April 2015

REPORT

REPORT ON

Foundation Investigation and Design
Proposed Culvert Replacement
Site No. 27-266C
Highway 34 Culvert
Township of Prescott and Russell, Ontario
W.P. 4110-11-01

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W.P. 4110-11-01

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with numerous culvert and bridge replacements and/or rehabilitations at various locations in the Eastern Region of Ontario as part of the 23 Structures MEGA 3 project.

This report presents the results of the foundation investigation conducted for the replacement of a structural culvert located at Site No 27-266c, which is constructed at the crossing of an Unnamed Creek and Highway 34, north of Highway 417, in the Township of Prescott and Russell, Ontario (WP 4110-11-01).

Initially, the culvert replacement at this site was planned to be undertaken as a Design-Build project. It is now understood that Dillon will be completing the detailed design of the culvert replacement. The purpose of the foundation investigation was to assess the subsurface conditions for the proposed culvert replacement by drilling boreholes and carrying out in-situ testing and laboratory testing on selected soil samples.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated August 2012. In addition, Golder's letter dated November 20, 2014 described the work plan for additional foundation engineering services for detail design.

The work was carried out in accordance with Golder's Quality Control Plan dated December 2012.

2.0 SITE DESCRIPTION

The existing culvert (Site No. 27-266c) is located at the crossing of Unnamed Creek and Highway 34, just north of the intersection of Nixon Road and the Highway 417 westbound off-ramp, in the Township of Prescott and Russell, Ontario. The existing culvert is located at about Station 10+242.

The existing culvert is a 3.2 metre wide by 2.0 metre high single span corrugated steel pipe arch culvert which is about 22 metres in length. The date of construction of the culvert is unknown. The invert of the existing culvert is at about Elevation 72 metres. The existing culvert location is shown on Drawing 1.

The existing pavement grade at the culvert location is at about Elevation 75 metres to 75.1 metres and there is approximately 1.1 metres to 1.4 metres of cover over the existing culvert. In this area, Highway 34 is one lane wide in each direction (i.e., 2-lane highway). The existing embankment slopes at the culvert locations are up to about 2 metres in height and are at a slope of about 1.5H:1V and appear to be stable. Based on a visual observation at the time of the field investigation, the existing pavement structure is in satisfactory condition. The existing culvert is generally in poor condition with severe corrosion and undermining of the culvert

Available information from nearby sites including the Hwy 34 bridge over Hwy 417 indicates that the subsurface conditions typically consist of compressible clay deposits overlying till overlying shale bedrock. The clay is highly variable in thickness and not present at all locations. The till deposit also has a variable thickness but often consists of a thin veneer above the bedrock surface.

The culvert is planned to be replaced with a 25.9 metres long, precast, segmental concrete box culvert that is 3 metres wide by 1.8 metres high (internal dimensions). Current design information indicates that the new culvert will be constructed in approximately the same location as the existing culvert alignment. No roadway grade raise is proposed as part of the culvert replacement project. However, realignment of the E-N/S Ramp to Highway 34 northbound curb line is proposed to improve road conditions and turning radius for truck traffic. This realignment would include a minor localized modifications to the existing embankment for the highway off-ramp located south of the eastern extent (inlet) of the new culvert.

Wingwalls are required on either side of the inlet for the new culvert. Consideration is being given to constructing these walls using permanent, cantilevered sheet pile walls.

It is understood that the culvert replacement will be carried out in stages. Stage 1 will involve shifting a single lane of traffic to the west (partially onto the south-bound lane and shoulder) and removing/replacing the east half of the culvert. Stage 2 will involve shifting a single-lane of traffic to the east onto the reconstructed half of the culvert and removing/replacing the west half of the culvert.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the culvert replacement was carried out in two stages. During the first stage, a preliminary investigation was carried out for a design build project between May 28 and 30, 2013. A second stage of investigation for a detail design was carried out on January 27, 2015. Overall, six boreholes (numbered 13-1 to 13-4, 15-1 and 15-2) were advanced at the locations shown on Drawing 1. The boreholes were advanced as follows:

- Boreholes 13-1, 13-2, and 13-4 were advanced using 108 millimetres inner diameter (I.D.) continuous-flight hollow-stem augers by a truck-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. These boreholes were advanced to a depth of about 12.2 metres below the existing ground surface.
- Borehole 13-3 was advanced using portable drilling equipment, supplied and operated by OGS Inc. of Almonte, Ontario. This borehole was advanced to a depth of about 10.4 metres below the existing ground surface.
- Boreholes 15-1 and 15-2 were advanced using 108 millimetre inner diameter (I.D.) continuous-flight hollow-stem augers by a truck-mounted drill rig, supplied and operated by George Downing Estate Drilling of Grenville-sur-la-rouge, Quebec. Upon reaching depths of about 16.8 and 1.5 metres in boreholes 15-1 and 15-2, respectively, the boreholes were advanced, without sampling, to depths of 19.8 metres using dynamic cone penetration testing (DCPT) to determine the refusal level as required for settlement analysis.

Soil samples in the boreholes were obtained at vertical intervals ranging from about 0.6 metres to 1.5 metres of depth, using a 50 millimetres outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. In-situ vane testing, using an MTO “N”-size vane was carried out to measure the undrained shear strength of the cohesive soils encountered at the site. An MTO “B”-size vane was used in the portable borehole (i.e., Borehole 13-3). Six relatively undisturbed, 73 millimetre diameter thin-walled Shelby tube samples of the clay were retrieved using a fixed piston sampler.

A standpipe piezometer was installed in Borehole 13-3 to monitor the groundwater level at the site. The standpipe consists of a 32 millimetre diameter rigid PVC pipe with a 1.5 metre long slotted screen section, installed within silica sand backfill and sealed by a 0.6 metre long section of bentonite pellet backfill. The water level in the standpipe piezometer was measured on July 10, 2013, some six weeks after installation.

The boreholes were backfilled with bentonite pellets, mixed with native soils. The site conditions were restored following completion of the work.

The field work was supervised throughout by members of Golder’s technical staff, who located the boreholes, observed the drilling, sampling and in-situ testing operations, logged the subsurface conditions encountered in the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder’s laboratories in Ottawa and Mississauga for further examination and testing. Index and classification tests consisting of grain size distribution, water content, and Atterberg limit testing were carried out on selected soil samples at the Ottawa laboratory. An oedometer (consolidation) test was carried out on one sample of the clay from each of Boreholes 13-4 and 15-1. This testing was carried out at the Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

The borehole locations and ground surface elevations were surveyed by Golder Associates Ltd. using a Trimble R8 GPS unit. The borehole locations, including MTM NAD83 northing and easting coordinates, and ground surface elevations referenced to Geodetic datum are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
13-1	Centre of the culvert alignment	5037437.2	212642.5	75.1
13-2	Centre of the culvert alignment	5037428.4	212639.6	75.0
13-3	West end of the culvert	5037432.8	212629.5	73.0
13-4	East end of the culvert	5037436.2	212649.9	74.9
15-1	South east of culvert	5037418.4	212659.4	74.7
15-2	East end of the culvert	5037437.4	212650.7	75.0

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment is located within the physiographic region known as the *Winchester Clay Plain*, as delineated in *The Physiography of Southern Ontario*[1], which lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Winchester Clay Plain lies between the Glengarry Till Plain and the sand plains of the United Counties of Prescott and Russell and composes an area of 580 square kilometres. It is a flat lying area located almost entirely within the drainage basin of the South Nation River[1]. The Winchester Clay Plain is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that overlie relatively thin, commonly reworked glacial till and glacial fluvial deposits that in turn overlie bedrock. This region is underlain by sedimentary rock, consisting of limestone interbedded with shale.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, six boreholes were advanced along the alignment of the existing culvert. The borehole locations, ground surface elevations and an interpreted stratigraphic profile are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the in-situ and laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets in Appendix A and on Figures 1 to 8.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the locations of the proposed culvert replacement consist of a pavement structure up to about 0.4 metres in thickness (where present) over 1.5 to 2.8 metres of embankment fill, underlain by a deposit of sensitive clay which extends to ranging from about 10.4 metres (the final depths of some boreholes) to 18.3 metres below the existing ground surface. The upper 0.9 to 1.4 metres of the clay deposit in boreholes 13-2 and 13-4 have locally been weathered to a grey brown firm to very stiff crust. The clay beneath the fill material at the other three borehole locations and beneath the weathered portion is unweathered and grey in colour and has a very soft to firm consistency.

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

4.2.1 Pavement Structure and Fill Material

The pavement structure of the travelled lanes of the highway was penetrated within three boreholes (13-1, 13-2 and 13-4). The pavement structure of the shoulder of the highway 417 off-ramp was penetrated within one borehole (15-1). At the borehole locations, the pavement structure consists of about 0.1 to 0.2 metres of asphaltic concrete over 0.2 metres of crushed stone base. The crushed stone base of the pavement structure was not present in borehole 13-4 and 15-1. The pavement structure is underlain by about

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

1.6 to 2.8 metres of embankment fill. The embankment fill generally consists of sand and gravel with traces of silt and clay to silty sand with some clay and varying amounts of gravel. About 1.5 metres of fill material was also present at ground surface in borehole 13-3, which was advanced at the west end of the culvert. The fill at this location consist of clay with varying amounts of silt, sand and gravel.

The fill was fully penetrated to depths ranging from 1.5 metres to 3.2 metres below the ground surface (corresponding to Elevations of 71.5 to 73.1 metres) at all borehole locations.

Standard penetration test ‘N’ values measured within the fill range from 1 in the clay fill in borehole 13-3 beyond the roadway, and from 10 to greater than 50 blows per 0.3 metres of penetration in the sand and gravel fill.

The results of grain size distribution testing carried out on five samples of the fill material are provided on Figure 1. The results of Atterberg limit testing on one sample of the clay fill materials indicate a plasticity index value of about 30 percent and a liquid limit value of about 54 percent, as shown on Figure 2, indicative of a clay of high plasticity. The measured water contents of five samples of the fill ranged from approximately 3 percent (in the sand and gravel fill) to 35 percent (in the clay fill, off the roadway).

4.2.2 Sensitive Clay

The embankment fill is underlain by a deposit of sensitive clay. Boreholes 13-1 to 13-4 were terminated within the deposit. Based on DCPT results, Boreholes 15-1 and 15-2, the clay deposit extends to depths of about 18.3 and 18.9 metres below the existing ground/embankment surface level, at about Elevation 56.4 and 56.1 metres.

At Borehole 13-2 and 13-4, the upper 1.4 and 1.1 metres, respectively, of the clay has been weathered to a grey brown crust. Two measured SPT “N” values in this material range from ‘Weight of Hammer’ to 3 blows per 0.3 metres of penetration, indicating a firm to very stiff consistency of the weathered crust.

The results of grain size distribution testing on two samples of the weathered clay are shown on Figure 3. The results of Atterberg limit testing on two samples of the weathered material indicate plasticity index values of 39 and 40 percent and liquid limit values of 62 and 64 percent, as shown on Figure 4, indicating a clay of high plasticity. Four measured natural water contents of the weathered material range from about 41 to 71 percent.

The clay below the depth of weathering at Boreholes 13-2 and 13-4 and below the fill materials at Boreholes 13-1, 13-3, 15-1 and 15-2 is grey in colour and extends to depths of at least 10.2 to 18.9 metres.

The results of in situ vane testing carried out in this material indicate undrained shear strengths which range from about 12 to 42 kPa indicating a very soft to firm consistency. The lower shear strength values (i.e., below about 20 kPa) were obtained from Borehole 13-3, which was advanced just west of the existing culvert beyond the roadway using portable drilling equipment and a “B”-size vane.

The results of grain size distribution testing on four samples of the unweathered clay are shown on Figure 5. The results of Atterberg limit testing on nine samples of this material indicate plasticity index values which range from about 25 to 60 percent and liquid limit values that range from 49 to 84 percent, as shown on Figure 6, typically indicate a clay of high plasticity. The measured natural water content of this unweathered material ranges from about 68 to 89 percent. These natural water contents are generally near or above the measured liquid limits.

Oedometer consolidation testing was carried out on two samples of the unweathered clay from Boreholes 13-4 and 15-1 between about Elevation 69.9 and 70.3 metres and Elevation 68.6 and 68.1 metres, respectively. The results of that testing, which are provided on Figures 7 and 8 are summarized in the table below and indicate that this material is near normally consolidated, with a preconsolidation pressure of around 65 to 75 kPa and overconsolidation ratio of 1.0 to 1.1.

Borehole/Sample Number	Sample Depth/Elevation (m)	Unit Weight (kN/m^3)	σ_P' (kP)	σ_{VO}' (kP)	$\sigma_P' - \sigma_{vo}'$ (kPa)	Cc	Cr	e_o	OCR
13-4 / 5	4.6 - 5.0 / 69.9 - 70.3	15.2	75	67	8	1.63	0.037	2.22	1.1
15-1 / 7	6.1 - 6.6 / 68.6 - 68.1	15.2	65	65	0	1.93	0.004	2.27	1.0

Notes:

σ_P' - Apparent preconsolidation pressure
σ_{VO}' - Computed existing vertical effective stress
Cc - Compression index
Cr - Recompression index
e_o - Initial void ratio
OCR - Overconsolidation ratio

4.2.3 Inferred Till and Refusal

DCPT resistance values increased significantly in Boreholes 15-1 and 15-2 at depths of about 18.3 and 18.9 metres below the existing ground/embankment surface level, at about Elevation 56.4 and 56.1 metres, respectively. Based on available subsurface information in the area, this depth would likely represent the interface between the clay and the till.

