, the SEM excavation procedures would incrementally change between those required for rock to those required in soft ground.

4.8.4.2 The required cover shall be determined by analysis of the geotechnical conditions, the settlement control required, the face segmentation, pre-support to be installed above and in front of the excavation, the excavation advance length, initial support to be installed, and ground treatment to be applied at the location in question. This analysis shall require the application of complex Finite Element or Finite Difference ground/structure interaction modelling to determine the effects of the excavation increments.

4.8.5 Minimum Cover for Cut and Cover Running Structures (Box Structures)

4.8.5.1 The minimum cover for cut and cover running structures is not determined by geo-structural analysis but by requirements for clearances, for instance for utilities. Usual requirements for such clearances are 2.5 m above structures within the street right-of-way.

4.8.6 Minimum Cover for Cut and Cover Stations

4.8.6.1 The minimum cover for cut and cover station structures is zero since such structures shall interface with the street surface. Within the street right-of-way, except at entrances, vent shafts and emergency exits, the required cover for stations would be as for the cut and cover running structures.

4.8.7 Minimum Width of Pillar for Twin Tunnels

4.8.7.1 The normal pillar width between twin tunnels is usually determined by the spacing of the tunnels required to accommodate a centre platform between the two tracks. The alignments are, for simplicity, kept at this spacing for the remaining parts of the transit line. The platform width required, and the vehicle technology employed will determine the pillar width. For instance, the Eglinton Crosstown has LRT track spacing of 10.90 m and a tunnel excavated diameter of 6.72 m giving a pillar width of 4.18 m; for TYSSE the subway track spacing is 13.59 m (due to a wider platform) and a tunnel excavated diameter of 6.13 m giving a pillar width of 7.46 m. These pillar widths are considered to be generous, demonstrated somewhat by the Eglinton Crosstown LRT having a larger tunnel and a smaller pillar width compared with the TYSSE Subway, with no measurable detriment.

4.8.7.2 Should narrower pillar widths be required, these should be justified by analysis of the geotechnical conditions, the settlement control required, the face pressure needs to maintain stability, and ground treatment to be applied at the location in question.

4.8.8 Buoyancy Effects for Large Shallow Tunnels

4.8.8.1 Buoyancy effects large tunnels can be challenging. The analysis is based on the comparison of the upward force, the weight of water displaced by the tunnel envelope, and the downward force based on the weight of the tunnel structure and any overburden above it. This comparison should achieve the downward forces being 10% higher than the upward forces.

4.8.8.2 Generally, the friction values along the failure surfaces are not considered, but these may be included if sufficient investigation has been conducted to confidently predict these values. In cases where concrete walls are directly placed against rock surfaces, significant friction values or mechanical interlock will be available. If a membrane is included in these interfaces, significantly lower friction values shall be considered.

5. Elevated and Surface Structures

This section has been included but it is unlikely that these elements shall need significant revision.

5.1 Bridges and Elevated Guideways

5.2 Catenary Structures

5.3 Retaining Walls

5.4 Track Slabs

6. Geotechnical

6.1 Site Investigations

6.1.1 General

6.1.1.1 Work Included

a) The supervision, recording and reporting of site investigations, including but not limited to, borehole drilling, test pit excavation, sampling, and testing.

b) For the use geophysical investigations refer to Appendix D.

c) Subsurface investigations are best performed in several phases. However, during each phase the maximum possible number of sampling and testing, including in-situ and advanced laboratory testing, shall be carried out. Subsurface investigation work plans, reviewed and approved by Metrolinx, shall conform to the minima shown in Table 11 and Table 12 and shall provide adequate data for design interpretations as well as for preparation of the Geotechnical Baseline Report (GBR).

Table 11 Borehole Recommendations for Tunnels (modified from Infrastructure Ontario Site Investigation Guidelines)

Type of Structure	Method of Construction	Site Conditions	Subsurface Medium	Medium Conditions	Recommended Exploration Extent		
					Location	Spacing (m)	Minimum Depth (m)
Tunnel	Bored/Mined (e.g., Tunnel Boring Machine (TBM)/ Pipe Jacking/Box Jacking / Drill and Blast Method, etc.)	Previously Developed	Rock	Adverse	Single tunnel - 2 m offset from outside edge, staggered on sides	25 to 50	Greater of 6 m or 2 tunnel D below invert
					Twin tunnels - middle line	50 to 100	
				Favourable	Single tunnel - 2 m offset from outside edge, staggered on sides	50 to 100	Greater of 6 m or 1.5 tunnel D below invert
					Twin tunnels - middle line	75 to 150	
		Never Developed		Adverse	Single tunnel - 2 m offset from outside edge, staggered on sides	50 to 100	Greater of 6 m or 2 tunnel D below invert

Type of Structure	Method of Construction	Site Conditions	Subsurface Medium	Medium Conditions	Recommended Exploration Extent Location	Spacing (m)	Minimum Depth (m)
					Twin tunnels - middle line	50 to 100	
				Favourable	Single tunnel - 2 m offset from outside edge, staggered on sides	75 to 150	Greater of 6 m or 1.5 tunnel D below invert
					Twin tunnels - middle line	100 to 200	
	Bored/Mined (e.g., Tunnel Boring Machine (TBM)/ Pipe Jacking/Box Jacking / Sequential Excavation Method (SEM), etc.)	Previously Developed	Soil/Soil-Rock Interface	Adverse	Single tunnel - 2 m offset from outside edge, staggered on sides	25 to 50	Greater of 6 m or 3 tunnel D below invert
					Twin tunnels - middle line	50 to 100	
				Favourable	Single tunnel - 2 m offset from outside edge, staggered on sides	50 to 100	Greater of 6 m or 3 tunnel D below invert
					Twin tunnels - middle line	50 to 100	
		Never Developed		Adverse	Single tunnel - 2 m offset from outside edge, staggered on sides	25 to 50	Greater of 6 m or 3 tunnel D below invert
					Twin tunnels - middle line	50 to 100	
				Favourable	Single tunnel - 2 m offset from outside edge, staggered on sides	50 to 100	Greater of 6 m or 3 tunnel D below invert
					Twin tunnels - middle line	50 to 100	
	Shafts	Minimum one additional borehole at temporary (launch and removal) and permanent shafts location (Area < 225 m2). Minimum three additional boreholes at permanent shafts location (Area ≥ 225 m2) to depth as specified above. One deep borehole, every 1 km along tunnel alignment, to be extended to a depth below the ground surface of 2 x the depth of the tunnel invert below the ground surface.					
	Cut and Cover / Open Cut / Pre-deck Method	Previously Developed	Soil and Bedrock	Adverse	Centre line	25 to 50	Greater of 6 m or 2 tunnel D below invert. The purpose of outside edge / middle line boreholes is to predict the impact of excavation on
					Outside edge / Middle line	50 to 100	
				Favourable	Centre line	50 to 100	
					Outside edge / Middle line	50 to 100	
				Adverse	Centre line	25 to 50	

Type of Structure	Method of Construction	Site Conditions	Subsurface Medium	Medium Conditions	Recommended Exploration Extent		
					Location	Spacing (m)	Minimum Depth (m)
		Never Developed			Outside edge / Middle line	50 to 100	surrounding areas and to establish design parameters for SOE.
				Favourable	Centre line	50 to 100	
					Outside edge / Middle line	75 to 150	
		Previously Developed	Soil	Adverse	Centre line	25 to 50	Greater of 9 m or 3 tunnel D below invert. The purpose of outside edge / middle line boreholes is to predict the impact of excavation on surrounding areas and to establish design parameters for SOE.
					Outside edge / Middle line	50 to 100	
				Favourable	Centre line	50 to 100	
					Outside edge / Middle line	50 to 100	
		Never Developed		Adverse	Centre line	25 to 50	
					Outside edge / Middle line	50 to 100	
				Favourable	Centre line	50 to 100	
					Outside edge / Middle line	75 to 150	
	Submerged (Precast)	Underwater	Soil/Bedrock		Alignment - Staggered	75 to 150	Greater of 6 m or 2 tunnel D below invert

Table 12 Borehole Recommendations for Subsurface Stations/Structures (modified from Infrastructure Ontario Site Investigation Guidelines)

Type of Structure	Method of Construction	Site Conditions	Subsurface Medium	Medium Conditions	Recommended Exploration Extent		
					Location	Number	Minimum Depth (m)
Subsurface Stations / Structures	Cut and Cover / Top-Down Construction / Sequential Excavation Method (SEM)	Previously Developed	Soil and Bedrock	Adverse	Excavation Area	12 per station, including 1 at each headwall. 1 per 1000 m2 area for other structures.	6 m below foundation base elevation or below pile tip elevation. Outer rows of boreholes shall be as close as possible to the perimeter of the excavation.
				Favourable	Excavation Area	10 per station, including 1 at each headwall. 1 per 1000 m^2 area for other structures.	
		Never Developed		Adverse	Excavation Area	12 per station, including 1 at each headwall. 1 per 1000 m^2 area for other structures.	
				Favourable	Excavation Area	10 per station, including 1 at each headwall.	

Type of Structure	Method of Construction	Site Conditions	Subsurface Medium	Medium Conditions	Recommended Exploration Extent: Location	Number	Minimum Depth (m)
						1 per 1000 m^2 area for other structures.	
		Previously Developed	Soil	Adverse	Excavation Area	12 per station, including 1 at each headwall. 1 per 1000 m^2 area for other structures.	9 m below foundation base elevation or below pile tip elevation. Outer rows of boreholes shall be as close as possible to the perimeter of the excavation.
				Favourable	Excavation Area	10 per station, including 1 at each headwall. 1 per 1000 m^2 area for other structures.	
		Never Developed		Adverse	Excavation Area	12 per station, including 1 at each headwall. 1 per 1000 m^2 area for other structures.	
				Favourable	Excavation Area	10 per station, including 1 at each headwall. 1 per 1000 m^2 area for other structures.	

d) Notes for Table 11and Table 12:

1) Table 11and Table 12 are the recommended guidelines for determining the exploratory locations, depths and spacing for the respective intrusive method (boreholes, test pits, probes, etc.) for design purposes only. In addition to these guidelines, the Consultant shall consider the Guidelines and/or Specifications of the Project Sponsor(s) and/or Co-Sponsor(s) and adhere to whichever is more stringent/comprehensive. For example, if the Ministry of Transportation of Ontario is a Project Co-Sponsor, then their guidelines shall be considered in addition to these Guidelines.

2) The above-mentioned scope in terms of lateral and vertical extent is a guideline only and engineering judgment is required to determine the appropriate depths, locations and spacing of boreholes. The Consultant's Geotechnical Engineer shall review site conditions and proposed development and may propose a different scope of work, subject to approval from the Project Sponsor(s). The Consultant's Geotechnical

Engineer may also propose different methods and means of investigations as required and if site conditions permit. For tunnel projects, an Engineering Geologist shall be part of project investigation team. Any proposed changes to the scope of work or methods and means of investigations shall be approved by the Project Sponsor(s).

3) The Consultant’s Geotechnical Engineer assigned to the project shall complete at least one site visit, prior to start-up of the field investigation, to familiarize themselves with the site conditions and to help in determining the borehole/test hole locations.

4) The Consultant's Geotechnical Engineer shall determine the adverse/favourable conditions based on the review of published as well as available previously completed subsurface investigation information in the area, of overburden soil, rock, and groundwater conditions. Some of the factors that may contribute to adverse geological conditions include known, suspected or inactive faults, frequent highly jointed sedimentary formation, lava or volcanic formations, the boulder-soil matrix (BSM) mixed ground, the layered-banded rock (LBR) mixed ground, beach and fine sugary sandy conditions, soft clayey and peaty conditions, permafrost and frozen soil conditions, man-made features such as existing sub-structure remains and utilities, presence of aged adjacent or overlying utilities within zone of influence, and erratic groundwater characteristics and aggressiveness. Consideration shall be given when swampy/muskeg/bog conditions exist. The Consultant's Geotechnical Engineer shall determine the appropriate investigation methodology based on the known conditions and with respect to the intended purpose of the investigation.

5) The local geology of the Site shall be considered when determining the application of these tables in terms of number of testing location, depths, in-situ test types, frequencies and sample collection methods. Special consideration should be given to sites which contain bedrock at or near the ground surface, organic swampy (muskeg) conditions, buried peat/compressible soils, very soft/very loose conditions, buried structures/fill, permafrost, adjacent structures, environmentally sensitive areas and other special site conditions which may warrant increasing or decreasing the number of test locations, depths, spacing, etc.

6) Where possible, the same borehole shall be used to collect information for multiple components of the project provided that the required information for each project component is obtained.

7) Flexibility of the borehole locations in terms of lateral and vertical extent of investigation points could be considered. For

example, for horizontally stretched structures, the number of investigation points can be adjusted provided the required information is gathered.

8) A sufficient number of boreholes shall be converted to monitoring wells/piezometers to investigate, test and monitor subsurface soil and groundwater conditions; to report perched and subsurface groundwater conditions to a minimum depth of 3 m below the anticipated excavation bottom; establish hydrogeological parameters to determine groundwater quantity and quality for short and long term management; establish stabilized and seasonal elevation trends including horizontal and vertical groundwater flow directions; and predict impact of short and long term groundwater management on surrounding infrastructure and land features.

9) Where tunnel lengths are relatively short (less than 100 m), a minimum of 1 borehole is required at each temporary/permanent shaft or portal. The borehole spacing shall not exceed 50 m.

10) Sampling interval shall be semi-continuous at 0.75 m intervals to 2 tunnel diameters above tunnel invert, then continuous sampling to tunnel invert. Sampling interval increases to 1.5 m below tunnel invert.

11) Laboratory testing shall consist of 25% of samples for grain size distribution and Atterberg Limits. Natural Moisture Content analysis on all samples.

6.1.1.2 References

a) In all cases, the latest editions of the codes and standards shall be referenced. These standards are baseline standards. A set of consistent and equivalent widely accepted standards can be used, however mixing and matching shall not be allowed unless detailed analysis and reviews are provided. Where conflict exists between these codes and standards and this standard, then this standard shall prevail.

b) Ontario Occupational Health and Safety Act and Regulations for Construction Projects.

c) Canadian Foundation Engineering Manual (CFEM).

d) Toronto Transit Commission (TTC), Geotechnical Standards - Direction for Conducting Site Investigations (latest version).

e) Toronto Transit Commission (TTC), Design Manual (latest version).

f) Infrastructure Ontario (IO) AFP Geotechnical, Hydrogeology, Environmental Due Diligence Technical Requirements - Civil.

g) Infrastructure Projects (latest version).

h) Toronto and Region Conservation Authority (TRCA) Manuals and

Guidelines, including but not limited to the Geotechnical Engineering Design and Submission Requirement (latest version).

i) American Society for Testing and Materials (ASTM) Standards:

1) ASTM D1452, Soil Investigation and Sampling by Auger Borings.

2) ASTM D1586, Penetration Test and Split-Barrel Sampling of Soils.

3) ASTM D1587, Thin-Walled Tube Sampling of Soils.

4) ASTM D2113, Rock Core Drilling and Sampling of Rock for Site Investigation.

5) ASTM D2573, Field Vane Shear Test in Cohesive Soil.

6) ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

7) ASTM D2488, Standard Practice for Description and Identification of Soils (Visual Manual Procedure),

8) ASTM D3441, Mechanical Cone Penetration Tests of Soil.

9) ASTM D4220, Preserving and Transporting Soil Samples.

10) ASTM D4719, Prebored Pressuremeter Testing in Soils.

11) ASTM D4750, Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well).

12) ASTM D5079, Preserving and Transporting Rock Core Samples.

13) ASTM D5092, Design and Installation of Groundwater Monitoring Wells.

14) ASTM D5784, Use of Hollow-Stem Augers for Geo-environmental Exploration and the Installation of Subsurface Water Quality Monitoring Devices.

15) ASTM D6151, Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling.

16) The Ontario Traffic Manual Book 7: Temporary Conditions.

6.1.1.3 Approval of Staff and Laboratories

a) For the supervision of drilling, sampling and in-situ testing there shall be one full-time field supervisor for each drilling or testing rig. The full-time supervisor shall be a trained Intermediate Engineer, Intermediate Hydrogeologist/Geo-Scientist or Senior Technician, based on industry standards. No drilling, sampling or in-situ testing shall be carried out unless the field supervisor is present.

b) The name, work history and qualifications of the field supervisor shall be submitted to Metrolinx's Representative for approval. Only engineers, scientists and technicians approved by the Metrolinx's

Representative shall be permitted to act as "field supervisors".

c) The field supervisor shall ensure that the field work is carried out as required in this specification, safely and in accordance with appropriate legislation.

d) The planning and coordination of the work, and the preparation of the final report(s) may be carried out by suitably qualified persons other than the approved field supervisors but shall be carried out under the supervision of a Professional Engineer, or where applicable by a Professional Geo-Scientist.

e) Geotechnical Laboratory testing shall be carried out by a laboratory which is a member of the Canadian Council of Independent Laboratories and approved by Metrolinx's Representative. Advanced rock testing shall be undertaken at specialized rock testing laboratories such as Queen's University Advanced Geomechanics Testing Laboratory, Geomechanica Rock Mechanics Laboratory and the University of Western Ontario Geotechnical Research Centre. Where applicable, the laboratory shall have participated and met the MTO correlation program for soil testing.

f) Chemical (Geo-environmental) Laboratory testing shall be carried out by a laboratory which is accredited by the Canadian Association for Environmental Analytical Laboratories and approved by Metrolinx's Representative.

6.1.1.4 Submittals

a) A work plan shall be prepared by the Consultant and agreed with Metrolinx's Representative before starting work. The work plan shall include:

1) The number of drill holes, test pits and probes.

2) The approximate location and depth of drill holes, test pits and probes.

3) Sampling and in-situ testing methods, with approximate depths or frequency.

4) Number and type and piezometer/monitoring well installations.

5) The name of the designated field supervisor.

6) The names of the designated drilling and testing companies.

7) The types of drilling, excavation and field-testing equipment which will be mobilized to site.

b) A Health and Safety plan shall be prepared and submitted to Metrolinx's Representative for approval at least two weeks before starting work. The Health and Safety plan shall clearly specify:

1) Personal protection protocols and procedures that shall be used for all the work.

2) Levels of enhanced personal protection protocols and procedures, and the triggers that would require the implementation of the enhanced measures.

3) Emergency procedures.

4) Location and route to nearest hospital.
   A. The Health and Safety plans for potentially contaminated sites shall be site specific and take due account of the possible types of contamination and the physical constraints at the site.
   B. A copy of the Health and Safety Plan shall be always on-site during field operations.

c) Where the work takes place in close proximity to vehicular traffic, (i.e., city roadways, bus loops, garage areas, parking lot areas, etc.), a Traffic Protection Plan and Traffic Control Plan shall be prepared and submitted to Metrolinx's Representative for review at least two weeks before starting work. The Traffic Plan shall clearly specify:

1) Signage and traffic cones/barriers required.

2) Diagrams of proposed traffic plan.
   A. The Ontario Traffic Manual Book 7 Temporary Conditions, by the Ministry of Transportation, shall be consulted in the preparation of the traffic plan. Representative diagrams best illustrating the situation shall be included in the traffic plan.
   B. A copy of the Traffic Protection/Control Plan shall be always on-site during field operations.

6.1.1.5 Permits

a) Before starting work all necessary easements and permits shall be obtained.

6.1.1.6 Service Clearance

a) Before starting work the locations of buried services shall be determined, and borehole locations cleared by those agencies which could have services in the area.

b) Boreholes, probes or test pits shall be located outside the clearance limit established by each of the agencies.

c) If it is not possible to identify services, or if it is necessary to work inside the clearance limits, then a trial pit or “daylighting” (hydro-vac) shall be excavated to verify that the drilling location is free of services. When daylighting is required other means of collecting samples from shallow depths, missed due to daylighting, shall be provided.

d) Geophysical methods may be used to help identify underground structures or services, but the feature location shall be checked using a trial pit or probe hole and shall not be based solely on geophysical results.

e) Where a borehole is located within Metrolinx's right-of-way and/or facilities obtain the plans of buried utilities and structures from Metrolinx's Representative. These plans may not show exact "as-built" locations, and the structure or utility shall be physically located if it is intended to drill close to it.

6.1.2 Planning

6.1.2.1 Require that the drilling or testing company mobilizes sufficient plant, casings, consumables, parts and testing equipment to the site to complete the planned work without delay.

6.1.2.2 Verify that the plant, casings, consumables and testing equipment provided are clean and serviceable. Equipment shall comply with the relevant ASTM standard, unless otherwise approved by Metrolinx's Representative.

6.1.2.3 Require that an adequate number of sample containers be available on the site. The type of sample container to be provided will depend on the type of sample obtained and the testing envisaged.

6.1.2.4 For samples which are taken to be sent for chemical testing, require the testing laboratory to provide the correct sample containers, stabilizing agents and site storage facilities.

6.1.2.5 Require that suitable arrangements be made to transport any cuttings or potentially contaminated water resulting from drilling or sampling. The arrangements for storage, transportation and disposal shall comply with relevant regulations and requirements (including Ministry of the Environment, Conservation and Parks (MECP), The Toronto and Region Conservation Authority (TRCA) and other Conservation Authorities, as applicable) which are current at the time of storage or disposal. Requirements for the geotechnical consultant, current at the time of preparation of the specification, include, but are not limited to:

a) Lugger boxes or clean, sealable 205 L drums for collection and transport of drill cuttings and water. Labels shall be attached to the boxes or drums and shall include the following minimum information:

   1) Consultant's company name and office location.

   2) Contact number.

   3) Metrolinx contract number.

b) Chemical testing appropriate for the proposed approved disposal facility on a composite sample from each container.

c) Segregation of cuttings where noxious odors or visual impacts are observed.

d) Use of Ministry of Environment licensed haulers for contaminated materials and wastes.

e) Certificate of Approval for any temporary storage facility.

f) Obtain emergency waste generator number from the Ministry of Environment prior to disposal of contaminated cuttings or water.

g) Disposal of cuttings and water at an approved disposal facility.

6.1.3 Execution

a) The field supervisor shall arrange that the work is carried out to the requirements of Metrolinx and in accordance with the work plan and the Health and Safety plan. Refer to Appendix A for additional information. The main field supervision tasks are specified below:

6.1.3.2 Drilling, Sampling and Installations

a) Drilling methods and procedures shall be submitted by the drilling contractor for review by Metrolinx. Drilling methods and procedures should be selected to minimize disturbance to the collected samples. The drilling methods shall allow for collecting samples for advance laboratory testing in addition to samples collected for routine index testing,

b) When drilling through more than one major aquifer, or where there is potentially contaminated ground, precautions shall be taken to avoid cross- aquifer contamination.

c) When drilling in artesian conditions it may be necessary to use a weighted drilling mud. Where such conditions are anticipated the drilling company used shall be experienced in this type of work.

d) Generally, the methods and frequency of sampling shall follow that outlined in the work plan. Refer to Appendix A, A.6.1, for minimum frequency of sampling. However, the field supervisor shall vary the method or frequency of sampling where conditions warrant such change. Require that equipment used to obtain samples for chemical testing is cleaned to the necessary standard.

e) All boreholes, probes and test pits shall be surveyed for location and elevation and referenced to Geodetic datum. Survey shall include the following:

1) Northings and Eastings to ±2 cm.

2) Ground surface Elevations to ±1 cm.

3) Metrolinx MTM GCS defined as:
   A. Horizontal datum: North American Datum of 1983, Canadian Spatial Reference System Version 6 (“NAD83 CSRS v6”), also referred to as “NAD83 CSRS epoch2010”.
   B. Vertical datum: Canadian Geodetic Vertical Datum of 1928:1978 adjustment (CGVD28:78adj); and
   C. Map projection: MTM, Zone 10.

f) Surface materials at all borehole locations shall be accurately measured. For borings carried out on vegetated surfaces, a hand dug test pit shall be carried out adjacent to the borehole to provide an accurate measurement of the topsoil thickness (measured to the nearest cm). Thickness of concrete or asphalt surfaces shall be measured from extracted cores or from measurements taken inside the open boring (measured to the nearest mm). The granular courses below the pavement surface shall be accurately measured from inside the open boring (measured to the nearest cm). Where practical, distinction shall be made between the granular base and sub-base material.

g) Field records for boreholes shall be prepared during drilling and shall include the following minimum information:

1) Borehole Number Drilling Date(s)

2) Time: Drilling time shall be recorded for contractor payment purposes. Productive time, standby time and delay time shall be separately recorded, and average daily rates of advance shall be calculated.

3) Drilling Contractor/Equipment: The name of the drilling contractor and his site representative (the driller) shall be recorded, as shall the make and model of the drill rig, the size and type of augers or casings used and the size and weight of the standard samplers.

4) Weather: Average Daily Temperatures and general weather conditions shall be recorded.

5) Location: A written description and a field sketch (with dimensions) shall be provided.

6) Topsoil/Pavement: The thickness and nature of surface materials shall be recorded (topsoil thickness to the nearest cm from shallow hand dug test pit adjacent to the borehole). For pavements this includes the thickness of asphalt, concrete and granular base. Asphalt and concrete thicknesses shall be reported to the nearest mm, and granular thicknesses to the nearest cm. Where practical, distinguish between granular base and sub-base.

7) In-Situ Tests: The number, depth and result of each in-situ test shall be recorded.

8) Drilling Advance: Comments on the rate of advance or relative difficulty of borehole advance shall be provided. The depth of obstructions and any indications of obstructions such as grinding of the augers shall be noted.

9) Sampling: The number, depth, type and size of each soil sample shall be recorded, and the actual amount of soil recovered, and a description of the recovered soil shall be provided.

10) Groundwater: The location at which groundwater is first encountered shall be recorded as shall the degree of wetness of the sampler; the groundwater level within the hole at the

beginning and end of each day shall also be recorded.

11) Strata Summary: A summarized description of each soil stratum encountered shall be provided, together with the interpreted or observed boundary between the strata.

12) Consumables: All consumable items used by the drilling contractor shall be listed on the field borehole log.

13) Emphasis shall be placed on those observations which can only be made in the field, i.e., (Depth where sampler becomes wet, Scraping, grinding or damage to drilling or sampling equipment, Visual or olfactory indication of contamination, Time and consumables used)

h) Field records for test pits shall be prepared during drilling and shall include the following minimum information:

1) Test Pit Number Excavation Date

2) Time: Excavation time shall be recorded as productive time, standby time and delay time for contractor payment purposes.

3) Contractor/Equipment: The name of the excavation contractor and his site representative shall be recorded, as shall the make and model of the backhoe.

4) Weather: Average daily temperature and general weather conditions shall be recorded.

5) Location/Size: A written description and dimensioned field sketch shall be provided showing the test pit location and size.

6) In-Situ Tests: The number, depth, location within the test pit and result of each in-situ test shall be recorded.

7) Obstructions: The location, size, shape and type of each obstruction shall be recorded.

8) Sampling: The number, depth and location of each soil sample shall be recorded, and a description of each sample shall be provided.

9) Groundwater: The location of seepage zones shall be noted, and the quantity of seepage estimated. The depth of standing water at the end of excavation shall be noted as shall any changes in water level prior to backfilling.

10) Strata Summary: A summary sketch showing the stratigraphy in the wall of the test pit shall be provided. The sketch shall indicate any dip in strata boundaries and shall include surficial materials such as topsoil or pavement structure.

11) Backfill Details: The type and method of backfill placement shall be recorded.

i) Copies of the field records shall be submitted to Metrolinx's

Representative if requested.

j) All piezometer/well installations shall be carried out in accordance with MECP Regulation 903 and its amendments. Boreholes without installations shall also be decommissioned as per MECP Regulation 903 (as amended). Decommissioning shall withstand pressure induced by tunnelling. No soil or rock obtained during drilling shall be placed back into the borehole, unless the borehole is less than 3.0 m in depth, as per MECP Regulation 903 (as amended), provided the groundwater table has not been encountered.

k) Protective steel casings shall be installed at the top of each borehole, secured with concrete or neat cement grout. For boreholes in roads or sidewalks the steel casing shall have a cap that is flush with the surface. For boreholes in grassed areas the casing shall project above ground level and have a lockable cap.

l) In-situ probe holes (i.e., dilatometer, pressuremeter, etc.) shall be grouted to ground surface using a tremie pipe. Grouting shall be carried out after the probe is removed by inserting a tremie pipe into the open hole.

m) Unless otherwise directed, test pits and trial pits shall be backfilled using either excavated material or compacted granular fill, as required. Additional information on test pit installation is included in Appendix C.

n) Make good roads, parking areas and sidewalks in accordance with applicable City Standards.

o) Materials presented in the MECP documents titled: "Guidance on Sampling and Analytical Methods for Use at Contaminated Sites in Ontario” (1996) and “Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” (2004 as amended) shall be utilized where appropriate.

6.1.3.3 Soil and Rock Sampling

a) Soil and rock samples obtained using a split-spoon sampler shall be placed in 500 mL glass jars and the sample shall be separated from the jar lid using aluminum foil. Alternatively, the samples may be placed in two plastic bags with both bags sealed independently of each other. Refer to Appendix A, (A.5.1) for other types of samplers.

b) A representative portion of each sample retrieved shall be placed in the compartmentalized cardboard boxes or equivalent. This sample shall be approximately 30 to 40 mm in length.

   1) If there is a significant change in soil type in the same sample interval, the two soil types shall be placed in separate bags and separate boxes with the depth of the sample correctly indicated on both the bags and boxes.

   2) It shall be recorded on the field log, when a sample has been

stained or has an odour. The colour and degree of stain, type of odour shall be described. Any indication that the soil may be fill shall be recorded.

3) The screening process, using visual, olfactory, and organic vapour observations, described here, is intended to provide an index of potential impact for sites where there is no identified impact risk. Where there are reasons (historic land use, prior boreholes) for expecting contamination, screening, testing and safety precautions shall be decided on a site-specific basis.

4) Where the visual, olfactory, or organic vapour observations, described above, provide evidence that part of the site may be impacted, the field supervisor shall immediately report this to Metrolinx's Representative.

6.1.3.4 Rock Sampling

a) Rock cores shall be wrapped in plastic sheeting and placed in rock core boxes for transport. As soon as practicable, the core shall be stored in a humid room. All rock core runs shall be separated to allow easy reviewing of the individual sample runs.

b) It shall be recorded on the field log, when a sample has been stained or has an odour. The colour and degree of stain, type of odour shall be described.

c) The screening process, using visual, olfactory and organic vapour observations described here is intended to provide an index of potential impact for sites where there is no identified impact risk. Where there are reasons (historic land use, prior boreholes) for expecting contamination, screening, testing and safety precautions shall be decided on a site-specific basis.

d) Where the visual, olfactory or organic vapour observations, described above, provide evidence that part of the site may be impacted, the field supervisor shall immediately report this to Metrolinx's Representative.

e) Photograph all rock cores obtained.

6.1.3.5 In-Situ Testing

a) In-situ testing equipment shall be complete and calibrated, as required in the relevant ASTM standard or the manufacturer's handbook.

b) In-situ testing shall be carried out at the frequency required in the work plan, or as necessary to obtain the information required.

c) In-situ testing shall be carried out by staff experienced in the use of the equipment.

d) Readings shall be taken in all piezometers/wells at periodic time intervals until the readings have stabilized, or as directed in the work plan.

e) In-situ permeability tests shall be carried out in wells or piezometers as required in the work plan.

f) Commonly used in-situ testing methods are described in Appendix A (A.6).

6.1.3.6 Groundwater Sampling

a) Groundwater samples from wells/piezometers shall be taken as required by the work plan or following a vapour emission survey (see section 6.1.3.5) or site evaluation.

b) The well/piezometer shall be developed prior to the taking of a groundwater sample.

c) pH, temperature and specific conductivity of water extracted from well shall be monitored and recorded regularly. Development of a new well/piezometer is complete when:

   1) pH, temperature and specific conductivity of extracted water is constant; and,

   2) At least three times the volume of water within the screen and riser pipe have been removed.

   3) If the wells recharge very slowly, item ii) can be modified to a lower value by the field supervisor or the well shall be purged dry. Such modification shall be noted in the field records.

d) Where a well/piezometer has been developed before, then it shall be redeveloped prior to sampling. For redevelopment, the minimum volumes of water that shall be removed from the well are:

   1) Well/piezometer sealed into permeable soil (D10 >0.075 mm): four well volumes.

   2) Well/piezometer sealed into less permeable soil (D10 <0.075 mm): one well volume.

e) Sampling protocols shall be determined for each site and situation. The protocols, including sampling methods, filtration, preservation and custody shall be determined in consultation with the analytical laboratory prior to sampling.

f) Immediately before use, the sampling equipment shall be thoroughly cleaned with distilled water and decontaminated.

g) Sample collection and storage requirements shall follow MECP "Guidance on Sampling and Analytical Methods for Use at Contaminated Sites in Ontario, Ontario Ministry of Environment, Standards Development Branch, May, 1996", and “Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” (2004, as amended). Where not covered in the above document, recognized guidelines or standards such as EPA, APHA, MISA and other MECP methods should be followed.

6.1.3.7 Organic Vapour Testing

a) Organic vapour tests shall be carried out for all soil samples obtained from a split spoon sampler and at the head of every standpipe piezometer or well installed.

b) Soil samples obtained from the split spoon shall be stored in 500 mL glass jars or polyethylene bags. When using jars, aluminum foil shall be placed over the neck of the jar, such that the foil and the lid form an airtight seal to the jar, and the lid is separated from the sample by the foil.

c) After the jar/bag containing the sample has sat for at least two hours in an ambient temperature of 20°C or more, the concentration of organic vapour in the "headspace" (between the soil and the lid of the jar/bag) shall be measured using a photo-ionization unit. The probe of the photo-ionization unit shall be pushed through the aluminum foil cap or into the bag and the vapour at the top of the jar/bag shall be drawn into the unit to take a reading. The photo-ionization unit (HNu or OVA) shall be calibrated against isobutylene or equivalent.

d) An organic vapour survey shall be carried out of the head of every standpipe piezometer and well installed, using a photo-ionization unit (HNu or OVA) calibrated to isobutylene or equivalent, and a methane detector. Immediately after removing the cap of the standpipe, the probes of each of these units shall be inserted, in turn, into the riser pipe and a sample drawn into the unit to take a reading.

e) Photo-ionization units and methane detectors shall be calibrated daily, or more frequently if weather conditions are variable.

f) Record the results of organic vapour tests together with the serial number of the instruments used, weather conditions (if outdoors) and the temperature at the time of the survey.

6.1.3.8 Environmental Testing

a) Testing of soil and groundwater for selected parameters shall be based on visual inspection, organic vapour test results, odour, colour, known past history of the site or adjacent sites and as required in the work for site assessment or characterization.

b) Quality assurance and quality control procedures requirements shall follow Section 8. titled: "Analytical Methods and Laboratory Quality Assurance and Quality Control" of MECP "Guidance on Sampling and Analytical Methods for Use at Contaminated Sites in Ontario, Ontario Ministry of Environment, Standards Development Branch, May, 1996" and “Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” (2004 as amended).

6.1.3.9 Labelling, Transportation and Storage of Samples

a) All containers for soil, rock or water samples shall be clearly labelled. As a minimum, the label should identify:

1) The Project.

2) Sampling Date.

3) Borehole Number.

4) Sample Number.

5) Depth Interval.

6) Initials of Field Supervisor.

b) Soil, rock and water samples for testing shall be transported to the testing laboratory in a manner and within the time required to ensure that the testing is representative of the in-situ condition.

c) A 'Chain of Custody' form shall accompany each sample sent for testing. The chain of custody shall show, as a minimum:

1) The borehole/well/test pit and sample identifiers.

2) The depth at which the sample was obtained.

3) The names, in order, of every person responsible for taking, storing, transporting and receiving the samples.

4) A dated signature for each person responsible for the sample, at the time that the person received custody.

d) All samples shall be retained for at least three months past the duration of the Contract or upon approval by Metrolinx's Representative. Make samples available for inspection by Metrolinx's Representative when requested.

e) Metrolinx's Representative will instruct the Geotechnical Consultant either to deliver the samples to a designated storage facility or to dispose of them in accordance with regulatory requirements.

f) Sample collection and storage requirements shall follow MECP "Guidance on Sampling and Analytical Methods for Use at Contaminated Sites in Ontario, Ontario Ministry of Environment, Standards Development Branch, May, 1996", and "Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act" (2004). Where not covered in the above document, recognized guidelines or standards such as EPA, APHA, MISA and other MECP methods should be followed.

6.1.3.10 Geotechnical Laboratory Testing

a) Moisture content testing shall be carried out on all soil samples retrieved from the site.

b) Routine Geotechnical Laboratory testing including Grain Size Distribution and Atterberg Limits testing shall be carried out on 25% of the soil samples obtained from the site.

c) Geotechnical testing shall be carried out on samples from the middle of the soil deposit. Plasticity Index testing and Grain Size Distribution testing shall be carried out on the same sample, preferably from the middle of the soil stratum being characterized. Where there is insufficient material to permit this, two samples from the same layer may be combined and tested. A note in this regard shall be added to the Record of Borehole Sheet.

d) It is noted that any inclusions such as large pieces of gravel shall not be more than 2% of the total weight of the sample.

6.1.3.11 Chemical Laboratory Testing

a) The chemical testing laboratories shall report results within ten working days of sample collection.

b) All scheduled testing shall be carried out and reported.

c) Properly signed, original, copies of laboratory tests shall be received and compiled.

6.1.4 Reporting, Communication and Report Files

6.1.4.1 A Geotechnical Data Report and/or a Geotechnical Interpretation and Design Report shall be prepared in accordance with section 6.2.

6.1.4.2 The field supervisor may have identified, during the investigation, geotechnical, hydrogeological or environmental conditions which he/she considers warrant further investigation. The Geotechnical Consultant shall inform Metrolinx's Representative, in writing, of the relevant observations and give recommendations for further investigation.

6.1.4.3 Together with the required number of hard copies of the Geotechnical Data Report/Geotechnical Interpretation and Design Report the Geotechnical Consultant shall submit a copy of the report(s) in PDF format. The PDF version of the report shall not be locked or secured, as Metrolinx shall be able to insert the file electronically into the contract documents.

6.1.4.4 All correspondence to and from Metrolinx and report files shall first include the following:

a) Contract Name, Project Location - Description
i.e., Advance Tunnels for Eglinton Crosstown Western Extension (ATECWE), Eglinton Crosstown LRT - Geotechnical Investigation.

6.2 Geotechnical Reports

6.2.1 Introduction

6.2.1.1 Depending on the required information, a geotechnical investigation may include one or more of three types of deliverables:

a) Geotechnical Data Report (GDR).

b) Geotechnical Interpretation and Design Report (GIDR).

c) Geotechnical Baseline Report (GBR).

6.2.1.2 After each site investigation a GDR shall be prepared. The report shall contain all the data obtained during the investigation and record how this data was obtained. The only interpretative information in this report shall be geo-environmental comments on soil and groundwater disposal. This report shall be included in the contract construction documents.

6.2.1.3 A GIDR may also be requested, which shall include comments and recommendations on how to interpret and implement the data from the GDR for the proposed construction. The GIDR shall be a stand-alone report and shall not be included in the contract documents.

6.2.1.4 Underground construction projects, for example tunnel projects, shall include a GDR that is followed by a GBR. Both these reports shall be included in the construction contract documents. The GBR shall provide a baseline description of the geotechnical conditions that the contractor shall rely on for bidding and contractual purposes.

6.2.1.5 This part provides a standard format for the presentation of results and a Table of Contents for the reports.

6.2.2 Contents of the Geotechnical Reports

a) The GDR, prepared by the geotechnical Consultant, shall present only the factual geotechnical, geological, and hydrogeological information obtained from subsurface investigations, and in-situ field and laboratory test results.

b) When a GDR is to be completed, it shall include the subsurface conditions encountered based on the observations, in-situ test results, and laboratory test results.

6.2.2.1 Geotechnical Data Report (GDR).

The GDR shall include the following information:

a) Report Cover Page: This page shall (at a minimum) include information such as the name of the report, the report version, the project name and location, who the report was prepared by (Consultant information), who the report was prepared for (client information), and the date of issue.

b) Table of Contents- Table of contents shall identify numbered sections and various levels of subsections' titles and page numbers, as well as the following information:

1) List of Tables (table number, table name, page number).

2) List of Figures (figure number, figure name, page number).

3) List of Appendices (appendix letter, appendix name, appendix content).

c) Introduction-This section shall provide the following information:

1) Project description and background.

2) Objectives of investigation and scope of work.

3) Data, information, and report organization.

d) Project Area Description- This section shall provide the following information:

1) Existing site features and current and past use(s) of the project area.

2) Topography and surrounding properties current and past use.

e) Geological Setting and Features- This section shall provide the following information:

1) Topography.

2) Geological setting.

3) Quaternary regional geology; overburden; bedrock; land subsidence; landslides; and sinkholes.

4) Seismic setting.

5) Regional seismic conditions; published seismic site class.

f) Previously Completed Investigations and/or Studies- This section shall provide the following information:

1) Relevant information from previously completed investigations and/or studies.

2) Summary of extrapolated data.

3) Information that has been relied upon for the current geotechnical investigation.

g) Field Investigations Methodology- This section shall provide the following information:

1) Names and qualifications of those involved with the investigation; site location plan; health and safety plans; borehole location plan; utility clearances; investigation plans; and restrictions/obstacles encountered during Investigation.

2) Investigation means and methods: drilling contractor; drill rigs; hammer types; equipment calibration record; and in-situ test description.

3) Sampling methods and equipment (for example, split spoon; Shelby tube; HQ or NQ cores); sampling interval (e.g., standard or continuous).

4) In-situ tests completed, procedures, equipment used for tests and frequency (e.g., standard penetration tests, cone penetration tests, Pocket Penetrometer, Torvane, field vane); sample handling procedures; borehole completion and abandonment details; groundwater level measurements in

boreholes.

5) Observation / Monitoring Wells / Piezometers installations.

6) Shallow wells; deep wells; bedrock wells; water table measurement procedures; frequency of measurement; data logger installations (if any); results of all measurement(s).

7) Sample transportation and preservation methods.

8) Laboratory testing schedule, total number of samples for each test, summary of sample locations, sample depths, laboratory name and location for geotechnical and chemical testing, and laboratory qualifications.

h) Subsurface Stratigraphy- This section shall provide the following information:

1) Summary of encountered soil strata, summary of depths encountered, elevations, total thickness of each soil strata, locations.

2) Typical reporting includes summarizing the encountered surficial soils/materials (concrete, asphaltic concrete, etc.), followed by the encountered soils/materials. below the surficial soils (granular fill, earth fill, uncontrolled fill), native soil deposits/overburden, bedrock, and groundwater conditions.

3) Summary of the in-situ tests completed within each soil strata, results of in-situ tests, and results of laboratory tests.

i) Laboratory Testing Results- This section shall provide the following information:

1) Laboratory visual classification.

2) Index properties, engineering properties.

3) Summary of the results of each sample by test completed, alongside tables and graphs that present information clearly.

4) Summary of depths encountered, elevations, location.

j) Conclusions- This section shall provide the following information:

1) Summary of anticipated general subsurface stratigraphic conditions based on those encountered.

2) Confirmation of whether the investigation was completed as outlined in the work plan and/or Proposal.

3) Discussion and rationale if any deviation from the work plan and/or Proposal.

k) Limitations- The Limitations section shall describe the report intentions, applicability, investigation procedures that deviated from typical standards and specifications and reasons why, physical limitations, and implications.

l) Signatures- When applicable, appropriate signing authority persons (i.e., P.Eng.) are required to sign and stamp the report.

m) References- This section shall provide references to all publications, manuals, books and standards and design codes referred to in the report.

n) Figures:

1) A title page that lists each figure, their names and figures numbers (as 1, 2, 3,…), followed by all figures for the report.

2) At a minimum, a site location plan and a borehole and test pit locations plan are required.

o) Tables:

1) A title page that lists each table included, their names and table numbers, followed by all tables for the report.

p) Appendices

1) A title page that lists each appendix to the report, their names, numbers (as A, B, C,….), contents, followed by all appendices for the report.

2) At a minimum, the appendices shall include the borehole logs (with explanation page[s] of the reported logs), all laboratory test results, and certificates of authentication where applicable. The following minimum information is required on the borehole logs:

A. Boreholes Logs: Client name; project name; project location; drilling methods and dates drilled; field supervisor/technician name; name of engineer who completed tactile examination; borehole locations' coordinates; geodetic benchmark number and location; geodetic elevation of ground surface at boreholes; thickness and description of materials just below the ground surface, which may or may not have been sampled, such as topsoil, pavement structures, fill, etc.; soil stratification including, thickness of each distinct layer and the elevation of first groundwater strike as observed by the field supervisor overseeing the drilling activities.

B. Field Observations Record: Recovery; sampler blow count per 300 mm; obstructions encountered: spoon refusal; auger refusal due to boulder, concrete, metallic, non-metallic obstruction, dense soil or bedrock; gas observation and vapour testing; groundwater observation: groundwater depth in boreholes, borehole completion depth; collapsed or open.

C. Sample Tactile Examination: Colour; odour; compactness; consistency; soil classifications; moisture content; organic content; and deleterious presence.

D. Results of In-Situ Tests: Standard penetration tests, cone penetration tests, field vane tests, dynamic cone penetration tests, pocket penetrometer, Torvane, etc.

E. Results of Laboratory Tests: Soil classifications; soil index properties; shear strength; compressibility; permeability; density; moisture/water content; results of chemical testing (including Quality Assurance/Quality Control (QA/QC) testing); a comparison of the results of chemical testing to the relevant MECP Standards and comment on the soil and groundwater disposal options.

F. Monitoring Wells/Piezometers: Construction details; groundwater elevations (stabilized)-all rounds; adjacent water body elevation if applicable; in-situ permeability test results.

G. Bedrock if Encountered: Depth to bedrock; rock core type; recovery; bedrock geological classification; weathering; structural features of rock mass; discontinuity-characteristic; discontinuity-orientations; discontinuity-spacing; Rock Quality Designation (RQD).

q) In addition to these items, the Geotechnical Data Report shall include other relevant factual information obtained during the investigation. Sufficient factual information is required to determine:

1) The vertical extent of subsurface materials (including both soil and rock) and their pertinent engineering properties.

r) Additional site-specific information may be required, depending on specific site conditions, the project, or proposed development requirements.

s) The GDIR shall include reference to the GDR but does not need to append the report.

t) The GIDR shall generally adhere to the following format:

1. Engineering Discussion and Assessment

1.1 Summary of the Findings of the GDR

1.2 Earthwork

1.2.1 Geotechnical Design Parameters

1.2.2 Preliminary Design Criteria

1.2.3 Construction Parameters and Recommendations

1.2.4 Soil Lateral Movement Monitoring Recommendation

1.2.5 Soil Vertical Movement Monitoring Recommendation

1.2.6 Applicable Standards and Specifications

1.2.7 Testing Methods and Frequencies

1.2.8 Reuse of Existing On-site Materials

1.2.9 Bulking and Shrinkage Factors

2. Site Soil and Ground Improvement

2.1 Geotechnical Design Parameters and Criteria

2.2 Construction Parameters and Recommendations

2.3 Risk Management

3. Foundations

3.1 Foundation Options including Shallow and Deep Foundations

3.2 Geotechnical Design Parameters and Criteria for Each Option

3.3 Construction Parameters and Recommendations

3.4 Size, Locations, Capacities, Ultimate Limit States and Serviceability Limit States

3.5 Anticipated Settlement

3.6 Testing Techniques and Methods

3.7 Impact on Surrounding/Adjacent Foundations/Structures

3.8 Slab-on-grade Considerations

4. Drainage

6.2.2.2 Geotechnical Interpretation and Design Report (GIDR)

a) The GDIR shall include reference to the GDR but does not need to append the report.

b) The GIDR shall generally adhere to the following format:
1. Engineering Discussion and Assessment
1.1 Summary of the Findings of the GDR
1.2 Earthwork
1.2.1 Geotechnical Design Parameters
1.2.2 Preliminary Design Criteria
1.2.3 Construction Parameters and Recommendations
1.2.4 Soil Lateral Movement Monitoring Recommendation
1.2.5 Soil Vertical Movement Monitoring Recommendation
1.2.6 Applicable Standards and Specifications
1.2.7 Testing Methods and Frequencies
1.2.8 Reuse of Existing On-site Materials
1.2.9 Bulking and Shrinkage Factors
2. Site Soil and Ground Improvement
2.1 Geotechnical Design Parameters and Criteria
2.2 Construction Parameters and Recommendations

2.3 Risk Management
3. Foundations
3.1 Foundation Options including Shallow and Deep Foundations
3.2 Geotechnical Design Parameters and Criteria for Each Option
3.3 Construction Parameters and Recommendations
3.4 Size, Locations, Capacities, Ultimate Limit States and Serviceability Limit States
3.5 Anticipated Settlement
3.6 Testing Techniques and Methods
3.7 Impact on Surrounding/Adjacent Foundations/Structures
3.8 Slab-on-grade Considerations
4. Drainage
4.1 Geotechnical Design Parameters and Criteria
5. Utility Servicing
5.1 Geotechnical Design Parameters
5.2 Construction Parameters and Recommendations
5.3 Bedding, Backfill and Compaction Requirements
5.4 Municipal and Provincial Standards and Specifications
6. Pavements
6.1 Anticipated Traffic Use
6.2 Geotechnical Design Parameters
6.3 Subgrade Preparation
6.4 Construction Parameters and Recommendations
6.5 Minimum Municipal and Provincial Standards
6.6 Drainage Adequacy and Options
6.7 Forensic Results of Pavement Distress Areas
6.8 Lifecycle Costing and Pavement Type Comparison\
7. Excavation and Temporary Shoring
7.1 Options and Recommendations for Support of Excavation Systems
7.2 Geotechnical Design Parameters
7.3 Construction Parameters and Recommendations
7.4 Applicable Regulations Guidelines
7.5 Soil Types and Respective Grade Requirements
8. Earth Pressures
8.1 Geotechnical Design Parameters
8.2 Construction Parameters and Recommendations
8.3 Lateral Parameters and Engineering Properties
8.4 Uplift and Friction
8.5 At-rest and Active Properties
8.6 Rock Swelling Characteristics/Pressures
9. Excavation Base Stability Including Global Stability
9.1 Geotechnical Design Parameters
9.2 Construction Parameters and Recommendations
9.3 Bearing Capacity, Sliding Resistance, Turning Resistance
9.4 Failure Criteria
10. Temporary and Permanent Dewatering Requirements
10.1 Options
10.2 Geotechnical Design Parameters
10.3 Potential Effect on Local Groundwater Resources and Natural

Features
10.4 Potential Effect on Local Structures (Settlement)
11. Site Seismic Classification
11.1 Perform Analysis
11.2 Classify Seismic Class
11.3 Provide Shear Wave Velocity Values
12. Soil Frost Susceptibility
12.1 Perform Analysis
12.2 Recommendations
12.3 Depth of Expectant Frost Penetration
12.4 Thermal Cover Techniques and Methods
12.5 Freeze-thaw Cycle Implications
13. Soil Resistivity and Corrosivity Potential
13.1 Perform Analysis
13.2 Recommendations
14. Engineered Fill/Earthwork
14.1 Discussion
14.2 Material Recommendations
14.3 Construction Recommendations/Compaction
14.4 Engineering Monitoring Recommendations
14.5 Municipal and Provincial Standards and Specifications
14.6 Testing Types, Frequency, Requirements
15. Existing Slopes
15.1 Discussion
15.2 Geotechnical Design Parameters
15.3 Construction Parameters
15.4 Slope Stability Analysis
15.5 Proposed Set Back
15.6 Vegetation, Erosion, Seepage, Height, Proximity to atercourse
16. Soil Retaining Structures/Mechanically Stabilized Earth Walls
16.1 Geotechnical Design Parameters
16.2 Soil Lateral Movement Monitoring Recommendations
16.3 Soil Vertical Movement Monitoring Recommendations
16.4 Global Stability Assessment
16.5 Turning, Sliding, Failure Criteria
16.6 Bearing Capacity, Mechanical Recommendations
17. Linear Infrastructure
17.1 Geotechnical Design Parameters
17.2 Engineering Inspection and Methods for Confirmation of Risks Associated with Potential Data Gaps
18. Bridges
18.1 Configuration
18.2 Foundations
18.3 Ground Conditions
18.4 Scour
18.5 Geotechnical Design Parameters (static/seismic)
18.6 Soil Lateral Movement Monitoring Recommendations
18.7 Soil Vertical Movement Monitoring Recommendations
18.8 Stability and Settlement of Approach Embankments
19. Tunnels

19.1 Subsurface Conditions
19.1.1 Soft Ground
19.1.2 Bedrock
19.1.3 Sandy Water Bearing Pockets/Layers
19.1.4 Boulders, Removal, Cost Implications
19.1.5 Groundwater, Artesian Pressure
19.1.6 Natural Features
19.1.7 Human Made Features
19.1.8 Potential Hazardous Materials Encounter, Handling, and Improvements
19.1.9 Solubility of Rocks
19.1.10 Risk Management
20. Geotechnical Monitoring Requirements
20.1 Settlement Monitoring
20.2 Adjacent Structures
20.3 Excavation Support Structures
20.4 Tunnel Linings

6.2.2.3 Geotechnical Baseline Report (GBR)

a) The GBR shall refer to but not repeat information contained in the GDR, which may be updated during the bidding process. As indicated in the Geotechnical Baseline Reports for Construction, Suggested Guidelines, (Essex, 2007), a GBR shall provide key baselines of anticipated physical conditions.

b) The content of the GBR shall follow the structure and content defined in the GBRs for Construction (ASCE, 2007).

6.2.3 Presentation of Results

6.2.3.1 Borehole logs and standard laboratory test results shall be prepared to a standard format as shown in Figure 7 to Figure 9.

6.2.3.2 Generally, the author of a GDR shall avoid making any interpretation of the ground conditions or likely behaviour. This is to avoid having a number of potentially conflicting interpretations. One of the exceptions to this principle is comments on chemical analytical results for soil and groundwater disposal purposes. The intent in this instance is for the Consultant, that has ensured the sampling method and sample integrity has followed proper environmental protocols, be responsible for comments on soil/groundwater disposal. However, the GDR shall include:

a) Summary graphs or tables where a number of tests of the same type have been carried out.

b) Calculated parameters from testing (e.g., permeability from a test in a piezometer; Cc, Cr, Mv from an oedometer test).

6.2.3.3 For in-situ tests such as the Pressure meter or Dilatometer, where primary output is typically converted to graphical form for presentation, the GDR shall include:

a) The graphical presentation.

b) A tabulated print-out of the original data.

c) For geophysical testing, the data shall be interpreted by the geophysicist responsible for the work. This is an exception to the general principle of avoiding interpretation in the GDR. The interpretation made by the geophysical company shall be reproduced in an appendix but shall not be discussed further.

6.2.4 Borehole, Test and Sample Designators

6.2.4.1 Each borehole, test pit, in-situ test and sample shall have a unique designator, to avoid potential confusion.

6.2.4.2 Designators will be provided by Metrolinx's Representative prior to any stage of investigation commencing and shall generally consist of the following:

a) Contract Name - Borehole Series Number- Borehole Number

b) Example: Advance Tunnels for Eglinton Crosstown West Extension (ATECWE) - EWE - BH07

6.2.5 Retention of Records

6.2.5.1 Original records, that are either not directly presented or reproduced in the GDR, shall be retained by the geotechnical consultant. These records shall include:

a) Original field logs and notes used in the preparation of borehole and test pit records.

b) The original signed certificates for laboratory tests.

c) QA/QC records for laboratory testing.

d) Chain of custody forms.

e) Digital photographs.

6.2.5.2 These records shall be made available to Metrolinx's Representative on request.

6.2.6 Drawings

6.2.6.1 Drawings, including site plans, borehole and test pit location plans, etc. shall be prepared in accordance with Metrolinx standards using the Metrolinx standard borders on 11" x 17" paper.

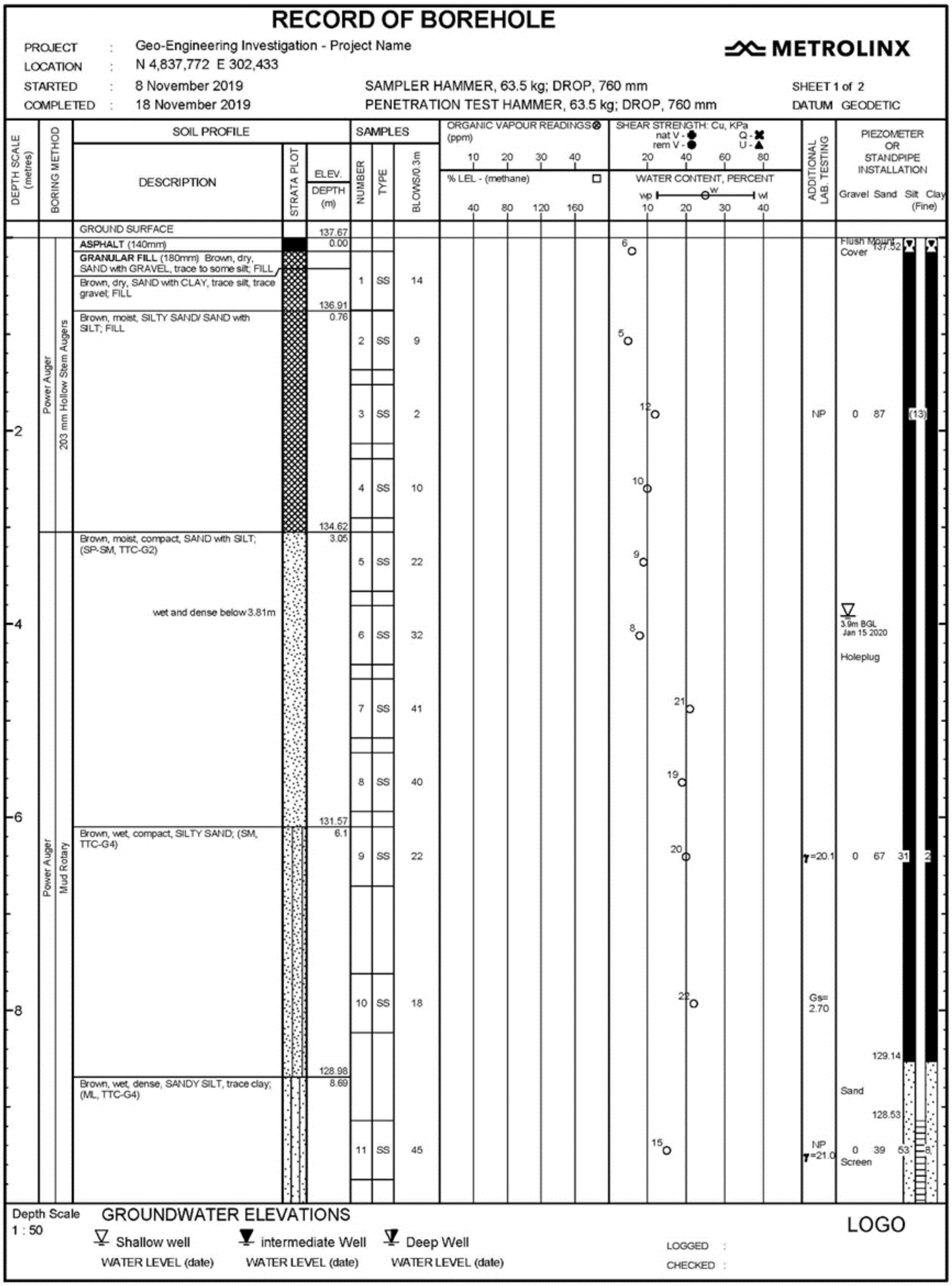


Figure 7 Example- Record of Borehole (Overburden)

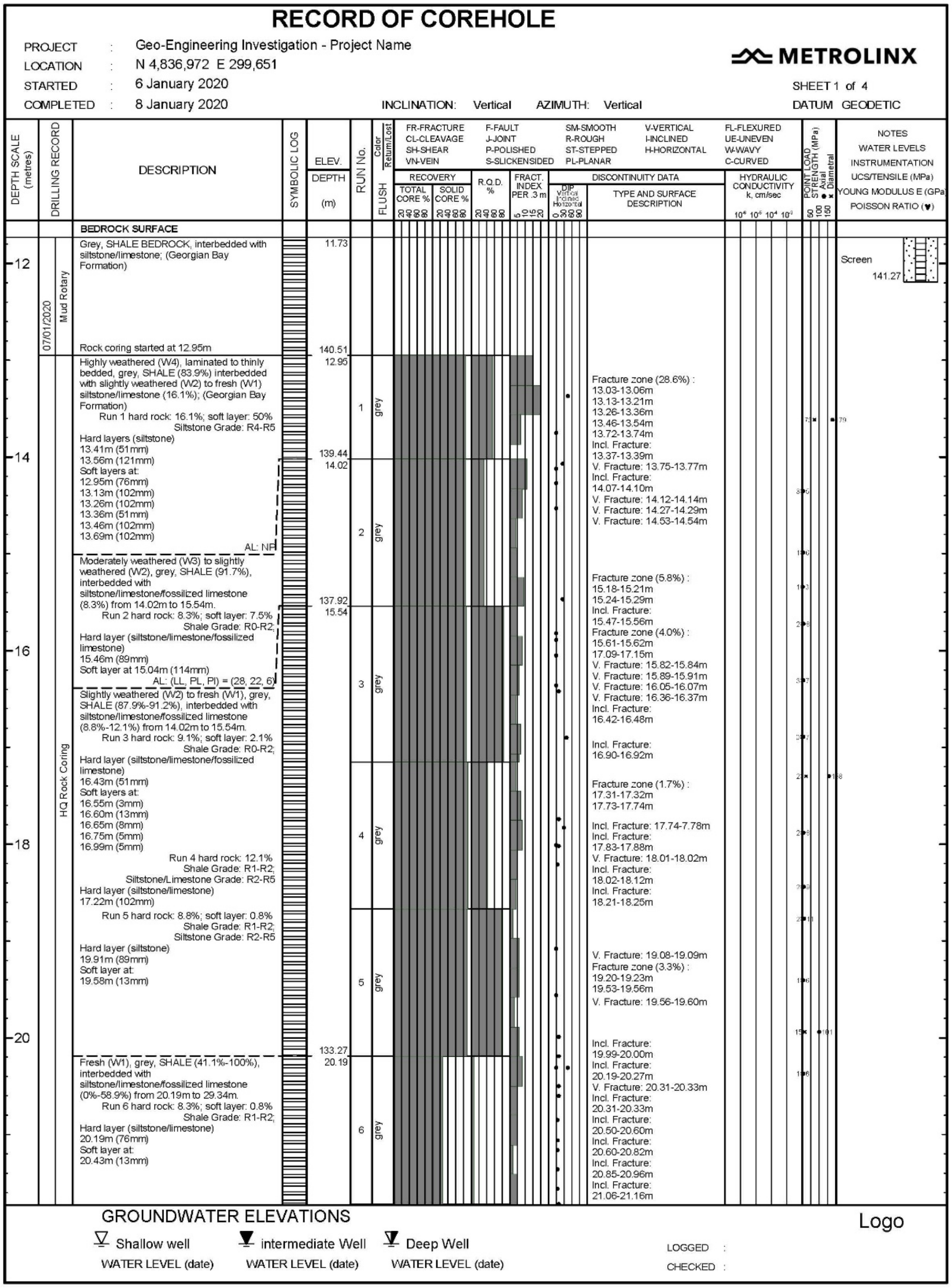


Figure 8 Example- Record of Borehole (Rock)

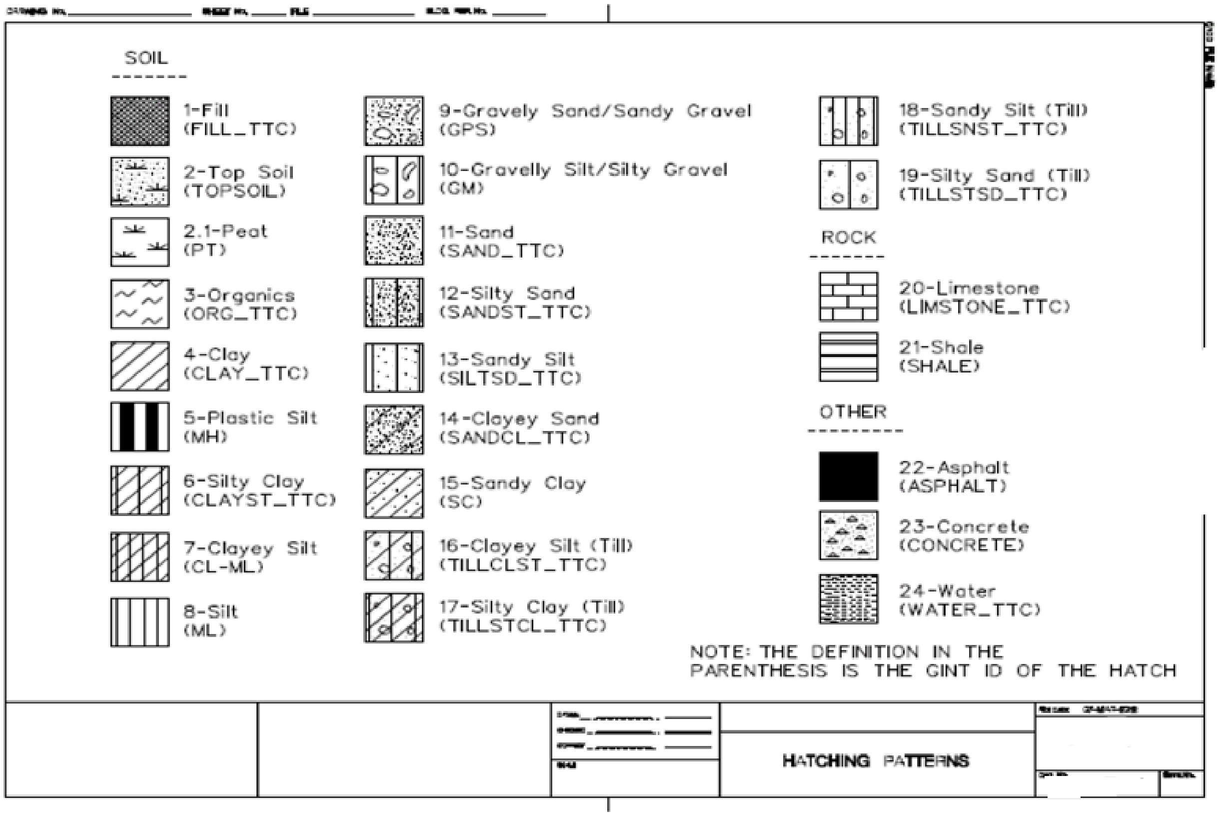


Figure 9 Hatching Patterns

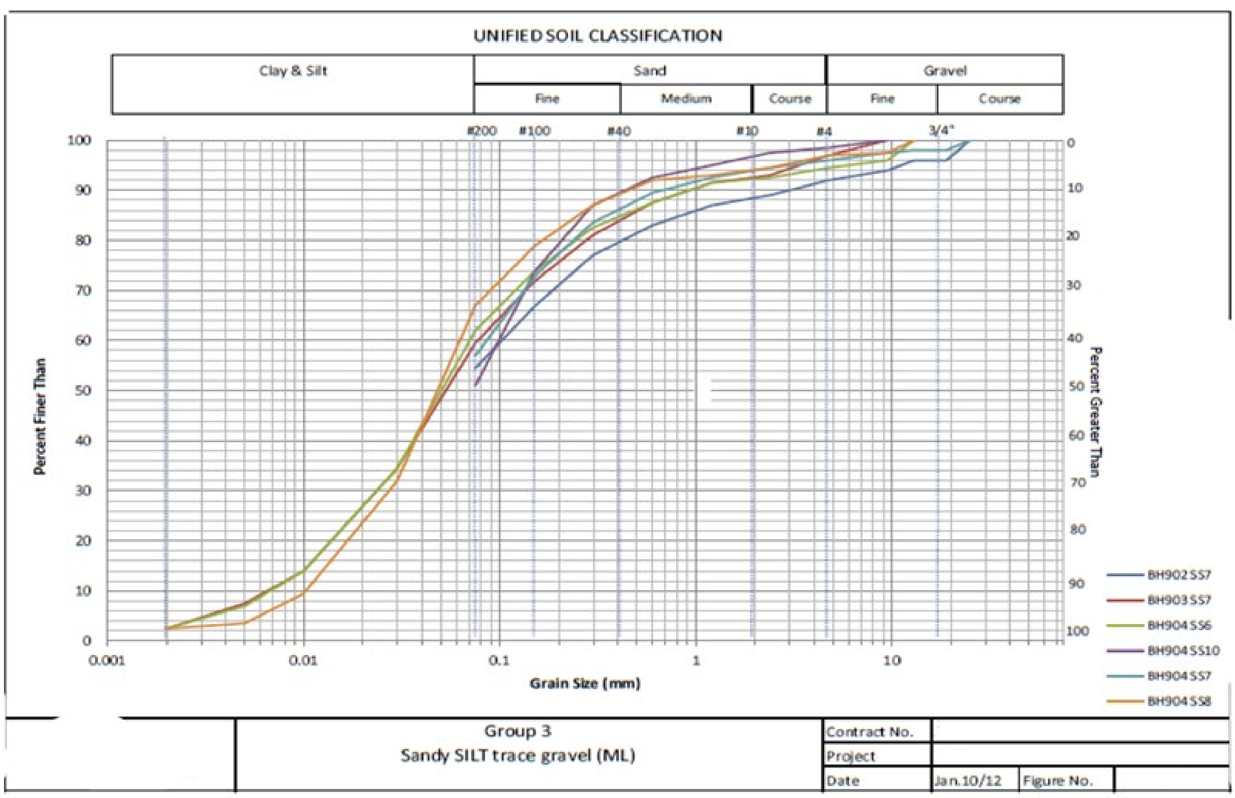


Figure 10 Example Grain Size Distribution

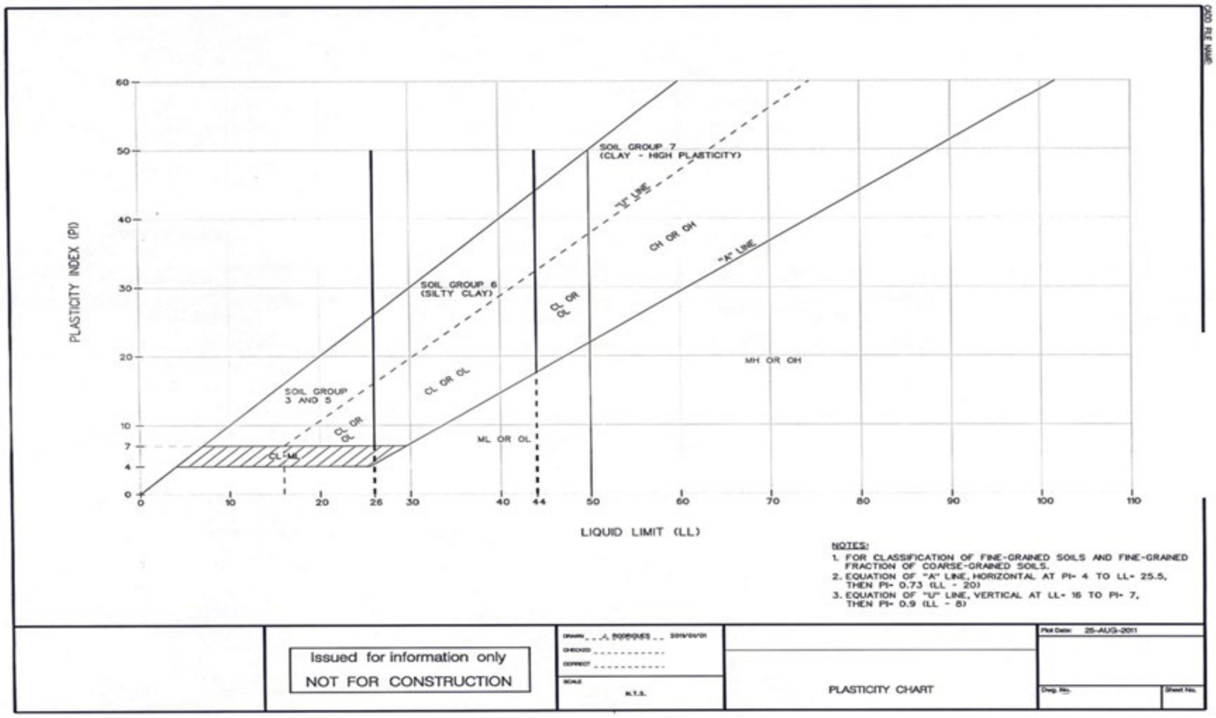
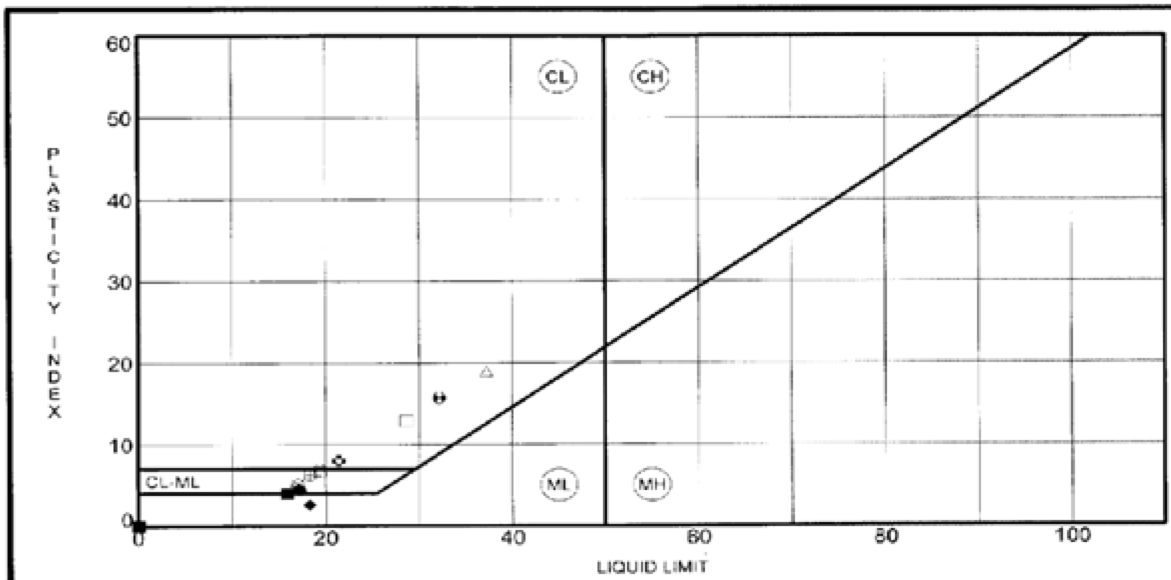


Figure 11 Bank Atterberg Limits Test Results



Specimen Identification		LL	PL	PI	Fines	Classification
J85-22-200	3.05	17	13	4	60	SANDY SILTY CLAY(CL-ML)
J85-22-200	9.15	NP	NP	NP	44	SILTY SAND(SM)
J85-22-200	15.24	NP	NP	NP	41	SILTY SAND(SM)
J85-22-200	18.20	NP	NP	NP	85	SILT(ML)
J85-22-201	13.72	NP	NP	NP	81	SILT with SAND(ML)
J85-22-206	0.76	21	13	8	55	SANDY LEAN CLAY(CL)
J85-22-206	15.24	NP	NP	NP	99	SILT(ML)
J85-22-206	19.81	37	18	19	100	LEAN CLAY(CL)
J85-22-207	0.00	17	12	5	58	SANDY SILTY CLAY(CL-ML)
J85-22-207	3.05	18	12	6	60	SANDY SILTY CLAY(CL-ML)
J85-22-207	18.28	29	16	13	99	LEAN CLAY(CL)
J85-22-207	19.80	32	16	16	99	LEAN CLAY(CL)
J85-22-208D	10.67	NP	NP	NP	45	SILTY SAND(SM)
J85-22-208D	13.72	NP	NP	NP	20	SILTY SAND(SM)
J85-22-209	0.00	19	13	6	60	SANDY SILTY CLAY(CL-ML)
J85-22-209	7.62	16	12	4	54	SANDY SILTY CLAY(CL-ML)
J85-22-211	0.76	18	16	2	56	SANDY SILT(ML)
J85-22-212	3.05	17	12	5	64	SANDY SILTY CLAY(CL-ML)
J85-22-212	12.20	NP	NP	NP	96	SILT(ML)
J85-22-212	16.76	NP	NP	NP	75	SILT with SAND(ML)

Logo

ATTERBERG LIMITS' RESULTS

Project

Location:

Number: 700467-053

Figure 12 Example Atterberg Limits Test Results

6.3 Soil Classification, Definitions and Descriptions

6.3.1 Soil Classification

6.3.1.1 General

a) To minimize the confusion in describing soils, Metrolinx has adopted the Unified Soil Classification System, as outlined in the following ASTM standards for use on all Metrolinx projects:

1) ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

2) ASTM D2488, Standard Practice for Description and Identification of Soils (Visual- Manual Procedure).

b) The consultant shall use the most current ASTM Standard when classifying soils for Metrolinx projects.

c) ASTM D2487 uses a series of flowcharts, which use grain size distribution and plasticity index test results, to determine the description. Some modifications to these flow charts were carried out by Metrolinx and the modified charts are attached to this Standard as Figure 13Figure 17. Table 13 outlines the ASTM chart and corresponding Metrolinx replacement chart.

Table 13 Metrolinx/ASTM Flow Chart Replacement Comparison

ASTM Standard Flow Charts and Figures	Metrolinx Flow Charts and Figures
Figure 1 - Flow Chart for Classifying Fine-Grained Soils (50% or More Passes No. 200 Sieve)	**Figure C1 - Flow Chart for Classifying Fine-Grained Soils (50% or More Passes No. 200 Sieve)**
Figure 2 - Flow Chart for Classifying Organic Grained Soil (50% or More Passes No. 200 Sieve)	**Figure C2 - Flow Chart for Classifying Organic Grained Soil (50% or More Passes No. 200 Sieve)**
Figure 3 - Flow Chart for Classifying Coarse Grained Soils (More than 50% Retained on No. 200 Sieve)	**Figure C3 and C4 - Flow Chart for Classifying Coarse Grained Soils (More than 50% Retained on No. 200 Sieve)**
Figure 4 - Plasticity Chart	**Figure C5 - Plasticity Chart**

d) The most significant difference between the ASTM Flow Charts and Metrolinx's Flow Charts is in the naming convention. The ASTM charts use the terms "lean and fat" to describe clay as well as "poorly graded" and "well graded" to describe sands and gravels. These terms have been removed in the Metrolinx flow charts.

6.3.1.2 Organic Soils

a) The description of organic soils depends on the percentage, type and distribution of organic matter within the soil.

b) Soils containing up to 5% by weight of organic matter shall be described in accordance with the standards for inorganic soils, with the modifier "trace organics" added to the written description. Where un-decomposed organic material is discernible in the soil, these components shall be listed in brackets (example: roots, leaves, twigs, shells).

c) Soils containing more than 5% matter shall be considered organic soils and shall be classified and described in accordance with Table 14 and as per ASTM D4318.

Table 14 Classification of Organic Soils

Category	Name	Organic Content (% by weight)	Group Symbol	Distinguishing Characteristics for Visual Identification
Organic Matter	Fibrous Peat	75% to 100%	PT	Light weight, spongy and often elastic at natural water content. Shrinks considerably upon air drying. Much water squeezes from sample.
	Amphorous Peat	75% to 100%	PT	Light weight, spongy but not usually elastic at natural water content. Shrinks considerably on air drying. Much water squeezes from sample.
Highly Organic Soils	Silty Peat	30% to 75%	PT	Relatively light weight, spongy. Thread usually weak and spongy near plastic limit. Shrinks on air drying; medium dry strength. Usually, can squeeze water from sample readily. Low dilatancy.
	Sandy Peat	30% to 75%	PT	Sand fraction visible. Thread weak and friable near plastic limit, shrinks on air drying, low dry strength. Usually, can squeeze water from sample readily, high dilatancy, "gritty".
Organic Soils	Organic Clayey Silt	5% to 30%	OH	Often has strong hydrogen sulfide (H_2S) odour. Thread may be tough depending on clay fraction. Medium dry strength, low dilatancy.
	Organic Sand or Silt	5% to 30%	OL	Threads weak and friable near plastic limit, or threads may not be rolled. Low dry strength; medium to high dilatancy.

Reference: Table based on Naval Facilities Engineering Command (NAVFAC) Design Manual 7.01, 1986, with some modifications.

6.3.1.3 Classification Procedures

a) The preceding sets out a standardized system for classifying and describing soil constituents (gradational and plasticity characteristics) and closely follows ASTM Standards. However, this is only part of a complete geotechnical soil description.

b) To provide a full, complete and uniform field description of soils, the following information should be determined and presented in the sequence indicated. This is also in accordance with the ASTM Standards identified above:

1) Colour.

2) Moisture Condition.

3) Compactness/Consistency (Based on N-values and shear strength).

4) Soil Constituents (as per attached charts).

5) Other Inclusions/Ancillary Information.

6) Odour.

7) Group Symbol (as per attached charts).

c) In addition to the above, the presence of cobbles and boulders within a soil deposit can be very significant in construction, and it is imperative that their presence be noted.

d) As a standard procedure, cobbles and boulders, if encountered in boreholes, test pits or exposures shall be noted, and the approximate depth encountered recorded.

e) Ancillary information shall also be included the soil description.

f) For fill soils there may be a number of different inclusions within a sample. It is essential that all of the observed inclusions are recorded, as the type of inclusion could affect the choice of soil testing required to classify the soil for the purpose of disposal.

g) Any ancillary information that is pertinent to the origin, composition or behaviour of the soil shall be noted, and where possible, the relative quantities of the materials shall be estimated and described using the modifiers, provided in the ASTM Standards. An example is that of non-soil inclusions in fill (e.g., "trace to some brick fragments and cinders").

6.3.2 Soil Groups

6.3.2.1 Once classified according to applicable standards, soils shall be grouped for presentation and discussion in the GDR and GDIR.

6.3.2.2 A review of key soil characterization indicators, specifically grain size distribution and Atterberg Limits test results, was carried out to assist in grouping soils. Reviewing approximately 600 grain size distribution tests and over 200 Atterberg Limits tests from various sites and projects, the soils in the Toronto area for glacial deposits can generally be classified into seven groups (excluding organic soils, lacustrine, alluvial, river deposits and fill material) as outlined in Table 15.

Table 15 Metrolinx Soil Group Names

Soil Group Number	Soil Group Name	Soil Group Envelope Figure Number
1	Sand and gravel	18 (C6)
2	Sand	19 (C7)
3	Glacial Till (silty clay till to silty sand till)	20 (C8)
4	Silty Sand/Sandy Silt	21 (C9)
5	Silt/Clayey Silt	22 (C10)
6	Silty Clay	23 (C11)

Soil Group Number	Soil Group Name	Soil Group Envelope Figure Number
7	Clay - High Plasticity	24 (C12)

6.3.2.3 A grain size distribution envelope for each soil group is provided in Figure 18 (C6) through Figure 24 (C12). Atterberg Limit envelopes for the plastic soils (silts and clays) are provided on Figure 25 (C13). These shall be used to compare laboratory test results and determine an appropriate soil group.

6.3.2.4 A summary of the grain size and Atterberg Limits envelopes is provided in Figure 16. The grain size distribution and Atterberg limits test results shall be compared to the envelopes provided above to determine an appropriate soil group.

6.3.3 Presentation of Data (Record of Borehole Sheet Descriptions)

6.3.3.1 General

a) Field and laboratory observations and the results of laboratory tests shall be used to prepare final soil descriptions that shall be presented on Record of Borehole and Test Pit sheets and used in the text of reports. The format for final Record of Borehole and Test Pit sheets is presented in Section 6.2. The standards that follow shall be used to prepare the written descriptions that shall appear on the final Records.

6.3.3.2 Final Borehole/Test Pit Log Descriptions

a) The basic soil descriptions form is as follows:

1) Colour, Moisture Conditions, Compactness/Consistency, Metrolinx STANDARD SOIL DESCRIPTION, Minor Soil Constituents (trace, some), Grading, Other Inclusions, Odour, other ancillary information (GROUP SYMBOL(S)).

2) The items in italics shall only be included if present or if significant. The descriptions shall be written with all items in standard lowercase print with the exception of the predominant soil type and group symbol, that shall be presented in block capital letters.

3) The description given on a borehole or test pit record shall generally incorporate several consecutive samples that have similar characteristics and that are anticipated to behave in a similar manner. The written description on the record shall reflect the range of characteristics of individual samples, the results of tests over the described interval and shall also include any characteristics or constituents implied from drilling or excavation behaviour.

6.3.3.3 Presentation of Laboratory Characteristic Testing Data

a) All laboratory data shall be assembled and compiled into soil groups and each soil group presented on a single laboratory test data sheet i.e., grain size distribution and Atterberg Limits sheets.

b) Grain size and Atterberg test results shall not be provided based on tests carried out at a specific borehole location, i.e., a sand and gravel, silty sand and clayey silt all on one grain size curve. These shall be presented with like-soil groups.

c) Grain size distribution curves for non-cohesive soils shall include the Coefficient of Uniformity (Cu) and the Coefficient of Curvature (Cc).

6.3.3.4 Presentation of Consolidation and Triaxial Laboratory Testing Data

a) All data for Consolidation and Triaxial testing shall be submitted in an approved tabular and graphical format, both PDF and Excel files.

b) Consolidation test data shall include an interpretation of the following:

1) Pre-Consolidation Pressure - Pc’.

2) Initial Void Ratio - e_o.

3) Compression Index - Cc.

4) Recompression Index - Cr.

c) Triaxial test data shall include an interpretation of:

1) C, φ, C’ and φ’ using Mohr Stress Circles (total stress and effective stress).

2) C’ and φ’ using Stress Path Method.

Table 16 Soil Group Envelopes

Coarse Material

US Sieve classification Sieve Opening Size (mm)	3/4″ 19	3/8″ 9.5	#4 4.75	#10 2	#20 0.85	#40 0.425	0.3	#100 0.15	#200 0.075
Soil Group # and Name	**Percent Passing**								
1 (SAND/GRAVEL)	90-100	75-100	55-100	44-100	32-92	25-60	20-55	10-40	3-25
2 (SAND)	100	100	100	100	92-100	80-100	50-100	20-80	3-30
3 (Sandy Silty CLAY trace gavel/sandy clayey SILT trace gravel (TILL))	95-100	90-100	85-100	80-100	77-100	72-100	-	55-90	40-80
4 (Silty SAND/Sandy SILT)	100	97-100	95-100	90-100	85-100	82-100	-	60-97	30-75
5 (SILT/Clayey SILT)	100	100	98-100	97-100	95-100	92-100	-	85-100	75-100
6 (Silty CLAY)	100	98-100	97-100	96-100	95-100	92-100	-	89-100	85-100
7 (CLAY high plasticity)	100	100	100	100	99-100	98-100	-	97-100	96-100

Fine Material: Passing 200 Sieve

Particle Diameter (mm)	0.04	0.03	0.02	0.01	0.005	0.0015	Liquid limit, %
Soil Group # and Name	**Percent Passing**						
1 (SAND/GRAVEL)	-	2.5-18	-	2-12	-	0-10	NP
2 (SAND)	-	2.5-18	-	2-18	-	0-10	NP
3 (Sandy Silty CLAY trace gavel/sandy clayey SILT trace gravel (TILL))	-	15-70	-	5-55	-	0-30	12-26 Above the A line
4 (Silty SAND/Sandy SILT)	10-50	-	6-35	3-25	2-18	2-13	NP/ML Below the A line
5 (Clayey SILT/SILT)	55-100	45-98	35-95	25-70	15-50	5-30	14-26 Above the A line
6 (Silty CLAY)	78-98	-	-	60-93	-	23-55	26-44 Above the A line
7 (CLAY high plasticity)	-	-	92-100	-	80-95	55-75	>44 Above the A line

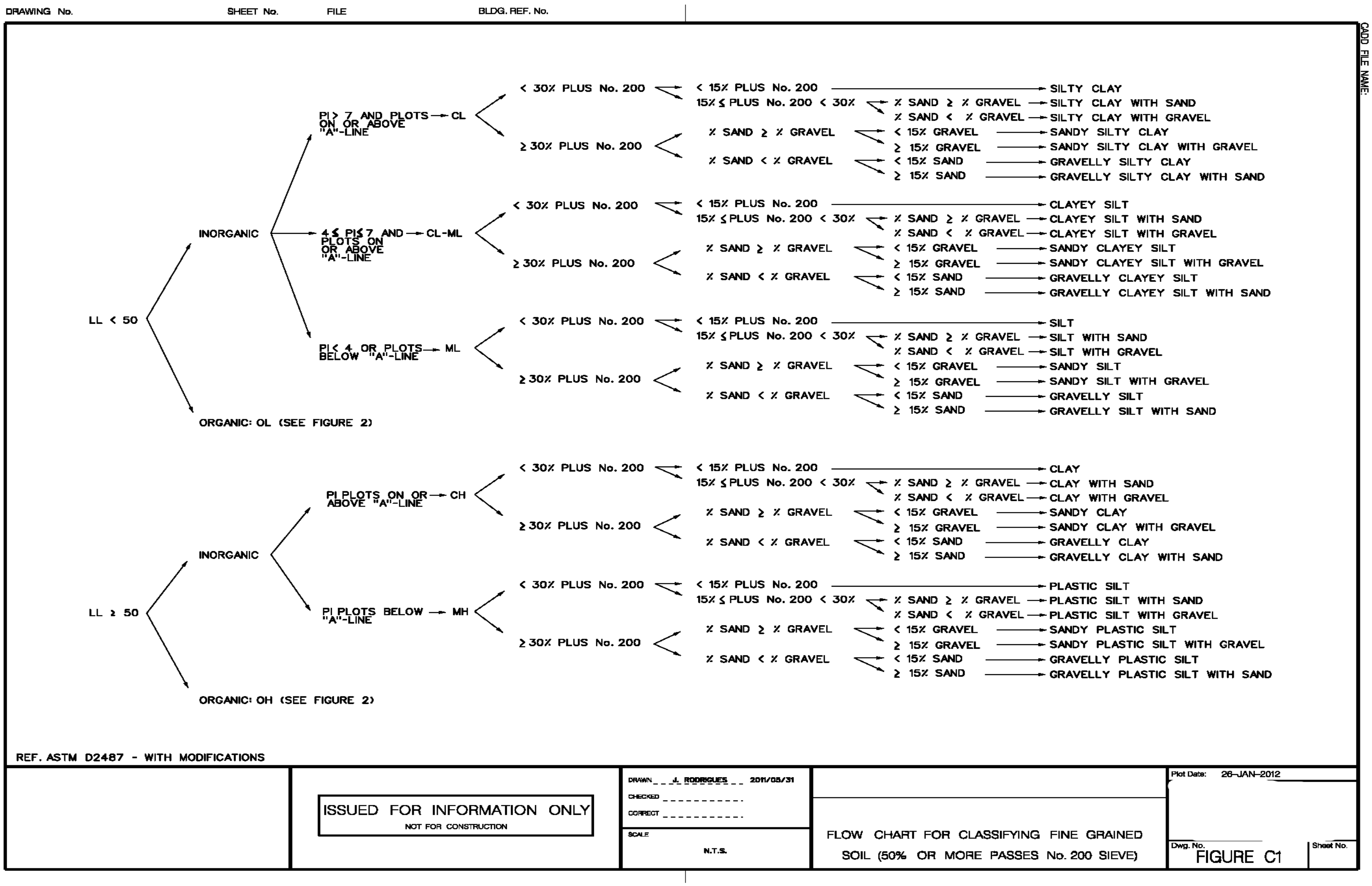


Figure 13 (C1) Flow Chart for Classifying Fine Grained Soil (50% or More Passes No. 200 Sieve)

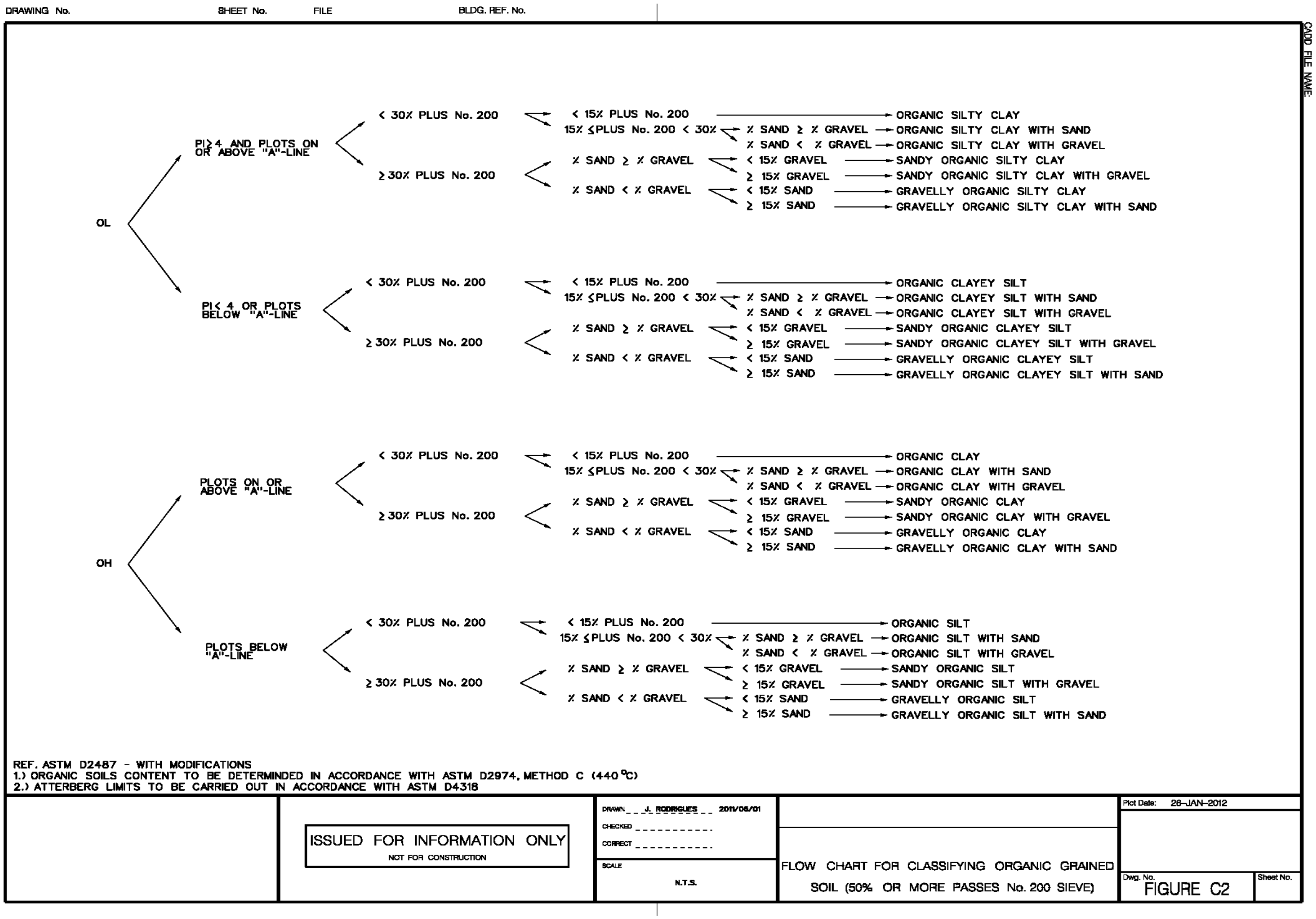


Figure 14 (C2) Flow Chart for Classifying Organic Grained Soil (50% or More Passes No. 200 Sieve)

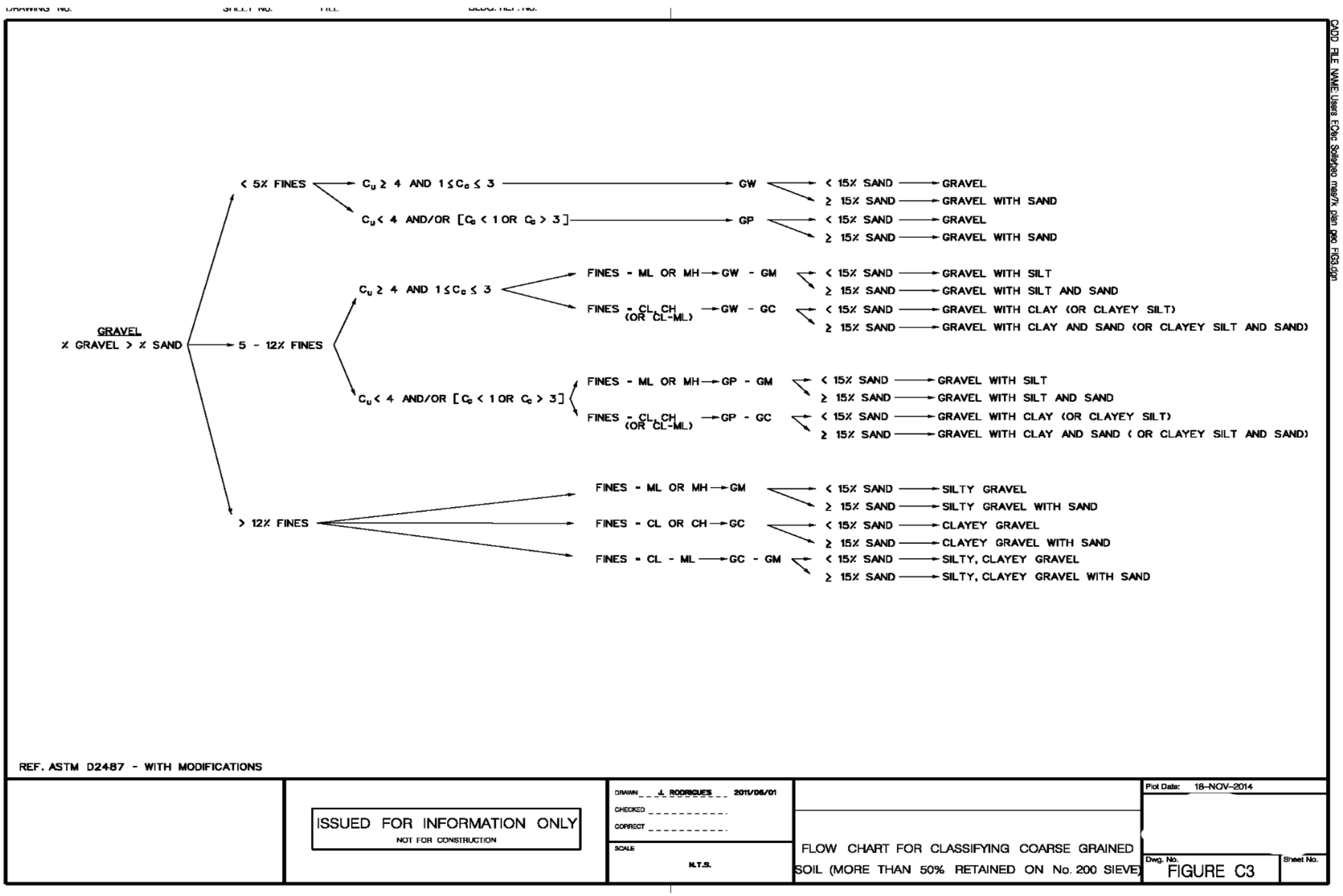


Figure 15 (C3) Flow Chart for Classifying Coarse Grained Soils (More than 50% Retained on No. 200 Sieve)

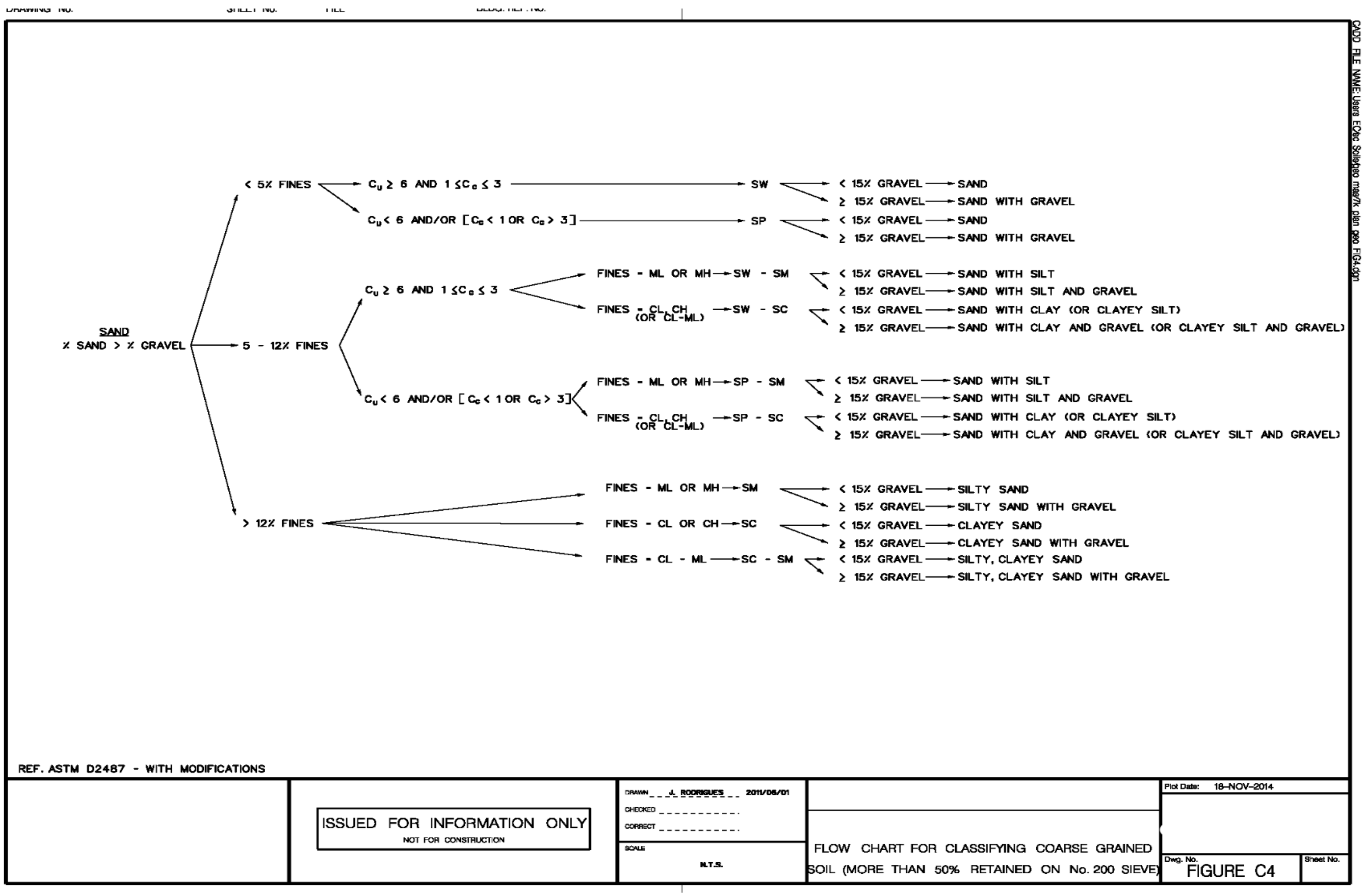


Figure 16 (C4) Flow Chart for Classifying Coarse Grained Soils (More than 50% Retained on No. 200 Sieve)

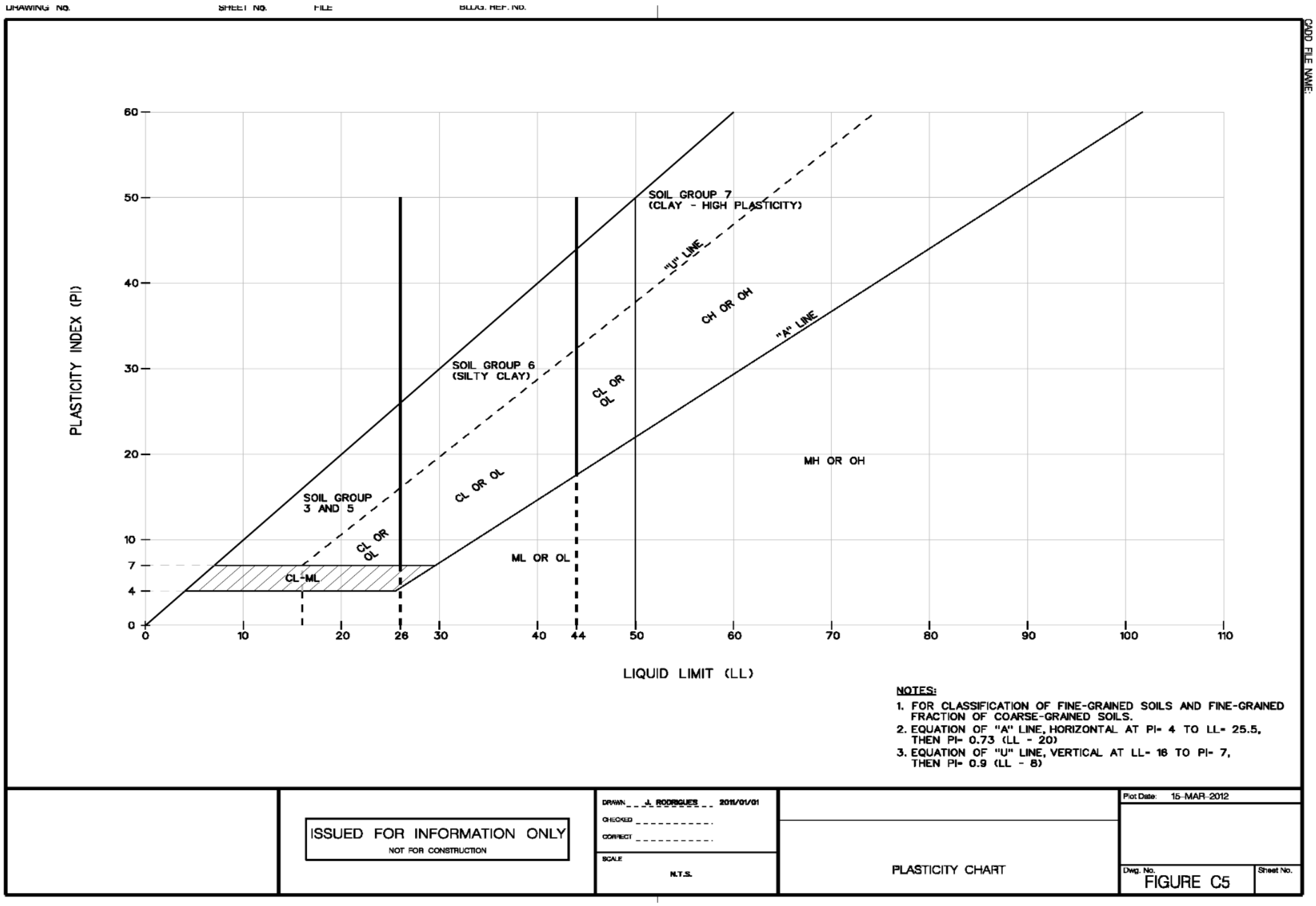
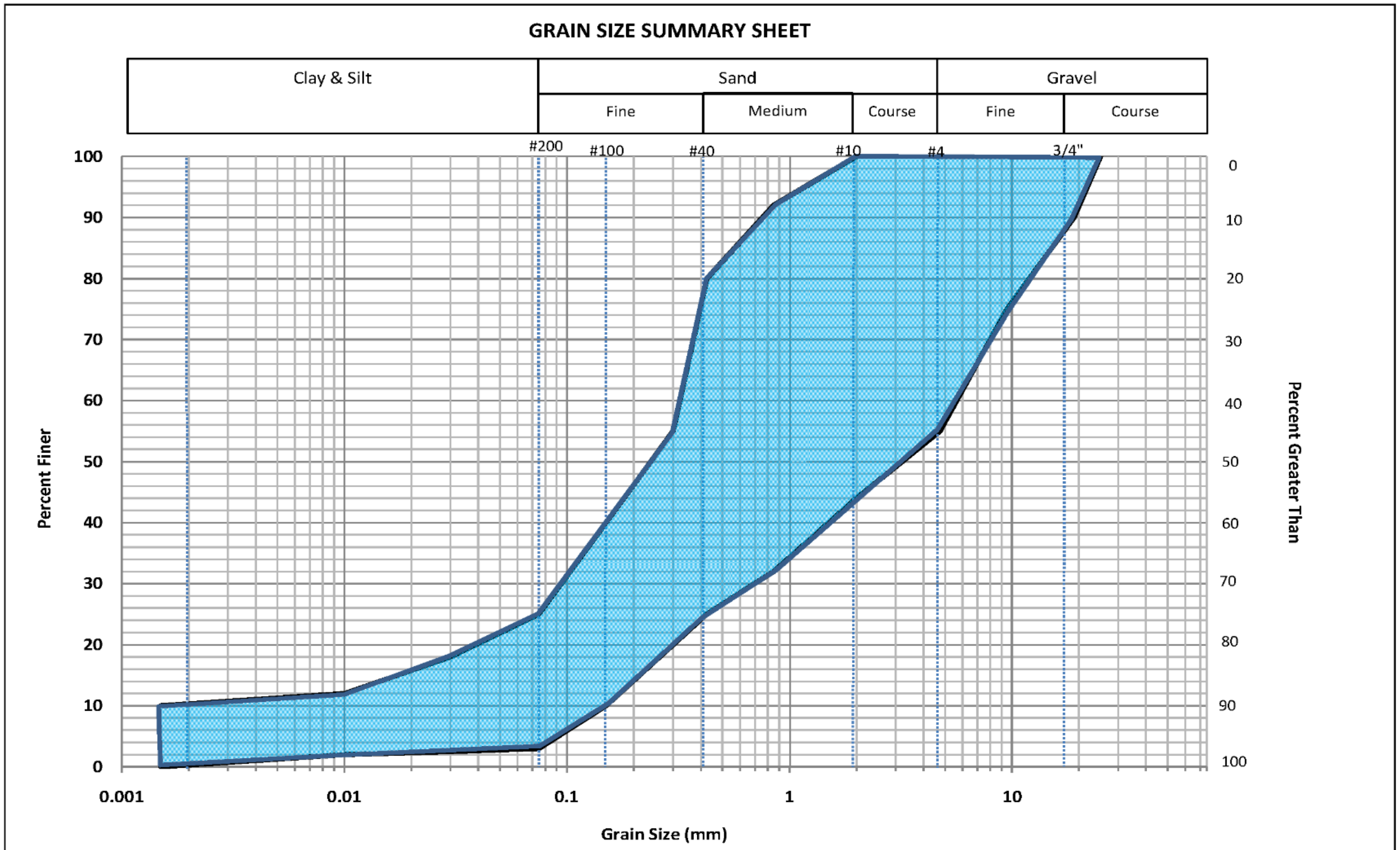
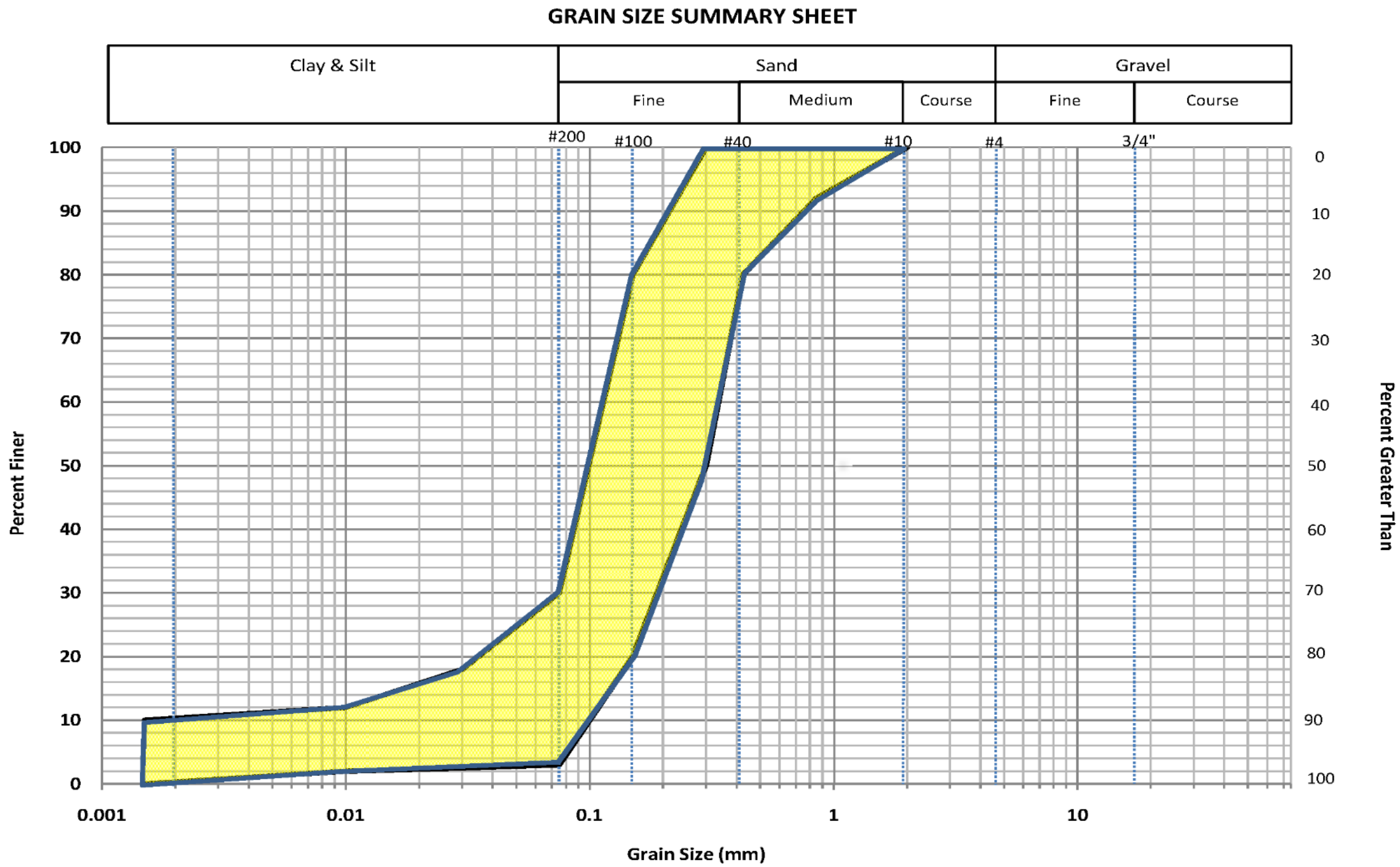