Refusal to the advancement of the DCPT was encountered in Borehole 15-1 at a depth of about 19.8 metres below the existing ground/embankment surface level, at about Elevation 54.9 metres. This refusal could indicate the bedrock surface or the presence of cobbles and boulders in the till deposit.

4.2.4 Groundwater Conditions

The groundwater level in the piezometer in Borehole 13-3 was measured on July 10, 2013. The piezometer was sealed into the clay deposit.

The groundwater level in the piezometer is summarized in the table below.

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
13-3	73.0	0.6	72.4	July 10, 2013

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

5.0 CLOSURE

This report was prepared by Ms. Kim Lesage, P.Eng., and reviewed by Mr. Fintan Heffernan P.Eng., the designated MTO contact for this project.

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PART B

FOUNDATION DESIGN
PROPOSED CULVERT REPLACEMENT
SITE NO. 27-266C
UNNAMED CREEK, HIGHWAY 34
TOWNSHIP OF PRESCOTT AND RUSSELL, ONTARIO
W.P. 4110-11-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the Unnamed Creek Culvert on Highway 34 (Site 27-266c). The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

The existing culvert alignment is shown on Drawing 1. The existing culvert is a 3.2 metres wide by 2.0 metres high single span structural plate corrugated steel pipe arch culvert which is about 22 metres in length. The existing culvert has approximately 1.1 metres to 1.4 metres of cover beneath the travelled surface of the highway.

Current design information indicates that the new culvert will be constructed at approximately the same location as the existing culvert alignment. The proposed culvert invert level will be at about Elevation 71.3 to 71.4 metres. Wingwalls/retaining walls are also being considered at the inlet. Consideration is being given to constructing these walls using permanent, cantilevered sheet pile walls. No significant roadway grade raise is proposed as part of the culvert replacement project; however, a grade raise is required immediately behind the proposed wingwalls and relatively minor modifications to the highway embankments are planned to facilitate improvements to the turning radii for the intersection to the south of the culvert.

The culvert is planned to be replaced with an approximately 26 metres long, segmental precast, concrete rigid-frame box culvert that is 3 metres wide by 1.8 metres high (internal dimensions) which will be constructed in two stages. Other foundation types are also discussed in the following sections and a comparison of the foundation alternatives is provided in Table 1.

6.2 Culvert Foundations

The subsurface conditions at this site generally consist of up to about 3.2 metres of fill, underlain by sensitive and compressible clay that extends to depths in excess of 18 metres below road level.

The existing clay soils at this site are close to normally consolidated. Therefore, a significant concern for design of the replacement culvert is compression and settlement of the underlying clay soil and the impacts that those settlements could have on the performance of the culvert. The clay soils at this site have very limited capacity to accept any increase in load without undergoing significant settlement. As discussed below in Section 6.2.1, it is considered generally feasible to support the culvert on or within the native clay subgrade but it is important to limit the magnitude of the foundation stresses since stresses higher than existing conditions will result in increased magnitudes of settlement. The use of lightweight backfill materials is planned to limit foundation loads.

A pre-cast concrete closed box culvert is considered to be feasible for this site since the foundation loads are distributed over a larger area, resulting in lower foundation stress levels, and therefore reduced settlement magnitudes in comparison to open bottom culvert configurations. This is considered to be the preferred option from a foundations perspective. A pre-cast culvert system is preferred over a cast-in-place culvert because it will be more accommodating to the expected settlements at this site and take less time to install.

The use of a rigid frame open box culvert would result in larger settlements than a closed box culvert, due to the higher concentration of foundation stresses, and is therefore not considered suitable for this site.

Tunnelling was not considered as a replacement option for the culvert at this crossing. As a general guideline, trenchless crossings in overburden should only be considered where the cover above the crown of the tunnel/bore would be at least twice the tunnel/bore diameter relative to the ground surface. Lesser amounts of cover could jeopardize the stability of the working face (depending on the method) or lead to ground loss and settlement. It is therefore considered that tunnelling is not considered a feasible option for this site.

Also, sliplining was not considered as a suitable option at this site because relining the culvert with grout would add additional loading (i.e., the weight of the liner as well as the grout) onto the underlying normally consolidated clay deposit, with the potential for causing significant settlements. Furthermore, the new liner may not leave sufficient space for the required culvert flow.

It is also not considered to be a practical or economic option to support the culvert on deep foundations since the available subsurface information indicates the bedrock surface is at a depth greater than 15 metres below founding level and would result in long and relatively expensive deep foundations. Furthermore, the use of deep foundations could lead to long-term differential settlement between the culvert and adjacent roadway embankment due to ongoing creep of the underlying clay resulting from the original embankment construction. Therefore, detailed design guidelines are not provided for deep foundations since they would not be economical or practical.

It is not necessary to found the box culvert at the standard depth for frost protection purposes as box structures are tolerant of small magnitude movements related to freeze-thaw cycles should these occur. The box culvert should, however, be founded below any existing fill and surficial soils containing organic matter. Footings for any associated wingwalls/retaining walls should be founded at a minimum depth of 1.8 metres below the lowest surrounding grade, to provide adequate protection against frost penetration.

Based on the above, the use of a concrete box culvert is considered the preferred design option from a foundation perspective at this site. Recommendations for closed box culvert design are presented in the following sections.

6.2.1 Box Culvert

6.2.1.1 Geotechnical Resistance

It is understood that the box culvert will be founded on or within the soft to firm clay subgrade, and it has been assumed that the culvert will be founded at about elevation 71.3 to 71.4 metres. It is recommended that a minimum 300 millimetres thick layer of Ontario Provincial Standard Specification (OPSS.PROV) 1010 *(Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material)* Granular A be placed below the base slab on the subgrade to form a bedding layer for the culvert segments, and to limit the degradation of the sensitive clay subgrade.

The factored geotechnical resistance at Ultimate Limit States (ULS) for the culvert will be controlled by the shear strength of the underlying soft grey clay as well as by the depth of embedment below ground surface level. A factored ULS geotechnical resistance of 90 kPa may be used.

The geotechnical resistance at Serviceability Limit States (SLS) is controlled by the compressibility of the soft clay deposit present at a shallow depth below founding level. In considering the height and weight of the existing embankment fills beneath the driving lanes, the available information indicates that the clay deposit is near normally consolidated (i.e., the existing effective pressure at depth is at or very close to the deposit's preconsolidation pressure). Therefore, beneath the existing lanes, an increase in stress above the existing values could resulting in significant settlements as a result of primary and secondary consolidation settlement. Due to these conditions, a design philosophy incorporating the use of light-weight fill materials to negate/minimize additional loads resulting from the culvert reconstruction should be adopted.

Further discussion on lightweight fill thicknesses and requirements are discussed in Sections 6.4.1 and 6.6.5. Provided that sufficient lightweight fill is used to provide for a 'no new net load' condition, settlements of the culvert are expected to be less than 25 millimetres.

These geotechnical resistances are given under the assumption that the loads will be applied perpendicular to the surface of the foundations. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.2 of the *Canadian Highway Bridge Design Code (CHBDC).*

The footing subgrade should be inspected by a Quality Verification Engineer (QVE) in accordance with OPSS 902 (*Construction Specification for Excavating and Backfilling – Structures*). Further discussion regarding subgrade preparation and protection is provided in Section 6.6.4.

6.2.2 Resistance to Lateral Loads

The resistance to lateral forces/sliding for the culvert should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The resistance values will also depend on the founding levels/strata.

The culvert will be constructed on a granular pad on the unweathered clay. For this case, the following parameters should be used:

Interface and Loading Condition	Parameter
Concrete – granular pad: short or long term loading	Effective friction angle = 33 degrees
Granular A pad – clay subgrade: short term loading	Undrained cohesion = 20 kPa
Granular A pad – clay subgrade: long term loading	Effective friction angle = 28 degrees

However, where EPS Geofoam lightweight fill will be placed beneath the culvert, the potential for shearing across the interface with the insulation should also be checked using a friction angle of 25 degrees.

These values are unfactored; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3 Lateral Earth Pressures for Culvert Design

The lateral earth pressures acting on the box culvert walls will depend on the type and method of placement and slope of the backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

6.3.1 Lightweight Fill

As discussed later in Section 6.4.1 of this report, the use of expanded polystyrene (EPS) lightweight fill is proposed for mitigating the potential roadway settlements due to compression of the underlying clay deposit. In regards to the lateral earth pressures described in Section 6.3.2, the low unit weight (in comparison to soil) will alter the design lateral earth pressures. For design purposes, the EPS could be assumed to have a unit weight of 1 kilonewton per cubic metre; this low unit weight should be considered in the calculation of the vertical stress acting on the underlying granular backfill, and thus the horizontal lateral pressure applied to the wall. Where the backfill is relied upon to provide passive resistance to the walls, the contribution of the EPS itself should be neglected, but the effect of the lower unit weight and lower vertical stress level must be considered in assessing the passive resistance from the underlying backfill.

6.3.2 Granular Fill

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

- Select free-draining granular fill meeting the specifications of the provincial version of the Ontario Provincial Standard Specifications (OPSS.PROV) Granular ‘A’ or Granular ‘B’ but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in accordance with OPSS.PROV 501 (Compaction). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3101.150 and 3121.150.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501 (Compaction). Other surcharge loadings should be accounted for in the design as required.

- The granular fill may be placed either in a zone with a width equal to at least 1.8 metres behind the back of the abutment stem (Case I) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II).

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material:

Soil Unit Weight	20 kN/m^3
Coefficients of Static Lateral Earth Pressure: Active, K_a At rest, K_o	 0.35 0.50

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

Soil Unit Weight	Granular 'A'	Granular 'B' Type II
	22 kN/m^3	21 kN/m^3
Coefficients of Static Lateral Earth Pressure: Active, K_a At rest, K_o	 0.27 0.43	 0.27 0.43

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding (i.e., the wingwalls), at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows:
  - Rotation (i.e., of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
  - Horizontal translation of 0.001 times the height of the wall; or,
  - A combination of both.
- Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the CHBDC, the site-specific zonal acceleration ratio for the area of the site is 0.2. Based on experience, for the subsurface conditions at this site, a 10 percent amplification of the ground motion could occur, resulting in an increase in the ground surface acceleration to 0.22 g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.22.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding (i.e., culvert walls), the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., k_h = 0.33). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., k_h = 0.11).
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Seismic Active Pressure Coefficients, K_{AE}

	Case I	Case II	
		Granular 'A'	Granular 'B' Type II
Yielding wall	0.40	0.33	0.33
Non-yielding wall	0.65	0.50	0.50

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.22. This corresponds to displacements of up to approximately 55 millimetres at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \cdot \gamma \cdot d + (K_{AE} - K_a) \cdot \gamma \cdot (H-d)$$

Where: $\sigma_h(d)$ = the lateral earth pressure at depth, d, (kPa);

K_a = the static active earth pressure coefficient;

K_{AE} = the seismic active earth pressure coefficient;

γ = the unit weight of the backfill soil (kN/m^3), as given previously;

d = the depth below the top of the wall (m); and,

H = the total height of the wall (m).

6.4 Settlement

It is understood that the new box culvert will be situated partially over the existing culvert location. Furthermore, realignment of the E-N/S Ramp to Highway 34 northbound curb line is proposed to improve road conditions and turning radius for truck traffic. This realignment would include a minor localized widening of the existing embankment of the Highway 417 off-ramp located south of the eastern extent of the new culvert.

6.4.1 Culvert Settlement Considerations

Since the clay deposit is near normally consolidated, any increase in stress beyond the existing condition (i.e., due to increased loading from the replacement culvert) could result in significant settlement. Similar settlements to those mentioned above would also result adjacent to the culvert due to the increased weight of the granular materials which will be used as backfill alongside the culvert. This could result in 'sags' in the pavement profile leading up to and away from the culvert. Following discussions with Dillon and MTO, a design philosophy incorporating the use of light-weight fill materials to negate/minimize additional loads resulting from the culvert reconstruction was adopted.

The use of Expanded Polystrene (EPS) is recommended over other lightweight fill types (e.g. lightweight concrete, tire derived aggregate, slag, etc.) in order to reduce the quantity of lightweight fill required.

The commercially-available 'Settle-3D' software was used to review loading conditions associated with the existing and proposed culvert configurations in order to determine the amount of EPS lightweight fill required to result in a no additional load condition. For the main part of the new culvert (i.e., the central section where the existing culvert crosses through the existing embankment), EPS is recommended to be placed at the following locations to account for the additional loads imposed by the construction of the new culvert:

- A thickness of 0.35 metres of EPS should be placed above the culvert; and
- Additional EPS materials measuring 1.35 metres in thickness and 1.2 metres in width should be placed on both sides along the length of the culvert. The top of the EPS materials on the sides of the culvert should match the top of the EPS materials placed above the culvert. For this configuration, the base of the EPS on the sides would be just above the drain holes in the culvert.

Additional EPS would be required at the outlet where the culvert will be extended, to account for increased loading associated with the new culvert segments, and removal of clay and replacement with granular soils beneath the culvert. There does not appear to be any cover on the culvert in this area and, as such, it is anticipated that EPS would need to be installed below the culvert at this location. The thickness of EPS should be determined based on the 'no net load increase' principal using unit weights of 1 kN/m^3 and 16 kN/m^3 for the EPS and the existing soils for the assessment.