Figure 17 (C5) Plasticity Chart



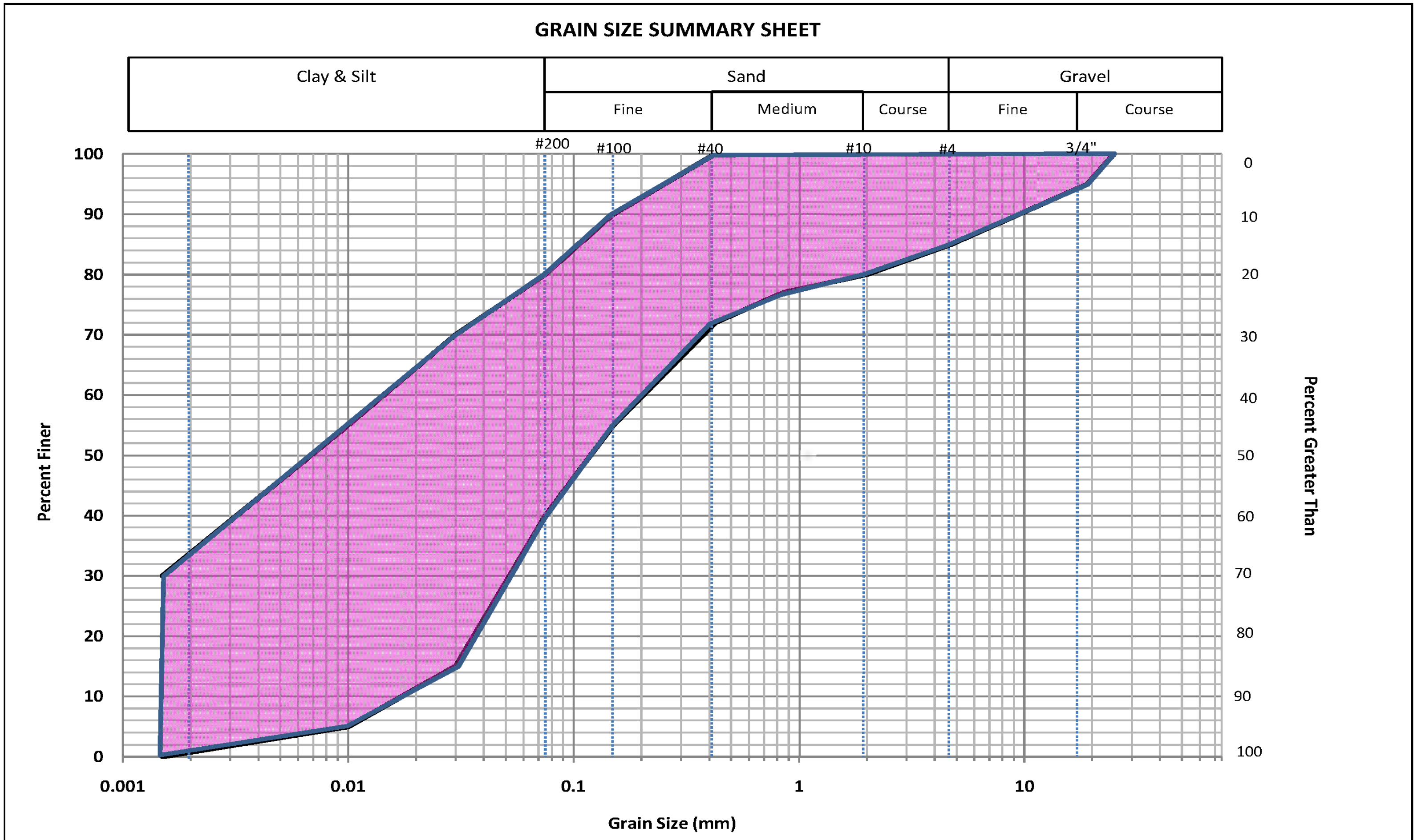
	GRAVEL & SAND- Group1	Contract No.			
		Project			
		Date:	July 2011	Figure No.	C6

Figure 18 (C6) Grain Size Summary Gravel & Sand (Group 1)



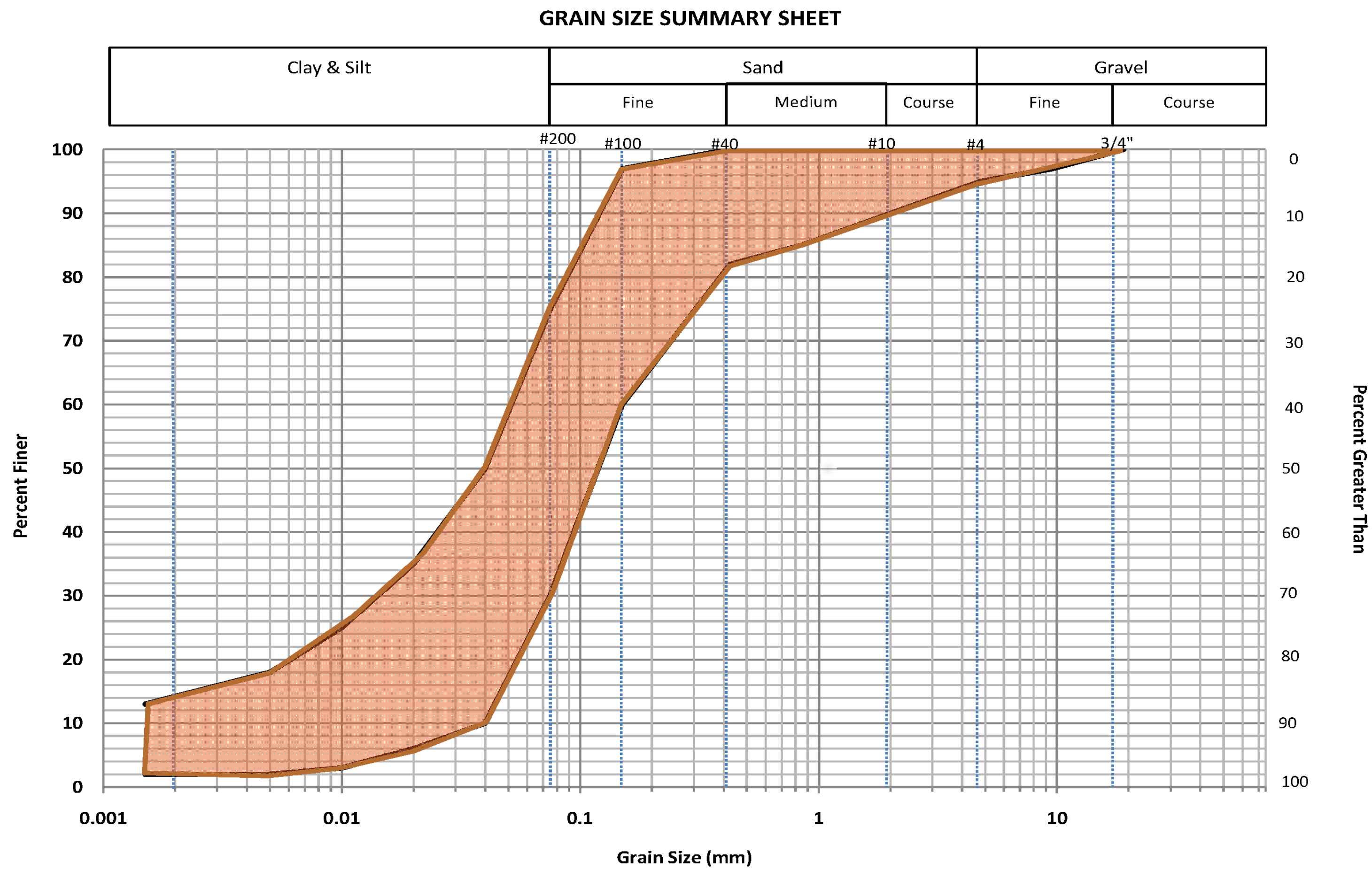
	SANDS- Group2	Contract No.			
		Project			
		Date:	July 2011	Figure No.	C7

Figure 19 (C7) Grain Size Summary Sands (Group 2)



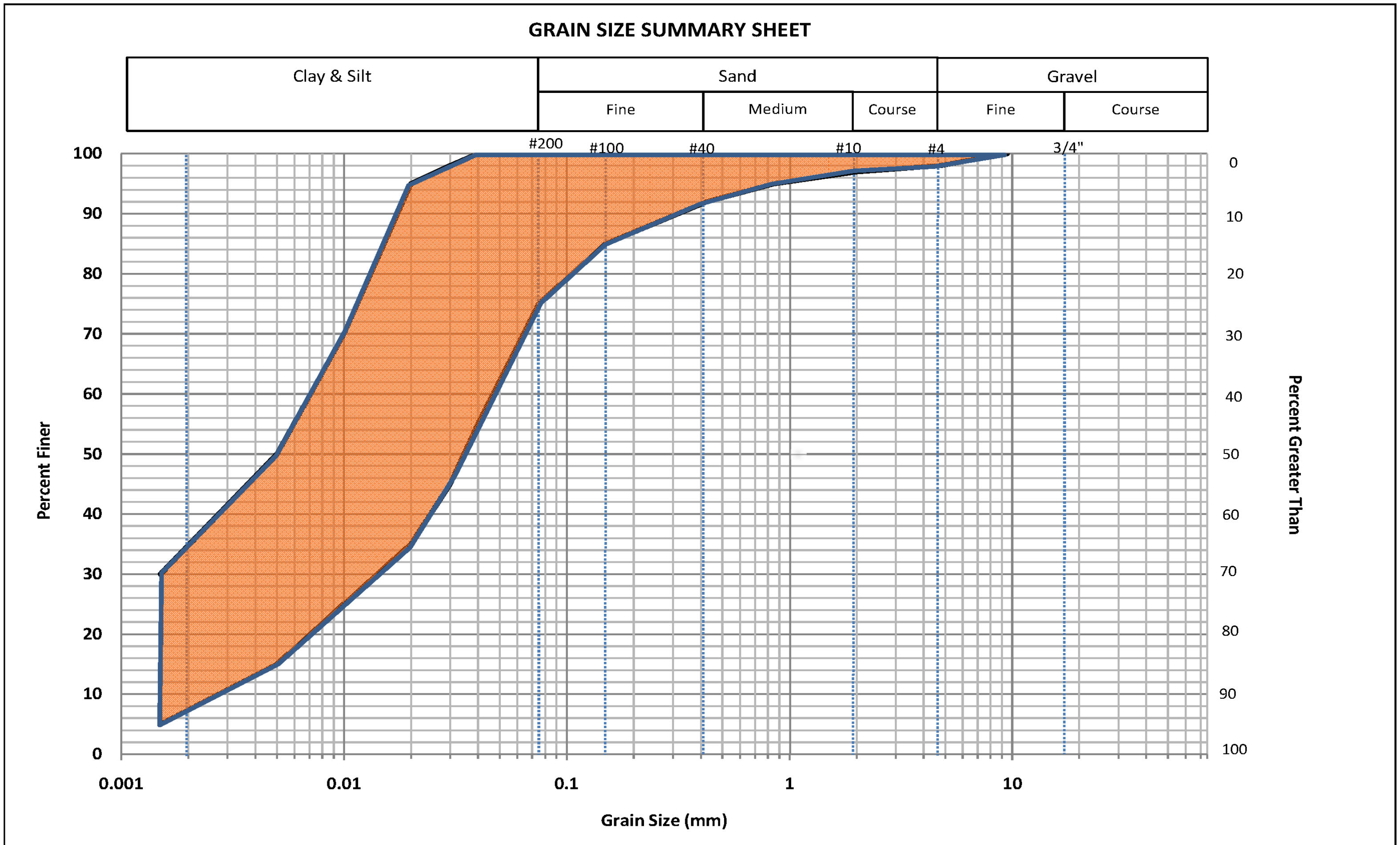
	TILL- Group3	Contract No.			
		Project			
		Date:	July 2011	Figure No.	C8

Figure 20 (C8) Grain Size Summary Till (Group 3)



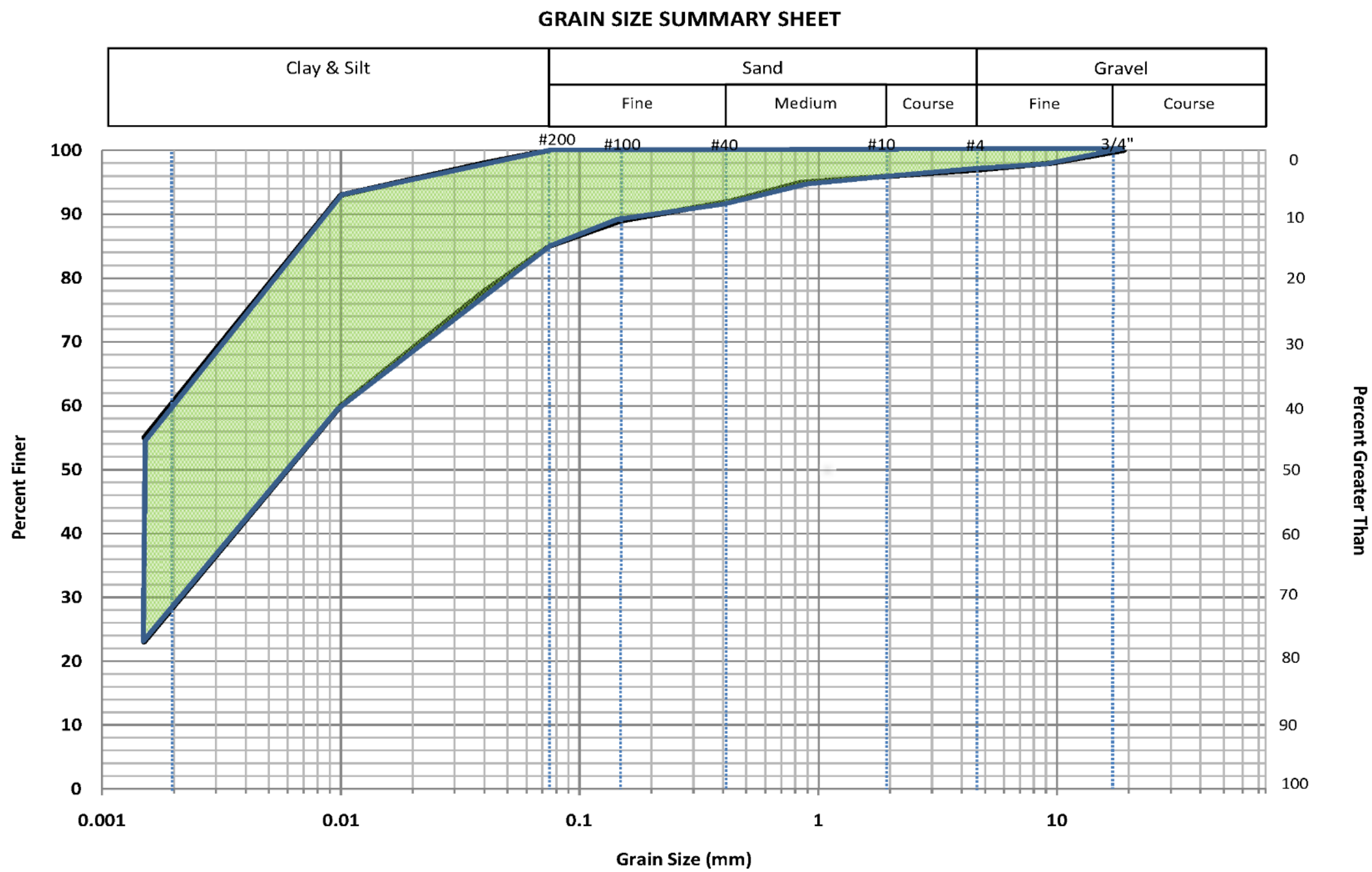
	Silty SAND/sandy SILT- Group 4	Contract No.			
		Project			
		Date:	July 2011	Figure No.	C9

Figure 21 (C9) Grain Size Summary Silty Sand/Sandy Silt (Group 4)



	CLAYEY SILT/SILT- Group5	Contract No.			
		Project			
		Date:	July 2011	Figure No.	C10

Figure 22 (C10) Grain Size Summary Clayey Silt/Silt (Group 5)



	SILTY CLAY- Group6	Contract No.			
		Project			
		Date:	July 2011	Figure No.	C11

Figure 23 (C11) Grain Size Summary Silty Clay (Group 6)

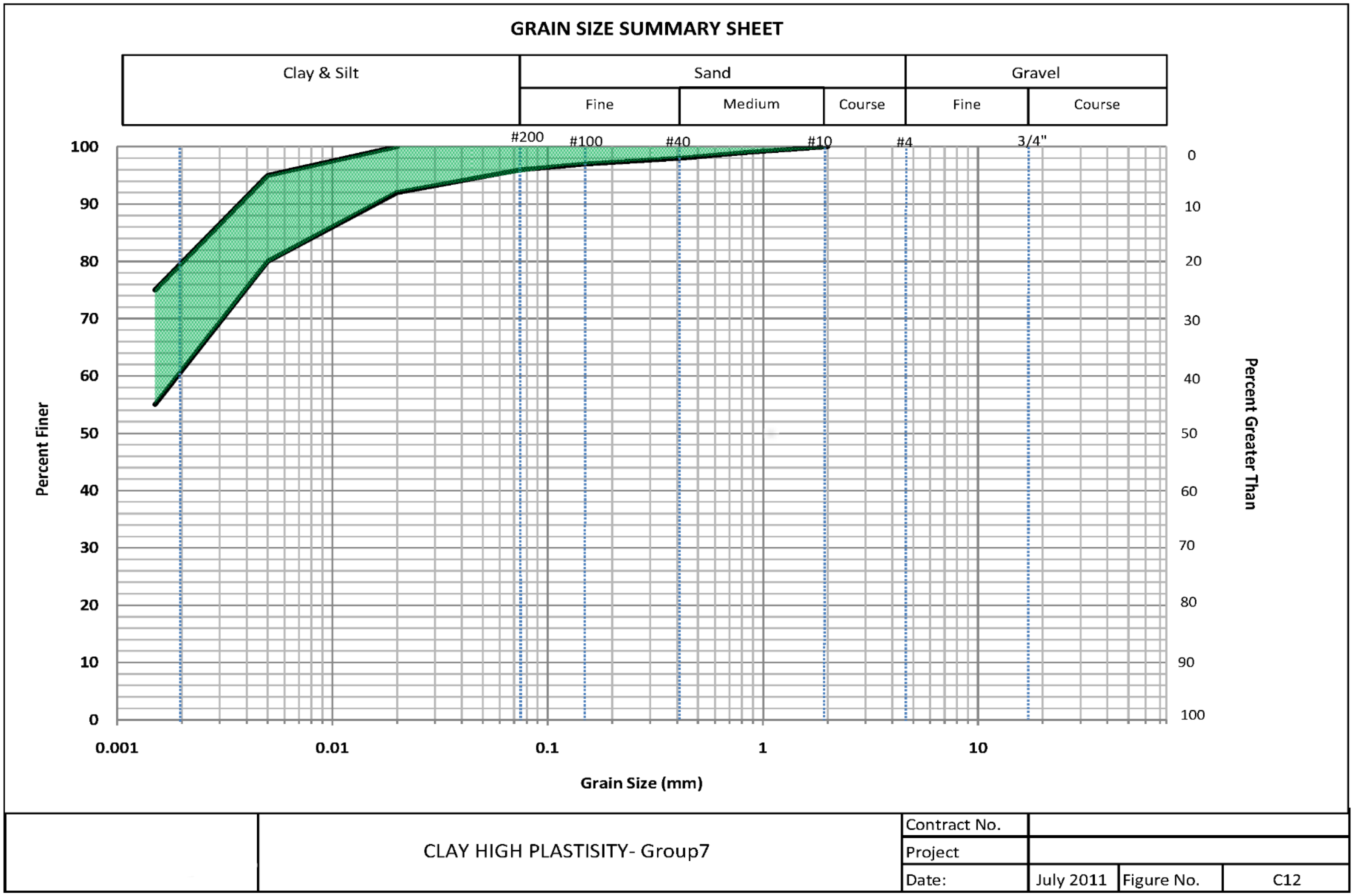
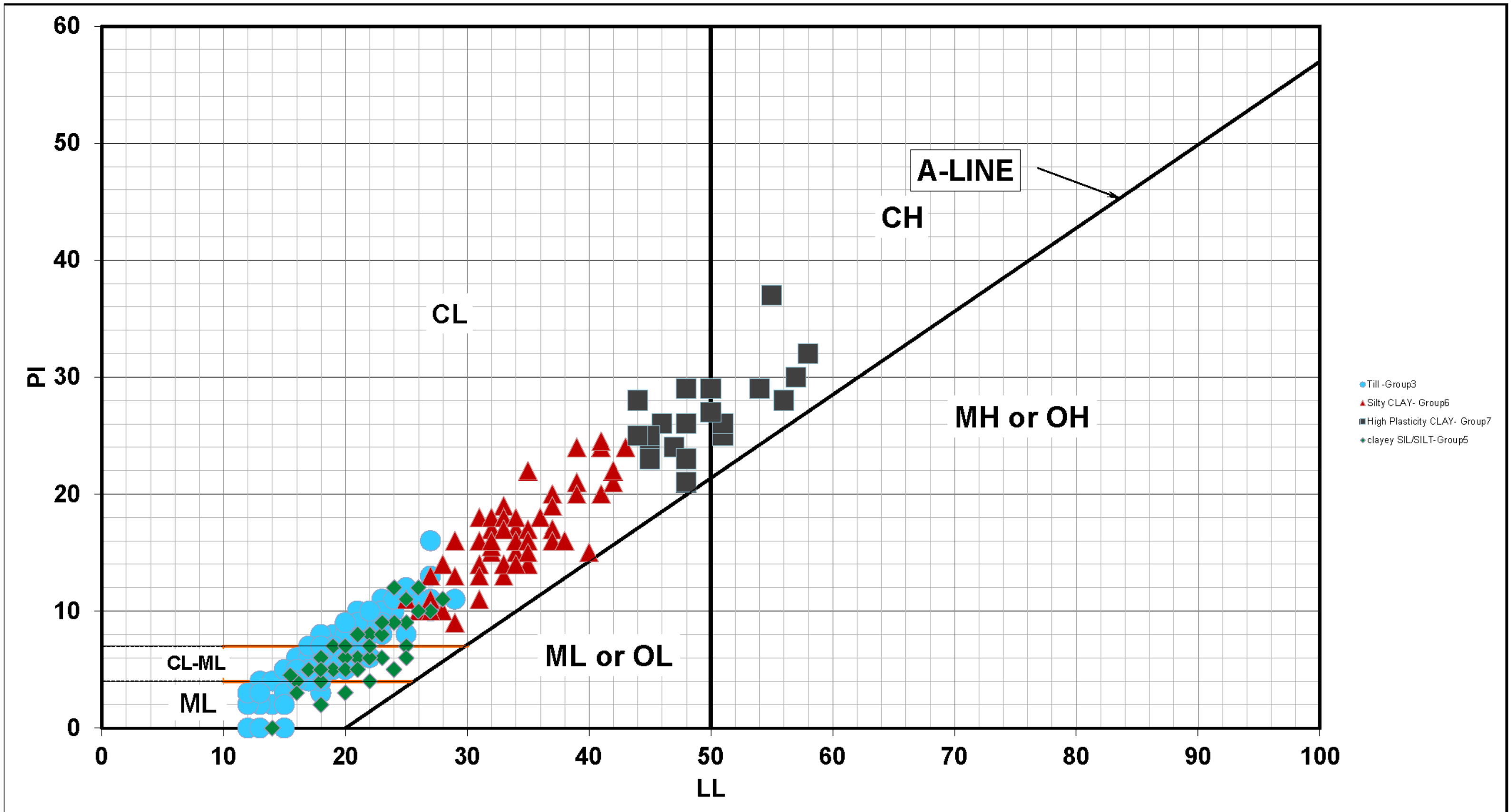


Figure 24 (C12) Grain Size Summary Clay High Plasticity (Group 7)



	PLASTISITY CHART	Contract No.			
		Project			
		Date:	May-14	Figure No.	C13

Figure 25 (C13) Atterberg Limit Envelopes for Plastic Soils

6.4 Rock Classification, Definitions and Descriptions

6.4.1 Rock Classification

6.4.1.1 The GTA is underlain by shale of the Blue Mountain and Georgian Bay Formations. The Blue Mountain Formation is covered by a thick layer of overburden and is typically not encountered in work carried out for Metrolinx projects. The Georgian Bay Formation is predominantly shale but contains interbeds of limestone and siltstone.

6.4.1.2 The standard rock description and definitions adopted by Metrolinx shall be tailored to this specific rock type formation in the GTA yet shall also conform to nationally and internationally recognized systems.

6.4.1.3 The descriptive terms and definitions provided in this section are consistent with and based on international standard practice as set out in the Journal of the International Society of Rock Mechanics (ISRM), the Quarterly Journal of Engineering Geology (QJEG) and the bulletin of the International Association of Engineering Geology (IAEG). The sequence of rock descriptions has been selected to reflect the characteristics of the Georgian Bay Formation that will most significantly affect design and construction.

6.4.1.4 This system of rock classification, definition and description shall be used for all geotechnical reports prepared for Metrolinx.

6.4.1.5 If a Metrolinx project is impacted by other rock types, including but not limited to, Queenston Shale, in other areas beyond the GTA then rock classifications, properties and features of these rock types shall be applied.

6.4.2 Sequence of Rock Description

6.4.2.1 To provide a full, complete and uniform description of rock, the following information shall be determined and presented in the sequence indicated.

a) Weathering State (1.1)

b) Structure (1.2)

c) Colour (1.3)

d) Strength (1.4)

e) Rock Type (1.5)

f) Discontinuity Data (1.6)

  1) Lost Core (1.6.1)

  2) Total Core Recovery (1.6.2)

  3) Rock Quality Designation (1.6.3)

  4) Fracture Index (1.6.4)

  5) Discontinuity Orientation (1.6.5)

6.4.2.2 Weathering State

a) The degree of weathering of a rock directly affects its strength, permeability and ripability. The degree of weathering of rock core shall be assessed as soon as it is removed from the ground, as exposure can result in further degradation of the rock.

b) The weathering terms that shall be used for the description of rock cores are provided in Table 17.

Table 17 Weathering Classification

Term	Description	Symbol
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces	W1
Slightly Weathered	Discolouration indicates weathering of rock material on discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker than in its fresh condition.	W2
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	W3
Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	W4
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	W5

c) The weathering symbol provided in this table shall be included in a separate column on the rock core drill hole record.

d) In the GTA it is common to encounter weathered shales with interbeds of less weathered limestone or siltstone. The interbeds are often thin (<200 mm) but can still have an adverse effect on excavation due to their strength. Where weathered rock with less weathered interbeds are found in a rock core, both rock types shall be fully described, with the elevation and thickness of the interbeds recorded.

6.4.2.3 Structure

a) The predominant structural feature of the Georgian Bay Formation underlying the GTA is its bedding.

b) The bedding is essentially horizontal and is typically characterized by relatively thick beds of shale and thinner interbeds of limestones and siltstones.

6.4.2.4 Colour

a) The colour(s) of the predominant rock type and of the interbeds shall be provided.

b) Single word colour descriptions, modified by an appropriate adjective are sufficient.

c) The colour shall be described when the rock is wet; staining of the rock or on the surfaces of its discontinuities shall also be noted.

6.4.2.5 Strength

a) The strength of a rock shall be described qualitatively based on the uniaxial compressive strength of intact samples.

b) This strength shall either be measured directly by a uniaxial compressive strength test, correlated using the measured point load index or inferred in the field using a pocket knife and geological hammer.

c) The descriptive terms provided are based on the International Society of Rock Mechanics (ISRM) standard classification system.

d) It shall be noted that a variety of rock strength classifications have been developed in various parts of the world and in some cases, "strong" rock under one classification is considered "weak" rock under another classification.

e) Consistent adherence to Table 18 is, therefore, essential; it provides both the range of uniaxial compression strengths for each descriptive term and a field procedure for the qualitative assessment of the rock strength.

Table 18 Rock Strength Classification

Description	Field Identification	Approx. Range of Uniaxial Compressive Strength (MPa)	Grade
Extremely weak	Indented by thumbnail	.25-1.0	R0
Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0	R1
Weak	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25	R2
Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of geological hammer	25-50	R3
Strong	Specimen requires more than one blow of geological hammer to fracture it	50-100	R4
Very strong	Specimen requires many blows of geological hammer to fracture it	100-250	R5
Extremely strong	Specimens can only be chipped with geological hammer	>250	R6

f) As with weathering, a symbol is provided to identify the rock strength classification. This symbol should be placed on the rock log and the symbol used shall apply to the predominant rock type.

g) The appropriate strength classification shall also be included with the description of each predominant type of interbed.

6.4.2.6 Rock Type

a) Detailed petrographic rock descriptions, focused on mineral constituents, their distribution and crystal size, are not necessary. Such characteristics do not influence engineering behaviour as much as weathering, strength and discontinuity distribution.

b) The predominant rock type in the GTA is shale containing interbeds of siltstone and limestone.

c) Within the written rock description, the rock type should be identified with capital letters.

d) Rock types and descriptions anticipated are defined below:

1) SHALE: A dark grey to black, very fine grained and fissile rock.

2) CRYSTALLINE LIMESTONE: Contains calcium carbonate (fizzes in presence of hydrochloric acid (HCl)) minerals in crystalline form. Crystals are often visible with the unaided eye and fossils are often present and visible.

3) SHALY LIMESTONE: A very fine-grained fissile rock cemented with calcium carbonate. It is often lighter in colour than shale and reacts with HCl.

4) SILTSTONE: A very fine grained, non-fissile and non-reactive rock.

e) After each rock type is named, the percentage of the rock type shall be listed in brackets. The percentage shall be based on the total recovered length of core as opposed to the total length of core drilled (i.e., all percentages shall add up to 100%).

f) The rock description shall include a listing of all interbeds greater than 50 mm thick. The elevation of the centre of the interbed, its thickness and rock type should be provided. Example;
92.65 m: 75 mm siltstone
92.10 m: 60 mm shaly limestone

6.4.2.7 Discontinuity Data

a) General

1) Rock discontinuity data of core samples shall be recorded as soon as possible after recovery from the ground.

2) If a split tube liner is used, then the discontinuity data shall be recorded with the rock still in the tube.

3) If a solid tube is used, then the discontinuity data shall be recorded immediately after placing the rock in a core box.

4) Measurements of RQD and Fracture Index shall be based on natural fractures only; drilling and handling induced fractures

shall be excluded. As a guide, clean, fresh surfaces that are oriented at close to 90 degree to the core axis and that can be rejoined with only a hairline separation are typically drilling or handling induced. If there is doubt about the origin of a fracture, it shall be counted as a natural fracture. While this standard is conservative from the design perspective, for construction purposes it may suggest a rock mass that is easier to excavate. Therefore, every effort shall be made to delineate unnatural fractures in the field and, as these are identified, they shall be marked directly on the core.

5) The various rock discontinuity terms are defined in the following sections as are standards for measuring these items.

b) Lost Core

1) Lost core generally indicates poor ground conditions and when it occurs, it shall be clearly identified on the borehole log. When the core loss is significant, which for GTA rock may be taken as 150 mm, the core loss interval shall be blocked on the rock log and labelled "NO CORE". The actual interval shall be inferred from grind marks on ends of recovered core, rubble zones within the core, drill rods "dropping" during drilling or by rapid advance at the start of a run-in weak rock as the rock is washed away. If the location of the "NO CORE" interval cannot be identified, the zone shall be assigned to the lower end of the core run.

c) Total Core Recovery

1) Total Core Recovery (TCR) is the total cumulative length of all core recovered in the core barrel divided by the length drilled and shall be recorded on a "per run" basis. Prior to measuring the total recovered length, the core shall be assembled with joints aligned and with rubble zones reassembled to the extent practicable. The TCR shall be presented on logs as a percentage.

d) Rock Quality Designation

1) Rock Quality Designation (RQD) is the cumulative length of intact core that is greater than 100 mm long, divided by the length drilled (Standardized for N sized drill core). The RQD shall be reported as a percentage on a "per run" basis. The length of intact core shall be measured along the core axis.

2) The RQD is often used as an overall indicator of rock mass quality. The descriptions of rock quality as related to RQD are provided in Table 19.

Table 19 Rock Quality Description Based on RQD

RQD (%)	Description
0 - 25	Very poor
25 - 50	Poor

RQD (%)	Description
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

3) It shall be noted that the above system applies to all rock types, and therefore, covers a very broad range of rock quality. When RQD is based on rock core other than ‘N’ size, this information shall be noted on the logs.

e) Fracture Index

1) Fracture index is defined as the number of fractures occurring per 0.3 m length of core. This provides a more sensitive indication of fractured zones within a rock mass because it is measured over a shorter interval then either TCR or RQD. The Fracture Index shall be reported as a simple count of fractures. If there are more than 25 fractures/0.3 m length, the Fracture Index shall be reported as >25.

f) Discontinuity Orientation

1) The discontinuity orientation shall be recorded as an angle relative to the core axis. Fractures that are perpendicular to the core axis are at 90 degrees and those parallel to the core axis are at 0 degree. Where the relatively highly fractured Georgian Bay Formation is encountered, the orientation of fractures shall be recorded on the logs for each 0.3 m interval. For example, a 0.3 m interval may contain seven fractures perpendicular to the axis and one at 45 degrees to the axis. In this case the fracture Index shall be eight and orientations of 45 degrees and 90 degrees will be indicated on the log for that interval. It shall not be necessary to indicate the fracture orientation for each individual fracture.

6.4.3 Presentation of Data (Drillhole Record Descriptions)

6.4.3.1 General

a) Field and laboratory observations and the results of laboratory tests shall be used to prepare final rock descriptions that shall be presented on Drillhole Records and used in the text of the reports. The format for the final Drillhole Records is presented in Section 6.2.

b) The standard Drillhole Record format includes the presentation of discontinuity data in graphical form. The standards that follow shall be used to prepare the written description that shall appear on the final records.

c) The basic rock description form is:

1) WEATHERED STATE, COLOUR, ROCK TYPE [REPEAT FOR INTERBEDS]

LIST INTERBEDS >50 mm THICK WITH ROCK TYPE AND ELEVATION.

d) The description shall be written with all items in standard lower-case print with the exception of the rock types, which should be presented in block capital letters.

6.4.3.2 Example Rock Descriptions

a) The standard form of the rock description is illustrated by the following example:

"Moderately weathered to fresh, dark grey, very weak to weak, SHALE, with slightly weathered to fresh, light grey, weak to medium strong CRYSTALLINE LIMESTONE and SHALY LIMESTONE.
END OF DRILL HOLE
Limestone layers generally less than 50 mm thick except at the following elevations:
92.35 mm: 100 mm x.1.
91.80 mm: 50 mm x.1.
91.70 mm: 60 mm s.1.
91.55 mm: 50 mm x.1.
NOTES
x.1. - crystalline limestone.
s.1. - shaly limestone.

6.5 Rock Testing

6.5.1 Introduction

6.5.1.1 The Georgian Bay Formation (GBF) sedimentary bedrock possesses unique characteristics. The Georgian Bay Formation (GBF) is characterized by noncalcareous, greenish to bluish-grey shale, interbedded with limestone, siltstone, and sandstone (interbeds/hard beds/hard layers). The abundance and thickness of these hard beds decrease stratigraphically downward. The sandstone and siltstone beds are commonly calcareous, and typically fossiliferous. The main geo-structural features of the GBF are lamina, bedding and sub-vertical joints, and shear zones.

6.5.1.2 Adequate in-situ and laboratory testing shall be carried to characterize the bedrock. This section provides the testing requirements in addition to the requirements stipulated in the previous sections.

6.5.1.3 The requirements of this section are mandatory when underground excavations include transit tunnels, caverns, shafts and underground stations that will be founded in bedrock. Presence of bedding, bedding joints, weak bedding planes, subvertical joints as well as random joints along with joint infillings are collectively called anisotropic features of GBF. These features dominate the behaviour of GBF bedrock during construction. Design shall be developed such that anisotropic features of GBF are clearly accounted for.

6.5.2 Bedrock Testing

6.5.2.1 General

a) Vertical boreholes, inclined boreholes, in-situ and laboratory testing, and geophysical surveys that are justified and calibrated with boreholes shall be carried out. All boreholes that are extended into bedrock in addition to recovering rock cores for logging shall be surveyed with Optical Televiewer (OTV) and Acoustic Televiewer (ATV) surveys for the rock section.

6.5.2.2 Inclined Boreholes

a) General

1) The subvertical and bedding joint spacing, persistence and joint properties for large caverns, particularly with small rock cover, shall affect the construction/mining staging, support requirements as well as the extent of the zone of influence.

b) Spacing and Persistence of Subvertical Joints:

1) While vertical borehole logs and televiewer survey tracks can be used to extract data to form statistical basis for spacing of bedding joints, data on the subvertical joint spacing and other characteristics can be obtained by advancing inclined boreholes. Properly configured inclined boreholes are expected to provide information on subvertical joint spacing and persistence. Figure 26and Figure 27 illustrate subvertical joints, their persistence and the use of inclined boreholes.

2) Where land is available and surface obstacles and other constraints such as subsurface and overhead utilities clearances allow, inclinations in the order of 30 degrees to 40 degrees from vertical have been successfully achieved for transit projects.

3) Adequate number of inclined boreholes shall be advanced to capture subvertical joint characteristics. Inclined boreholes shall be deep enough to reasonably cover adequate horizontal plan projected length of underground structures and stations. Inclined borings shall be oriented with the boring direction ideally perpendicular to the strike and dip direction of the joint orientations expected to be encountered in the project area. By advancing inclined boreholes, data shall be collected for different subvertical joint spacing and indications of joint persistence. This shall require careful survey of borehole elevation and logging of rock cores retrieved from boreholes. Close correlation shall be carried out between core logs and televiewer survey tracks to reduce anomalies to an acceptable accuracy. Based on the data that shall be obtained from a cluster of inclined boreholes, an estimate of joint persistence can be made.

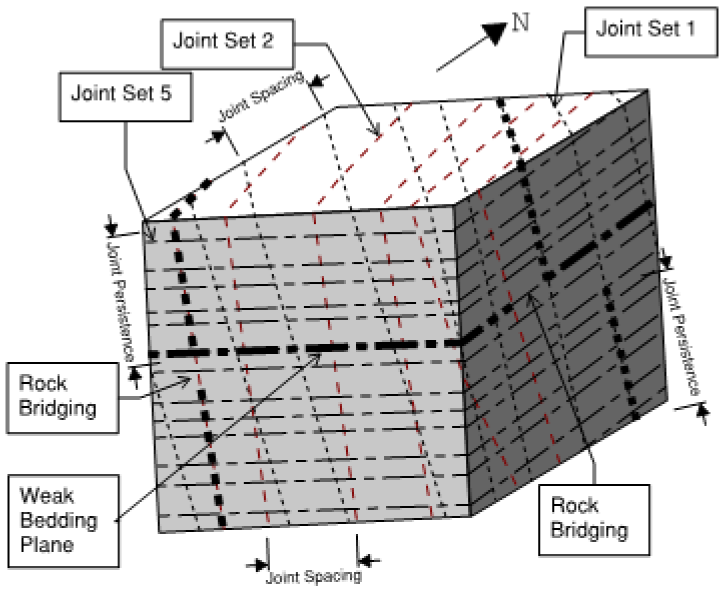


Figure 26 Joint Spacing, Joint Persistence and Rock Bridging. (Joint sets orientations are for illustration and do not represent joint sets orientations in GBF. Diagram by Hatch)

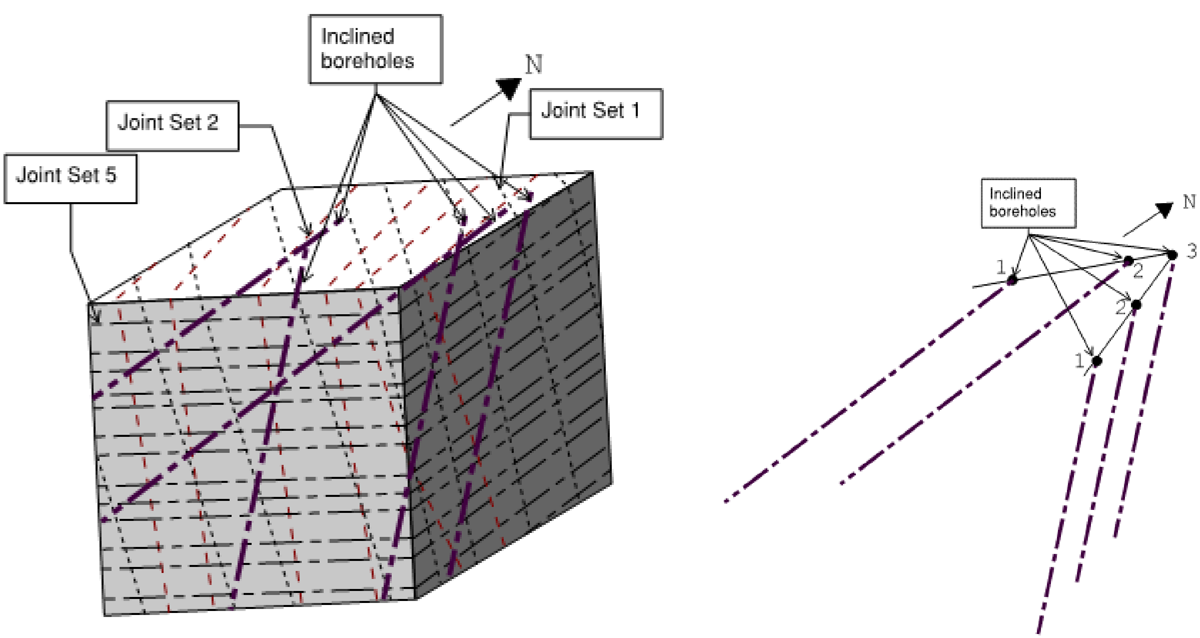


Figure 27 Sample Bedrock Block Diagram - A Cluster of Testing Boreholes

6.5.2.3 Direct Shear Test

a) Large Scale In-situ Direct Shear test

1) When site conditions or design and construction staging allows, the GBF Shale shall be tested in direct shear test using large scale testing methods. Shear stiffness of rock joints is a doubly scale dependent parameter and, hence for numerical

modelling of bedrock the results of large-scale test can be assumed to be a proper representation of in-situ rock mass behaviour subject to shearing mechanisms. Large scale direct shear test includes the effect of joints length and joint surface characteristics both for intact large specimens, for specimens with pre-existing discontinuities and for pre-existing discontinuities with infilling and clay gouge.

2) It should be noted that there are standards for large scale direct shear test on rock joints. ASTM D4554-2012, "Standard Test Method for In Situ Determination of Direct Shear Strength of Rock Discontinuities", along with many other studies conducted for dam foundations and mines that provide guidelines, procedures and interpretations for large scale in-situ direct shear testing.

b) Laboratory Direct Shear Test

1) In lieu of large-scale shear test, laboratory direct shear tests can be carried out, if scale effects and joint surface properties are accounted for in estimating joint shear stiffness as per methodologies provided in the existing large body of publications such as:

1. Bandis, S.C., Lumsden, A.C., and N.R. Barton. Fundamentals of Rock Joint Deformation. Int. J. Rock Mech. Min. Sci. & Geomech. Abst. Vol. 20, No. 6, pp. 249-268, 1983.

2. Barton, N.R. Deformation Phenomenon in Jointed Rock. Geotechnique 36, No. 2, pp. 147-167. 1986.

3. Barton, N. Shear Strength Criteria for Rock, Rock Joints, Rockfill and Rock Masses: Problems and Some Solutions. J. Rock Mechanics and Geotechnical Engineering. 5 (2013) 249-261.

2) Since inclined boreholes, if configured properly, intersect subvertical as well bedding joints, in addition to uniaxial compressive strength of intact rock, direct shear test shall be done on samples containing bedding and subvertical joint surfaces. Direct shear tests shall be done with measurement of horizontal as well as normal displacement for several constant normal stresses. At least three levels of normal stresses commensurate with the states of normal stresses expected at various elevations of underground structures shall be included.

3) Direct shear tests shall include intact samples, samples containing discontinuities (e.g., bedding joints, weak bedding planes, subvertical joints) and discontinuities with infilling and clay gauge. Direct shear tests on initially intact samples shall include a stage of testing after shear planes are fully developed (multistage shear test with repositioning). All tests shall be carried out according to applicable standards such as ASTM D5607 "Standard Test Method for Performing Laboratory

Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force" and ISRM Suggested Method for Laboratory Determination of the Shear Strength of Rock Joints.

c) Joint Roughness Coefficient (JRC) and Joint Wall Compressive Strength (JCS)

1) The JRC and JCS of existing discontinuity shall be measured before and after shear tests as well as on specimens that are collected only for joint surface tests.

2) Joint Roughness Coefficient (JRC): Using a roughness-profile gauge "comb" method, the amplitude of joint surface asperities (a) per measurement of length (L) shall be measured. The roughness measurement is illustrated on the top right corner of Figure 28. Typical roughness profiles for JRCs are shown on Figure 29. The index tests shall be performed on at least ten samples of each joint set.

3) Joint Wall Compression Strength (JCS): The joint wall compression index shall be done on saturated joint wall surfaces. For this purpose, a Schmidt Hammer shall be used to perform the test on clamped water saturated joint samples retrieved from borehole cores. The JCS is generally expected to be less that UCS of rock. The values of JCS from Schmidt Hammer tests on saturated fresh or weathered joint surfaces shall be converted using a density-uniaxial compression strength conversion according to existing standards and guidelines.

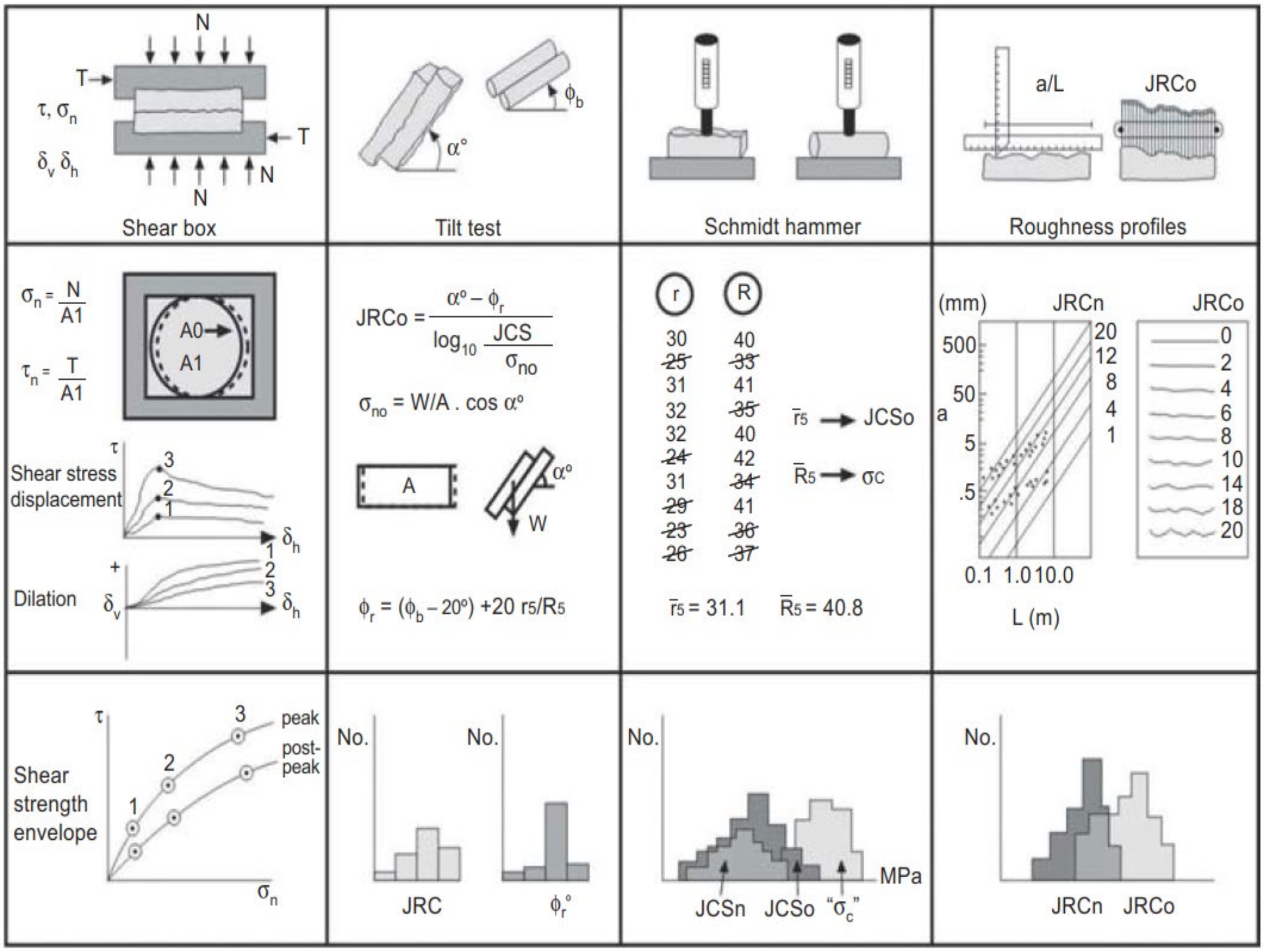


Figure 28 Shear Box and Testing of Rock Joints

(Note that at the bottom row the statistical distribution of each key joint strength and stiffness parameters is schematically shown). Four columns of diagrams showing 1) direct shear tests principals, 2) tilt test principals, 3) Schmidt hammer test principals, and 4) roughness recording principals (diagram from Barton 2013)

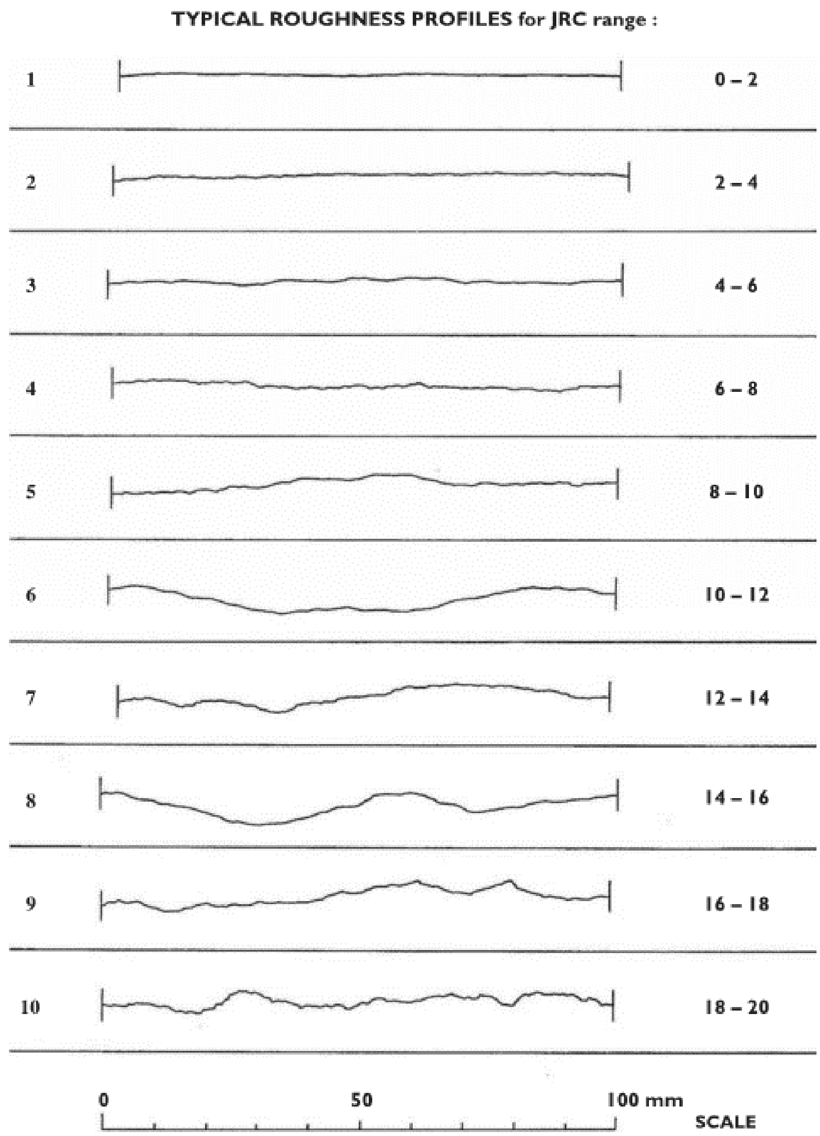


Figure 29 Typical Roughness Profiles for JRC (Bandis et al, 1983)

6.5.2.4 Other Laboratory Tests

a) General

1) Other laboratory tests including Density, Point Load Test (PLT), Uniaxial Compressive Strength (UCS), Triaxial Compressive Strength (TCS), Elastic modulus (E), Poisson's ratio, bulk density test, Indirect Tensile Strength (Brazilian Test - BTS), Swelling Strain and Pressure, Cerchar Test, and Slake Durability, shall be carried out on samples obtained from rock cores. The elastic modulus measurements shall also be carried out on secondary cores in a horizontal direction.

b) Unconfined Compressive Strength Test

1) Unconfined compressive strength (UCS) test shall be caried out

in accordance with ASTM D7012-14. UCS tests shall be carried out on vertically obtained cores as well as secondary cores taken in a horizontal direction.