Similarly, additional thickness of EPS will be required at and around the inlet of the culvert to counteract the additional load that will be imposed by the extended culvert as well as the backfill behind the permanent sheet pile walls (i.e., wingwalls) associated with the proposed increased grade. Based on the proposed geometry, it is expected that this EPS would have to be placed behind the walls (i.e., adjacent to the sides of the culvert) and underneath the new culvert.

Based on the recommended use of light-weight fill materials and the 'no net load increase' principal, then the total and differential culvert settlements should be minimal (i.e., less than 25 millimetres).

Consideration should be given to wrapping the culvert joints with a geotextile (particularly for the joints near the ends of the culvert where extensions are being added) to avoid the potential for material loss in the event of some differential settlement/distortion of the culvert occurs.

It should be noted that since the clay at this site is normally consolidated or just slightly overconsolidated, small changes in loading can result in significant settlement magnitudes. Increases in loading could occur during construction due to a number of factors that are outside the control of the designer (i.e., such as over-excavation resulting in increased thicknesses of granular material or increased concrete thicknesses). The risk of larger than tolerable settlements, due to unforeseeable construction or site circumstances that result in modest load increases, should be considered during the design. A Non-Standard Special Provision (NSSP) on this matter is provided in Appendix B.

6.4.2 Modifications to Highway Embankments

Minor, localized modifications to the sideslopes of the existing Highway 417 E-N/S Ramp to Highway 34 embankment located south of the eastern extent of the new culvert is proposed to improve road conditions and turning radius for truck traffic. The proposed modification would consist of a slight grade raise of about 0.25 metres over a width of about 10 metres in the shoulder and sideslope area of the embankment. The settlements relating to this minor embankment modification could result in increased pavement maintenance alongside the E-N/S Ramp to Highway 34 northbound curb (i.e., padding).

6.4.2.1 Existing Services

The proposed localized embankment widening is planned to be carried out above/adjacent to two Enbridge Gas Distribution (Enbridge) high pressure gas mains (with diameters of 150 and 200 millimetres), which are located just east of the existing culvert and have inverts at about Elevation 70.2 and 70.5 m, respectively.

As stated above, the results of the foundation investigations indicate that the clay deposit has limited capacity to support additional stress (such as from foundation loads or the weight of additional embankment fill) without being overstressed, which would lead to settlements. Therefore, the localized embankment modifications, although minor, could result in settlement of the gas mains which are underlain by the compressible clay materials. Although the greatest settlements of the embankment widening would occur beneath the crest of the embankment, the effects of the embankment loading will extend somewhat beyond the footprint and the toe of the embankment. In order to limit the magnitude of potential settlements, reconstruction of the edges of the embankment using rock fill materials (with partial removal of the existing granular) has been proposed as these materials can be constructed at steeper angles (~1.25H:1V) and are slightly lighter than the existing sand and gravel fill.

Enbridge has indicated that up to 50 millimetres of settlement of the high pressure gas mains at the site would be acceptable. In order to estimate the magnitude of settlement of the clayey soils underlying the embankment modifications, analyses were carried out using the commercially-available ‘Settle-3D’ software. These analyses were carried out using the preconsolidation pressure profile and consolidation parameters obtained from the foundation investigations, and rock fill was used as the material for the widening. Our current analysis indicates that the settlements caused by the proposed widening would be limited to less than 40 millimetres in 50 years below the crest of the widening, which is within the tolerance value. Of this settlement, some 25 millimetres will be primary consolidation settlement and will take place within one year. The remaining 15 millimetres will be long term secondary consolidation settlement.

A monitoring program will need to be implemented to monitor the settlements prior to, during, and following construction along the portion of the gas mains within the construction zone. Monitoring shall, at a minimum, consist of surveying of the elevations of 4 in-ground settlement monitoring points (i.e., settlement rods) established between the pipes. The elevations of the monuments should be surveyed prior to the start of construction (at least three times), at least three times daily while construction is in progress, weekly for at least three months thereafter, and monthly for another three months thereafter. The results should be reviewed at that time to determine the settlement patterns and whether further monitoring is required.

A settlement monitoring plan is provided in Appendix C of this report. A Non Standard Special Provision for settlement rods should be included in the contract documents and has also been included in Appendix C.

6.5 Wingwalls/Retaining Walls

Wingwalls are proposed to be constructed on either side of the culvert inlet. The wingwalls are understood to have a maximum exposed height (top of wall to base of wall at creek level) of about 2 metres near the edges of the new culvert and will transition to minimal height within about 4.5 to 5 metres of the culvert. The embankment above the wingwalls is planned to have a 3H:1V slope.

As the culvert will be extended, a grade raise is required in and behind the area of the wingwalls. The clay deposit at this site has very limited capacity to support the foundation loads of retaining walls or the weight of the backfill soils. As discussed previously, a no new net-load philosophy has been used for the design of the new culvert crossing and roadway improvements in order to limit the potential for significant settlements of the soft clay deposit that underlies the site. It is understood that a small grade raise (typically less than 1 metre) is proposed in the area of the roadway shoulders adjacent to the sheet pile retaining walls. In this regard, portions of the existing materials present behind the wall locations would need to be replaced with lightweight fill materials in order to offset the loads from the new fill. Lightweight fill will therefore form, at least in part, the backfill soils behind the walls.

Various wingwall systems were reviewed with Dillon (i.e., steel sheet pile, armour stone, gabion, reinforced concrete gravity or cantilever, and RSS walls systems). Retaining wall systems, such as reinforced concrete gravity or cantilever walls and armour stone walls that require high bearing resistances and that would impart significant new loads on the site soils are not feasible as they would result in unacceptable settlements. Furthermore, it is general practice to not use RSS wall systems wherever significant ground settlements are expected because, although the wall system itself is somewhat flexible, the resulting opening of the joints is typically unacceptable from an aesthetic standpoint. Cantilevered steel sheet pile wingwalls are the preferred alternative to be constructed in order to limit the loading on the clay deposit at the east end of the proposed culvert replacement.

Given the proximity of the pipelines to the wingwall, the sheet piles will need to be installed using low vibration installation methods in order to limit vibrations to below the utility owner's specified vibration tolerances.

The design of the cantilever walls should consider the net pressures acting on the wall in accordance with standard design procedures. The following provides soil parameters that may be used in determining lateral earth pressures (for static conditions) acting on the proposed cantilever sheet pile walls.

The existing embankment fill materials would impart lateral earth pressures on the lightweight fill. Due to the limited width of lightweight fill behind the wall, it is recommended that the design of the wall take into account lateral earth pressures based on the granular embankment fill materials. In this regard, the active pressures acting on the walls within the backfill zone (i.e., above the native clayey soils) should be based on the existing granular fill materials within the embankment and the following values may be used in design.

Material	Granular Fill
Bulk Soil Unit Weight: Submerged Soil Unit Weight:	21 kN/m^3 11.2 kN/m^3
Coefficient of static lateral earth pressure (for 3H:1V slope above wall): Active, K_a	0.39

The following equation should be used to calculate the 'active' horizontal pressure acting on the sheet pile wall from the fill materials:

$$\sigma_h'(z) = K_a (\sigma_{vo}' + q)$$

where $\sigma_h'(z)$ is the lateral earth pressure at depth, z, (kPa);

σ_{vo}' is the vertical effective stress at depth, z, and (kPa)

K_a is the static active earth pressure coefficient (see table); and,

q is the surcharge at ground surface

For the cohesive soils encountered beneath the fill at elevations below about 72 metres at this site, the lateral earth pressures/resistances should be checked under both drained and undrained conditions to determine which case will govern. The near surface materials present within the ditch in front of the wall were likely disturbed as a result of construction of the gas pipelines and may also have reduced strength as a result of softening during freeze/thaw cycles. Standard practice is to neglect or reduce the passive resistance in the frost zone. For this low wall, a reduced passive resistance based on residual strength parameters in the frost zone (i.e., within 1.8 metres of ground surface on the front side of the wall) is recommended.

The following soil parameters for the clayey soils may be used for lateral earth pressure calculations for the sheet pile wall under drained, static loading conditions.

Material	Clay
Bulk Soil Unit Weight: Effective Soil Unit Weight	15.5 kN/m^3 5.7 kN/m^3
Coefficients of lateral earth pressure: Active, K_a Passive, K_p Passive (Frost Zone), K_p	 0.4 2.45 1.8

For undrained conditions, the total active (Pa) and passive earth pressures (Pp) within the clay deposit may be calculated using the following formulas. An undrained shear strength (Cu) of 20 kPa may be considered in design for the clayey soils present beneath an elevation of about 72 metres; a reduced Cu value of 5 kPa should be considered in the frost zone in front of the wall.

where $P_a =$ (total vertical stress + surcharge) – $2C_u$

$P_p =$ total vertical stress + $2C_u$

A Factor of Safety of 2 should be applied to the calculated passive resistance in order to limit wall deflections to around 0.01 (deflection of top of wall over the height of the exposed face of the wall). A different FoS may be appropriate if a different deflection is required.

Hydrostatic water pressures should be considered in the design of the wall if there will be unbalanced water levels on either side of the wall.

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

Seismic loading will result in increased lateral earth pressures acting on the wall from the granular backfill materials. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. The site-specific zonal acceleration ratio (A) for the site is 0.2. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.2. In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, (kh) is taken as 0.5 times the zonal acceleration ratio for structures which allow lateral yielding.

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution within the granular fill materials, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at the base of the granular layer (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K\,\gamma\,d + (K_{AE} - K)\,\gamma\,(H-d)$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d, (kPa);

K is the static active earth pressure coefficient, K_a (to be used for yielding walls);

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), as given previously;

d is the depth below the top of the wall (m); and,

H is the height of the wall (m).

The seismic active pressure coefficients (K_{AE}) within the granular backfill zone may be taken as 0.6 accounting for the sloping backfill and amplification within the soft clay deposit. It should be noted that this seismic earth pressure coefficients assume that the back of the wall is vertical. The above KAE value is for a yielding walls and is applicable provided that the wall can move up to 250A millimetres, where A is the design zonal acceleration ratio of 0.2. This corresponds to displacements of up to approximately 55 millimetres at this site.

6.6 Construction Considerations

6.6.1 Groundwater and Surface Water Control

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement, to allow excavation and foundation construction to be carried out in dry conditions.

Depending on the flow of the creek at the time of construction, the surface water flow could be passed through the culvert area by means of a temporary pipe, or diverted by pumping from behind a temporary cofferdam. Temporary cofferdams should not be constructed in close proximity to the existing gas pipelines as this could lead to settlement of the pipelines. Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the sensitive clay subgrade soils; further discussion on this aspect is provided in Section 6.6.4.

A sample NSSP for groundwater and surface water control is provided in Appendix B.

6.6.2 Excavations and Temporary Roadway Protection

Temporary excavations for the culvert replacement will be made through the existing fill and are expected to terminate or extend into the very soft to firm unweathered clay deposit. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The existing fill would be classified as Type 3 soil and the underlying very soft to firm clay would be classified as Type 4 soil, based on the OHSA. According to OHSA, excavations that extend to, or into, Type 4 soils should be made with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V). Flatter excavation side slopes may be necessary due to the nature of the underlying soft sensitive clay deposit at this site. A sample NSSP regarding this issue is included in Appendix B.

Based on the proposed, staged culvert construction, temporary protection systems will be required adjacent to the active highway lanes and construction work areas. This support system should be designed and constructed by the contractor in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539.

Excavated soils should not be stockpiled adjacent to the crest of the excavation side slopes (or above shoring) due to the potential to reduce the factor of safety against side slope or basal instability and the increased loads acting on shoring systems.

A conventional shoring system for the subsurface conditions at this site would consist of an interlocking steel sheet piling supported against lateral movement. The design of the shoring system should be entirely the responsibility of the contractor. The potential methods of lateral support will be limited by the subsurface conditions as the soft clay soils are not expected to provide sufficient support for either tie-back anchors or raker footings. In this regard, the lateral support would need to be provided using walers and internal struts/braces or by tie-backs connected to dead-men anchors or secondary sheet pile walls, or by deep ground anchors extending into the till and/or bedrock..

Furthermore, the design of that shoring must consider the very soft to soft clay deposit at depth and the potential for basal instability of the excavation. Basal instability occurs when the soil beneath the sheeting is sheared by the unbalanced weight between the soil outside and inside the excavation and could lead to the flow of sheared/disturbed clay into the excavation, significant deformation (settlement and ground slumping) behind the sheeting, and possible collapse of the shoring system. The potential for basal instability is dependent on the size/depth/geometry of the excavation as well as construction equipment loads in the vicinity of the excavation. Calculations indicated that a conventional shored excavation with full depth vertical walls to the base of the excavation would have a marginal factor of safety against basal instability (i.e., have a factor of safety of about 1.4 to 1.5). Unloading (by excavation of the adjacent ground) or extensions of the sheet piles significantly below the excavation floor level could therefore be required to limit the potential for basal instability.

In addition, the design of the sheeting protection for deeper excavations would also need to resist the lateral loading imposed by the clay. This may require a very heavy/strong sheeting section and potentially driving soldier piles in front of the sheeting and into the glacial till at depth. Even with these measures, some distortion of the ground above/behind the sheeting should be expected.