2) Young's modulus and Poisson's ratio shall be determined from UCS test and reported.

c) Triaxial Test

1) Triaxial compressive strength tests shall be carried out according to ASTM D7012-14 for both vertical and re-cored (secondary cored) rock samples. Results of triaxial tests along with vertical triaxial moduli and Poisson ration and triaxial moduli and Poisson's ratio for the secondary cores in horizontal direction shall be reported. Test samples shall be taken from shale, shale with hard layers, as well as for hard layers.

d) Brazilian Tensile Test

1) Brazilian tensile test in accordance with ASTM D3967-16 shall be carried out on representative samples of shale, shale with hard layers, and hard layers.

e) Point Load Test

1) Point load test shall be conducted according to ASTM D5731-16. Both axial and diametral tests shall be carried out. Point Load Strength Index values $Is_{(50)}$ shall be calculated and reported.

f) Slake Durability Test

1) Slake Durability tests in accordance with ASTM D4644-16 shall be carried out on shale samples.

g) Swell Test

1) Swell test on laboratory samples shall be carried out for both horizontal and vertical swelling potentials as follows:

A. Free Swell Test.

B. Semi-confined Swell Test.

C. Null Swell Test.

2) Moreover, water content, salinity and calcite content tests shall also be carried out on GBF shale samples.

3) Swell test results shall be reported using plots for horizontal and vertical swelling potentials similar to Figure 30.

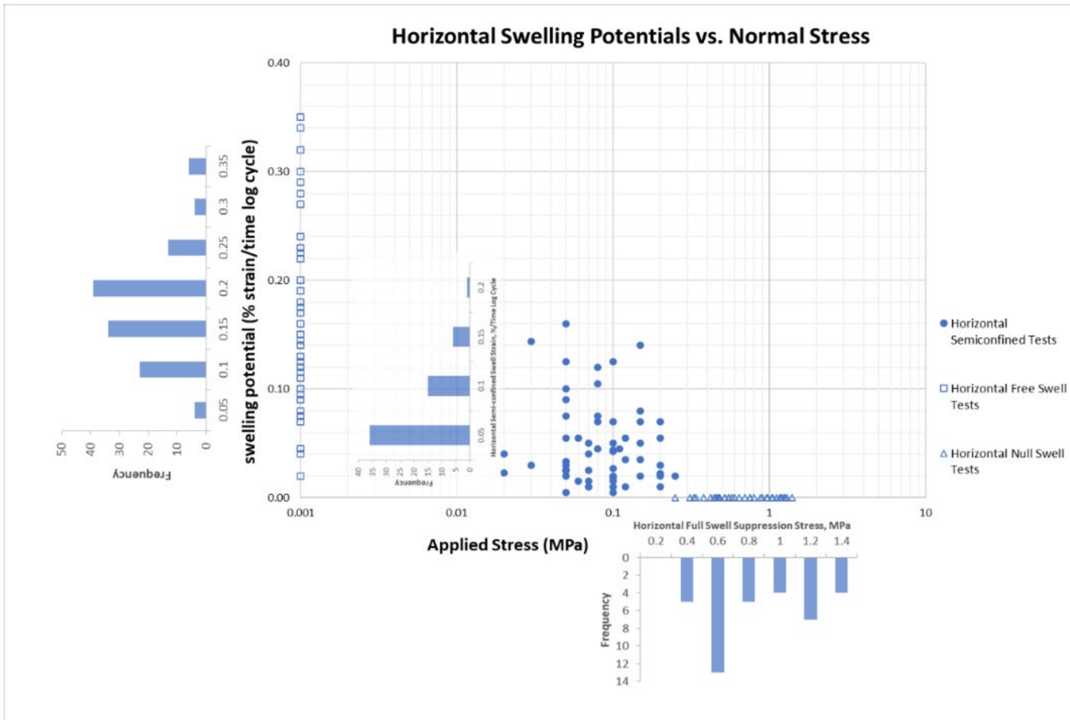


Figure 30 Typical Plots for Swelling Potentials

Data points on the vertical axis represent the free swell test results, data point on the horizontal axis represent the null swell test results and other data points are from semi-confined swell test results. (Format by Hatch)

h) Abrasivity Test

1) Cerchar abrasion testing on rock samples shall be carried out according to ASTM D7625-10 to provide a measure of the bedrock abrasivity for determining cutter wear for rock excavation. Tests shall be carried out on samples of shale and hard interbeds to be representative of rock mass conditions.

i) Rock Drillability Test

1) Rock drillability (SINTEF) testing shall be carried out on rock cores and shall include Abrasion Value (AV), Abrasion Value Cutter Steel (AVS), Brittleness Value (S20), Sievers's J-Value (SJ), Drilling rate Index(DRI), Bit Wear Index (BWI), and Cutter Life Index (CLI).

6.6 Hydrogeology Assessment

6.6.1 Statement of Purpose

a) The purposes of a detailed hydrogeological assessment are to:

1) Characterize existing hydrogeological conditions.

2) Assess changes in groundwater quality across a project area.

3) Determine the need for and options for groundwater control

such as during construction dewatering and long-term groundwater control.

4) Quantify potential impacts of construction to the local groundwater regime (e.g., impacts to nearby groundwater users such as private well owners or nearby natural features such as wetlands and streams, or effects of construction dewatering on the settlement of adjacent structures).

5) Determine the need for and nature of any mitigation measures.

b) To support these objectives, the level of detail within a hydrogeological assessment shall be sufficient to support the submission of permit applications (e.g., Permit To Take Water [PTTW], Environment Activity and Sector Registry [EASR], or municipal approvals), and to assess potential impacts to local water resources and water-related features. Table 20 lists some key considerations when developing the scope of work for and while conducting a hydrogeological assessment.

Table 20 Key Considerations for the Hydrogeological Assessment

	Considerations
1	What are the hydro-stratigraphic unit(s) (HUs) and their hydraulic properties?
2	What are the groundwater flow conditions for each identified hydro-stratigraphic unit, including seasonal variation?
3	Will there be a potential need for waterproofing structures, dewatering during construction activities, or permanent dewatering?
4	Will a PTTW be required and, if so, what information is needed to support the application?
5	Will an Environmental Activity and Sector Registry (EASR) registration be required and, if so, what information is needed to support it?
6	Is there potential for a perched water table to be present?
7	Is there potential for artesian conditions and, if so, what mitigation options might be applicable?
8	Is there potential for groundwater to be under the influence of surface water?
9	What is the groundwater quality at the site (i.e., project area), and how does it compare to applicable standards such as Provincial Water Quality Objectives (PWQO), municipal Sewer By-Laws or others as applicable?
10	Is there potential for contaminants of concern to be migrating into or out of the project area?
11	What is the site infiltration rate in support of design of infiltration facilities?

6.6.2 Approach

6.6.2.1 Certifications

a) As a minimum, consulting firms undertaking a hydrogeological assessment shall have one of the following certifications:

1) A valid Certificate of Authorization (C of A) to practice provided by the Association of Professional Geoscientists of Ontario and shall adhere to the professional liability insurance regulations in accordance with the Professional Geoscientists Act.

2) A valid C of A under the Professional Engineers Act that qualifies it to provide services that would also constitute the practice of professional geoscience.

b) Practitioners shall either be a Professional Geoscientist (P.Geo.) or a Professional Engineer (P.Eng.) experienced in hydrogeological assessment. Additional accreditation may be required to complete work for specific projects, for example, Ministry of Transportation of Ontario Registry, Appraisal and Qualification System (RAQS) qualifications and Qualified Persons (QP) under Regulation 153/04 may be necessary to complete work related to these types of sites.

c) Laboratories completing chemical analysis shall be certified under the Standards Council of Canada (SCC) or the Canadian Association for Laboratory Accreditation Inc. (CALA). The requirements for laboratory certifications shall be confirmed before any laboratory testing takes place.

d) The well contractor and the well technician shall be licensed in accordance with the Wells Regulation (Ontario Regulation 903, as amended) under the *Ontario Water Resources Act*.

e) When other third parties are involved in the scope of investigation, their qualifications or experience performing the required task, or both, shall be readily available if requested.

6.6.2.2 Hydrogeological Assessment Standards and Practices

a) The hydrogeological assessment, including all aspects, tasks, and in-situ and laboratory testing, shall follow the most recent editions of the following standards and practices:

1) Ontario Water Resources Act and Regulations

A. Ontario Regulation 903 (O. Reg. 903), Wells

B. Section 34 of the Ontario Water Resources Act for Permit to Take Water

C. Ontario Regulation 387/04 Water Taking and Transfer

D. Ontario Regulation 63/16, Registrations Under Part II.2 of the Act - Water Taking)

2) Ontario Clean Water Act (2006)

3) Ontario Provincial Standards and Ontario Provincial Drawings

4) Municipal Standards and Guidelines (e.g., storm and sanitary by-laws)

5) ASTM International (ASTM) Standards (Table 21) lists some pertinent ASTM standards; however, other ASTM Standards may also be relevant)

b) Where published standards are not available, local industry practices may be considered acceptable. In addition, the policy and guidelines of authorities having jurisdiction should be considered in planning the investigation. Typically, this shall include policies and guidelines of local and regional conservation authorities or provincial agencies such as the Ontario Ministry of Natural Resources and Forestry, Fisheries and Oceans Canada, and the Canadian Water and Wastewater Association.

6.6.3 Process

6.6.3.1 General

a) A competent Professional Engineer or Professional Geoscientist shall supervise a site reconnaissance, planning and coordination of the investigation scope, the planning and execution of laboratory testing and analysis, and the preparation of report(s). Where there are significant natural features such as Environmentally Significant Areas, Areas of Natural and Scientific Interest, or other areas of natural significance, the local conservation authority or the Ontario Ministry of Natural Resources and Forestry office shall be consulted to determine the sensitivity of these features and requirements for mitigation strategies, related to potential groundwater fluctuations.

6.6.3.2 Background Information Collection and Review

a) While preparing for the field investigation, published and publicly available information shall be reviewed under the supervision of an experienced Professional Engineer or Professional Geoscientist to assess regional and local geological and hydrogeological conditions. In particular, information regarding regional and local geology and hydrogeology, water levels, local surface water, groundwater resources, well head protection areas (WHPAs), source water protection areas, and other significant water-related features shall be reviewed. This review shall include source water protection information that is available from local municipalities and conservation authorities. This review shall be supplemented with a site reconnaissance to verify site conditions and corroborate information collected in the background information collection and review.

6.6.3.3 Investigation Work Plan

a) General

1) Based on the background review, a detailed investigation work plan shall be developed to characterize the existing hydrogeological conditions of the project area. The detailed investigation work plan shall include, at a minimum: proposed investigation borehole and monitoring well locations and depth, sampling and testing requirements (soil and groundwater, field and laboratory), hydraulic testing (such as a single-well response test and pumping test), infiltration tests and measurements, water level measurement, surface water investigations, and the desired outcome. The investigation work plan shall include the recommended field procedures such as standard operation procedures and methodologies.

Depending on the nature of the project, there may be a requirement for soil sampling/testing at specific intervals (e.g., every 0.75 m) such as continuous / semi continuous sampling to certain depth for critical zones. Additionally, the suitable sampling interval, shall be defined for specific projects such as MTO projects.

2) If the hydrogeological report is anticipated to be used in support of municipal development application approval process, consultants shall consider the respective Upper/Lower tier Municipality/Conservation Authority's requirements when preparing the investigation work plan.

3) Whenever possible, efforts shall be made to coordinate the field components of Phase 2 ESA, geotechnical investigations, geophysical surveys, hydrogeological investigations, and utility mapping to allow for optimized efficiency and overall project cost savings, provided the scope of work for each individual investigation is satisfied.

b) Hydrogeological Assessment Considerations

1) Monitoring wells shall be installed in accordance with O. Reg. 903 (as amended) and if no longer required or used, decommissioned in accordance with O. Reg. 903, as amended. Sampling for environmental investigations shall follow O. Reg. 153/04, as amended.

c) Construction Requirements for Monitoring Wells

1) The locations for boreholes and groundwater observation wells shall be distributed in a way that ensures the hydrostratigraphic unit(s) and groundwater conditions are well-defined for the project area. Any specific site features (wells, surface water bodies, etc.) shall be investigated as follows:

A. Drill boreholes to determine the site-specific geology (stratigraphy and depth to bedrock). The number and depth of boreholes shall depend on the size of the proposed project area, the available background data, and the expected geological complexity of the area. During the planning stages of a geotechnical investigation or Phase 2 Environmental Site Assessment, the Consultant shall plan the location of boreholes and monitoring wells so that the locations shall also provide data that is useful for the hydrogeological investigation.

B. Collect soil samples from sufficient boreholes and test for grain size to characterize the soil types and to assist in determining soil hydraulic conductivity, and test for soil quality, as needed.

2) The network of groundwater observation wells shall be established in a manner that allows the groundwater elevations

for each hydro-stratigraphic unit and vertical and horizontal groundwater hydraulic gradients to be determined. Consideration shall be given to the type and overall objectives of the proposed project. For example, for sewer construction projects, the proposed wells along the sewer alignment are used to determine the elevation, hydraulic properties, quality, and dewatering requirements.

3) The number of monitoring wells, temporary drive points, permanent piezometers, nested wells, open boreholes or vibrating wire piezometers to be installed will depend on the size of the project area, the complexity of drainage, nearby environmental features, the locations of groundwater divides, and the available background data. Where available, existing observation wells shall be used. A minimum of three monitoring wells are needed in each hydro-stratigraphic unit, and the installation of four wells is preferred to provide added flexibility in data collection and safeguard against possible damage to wells. The number of wells may be adjusted based on the size and complexity of the hydrogeological conditions within the project area. The well completion details, such as well diameter, screen material, size, screen length, filter pack, seal, and finish, depend on the specific site conditions and are at the discretion of the consultant, but shall be consistent with the applicable regulatory requirements. For example, monitoring wells that shall be also used for environmental sampling shall have well screens that are no more than 3.05 m long, as per the requirements of O. Reg. 153/04. The monitoring well screen length shall be consistent with the desired monitored interval and geological conditions encountered (that is, stratigraphy and water table elevation). Screens shall not straddle multiple hydro-stratigraphic units and shall be properly sized and placed to avoid creating preferential pathways for contaminants to migrate between hydro-stratigraphic units.

4) Monitoring wells shall be constructed with permanent monument or flush-mounted casings and equipped with locks. All monitoring locations shall be surveyed for coordinates and geodetic elevations. The newly installed wells shall be developed to remove the fluids introduced during drilling, and to remove particulates that have become entrained in the well and filter pack prior to the start of monitoring. Well development can be conducted mechanically, using a submersible pump powered by a portable generator or manually using a bailer or an inertial foot valve placed at the end of tubing. The volume and/or physical description (e.g., appearance and odour) of water removed during well development shall be measured and recorded.

5) Water Level Measurement: Before water level measurement and sampling, well conditions shall be assessed to identify

wells in need of repair or replacement, and to evaluate whether they are properly sealed at the ground surface to minimize the potential for surface water to directly contact groundwater, that could impact sample integrity. Groundwater elevation of all observation wells within a project area shall be collected during a 24-hr time frame to collect a “snap-shot” of near simultaneous conditions that allows for the evaluation of hydraulic gradient and groundwater velocity. The monitoring period shall be sufficient to demonstrate fully recovered stable water level conditions after well installation or sampling events. Additionally, the monitoring period shall be sufficient to address potential seasonal fluctuation in groundwater elevation. For example, groundwater elevation monitoring shall be collected manually or by transducers and take place at least quarterly for a period of one year to identify the seasonal fluctuation.At a contaminated site, an oil/water interface probe shall be used to identify the potential for the presence of free-phase liquid product such as light non-aqueous phase liquid or dense non-aqueous phase liquid. Equipment shall be cleaned and decontaminated between monitoring locations.

6) Hydraulic Testing: Single-well response tests, pumping tests, or other appropriate field tests shall be conducted to assess the hydraulic properties (such as hydraulic conductivity, transmissivity, and storativity) of the defined hydro-stratigraphic unit(s). The most suitable test for hydraulic properties shall depend on the site objectives and data requirements. For instance, a single well response test shall be used to define the general hydraulic properties of hydro-stratigraphic unit(s) and can be used for groundwater velocity calculations and preliminary evaluation of groundwater control. A pumping test is needed to support an application for a Category 3 PTTW or EASR.

7) Groundwater Sample Collection: Monitoring wells shall be purged before sampling, to remove stagnant water in the wells, and to obtain groundwater samples representative of formation water. The volume to be purged from the monitoring wells shall consider the sampling methodology, the diameter of the well, the sand pack, and the water levels. Water quality parameters such as pH, temperature, dissolved oxygen, electrical conductivity, and oxidation-reduction potential shall be recorded during the purging process. Aesthetic observations shall be carried out on groundwater samples to determine the presence or absence of environmental impairment (such as sheen or odours). Sampling shall take place shortly after purging has been completed and the well has been given sufficient time to recover. Sampling using low-flow techniques shall be considered where the groundwater recovery is sufficient to allow the water elevation to be maintained. A sufficient number of groundwater samples shall

be collected from an adequate number of monitoring wells to undergo representative analysis for groundwater quality. Groundwater samples shall be analyzed for environmental purposes (if a Phase 2 ESA is being completely concurrently) or analyzed against the applicable by-laws and guidelines from local municipalities and conservation authorities. Sampling programs shall include requirements for both field-filtered and lab-filtered metals analysis where appropriate.

8) Groundwater Surface Water Interaction: The consultant shall evaluate the potential for impacts from proposed underground services to shallow groundwater conditions adjacent to a surface water body. If a surface water body (such as a wetland, stream or river) is located on or near the project location, surface water shall be monitored (including water level and flow rate measurements). These measurements shall be used to establish baseline conditions of groundwater and surface water interaction, and to evaluate the potential impacts from the proposed dewatering activities during construction or long-term groundwater control. Percolation tests shall be needed to support the design of infiltration facilities. The number of tests shall be dependent on the size of the facility and the site-specific soil conditions.

9) Investigation Derived Waste: Waste sampling and waste disposal shall be considered during investigation planning. Investigation-derived waste, including soil cuttings and groundwater (e.g., development water or purge water), shall be containerized in appropriate containers, such as drums, during the investigative field activities. Appropriate characterization samples that consider the Contaminants of Concern for the project area, as identified by the Phase 1 ESA (if applicable), shall be collected and analyzed. This shall include the collection of composite soil sample(s) for toxicity characteristic leaching procedure testing. The containerized waste shall be removed by a licensed waste hauler and disposed of at an appropriate Ministry of Environment, Conservation and Parks (MECP) licensed facility within a reasonable timeframe after completion of the field work and preferably before the start of the winter season.

10) Monitoring Well Decommissioning: Once hydrogeological assessment activities have been completed, the consultant shall follow the regulatory requirements (O. Reg. 903) for decommissioning any on-site wells (water supply and/or monitoring wells) that are no longer being used. Consultants shall include the decommissioning of wells in their program budget, however the approval of well owners (i.e., the property owner), is required, before decommissioning work begins.

6.6.4 Field Investigation

6.6.4.1 General

a) A qualified field supervisor shall be designated to oversee the field investigation. Additional accreditation may be required to complete work for specific projects; for example, Ministry of Transportation of Ontario (MTO) Registry, Appraisal and Qualification System (RAQS) qualifications may be necessary to complete work related to these types of sites. The qualifications and experience of the Professional Engineer or, where applicable, Professional Geoscientist supervising the work, and the designated field supervisor, shall be established and confirmed when retaining the consultant to complete the work, based on the type of project and scope required.

b) The field supervisor shall be responsible for ensuring the field investigation conforms to the safety plans, investigation scope, and applicable legislation(s), and shall supervise the borehole advancement, in-situ testing, and monitoring well construction. They shall confirm that the boreholes are properly logged, and results of the in-situ testing and field observations are recorded.

c) Generally, the following key activities shall be required to complete the hydrogeological assessment:

1) Work Plan/Desktop Study/Requirements for Development.

2) Communication, Permits and Approvals.

3) Utility Locates.

4) Field Program Preparation and Undertaking.

5) Laboratory Program Testing and Analysis.

6) Hydrogeological Reporting and Permit Applications.

7) Meetings, Consultations and Clarifications.

6.6.4.2 Work Plan

a) Before work begins, the consultant shall prepare a work plan outlining the minimum requirements for the safe and adequate execution of the work. This shall include provisions to supply any requested clarifications, data, design parameters, and reports. It shall also provide allowances for meetings and consultations with the project planning, design, and compliance team. Work plan development shall begin with a desktop study completed by an experienced Professional Engineer or Geoscientist; this effort involves reviewing published geological information and completed subsurface investigation studies (if available) relevant to the project area. Efforts shall be made to obtain existing information, as it may provide an indication of the expected subsurface conditions that are prevalent in the area and are likely to be encountered during the hydrogeological investigation and it can support the selection of the extent of the hydrogeological investigation and the borehole locations.

b) The work plan shall include the following components, at a minimum:

1) Investigation Details Plan: The type, location, size, and depth of

explorations and testing shall depend on the nature, size, and location of the project, and on the degree of complexity of the subsurface conditions. All relevant available information about site conditions and the project shall be considered while developing the Investigation Details Plan. The consultant shall determine the character of equipment and facilities needed during the execution of work. Where necessary, contingency plans shall be developed where the potential for risk of spills, artesian conditions, hazardous gases, or otherwise unfavourable conditions may affect the costs and schedule of the deliverables. The Investigation Details Plan shall outline the details and specific deliverables of the investigation.

2) The professional overseeing the work shall consider the project needs and overarching goals of the investigation in developing the Investigation Details Plan and based on professional knowledge and experience, make site-specific decisions for the investigation program. The Investigation Details Plan shall include the following information:

A. Number of investigation locations, (boreholes, test pits, probe holes, etc.).

B. Approximate location and depth of investigation locations.

C. Types of drilling, excavation and field-testing equipment used for site works.

D. Sampling and in-situ testing types and methods, with approximate depths and/or frequency.

E. Number and type of piezometer or monitoring well installations.

F. Number of required water level measurements during the investigation.

G. Number of required water level measurements for seasonal trends.

H. Number of wells where transducers are to be installed for long term groundwater monitoring.

I. Laboratory testing.

J. Name of the designated field supervisor and primary contact.

K. Names of the Professional Engineer(s), Geoscientists, or both, supervising the planning and coordination of the investigation scope.

3) The Investigation Details Plan shall adequately identify potential issues that could impact design or construction, or both. The scope of the field work shall be planned to maximize

the amount of relevant information obtained during the investigation process. Planning shall be geared to determine foundation design parameters.

4) Health and Safety Plan: A site-specific Health and Safety Plan will be prepared in accordance with Metrolinx guidelines that discuss Project Planning and Approval requirements.

5) Traffic Protection and Traffic Control Plan: Where the site investigation work takes place near vehicular traffic (e.g., roadways or road rights-of-way (ROWs), bus loops, garage areas, parking lot areas, etc.), a Traffic Protection Plan and Traffic Control Plan (traffic plan) shall be prepared in accordance with Metrolinx guidelines that discuss Project Planning and Approval requirements.

6.6.4.3 Permits and Approval

a) The consultant shall obtain anticipated permits, approvals, and communications before field activities begin in accordance with Metrolinx guidelines that discuss Project Planning and Approval requirements.

6.6.4.4 Communication Management Plan

a) A Communication Management Plan (CMP) shall be prepared in accordance with Metrolinx guidelines that discuss Project Planning and Approval requirements.

6.6.4.5 Utility Locates

a) Before site works begin, all public and private utility locates shall be obtained in accordance with Metrolinx guidelines that discuss Project Planning and Approval requirements.

6.6.4.6 Field Program Preparation and Undertaking

a) The field work shall begin once all necessary permits, approvals, and utility locates have been obtained, and it shall follow the Investigation Details Plan. Based on the knowledge of the known and published subsurface conditions in the area, the field work shall be planned efficiently to obtain the required data. Special site conditions may be encountered, or proposed development requirements may arise that would require changes in the nature and extent of the field work set forth in the work plan. In these circumstances, the CMP shall be used to expedite applicable changes in the scope of field work safely and efficiently.

b) Coordination shall take place between the consultant and the drilling contractor or testing company to arrange for sufficient consumables, parts, and testing equipment to be mobilized to the site to complete the planned work. The consultant shall confirm the equipment is clean and serviceable. Equipment shall comply with the relevant standards, unless otherwise approved.

c) The consultant shall arrange for an adequate number of sample containers (e.g., sample bags, sample jars, etc.) to be available on-site. The type of sample container to be provided shall depend on the type of sample being obtained and the type of testing that is planned.

d) Suitable arrangements shall be made to containerize and transport any cuttings or potentially contaminated water resulting from drilling or sampling activities. Arrangements for storage, transportation and disposal shall comply with relevant and current regulations and requirements including the MECP, conservation authorities, and any other relevant provincial or municipal regulation.

e) The borehole advancement, in-situ testing, and sampling shall be completed as described in the work plan. As a minimum, the following list of procedures and protocols shall be maintained by the consultant during the observations, logging, and recording of the borehole advancement:

1) Drilling methods and procedures shall be agreed upon with the drilling contractor as required by the relevant standards, procedures, and guidelines. Drilling methods and procedures shall be selected to minimize disturbance to the ground.

2) In-situ testing equipment shall be completed and calibrated as required by the relevant standard and manufacturer's handbook.

3) In-situ testing shall be completed by staff or subcontractors experienced in the use of the equipment.

4) Generally, the methods and frequency of sampling shall follow those outlined in the work plan and relevant standards; however, the field supervisor may vary the method or frequency of sampling when and as needed. Equipment used to obtain samples for chemical/biological testing shall be cleaned to the necessary standard.

5) Field records for boreholes shall be prepared during drilling in accordance with applicable standards and procedures.

6) In terms of storing and transporting samples, the following instructions shall be followed:

A. All containers for soil, rock, or water samples shall be clearly labelled. As a minimum, the label shall identify the following information:

1. The Project Name and Number
2. Sampling Date
3. Borehole Number
4. Sample Number
5. Depth Interval
6. Initials of Field Supervisor

7. Name of Consulting Firm

B. Soil, rock, and water samples for testing shall be transported to the testing laboratory in a manner and within the time required so that the testing is representative of the in-situ conditions.

C. For samples sent for chemical/biological analysis, the consultant shall coordinate with the laboratory so that correct sample containers, stabilizing agents, and site storage facilities are provided.

D. For chemical/biological sampling, the consultant shall ensure a chain-of-custody form accompanies each sample sent for testing.

E. Retain all samples for at least three months past the end of the contract or as requested by Metrolinx and/or their representative(s). Make samples available for inspection when requested.

7) Piezometers/monitoring wells shall be installed in boreholes based on the work plan and at appropriate horizons and depths.

8) Where monitoring wells/piezometers are not installed, the boreholes or temporary monitoring wells shall be decommissioned as per O. Reg. 903. The sealing material shall be suitable for the encountered subsurface site conditions. For example, for soft soil conditions with high water table, the backfilling of boreholes is generally accomplished using a cement grout mixture by pumping the grout mix through drill rods or other pipes inserted into the borehole. In boreholes where groundwater or drilling fluid is present, grout shall be tremied from the bottom of the borehole.

9) Precautions shall be taken to avoid cross-aquifer contamination.

10) Water levels shall be monitored over a sufficient period based on the work plan, so that stabilized groundwater measurements are obtained. If fluctuations in groundwater levels may significantly impact the design and construction of the proposed development, consideration shall be given to monitoring groundwater levels over an extended period to reflect potential water level variations.

11) The designated field supervisor shall be present to witness the installation of monitoring wells and piezometers, to confirm proper installation. All piezometer and monitoring well installations shall be carried out in accordance with O. Reg. 903.

12) Roads, parking areas, and sidewalks shall be returned to their original state in accordance with applicable municipal

standards. The ground surface in and around boreholes and test pits shall be rehabilitated as close as possible to the original condition. This includes, but is not limited to, the repair of damage to the ground surface related to the movement of drill rigs and other vehicles on the site. Holes in pavements and slabs shall be patched with concrete or asphaltic concrete, as appropriate.

13) Addition information on piezometer/monitoring well installation is provided in Appendix B.

6.6.4.7 Laboratory Program/Soil-testing and Analysis

a) The laboratory program shall be dependent on the detailed work plan. Groundwater samples shall be collected from monitoring wells to undergo analysis for groundwater quality. Groundwater samples shall be analyzed for environmental purposes (if a Phase 2 ESA is being completed concurrently) or analyzed against the applicable by-laws and guidelines from local municipalities and conservation authorities.

1) Sampling programs shall include requirements for both field-filtered and lab-filtered metals analysis where appropriate.

2) Include the collection of soil and surface water samples and submit for lab analysis as per the detailed work plan.

3) Include additional laboratory analysis based on the work plan, such as grain-size analysis.

6.6.5 Hydrogeology Assessment Report

6.6.5.1 The hydrogeological assessment shall provide an overview of the regional geological setting and relate the local geological data to the regional setting. The results of the hydrogeological assessment shall address the issues outlined in the study requirements. The hydrogeological assessment report shall include as a minimum:

1. Introduction
1.1 Objectives
1.2 Scope of Work
2. Methodology
2.1 Drilling, Soil Sampling, and Observation of Well Installation
2.2 Groundwater Monitoring
2.3 Surface Water Monitoring
2.4 Water Quality Sampling Program
3. Physical Setting
3.1 Physiography and Topography and Climate
3.2 Drainage:
3.2.1 Surface Water Flow
3.2.2 Surface Water Quality
3.3 Bedrock Geology
3.4 Quaternary Geology

3.5 Hydro-stratigraphy
3.6 Aquifers and Local Groundwater Use
3.7 Groundwater Flow and Velocity
3.8 Groundwater/Surface Water Interaction, if applicable
3.9 Groundwater Quality
4. Development Considerations
4.1 Groundwater Control Evaluation
4.2 Potential Impacts and Risks
4.3 Mitigation Measures
4.4 Contingency Plan
4.5 Discharge Plan
5. Conclusions and Recommendations
Figures (Project Specific and Published, Cross-Sections, Groundwater Elevations Contours, Groundwater Hydrographs etc.)
Tables (such as water level elevations and well construction details)
Appendices:

a) Borehole Logs
b) Laboratory Test Results
c) Hydraulic Testing Results
d) Infiltration Testing Results
e) Groundwater Quality Results
f) Water Balance Analysis (if completed, e.g., in support of planning approval)
g) Trial Dewatering Calculations Sheets
h) Hydrogeological Conceptual Site Model (if completed, e.g., in support of MTO projects)
i) Drainage Conceptual Site Model (if completed, e.g., in support of MTO projects)

Table 21 Examples of Pertinent ASTM Standards for Hydrogeological Assessmento

	Standard
ASTM D420	"Standard Guide to Site Characterization for Engineering, Design and Construction Purposes"
ASTM D422	"Standard Test Method for Particle Size Analysis of Soils (Hydrometer Analysis)"
ASTM D1452	"Standard Practice for Soil Investigations and Sampling by Auger Boring"
ASTM D1586	"Test Method for Penetration Test and Split Barrel Sampling of Soils"
ASTM D1587	"Standard Practice of Thin-Walled Tube Geotechnical Sampling of Soils"
ASTM D2434	"Standard Test Method for Permeability of Granular Soils (Constant Head)"
ASTM D2487	"Standard Practice for Classification of Soils for engineering purposes (Unified Soil Classification System)"
ASTM D2488	"Standard Practice for Description and Identification of Soils (Visual Manual Procedure)"
ASTM D3550	"Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils"
ASTM D4044	"Standard Test Method for (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers"
ASTM D4050	"Standard Test Method for (Field Procedure) for Withdrawal and Injection Well Tests for Determining Hydraulic Properties of Aquifer Systems"
ASTM D4104	"Standard Test Method (Analytical Procedure) for Determining Transmissivity of Nonleaky Confined Aquifers by Overdamped Well Response to Instantaneous Change in Head (Slug Tests)"

	Standard
ASTM D4750	"Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)"
ASTM D4823	"Standard Guide for Core Sampling Submerged, Unconsolidated Sediments"
ASTM D5092	"Standard Practice for Design and Installation of Ground Water Monitoring Wells in Aquifers"
ASTM D5784	"Guide for the use of Hollow Stem Augers for Geo-environmental Exploration and Installation of Subsurface Water-Quality Monitoring Devices"
ASTM D5785	"Standard Test Method for (Analytical Procedure) for Determining Transmissivity of Confined Nonleaky Aquifers by Underdamped Well Response to Instantaneous Change in Head (Slug Test)"
ASTM D5786	"Standard Practice for (Field Procedure) for Constant Drawdown Tests in Flowing Wells for Determining Hydraulic Properties of Aquifer Systems"
ASTM D5881	"Standard Test Method for (Analytical Procedure) Determining Transmissivity of Confined Nonleaky Aquifers by Critically Damped Well Response to Instantaneous Change in Head (Slug)"
ASTM D5903	"Standard Guide for Planning and Preparing for a Groundwater Sampling Event"
ASTM D5920	"Standard Test Method (Analytical Procedure) for Tests of Anisotropic Unconfined Aquifers by Neuman Method"
ASTM D5979	"Standard Guide for Conceptualization and Characterization of Groundwater Systems"

6.7 Geotechnical Design

6.7.1 Ground Movement Assessments for Excavations in Soft Ground

6.7.1.1 The empirical calculation methodology for the prediction of ground movement for underground excavations in soft ground has been presented in 4.5. Figure 31 shows a typical settlement profile for underground excavations in soft ground.

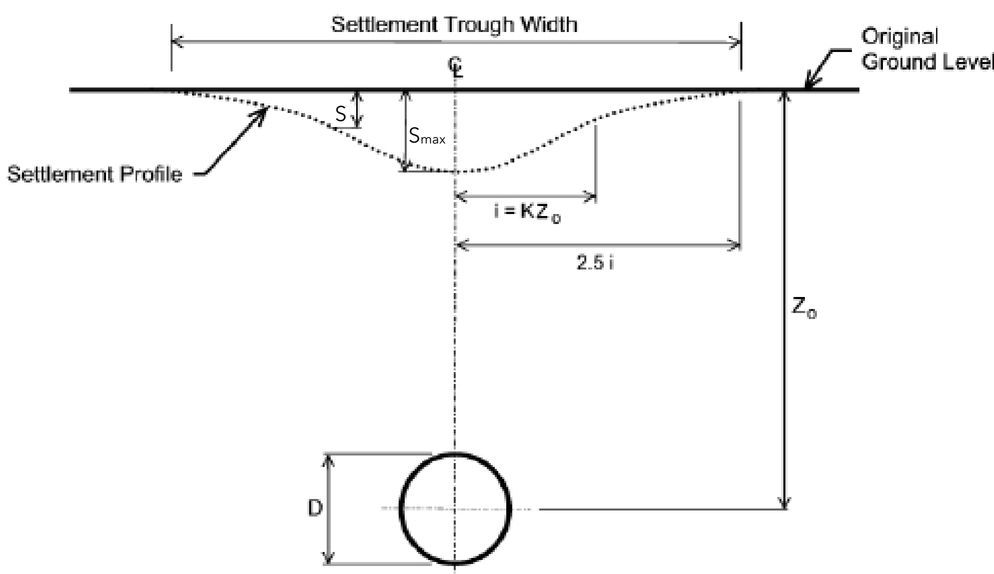


Figure 31 Typical Settlement Profile for a Soft Ground Tunnel (from Singapore LTA)

6.7.1.2 In summary,

$$S_{max} = 0.00314 \times f \times D^2/i$$

where, S_{max} = maximum vertical displacement at the tunnel centreline (mm)

D = excavated diameter of tunnel (mm)

f = face loss percentage of tunnel excavated area (%)

$i = K \times z_o$

K = trough width parameter

z_o = depth, surface to tunnel axis level (mm)

For the settlement curve equation,

$$S = S_{max} \times \exp(-y^2/2i^2)$$

where, S = vertical displacement

y = transverse distance from the tunnel centreline (mm)

6.7.1.3 These empirical equations provide good results for TBM driven tunnels where there is historical data to provide substantiated values of the achieved face loss percentages. For these projects, the empirical method tends to provide better predictions than the application of complex Finite Element or Finite Difference ground/structure interaction modelling where the choice of constitutive model has a significant effect on the results.

6.7.1.4 Similar equations are available for longitudinal settlement. For tunnelling, the longitudinal settlement is generally temporary but shall be important for analysis of settlement effects on utilities that are parallel to the tunnel alignment.

6.7.1.5 For underground excavations using SEM, the use of the empirical method would be very difficult, and likely inaccurate, due to the incremental nature of the excavation and support process with no excavations being circular. Since SEM excavations tend to be large, they are greatly influenced by the geotechnical conditions, certainly to a much greater extent than for a pressurized face TBM. For SEM, the best predictive tool would be Finite Element or Finite Difference ground/structure interaction modelling. These models would be complex 3-D models that address the staggered excavation advancement of the multiple drifts, the development of the strength of the excavation support system over time, the incorporation of additional supports such as Barrel Vaults or steel pipe arches, the incremental removal of supports installed in earlier steps, and the stability of the excavation face.

6.7.1.6 There are limited documented examples of ground movements around SEM excavations. One of the recent SEM excavations in the GTA was Laird Station on the Eglinton Crosstown LRT project. The platform vault was an excavation of approximate dimensions of 19 m width and 15 m height and had approximately 15 m of cover. The centreline surface settlement ranged from 25 to 30 mm. By approximating the excavation to an equivalent circle, the empirical method would give a face loss of about 0.3%. This was a well executed excavation in relatively good ground conditions with a significant cohesive cap over the tunnel.

6.7.1.7 A hydro-geological study shall be carried out to establish the hydraulic connectivity of ground water through permeable soil or rock layers and determine how the ground water connectivity will influence the extent of water table draw down beyond typical zone of influence. These effects shall be considered in the estimation of consolidation settlements by carrying out seepage analysis for all excavations.

6.7.1.8 Surface settlement contour plans shall be prepared for the area around excavations showing 1 mm, 5 mm, 10 mm, 15 mm, 20 mm, (increasing in increments of 10 mm as required) settlement contours. Horizontal movements and strains associated with excavations shall also be predicted.

6.7.2 Ground Movement Assessments for Excavations in Rock

6.7.2.1 Ground movement around excavations in rock is typically not assessed as the movements are very small. However, with larger excavations particularly in weaker rocks, rock movements shall be addressed. These movements are the result of general instability of the rock mass or of the movement of rock along pre-existent joints or faults.

6.7.2.2 For jointed rock masses, there are two classification systems that have been in general use. These are: the CSIR Geomechanics Classification of Jointed Rock Masses - Rock Mass Rating (RMR) by Bieniawski and the Norwegian Geotechnical Institute (NGI) Tunnelling Quality Index, Q by Barton, Lien and Lunde. These have been further developed into the Geological Strength Index (GSI) by Hoek and Brown, and Marinos, Marinos and Hoek. These systems provide tools for jointed rock mass strength estimation. However, care shall be applied when using these systems for shales of recent geological age.

6.7.2.3 The rock mass strength estimation shall be used in Finite Element or Finite Difference ground/structure interaction modelling, that considers the rock mass as an isotropic continuum, to predict ground movements and surface settlements. Interbedded shales and limestones, as in the Georgian Bay Shale, will be anisotropic. The use of simplified assumptions to treat this rock mass as isotropic may be appropriate. However, where faults and discrete joints exist, a more appropriate tool to model rock block movements, shall be a Discrete Element model. For the Georgian Bay Shale these models shall address in-situ stress redistribution, Time Dependent Deformation (TDD) if unmitigated, the tendency of the shale beds to break out in the shoulders of circular tunnels if unsupported, together with the substantial progressive overbreak that will occur in inadequately supported large span openings, and ground water inflow with associated surface degradation of the rock surface during construction. The models shall address the presence of steeply dipping joints that may strike sub-parallel to the excavation opening, if these have been identified at the location, and any reinforcement or initial support that may be incorporated to resist rock movements.

6.7.3 Protection of Existing Adjacent Structures

6.7.3.1 Condition Assessment Reports

a) Condition assessment inspections shall be performed on all Existing Adjacent Structures (EAS) where permissions to access have been provided by the property owners and on Existing Adjacent Utilities (EAU) where these are accessible. These inspections shall be performed as pre-construction inspections, not more than three months before the start of excavation; and as post-construction inspections, not less than three months and not more than six months after the completion of monitoring of construction activities.

b) The pre-construction condition assessments shall classify the condition of the buildings in accordance with Table 22.

Table 22 Guidance to Classify Existing Condition of Buildings From Visual Inspections (Goh and Mair, 2014)

<table>
<tr><th rowspan="2">Classification of Overall Building Condition</th><th rowspan="2">Observations on Structural Elements</th><th colspan="2">Adapted from BRE Digest 251 (1991)</th></tr>
<tr><th>Classification of visible damage to walls (brick, blockwork) with particular reference to ease of repair in italics</th><th>Classification of visible damage on ground floor slab settlement</th></tr>
<tr><td>0</td><td rowspan="2">No sign of any structural damage/ distress and material deterioration on visible structural elements</td><td>Hairline cracks of less than 0.1 mm</td><td>Hairline cracks between floor and baseboards</td></tr>
<tr><td>1</td><td>Fine cracks that can be treated easily using normal decoration. Damage generally restricted to internal wall finishes; cracks rarely visible in external brickwork. Typical crack widths* up to 1 mm.</td><td>Settlement of floor slab, either at a corner or along a short wall, such that a gap opens below the baseboards that can be masked by resetting the baseboard. No cracks in walls. No cracks in floor slab, although there may be negligible cracks in floor screed and finish. Slab reasonably level. Typical gap** up to 1 mm.</td></tr>
<tr><td>2</td><td rowspan="2">No sign of structural distress on visible structural elements, but some structural cracks and material deterioration (e.g., concrete spalling, corrosion of rebar) are observed. Generally, the defects are unlikely to affect structural ability and integrity of any part of the building.</td><td>Cracks easily filled. Recurrent cracks can be masked by suitable linings. Cracks not necessarily visible externally; some external repointing may be required to ensure weather-tightness. Doors and windows may stick slightly and require easing and adjusting. Typical crack widths* up to 5 mm.</td><td>Larger gaps below baseboards. Some obvious but limited local settlement leading to a slight slope of floor slab. Gaps can be masked by resetting baseboards and some local recreeding may be necessary. Fine cracks appear in internal partition walls that need some re-decoration; slight distortion of door frames so some ‘jamming’ may occur necessitating adjustment of doors. No cracks in floor slab although there may be very slight cracks in floor screed and finish. Slab reasonably level. Typical crack widths* up to 5 mm and gap** up to 13 mm.</td></tr>
<tr><td>3</td><td>Cracks that require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weather-tightness often impaired. Typical crack widths* are 5 to 15 mm or several of 3 mm cracks.</td><td>Significant gaps below baseboards with areas of floor, especially at corners or ends, where local settlements may have caused slight cracking of floor slab. Sloping of floors in these areas is clearly visible; (slope approximately 1 in 150). Some disruption to drain, plumbing or heating pipes may occur. Damage to internal walls is more widespread with some crack filling or replastering of partitions necessary. Doors may have to be refitted. Inspection reveals some voids below slab with poor to loosely compacted fill. Typical crack widths* up to 15 mm and gap** up to 19 mm.</td></tr>
</table>

Classification of Overall Building Condition	Observations on Structural Elements	Adapted from BRE Digest 251 (1991): Classification of visible damage to walls (brick, blockwork) with particular reference to ease of repair in italics	Adapted from BRE Digest 251 (1991): Classification of visible damage on ground floor slab settlement
4	Signs of structural distress and structural damage on visible structural elements. These defects are likely to affect the structural stability and integrity of any part of the building	Extensive damage that *requires breaking out and replacing sections of walls*, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably***. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths* are 15 to 25 mm, but also depends on number of cracks.	Large, localized gaps below baseboards; possibly some cracks in floor slab with sharp fall to edge of slab; (slope approximately 1 in 50, or more). Inspection reveals voids exceeding 50 mm below slab and/or poor to loose fill likely to settle further. Local breaking out, part refilling and re-laying of floor slab or grouting of fill may be necessary; damage to internal partitions may require replacement of some bricks or blocks or relining of stud partitions. Typical crack widths* range between 15 to 25 mm but may also depend on number of cracks, and gap** up to 25 mm.
5		Structural damage that *requires a major repair job, involving partial or complete rebuilding*. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths* are greater than 25 mm but depends on number of cracks.	Either very large overall floor settlement with large movement of walls and damage at junctions extending up into the second-floor area with possible damage to exterior walls, or large differential settlements across floor slab. Voids exceeding 75 mm below slab and/or poor to loose fill likely to settle further. Risk of instability. Most or all of floor requires breaking out and re-laying or grouting of fill; internal partitions require replacement. Typical crack widths* are greater than 25 mm but may also depend on number of cracks, and gap** greater than 25 mm.

* Crack width is one factor in assessing category of damage and should not be used on its own as a direct measure of it.
** Gap refers to space between baseboard and finished floor, caused by settlement after making appropriate allowance for discrepancy in building, shrinkage, normal bedding and the like.
*** For floors and walls, local deviation of slope from the horizontal or vertical of more than 1/100 will normally be clearly visible. Overall deviations in excess of 1/150 are undesirable.

Modified from Singapore LTA

6.7.3.2 Construction Impact Assessment Report - 1 (CIAR-1): Ground, structure, and pavement deformation analysis shall be performed and summarized in a Construction Impact Assessment Report - Level 1 (CIAR-1). This report shall include:

a) First level of assessment and screening for all EASs and EAUs based on Good Industry Practice outlined in:

1) New and O'Reilly (1982), paper "Settlements Above Tunnels in the United Kingdom - their Magnitude and Prediction".

2) New and O'Reilly (1991) paper "Tunnelling Induced Ground Movements; Predicting their Magnitude and Effects".

3) Burland, Broms and De Mello (1977) paper "Behaviour of Foundations and Structures".

4) Boscardin and Cording (1989) paper "Building Response to Excavation Induced Settlement".

5) Attewall, Yeates and Selby (1986) paper "Soil Movements Induced by Tunnelling and their Effects on Pipelines and Structures".

6) Lake, Rankin and Hawley (1992) paper "Prediction and Effects of Ground Movements caused by Tunnelling in Soft Ground Beneath Urban Areas".

7) Son and Cording (2005) paper "Estimation of Building Damage Due to Excavation -Induced Ground Movement".

8) Canadian Foundation Engineering Manual for settlement prediction next to vertical excavations.

b) Inventory of the EAS within ZOI including an overall evaluation of EAS potentially at risk.

c) Information related to EAS such as locations, dimensions, elevations, foundations, structural details, materials, and other information necessary to complete the assessment.

d) Utility inventory identifying EAU within the ZOI, categorized by utility type and use, including an overall evaluation of EAU potentially at risk.

e) Methodology used for assessment of the potential impacts.

f) Detailed definition of damage levels.

g) Assessment of the free field ground movement using empirical methods.

h) Determine deformation parameters (e.g., strain, settlement, and angular distortion) limits for each EAS.

i) Assessment of the potential impact to each EAS and EAU using empirical methods.

1) Damage levels (degree of damage) to the buildings shall be categorized as Negligible, Very Slight, Slight, Moderate, Severe, and Very Severe. These categories of damage levels to the buildings shall be in accordance with the damage levels and description of typical damage defined in Son and Cording (2005).

2) Damage levels (degree of damage) to the utilities shall be categorized based on the expected values of the deformation parameters for each utility in comparison to the allowable deformation parameters, as per below. Deformation parameters are movements that could potentially affect structural integrity, functionality, operability, and durability of the utilities, including but not limited to, joint rotation, longitudinal strain, curvature, total settlement, and differential settlement, where applicable depending on the utility type.

A. Negligible: Expected value of all deformation parameters is less than 33 percent of the allowable deformation parameters.

B. Very Slight: Expected value of any deformation parameter is between 33 and 50 percent of the allowable deformation parameters.

C. Slight: Expected value of any deformation parameter is between 50 and 75 percent of the allowable deformation parameters.

D. Moderate: Expected value of any deformation parameter is between 75 and 90 percent of the allowable deformation parameters.

E. Severe to Very Severe: Expected value of any deformation parameter is more than 90 percent of the allowable deformation parameters, or the condition of the utility is assessed to be poor or very poor.

A. Allowable deformation parameters of the utilities shall be obtained from the utility companies. Prior to obtaining allowable deformation parameters from the utility companies the allowable deformation parameters provided in Table 23 shall be used in preliminary calculations.

3) Determine EAS and EAU requiring a further CIAR-2 analysis.

Table 23 Allowable Deformation Parameters for Utilities

Material	Diameter (mm)	Increase in Tension Strain (microstrain)	Increase in Compression Strain (microstrain)	Allowable Joint Rotation (deg)
Brick Sewers	Any	150	Strain equivalent with 25% of allowable stress	N/A

Material	Diameter (mm)	Increase in Tension Strain (microstrain)	Increase in Compression Strain (microstrain)	Allowable Joint Rotation (deg)
Cast Iron (Lead-yarn joints)	Any	100	1200	0.1
Ductile Iron (Lead-yarn gasket joints)	Any	500	700	0.5
Ductile Iron (Rubber gasket joints)	Any	500	700	2.0
Steel	Any	450	450	1.5
Enwave deep lake water cooling pipe inside the tunnel		450	450	1.36
Vitrified Clay	<125	80	400	0.5
	125 to 350			2.0
	351 to 750			1.0
	>750			0.3
Concrete (unreinforced)	< 225	20	400	0.5
	225 to 350	40		2.0
	351 to 750	40		1.0
	>750	60		0.3
High Density Polyethylene (HDPE)	Any	9000	-	N/A
Note: The limit for Joint Pull Apart is 15 mm.				

From Project Agreement for Ontario Line.

6.7.3.3 Construction Impact Assessment Report - 2 (CIAR-2)

a) A Construction Impact Assessment Report - Level 2 (CIAR-2) shall be prepared to address EAS or EAU that:

1) The preceding CIAR-1 indicates EAS or EAU shall sustain unacceptable damage.

2) Include deep, multi-storey basements that could alter the slope of the ground settlement profile.

3) Have more than five storeys above grade.

4) Are of historical or cultural significance, heritage buildings or essential services buildings that exceed any CIAR-1 predicted impact levels.

5) Have superstructure or cladding more sensitive to movement.

6) Are utilities which present indications of vulnerability to stress and strain.

b) CIAR-2 analysis shall include the following as minimum:

1) Information related to EAS or EAU included in the CIAR-2 such as locations, dimensions, elevations, foundations, structural details, materials, and other information necessary to complete the assessment.

2) Methodology used for assessment of the potential impact.

3) Detailed definition of damage levels.

4) Determine deformation parameters (e.g., strain, settlement, and angular distortion) limits for each EAS and EAU.

5) Assessment of the potential impact using empirical methods. Son and Cording (2005) shall be used for buildings.

6) Determine EAS and EAU requiring a further CIAR-3 analysis.

c) Numerical simulation methods such as finite element method or finite difference methods using non-linear soil constitutive models that account for the stress paths (loading and unloading) shall be used for CIAR-2. The numerical simulation shall consider the planned stages of the excavation/mining and initial/temporary support. As a result of this selection process, the CIAR-1 and CIAR-2 analyses shall provide an estimate of the potential impacts due to construction, including but not limited to tunnelling and shaft excavations.

6.7.3.4 Construction Impact Assessment Report - 3 (CIAR-3)

a) A CIAR-3 analysis shall be conducted for EAS or EAU that meet any of the following criteria, unless a mitigation scheme is to be implemented to compensate for the movement of the EAS or EAU caused by the construction activities:

1) EAS or EAU with potential damage assessed by CIAR-2 to be Moderate or worse.

2) Mid-rise and high-rise buildings with five storeys or higher where underground mining occurs below the building.

3) Buildings with three-level basements or deeper where underground mining occurs below the building.

b) A CIAR-3 analysis shall be conducted for Enwave tunnels, Enwave deep lake water cooling pipes, other sensitive major utility tunnels and pipes and TTC tunnels where excavation occurs below or beside them. The CIAR-3 for the TTC and Metrolinx underground structures shall provide the predicted differential settlement at the joints between cut and cover units and lining displacements for tunnels.

c) The CIAR-3 analysis shall include the following as minimum:

1) Information related to EAS and EAU included in the CIAR-2 such as locations, dimensions, elevations, foundations, structural details, materials, and other information necessary to complete the assessment.

2) Methodology used for assessment of the potential impact.

3) Detailed definition of damage levels.

4) Assessment of the EAS and EAU movement using proper numerical simulation methods such as finite element method or finite difference method. The numerical simulation shall consider ground-structure interaction effects between the

ground (soil/rock) and the EAS or EAU. The numerical simulation shall be three-dimensional.

5) The numerical simulation shall take into account the planned stages of the excavation/mining, initial/temporary support, and details of the EAS or EAU.

6) Assessment of the potential impact on the EAS or EAU based on the results of the numerical simulation.

6.7.3.5 Impact Mitigation Design

a) An impact mitigation design shall be provided as part of CIAR-2 and CIAR-3, that:

1) Demonstrates structure-specific mitigation measures needed to prevent a loss in appearance, structural integrity, functionality, operability and durability of potentially impacted EAS and EAU and ensure safety and continued operation of the EAS and EAU.

2) Demonstrates the effectiveness of the proposed mitigation measures by engineering analysis.

3) Confirms and documents that the proposed mitigation is acceptable to all potentially affected EAS or EAU owners.

4) Determines instrumentation monitoring requirements.

5) Defines response levels (Review Levels and Alert Levels) for each EAS and EAU.

6.7.4 Instrumentation and Monitoring

6.7.4.1 General

a) Instrumentation shall be designed, supplied, installed, maintained and monitored to record and report groundwater elevation and pressure, ground movements due to excavations and tunnelling, movement of temporary structures (including piles, struts and tie-backs), permanent structures, utilities, highways, existing buildings, bridges, electrical towers and other entities impacted by the construction.

b) The contractor shall use the information obtained from the instrumentation to adjust construction activities as necessary to verify that the performance requirements and tolerances of the monitored facilities are met.

c) The Geotechnical Instrumentation and Monitoring Plan (GIMP) shall be produced and shall incorporate the findings of the CIAR-1, CIAR-2 and CIAR-3 in addition to the requirements outlined in this section.

d) The instrumentation drawings shall incorporate and implement the minimum requirements of the Geotechnical Instrumentation and Monitoring Drawings as per Appendix E.

6.7.4.2 Geotechnical Instrumentation and Monitoring Plan

a) The GIMP shall include:

1) The definition of the ZOI.

2) Identification of each of the monitored facilities within the ZOI.

3) An estimate of the movement and distortion of, and effects on the monitored facilities within the ZOI resulting from implementation of the project.

4) Identification of each of the monitored facilities where movements or distortions caused by implementation of the project are initially estimated to be in excess of the Limits of Movement values, and proposed mitigation measures.

5) Identification of the need, extent and locations of all instrumentation and monitoring equipment to be located within the ZOI, or elsewhere.

6) Description of instrumentation to be used, including measurement methodology and type of instrument. Instrumentation shall be capable of collecting and transmitting near real-time monitoring data obtained on a continuous basis to an Automated Data Acquisition and Management System (ADAMS).

7) A plan and visual break down of the flow of communication for each type and location of triggered alarm. The plan shall include the roles and responsibilities as well as an alternate at each level of responsibility in order to maintain the communication chain.

8) Overall schedule for all activities associated with the GIMP, including monitoring instrument installation, the baseline monitoring, routine monitoring, and the plan, procedure and schedule for handover, decommissioning and disposing of instruments.

b) A geotechnical instrumentation specialist team shall be assigned to the project, to plan, organize, implement, control, and monitor instrumentation. This team shall:

1) Be led by a Professional Engineer in the province of Ontario, or equivalent qualification accepted by Metrolinx (prior to commencement of the project), with a demonstrated knowledge (and a minimum of 10 years of direct field experience) with installation, calibration, monitoring, interpretation, and reporting on similar construction monitoring instrumentation processes on tunnelling projects of a similar size and scope.

2) Demonstrate previous relevant project experience with a list of similar projects and background information.

3) Provide a geotechnical instrumentation specialist at the site

during installation of all instrumentation to supervise and direct the installation technicians.

4) Perform all surveying of any instrumentation under the supervision of an Ontario Land Surveyor (OLS) licensed in the province of Ontario, or equivalent qualification accepted by Metrolinx (prior to commencement of the project) having a minimum of five years of field experience with automated survey systems and manual high accuracy surveying.

6.7.4.3 Limits of Movement

a) For each of the monitored facilities within the ZOI, the limits of movement shall be established by:

1) Determining the applicable limits of movement based on the condition of the facility, and complying with applicable codes, or guidelines from the facility owners.