Where the temporary protection system will be installed adjacent to EPS lightweight fill, the contractor should provide a methodology for removal of the protection system that will limit disturbance to the EPS during removal. Alternatively, the protection system can be left in-place.

A sample NSSP regarding basal heave is provided in Appendix B.

6.6.3 Culvert Bedding, Backfill and Erosion Protection

Prior to the placement of any engineered fill or bedding, all topsoil, organic material or loosened soil should be stripped from below the proposed culvert footprint. Buried organic matter may also be encountered. Those materials would need to be subexcavated if present below founding level.

The bedding, backfilling, and levelling pad requirements for the culvert should be in accordance with OPSS 422 *(Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut)* for pre-cast rigid frame box culverts. Box culvert replacements should be provided with at least 300 millimetres of OPSS.PROV 1010 Granular A material for bedding purposes.

Backfill and cover for the concrete culvert and any associated wingwalls/retaining walls should be completed in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010 *(Backfill and Cover for Concrete Culverts)* taking into account the need for EPS backfill behind the wingwalls described in Sections 6.4.1. Backfill for the box culvert walls should consist of granular fill meeting the requirements of OPSS.PROV Granular A or Granular B Type II, but with less than 5 percent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with OPSS.PROV 501 (Compaction). The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 500 millimetres in height. The culvert replacement should be designed for the full overburden pressure and live loads, assuming an embankment fill unit weight of 22 kN/m^3 for Granular A and 21 kN/m^3 for Granular B Type II. The performance of the box culvert is dependent on the construction procedures. Therefore, it is suggested that, if a box culvert is selected as the method of replacement, a Quality Verification Engineer should be retained to verify the construction procedures.

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall should be provided at the upstream and downstream ends of the culvert replacement. If clay seals are adopted, the clay material should meet the requirements of OPSS 1205 *(Material Specification for Clay Seal)*. The clay seals should have a thickness of 1 metre, and the seal should extend from a depth of 1 metre below the scour level to a minimum horizontal distance of 2 metres on either side of the culvert inlet/outlet opening, and a minimum vertical height equivalent to the maximum 100 year water level including treatment of the adjacent side slopes. Alternatively, clay blankets may be constructed, extending upstream/downstream to a distance equal to three times the culvert height. Normally, a clay blanket would extend along the adjacent embankment side slopes to a height of two times the culvert height or the high water level, whichever is higher; however, at this site where the cover over the culvert is relatively limited, it is recommended that a clay blanket, if adopted, extend to the top of the embankment side slope. If a cast-in-place concrete cut-off wall is adopted it should extend the full width of the culvert. The concrete cut-off should have a thickness of 400 millimetres and extend to a depth of 1.2 metres below the scour level. The cut-off walls should be earth formed within trenches cut for their construction or precast and backfilled with compactable clay to maintain intimate contact between the concrete and the native low permeability soils.

If the creek flow velocities are sufficiently high, a provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert inlet and outlet. The requirements for and design of erosion protection measures for the culvert inlet should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlet should be consistent with the standard Treatment Type A presented in OPSD 810.010 *(Rip-Rap Treatment for Sewer and Culvert Outlets),* with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket if adopted, including the drain side slopes and embankment fill slope adjacent to the culvert.

6.6.4 Subgrade Protection

The clay that is exposed at the founding/subgrade level will be susceptible to disturbance from construction traffic and ponded water.

Trafficking over the very soft to soft clay subgrade will not be possible. An Operational Constraint or a Non-Standard Special Provision should be included in the contract in this regard, which directs the contractor to not travel on the subgrade surface with equipment. Consideration could be given to providing a working slab some 100 millimetres thick of 20 MPa concrete.

The box culvert should be provided with a minimum 300 millimetres of OPSS.PROV Granular A bedding (as mentioned in Sections 6.2.1.1 and 6.6.3).

A sample NSSP for subgrade protection is provided in Appendix B.

6.6.5 Expanded Polystyrene Lightweight Fill Construction

In order to limit post-construction settlements to acceptable levels, lightweight backfill materials will need to be incorporated into the backfill of the culvert and wingwalls so that the final stress level in the underlying clay deposit would be maintained below existing conditions.

Considering the reduction in the loading that must be achieved (versus conventional earth fill construction), it is considered that expanded polystyrene (EPS) Geofoam will be the most practical lightweight fill type. Other lightweight fill materials could also be considered, such as blast furnace slag or cellular/foamed concrete, however it is considered that, in this case, the unit weights of these materials are not sufficiently low to achieve the needed reductions in the final stress level.

The Geofoam will need to be covered with a concrete slab to protect it from being overstressed by the traffic loads; overstressing of the Geofoam could lead to rutting of the pavement surface. A concrete slab thickness of 125 millimetres is typical for the protective slab. The EPS suppliers should be consulted for further requirements for the concrete slab.

The EPS Geofoam is potentially soluble in hydrocarbons. To guard against dissolution of the EPS in the case of an accidental release and infiltration of fuel (such as could occur in the case of a collision), it is general practice to cover the outside surface of the EPS with 10 mil polyethylene sheeting.

A 300 millimetres thick layer of OPSS.PROV Granular A or Granular B Type II would be appropriate as a levelling pad beneath the EPS Geofoam.

6.6.6 Existing Utilities – Settlements and Vibration Monitoring

There are several utilities in the area of the proposed culvert replacement. It is understood that the utilities will be relocated with the exception of the two Enbridge high-pressure gas mains. A discussion on the impact of the potential settlements to the existing pipelines is included in Section 6.4.3.

Vibration monitoring should be carried out during construction activities including sheet pile installation and compaction works to ensure that the vibration levels at the existing pipelines are maintained below tolerable levels set by the utility owner(s). A Non Standard Special Provision for vibration monitoring should be included in the contract documents. A NSSP for vibration monitoring as well as a vibration monitoring plan have been included in Appendix C of this report.

7.0 CLOSURE

This preliminary report was prepared by Ms. Kim Lesage, P.Eng., and the technical aspects reviewed by Mr. Kevin Nelson, P.Eng. and Associate. Mr. Fintan Heffernan P.Eng., the designated MTO contact for this project, conducted an independent review of this report.

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Table 1

Comparison of Foundation Alternatives

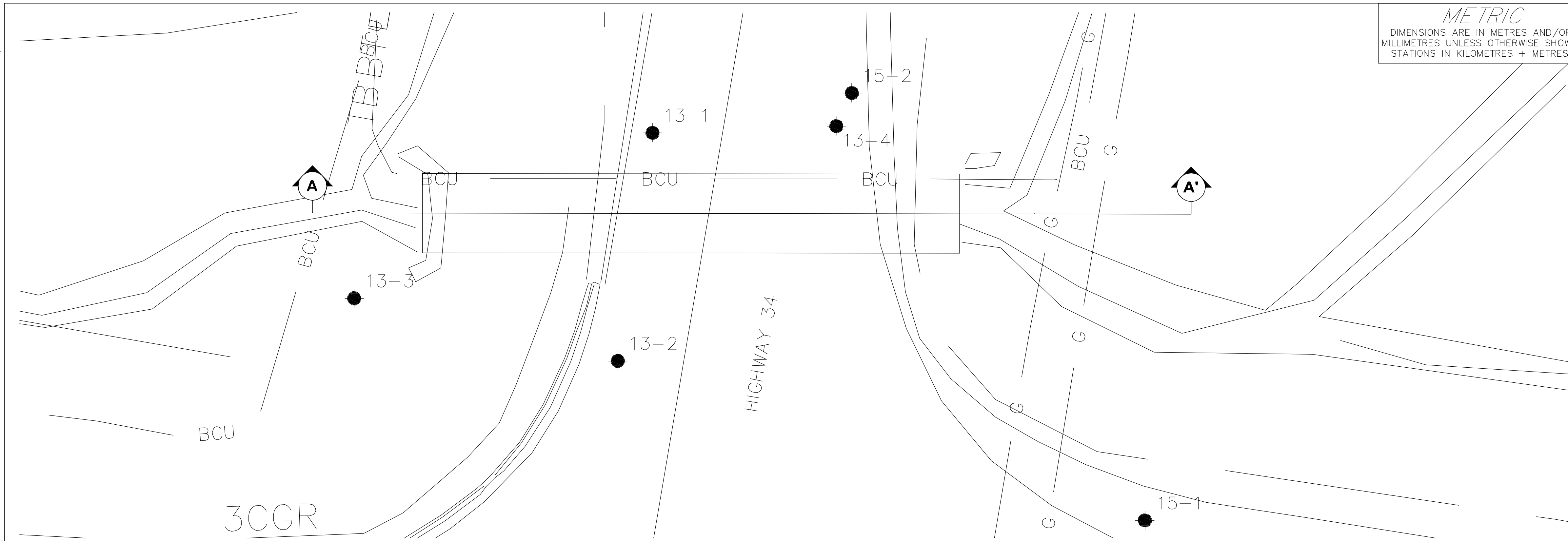
W.P. 4110-11-01

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1 Concrete Box	▪ Feasible	▪ Minimizes excavation depth ▪ Can be designed to reduce settlement magnitudes. ▪ Long design life	▪ Need to optimize culvert dimensions and incorporate lightweight backfill materials to minimize foundation stresses to current levels ▪ Deeper founding levels required for hydraulic design could result in higher settlement magnitudes adjacent to culvert ▪ Roadway protection (i.e., excavation shoring) needed, due to traffic staging. System may be complex and require non-standard design	▪ Moderate cost	▪ Generally low risk option (except for shoring) ▪ Risk of conflict with contractor over design of shoring system
Option 2 Rigid Frame Open Footing	▪ Not feasible	▪ N/A	▪ Large settlements due to higher concentration stresses from narrow foundations	▪ Moderate cost	▪ High risk option
Option 3 Deep Foundations	▪ Feasible but not required/practical	▪ Would not result in culvert settlement	▪ Would require relatively deep piles ▪ Culvert settlement would not conform to ground settlements. Could result in roadway distortion / differential settlement ▪ Also requires roadway protection system (same comments as for Option 1)	▪ Expensive option	▪ Low risk option

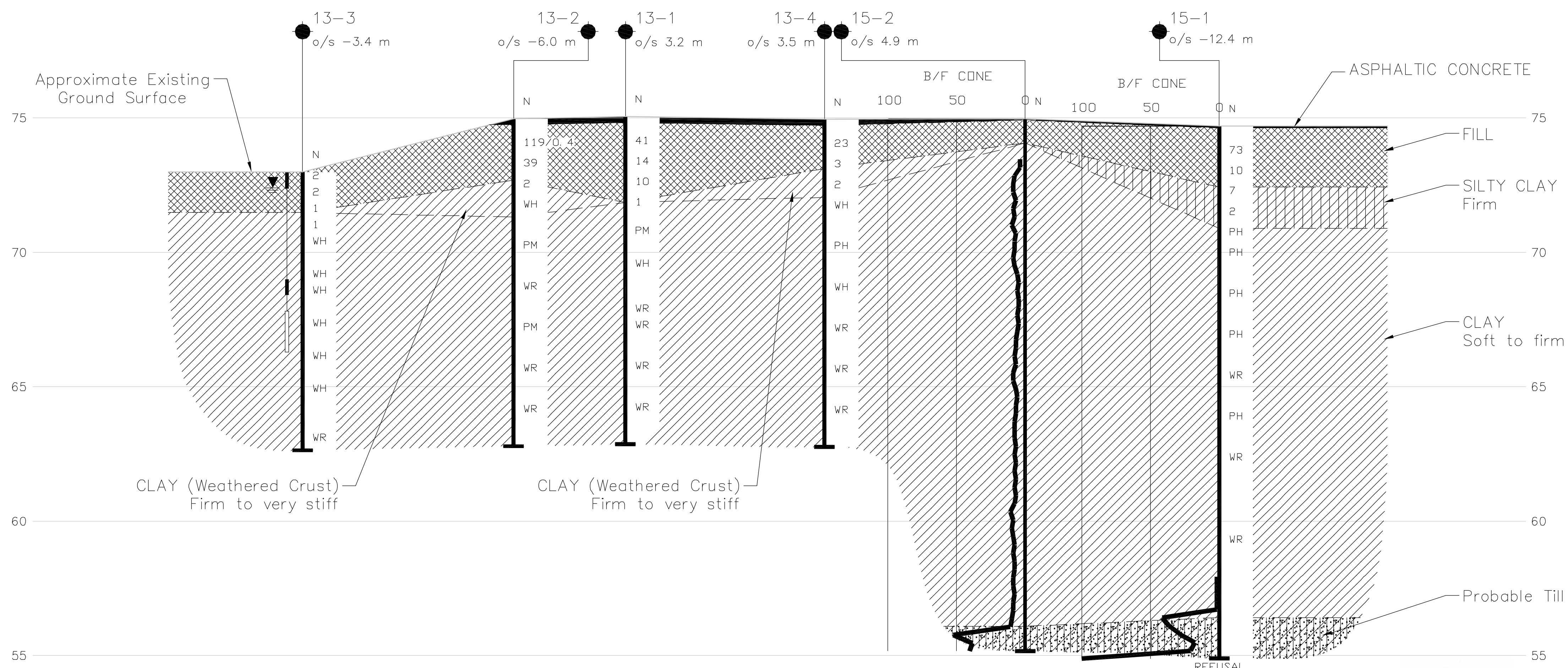
HIGHWAY 34 CULVERT BOREHOLE LOCATIONS AND SOIL STRATA

CONT No. 2015–4011
WP No. 4108–11–00
SHEET 17
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

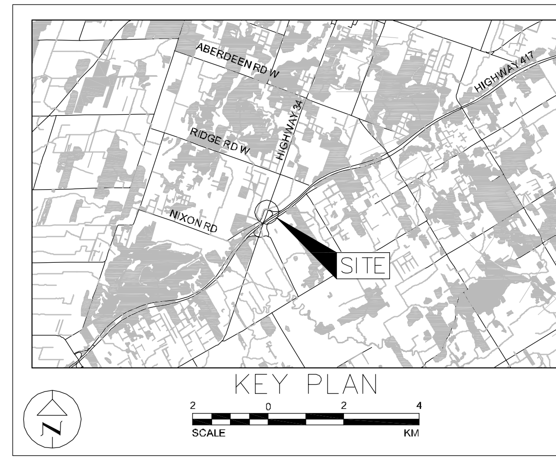
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.



PLAN
SCALE 2 0 2 4 m



A-A' CULVERT SECTION
SCALE 2 0 2 4 m



KEY PLAN
SCALE 2 0 2 4 KM

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- WL in piezometer, measured on July 10, 2013
- Dynamic Cone Penetration Test

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
13–1	75.1	5037437.2	212642.5
13–2	75.0	5037428.4	212639.6
13–3	73.0	5037432.8	212629.5
13–4	74.9	5037436.2	212649.9
15–1	74.7	5037418.4	212659.4
15–2	75.0	5037437.4	212650.7

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plan provided in digital format by Dillon, drawing file no. Contract B –BasePlan.dwg, received March 19, 2015.

NO.	DATE	BY	REVISION

Geocres No. 31G–247

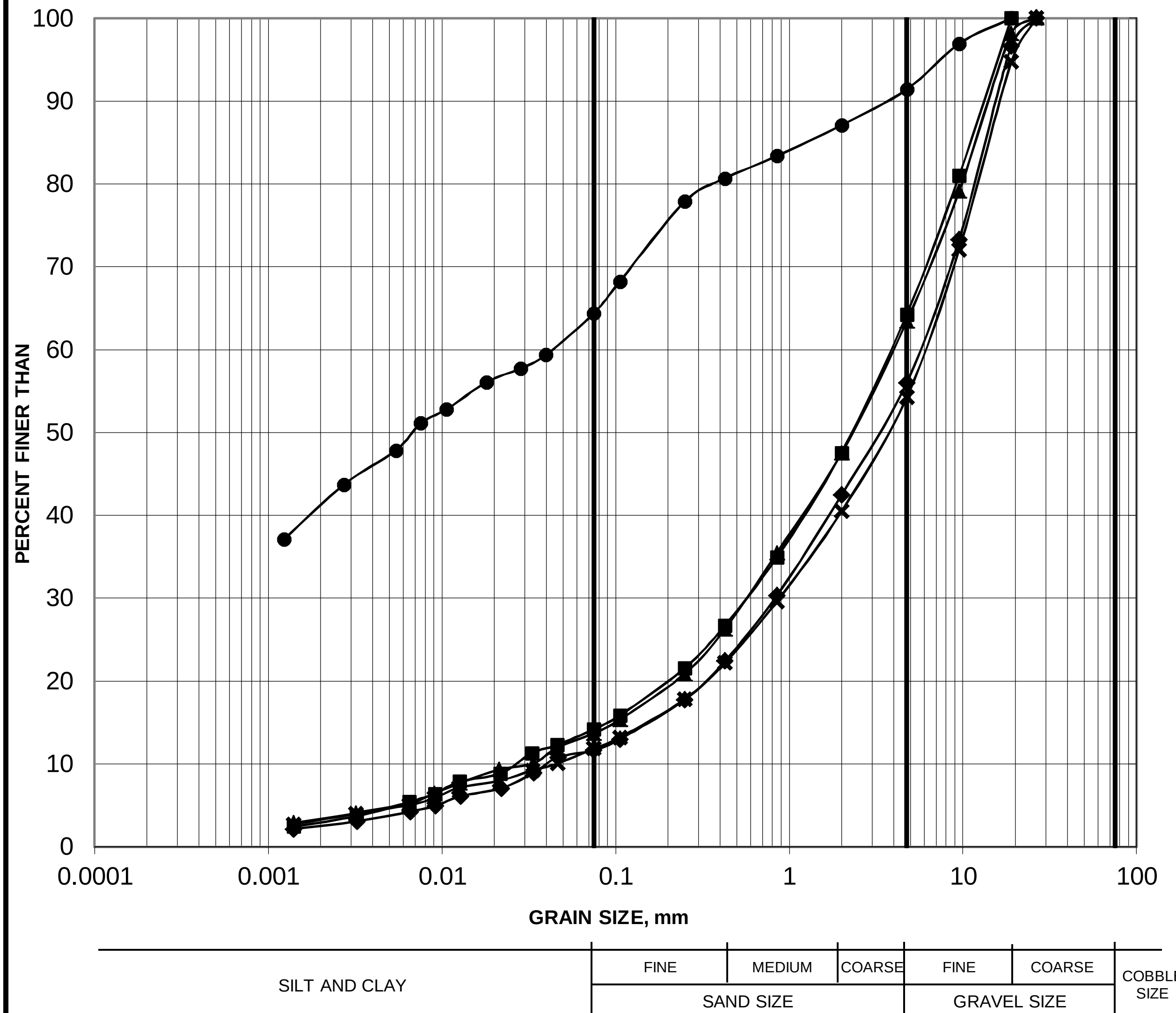
HWY. 34	PROJECT NO. 12–1121–0193		DIST.
SUBM'D. KSL	CHKD. KSL	DATE: 4/17/2015	SITE: 27–266/C
DRAWN: JFC/JM	CHKD. KSL	APPD. FJH	DWG. 1

PLOT DATE: April 17, 2015
FILENAME: N:\Active\2012\1121 – Geotechnical\12–1121–0193 Dillon Mega 3 Eastern Region\Spatial_IM\CAD\–NA– Unnamed Creek Culvert PH 1311\12112101930A001.dwg

LICENSED PROFESSIONAL ENGINEER K. S. LESAGE 100131998 Apr 24, 2015 PROVINCE OF ONTARIO
REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN 22/04/2015 PROVINCE OF ONTARIO

GRAIN SIZE DISTRIBUTION	FIGURE 1

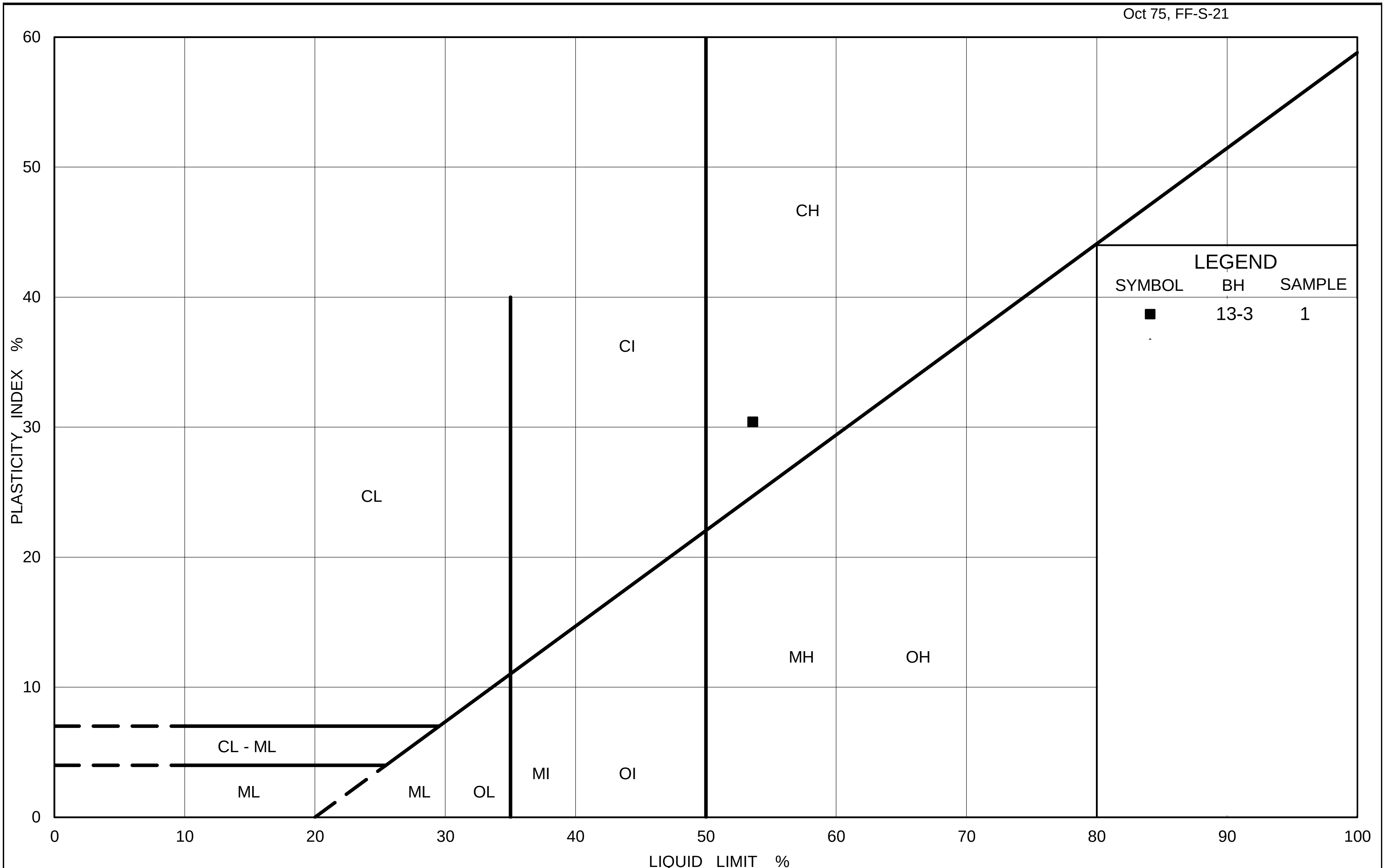
FILL



Borehole	Sample	Depth (m)
13-1	1	0.76-1.37
13-1	3	2.29-2.90
13-2	2	1.52-2.13
13-3	2	0.61-1.22
13-4	1	0.76-1.37

Project: 12-1121-0193-T1310 **Golder Associates**

Created by: CW
Checked by: CNM



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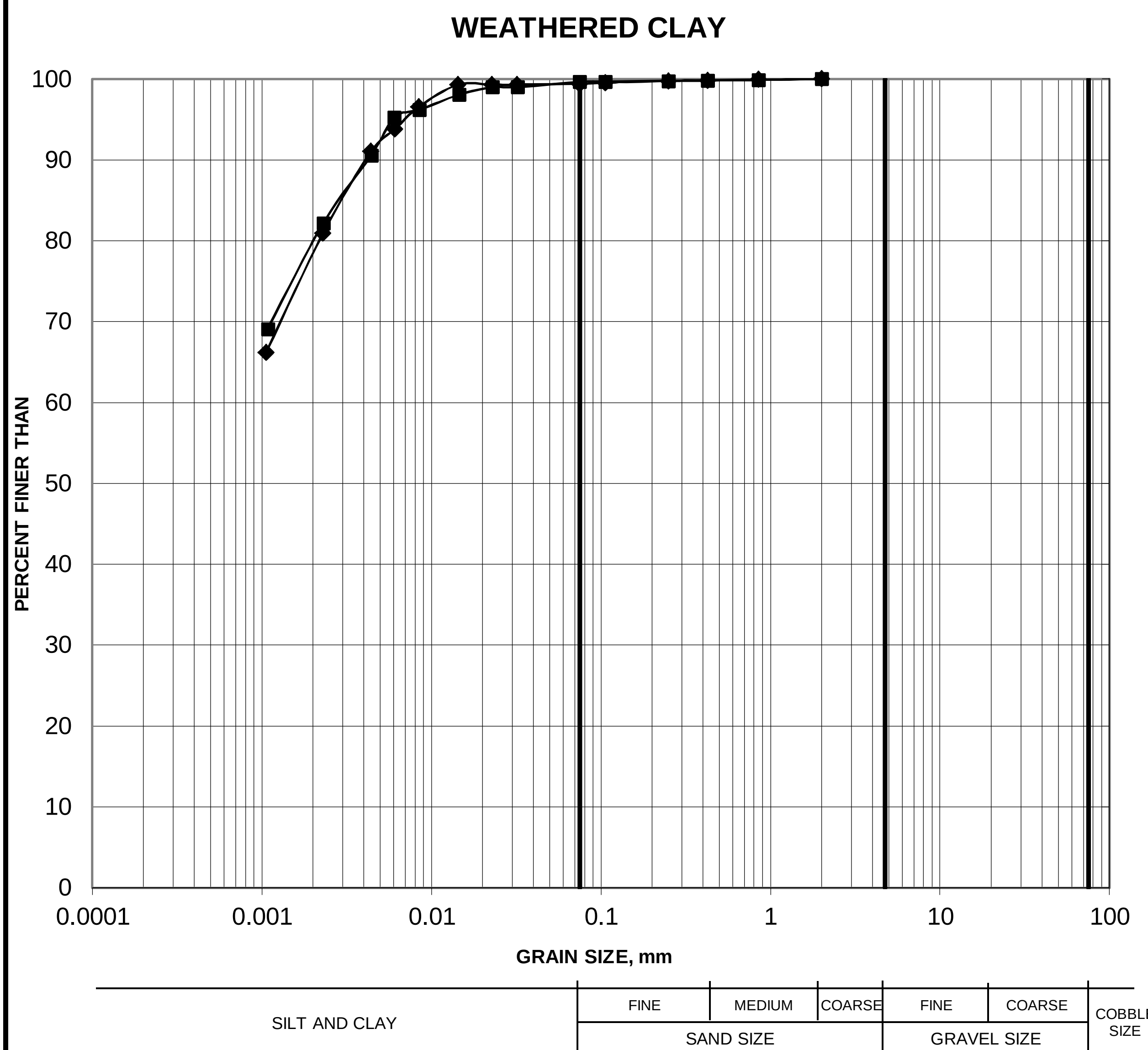
PLASTICITY CHART
FILL