2) Determining Review Levels and Alert Levels for all instruments to ensure the limits of movement will not be exceeded; at Review Levels construction methods shall be reviewed; at Alert Levels construction shall be stopped until mitigation measures are employed.

6.7.4.4 Automated Data Acquisition and Management System

a) A GIS and web-based Automated Data Acquisition and Management System (ADAMS) shall be provided and maintained that:

1) Receives, organizes, and stores near real-time monitoring data obtained on a continuous basis from all monitoring instruments installed and monitored as part of the project, including TBM data where:

A. Near real time means no longer than 15 minutes following collection of monitoring data.

B. Continuous means at intervals no greater than 15 minutes.

C. The above activity will span from the start of the baseline reading until completion of monitoring.

D. If manual readings are taken, the data shall be entered into ADAMS within four hours of the reading of initial measurement (e.g., level reading). A manual readings system shall only be permitted where an automated system is not feasible. A GIMP that includes manual measurements shall be verified by a Metrolinx Appointed Person.

2) Produces visualizations and other forms of reporting that are based on collected data.

3) Transmits, within five minutes of upload, notifications of exceedances of any limits of movement via phone and email to identified recipients.

4) Allows generation of user specified reports of instrumentation data.

5) Is equipped with a secure internet connection.

6) Metrolinx shall be provided with access credentials for a minimum of 50 simultaneous users of the ADAMS.

6.7.4.5 Minimum Instrumentation Required

a) Ground Movement around Shafts and Deep Excavations

1) As a minimum, instruments shall be installed and monitored as shown on the drawings in Appendix E.

2) Shafts and temporary retaining structures greater than 5 m in height shall be monitored for horizontal displacement using inclinometers at a minimum of one inclinometer per 20 m of wall length. Inclinometers shall extend to a minimum depth that is lesser of 5 m below the bottom of the embedded section of the retaining structure or 1.0 m into the competent bedrock. In addition, the monitoring shall meet the requirements of OPSS 539.

3) Vertical ground displacements shall be measured at regular intervals of distance around the perimeter of all temporary retaining structures at a maximum spacing of 10 m and at a maximum distance of 1.5 m from the edge of the temporary retaining structure. Where rigid pavements exist in these areas, the monitoring instruments shall extend below the rigid pavement.

b) Ground Movement along Tunnel Alignment

1) As a minimum, instruments shall be installed and monitored as shown on the drawings in Appendix E.

c) Ground Movement at Cross Passages

1) As a minimum, instruments shall be installed and monitored as shown on the drawings in Appendix E.

d) Movements of Monitored Facilities- The following shall be monitored:

1) Vertical displacements of all EAS and EAU by automated survey methods or other means reviewed by Metrolinx.

2) Horizontal displacements in addition to vertical displacements, where such movements are deemed likely and detrimental as identified by the CIAR process.

3) Tilt of EAS and EAU in addition to any other displacements, where such movements are deemed likely and detrimental as identified by the CIAR process.

4) Cracks (over 2 mm in width identified in precondition surveys) and other defects using crack gauges or other applicable

instruments.

5) Instruments and target locations as shown on the drawings in Appendix E.

6) If required by the utility owner, the vertical displacement of EAU located within the ZOI shall be monitored.

e) Groundwater Levels Around Deep Excavations

1) Piezometers shall be installed and monitored as shown on drawings in Appendix E.

f) Additional Monitoring Requirements of SEM Tunnel- Deformation Monitoring of the Initial Lining:

1) Deformations of the primary lining shall be monitored by observing movement of measuring bolts or prisms installed immediately after the excavation. Measuring bolts and prisms shall be fixed during the tunnel advance, after the application of the shotcrete within 1 m of the tunnel face. The following measurements shall be taken:

A. Optical survey of prisms using a high precision total station.

B. Distance between measuring bolts using tape extensometer.

2) Deformation measurements shall be taken in arrays at intervals along the tunnel as well as at junctions, bifurcations and in adjacent openings. The position of the measuring bolts and prisms shall be adjusted to suit the excavation sequence.

A. Stress Monitoring of the Initial Lining.

3) Strain gauges shall be installed in circumferential direction to assess the load build-up and distribution on the initial lining. The strain gauges shall be located at mid depth of the shotcrete as far as practicable.

g) Monitoring Accuracy: Instruments shall be installed with the following monitoring accuracy:

1) Elevations of benchmarks shall be established to 1 mm.

2) Coordinates of benchmarks shall be established to 5 mm.

3) Elevations and horizontal positions, as applicable, of instruments shall be established and monitored to within 1 mm.

4) Initial coordinates of instruments shall be established to 10 mm.

6.7.4.6 Baseline Readings

a) Baseline readings from all monitoring instruments shall be obtained, on a regular basis, minimum twice a week, for a minimum of one month prior to commencing a construction activity that may impact an instrument reading against which the limits of movement will be measured.

b) During the baseline monitoring, a maximum reading repeatability of ±2 mm for all monitoring instruments shall be achieved.

c) For each individual monitoring point a statistical analysis shall be conducted of all monitoring instrument readings collected during the baseline monitoring, including an evaluation of daily, weekly, and seasonal thermal influences on such readings; and include only monitoring instrument readings that lie within two standard deviations above or below the mean value obtained for any given monitoring instrument.

d) All processes facilitating development of the baseline shall be documented in a Baseline Monitoring Report, including details related to the methodology used to analyze data obtained during the baseline monitoring, the results of such analysis, and presentation of the baseline.

6.7.5 Routine Monitoring

6.7.5.1 All automated monitoring instruments shall be monitored and data delivered on a continuous basis during implementation of the project and the data shall be presented within the ADAMS;

6.7.5.2 All manual monitoring instruments shall be monitored no less frequently than:

a) One reading per no more than eight hours during construction of shafts.

b) One reading per no more than eight hours where the bored tunnel excavation face is within 25 m of the applicable instrument.

c) One reading per day where the bored tunnel excavation face is within 50 m of the applicable instrument.

d) One reading per week where the bored tunnel excavation face is within 100 m of the applicable instrument.

e) One reading per month where the bored tunnel excavation face is greater than 100 m from the applicable instrument, until the contract completion.

6.7.5.3 Data Presentation

a) The ADAMS shall be utilized to present data collected from all instrumentation installed and monitored as part of the Works, in time-series plots.

b) Weekly Instrumentation Monitoring Reports shall be produced, that include summary of collected data, highlighting any exceedances, together with the documentation of the actions carried out to address and mitigate the exceedances, and time-series plots for all active instruments. The time series plots shall contain a cumulative history of readings, including identifications of the associated instrumentation, time of monitoring, position stations of the construction, and tunnelling related activity at the time of each reading.

6.7.5.4 Final Readings

a) The final reading for each instrument shall occur when both of the following conditions are met:

1) No additional construction activity that could influence the instrument readings shall occur.

2) Readings for the instrument are stable (i.e., do not indicate ongoing movement or other changes) for a period of three months.

6.7.5.5 Ground Surface Settlement Monitoring Instruments

a) Total Station Instrument Based Systems

1) Total station survey instrument-based systems fall into two categories; automated (robotic) or manual.

2) Automated systems typically consist of a motorized total station, mounted on tower or similar stable location, that is programmed to survey, at time intervals, a series of fixed targets in the line of sight of the instrument. In some cases, the requirement for a fixed target is avoided with the instrument surveying a known location on a surface instead of a reflective target. The data is transmitted and delivered to the ADAMS automatically.

3) Manual systems achieve the same outcome but require an operator to use, read the instrument and to transmit the data, and an assistant to position the survey target.

4) The automated systems are preferred since they provided much faster data delivery at lower cost but are not applicable in all circumstances.

5) A notable issue is where a surface to be surveyed is exposed to frost heave movements. The impact of frost heave can be avoided by the use of surface settlement points that consist of a vertical pipe containing a rebar rod that is anchored below the frost level. The pipe has a cap at the surface that shall be removed to allow the reading to be taken, thus requiring a manually operated total station.

b) InSAR Surveys

1) Interferometric synthetic aperture radar (InSAR) surveys shall be provided over the whole project area within 500 m horizontal distance each side of the reference alignment.

2) Pre-construction and post-construction condition survey reports shall include satellite-based InSAR baseline survey colour images.

3) InSAR surveys shall facilitate 1 m × 2 m ground pixel resolution

or better, and measurement of vertical movement to ± 3 mm accuracy.

4) The InSAR data shall be analysed by a satellite interferometry specialist firm and processed with multi-image processing algorithms.

5) The InSAR data shall be presented in survey reports with colour images, contour plots of any vertical settlement.

6) InSAR data shall be acquired monthly at least three months before construction, once per week during underground structures construction, and monthly at least three months after completion.

7) The InSAR surveys shall be calibrated with ground-based total station surveys at least once every three months.

8) Metrolinx may elect to use InSAR surveys during the winter months ahead of the project to assess whether the surface monitoring point (SMP) locations will be exposed to frost heave and will require the use of recessed SMPs and manual surveying.

6.7.5.6 Vertical Movements at Depth Monitoring

a) Vertical movements at depth shall be measured by multi-point extensometers (MPBX). These shall have the ability to transmit movement data to the ADAMS automatically.

b) In some cases, where access to the surface is restricted, vertical movements shall be measured by shape arrays (SAA) installed in horizontal pipes.

6.7.5.7 Lateral Movement Monitoring

a) Lateral movements adjacent to vertical excavations shall be measured using vertical inclinometers. These can be installed adjacent or within the SOE system. These inclinometers can be conventional where movements are read manually or in-place inclinometers that can be connected to a data logger and shall transmit data to the ADAMS automatically.

b) Monitoring of Struts and Ground Anchors

1) At least 15% of all struts and ground anchors shall be monitored for load using strain gauges and/or load cells. At least 25% of the struts and anchors monitored using strain gauges shall also be monitored using load cells. Where strain gauges are used, a minimum of two strain gauges coupled with temperature monitoring shall be installed at each monitoring location for struts.

6.7.5.8 Groundwater Level Monitoring

1) Groundwater level monitoring shall be measured by vibrating

wire piezometers equipped with dataloggers and transmission devices. Most of these shall be installed during the preconstruction geotechnical investigation program. Additional devices shall be installed prior to construction as necessary to confirm the earlier findings.

6.7.5.9 Building Movement Monitoring

1) The simplest method of building movement monitoring is using survey targets that are attached to the building and monitored by an automated total station survey instrument. Where more intensive monitoring is required, shape arrays, tilt meters and electro-levels shall be used. These shall typically be installed in the basement of the subject building.

2) Building Movement Monitoring shall include, but not be limited to, measurements of horizontal and vertical movement as well as rotational movement (i.e., tilt). Monitoring instruments shall be affixed to the structure in a manner that best conveys the movement to be measured (e.g., vertical movement measured on foundation elements, tilt meters affixed to exterior wall columns).

6.7.5.10 Utility Movement Monitoring

1) Utility movement shall be measured utilizing instruments affixed to the utility structure itself, generally exposed using hydro-vac excavation. Where not feasible or permitted by the utility owner (e.g., Natural Gas or Electric) instrumentation may be install adjacent to the utility, as near as is permitted by the utility owner, on the side closest to the impacting construction activity.

6.8 Permanent Cut Slopes

6.8.1 General

6.8.1.1 Permanent cut slopes shall be designed to:

a) Provide an adequate factor of safety against shallow and deep seated slope failure.

b) Resist surficial erosion and slumping types of failure.

c) Control discharge of surface water and subsurface seepage.

d) Allow for regular maintenance of the slope surface.

e) Consider potential impact on adjacent properties and structures.

6.8.2 Deep-Seated Failure

6.8.2.1 The minimum design factor of safety against slope failure for a given location is related to the consequence of failure, degree of certainty or confidence in the ground and groundwater conditions (i.e., extent of geotechnical investigation), and methods used to determine the soil parameters and the type of ground forming the slope. Table 24 summarizes the minimum factors of safety as calculated by limit equilibrium methods or a strength reduction method for design of permanent cut slopes. A higher factor of safety may be required to accommodate particular or special site conditions. In these instances, the required minimum design factor of safety shall be provided in the Geotechnical Interpretation and Design Report.

6.8.2.2 The factor of safety shall be calculated using commercially available limit equilibrium or strength reduction methods (software) that are well accepted in industry and have a proven track record.

Table 24 Minimum Design Factors of Safety for Permanent Cut Slopes

Soil Type	Method Used to Assess Shear Strength Parameters	Minimum Factor of Safety
Cohesionless Soils: Gravels, Sands, Silty Sands	• Correlation between Standard Penetration Test 'N" Values, and angle of internal friction. • Laboratory test results (triaxial and direct shear tests). • Back analysis or experience with existing slopes in geotechnical conditions.	1.3*
Cohesive Soils: Clayey Silts to Clays	• Effective stress analysis using well established correlations between in-situ test, Plasticity Indices, and angle of internal friction and effective cohesion. • Back analysis or experience with existing slopes in geotechnical conditions.	1.5
	Effective stress analysis using: • Consolidated drained triaxial compression tests or consolidated undrained triaxial compression tests with porewater pressure measurement.	1.3*
* Factor of Safety 1.3 shall be increased to 1.5 where uncertainty of conditions exists and where consequences of failure are significant.		

From TTC Design Manual.

6.8.2.3 In assessing the minimum factor of safety for a permanent cut slope several different sections of the slope will require analysis. The sections of a slope to be considered for analysis are:

a) The section of slope formed of the weakest material.

b) The highest section of a slope.

c) The section of a slope with the steepest gradient.

d) The section with the highest groundwater conditions.

6.8.3 Drainage and Surface Treatment

6.8.3.1 General

a) Surface treatment and drainage works on permanent cut slopes are necessary to mitigate against surficial slope failures that can lead to short service interruptions and may ultimately lead to deep-seated slope failures.

b) A continuous slope with constant surface gradient meeting the design criteria shall be designed for slopes up to 8 m high. For slopes greater than 8 m in height, the slope surface shall incorporate horizontal benches at least 2 m wide. Each bench shall incorporate a surface drainage ditch or swale. The height between benches shall be based on stability and surface drainage considerations, as appropriate.

6.8.3.2 Surface Drainage Works

a) Slopes shall be designed so that surface water collected behind the slope crest is directed away from the slope surface. Water falling on the slope surface shall be collected and discharged, away from the slope, in a controlled manner.

b) The discharge capacity of slope ditches, swales and discharge pipes shall be designed using the Rational Method for sites of area less than 4 hectares or the Ottawa Hydraulic model (OTTHYMO) or equivalent for larger sites. In addition, the lining (vegetation, stone, concrete) of ditches and swales shall be designed to resist the erosional forces anticipated under design storm conditions.

6.8.3.3 Subsurface Drainage Works

a) Where water bearing deposits, layers, seams or lenses are present on the slope surface, subsurface drainage works shall be incorporated into the slope design to adequately control seepage and piping, and to control the build-up of excessive porewater pressures within the slope.

b) Options for the provision of suitable drainage works include surface and subsurface drainage blankets, systems of near surface French drains, and interceptor drains behind the slope crest. Selection of subsurface drains shall be based on the slope geometry, and the location, size and extent of water bearing soils.

c) All subsurface drainage works shall be provided with appropriate filters to adequately control migration of soil into and out of the drains. Drainage works shall generally follow the latest edition of OPSS.

6.8.3.4 Surface Treatment

a) The surface treatment of cut slopes shall be designed to adequately resist wind erosion and sheet run-off erosion. Consideration shall be given to the use of vegetation such as sod, mixtures of wild grasses and mixtures of wildflowers and legumes as surface treatment on cut slopes. Suitable vegetation cover normally provides sufficient surficial erosion protection if surface run-off behind the crest is well controlled, and slope benching is provided, and if adequate sunlight and moisture conditions are available to sustain plant growth.

b) For a permanently covered cut slope, such as under a bridge, inorganic slope surface treatment is to be provided. Concrete and suitably sized and placed rip-rap are feasible design options in these situations. Filters shall be provided beneath rip-rap to adequately control wash out of soil fines. Filters are to be designed and constructed to be stable under all conditions.

c) Concrete cover shall be designed with a suitable blanket granular underdrain to adequately control the build-up of porewater pressure under the concrete and discharge to a frost-free outlet.

6.8.3.5 Hydrogeological Effects of Slope Drainage

a) Potential adverse hydrogeological effects resulting from the permanent cut slopes shall be considered in design. Adverse effects of changing the groundwater regime could include the following:

1) Migration or changes in flow patterns of groundwater, including possible transport of contaminants.

2) Loss of adequate bearing under foundations.

3) Ground movement.

4) Reduction in the water level in existing wells, ponds or other bodies of water outside of the right-of-way.

b) The need to treat water collected in drains and sumps shall be assessed and confirmed, prior to disposal, both during construction and in the long term.

6.8.4 Maximum Slope

6.8.4.1 The maximum permanent slope gradient as calculated from analysis for resistance against deep-seated slope failure may exceed the serviceability requirements for the slope surface. If slopes are vegetated, maintenance requirements (lawn mowing, replanting, etc.) and the effects of soil softening because of freeze-thaw cycling, shall limit the maximum practical slope gradient to 2.5 horizontal to 1 vertical.

6.9 Ground Treatment

6.9.1 Dewatering

6.9.1.1 Dewatering is the most used form of ground treatment in the GTA. This is generally applied from the ground surface but can be used from the tunnel or from the base of an excavation. Although deep wells have been successfully employed in the GTA, it is usually necessary to include vacuum assisted wells, such as eductors, in the relatively fine grain deposits that predominate. It is generally necessary to lower the water table to below the base of the excavation to avoid instability at the excavation face.

6.9.2 Station Headwalls

6.9.2.1 For transit tunnels in the GTA it has become standard to include station headwalls. Although these are more exactly described as ground replacement than ground treatment, they have been included under the heading for the latter. These were developed on the Sheppard Subway Line in the 1990's. Generally, they consist of secant pile walls pre-installed at each end of the station platform tunnels. The tunnel boring machines then cut through these walls during the installation of the running tunnels. The reinforcement in the wall within the tunnel 'eye' (the section of the wall to be cut through) consists of fibreglass or similar cuttable material. This procedure avoids the complex process of installing SOE around pre-existing tunnels. The negative is that the tunnels within the station shall be dismantled and disposed of. Typically, the saving in tunnelling productivity, station excavation reduction and simpler SOE more than compensates for the loss of these tunnels.

6.9.2.2 Station headwalls shall be installed for all transit stations except where the tunnel horizon is within rock.

6.9.3 Jet Grouting

6.9.3.1 Jet grouting consists of the replacement of soil underground with a soil/cement mixture through a relatively small surface penetration. This is accomplished by using a grouting drill stem equipped with a device at its lower end that has horizontal jets. There are three types, a single fluid system (slurry grout jet), a double fluid system (slurry grout jet surrounded by an air jet, and a triple fluid system (water jet surrounded by an air jet with a separate slurry grout port). These are selected based on ground conditions. In each case, the drill stem is jetted into the ground and is rotated during the withdrawal process, during which the fluids from the horizontal jets erode and replace the soil forming an in-situ soil/cement column. Granular soils are more erodible and will result in larger columns. Some harder cohesive materials are not erodible using this process. The excess spoil is removed through the surface penetration during the grouting process. This technique is generally used for underpinning of structures or large utilities.

6.9.3.2 For the Eglinton Crosstown tunnels, this technique was used to construct station headwalls. This selection was driven by the fact that headwall installation laterally across a right-of-way always requires the relocation of utilities. This is typically handled by utility diversion around the headwall or temporary diversion followed by restoration through the headwall. For Eglinton Crosstown, the narrow right-of-way precluded reasonable utility relocation and the headwall extended the full width of the right-of-way. Jet grouted headwalls offered a solution, having relatively small surface penetrations that could be located to avoid the utilities. This was successful and cost completive with secant pile headwalls. Unfortunately, during the construction, the spoil from the jet grouting was designated as a ‘waste’ requiring pretreatment prior to transport and more costly disposal.

6.9.3.3 Jet grouting is a useful ground replacement technique, but cost shall be an important consideration in its application.

6.9.4 Permeation Grouting

6.9.4.1 Permeation grouting consists of the injection of fluid grouts to permeate the structure of, generally, a soil medium but also fractured rocks. The grouting is normally used to reduce water infiltration but can have a secondary effect of increasing the strength of the grouted medium. Figure 32 gives a general indication of the applicability of various types of for permeation of different ground conditions. This shows that cement grouts, being suspended solids grouts, have applicability limited to medium sands and coarser soils. Even acrylates and acrylamide, having viscosities close to water, can only permeate coarse silts and coarser soils.

6.9.4.2 Permeation grouts shall be used to limit water inflows in granular deposits, with low-cost cement grouts used initially, followed by true chemical solution grouts (having no suspended solids. Cement grouts have a lifespan of 100 to 200 years. Acrylamide has a lifespan exceeding 300 years, acrylates have lifespans of about 50 years, while sodium silicate grout is considered a temporary solution having a lifespan of only a few years.

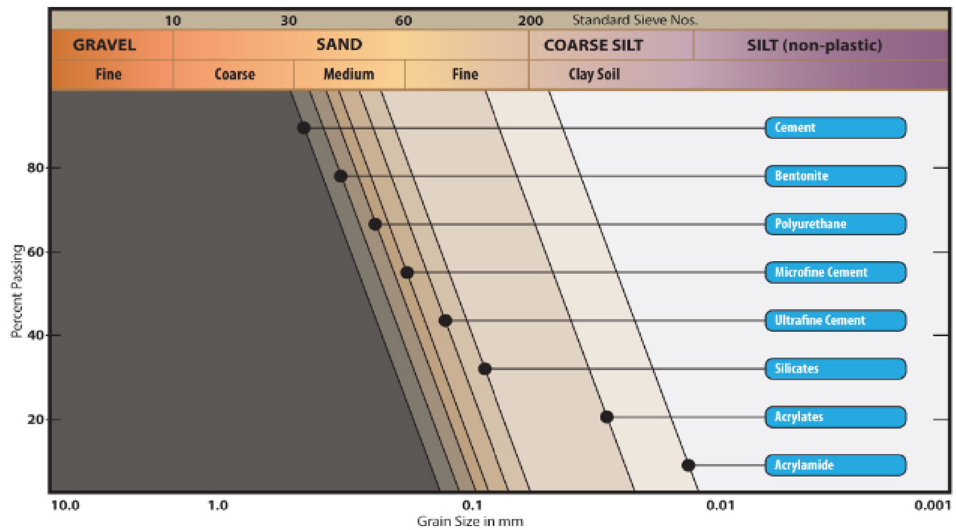


Figure 32 Applicability of Grout Types for Permeation Grouting in Various Ground Conditions (by Avanti International)

6.9.5 Consolidation or Compaction Grouting

6.9.5.1 Consolidation or compaction grouting is generally used where a densification of an in-place soil is desired. The process involves the injection of a relatively viscous, low slump, grout solution, usually cement grout, at high pressures to create a grout bulb that pushes and compacts the ground around it. Grouting pressures of 30 to 50 Bar shall be used.

6.9.5.2 The grouting is used in soft soils and soils where ground loss has occurred. Grouting shall be undertaken from the surface using a grid of drill holes and grouting points at vertical intervals within the targeted zone. Measurements shall be taken of continuous grouting pressure (accurate to 10 kPa), flow rate (accurate to 0.01 cubic metres per minute) and grout volume injected (accurate to 5 litres).

6.9.5.3 Compaction grouting shall generally be used ahead of tunnelling, but it can be used after tunnelling to address ground loss. In the latter event, care shall be taken to avoid damage to the installed tunnel resulting from the significant grout pressures involved. Monitoring of the movements of the ground surface, adjacent buildings and tunnels shall be an integral part of the grouting process.

6.9.6 Compensation Grouting

6.9.6.1 Compensation grouting is a variant of compaction grouting where a particular building or structure has been identified as requiring protection from settlement induced by adjacent tunnelling. It is typically applied where the tunnel route is directly below the subject structure. Compensation grouting shall be a reactive process where the building is restored to its original position immediately, as settlement occurs.

6.9.6.2 The process shall consist of the installation of a, or several, horizontal fans of tube-a-manchette pipes on a treatment plane below the structure foundations. These pipes shall allow the delivery of grout to discrete locations along each pipe. The building shall be instrumented with movement detection devices. On detection of movement, grout will be injected at an appropriate location to restore the structure, or structure element, to its previous position.

6.9.7 Ground Freezing

6.9.7.1 Ground freezing has seen limited use in the GTA. The technique consists of converting the soil into a frozen mass that therefore becomes stable and can be excavated by similar methods to those used for a soft rock. The freezing is achieved by circulating chilled brine through drilled lances or by the use of liquid nitrogen.

7. State of Good Repair - Tunnels and Structures

7.1 Classification of Assets

7.1.1 Location

7.1.1.1 The geospatial location of the asset shall be identified. This shall be by track chainage or by tunnel ring number. There shall be visual references on or near the asset to these locational identification systems. Where location by track chainage is not appropriate, the location shall be identified by mapping coordinates

7.1.2 Structure Type

7.1.2.1 The structure type shall be identified as one of the following:

a) Bored Running Tunnel

b) SEM Running Tunnel

c) Cut and Cover Running Tunnel

d) Ancillary Tunnel

  1) Cross Passages

  2) Access Tunnel

e) Underground Station (by Name)

  1) Underground Structure - Cut and Cover

  2) Underground Structure - SEM

  3) Surface Structure

  4) Emergency Exit Shaft

  5) Ventilation Shaft

f) Emergency Exit Building

  1) Underground Structure

  2) Surface Structure

g) Traction Power Substation

  1) Underground Structure

  2) Surface Structure

h) Surface Track

  1) On-street

  2) Off-street

i) Surface Station (by Name)

1) On-street

2) Off-street

j) Elevated Guideway

1) Deck Structures

2) Supporting Columns

3) Special Structures (Bridges, etc.)

k) Elevated Station (by Name)

Note: This list intends to address the structures only. Other lists would address equipment.

7.1.3 Construction Method

7.1.3.1 The list, in section 7.1.2, addresses the general construction methods. This descriptor shall identify any relevant features of the construction process relating to this structure, such as changed ground conditions, equipment breakdowns and related stoppages, significant ground movements, type of SOE, dewatering issues, reinforcement type in tunnel linings, tunnel break in and break out methodologies, weather conditions during construction, complexities such as underpinning of other structures, compensation grouting, other ground treatments, significant leaks and their repair.

7.1.4 Age

7.1.4.1 This shall be the age of the structure in years from the completion of its construction. Also, the duration of the construction shall be listed and the time between completion of construction and start of operation. The design life of the structure shall be documented under this heading.

7.1.5 Classification

7.1.5.1 The asset classification of the element shall be identified under the following classifications:

a) **Criticality Level 5**: Critical primary load bearing structure where loss of the structure would prevent operation of the transit vehicle.

b) **Criticality Level 4**: Critical primary load bearing structure but loss of the structure would not prevent operation of the transit vehicle.

c) **Criticality Level 3**: Secondary load bearing.

d) **Criticality Level 2**: Protective.

e) **Criticality Level 1**: Aesthetic.

7.1.6 Replacement Value

7.1.6.1 The replacement value of the structure shall be identified in current Canadian dollars. The value shall be based on replacement in a transit out-of-service operating condition.

7.2 Inspection and Testing of Asset

7.2.1 Types of Inspection

7.2.1.1 The categories of inspection defined in Table 25 shall be used to define the type of inspection required.

Table 25 Types of Inspection of Assets

Level	Type of Inspection	Description and Purpose
1	Initial	Establishes baseline or as-built condition
2	Routine	Periodic inspection
3	Damage	Assess damage from events - impact, fire, flood, seismic, blast
4	In-depth	Detailed inspection to investigate an identified condition
5	Analytical data	To acquire data for use in analysis
6	Special	Where significant deficiencies require monitoring

7.2.2 Inspection Team Qualifications

7.2.2.1 The inspection team leader shall be a Professional Engineer (P.Eng.) registered in Ontario, with 10 years of experience of the design of structures including five years of inspection of structures. Supporting inspectors shall be Engineering Technicians or Technologists registered in Ontario.

7.2.3 Asset Condition Ratings

7.2.3.1 The asset inspections shall deliver Asset Condition Ratings as defined in Table 26.

Table 26 Asset Condition Ratings

Asset Condition Rating	Description
5	Very good - near as-built condition
4	Good - minor repairs required
3	Fair - substantial repairs required
2	Poor - in need of immediate repair
1	Very poor - to be closed for transit use

7.2.4 Initial Inspection

7.2.4.1 On new structures, the initial inspection shall be conducted after the completion of construction activities and the testing of functional systems but prior to opening of the transit facility. At a minimum, the initial inspection shall consist of a sufficient number of observations and measurements to determine the physical and functional condition of the structure. These inspections are intended to be comprehensive covering the structural, civil, mechanical, electrical and lighting, fire and life safety, security, signs, and protective systems.

7.2.4.2 The initial structure inspection shall establish the baseline conditions of the tunnel; and it is used to field verify the initial structure inventory data. The baseline results shall be used to evaluate changes over time to the structures and to help identify trends.

7.2.5 Routine Inspection

7.2.5.1 Following the initial inspection, routine inspections shall be conducted within the intervals established in Section 7.3. Routine inspections shall be regularly scheduled inspections that help to ensure continued safe, reliable, and efficient service. These inspections shall be similar in scope to the initial inspection. Routine tunnel inspections shall record the changes to the structure over time and shall be used to help identify trends and predict future life expectancy of components.

7.2.5.2 At a minimum, routine inspections consist of a sufficient number of observations and measurements that can be used to determine the physical and functional condition of the tunnel. These inspections are intended to be comprehensive covering the structural, civil, mechanical, electrical and lighting, fire and life safety, security, signs, and protective systems.

7.2.6 Damage Inspection

7.2.6.1 Damage inspections shall be performed in response to natural disasters or human activities that damage the structure. Damage may occur by transit vehicle impact, fire, flood, earthquake, vandalism, or explosions. When severe damage occurs, the structure shall remain closed until a damage inspection has been completed. Structural analysis and follow-up emergency repairs may be needed.

7.2.6.2 Safety is of paramount importance after an incident. Devices such as breathing apparatus, protective clothing, and specialized equipment may be necessary. Inspection work shall be coordinated with emergency responders. It is important that the structure inspection teams shall develop detailed plans and conduct training exercises with the transit facility personnel in advance of these events.

7.2.7 In-depth Inspection

7.2.7.1 In-depth inspections shall be close-up, hand-on inspections conducted on one, several, or all the elements or functional systems of the reference structure. These inspections shall be used to identify deficiencies that are not readily detectable during initial, routine, or damage inspections. In-depth inspections may involve testing of the structure system, components, and materials. More extensive disassembly and cleaning of equipment parts may occur. This type of inspection shall be used to support a structural analysis or a functional system evaluation where more information is needed. In-depth inspections shall be scheduled based on the needs of the transit facility, inspection findings, and established written procedures.

7.2.8 Analytical Data Inspection

7.2.8.1 An inspection for analytical assessment shall provide the physical information about an asset necessary for the assessment of the asset to be undertaken. The inspection shall be a close inspection of all inspectable and/or critical elements of the asset carried out within touching distance to determine actual section sizes and the extent of any deterioration, or other features having an influence on the ability of the asset to perform its required duty.

7.2.8.2 The inspection shall cover not only the condition of the individual components, but also the condition of the structure as a whole, especially noting any signs of distress and their cause. The inspection for the analytical assessment shall be undertaken by the person carrying out the analytical assessment. Inspections for analytical assessment shall provide information on the loading applied to the assets and factors relevant to the assets structural resistance.

7.2.9 Special Inspection

7.2.9.1 A special inspection shall be typically performed after an initial, routine, damage or in-depth inspection when significant deficiencies have been discovered and shall be monitored. Special inspections are scheduled based on the needs of the transit facility, inspection findings, and established written procedures. These types of inspections shall continue, but perhaps at adjusted intervals or durations, until the deficiency is repaired, the component is removed from service, or further study determines that the conditions are no longer deteriorating at accelerated levels.

7.3 Inspection Frequency

7.3.1 Initial

7.3.1.1 The initial inspections shall be undertaken prior to the opening of the transit facility.

7.3.2 Routine

7.3.2.1 Routine inspections shall be scheduled so that each structure is inspected with a frequency of one year. A schedule shall be established to allow inspection teams to inspect structures on a rotational basis so that each structure is revisited one year after its previous inspection.

7.3.3 Damage

7.3.3.1 Damage inspections shall be undertaken as quickly as practicable after the damage has been discovered and reported and therefore cannot be scheduled ahead. These inspections shall be executed as an urgent response since the incident may involve a closure of the effected transit facility.

7.3.4 In-depth

7.3.4.1 In depth inspections shall be scheduled to investigate an issue of concern that requires more detailed investigation. These are typically required because of an issue found within a routine inspection and therefore cannot be scheduled ahead.

7.3.5 Analytical Data

7.3.5.1 Analytical data inspections shall be scheduled when additional information or physical testing is required. These shall be undertaken to resolve structural concerns discovered during other inspections and therefore cannot be scheduled ahead.

7.3.6 Special

7.3.6.1 Special inspections shall be to monitor an issue of concern that has been identified in other inspections. The scheduling of these inspections shall be by the inspection team based on the seriousness of the concern.

7.4 Record Keeping

7.4.1 BIM Model

7.4.1.1 The record keeping for the operations and maintenance period shall utilize the BIM model as the data collection tool. The maintenance and repair data for each structure or structural element shall be linked to that structure or element in the 3-D model. The data shall be collected and organized in accordance with the Metrolinx Asset Data and Information Standard.

7.4.2 Other Records and Samples

7.4.2.1 Physical samples shall be stored in appropriate storage facilities.

8. Sources of Information

8.1 Section 1

Toronto Transit Commission, Design Manual, Revision 42, August 20, 2021.
Metrolinx, Light Rail Transit, Design Criteria Manual, March 2016.
Metrolinx, Asset Data and Information Standards, 10/06/2020.
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Close Out Report for St Clair Tunnel, 1996.

8.3 Section 3

To be completed.

8.4 Section 4

Toronto Transit Commission, Design Manual, Revision 42, August 20, 2021.	
Metrolinx, Light Rail Transit, Design Criteria Manual, March 2016.	
Toronto Transit Commission, Geotechnical Standards - Direction for Conducting Site Investigations.	
	Part A: Specification for the Supervision and Reporting of Site Investigations.
	Part B: Geotechnical Report.
	Part C: Soil Classifications, Definitions and Descriptions.
	Part D: Rock Classifications, Definitions and Descriptions.

Part E: Geo-environmental Definitions.
Appendix 1: Notes on Borehole Drilling and Sampling.
Appendix 2: Guidelines for Piezometer Monitoring Well Installation.
Appendix 3: Guidelines for Test Pit Investigations.
Appendix 4: Use of Geophysical Investigations.
Appendix 5: Discussion on Geotechnical Laboratory Testing.
Appendix 6: Discussion on Chemical Testing.
Appendix 7: Well Decommissioning Procedures.

Infrastructure Ontario, Site Investigation Guidelines for Due Diligence and Design Purposes - Social and Civil Infrastructure Projects - Final, November 2018.
Metrolinx, Advance Tunnels for Eglinton Crosstown West Extension (ATECWE), Project Agreement - Output Specifications - Schedule 15 - Section 3.13 - Tunnel, 2021.
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8.7 Appendix B

Toronto Transit Commission, Geotechnical Standards - Direction for Conducting Site Investigations - Appendix 2.

8.8 Appendix C

Toronto Transit Commission, Geotechnical Standards - Direction for Conducting Site Investigations - Appendix 3.

8.9 Appendix D

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8.10 Appendix E

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9.3 Section 3

To be completed

9.4 Section 4

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ASTM D1587, "Thin-Walled Tube Sampling of Soils".
ASTM D2113, "Rock Core Drilling and Sampling of Rock for Site Investigation".
ASTM D2573, "Field Vane Shear Test in Cohesive Soil".
ASTM D2487, "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)".
ASTM D2488, "Standard Practice for Description and Identification of Soils (Visual Manual Procedure)".
ASTM D3441, "Mechanical Cone Penetration Tests of Soil".
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ASTM D4719, "Prebored Pressuremeter Testing in Soils".
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ASTM D4050, "Standard Test Method for (Field Procedure) for Withdrawal and Injection Well Tests for Determining Hydraulic Properties of Aquifer Systems".
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9.5 Section 5

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ASTM D5777, "Guide for Using the Seismic Refraction Method for Subsurface Investigation".
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A Appendix A- Notes on Borehole Drilling and Sampling

A.1 INTRODUCTION

A.1.1 Borehole drilling provides most of the geotechnical and hydrogeological design data. Borehole drilling shall likely be carried out during every design and construction phase.

A.1.2 All investigations shall require that a Health and Safety and work plan be developed for the site. Such plans shall incorporate drilling, sampling, borehole completion and personnel protection protocols that are as, or more, stringent than the minimum standards outlined below. The Health and Safety plans for potentially contaminated sites shall be site specific and take account of the possible types of contamination and the physical constraints at the site.

A.1.3 Traffic Control and Protection Plans shall also be required for any investigations that are carried out and where there are vehicle movements around the drilling crew, such as on city streets, bus loops, parking areas, access roads, and bus garages. The Traffic Control and Protection Plan shall make reference to the latest edition of the Ontario Traffic Manual Book 7 Temporary Conditions, by the Ministry of Transportation Ontario (MTO).

A.2 PREPARATION

A.2.1 General

A.2.1.1 At the beginning of a borehole investigation, a well-defined work plan shall be prepared that identifies the locations of boreholes, their depth, the types of in-situ tests to be carried out, the number and type of samples to be obtained and the location and types of piezometer/monitoring well installations.

A.2.1.2 The work plan shall be flexible and subject to revision as the details of the subsurface conditions are reviewed during the drilling process. Apart from the preparation of the work plan, a number of items shall be attended to prior to breaking ground with a drill rig. These items are addressed in the following sections.

A.2.2 Access Permits

A.2.2.1 For boreholes carried out on public property, permission to drill shall be obtained from the Authority Having Jurisdiction at the particular site. Boreholes advanced in parkland require permission from the appropriate department of Parks and Recreation, and boreholes on a Hydro right-of-way require authorization from the hydro authority.

A.2.2.2 If boreholes are advanced on private property on behalf of Metrolinx, authorization from the owner shall be obtained before moving equipment onto the site. All permission to enter onto private property, or other governmental jurisdictions typically will be made by Metrolinx.

A.2.2.3 For boreholes carried out within the City of Toronto right-of-way, road cut permits shall be obtained from the appropriate City of Toronto Works and Emergency, Permit Department. Drilling on City roads is often restricted to particular times of day and, therefore, where possible it is usually easier and more convenient to drill on adjacent side streets. These permits are usually conditional on adequate backfill being placed in the completed boreholes (discussed in Section 7). Road cut permits typically will be obtained by Metrolinx. Permits to access and drill within City of Toronto Parks Department typically shall be obtained by the Consultant.

A.2.3 Utilities / Service Clearance

A.2.3.1 Prior to beginning a borehole within the City right-of-way and public property, the locations of buried utilities and services shall be determined, and the borehole locations shall be cleared by all agencies that could have services in the area. Clearance shall generally be obtained during on-site meetings with representatives of the various agencies. In these situations, the location of nearby utilities and services should be staked or painted on the ground and a written record, usually provided by the service agency, of the utility location(s) shall be provided to the company supervising the drilling work.

A.2.3.2 Some of the agencies that may have buried utilities and services within the drilling area are listed below:

a) Bell Canada

b) Enbridge Consumer's Gas

c) Cable TV (Rogers, Shaw)

d) CN/CP Telecommunications

e) Hydro One

f) Toronto Hydro

g) City of Toronto for:

    1) Sanitary Sewers

    2) Storm Sewers

    3) Water Distribution

h) Ministry of Environment for:

    1) Trunk Sewers

i) Toronto Transit Commission

A.2.3.3 It is the responsibility of the Consultant to contact and obtain the necessary service clearances from all agencies, that may have buried utilities in the area of a borehole. All underground utility service clearance forms and a check list of contacted utility companies shall be provided to Metrolinx. It is also the responsibility of the Consultant to locate the borehole outside of the clearance limits established by each of the agencies. The responsibility for the "as-staked" locations of the service, however, shall remain with the utility company. In some situations, it may not be possible to locate a borehole the required distance from all area utilities. In this instance hand digging or “daylighting” (hydro-vac excavation) to locate the service shall be required prior to commencing drilling.

A.2.3.4 For boreholes on private property many agencies will not locate services beyond the property line and only the owner's records may be available. These records may not be reliable, and the responsibility for any damages to services will be less clear. Therefore, boreholes on private property shall be avoided where possible and, where carried out, a clear understanding of service clearance responsibilities shall be established, and the location of underground services should be reviewed and identified by a private locating firm. In addition, geophysical techniques and/or hand digging, or daylighting may be required.

A.2.4 Water Supply

A.2.4.1 Wash boring, auger drilling through wet sands, and rock coring all require an on-site water supply during drilling. Arrangements for water supply shall be made before the start of the borehole investigation to avoid project delays. Fire hydrants provide the most efficient water supply for borehole drilling and are generally available for use from May through to October. A hydrant use permit is required within the City of Toronto and shall be obtained by the Consultant before a hydrant is used.

A.2.4.2 In the winter months and in those areas distant from hydrants, the drilling contractor shall provide their own water supply. In these cases, it is best if the contractor has a water supply vehicle. It shall be agreed that the contractor will start each day with a full supply of water and the refilling station shall be selected before the start of the drilling project.

A.2.4.3 When drilling on potentially contaminated sites, the drilling contractor shall collect water used for drilling and/or cleaning downhole equipment. The drilling contractor shall provide suitable containers to store and transport the water and the consultant shall arrange for appropriate disposal of the water. The Consultant shall be prepared to arrange a disposal contract with a licensed hauler who has a "systems" Certificate of Approval and shall also obtain appropriate emergency waste generator numbers from the Ministry of Environment.

A.2.5 Cuttings Disposal

A.2.5.1 All boreholes deeper than 3 m in depth, or those shallower than 3 m that intersect the groundwater table, shall be grouted upon completion. In these cases, cuttings cannot be disposed of down the hole and, therefore, arrangements for cuttings disposal shall be agreed before the start of drilling. The regulations current at the time of disposal shall be followed. Current procedures are outlined below.

A.2.5.2 The drilling contractor shall be required to supply lugger boxes or clean, sealable 205 L drums for the collection and transport of drill cuttings. The company supervising the drilling work shall arrange for suitable disposal. Chemical testing appropriate for the proposed disposal site shall be required on a composite soil sample from each container. For example, soil with no visual or odour impacts may be directed for lakefill disposal but shall first require testing to satisfy the Toronto and Region Conservation Authority's "Improved Landfill Quality Control Program".

A.2.5.3 If during the course of drilling, noxious odours or visual impacts are observed, the cuttings shall be segregated and disposal shall be directed to a registered landfill site, after an MECP Regulation 347 (as amended by Regulation 558) TCLP test has been performed to classify the waste material.

A.2.5.4 Consultants shall ensure that MECP licensed haulers are used to remove any materials that are contaminated. The waste hauler shall have a Certificate of Approval listing as many types of licensed landfill waste types as possible.

A.3 DATA COLLECTION AND RECORDING

A.3.1 The Consultant's field supervisor shall be primarily responsible for ensuring that the borehole investigation work plan is carried out, and that, in the process, in-situ tests are properly executed, samples are properly obtained and that drilling, sampling and testing activities and observations are logged as they are carried out. The drilling supervisor shall also be responsible for ensuring that the location, northing, easting and geodetic elevation of the borehole are surveyed during or soon after drilling.

A.3.2 A standard field borehole log, that shall provide a permanent record of borehole investigation activities, shall be used for the means of recording information during drilling. The field borehole log shall include a description of each sample obtained and a summary of the stratigraphy encountered during the investigation. Descriptions shall be in accordance with the standards set out in Sections 6.3 and 6.4. Samples shall also be inspected and tested in the geotechnical laboratory. In the field, therefore, emphasis shall be placed on the recording of observations that can only be made during drilling. Auger scraping or grinding, for example, may indicate cobbles or boulders at depth, yet the presence of such obstructions shall not be evident from borehole samples. The presence or absence of water in samples and in the borehole shall be noted.

A.3.3 The minimum information that shall be recorded on a field borehole log is listed and described below:

A.3.3.1 Borehole Number - in accordance with 6.2 Drilling Date(s)

A.3.3.2 Time

a) Drilling time shall be recorded for contractor payment purposes. Productive time, standby time and delay time shall be separately recorded, and average daily rates of advance shall be calculated.

A.3.3.3 Drilling Contractor/Equipment

a) The name of the drilling contractor and the site representative (the driller) shall be recorded, as shall the make and model of the drill rig, the size and type of augers or casings used and the size and weight of the standard samplers.

A.3.3.4 Weather

a) Average Daily Temperatures and general weather conditions shall be recorded.

A.3.3.5 Location

a) A written description and a field sketch (with dimensions) shall be provided.

A.3.3.6 Topsoil/Pavement

a) Often samples are not obtained from ground surface, however, the thickness and nature of surface materials shall be recorded. For pavements this includes the thickness of asphalt, concrete and granular base.

A.3.3.7 In-Situ Tests

a) The number, depth and result of each in-situ test shall be recorded.

A.3.3.8 Drilling Advance

a) Comments on the rate of advance or relative difficulty of borehole advance shall be provided. The depth of obstructions and any indications of obstructions such as grinding of the augers shall be noted.

A.3.3.9 Sampling

a) The number, depth, type and size of each soil sample shall be recorded, and the actual amount of soil recovered, and a description of the recovered soil shall be provided.

A.3.3.10 Groundwater

a) The depth at which groundwater is first encountered shall be recorded as shall the degree of wetness of the sampler; the groundwater level within the hole at the beginning and end of each day shall also be recorded.

A.3.3.11 Strata Summary

a) A summarized description of each soil stratum encountered shall be provided, together with the interpreted or observed boundary between the strata.

A.4 DRILLING METHOD

A.4.1 Advancing the borehole and retrieving samples from the specified depths is the responsibility of the drilling contractor. The selection of the drilling method and techniques shall be agreed by the drilling contractor and the Consultant's field supervisor. The methods shall be based on knowledge of local conditions, experience with the drill rig and equipment, and on the sampling and testing requirements of the investigation.

A.4.2 In some ground conditions, the drilling method can affect the results of in-situ testing and the quality of samples. In such situations the Consultant's field supervisor shall direct the drilling contractor to change the method to minimize soil disturbance. Examples of situations in which the drilling method may induce excessive soil disturbance are provided below.

A.4.2.1 Solid Stem Augers: The use of solid stem augers for shallow boreholes is common, providing the most rapid means of augering and sampling shallow boreholes. Within wet sands, soft clay or organic soils, the uncased standard auger hole will usually not remain open to allow free access for the soil sampler. The sampler may become blocked or partially filled with sloughing soils on the way to the bottom of the hole and squeezing of the side soils and heave of the base soils shall affect the in-situ test results. In the above situations the drilling method shall be changed to hollow stem augering or wash boring.

A.4.2.2 Hollow Stem Augers: The removal of the auger plug within saturated sands and silts can "suck" soil into the augers, disturbing the bottom of the borehole. Filling the augers with water prior to plug removal, to provide a counter-balancing head, may reduce this problem, as may augering without the plug and washing the cuttings out with an interior bit.

A.4.2.3 Artesian Conditions: Artesian groundwater conditions, in which the total head of water is above the ground surface may require the use of a weighted drilling mud to counteract the porewater pressure. Such rotary drilling techniques may be outside the experience of some conventional geotechnical drilling contractors and, therefore, if such conditions are anticipated an experienced, specialist drilling contractor shall be retained.

A.5 SAMPLING

A.5.1 General

A.5.1.1 The proper retrieval, identification, storage and transport of soil and rock samples is of fundamental importance to a borehole investigation program. Once retrieved from the ground, every sample shall be immediately placed in a container and labelled. Included in this, a small representative portion of the sample shall be placed in a cardboard box as per the attached drawing and labelled.

A.5.1.2 As a minimum, the labels shall identify:

a) The Project

b) Sampling Date

c) Borehole Number

d) Sample Number

e) Depth Interval

f) Initials of Supervising Technician

A.5.1.3 Verification of the depth interval is especially important and shall be confirmed by measuring the sampler and the drill rods as they are lowered down the borehole.

A.5.2 Soil Sampling

A.5.2.1 The basic methods and procedures for soil sampling are regularly carried out by drilling contractors, geotechnical engineers and hydrogeologists in the Greater Toronto area. The intent of this section is not to provide step-by-step directions for obtaining soil samples but rather, to provide standard sample sizes for use on Metrolinx projects and to highlight sampling issues to which particular attention shall be paid. For example, as discussed in Section 6, the drilling methods which induce excessive disturbance of the borehole are to be avoided, however, if such disturbance occurs it shall be noted on the borehole log and on the sample label.

a) Split Spoon Sampler: The 50 mm O.D. diameter split-spoon sampler, used in conjunction with the Standard Penetration Test, is the most common sampling tool in the GTA, and this size shall be used as the standard. Within some fills, use of a 75 mm O.D. split spoon sampler may provide improved sample recovery and shall be used where sample recovery is more important than the results of the Standard Penetration Test. Likewise, the use of a "sand trap" will improve recovery when sampling saturated sands and silts. Upon retrieval of split spoon samples, the sample recovery and condition shall be immediately noted.

b) Shelby Tube Sampler: Shelby tube samplers are generally used to retrieve relatively undisturbed samples of soft to stiff clayey soils for subsequent strength and/or deformation testing. If possible, 75 mm Shelby tubes shall be used. To minimize disturbance, such tubes shall be pushed into the soil a distance of at least 75 mm less than the inside tube length. After the tube is inserted, it shall be rotated by hand and then allowed to "set up" for at least two minutes prior to retrieval. The sample recovery shall be recorded, and the ends of the tube sealed using either wax or tightly fitting plastic caps, secured with electrical tape. Transportation of the Shelby tube after sampling shall be done as per ASTM D- 4220.

c) Chemical Testing Requirements: Chemical testing of some soil samples shall be carried out as part of routine site screening and investigations or off-site disposal requirements and, minimum standards of sample care are required to ensure representative test results. These minimum requirements shall need to be enhanced for investigations on contaminated sites. Split spoon samplers, which are used repeatedly, shall be rinsed between sampling with clean water and decontaminated to prevent any cross contamination between the samples and boreholes. Furthermore, oil or grease shall not be applied to the threads of the sampler.

A.5.2.2 Split spoon, side barrel or wash samples shall be placed in 500 mL glass jars and foil shall be placed on top of the jar to separate the soil from the lid. Alternatively, samples shall be placed in two polyethylene bags, independently sealed (depending on the type of chemical analysis). All work should be performed in accordance with the Guideline For Use At Contaminated sites in Ontario (Ontario Ministry of Environment, June 1996, as revised) and Regulation 153/04 Records of Site Condition - Part XV.1 of the Environmental Protection Act (Ontario Ministry of Environment, 2004, as amended). Sample collection and storage requirements shall follow MECP "Guidance on Sampling and Analytical Methods for Use at Contaminated Sites in Ontario, Ontario Ministry of Environment, Standards Development Branch, May, 1966". Where not covered in the above document, recognized guidelines or standards such as EPA, APHA, MISA and other MECP Methods shall be followed.

A.5.3 Rock Sampling

A.5.3.1 Rock sampling is usually carried out using rotary rock coring techniques, although some samples of weathered shale in Toronto may be obtained using a split-spoon sampler. Typically, once auger resistance is encountered in a borehole, a split-spoon sampler shall be used to identify the source of the obstruction and the presence of possible bedrock. Any natural fractures within such a sample will be completely masked by the fractures caused during driving. The rock type and an impression of whether or not it is weathered will be discernable from split spoon samples and shall be noted, as shall the presence of any interlayers. Determination of the state of weathering requires rock coring.

A.5.3.2 Once rock has been observed within the split-spoon sampler, rock core samples shall be taken. In most cases the rock core shall be obtained using a conventional soil sampling rig, that rotates and retrieves the core barrel using conventional drill rods. The use of wireline rock coring equipment shall be considered when long sections of core are required from relatively deep boreholes. Prior to beginning a core run a casing shall be advanced 0.15 m to 0.3 m into the bedrock and the inside of this casing shall be cleaned using a rotary bi-cone or tri-cone bit down to the bottom of the casing.

A.5.3.3 As a minimum standard, an "N" sized (47.5 mm diameter core), swivel-type double tube core barrel shall be used for rock. With this type of core barrel, the inner tube does not rotate during drilling. Core shall typically be retrieved in 1.5 m to 3 m long runs. Immediately after removal from the ground the rock core shall be placed in a wooden core box with the top and bottom of the core clearly identified. The core shall be logged in the field with an emphasis placed on logging joints and fractures and on identifying mechanical fractures induced during coring. A separate field form shall be used for logging rock coring operations. The rock core shall be wrapped in plastic sheeting in the field and transported to the geotechnical laboratory for storage and subsequent inspection. The rock core shall be stored in a humid room until laboratory testing and inspection is complete. At the time that laboratory inspection is carried out the rock core shall be photographed.

A.6 IN-SITU TESTING

A.6.1 General

A.6.1.1 In-situ tests are carried out during borehole investigations, and they measure the "in-place" behaviour of soils and rock. The most common in-situ test that shall be carried out during the investigations will be the Standard Penetration Test. If soft to firm clayey soils are encountered, the field vane test shall be carried out. Because of the frequency with which these tests are likely to be carried out, minimum standards for their execution are provided in the following sections.

A.6.2 Standard Penetration Testing

A.6.2.1 The Standard Penetration Test is the most common test carried out during geotechnical investigations. The test is considered an index test, used to assess the consistency and relative density of cohesive and cohesionless soils, respectively. The test is carried out by driving a standard 50 mm O.D. open drive split barrel sampler from the base of a borehole with a 63.5 kg hammer, dropped from a height of 0.76 m.

A.6.2.2 The method of Standard Penetration Testing is documented in ASTM D1586, and this shall be adhered to. The testing interval shall typically be 0.75 m for the first 6 m below ground surface with a testing interval of 1.5 m thereafter. The testing interval shall be adjusted based on project specific requirements. For example, in tunneled sections of an alignment, a 1.5 m testing interval shall be used down to the tunnel zone, defined as extending from 1 D above the tunnel (D = excavated tunnel diameter) above the tunnel to 0.5 D below the tunnel, where a 0.75 m testing interval shall be used. Below this zone, the 1.5 m interval shall be restored. In addition, and at the discretion of Metrolinx Geotechnical staff, in the tunnel zone the split-spoon may be driven 600 mm into the ground, as opposed to the standard 450 mm penetration depth. This testing procedure will allow near continuous sampling over the tunnel interval. When the sampler is driven 600 mm, the 'N' value shall still be taken as to the number of blows of the hammer required to advance the sampler from 150 mm to 450 mm.

A.6.2.3 Similarly, for boreholes adjacent or within cut and cover excavation, the testing intervals shall be 0.75 m from the ground surface to the base of the excavation. Below this, the 1.5 m testing interval shall be used.

A.6.3 Vane Shear Testing

A.6.3.1 The in-situ vane shear test is carried out in cohesive soils at the base of a borehole, usually after a split spoon or Shelby tube sample has been obtained. The vane test is carried out to measure the undrained shear strength of soft to stiff cohesive soils. Standard vane test apparatus is described in ASTM D2573. In addition, the Ministry of Transport Ontario (MTO) has its own standard vane dimensions. A single standard vane type is not viewed as a necessary requirement for all site investigations. Any vane used, shall, however, have a minimum height of 100 mm, a height to diameter ratio of about 2 and the dimensions of the vane shall be measured and recorded. The vane factor, converting applied torque to shear resistance, shall be calculated based on the dimensions of the vane used during the site investigation.

A.6.3.2 The vane shear test shall be attempted each time a SPT 'N' value less than or equal to 10 blows/ 0.3 m penetration is recorded in cohesive soils.