FIG No. 2

Project No. 12-1121-0193-T1310

Compiled By: CW Checked By: CNM

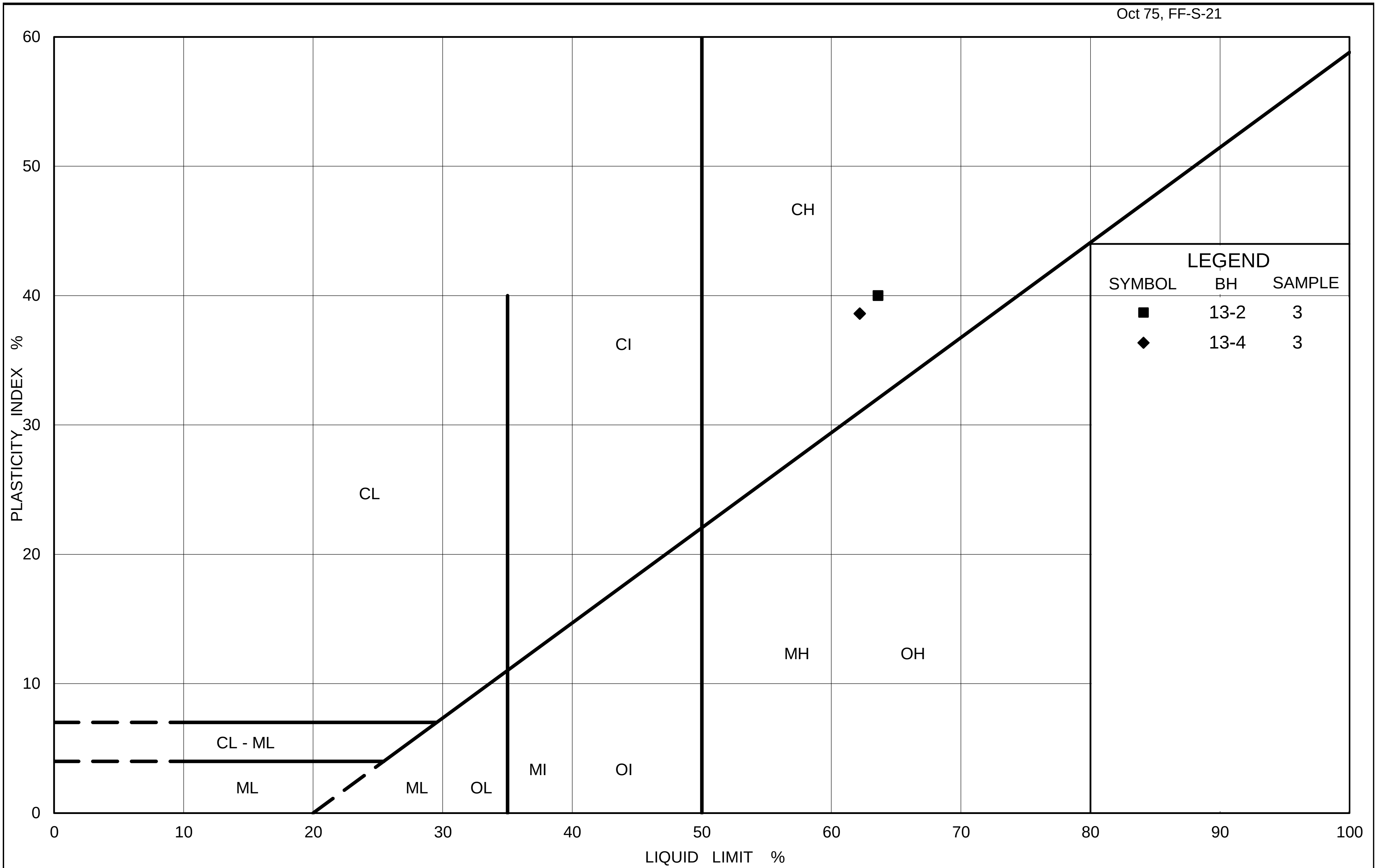
GRAIN SIZE DISTRIBUTION	FIGURE 3



Borehole	Sample	Depth (m)
13-2	3	2.29-2.90
13-4	3	2.29-2.90

Project: 12-1121-0193-T1310 **Golder Associates**

Created by: CW
Checked by: CNM



Ministry of Transportation

Ontario

PLASTICITY CHART
Weathered Clay

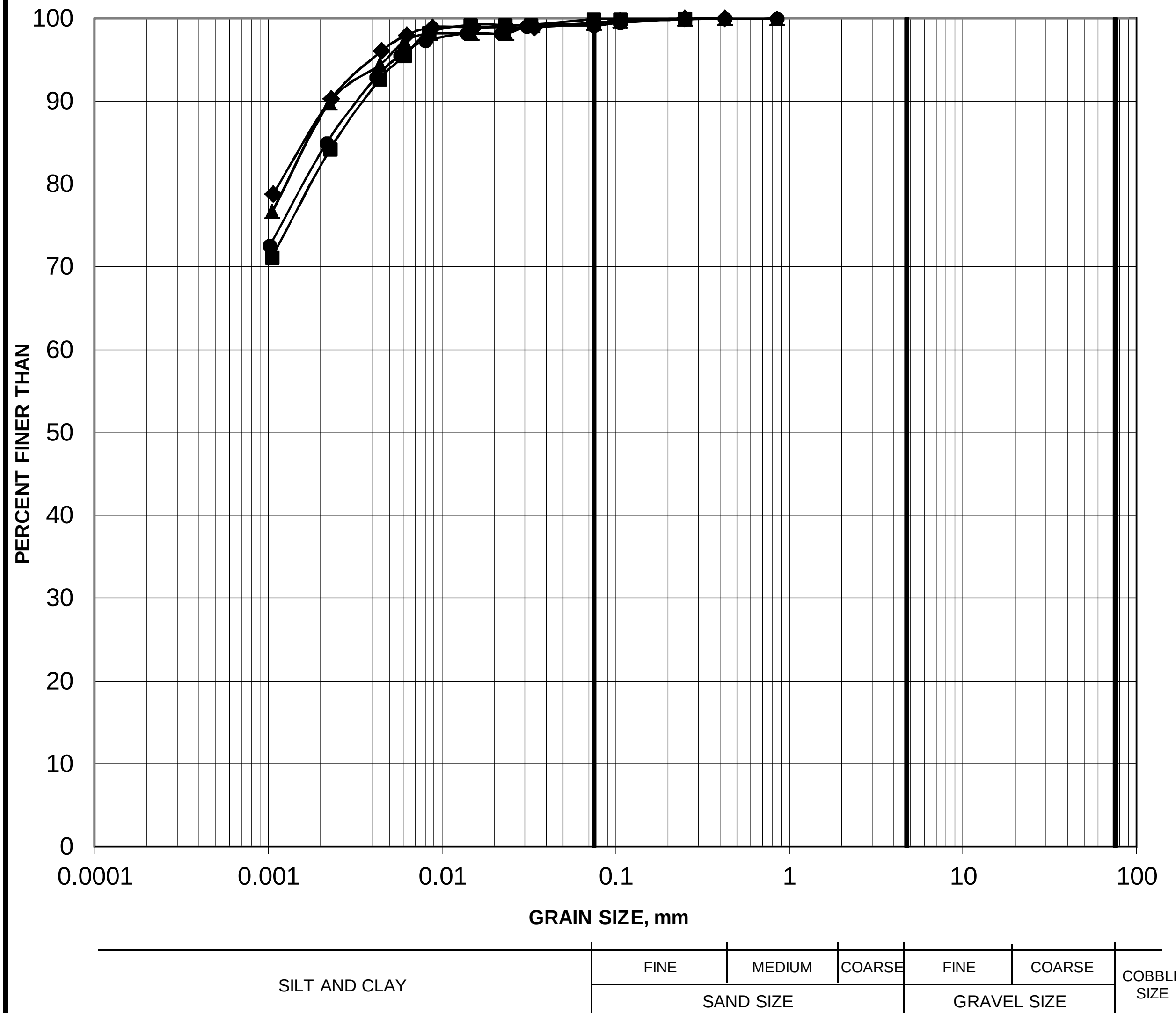
FIG No. 4

Project No. 12-1121-0193-T1310

Compiled By: CW Checked By: CNM

GRAIN SIZE DISTRIBUTION	FIGURE 5

UNWEATHERED CLAY

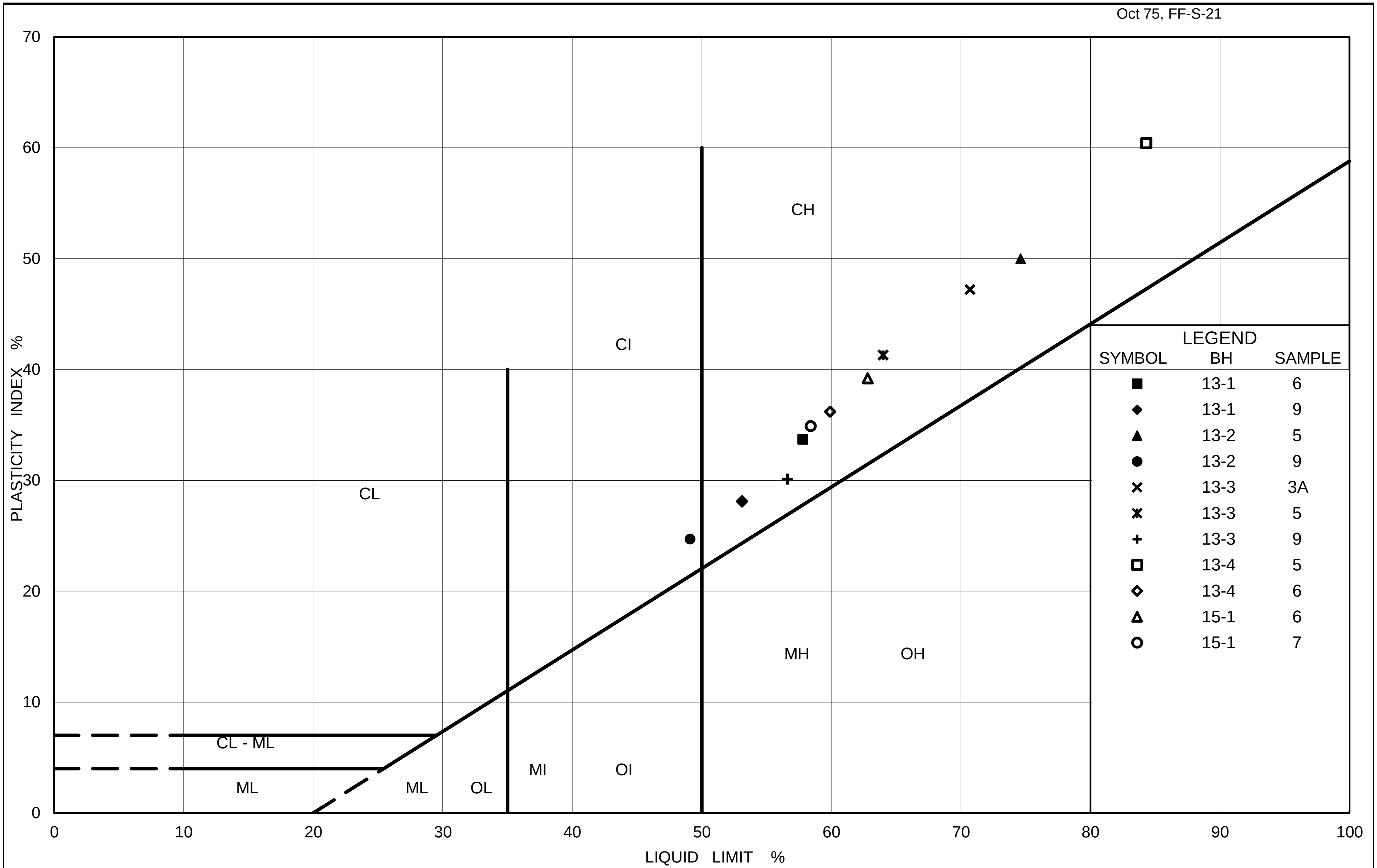


Borehole	Sample	Depth (m)
13-1	6	5.34-5.95
13-2	9	10.67-11.28
13-3	9	6.71-7.32
13-4	6	6.10-6.71