A.6.3.3 In carrying out the vane test, the vane shall be pushed, without turning, 0.45 m below the bottom of the borehole. The vane shall not be pushed through the hole left by a split-spoon sampler. If it is required to follow an SPT test with a vane, the borehole shall be drilled out to the bottom of the hole left by the split spoon before inserting the vane. Depth measurements with the vane shall be made from the centre of the vane. Once pushed into position the vane shall be slowly rotated using a torque wrench or spring scales and the maximum torque reached shall be recorded. If spring scales are used the reading on each scale and the distance between scales shall be recorded in addition to the torque. Once this test is complete the vane shall be rotated a minimum of five times and the test repeated to measure the remoulded strength of the clay. The spring balances or the torque wrench used for vane testing shall be calibrated at the beginning and end of each investigation program to verify that correct scale readings are being measured and recorded. Alternatively, an adjacent borehole shall be advanced where only vane tests are carried out at selected depths corresponding with the very soft to stiff zones in the adjacent sampled borehole.

A.6.4 Other In-Situ Tests

A.6.4.1 Other in-situ tests shall also be carried out as part of site investigations. These tests may include but not be limited to pressuremeter, self-boring pressuremeter, dilatometer, K_o step blade, and Peizocone (CPTu) testing.

A.6.4.2 Standards for the type and operation of the above in-situ testing tools vary. Rather than attempt to specify equipment and testing methods, consultants retained to carry out such testing shall have demonstrated experience in the execution and interpretation of such tests.

A.7 BOREHOLE COMPLETION

A.7.1 All boreholes deeper than 3 m, or if the groundwater table is encountered within 3 m of the ground surface, shall be grouted upon completion in order to prevent the abandoned boreholes from acting as conduits for contaminant transport and to minimize the impact of the borehole investigation on subsequent construction, as per MECP Regulation 903. Piezometer or monitoring well installation shall follow the standards set out in Appendix B. Once the installation is completed, the remainder of the borehole shall be grouted up to the ground surface.

A.7.2 Two types of grout that shall be considered for use are Cement-Bentonite Grout and Bentonite grout. The selection of which grout to use will be dependent on the experience and equipment available to the drilling contractor. Two grout mixtures appropriate for backfilling boreholes are provided below:

a) CEMENT-BENTONITE GROUT: 40 kg cement: 1.2 kg bentonite: 24 litres water BENTONITE GROUTS: 22.6 kg Benseal: 350 mL E-Z Mud: 120 litres water

b) Or 22.6 kg Benseal: 3.2 kg Quick-Gel: 120 litres water

c) Or 22.6 kg Volclay (Powered Bentonite): 90 litres water

A.7.3 The borehole grout shall be placed using tremie techniques. That is, the grout shall be pumped through a tube or pipe that is lowered to the bottom of the borehole. Pumping shall continue until grout is displaced up to the ground surface.

A.7.4 For boreholes that are to be completed as piezometers/monitoring well, the installations shall be protected at the ground surface by appropriate lockable caps. For boreholes drilled on streets, parking areas and sidewalks, a flush mounted protective casing shall be used. Boreholes located in grassed areas shall have lockable above ground steel protective casings. A copy of the key to the protective casing shall be supplied to the Metrolinx representative on site. These protective casings shall be secured in the ground using concrete or neat cement grout. Refer to Appendix B for more detailed information on piezometer/monitoring well installation.

B Appendix B- Guidelines for Piezometer/Monitoring Well Installation

B.1 INTRODUCTION

B.1.1 Piezometers are used to measure the porewater pressure within the ground. Conventional piezometers consist of a slotted pipe, sealed within the ground and connected to the ground surface by a solid riser pipe. The porewater pressure is indicated by the elevation to which water rises in the pipe and this is measured with an electronic sounding device. Monitoring wells have the same basic form as piezometers but are usually larger in diameter and the standard screen and riser pipe size recommended in the following section shall allow groundwater samples to be obtained from the installations. Although the primary purpose of the installation shall be to measure porewater pressure, each piezometer installation shall be able to function as a monitoring well.

B.1.2 As with the standards for borehole investigations, the installation and sampling standards provided for piezometers and monitoring wells are for conventional investigations. Well installation techniques, sizes, locations and sampling protocols for contaminated sites shall be addressed independently in the work plan and Health and Safety program for each contaminated site under investigation, as the nature of the contamination and the site shall influence the selection of equipment, materials and methods. Adherence to the standards proposed in this section shall provide piezometer and monitoring wells of sufficient quality to allow sites to be screened for groundwater contamination, to allow on-going groundwater level monitoring through to construction and to allow in-situ permeability tests to be carried out where required.

B.2 INSTALLATION STANDARDS

B.2.1 A piezometer or monitoring well shall be installed in selected boreholes advanced as part of the site investigation program up to the construction phase if the groundwater table is encountered and through consultation with Metrolinx Geotechnical and Geo-Environmental Design Section. The final selection of the location of the wells within the boreholes shall be determined by the field supervisor and shall depend on the stratigraphy, the type of structure proposed, the presence or suspected presence of groundwater contamination and on the installations present in the surrounding area. However, guidance for well installation shall be documented in the work plan for the investigation, and not left to the sole discretion of the field supervisor. The following provides a general prioritized list for zones over which wells shall be screened:

a) Zones within which contaminated groundwater is suspected.

b) Depth of particular interest for construction i.e., base of excavation, tunnelled interval.

c) Water bearing stratum.

d) Bottom of Borehole.

B.2.2 Installations at the bottom of a borehole are generally easiest and quickest because grouting is not necessary prior to the installation. Often an installation at the bottom of a borehole shall partly meet criteria No. 2, as this criterion will also govern the borehole depth.

B.2.3 If it is planned to install the well screen close to the groundwater table, the screened interval shall be extended to intersect the groundwater table. This shall allow detection of floating hydrocarbons, where present.

B.2.4 To allow groundwater sampling, and to allow multiple installations in a single well and to minimize cost, a standard well screen and riser pipe size of 50 mm internal diameter is recommended. Larger diameter wells shall be required at contaminated sites or where pump testing may be carried out. The PVC pipe shall be Schedule 40 thickness and shall use threaded connectors with "O-ring" seals. The well screen shall be a No. 10 slot size, should be a minimum of 1.5 m long and shall be fitted with a threaded end cap. The completed installation shall have a tightly fitting end cap placed on top of the riser pipe. The riser pipe shall not be left uncapped as it shall vent the vapours that are to be measured as part of the site screening process.

B.2.5 Prior to installing the well screen and pipe, filter sand shall be placed at the bottom of the borehole to provide a seat for the well screen. For installations in the bottom of a borehole, this seat shall be about 0.1 m to 0.2 m thick. For installations above the base of the borehole, grout shall be trimmed into place to fill the bottom of the borehole and 0.3 m to 0.6 m of filter sand shall be placed on top of the grout to provide a seat for the well screen. The filter sand used shall depend on the particle size distribution of the surrounding native soil. For most sand deposits and till deposits 3 mm silica sand shall provide a sufficient filter pack. For uniform silts, silty clays and clayey silts 1 mm silica sand shall be used for the filter pack. The filter pack shall extend at least 0.3 m above the well screen.

B.2.6 All piezometer and well installations and documentation shall be carried out in accordance with Regulation 903 of the Ontario Water Resources Act (as amended).

B.2.7 The documentation required for each well installation shall be recorded on the borehole drilling record and shall include the following information.

a) Diameter of borehole.

b) Depth and length of screen and riser pipe.

c) Diameter of well screen and riser pipe.

d) Type of filter sand.

e) Depth of top and bottom of filter pack.

f) Depth of top and bottom of seals.

g) Depth of grouted interval.

h) Length of well stick-up above ground surface.

B.3 REPORTING

B.3.1 All wells shall be registered with the MECP in accordance with Regulation 903. Copies of the registration documentation shall be provided to Metrolinx Geotechnical & Geo-Environmental Design Section no later than three weeks after installation of the piezometers and/or monitoring wells.

B.4 WATER LEVEL MONITORING

B.4.1 Water level monitoring within piezometers/monitoring wells shall reflect stabilized water levels. In addition to any water levels measured immediately after drilling, the level in the well shall be read at least one week after the installation is complete. The water level reading shall be made with an electronic sensor; sounding with a tape in monitoring wells is not acceptable. The water level measuring device shall be cleaned and decontaminated between well measurements to prevent cross contamination. The depth of the water level shall be referenced to the ground surface. Each groundwater level reading shall be recorded together with the date and time of the reading and the weather conditions at the time of the reading.

B.4.2 An ongoing record of water levels, which shall indicate seasonal variations in water level, shall provide useful data for design and construction. Regularly scheduled readings of all piezometers installed during the design process shall be carried out. Coordination of such readings shall be done by the Consultant.

B.5 GROUNDWATER SAMPLING

B.5.1 Before groundwater samples are retrieved from a well, it shall first be "developed". Well development is necessary to remove drilling water and to ensure that there is good communication between the geological formation and the well. Well development is carried out by removing water from the well, and thereby, allowing groundwater from the adjacent formation to flow into the well. At contaminated sites, the water removed during well development shall be collected and stored for subsequent disposal, after testing and characterization. Water shall be removed from the well using a variety of bailers, pumps or purge techniques. The type of apparatus used shall depend on the size and depth of the well and the equipment available to the consultant.

B.5.2 The quantity of water that shall be removed during well development shall be dependent on the size and depth of the well, on the type drilling method used and the size of the filtered zone. During development the pH, temperature and specific conductivity of the water removed from the well shall be measured in the field and recorded. Development shall continue until these readings are constant. As a minimum, three times the volume of water stored within the well screen and riser pipe shall be removed from the well during development. If the wells recharge very slowly, less development water shall be taken. However, this fact shall be reported together with the field test results.

B.5.3 Sample retrieval, storage and transport requirements are dependent on the type of analysis being carried out. Sampling protocols, especially for the analysis of organic chemicals, shall vary and sampling protocols shall be established for each particular site and situation. These protocols, that shall also include filtering, preservation and custody requirements shall be determined in consultation with the analytical laboratory prior to sampling.

B.5.4 If sampling is being carried out for the first time, then the sample shall be retrieved immediately after well development is complete. For wells that have previously been developed, four to five well volumes shall be removed from the well if it is sealed into a coarse-grained soil and one volume shall be removed from the well if it is sealed into a fine-grained soil. The sampling equipment shall be thoroughly rinsed with distilled water. Only new, clean sample containers supplied by an analytical laboratory shall be used for sample collection and transport purposes.

B.6 PERMEABILITY TESTING

B.6.1 In-situ permeability testing shall be carried out using wells sealed within a single soil deposit or rock formation. These tests will normally be carried out at later design phases when site specific design parameters are required. Typically, such tests shall be carried out in tunneled sections of a subway at locations where excavation dewatering shall be required, or where an assessment of containment flow is being conducted.

B.6.2 Both rising and falling head permeability tests shall be used to assess the permeability of soil or rock deposits within which piezometers or monitoring wells are installed. Prior to carrying out either of these tests the installation shall be developed as per the standards provided in the preceding section. Usually rising head tests are carried out in wells where the static water level is very near to the ground surface and falling head tests are carried out in wells where the static water level is relatively deep.

B.6.3 In carrying out rising head tests the water in the well shall be removed by appropriate pumping, bailing or purging methods. As with well development, if the groundwater in the well is contaminated it shall be collected, stored, chemically tested and properly disposed of. If falling head tests are carried out, only clean water shall be introduced into the well. Whichever technique is used, the recovery of the groundwater level (either upward or downward) shall be measured during the test. Water level readings shall be taken at frequent intervals using a pressure transducer, such as Solinst Levelogger. Readings shall not be taken after the water level has returned to the static level. In fine grained soils water levels may not have stabilized after 150 minutes. If the water level is more than 75% recovered, then further readings need not be taken. If the water level has recovered to less than 75% of its static level, then readings shall continue at rate a dictated by the measured well recovery rate.

B.6.4 All data from the permeability test shall be presented graphically and on a form that indicates:

a) Date, time

b) Well number

c) Well depth

d) Well diameter

e) Filter zone length and diameter

f) Static water level

g) Method of water removal (or placement)

h) Time of test start

i) Water level vs. time after the start of the test

C Appendix C- Guidelines for the Test Pit Investigations

C.1 INTRODUCTION

C.1.1 While test pits are an excellent means of shallow subsurface investigation, they are disruptive, especially in an urban environment. For this reason, test pits are not normally planned as a regular component of the phased site investigation approach. Rather, test pits are likely to be excavated at specific sites for specific purposes, such as to locate buried tanks, utilities or building foundations. Test pits shall also be used to sample and delineate fill thickness and finally, test pits shall be dug at the tendering stage to allow contractors to observe the nature and behaviour of the subsoils.

C.1.2 Test pits are normally excavated using backhoe, the size of which is dependent on the depth of excavation required. Hand dug or hydro-vac test pits shall also be excavated in "tight areas" specifically if buried utilities or foundations are being located and exposed.

C.1.3 The standards for preparation and data collection for test pit investigations are similar to those for borehole investigations, and thus, reference will be made to Appendix A. As with the borehole investigation standards, these test pit investigation standards are for "conventional" geotechnical investigations.

C.2 PREPARATION

C.2.1 As with borehole investigations, the first step of a test pit investigation shall be the preparation of a work plan that identifies the test pit location, size, depth and the type and number of in-situ tests to be carried out and the number and types of samples to be retrieved. As discussed in Appendix A, access permits shall be obtained, and the location of buried services shall be cleared before the test pit investigation is started.

C.2.2 In some cases, the purpose of the test pit shall be to locate a buried service. In these instances, the approximate location in plan and the approximate depth of the service shall be obtained from the appropriate utility. Many utilities will require that their representative be on site as the service is unearthed, and therefore, scheduling of the investigation shall take this into account.

C.2.3 If impacted materials are removed from the test pit, then these materials shall be sampled, securely stored, chemically tested and disposed of at an appropriate facility by a licensed waste hauler.

C.3 DATA COLLECTION AND RECORDING

C.3.1 All excavated test pits shall be carried out under the full-time supervision of a trained geotechnical, geoscientist, hydrogeological technician or engineer. The test pit supervisor shall be responsible for ensuring that the investigation work plan is carried out. The supervisor shall normally be responsible for obtaining the samples, carrying out the in-situ tests, and logging activities and observations as they occur. The excavator operator shall be responsible for the safe operation of the equipment and for excavating in the area and to the depth specified by the field supervisor.

C.3.2 A standard test pit log shall be used to record all observations and shall serve as a permanent record of the test pit investigation program. The test pit log shall include a description of each sample obtained, the result of each in-situ test and a sketch of the stratigraphy as observed in the side wall of the test pit. The test pit provides an extra dimension to a site investigation, and the field notes and sketches should reflect this; dip in strata boundaries or lateral variations shall be noted. Obstructions encountered during the test pit excavation shall be detailed, providing information on the size, shape and nature of the obstruction.

C.3.3 The minimum information that shall be recorded on a test pit log is similar to that required on a borehole log, and is repeated here for clarity:

a) Test Pit Number (in accordance with Metrolinx Standards).

b) Excavation Date.

c) Time: Excavation time shall be recorded as productive time, standby time and delay time for contractor payment purposes.

d) Contractor/Equipment: The name of the excavation contractor and the site representative shall be recorded, as shall the make and model of the backhoe.

e) Weather: Average daily temperature and general weather conditions shall be recorded.

f) Location/Size: A written description and dimensioned field sketch shall be provided showing test pit location and size.

g) In-Situ Tests: The number, depth, location within the test pit and result of each in-situ test shall be recorded.

h) Obstructions: Location, size, shape and type of each obstruction shall be recorded.

i) Sampling: The number, depth and location of each soil sample shall be recorded, and description of each sample shall be provided.

j) Groundwater: The location of seepage zones shall be noted, and the quantity of seepage estimated. The depth of standing water at the end of excavation shall be noted as shall any changes in water level prior to backfilling.

k) Strata Summary: A summary sketch showing the stratigraphy in the wall of the test pit shall be provided. The sketch shall indicate any dip in strata boundaries and shall include surficial materials such as topsoil or pavement structure.

l) Backfill Details: The type and method of backfill placement shall be recorded.

C.3.4 In addition to the test pit log, photographs of the test pit wall, seepage conditions and obstructions shall be taken and shall form part of the permanent test pit record. The photo number(s) shall be recorded on the test pit log and once processed; the photographs shall be dated and labelled. For test digs, organized for contractors to view the subsurface conditions, a video record of the excavations is recommended in addition to a field log and photographs.

C.4 SAFETY

C.4.1 Test pit excavations shall be carried out in accordance with the current provisions of the Ontario Occupational Health and Safety Act. Generally, test pit excavations shall be carried out in open cut in which case the side slope shall generally be cut at a one horizontal to one vertical gradient from 1.2 m above the base of the excavation to the ground surface. For soft, loose or sensitive soil or if space is restricted, the excavation shall be shored in accordance with the provision of the Act.

C.4.2 In addition to the trench sidewall safety requirements, special safety requirements shall be necessary at sites where there is subsurface impact or where subsurface impact is suspected. These requirements shall be specified in a Health and Safety Plan prepared before the start of the investigation. If subsurface contamination is unexpectedly encountered, work on the site shall cease, the Metrolinx Representative shall be contacted, and an appropriate Health and Safety Plan prepared by the site investigation consultant. The excavation equipment shall not leave the site until the nature of the contamination is ascertained and appropriate decontamination procedures are carried out.

C.5 SAMPLING AND TESTING

C.5.1 There are three basic types of samples that shall be retrieved from a test pit excavation: grab samples, bulk samples and block samples.

C.5.2 Block samples provide the best form of undisturbed sample for specialized strength and deformation testing. Such samples are, however, rarely obtained because of the difficulty in obtaining, transporting and storing them.

C.5.3 Grab samples are typically small hand samples, retrieved directly from the sidewall of a test pit or from the backhoe bucket. These samples are usually less than 1 kg in size and are used for index testing in the geotechnical laboratory. The samples should be transported and stored in a manner that will preserve the natural water content. If jars are used, the lid shall be secure and if plastic bags are used, the sample shall be double bagged.

C.5.4 Bulk samples are simply larger grab samples, often being 10 kg to 20 g in size. These are often retrieved if there are larger sized particles (gravel and cobbles) in the soil and if tests such as the Standard Proctor tests are to be carried out. Heavy plastic, jute or cotton bags are used to transport and store these samples and hence, preservation of the in-situ moisture content is often not possible. In obtaining bulk samples, it is particularly important to retrieve a representative sample and to avoid the loss of coarse sizes. Typically, a small hand excavation will be made and all materials from within the excavation shall be collected and placed in the sample bag.

C.5.5 It is fundamentally important that as each sample is retrieved, it is labelled and that the label provides the test pit and sample number, the depth, the project, the date and the initials of the supervising technician or engineer.

C.5.6 The most common in-situ test carried out within a test pit is the in-situ density test. This is often performed at regular depths as the test pit is excavated, particularly if the near surface soils are to eventually support load. The in-situ density of the soil is commonly measured using a nuclear density gauge. If such instruments are used within a test pit, the moisture correction calibration for trench sidewalls shall be performed before each test is carried out.

C.5.7 The depth of all samples and in-situ tests shall be referenced to the ground surface. The ground surface shall, in turn, be surveyed such that its elevation may be referenced to Geodetic Datum. Where the test pit is located on sloping ground, the elevation shall be determined at the high and low ends of the test pit and sample and test depths shall be referenced to one of these two locations.

C.6 BACKFILL AND RESTORATION

C.6.1 The backfill and restoration requirements shall vary from site and shall be agreed upon at the time access permits for the work are procured. In the case of test pits excavated on roadways, it shall be necessary to backfill with “unshrinkable” fill, which clearly will necessitate off-site disposal of the excavated fill. In this instance, the guidelines discussed in Appendix A for drill cuttings shall be followed.

C.6.2 In situations where backfill standards are not specified by the owner; certain minimum standards shall be adhered to. Backfill with native material is acceptable. It shall be placed in the test pit in 0.3 m lifts and be thoroughly tamped with the backhoe bucket. If the test pit is excavated in a grassed area, the sod shall be cut away before the start of the excavation and replaced upon completion of backfilling. For excavations in paved areas, the pavement base and subbase materials shall be excavated and stored separately and shall be replaced and compacted on top of the other backfill. An asphalt patch shall be placed over the completed excavation. Finally, the area shall be swept clean at the completion of backfilling work.

D Appendix D- Use of Geophysical Investigations

D.1 STATEMENT OF PURPOSE

D.1.1 A wide variety of geophysical techniques are available to replace, assist and supplement conventional site investigation techniques. Geophysical techniques have an advantage in that they are a non-destructive means of "looking" below the ground surface. The application of geophysical methods shall be site and design specific and such methods are not proposed as part of the standard phased site investigation approach. A geophysical assessment can be targeted to address multiple purposes, including:

a) Characterize known, unknown, or changes in subsurface conditions and compare them with anticipated or suspected situations to confirm or refute conditions identified through historical records (e.g., fire insurance plans, previous land use, tank removal, etc.).

b) Support the identification of buried objects, disposal pits, debris, buried geo-environmental hazards, landfill boundaries, and metallic objects (e.g., drums, tanks, debris, shoring piles, and similar objects).

c) Support the selection of borehole/test pit locations for environmental and geotechnical site assessments where further intrusive investigation is needed (e.g., anomaly investigation).

d) Support the development of a conceptual site model by assisting in quantifying geotechnical parameters (e.g., bedrock profiling, soil stratigraphy mapping, and soil moisture content assessment).

D.1.2 Table D-1 lists some key considerations when developing the scope of work for and while conducting a geophysical assessment. It shall be noted that geophysical surveys are typically non-intrusive tools for the assessment of subsurface conditions present within a project area and should be 'ground-truthed' via intrusive methods as required.

Table D-1: Key Considerations for the Geophysical Assessment

	Consideration
1	Is there evidence to suggest the presence of buried objects or subsurface infrastructure (e.g., foundation rebar, underground utility lines, underground storage tanks [USTs], or other unknown obstacles) and, if so, how does this affect the selection of borehole locations for further investigation or interfere with the development of the property?
2	What geophysical anomalies are within the project area that may require further investigation to understand the potential for contamination or the potential for site design and construction challenges such as shoring piles or tie-backs extending from adjacent properties onto the project area?
3	What is the soil moisture content across the project area and what does this suggest in terms of depth to groundwater?
4	What are the fill/bedrock conditions across the project area?
5	What is the seismic site class for the project area as per the Building Code?

6	Based on the geophysical assessment, what uncertainties remain in terms of understanding site conditions?

D.2 APPROACH

D.2.1 Certifications

D.2.1.1 All consulting firms shall have a valid Professional Geoscientists of Ontario Certificate of Authorization or a valid Professional Engineer of Ontario Certificate of Authorization to practice as a geophysical Consultant. One exception is a Professional Geoscientist cannot practice geophysics at a consulting firm with only a Certificate of Authorization for Professional Engineering.

D.2.2 Geophysical Assessment Standards and Practices

D.2.2.1 Some geophysical methods that may be suitable for the investigation purposes include:

a) Seismic (refraction or reflection and/or multi-channel analysis of surface waves [MASW]).

b) Ground-penetrating radar (GPR).

c) Electrical resistivity imaging (ERI).

d) Frequency domain electromagnetics (FDEM).

e) Gravity.

f) Downhole geophysical methods.

D.2.2.2 The selection of the appropriate geophysical technology and survey procedures should follow ASTM D6429-99 in conjunction with, and in consideration of the site history, site investigation objectives, potential site use, other parameters requiring qualification, and the recommendations of an experienced, professional geoscientist or professional engineer. The geophysical investigation shall conform to the most recent editions of the ASTM standards.

D.2.2.3 Table D-2 provides a partial list of current accepted reference ASTM methods for geophysical assessment technologies. Other ASTM Standards may also be relevant.

Table D-2: Examples of Pertinent ASTM Standards for Geophysical Assessment

Reference	Standard
ASTM D6429-99	“Standard Guide for Selecting Surface Geophysical Methods”
ASTM D5753	“Guide for Planning and Conducting Borehole Geophysical Logging”
ASTM D7400	“Standard Test Methods for Downhole Seismic Testing”
ASTM D4428/D4428M	“Test Methods for Crosshole Seismic Testing”
ASTM D5777	“Guide for Using the Seismic Refraction Method for Subsurface Investigation”
ASTM D7128-05	“Standard Guide for Using the Seismic-Reflection Method for Shallow Subsurface Investigation”
ASTM D6285	“Guide for Locating Abandoned Wells”
ASTM G57	“Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method”

Reference	Standard
ASTM D6431-99 (2010)	"Standard Guide for Using the Direct Current Resistivity Method for Subsurface Investigation"
ASTM D6432	"Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation"
ASTM D6430-99 (2010)	"Standard Guide for Using Gravity Method for Subsurface Investigation"
CSA S250-11	"Mapping of Underground Utility Infrastructure"
ASCE 38-02	"Standard Guidelines for the Collection and Depiction of Existing Subsurface Utility Data"

D.2.2.4 A geophysical assessment can be applied for utility locating in support of drilling work (that is, confirming that drilling locations are clear of utilities), as well as to complete utility mapping surveys, for example, to provide an inventory of underground infrastructure, review existing assets for facility expansion, etc.

D.3 PROCESS

D.3.1 General

D.3.1.1 Whenever possible, efforts shall be made to coordinate the field components of Phase 2 ESAs, geotechnical investigations, geophysical surveys, hydrogeological investigations, and utility mapping to allow for optimized efficiency and overall project cost savings, provided the scope of work for each individual investigation is satisfied.

D.3.2 Determination of Geophysical Survey Approach

D.3.2.1 The decision-making process for evaluating the appropriate geophysical methods to employ for a project involves the following steps:

a) Review available site information such as Phase 1 ESA results, project-specific plans regarding planned site use/construction, other historical reports, and as-built drawings, as relevant.

b) Identify the additional site information (i.e., data gaps or uncertainties) that needs to be assessed via the geophysical assessment.

c) Determine which geophysical assessment method, or combination of methods, shall optimize the collection of this information and minimize the potential for data gaps. For example, if the work is being completed in an urban environment, significant interference issues can arise with some geophysical assessment techniques due to the presence of other buildings, fences, above ground and underground power lines, road salt contamination etc. In some cases, a preliminary assessment using a certain technique shall be applied and then supplemented with additional assessment methods using complimentary techniques to compensate for the missing information. This approach can help focus the secondary evaluation on specific areas where there are subsurface anomalies and/or uncertainties due to interference.

d) Determine the necessary data density and coverage to achieve the desired outcome.

e) Section D-4 and Table D-3 summarize the most common geophysical techniques. This is not meant to be a complete list of the accepted technologies for use in Metrolinx projects. For a full description of the following methods, please refer to ASTM D6429-99, Standard Guide for Selecting Surface Geophysical Techniques. If the professional geophysicist/geoscientist involved believes a different technology, not listed in Table D-3, shall be more efficient in acquiring the desired information, the technology may be used when following the appropriate ASTM or professional practices, subject to approval of Metrolinx.

D.3.3 Completion of Geophysical Survey

D.3.3.1 The specific program of the geophysical survey shall be developed based on the objectives for the project and based on whether the work is completed as part of a preliminary investigation to gain a better understanding of a project area during the planning stage. This shall aid in the selection of the number of boreholes and their locations for example, and/or a more detailed investigation such as after construction footprints and alignments are finalized.

D.3.3.2 A reconnaissance-level investigation shall be completed to ascertain a basic geophysical understanding of subsurface site conditions as a minimum. Typically, this is completed early in a project and may aid in planning subsequent investigations for the project area. More detailed coverage may be required in potentially contaminated areas, areas in which there is data interference (underground or above ground power lines) or highly sensitive areas through either additional survey methods or a higher density of data collection (additional cross section lines for example). All data shall be captured by a global positioning system (GPS) and geo-referenced.

D.3.4 Reporting

D.3.4.1 The consultant shall prepare a stand-alone report, which may be appended to a geotechnical and/or environmental report, that contains the following information:

a) Discussion of survey objectives

b) General description of existing site conditions

c) Description of survey instrumentation and applied technology

d) Description of survey procedures

e) Description of data processing procedures (application of filters, etc.)

f) Presentation of data in colour contour map (or seismic sections) format

g) Discussion of findings

h) Recommendations for follow-up work, if required

D.3.4.2 Raw and processed data shall be presented in soft-copy format with the geophysical investigation report.

D.3.4.3 The final report shall include a project area description, survey methodology, a discussion of the processing applied to all collected data, all colour contour maps generated with surface features that may have provided interference to the collected data and positioning reference data (e.g., buildings, trees, curbs, etc.) with and without line and station spacing to show data density, and an interpretation of the results (hard copies), as well as the raw and processed data (soft copy).

D.4 TECHNIQUES

D.4.1 Seismic Refraction Survey

D.4.1.1 To profile the top of 'competent' rock, a seismic refraction survey can be completed. Care shall be taken to review the nearby 'cultural noise' (such as construction projects, airports, vibrating machinery, etc.) to establish the viability of this investigation. The source for the survey shall also be considered: a sledge hammer, weight drop, or elastic hammer or equivalent, striking a steel plate shall be sufficient to map to 10 m depth, but explosives may be required for deeper sounding. Caution shall be exercised with following local by-laws regarding noise and explosives use permits. The data shall be captured by a seismograph capable of digital recording to 'stack' successive signals and/or filter unwanted 'noise.' The spacing between geophones shall be no greater than the anticipated depth of the bedrock profile and sufficient shot points shall be engaged to permit interpretation of the data completed using the Generalized Reciprocal Method or better. Data shall be presented as processed sections as depth below surface (or in units of elevation of topographic information if available), demonstrating variations in bedrock seismic velocities that can be compared to published charts of 'rippability'.

D.4.2 Multi-channel Analysis of Surface Waves (MASW)

D.4.2.1 MASW is a seismic method measuring the Rayleigh waves produced from a passive or active source (traffic or weight drop). From the data collected, the shear modulus and damping coefficient of the near-surface soil shall be calculated for estimates of soil and rock strength or stiffness and mapping subsurface lithology, top of bedrock, and to aid in liquefaction potential analysis. A detailed description of the method can be found in the paper Multi-channel Analysis of Surface Waves.

D.4.3 Ground-Penetrating Radar (GPR)

D.4.3.1 GPR can detect both conductive and nonconductive targets, soil stratigraphy, and bedrock profiles, and provide detailed definition, position, and orientation of other features detected by other geophysical investigations. The production radar survey parameters shall be optimized via a series of tests to calibrate the radar findings with known subsurface conditions. Ideally, the test radar data shall be acquired and evaluated with two to three radar frequencies. The optimized radar system parameters shall be used to acquire the radar data along parallel profile lines at a separation sufficient to image the target with station spacing equivalent to a maximum distance of one wavelength of the selected frequency. Interpreted results shall be presented in either a processed section format, colour-contoured time-slice sections, colour-contoured bedrock surface/isopach maps, or a combination thereof, along with recommendations for follow-up work, along with a survey summary.

D.4.4 Frequency Domain Electromagnetics (FDEM)

D.4.4.1 FDEM, also referred to as electromagnetic or terrain conductivity, can be used to map lateral changes in geological or hydrogeological subsurface conditions due to conditions such as the presence of metallic targets and changes in soil conductivity due to changes in soil type or soil moisture content. These changes may infer the presence of inorganic groundwater contamination or environmentally impacted soils. An instrument such as the EM31-MK2 terrain conductivity meter (manufactured by Geonics Limited), or equivalent, may be used to complete an electromagnetic survey. Other tools are available, and selection is based on the project area and target conditions (e.g., expected depth, surface conditions, target anomaly properties, etc.).

D.4.4.2 The optimal use of this method includes establishing a survey grid where the selection of station spacing and line spacing is relative to the smallest target size (e.g., metallic drum versus underground storage tank) and instrument used (e.g., EM31-MK2 vs EM38-MK2 - transmitter and receiver separation). Ideally, a continuous stream of data is received from the electromagnetic unit by a GPS unit that can allow for differential correction of the GPS data (real-time kinetics or similar). The data density shall allow for 100% coverage of the target area to reduce the need for interpolation between data points (for example, the EM31-MK2 would ideally have a 1 m line spacing where possible when collecting continuous station readings).

D.4.4.3 Where signal saturation or significant interference occurs (near buildings, fences or metallic slab rail ballast, or the presence of overhead or underground high voltage hydro-electric transmission lines or of numerous subsurface utilities, for instance), a secondary geophysical technique, such as an EM61-MK2 or GPR survey, may aid in supplementing the recorded data.

D.4.5 Other

D.4.5.1 Electrical resistivity imaging (ERI) or electrical resistivity tomography, is the modelling of the subsurface through the induction of a current into the ground to create resistivity pseudo section depth profiles. The sections, in support of borehole data, EM31-MK2 survey, or similar, can aid in development of the geological conceptual site model and in understanding the changes in lithology, hydrology, or subsurface geologic formations (voids and the like), or a combination thereof. Electrode spacing and arrangement can vary and would depend on the objectives of the survey and site conditions.

D.4.5.2 Downhole geophysical tools such as acoustic, electric and induction, fluid logs, nuclear, caliper, borehole video and magnetic susceptibility, in addition to others, can aid in providing information on lithology, physical properties, mineral composition, rock structure, and fluid parameters. While a few tools, such as the caliper or gamma, have no restrictions on their use, some tools can only be used within cased boreholes (that is, completed or screened wells, or both), while others require open or nonconductive boreholes and may require the hole to be dry or fluid-filled. Many of the logs created need to be used as part of a combination of downhole tools to correct measurements for borehole effects and aid in the interpretation of the data collected. When geotechnical parameters such as the shear modulus, Young's modulus, Poisson's ratio, and the potential for liquefaction are needed, seismic crosshole testing can allow for the recording of P- and S- wave velocities to complete calculations for these parameters. Due to the variety of instruments available for use, and the complexity potential of a downhole geophysical program, it is strongly suggested to plan the logging program with an experienced and certified geophysical consultant.

D.4.5.3 While the geophysical program would be site-specific, some technologies and common uses that may be considered are discussed in the following Table D-3.

Table D-3: Common Uses and Limitations of Various Geophysical Testing Methods

Test Methods	Common Use
Frequency Domain Electromagnetics (FDEM)	To locate buried tanks, pipes and unexploded ordnance. To map soil salinity changes, delineating contamination plumes, map lateral changes in lithology (and vertical changes when used in conjunction with electrical profiling), leachate mapping, landfill mapping, archaeology, etc. Various instruments and techniques available depending on goals of the survey (EM31-MK2, EM61-MK2, EM34, EM38, etc.). Surveys results and quality of data can be negatively impacted by overhead powerlines, high density metallic structures on a site or on the edges of surveys, etc. and shall be considered during the planning phase
Ground Penetrating Radar (GPR)	To locate buried targets (drums, underground storage tanks (USTs), buried utility lines), to map soil stratigraphy and bedrock, fractures, void detection, landfill boundaries, utility mapping, void mapping, archeological surveys, pavement thickness, etc. The survey method is not suitable in areas with high clay content in the soil and may have interference from anthropogenic sources.
DC Resistivity	To measure the electrical properties of the subsurface (such as Wenner Array, dipole-dipole, etc.) to map features such as lithology, fractures, depth to groundwater. Can work well with EM31-MK2 surveys to estimate plume volumes of conductive materials. There are some limitations with accuracy and resolution as data results use a weighted average over a large field which is common in measuring potential fields and may have interference from anthropogenic sources.
Electrical Resistivity Imaging (ERI) or Electrical Resistivity Tomography	To model the subsurface through the induction of a current into the ground to create resistivity pseudo-section depth profiles. The sections, in support of borehole data, electromagnetic survey or other, can aid in development of the conceptual site model and in understanding the changes in lithology, hydrology, or subsurface geologic formations (voids, and the like), or a combination thereof. Electrode spacing and arrangement can vary and would depend on the objectives of the survey and site conditions. As with DC resistivity, there are some limitations with accuracy and resolution as data results use a weighted average over a large field which is common in measuring potential fields and may have interference from anthropogenic sources.
Gravity	To locate buried channels, voids, caves, structures, bedrock features, landfill characterization, USTs. Potentially suitable for an urban environment when looking to map where the footings of an adjacent building are in relation to the subject property or for void detection. The survey shall map lateral changes in gravity variations in relation to a local base station and can be used to identify between natural and man-made materials. Survey stations require a precise elevation survey that may not be conducive for a site and the equipment is very delicate.

Test Methods	Common Use
Seismic Refraction	To map geological conditions as depth slices for the possible identification of stratigraphic layers, bedrock, and depth to groundwater table, fractures, characterization of rock type through velocity analysis. Depending upon the depth of investigation, long survey lines may be required. It can be difficult to resolve strongly dipping stratigraphic units in the processing and interpretation phase.
Seismic Reflection	To map lithological layers, depth to bedrock, depth to water table, soil and rock properties. Data processing can be labour-intensive. Depth estimates can be unreliable depending on the survey design and site conditions.
Seismic Cone Penetrometer	Now used more commonly to generate site-specific profiles of shear wave velocity. These are needed in connection with development of seismic site response analyses required by building codes in weaker soil profiles.
Multi-Channel Analysis of Surface Waves (MASW)	Shear wave velocity analysis to aid in soil/bedrock interface mapping, void detection, site characterization, soil properties, anomaly detection, compaction evaluation, etc. Resolution can be highly variable with depth.

E Appendix E- Typical Drawings for Instrumentation and Monitoring

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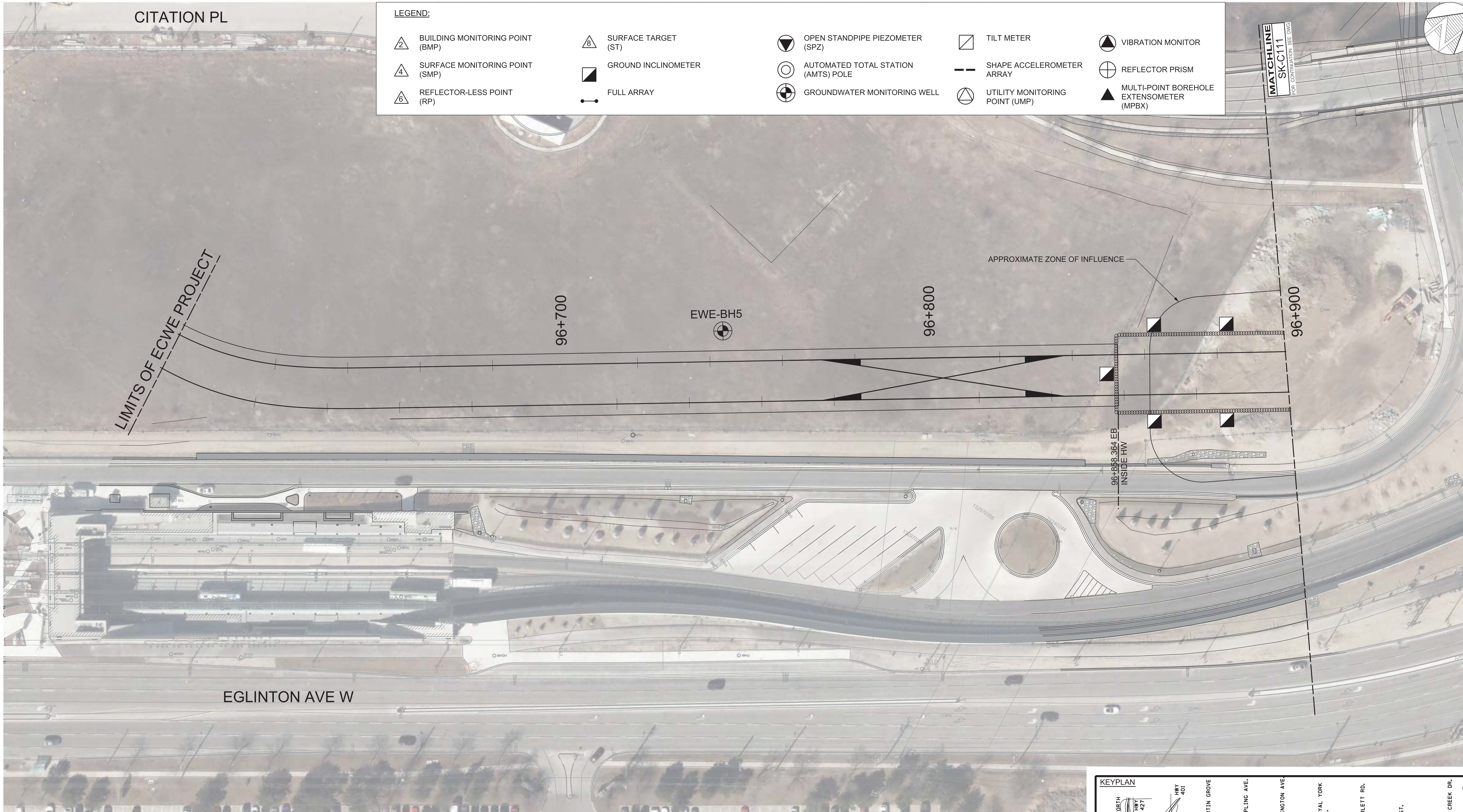
ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019

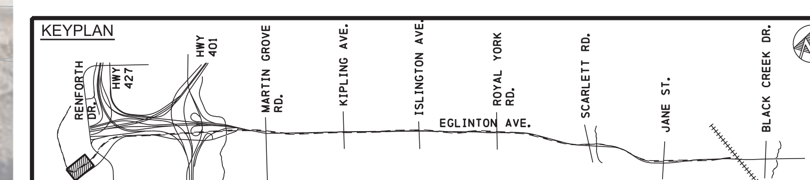


PLAN

STRUCTURE PERFORMANCE MONITORING

1

SK-C110



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4TRANSIT

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DRAWN	K. COFFILL	2020/05/25
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SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 96+600 TO STA. 96+900

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C110 Rev. No. Sheet No.

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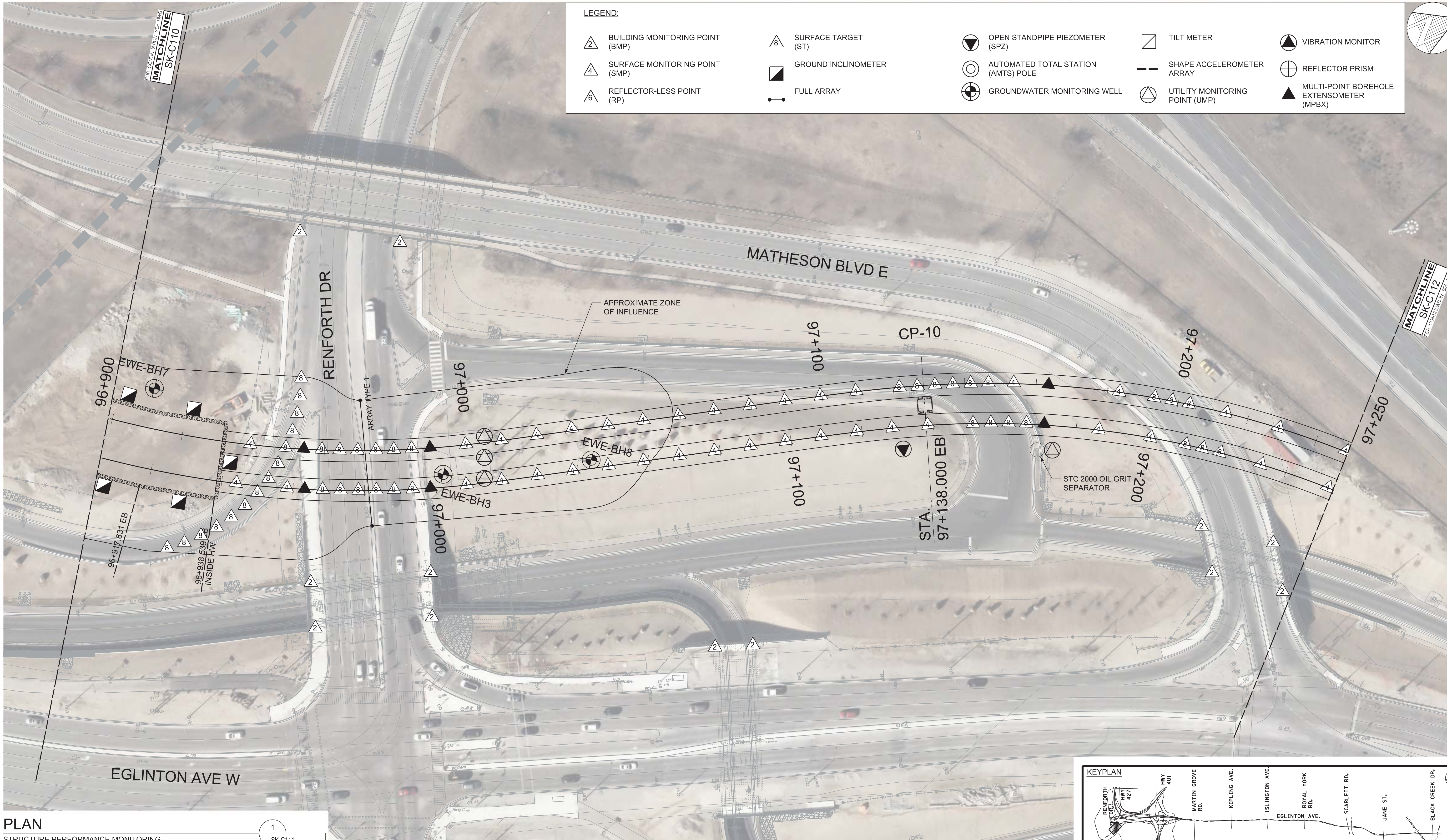

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1 SK-C111

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EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS

GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 96+900 TO STA. 97+250

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. | Rev. No. | Sheet No.

SK-C111

Plot Date: 8/18/2020

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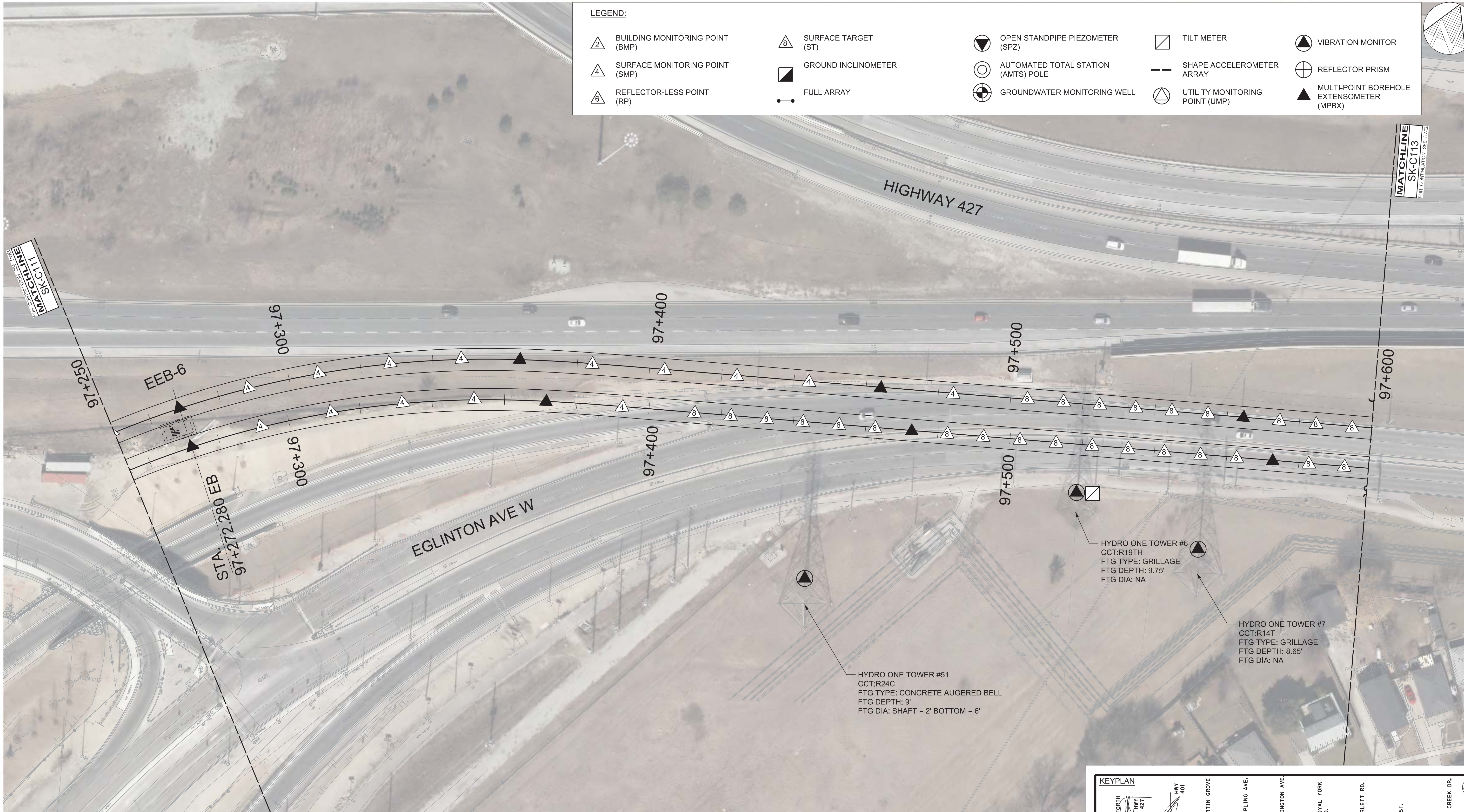

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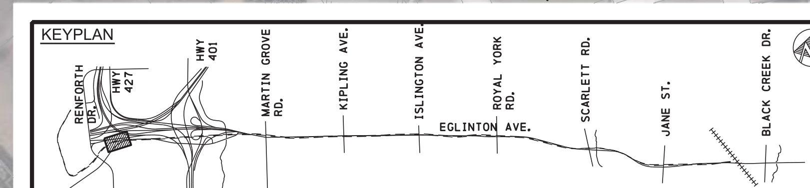
1 SK-C112



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CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS
GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING
STA. 97+250 TO STA. 97+600

Contract No.: TC85-3A Package No.:

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SK-C112

Plot Date: 8/18/2020

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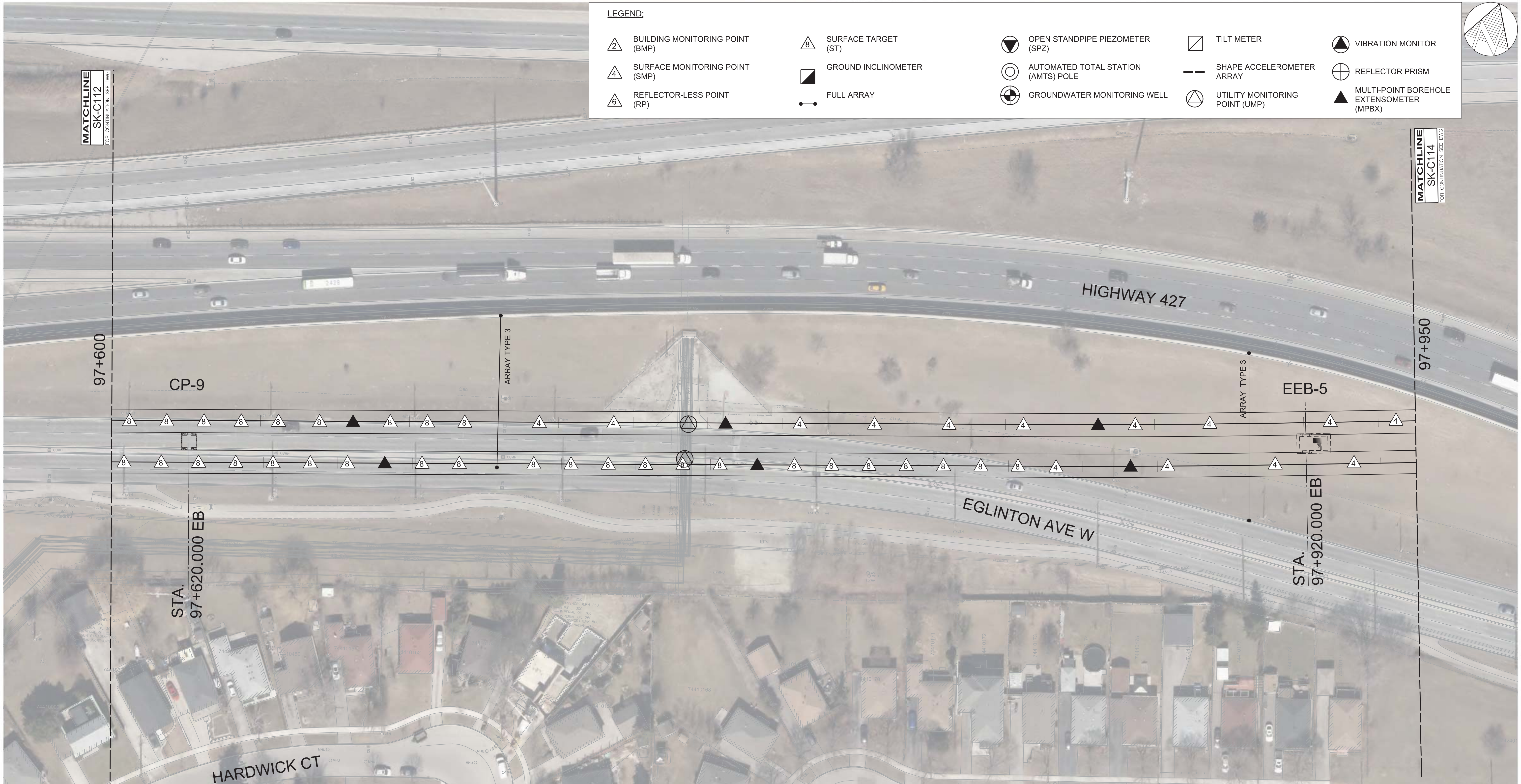

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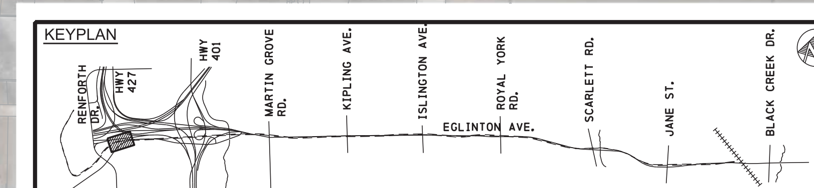
1 SK-C113



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CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS

GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 97+600 TO STA. 97+950

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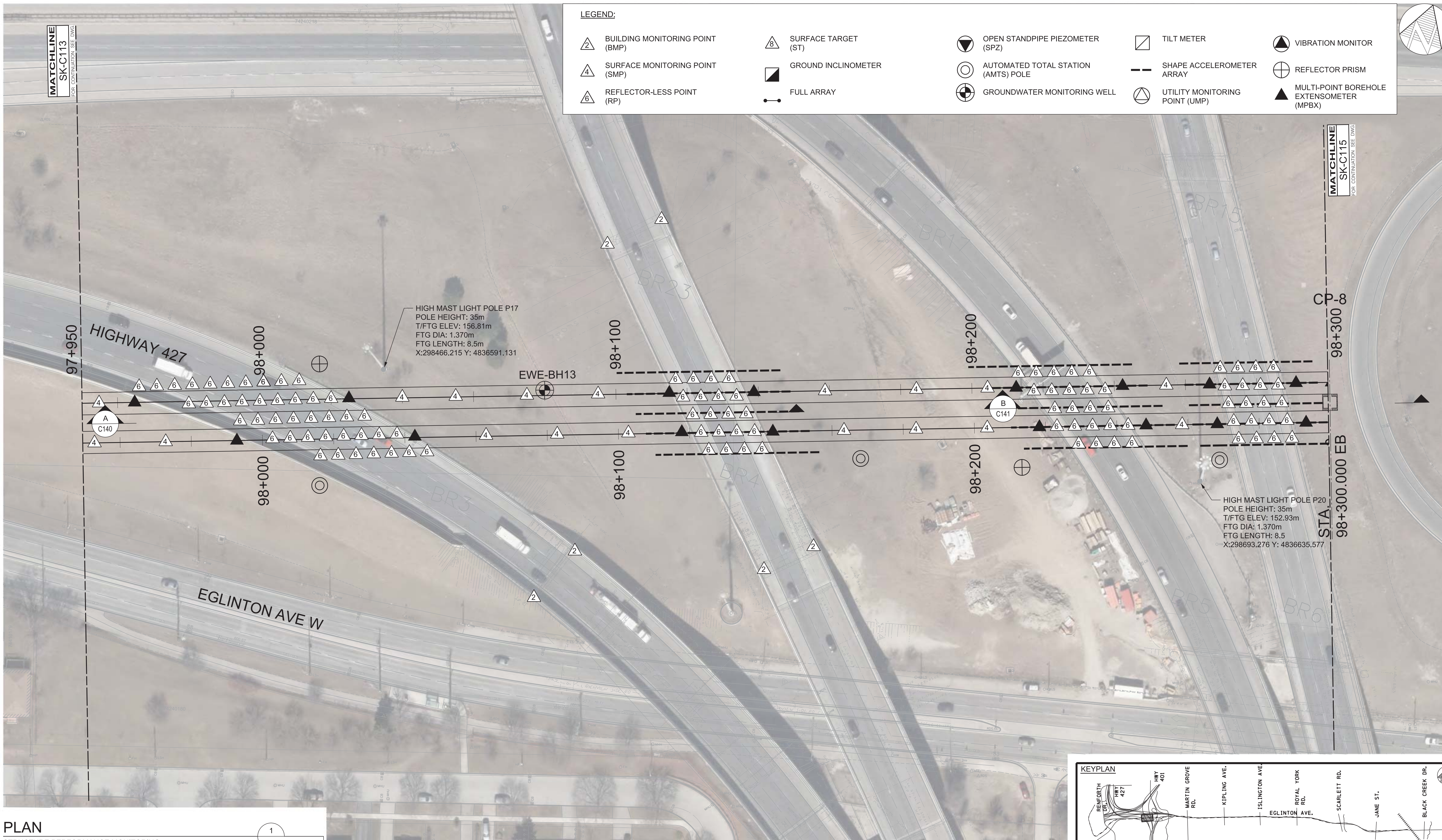

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STRUCTURE PERFORMANCE MONITORING

1 SK-C114

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CHECKED	R. SHOBAYRY	2020/08/18
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SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING STA. 97+950 TO STA. 98+300

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C114 Rev. No. Sheet No.

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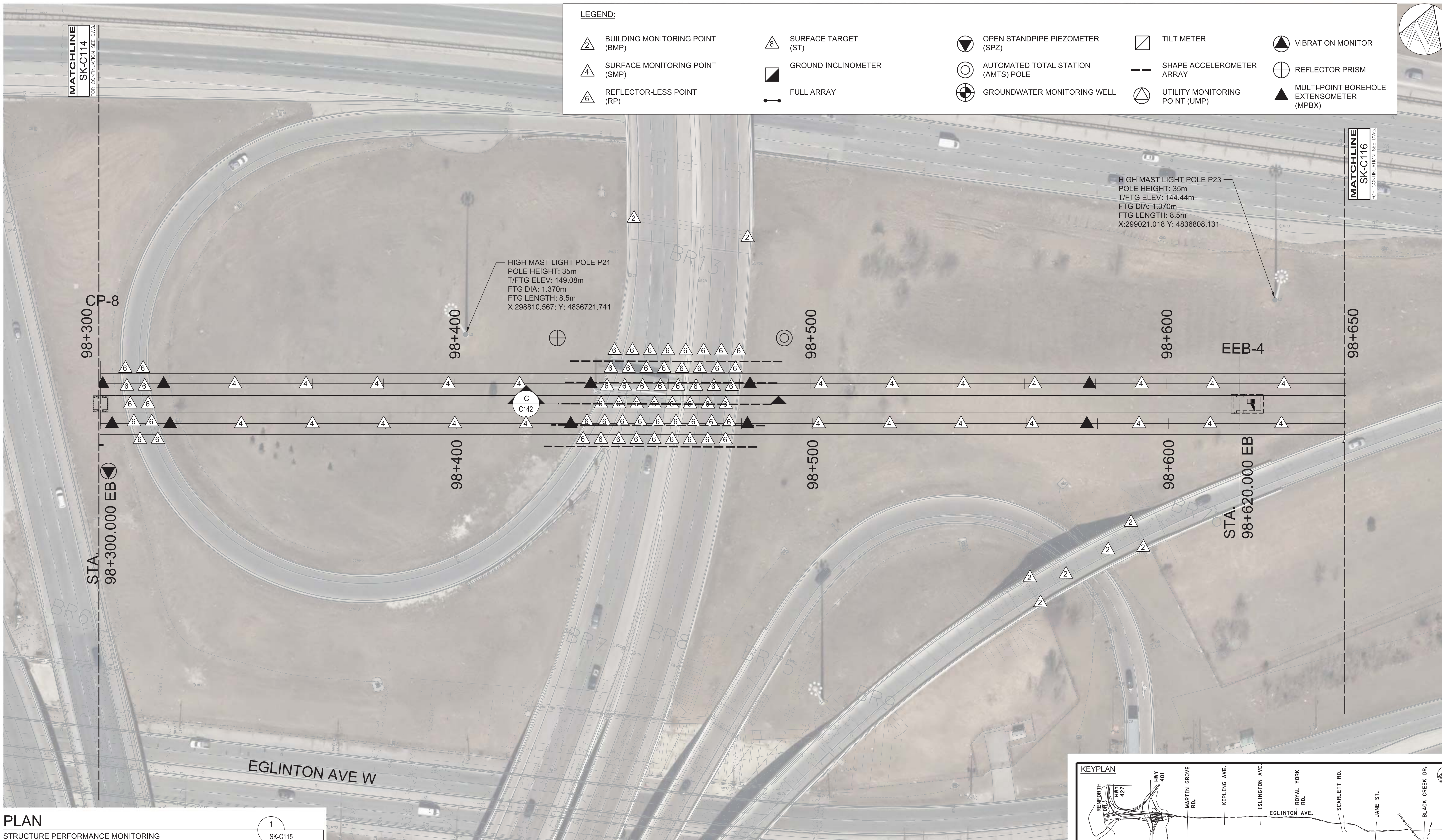

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STRUCTURE PERFORMANCE MONITORING

1 SK-C115

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CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 98+300 TO STA. 98+650

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C115 Rev. No. Sheet No.

Plot Date: 8/18/2020

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ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019

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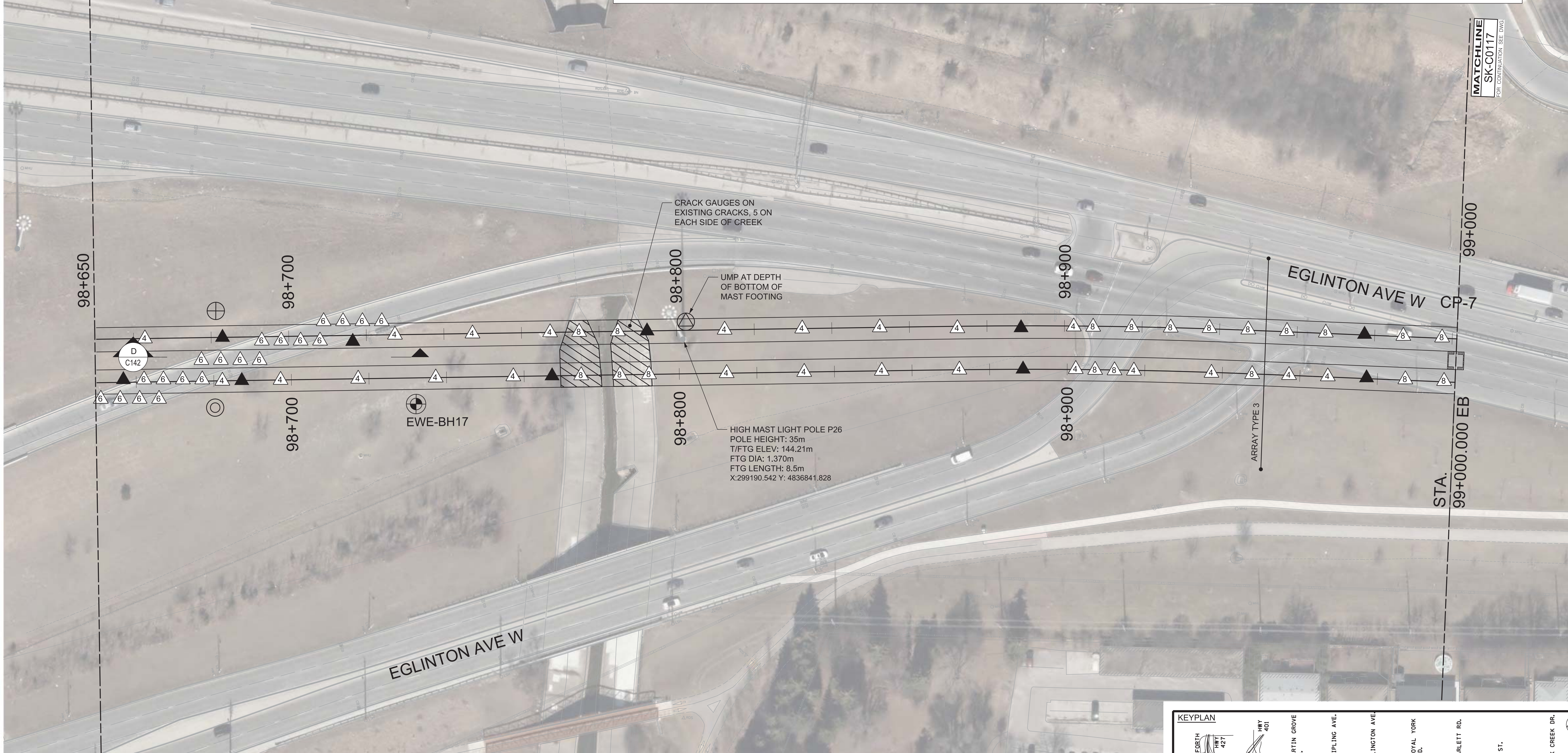
4	SURFACE MONITORING POINT (SMP)		GROUND INCLINOMETER
6	REFLECTOR-LESS POINT (RP)		FULL ARRAY
	OPEN STANDPIPE PIEZOMETER (SPZ)		TILT METER
	AUTOMATED TOTAL STATION (AMTS) POLE		SHAPE ACCELEROMETER ARRAY
	GROUNDWATER MONITORING WELL		UTILITY MONITORING POINT (UMP)
	VIBRATION MONITOR		REFLECTOR PRISM
	MULTI-POINT BOREHOLE EXTENSOMETER (MPBX)		



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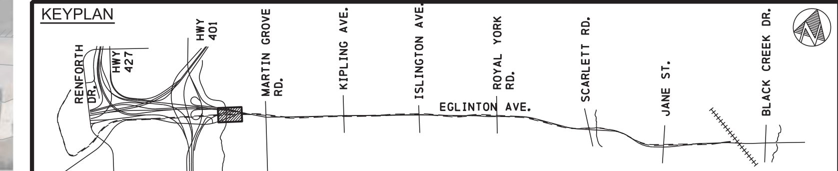
STRUCTURE PERFORMANCE MONITORING

1
SK-C116

KEYPLAN



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SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS
GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING
STA. 98+650 TO STA. 99+000

Contract No.: TC85-3A Package No.:

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Dwg. No. Rev. No. Sheet No.
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Plot Date: 8/18/2020

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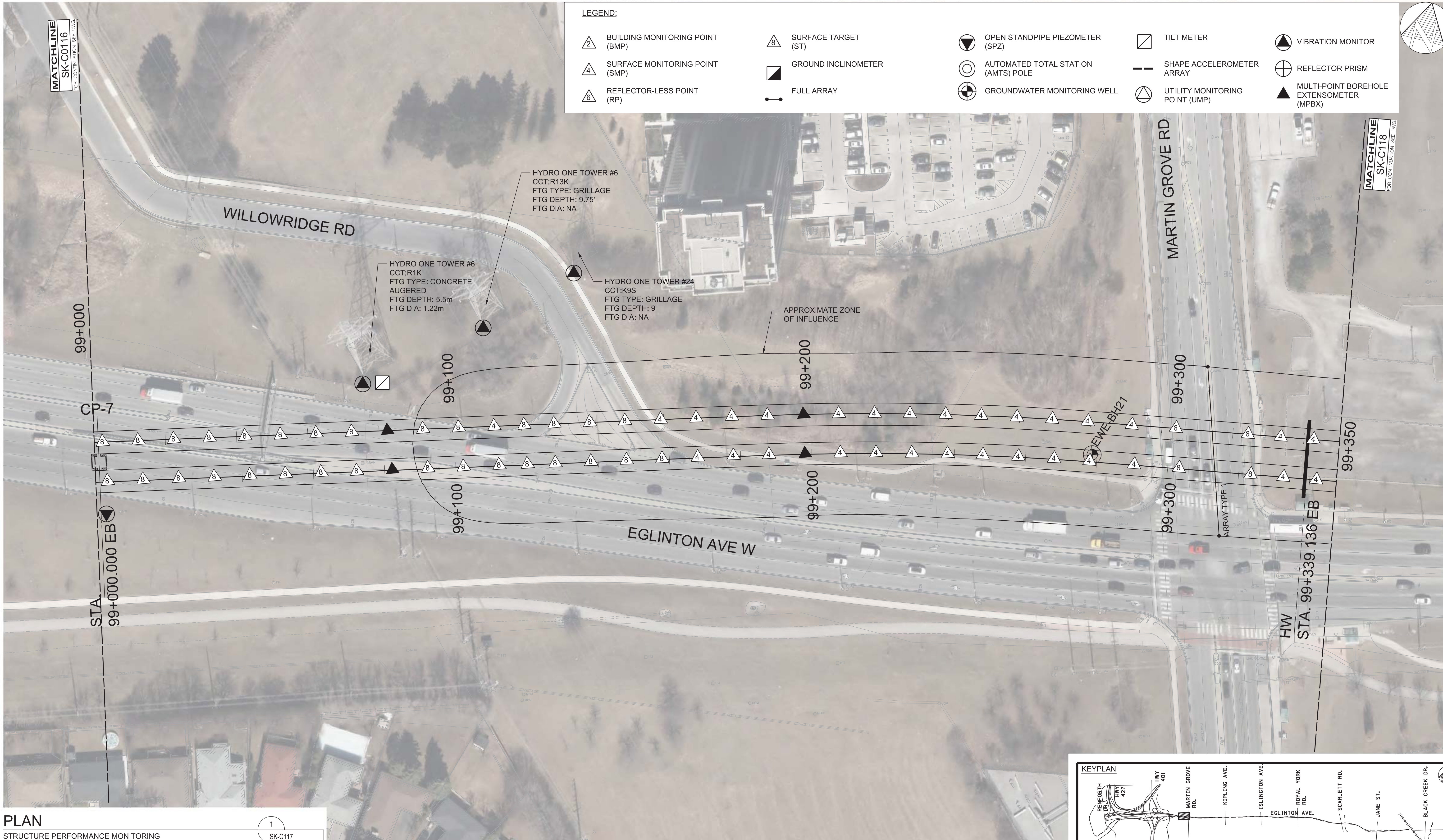

PLAN
STRUCTURE PERFORMANCE MONITORING
1 SK-C117

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CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION
TWIN TUNNELS
GEOTECHNICAL
STRUCTURE PERFORMANCE MONITORING
STA. 99+000 TO STA. 99+350

Contract No.:TC85-3A Package No.:
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Dwg. No. SK-C117 Rev. No. Sheet No.
Plot Date: 8/18/2020

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PLAN

STRUCTURE PERFORMANCE MONITORING

1 SK-C118

REVISIONS	REVISIONS

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4TRANSIT

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CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS

GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 99+350 TO STA. 99+700

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C118 Rev. No. Sheet No.

Plot Date: 8/18/2020

ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019

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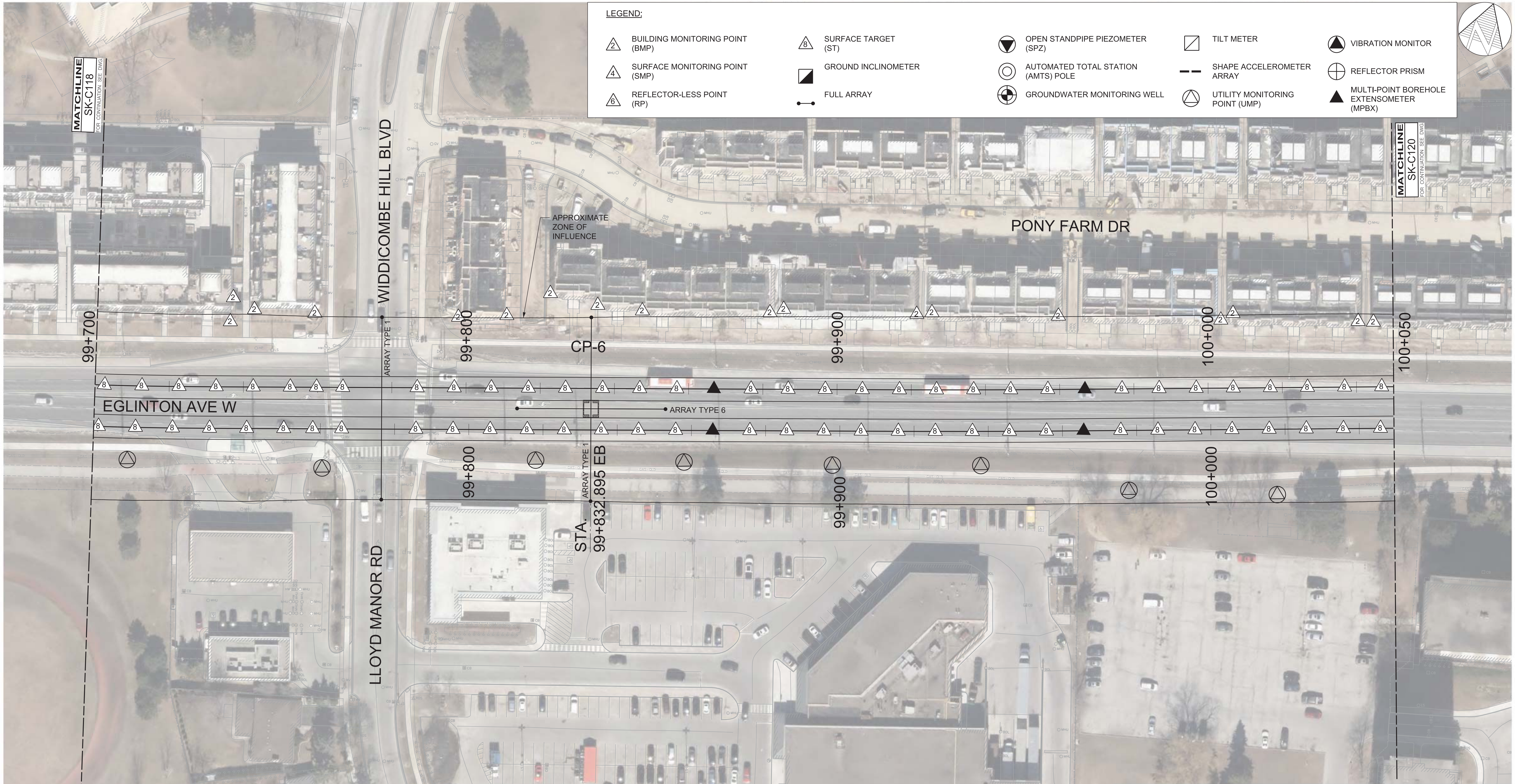

PLAN

STRUCTURE PERFORMANCE MONITORING

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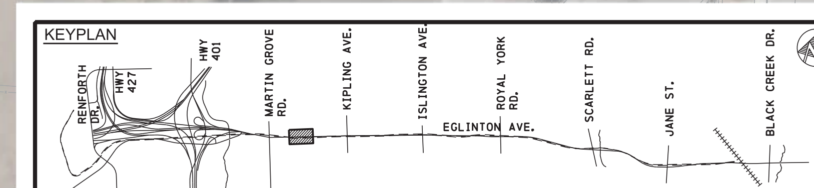
SK-C119



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DRAWING No. SK-C119

REVISIONS	REVISIONS

CONCEPTUAL DESIGN

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4TRANSIT

PARSONS HATCH WSP

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CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING STA. 99+700 TO STA. 100+050

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C119 Rev. No. Sheet No.

Plot Date: 8/18/2020

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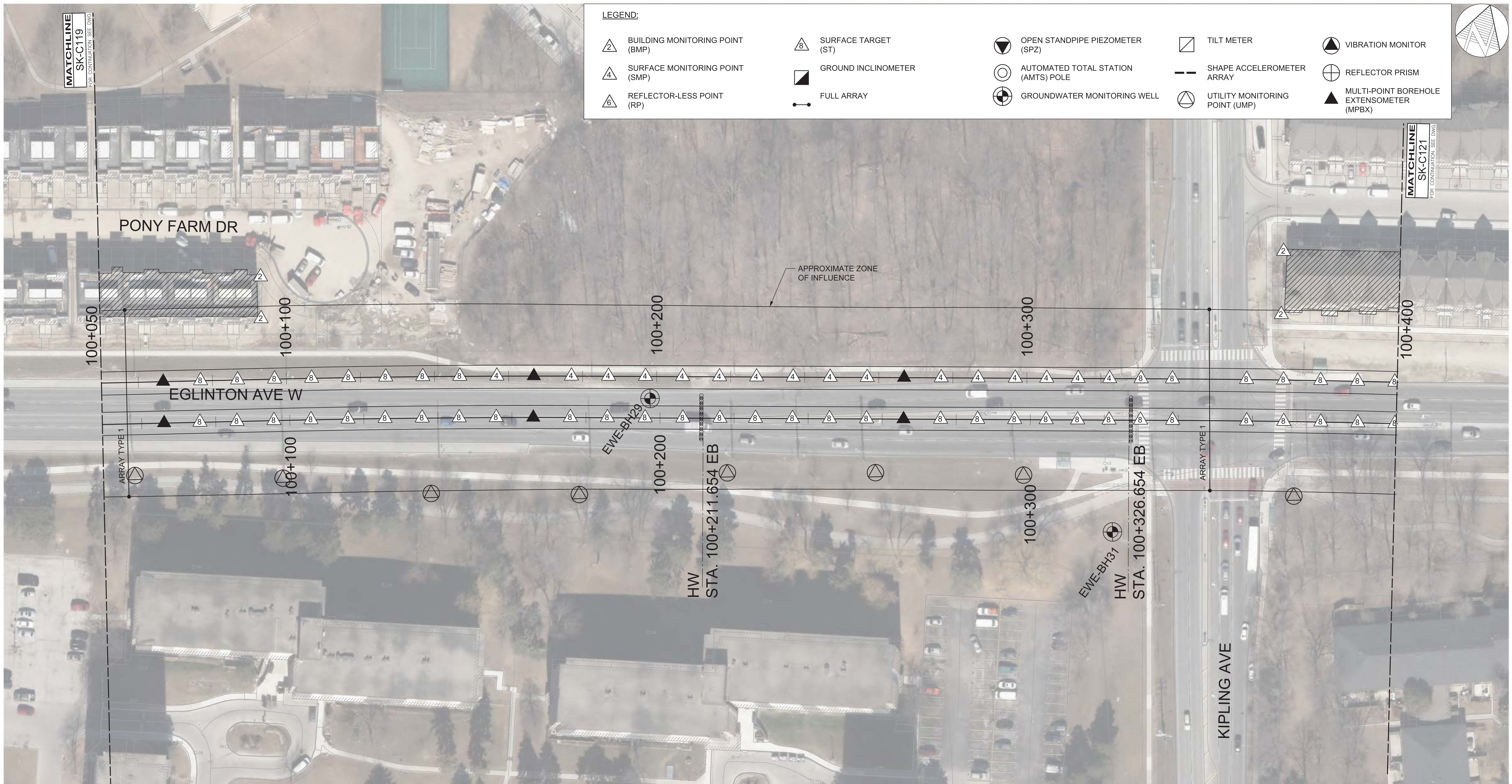

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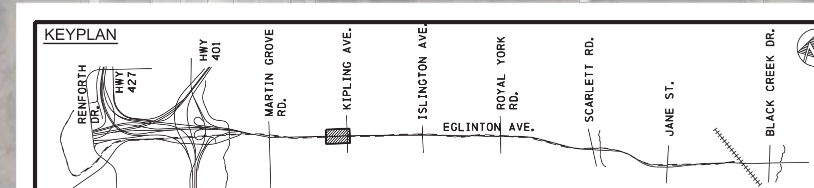
STRUCTURE PERFORMANCE MONITORING

1 SK-C120



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PARSONS HATCH WSP

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SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS

GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 100+050 TO STA. 100+400

Contract No.: TC85–3A Package No.:

METROLINX

Dwg. No. Rev. No. Sheet No.

SK–C120

Plot Date: 8/18/2020

ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

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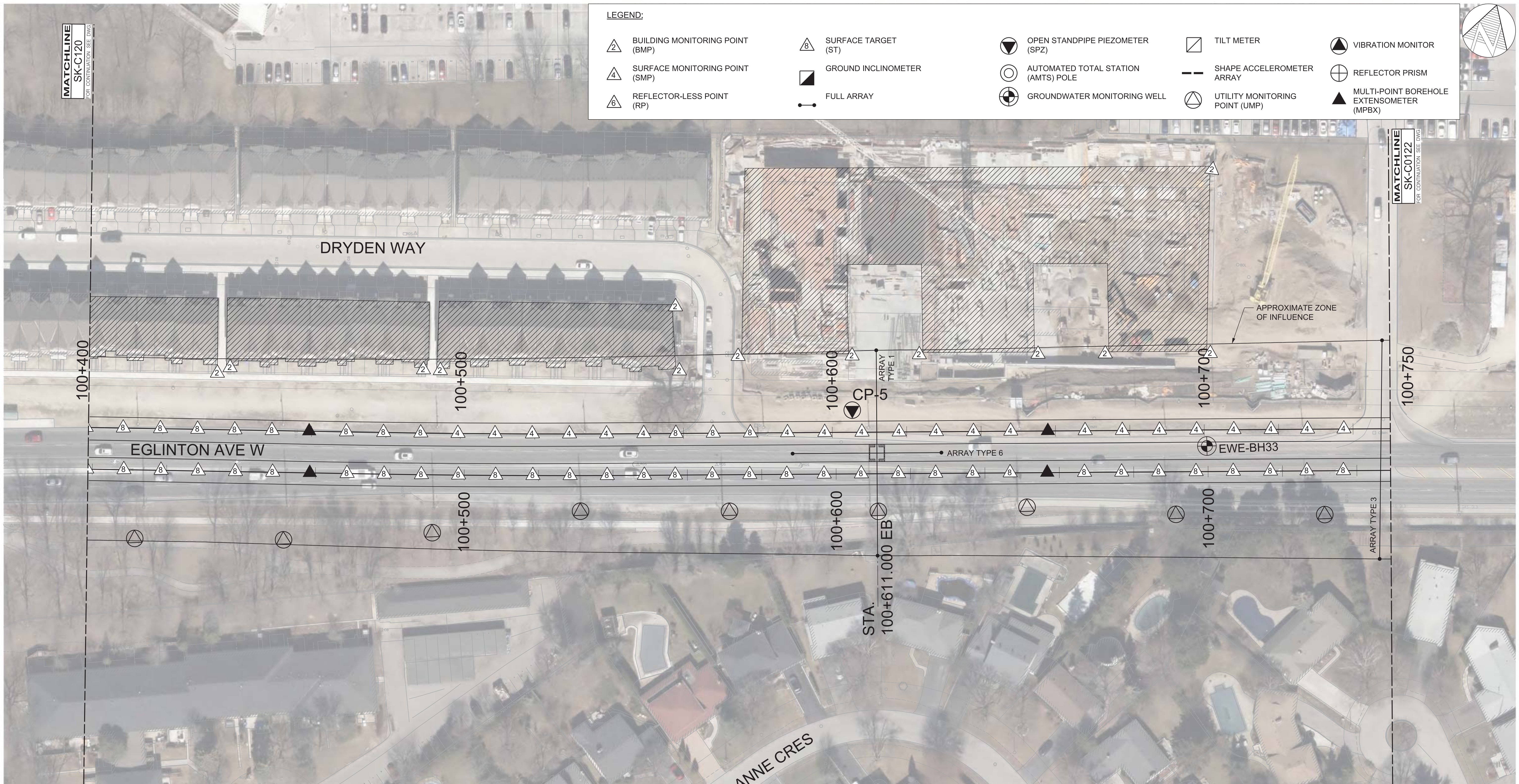
ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

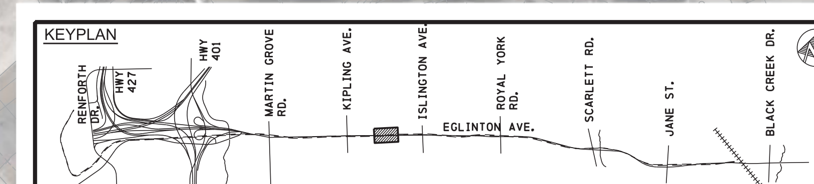


PLAN

STRUCTURE PERFORMANCE MONITORING

1

SK-C121



BLDG. REF. No.

FILENAME: hina_7k_pln_geo_121.dgn

SHEET No.

DRAWING No. SK-C121

REVISIONS	REVISIONS

CONCEPTUAL DESIGN

NOT FOR CONSTRUCTION

AUGUST 20, 2020

4TRANSIT

PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/27
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS

GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 100+400 TO STA. 100+750

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C121 Rev. No. Sheet No.

Plot Date: 8/18/2020

CADD FILE NAME: hina_7k_pln_geo_122.dgn

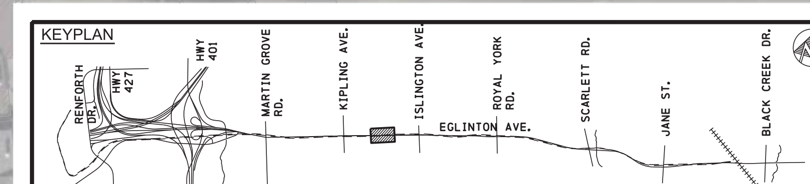
ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

PLAN

STRUCTURE PERFORMANCE MONITORING

1

SK-C122



BLDG. REF. No.

FILENAME: hina_7k_pln_geo_122.dgn

SHEET No.

DRAWING No. SK-C122

REVISIONS	REVISIONS

CONCEPTUAL DESIGN

NOT FOR CONSTRUCTION

AUGUST 20, 2020

4TRANSIT

PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/27
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS
GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING
STA. 100+750 TO STA. 101+100

Contract No.: TC85-3A Package No.:

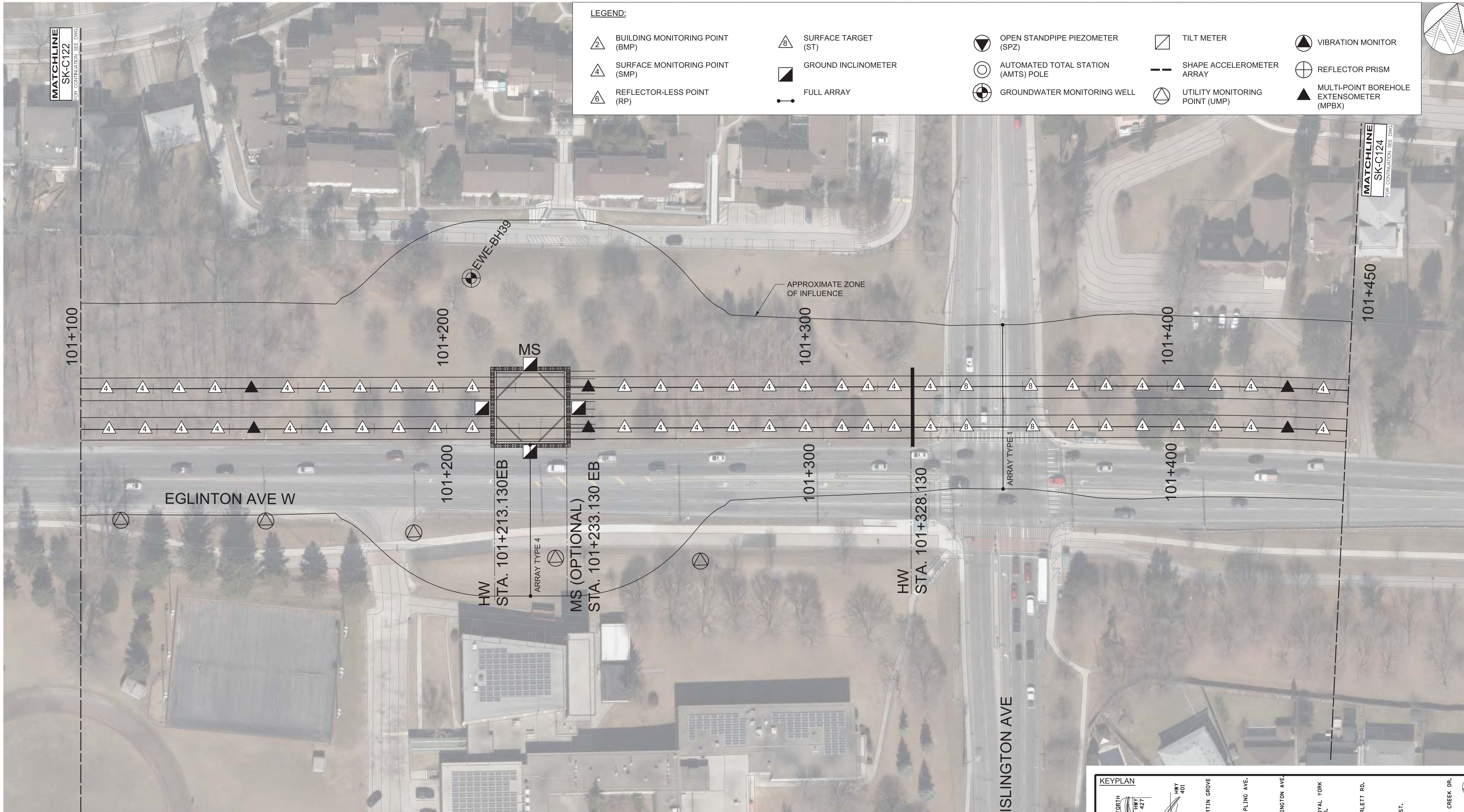
METROLINX

Dwg. No. Rev. No. Sheet No.

SK-C122

Plot Date: 8/18/2020

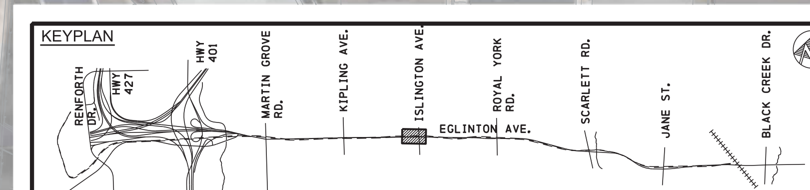
CADD FILE NAME: hina_7k_pln_geo_123.dgn



PLAN

STRUCTURE PERFORMANCE MONITORING

1
SK-C123



REVISIONS	REVISIONS

CONCEPTUAL DESIGN

NOT FOR CONSTRUCTION

AUGUST 20, 2020

4 TRANSIT

PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/27
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS

GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 101+100 TO STA. 101+450

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C123 Rev. No. Sheet No.

Plot Date: 8/18/2020

ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019

BLDG. REF. No.

FILENAME: hina_7k_pln_geo_123.dgn

SHEET No.

DRAWING No. SK-C123

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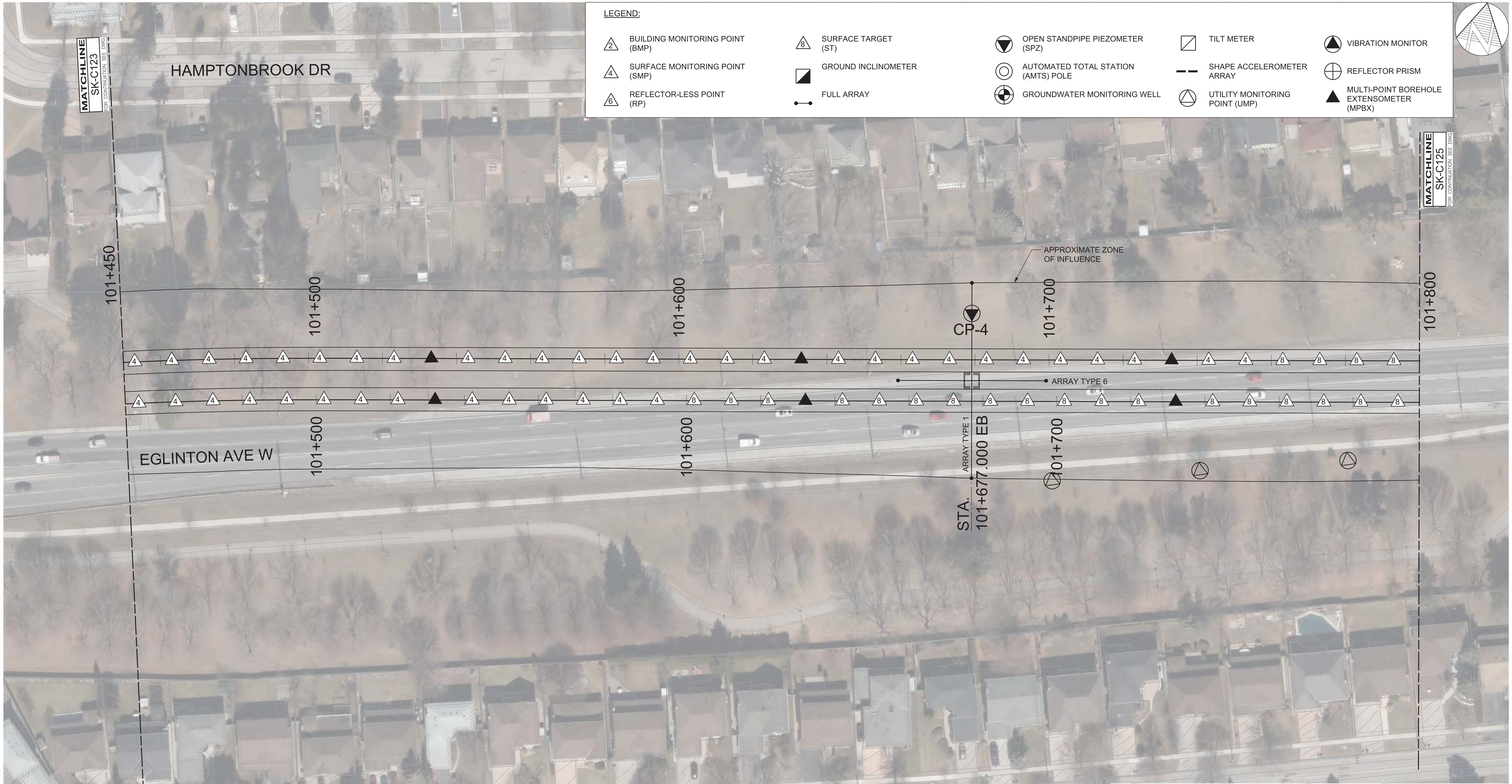
ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019



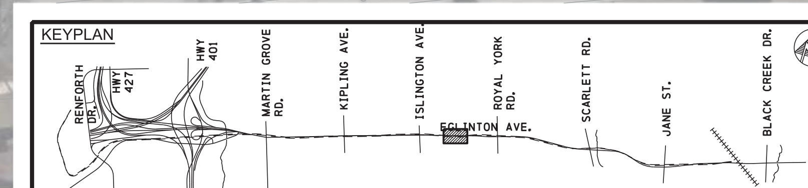
PLAN

STRUCTURE PERFORMANCE MONITORING

1

SK-C124

KEYPLAN



REVISIONS	REVISIONS

CONCEPTUAL DESIGN

NOT FOR CONSTRUCTION

AUGUST 20, 2020

4TRANSIT

PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/27
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 101+450 TO STA. 101+800

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C124 Rev. No. Sheet No.

Plot Date: 8/18/2020

BLDG. REF. No.

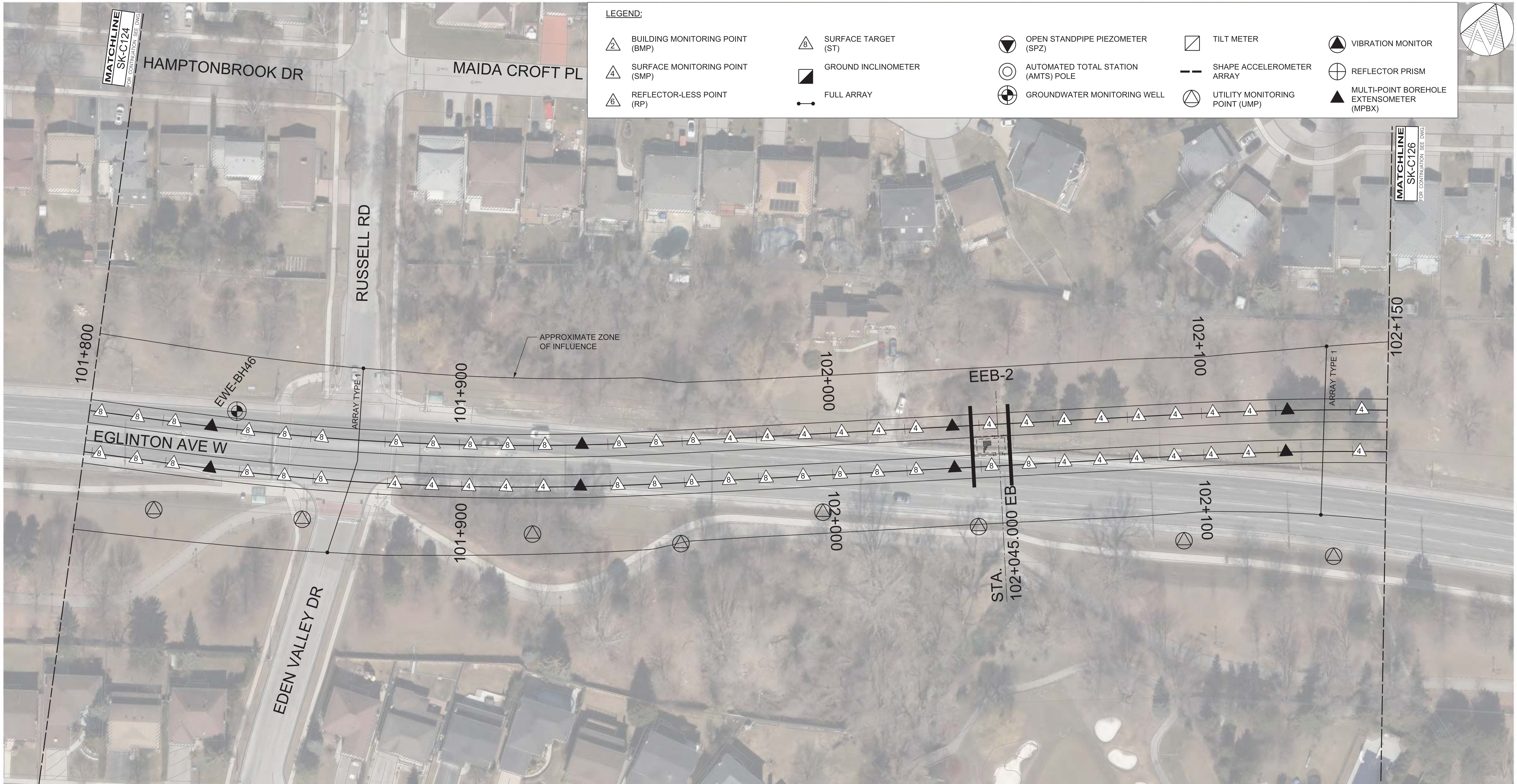
FILENAME: hina_7k_pln_geo_124.dgn

SHEET No.

DRAWING No. SK-C124

CADD FILE NAME: hina_7k_pln_geo_125.dgn

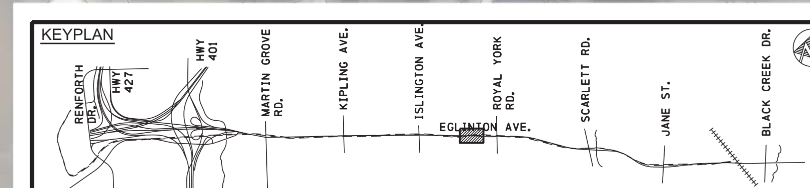
ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019



PLAN

STRUCTURE PERFORMANCE MONITORING

1
SK-C125



REVISIONS	REVISIONS

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION
AUGUST 20, 2020

4TRANSIT
PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS
GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING
STA. 101+800 TO STA. 102+150

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C125 Rev. No. Sheet No.

Plot Date: 8/18/2020

BLDG. REF. No.

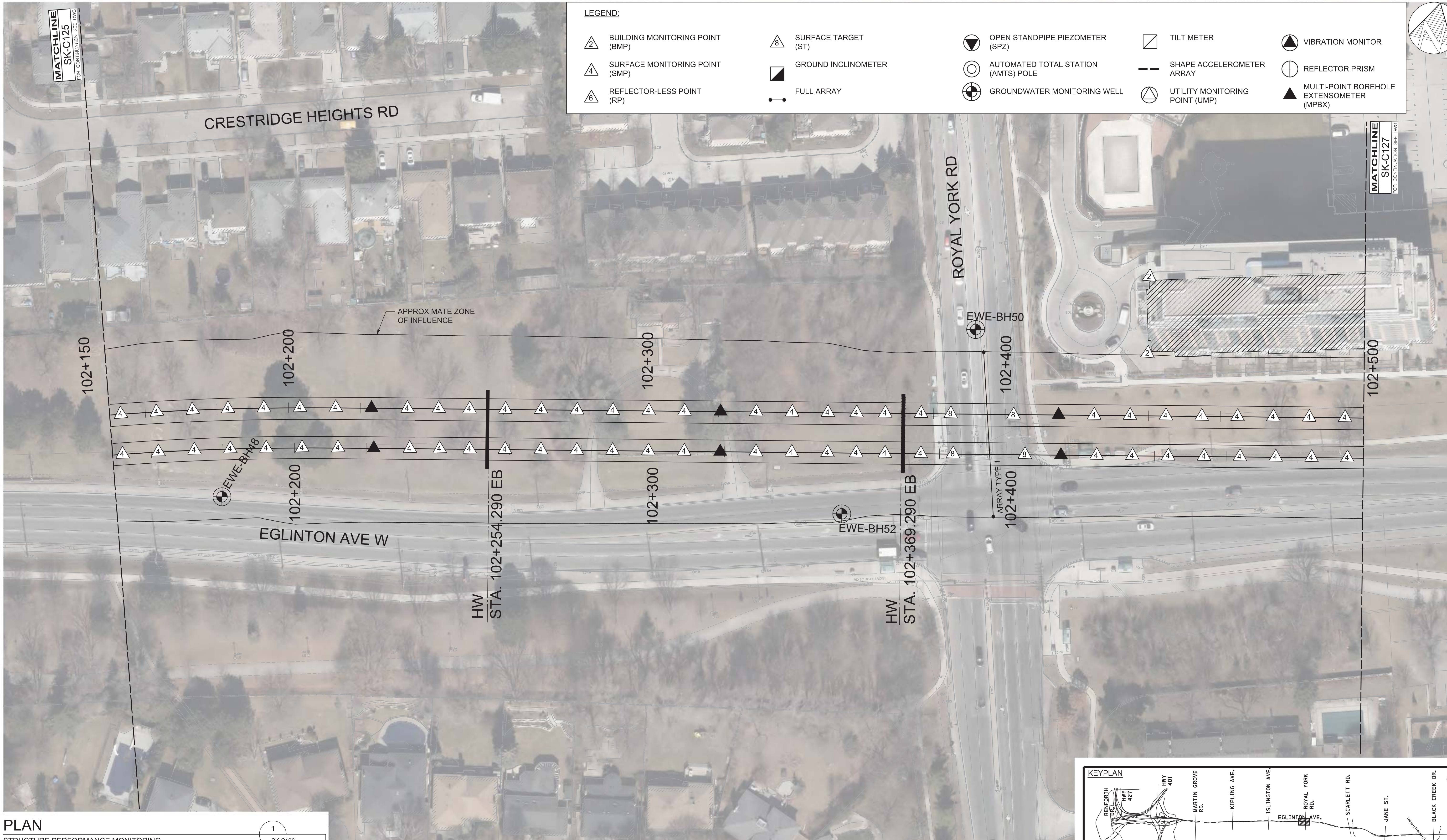
FILENAME: hina_7k_pln_geo_125.dgn

SHEET No.

DRAWING No. SK-C125

CADD FILE NAME: hina_7k_pln_geo_126.dgn

ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019



LEGEND:

- BUILDING MONITORING POINT (BMP)
- SURFACE MONITORING POINT (SMP)
- REFLECTOR-LESS POINT (RP)
- SURFACE TARGET (ST)
- GROUND INCLINOMETER
- FULL ARRAY
- OPEN STANDPIPE PIEZOMETER (SPZ)
- AUTOMATED TOTAL STATION (AMTS) POLE
- GROUNDWATER MONITORING WELL
- TILT METER
- SHAPE ACCELEROMETER ARRAY
- UTILITY MONITORING POINT (UMP)
- VIBRATION MONITOR
- REFLECTOR PRISM
- MULTI-POINT BOREHOLE EXTENSOMETER (MPBX)

PLAN

STRUCTURE PERFORMANCE MONITORING

1 SK-C126

KEYPLAN

REVISIONS	REVISIONS

CONCEPTUAL DESIGN

NOT FOR CONSTRUCTION

AUGUST 20, 2020

4TRANSIT

PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 102+150 TO STA. 102+500

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C126 Rev. No. Sheet No.

Plot Date: 8/18/2020

BLDG. REF. No.

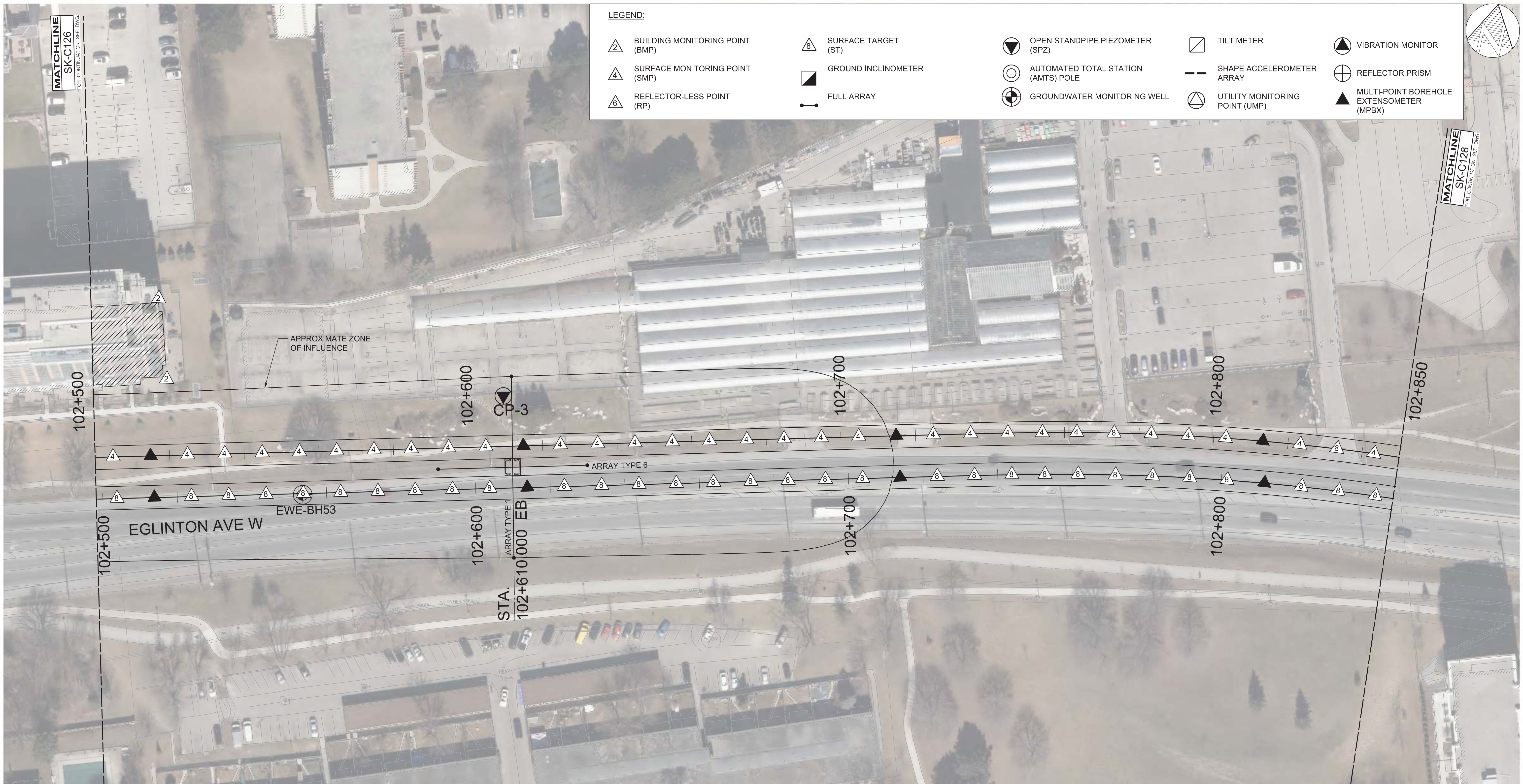
FILENAME: hina_7k_pln_geo_126.dgn

SHEET No.

DRAWING No. SK-C126

CADD FILE NAME: hina_7k_pln_geo_127.dgn

ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

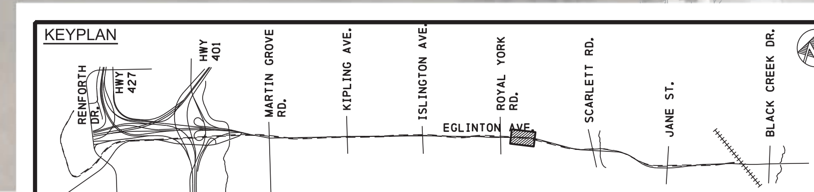


PLAN

STRUCTURE PERFORMANCE MONITORING

1

SK-C127



REVISIONS	REVISIONS

CONCEPTUAL DESIGN

NOT FOR CONSTRUCTION

AUGUST 20, 2020

4TRANSIT

PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS

GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 102 + 500 TO STA. 102 + 850

Contract No.: TC85–3A Package No.:

METROLINX

Dwg. No. SK–C127 Rev. No. Sheet No.

Plot Date: 8/18/2020

BLDG. REF. No.

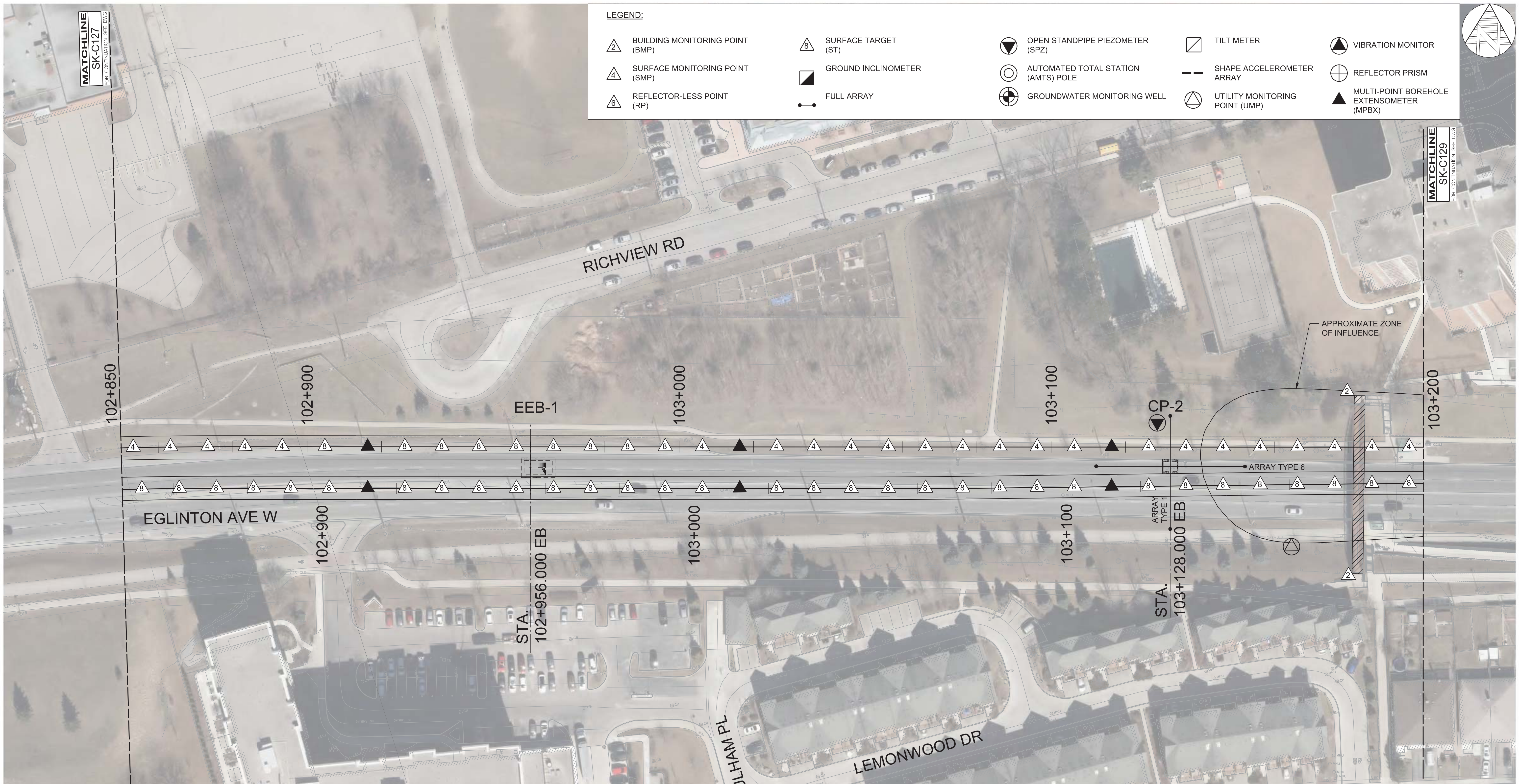
FILENAME: hina_7k_pln_geo_127.dgn

SHEET No.