Project: 12-1121-0193-T1310

Golder Associates

Created by: CW
Checked by: CNM



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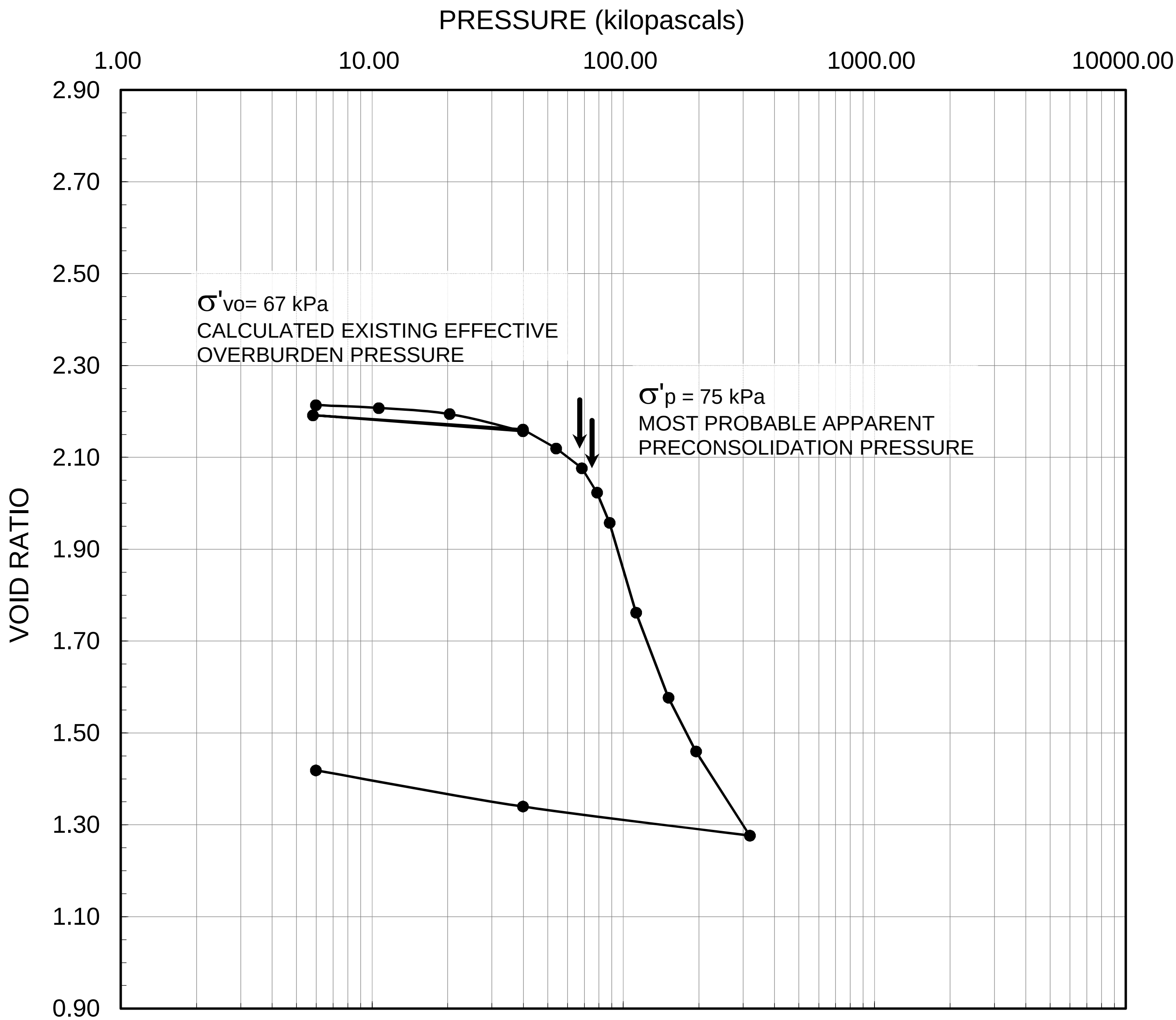
Ontario

PLASTICITY CHART
Unweathered Clay

FIG No. 6

Project No. 12-1121-0193-T1310

Compiled By: MI Checked By: CNM

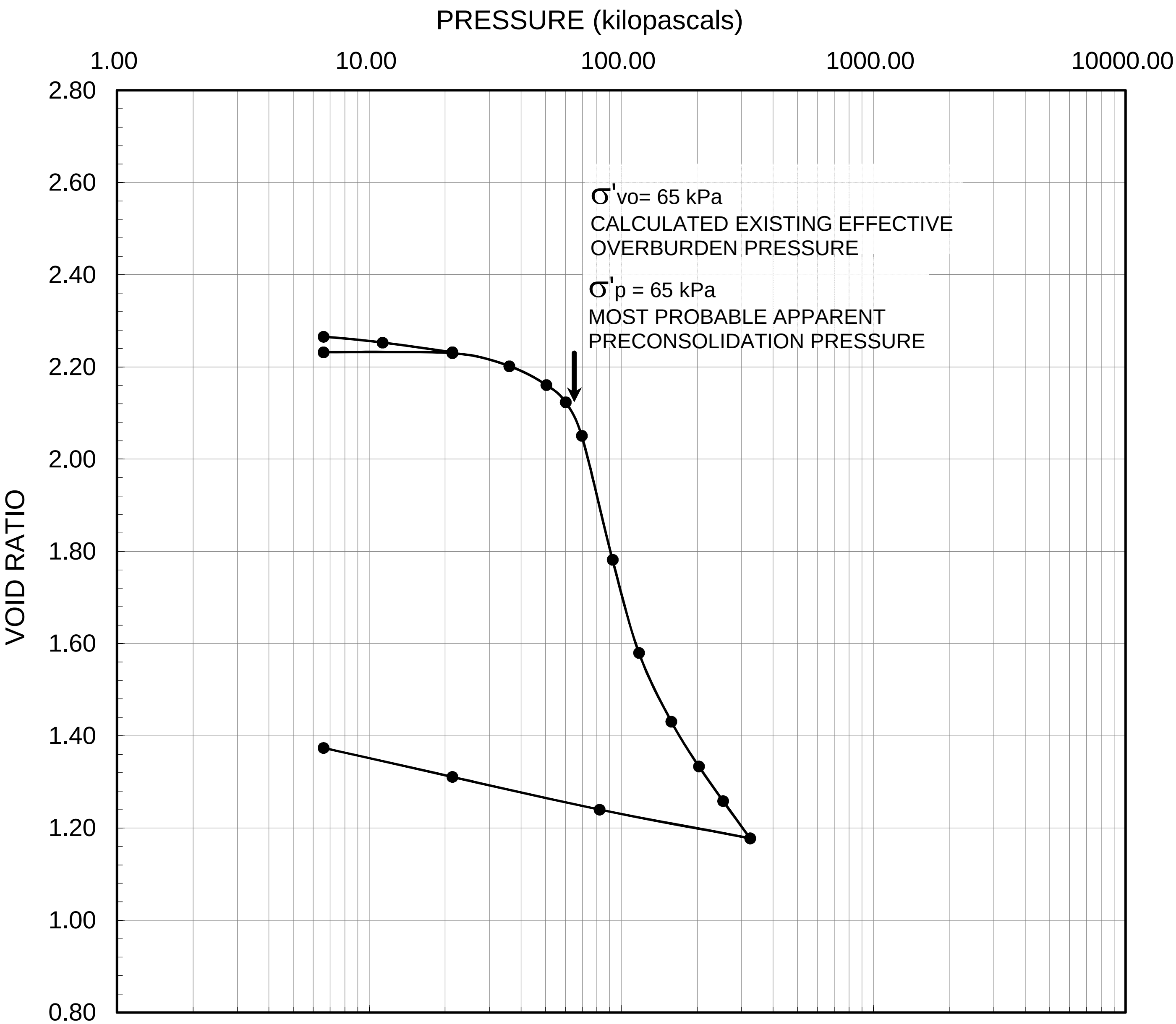


LEGEND

Borehole:	13-4	w_i = 80%	S_o = 100%	γ = 15.2 kN/m³
Sample:	5	w_f = 54%	e_o = 2.22	G_s = 2.77
Depth (m):	4.57-5.03	w_l = 84%	C_c = 1.63	
Elevation (m):	69.9-70.3	w_p = 24%	C_r = 0.037	

Golder Associates

SCALE	AS SHOWN	TITLE	
DATE	08/26/13		
CADD	N/A	**CONSOLIDATION TEST RESULTS**	
ENTERED	MI		
FILE No. Consolidation summary	CHECK	CNM	
PROJECT No. 12-1121-0193-1310 REV. 2	REVIEW	KSL	FIGURE 7



LEGEND

Borehole:	15-1	w_i = 82%	S_o = 101%	γ = 15.2 kN/m³
Sample:	7	w_f = 53%	e_o = 2.27	G_s = 2.79
Depth (m):	6.10-6.58	w_l = 58%	C_c = 1.93	
Elevation (m):	68.6-68.1	w_p = 24%	C_r = 0.004	

Golder Associates

SCALE	AS SHOWN
DATE	03/26/15
CADD	N/A
ENTERED	MI
CHECK	CNM
REVIEW	KSL

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

PROJECT No. 12-1121-0193-1321 REV. 1

FIGURE **8**

APPENDIX A

List of Abbreviations and Symbols
Record of Borehole Sheets

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures, and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open-ended, driven or pushed tube samplers
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual tube sample
DD	Diamond drilling

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive an uncased 50 mm (2 in.) diameter, 60^0 cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

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

Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	**C_u or S_u** kPa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	Water content
w_p or PL	Plastic limited
w_1 or LL	Liquid limit
C	Consolidaiton (oedometer) test
CHEM	Chemical analysis (refer to text)
CID	Consolidated isotropically drained triaxial test[1]
CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	Relative density
DS	Direct shear test
Gs	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
SO_4	Concentration of water-soluble sulphates
UC	Unconfined compression test
UU	Unconsolidated undrained triaxial test
V	Field vane test (LV-laboratory vane test)
γ	Unit weight

Note: [1] Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$ x or log x	logarithm of x to base 10
g	acceleration due to gravity
t	time
FOS	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

w	water content
w_1 or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity Index = $(w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_c	consistency index = $(w_1 - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:
[1] $\tau = c' + \sigma' \tan \phi'$
[2] shear strength = (compressive strength) / 2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of rock material weathering
Faintly Weathered: weathering limited to the surface of major discontinuities.
Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.
Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable
Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.
Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very Thickly Bedded	> 2 m
Thickly Bedded	0.6 m to 2m
Medium Bedded	0.2 m to 0.6 m
Thinly Bedded	60 mm to 0.2 m
Very Thinly Bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly Laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very Wide	> 3 m
Wide	1 – 3 m
Moderately Close	0.3 – 1 m
Close	50 – 300 mm
Very Close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns – 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	< 2 microns

Note: *Grains > 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^0 angle is horizontal.

Description and Notes
An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

BD -	Bedding	PY -	Pyrite
FO -	Foliation/Schistosity	Ca -	Calcite
CL -	Clean	PO -	Polished
SH -	Shear Plane/Zone	K -	Slickensided
VN -	Vein	SM -	Smooth
FLT -	Fault	RO -	Ridged/Rough
CO -	Contact	ST -	Stepped
JN -	Joint	PL -	Planar
FR -	Fracture	IR -	Irregular
MB -	Mechanical Break	UN -	Undulating
BR -	Broken Rock	CU -	Curved
BL -	Blast Induced	TCA -	To Core Axis
II -	Parallel To	STR -	Stress Induced
OR -	Orthogonal		