DRAWING No. SK-C127

CADD FILE NAME: hina_7k_pln_geo_128.dgn

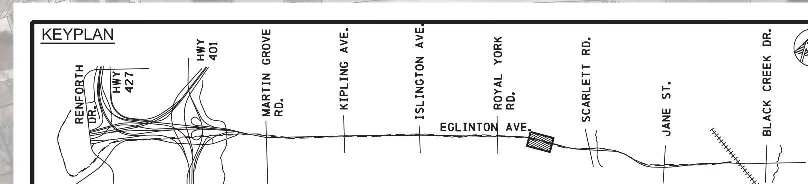
ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019



PLAN

STRUCTURE PERFORMANCE MONITORING

1
SK-C128



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FILENAME: hina_7k_pln_geo_128.dgn

SHEET No.

DRAWING No. SK-C128

REVISIONS	REVISIONS

CONCEPTUAL DESIGN

NOT FOR CONSTRUCTION

AUGUST 20, 2020

4TRANSIT

PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING
STA. 102+850 TO STA. 103+200

Contract No.: TC85-3A Package No.:

METROLINX

Dwg. No. SK-C128 Rev. No. Sheet No.

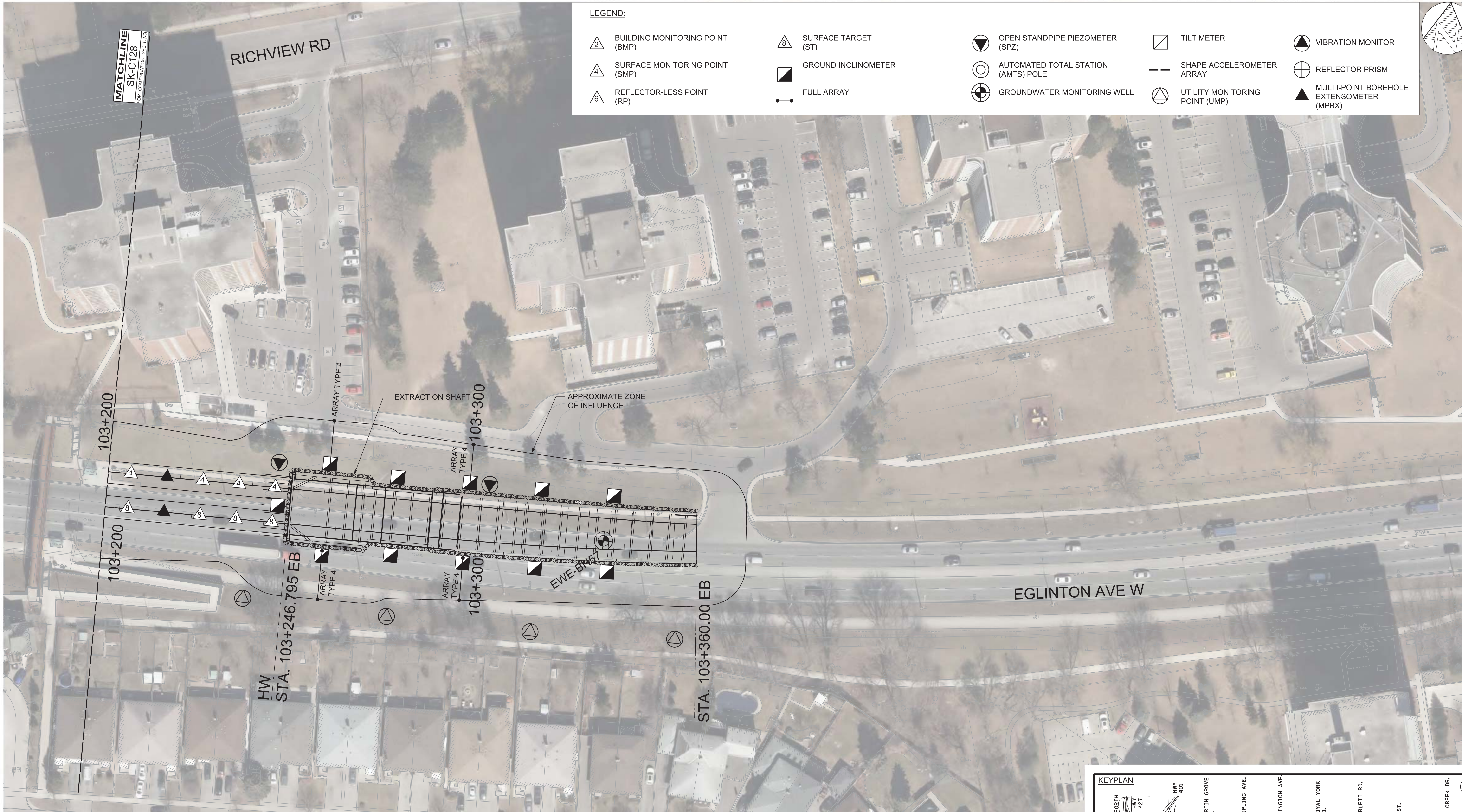
Plot Date: 8/18/2020

CADD FILE NAME: hina_7k_pln_geo_129.dgn

ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019

LEGEND:

- BUILDING MONITORING POINT (BMP)
- SURFACE MONITORING POINT (SMP)
- REFLECTOR-LESS POINT (RP)
- SURFACE TARGET (ST)
- GROUND INCLINOMETER
- FULL ARRAY
- OPEN STANDPIPE PIEZOMETER (SPZ)
- AUTOMATED TOTAL STATION (AMTS) POLE
- GROUNDWATER MONITORING WELL
- TILT METER
- SHAPE ACCELEROMETER ARRAY
- UTILITY MONITORING POINT (UMP)
- VIBRATION MONITOR
- REFLECTOR PRISM
- MULTI-POINT BOREHOLE EXTENSOMETER (MPBX)



PLAN

STRUCTURE PERFORMANCE MONITORING

1 SK-C129

KEYPLAN

BLDG. REF. No.

FILENAME: hina_7k_pln_geo_129.dgn

SHEET No.

DRAWING No. SK-C129

REVISIONS	REVISIONS

CONCEPTUAL DESIGN

NOT FOR CONSTRUCTION

AUGUST 20, 2020

4TRANSIT

PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10 15 20m

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS

GEOTECHNICAL

STRUCTURE PERFORMANCE MONITORING

STA. 103+200 TO STA. 103+550

Contract No.:TC85-3A Package No.:

METROLINX

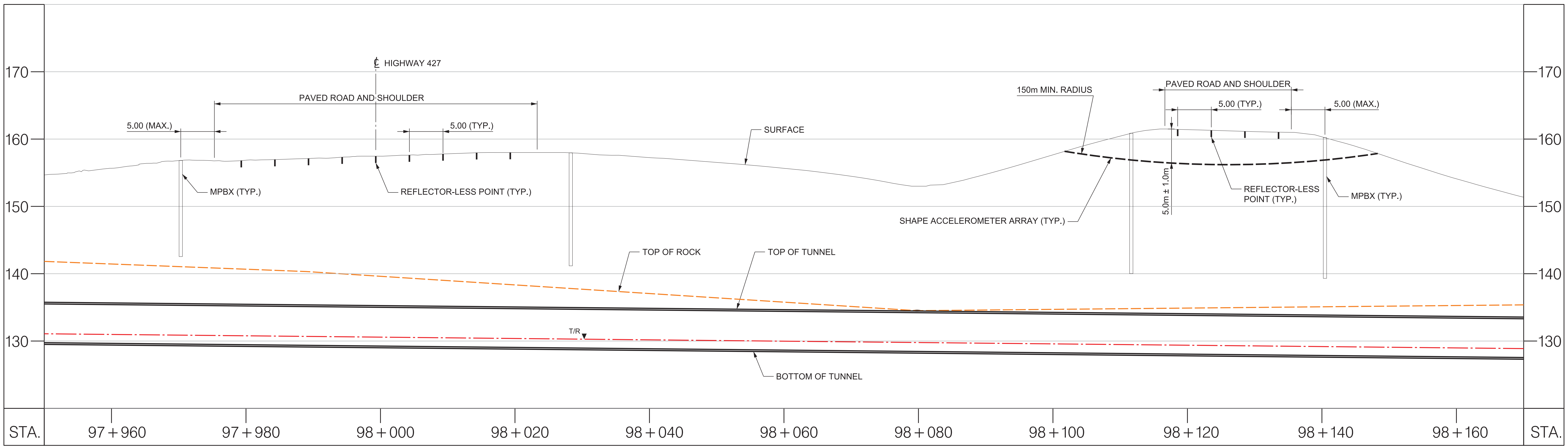
Dwg. No. SK-C129 Rev. No. Sheet No.

Plot Date: 8/18/2020

CADD FILE NAME: hina_7m_sec_geo_140.dgn

ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

NOTE:
1. CHAINAGES ARE BASED ON WESTBOUND TUNNEL.



SECTION A / SK-C114
STA. 97+950

REVISIONS	REVISIONS
△	△
△	△
△	△

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION
AUGUST 20, 2020

4TRANSIT
PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 2 4 6 8 10 12m

EGLINTON CROSSTOWN WEST EXTENSION
TWIN TUNNELS
GEOTECHNICAL
TUNNEL AT HIGHWAY 427
SECTION A

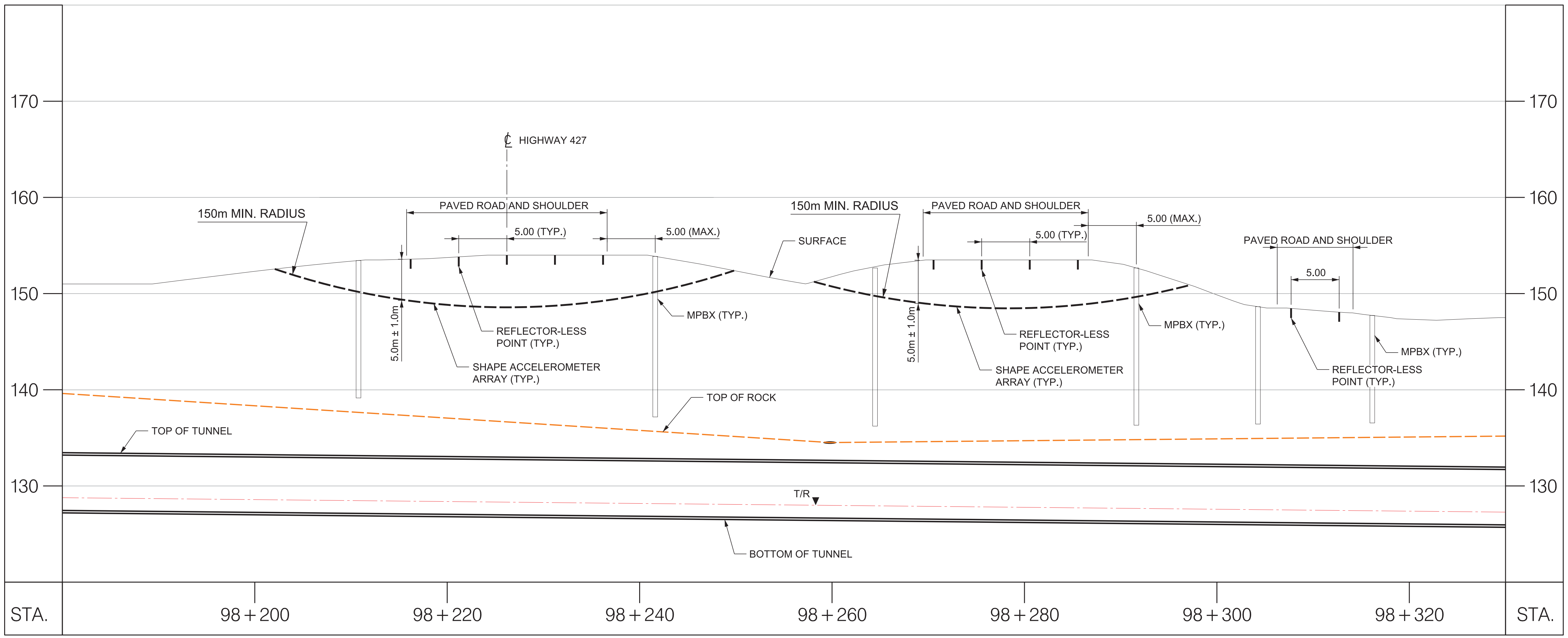
Contract No.: TC85–3A Package No.:
METROLINX
Dwg. No. SK–C140 Rev. No. Sheet No.
Plot Date: 8/18/2020

BLDG. REF. No.
FILENAME: hina_7m_sec_geo_140.dgn
SHEET No.
DRAWING No. SK–C140

CADD FILE NAME: hina_7m_sec_geo_141.dgn

ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

NOTE:
1. CHAINAGES ARE BASED ON WESTBOUND TUNNEL.



SECTION
STA. 98+180
B
SK-C114

BLDG. REF. No.
FILENAME: hina_7m_sec_geo_141.dgn
SHEET No.
DRAWING No. SK-C141

REVISIONS	REVISIONS

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION
AUGUST 20, 2020

4TRANSIT
PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10m

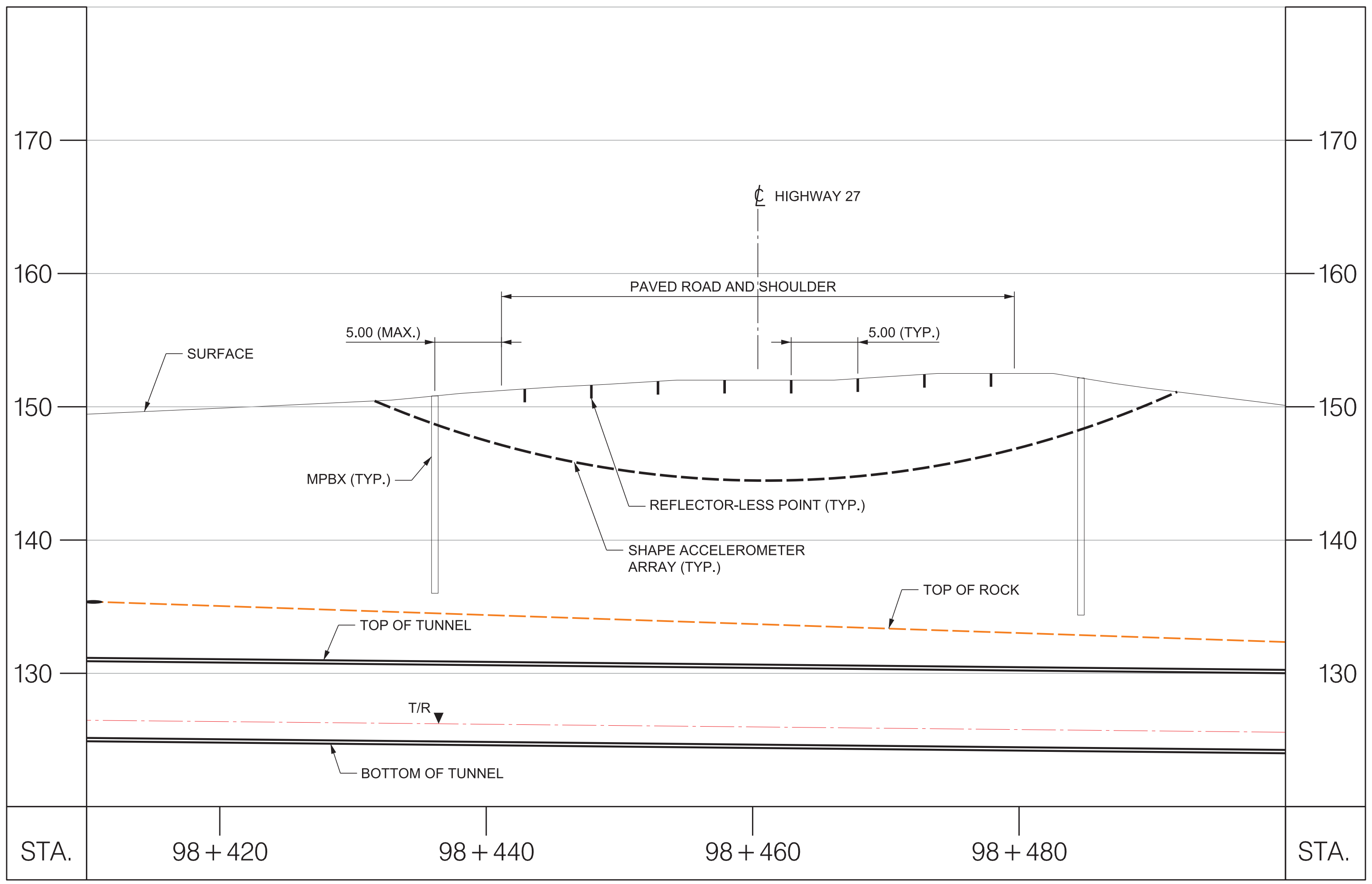
EGLINTON CROSSTOWN WEST EXTENSION
TWIN TUNNELS
GEOTECHNICAL
TUNNEL AT HIGHWAY 427
SECTION B

Contract No.: TC85-3A Package No.:
METROLINX
Dwg. No. SK-C141 Rev. No. Sheet No.
Plot Date: 8/18/2020

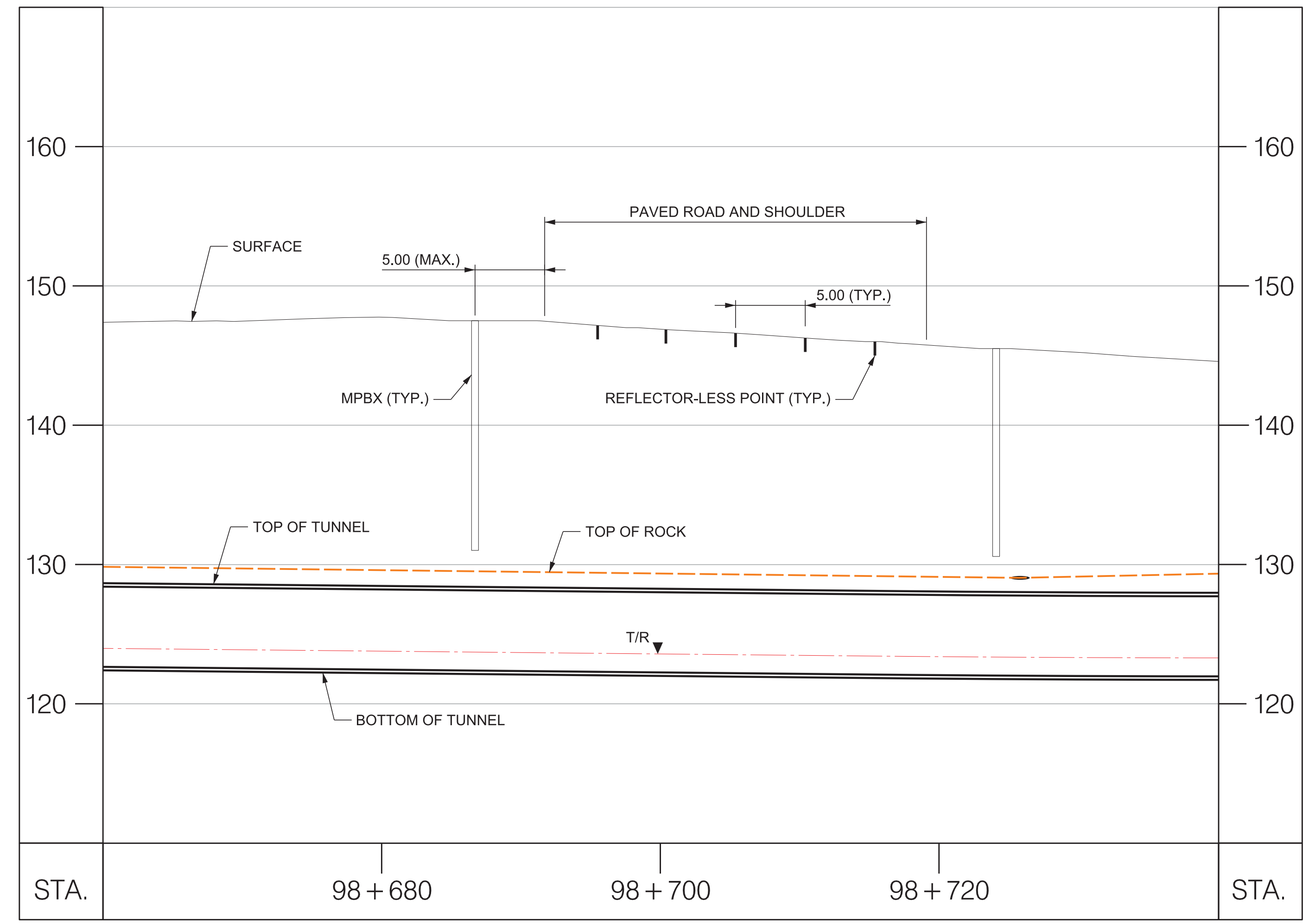
CADD FILE NAME: hina_7m_sec_geo_142.dgn

ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

NOTE:
1. CHAINAGES ARE BASED ON WESTBOUND TUNNEL.



SECTION C SK-C115
STA. 98+410



SECTION D SK-C116
STA. 98+660

REVISIONS	REVISIONS

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION
AUGUST 20, 2020

4TRANSIT
PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 5 10m

EGLINTON CROSSTOWN WEST EXTENSION
TWIN TUNNELS
GEOTECHNICAL
TUNNEL AT HIGHWAY 427
SECTIONS C & D

Contract No.: TC85-3A Package No.:
METROLINX
Dwg. No. SK-C142 Rev. No. Sheet No.
Plot Date: 8/18/2020

BLDG. REF. No.
FILENAME: hina_7m_sec_geo_142.dgn
SHEET No.
DRAWING No. SK-C142

CADD FILE NAME: hina_7k_det_geo_143.dgn

ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

GENERAL NOTES:

1. LOCATIONS, QUANTITY AND TYPES OF INSTRUMENTS SHOWN ON MONITORING DRAWINGS ARE PRESCRIBED WITH THE FOLLOWING EXCEPTIONS:

   - MINOR ADJUSTMENT TO LOCATION OF INSTRUMENTS ARE ALLOWED TO MAKE INSTRUMENTS CLEAR OF EXISTING UTILITIES AND/OR OBSTRUCTIONS.
   - LOCATION OF AUTOMATED TOTAL STATION POLES ARE CONCEPTUAL.

2. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS NOTED OTHERWISE.
3. DIMENSIONS SHOWN FOR LOCATIONS OF THE INSTRUMENTATION ARE APPROXIMATE. ACTUAL LOCATIONS SHALL BE DETERMINED IN THE FIELD FOR PHYSICAL OBSTRUCTIONS AND UTILITY CLEARANCES.
4. FOR THE MONITORING STATIONS AND PIEZOMETER INSTALLATION, REFER TO INSTRUMENTATION PLAN DRAWINGS.
5. GROUND MOVEMENT MAY OCCUR OVER A NARROWER OR WIDER ZONE IN PRACTICE.
6. MPBX ANCHOR POINTS TO BE INSTALLED AT EQUIDISTANT SPACING.
7. INCLINOMETERS AT CUT AND COVER EXCAVATIONS SHALL BE INSTALLED WITHIN THE VERTICAL MEMBER OF THE EXCAVATION SUPPORT SYSTEM.
8. SMP'S IN AN ARRAY TO BE INSTALLED AT EQUIDISTANT SPACING.
9. THE TIP OF THE LOWEST MPBX ANCHOR POINTS SHALL BE 1.0m FROM THE TUNNEL CROWN.
10. IN CASE OF DISCREPANCY IN ELEVATION OF TOP OF COMPETENT ROCK SHOWN ON INSTRUMENTATION AND MONITORING DRAWINGS AND GBR, GBR INFORMATION TAKES PRECEDENCE.
11. MAINTENANCE SHAFT INSTRUMENTATION, THAT INCLUDES FOUR INCLINOMETERS AND ONE ARRAY TYPE 4, ARE REQUIRED ONLY IF THE SHAFT IS PLANNED TO BE CONSTRUCTED.
12. INCLINOMETER FOR SHAFTS TO BE INSTALLED ADJACENT OR WITHIN THE SOE WALL.

SHORING AND HEADWALL PILE MONITORING NOTES:

1. PILING IS REQUIRED TO BE INSTALLED A MINIMUM OF 1.0m CLEAR OF ANY EXISTING CITY INFRASTRUCTURE, UNLESS OTHERWISE NOTED ON THE PLANS.
2. DURING SHORING PILE INSTALLATION ANY UTILITY WITHIN 1.0m OF A PILE WILL BE HYDROVAC LOCATED, VISUALLY INSPECTED AND SURVEYED WITH UTILITY PIPE IDENTIFIED PRIOR TO DRILLING. UTILITY INSPECTOR TO BE CONTACTED PRIOR TO PIPE INSTALLATION. IF THE UTILITY IS A WATERMAIN WITH AN UNRESTRAINED JOINT LOCATED WITHIN THE EXCAVATION, A JOINT RESTRAINT SHALL BE PROVIDED. A STEEL PLATE (DESIGNED BY TUNNEL CO) SHALL BE INSTALLED BESIDE ANY UTILITY WITHIN 3.0m OF A PILE LOCATION PRIOR TO DRILLING.
3. PILES SHALL BE AUGERED, NOT DRIVEN.

SURFACE MONITORING NOTES:

1. GROUND MOVEMENT MAY OCCUR OVER A NARROWER OR WIDER ZONE OF INFLUENCE.
2. WHERE ARRAY IS NOT PERPENDICULAR TO TUNNEL ALIGNMENT SPECIFIED, INSTRUMENT DISTANCE SHALL BE MEASURED PERPENDICULAR TO TUNNEL CENTRELINE.
3. USE REFLECTOR-LESS POINTS FOR SETTLEMENT MONITORING ONLY ON ASPHALT OR CONCRETE PAVED SURFACES OF HIGHWAY 427. USE SURFACE TARGETS ON ASPHALT OR CONCRETE PAVED SURFACES ELSEWHERE. USE SMPS OUTSIDE OF ASPHALT AND CONCRETE PAVED SURFACE.
4. COMPLEMENT REFLECTOR-LESS POINTS WITH REFLECTIVE PRISM ON THE SIDES OF THE ROAD.
5. DETERMINE LOCATIONS AND HEIGHT OF POLE SO THAT AMTS ARE ABLE TO READ ALL PLANNED REFLECTOR-LESS POINTS WITH THE SPECIFIED ACCURACY. WHERE POSSIBLE, AUTOMATED TOTAL STATION (AMTS) POLES TO BE INSTALLED OUTSIDE ZONE OF INFLUENCE (ZOI).
6. ANY SURFACE MONITORING POINT (SMP) CAN BE REPLACED BY A SINGLE POINT BOREHOLE EXTENSOMETER TO ALLOW AUTOMATIC MONITORING.
7. INSTALL SHAPE ACCELEROMETER ARRAYS USING DIRECTIONAL DRILLING TECHNIQUE AT A SAFE DEPTH TO AVOID DAMAGES TO MTO PAVEMENT AND INFRASTRUCTURE.
8. MANUFACTURER REPRESENTATIVE TO BE PRESENT ON-SITE AT ALL TIMES DURING INSTALLATION OF THE SHAPE ACCELEROMETER ARRAYS AND PROVIDE BENCHMARKED LONG-TERM MEASUREMENT DATA AND CERTIFICATION FOR EACH SUPPLIED INSTRUMENT TO DEMONSTRATE THAT INSTRUMENT DRIFT WILL SATISFY PROJECT TOLERANCES.

GROUNDWATER MONITORING NOTES:

1. VOLUME AND FLOW RATE OF LOW-FLOW DEWATERING MONITORED DAILY.
2. GROUNDWATER LEVELS SHALL BE MONITORED WEEKLY AFTER INSTALLATION UNTIL A MINIMUM OF 4 STABLE READING ARE RECORDED PRIOR TO START OF CONSTRUCTION, EXCAVATION OR DEWATERING.
3. GROUNDWATER LEVEL ONCE A WEEK FOR THREE WEEKS PRIOR TO PUMPING.
4. GROUNDWATER LEVEL ONCE A DAY FOR FIRST WEEK OF PUMPING.
5. GROUNDWATER LEVEL ONCE A WEEK FOR THE NEXT THREE WEEKS OF PUMPING.
6. READING EVERY THREE DAYS DURING EXCAVATION AND FOUR WEEKS AFTER REACHING PLANNED EXCAVATION DEPTH.
7. GROUNDWATER LEVEL ONCE A MONTH UNTIL CONSTRUCTION IS SUBSTANTIALLY COMPLETE.
8. WEEKLY READINGS ONCE DEWATERING SYSTEM IS TURNED OFF FOR A MINIMUM OF 3 MONTHS OR UNTIL GROUNDWATER LEVEL RETURNS TO THE PRE-CONSTRUCTION LEVELS, CONFIRMED WITH A MINIMUM OF 3 STABLE READINGS.
9. GROUNDWATER LEVEL ONCE A WEEK FOR FOUR WEEKS AFTER CONSTRUCTION IS SUBSTANTIALLY COMPLETE.
10. ADDITIONAL READING AS DIRECTED BY METROLINX REPRESENTATIVE.

BLDG. REF. No.

FILENAME: hina_7k_det_geo_143.dgn

SHEET No.

DRAWING No. SK-C143

REVISIONS	REVISIONS
△	△
△	△
△	△

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION
AUGUST 20, 2020

4TRANSIT
PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	A. JUNAYET	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20
SCALE		

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS
GEOTECHNICAL
TUNNEL MONITORING
TYPICAL LAYOUT AT TUNNELS

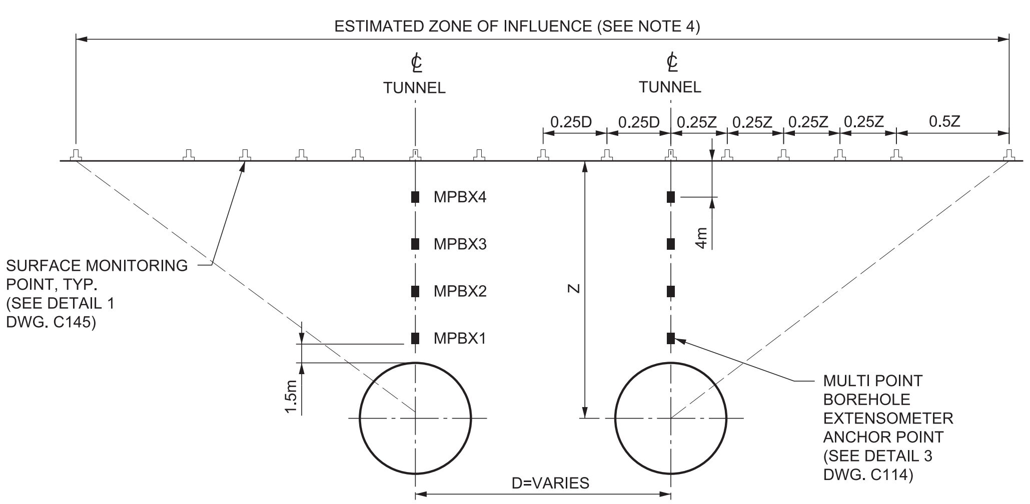
Contract No.:TC85–3A Package No.:

METROLINX

Dwg. No. SK–C143 Rev. No. Sheet No.

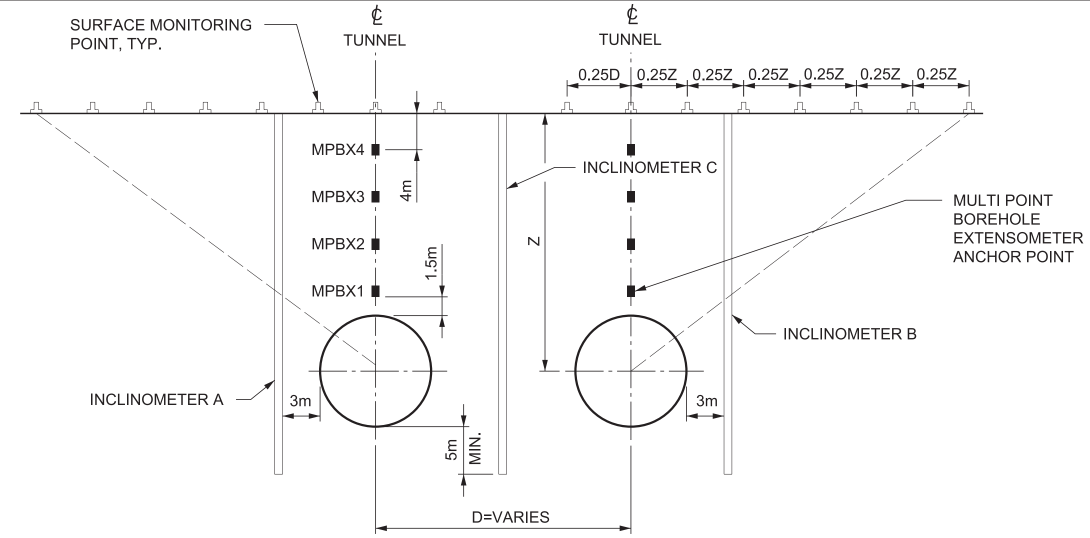
Plot Date: 8/18/2020

CADD FILE NAME: hina_7k_det_geo_144.dgn



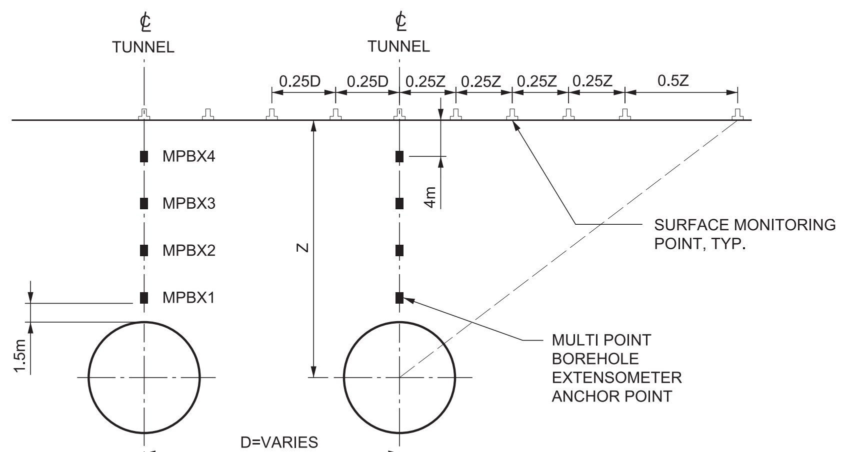
DETAIL 1 SK-C144

TYPICAL LAYOUT
SETTLEMENT ARRAY TYPE 1
FOR TWIN TUNNELS



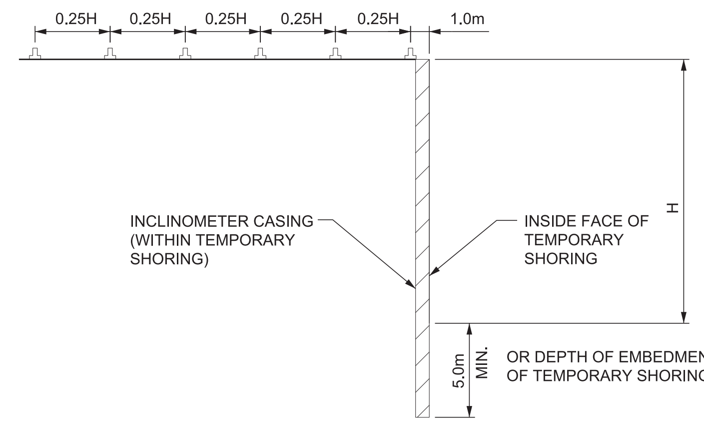
DETAIL 2 SK-C144

TYPICAL LAYOUT
SETTLEMENT ARRAY TYPE 2
FOR TWIN TUNNELS



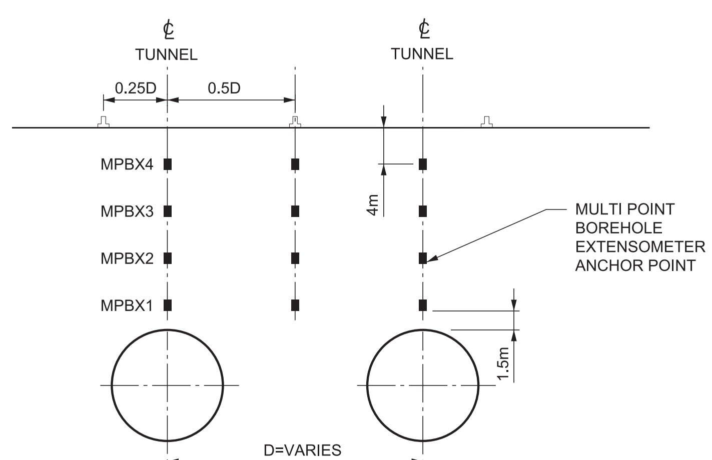
DETAIL 3 SK-C144

TYPICAL LAYOUT
SETTLEMENT ARRAY TYPE 3
FOR TWIN TUNNELS



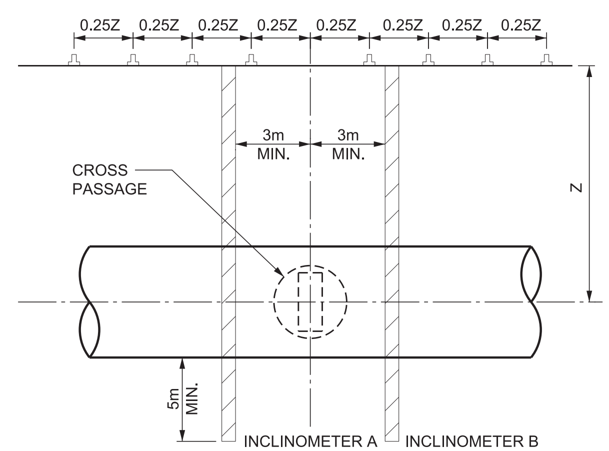
DETAIL 4 SK-C144

TYPICAL LAYOUT
SETTLEMENT ARRAY TYPE 4



DETAIL 5 SK-C144

TYPICAL LAYOUT
SETTLEMENT ARRAY TYPE 5
FOR TWIN TUNNELS



DETAIL 6 SK-C144

TYPICAL LAYOUT
SETTLEMENT ARRAY TYPE 6
(TRANSVERSE TO CROSS PASSAGE)

REVISIONS	REVISIONS

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION
AUGUST 20, 2020

4TRANSIT
PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	A. JUNAYET	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE 0 10 20 30 40m

EGLINTON CROSSTOWN WEST EXTENSION
TWIN TUNNELS
GEOTECHNICAL
TUNNEL MONITORING
TYPICAL LAYOUT AT TUNNELS

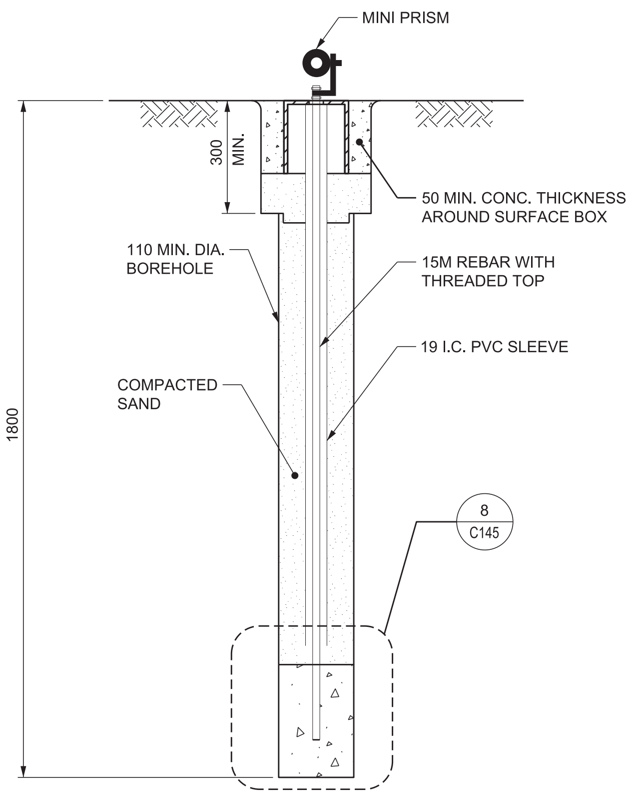
Contract No.: TC85-3A Package No.:
METROLINX
Dwg. No. SK-C144 Rev. No. Sheet No.

Plot Date: 8/18/2020

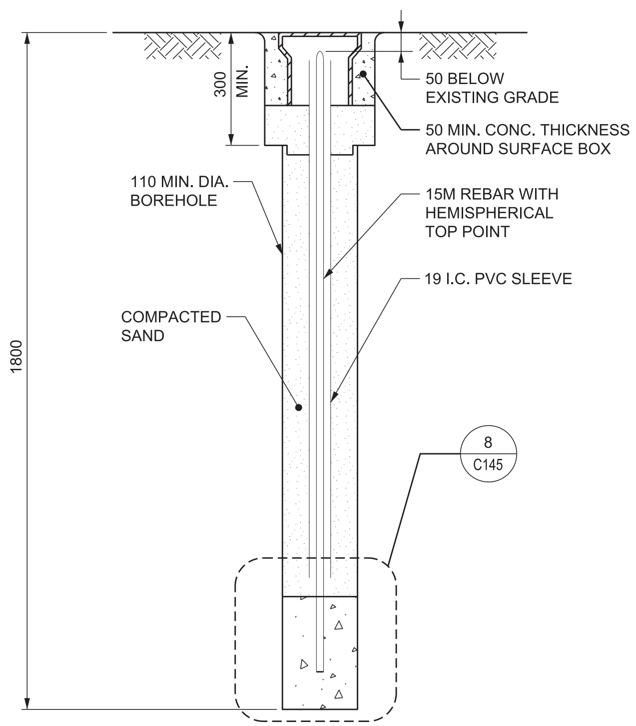
ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019
BLDG. REF. No.
FILENAME: hina_7k_det_geo_144.dgn
SHEET No.
DRAWING No. SK-C144

CADD FILE NAME: hina_7k_det_geo_145.dgn

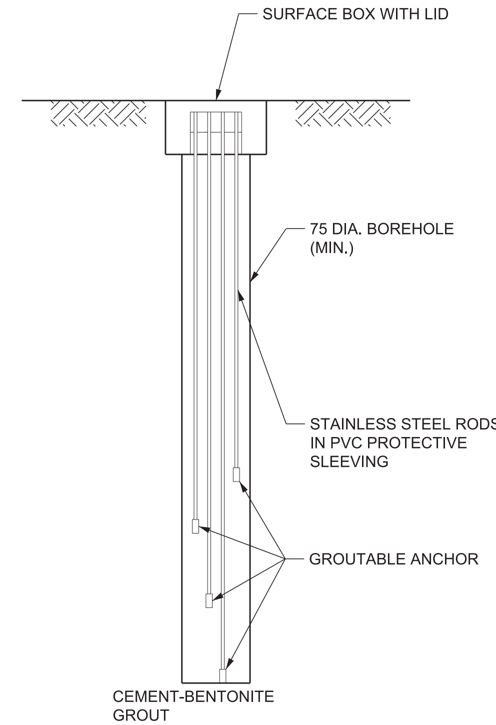
ECWE_engbord3.dgn (BORDER) OCTOBER 2, 2019



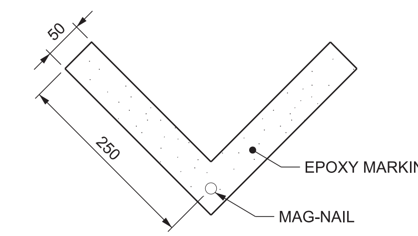
DETAIL 0 100 200 300 400mm 1
SURFACE MONITORING POINT (SMP) GENERAL USE 4 SK-C145



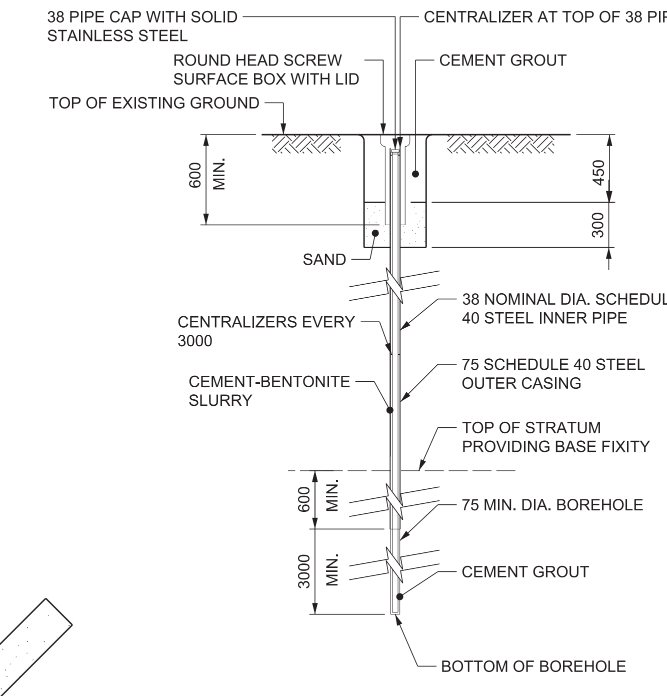
DETAIL 0 100 200 300 400mm 2
SURFACE MONITORING POINT (SMP) FOR USE IN ARRAYS WHERE SMP CONFLICTS WITH ROAD TRAFFIC 4 SK-C145



DETAIL N. T. S. 3
MULTI POINT BOREHOLE EXTENSOMETER (MPBX) SK-C145



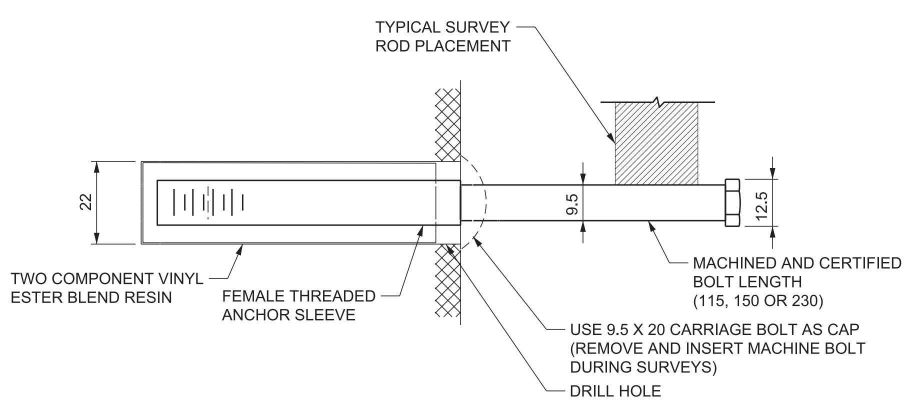
DETAIL 0 100 200mm 4
SURFACE TARGET 8 SK-C145



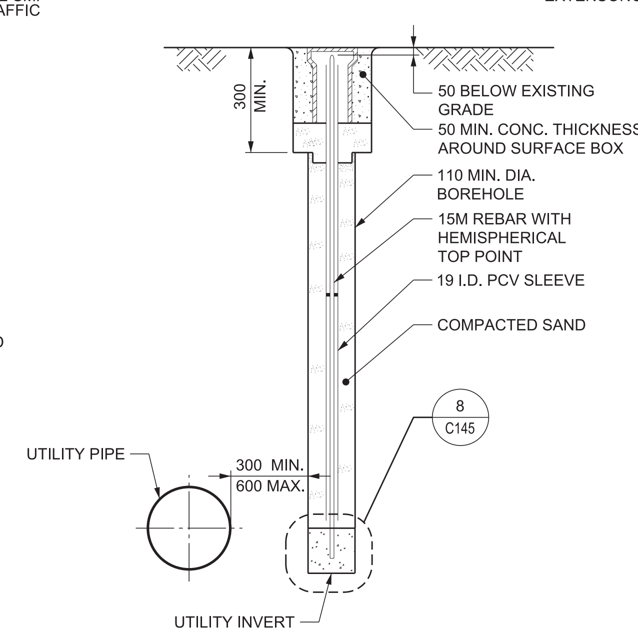
NOTE:

1. CEMENT-BENTONITE SLURRY TO BE DRILLED / CLEANED OUT OF OUTER PIPE BEFORE PLACEMENT AND FIXING OF THE INNER PIPE.

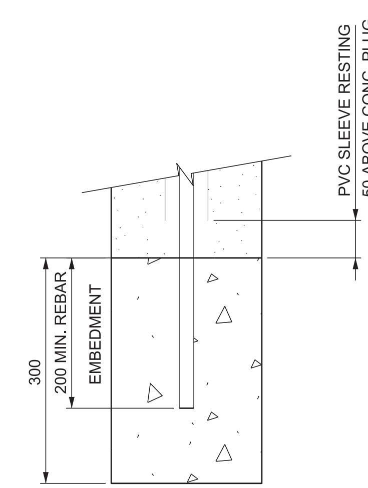
DETAIL 0 0.5 1m 5
BENCHMARK SK-C145



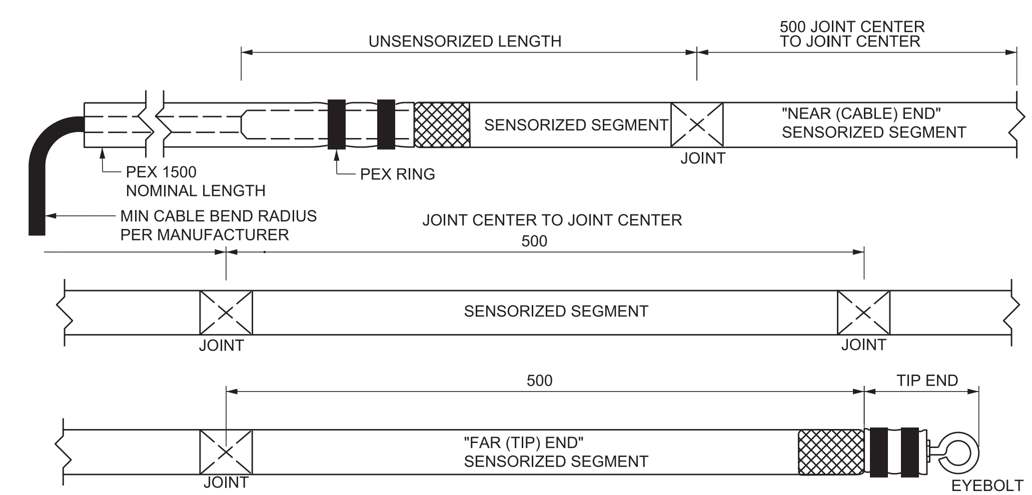
DETAIL 0 100 200 300 400mm 6
BUILDING MOVEMENT MONITORING POINT (BMP) 2 SK-C145



DETAIL 0 0.5 1m 7
UTILITY MONITORING POINT (UMP) SK-C145



DETAIL 0 100 200mm 8
REBAR EMBEDMENT SK-C145



DETAIL N. T. S. 9
SHAPE ACCELEROMETER ARRAY SK-C145

BLDG. REF. No.

FILENAME: hina_7k_del_geo_145.dgn

SHEET No.

DRAWING No. SK-C145

REVISIONS	REVISIONS

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION
AUGUST 20, 2020

4TRANSIT
PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	K. COFFILL	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20

SCALE
AS NOTED

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS
GEOTECHNICAL
TUNNEL, STRUCTURE, AND UTILITY MONITORING INSTRUMENTATION DETAILS

Contract No.:TC85-3A Package No.:

METROLINX

Dwg. No. SK-C145 Rev. No. Sheet No.

Plot Date: 8/18/2020

CADD FILE NAME: hina_7k_det_geo_146.dgn

ECWE_engbord3.dgn (BORDER) OCTOER 2, 2019

BLDG. REF. No.

FILENAME: hina_7k_det_geo_146.dgn

SHEET No.

DRAWING No. SK-C146

TABLE 1.0

TOATAL RESPONSE LEVELS FOR MTO HIGHWAY CROSSINGS SETTLEMENT DUE TO TUNNELLING AND CROSS PASSAGE CONSTRUCTION		
HIGHWAY CROSSING	REVIEW LEVEL	ALERT LEVEL
ANY	10 mm	15 mm

TABLE 2.0

RESPONSE LEVELS FOR VIBRATION MONITORING OF HYDRO TOWERS DUE TO TUNNELLING AND CROSS PASSAGE CONSTRUCTION		
HYDRO TOWER	REVIEW LEVEL	ALERT LEVEL
ANY	40 mm/s	50 mm/s

TABLE 3.0

SUMMARY OF RESPONSE LEVELS FOR BUILDING SETTLEMENT DUE TO TUNNELLING ONLY							
STA.	STREET ADDRESS	RESPONSE LEVEL (1)					
		REVIEW			ALERT		
		mm	ß	ε	mm	ß	ε

TABLE 4.0

SUMMARY OF RESPONSE LEVELS FOR BUILDING SETTLEMENT DUE TO CUT-AND-COVER EXCAVATION ONLY							
STA.	STREET ADDRESS	RESPONSE LEVEL (1)					
		REVIEW			ALERT		
		mm	ß	ε	mm	ß	ε

TABLE 5.0

SUMMARY OF RESPONSE LEVELS FOR BUILDING SETTLEMENT DUE TO CROSS PASSAGE EXCAVATION ONLY							
STA.	STREET ADDRESS	RESPONSE LEVEL (1)					
		REVIEW			ALERT		
		mm	ß	ε	mm	ß	ε

(1) ANGULAR DISTRIBUTION, ß ($x10^{3}$) AND HORIZONTAL STRAIN, ε ($x10^{3}$)

NOTE:

1. TUNNEL CO. TO DETERMINE RESPONSE LEVELS FOR ALL STRUCTURES ALONG THE ALIGNMENT WITHIN THE ZOI AND COMPLETE TABLES 3, 4 AND 5.

REVISIONS	REVISIONS
△	△
△	△
△	△

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION
AUGUST 20, 2020

4TRANSIT
PARSONS HATCH WSP

		YYYY/MM/DD
DRAWN	A. JUNAYET	2020/05/29
CHECKED	R. SHOBAYRY	2020/08/18
CORRECT	H. BIDHENDI	2020/08/20
SCALE	N. T. S.	

EGLINTON CROSSTOWN WEST EXTENSION

TWIN TUNNELS
GEOTECHNICAL
INSTRUMENTATION PROGRAM TABLES

Contract No.:TC85-3A Package No.:

METROLINX

Dwg. No. SK-C146 Rev. No. Sheet No.

Plot Date: 8/18/2020