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Foundation Design

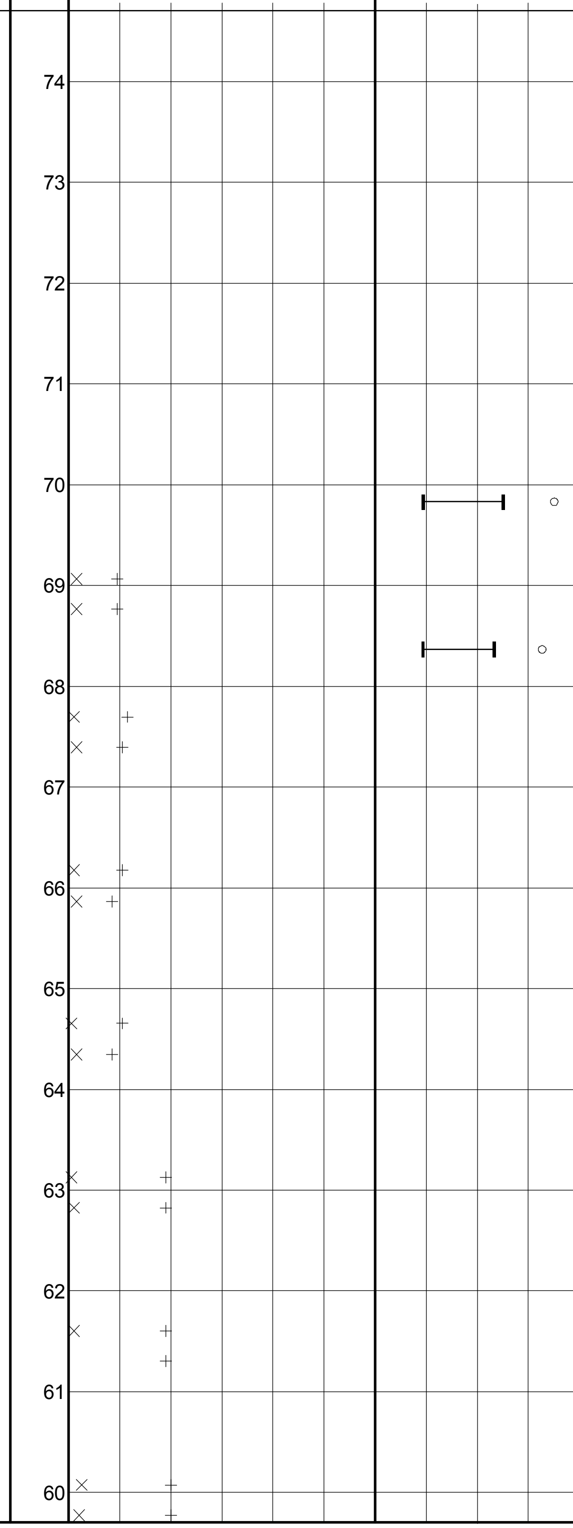
RECORD OF BOREHOLE No 15-1

SHEET 1 OF 2 **METRIC**

PROJECT 12-1121-0193-1311
W.P. 4108-11-00
LOCATION N 5037418.4 ;E 212659.4
ORIGINATED BY DWM
DIST HWY 34
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/DCPT
COMPILED BY JM
DATUM Geodetic
DATE January 27, 2015
CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
74.7	GROUND SURFACE				
0.0 / 0.1	ASPHALTIC CONCRETE				
	Gravelly silty sand (FILL) Very dense Grey-brown Moist	1	SS	73	
73.2 / 1.5	Silty sand, some clay, trace gravel (FILL) Loose to compact Grey Dry	2	SS	10	
72.4 / 2.3	SILTY CLAY, trace sand and organic matter Greenish-grey Firm Moist	3	SS	7	
		4	SS	2	
70.9 / 3.8	CLAY Soft Grey Moist to wet	5	SS	PH	
		6	TP	PH	
		7	TP	PH	15.2
		8	SS	PH	
		9	TP	WR	
		10	TP	PH	
63.1 / 11.6	CLAY Firm Grey Wet	11	SS	WR	

Columns: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



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+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 1311\1211210193-1311.GPJ GAL-GTA.GDT 04/17/15 JM

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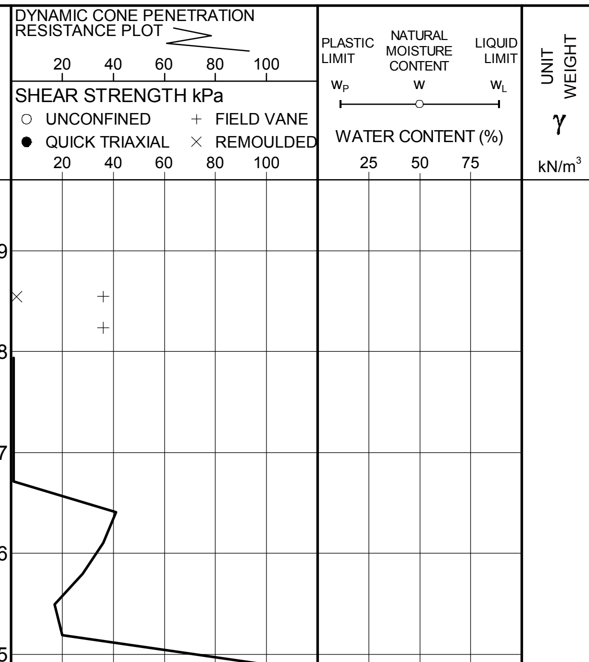
Foundation Design

RECORD OF BOREHOLE No 15-1

SHEET 2 OF 2 **METRIC**

PROJECT 12-1121-0193-1311
W.P. 4108-11-00 LOCATION N 5037418.4 ;E 212659.4 ORIGINATED BY DWM
DIST HWY 34 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/DCPT COMPILED BY JM
DATUM Geodetic DATE January 27, 2015 CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---							
	CLAY Firm Grey Wet		12	SS	WR		59	
57.9 16.8	Probable Silty Clay						58 57	
56.4 18.3	Probable Till						56 55	
54.9 19.8	END OF BOREHOLE DCPT REFUSAL							



+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 1311\1211210193-1311.GPJ GAL-GTA.GDT 04/17/15 JM

Golder Associates

RECORD OF BOREHOLE No 15-2

SHEET 1 OF 2

METRIC

PROJECT 12-1121-0193-1311
W.P. 4108-11-00
LOCATION N 5037437.4 ;E 212650.7
ORIGINATED BY DWM
DIST HWY 34
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/DCPT
COMPILED BY JM
DATUM Geodetic
DATE January 27, 2015
CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.0	GROUND SURFACE										
0.0	ASPHALTIC CONCRETE										
0.1	Gravelly sand (FILL) Grey-brown										
74.1 0.9	CLAY Grey						74				
73.5 1.5	Probable Silty Clay						73				
							72				
							71				
							70				
							69				
							68				
							67				
							66				
							65				
							64				
							63				
							62				
							61				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 1311\1211210193-1311.GPJ GAL-GTA.GDT 04/17/15 JM

Golder Associates

RECORD OF BOREHOLE No 15-2

SHEET 2 OF 2 **METRIC**

PROJECT 12-1121-0193-1311
W.P. 4108-11-00 LOCATION N 5037437.4 ;E 212650.7 ORIGINATED BY DWM
DIST HWY 34 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/DCPT COMPILED BY JM
DATUM Geodetic DATE January 27, 2015 CHECKED BY KSL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L / WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Probable Silty Clay						59 58 57				
56.1 18.9	Probable Till						56				
55.2 19.8	END OF BOREHOLE										

GTA-MTO 001 N:\ACTIVE\2012\1121 - GEOTECHNICAL\12-1121-0193 DILLON MEGA 3 EASTERN REGION\SPATIAL_IM\GINT\PHASE 1311\1211210193-1311.GPJ GAL-GTA.GDT 04/17/15 JM

+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

![Golder Associates]

Foundation Design

RECORD OF BOREHOLE No 13-1

SHEET 1 OF 2 **METRIC**

PROJECT 12-1121-0193-1310
W.P. 4108-11-00 LOCATION N 5037437.2 ;E 212642.5 ORIGINATED BY HEC
DIST HWY 34 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) COMPILED BY JM
DATUM Geodetic DATE May 28-30, 2013 CHECKED BY KSL

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.1	GROUND SURFACE				
0.0 / 74.9	ASPHALTIC CONCRETE				
	Crushed stone (FILL), Grey				
0.4	Sand and gravel, trace silt and clay (FILL), Dense to compact, Brown, Moist	1	SS	41	35 50 11 3
		2	SS	14	
72.8 / 2.3	Sand and gravel, trace silt and clay (FILL), Compact to loose, Grey-brown, Most to wet	3	SS	10	44 44 10 2
71.9 / 3.2	SILTY CLAY, Firm to stiff, Grey-brown to grey, Wet	4	SS	1	
71.4 / 3.7	SILTY CLAY, Soft, Grey, Wet	5	SS	PM	
		6	SS	WH	0 0 16 84
		7	TP	WR	
		8	SS	WR	
66.6 / 8.5	SILTY CLAY, Firm, Grey, Wet	9	SS	WR	

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (75 to 66); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³.

Continued Next Page

+3, ×3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 13-1

METRIC

PROJECT 12-1121-0193-1310
W.P. 4108-11-00
LOCATION N 5037437.2 ;E 212642.5
ORIGINATED BY HEC
DIST HWY 34
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)
COMPILED BY JM
DATUM Geodetic
DATE May 28-30, 2013
CHECKED BY KSL

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	PLASTIC LIMIT / NATURAL MOISTURE CONTENT / LIQUID LIMIT (WATER CONTENT %)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY Firm Grey Wet						65				
			10	SS	WR		64				
							63				
62.9 12.2	END OF BOREHOLE										

Legend: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED

+ 3, × 3: Numbers refer to Sensitivity
○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 13-2

SHEET 1 OF 2

METRIC

PROJECT 12-1121-0193-1310

W.P. 4108-11-00 | LOCATION N 5037428.4 ;E 212639.6 | ORIGINATED BY HEC

DIST | HWY 34 | BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) | COMPILED BY JM

DATUM Geodetic | DATE May 28, 2013 | CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.0	GROUND SURFACE						
74.8 0.0	ASPHALTIC CONCRETE						
0.4	Crushed stone (FILL) Grey Sand and gravel, trace silt and clay (FILL) Very dense to dense Brown Moist	1	SS	119/0.4	74		
		2	SS	39	73	○	37 49 11 3
72.7 2.3	SILTY CLAY (Weathered Crust) Stiff to very stiff Grey-brown Moist to wet	3	SS	2			0 0 20 80
		4	SS	WH	72		
71.3 3.7	SILTY CLAY Firm to soft Grey Wet				71		
		5	SS	PM	70		
		6	SS	WR	69		
		7	SS	PM	68, 67		
		8	SS	WR	66		

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

Foundation Design

RECORD OF BOREHOLE No 13-2

SHEET 2 OF 2

METRIC

PROJECT 12-1121-0193-1310

W.P. 4108-11-00 LOCATION N 5037428.4 ;E 212639.6 ORIGINATED BY HEC

DIST HWY 34 BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem) COMPILED BY JM

DATUM Geodetic DATE May 28, 2013 CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---							
	SILTY CLAY Firm to soft Grey Wet							
			9	SS	WR		64	0 0 12 88
63.7 11.3	SILTY CLAY Firm Grey Wet						63	
62.8 12.2	END OF BOREHOLE							

Chart column headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³

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$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-3

SHEET 1 OF 2

METRIC

PROJECT 12-1121-0193-1310			
W.P. 4108-11-00	LOCATION N 5037432.8 ;E 212629.5		ORIGINATED BY HEC
DIST	HWY 34	BOREHOLE TYPE Portable Drill	COMPILED BY JM
DATUM Geodetic	DATE May 24, 2013		CHECKED BY KSL

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
73.0	GROUND SURFACE
0.0	Silty clay, trace to some sand, trace gravel (FILL) Stiff Grey-brown Moist to wet
71.5 1.5	SILTY CLAY Firm to stiff Grey Wet
70.6 2.4	SILTY CLAY Very soft Grey Wet
68.1 4.9	SILTY CLAY Soft Grey Wet
63.6 9.5	SILTY CLAY Soft to firm Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	2	
2	SS	2	9 27 22 42
3	SS	1	
4	SS	1	
5	SS	WH	
6	SS	WH	
7	SS	WH	
8	SS	WH	
9	SS	WH	0 0 12 88
10	SS	WH	

STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE: 72, 71, 70, 69, 68, 67, 66, 65, 64

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

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Golder Associates

PROJECT 12-1121-0193-1310

RECORD OF BOREHOLE No 13-3

METRIC

W.P. 4108-11-00 | LOCATION N 5037432.8 ;E 212629.5 | ORIGINATED BY HEC

DIST HWY 34 | BOREHOLE TYPE Portable Drill | COMPILED BY JM

DATUM Geodetic | DATE May 24, 2013 | CHECKED BY KSL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---		11	SS	WR						
62.6 10.4	END OF BOREHOLE NOTES: 1. Water level in open borehole at a depth of 0.6 m below ground surface (Elev. 72.4 m), measured on July 10, 2013.										

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$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 13-4

SHEET 1 OF 2 **METRIC**

PROJECT 12-1121-0193-1310
W.P. 4108-11-00
LOCATION N 5037436.2 ;E 212649.9
ORIGINATED BY HEC
DIST HWY 34
BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)
COMPILED BY JM
DATUM Geodetic
DATE May 30, 2013
CHECKED BY KSL

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ (kN/m³)	GR	SA	SI	CL
74.9	GROUND SURFACE								
0.0	ASPHALTIC CONCRETE								
74.7 / 0.2	Sand and gravel, trace silt and clay (FILL) Compact to very loose Brown Moist								
		1	SS	23		46	42	9	3
73.1 / 1.8	SILTY CLAY (Weathered Crust) Stiff to firm Grey-brown Moist to wet	2	SS	3					
		3	SS	2		0	0	21	79
72.0 / 2.9	SILTY CLAY Firm to soft Grey Wet								
		4	SS	WH					
		5	TP	PH	17.3				
		6	SS	WH		0	1	14	85
		7	SS	WR					
66.1 / 8.8	SILTY CLAY Firm Grey Wet								
		8	SS	WR					

ELEVATION SCALE: 74, 73, 72, 71, 70, 69, 68, 67, 66, 65

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

GROUND WATER CONDITIONS

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+3, ×3: Numbers refer to Sensitivity   ○3% STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-4

SHEET 2 OF 2

METRIC

PROJECT 12-1121-0193-1310

W.P. 4108-11-00

LOCATION N 5037436.2 ;E 212649.9

ORIGINATED BY HEC

DIST HWY 34

BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)

COMPILED BY JM

DATUM Geodetic

DATE May 30, 2013

CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY CLAY Firm Grey Wet										
			9	SS	WR		64				
							63				
62.7 12.2	END OF BOREHOLE										

Plot legend: SHEAR STRENGTH kPa — ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; scale 20 40 60 80 100. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75.

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

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APPENDIX B

Sample Non-Standard Special Provisions

CULVERT SETTLEMENT – Item No.

Non-Standard Special Provision

The clay subgrade soil at this site is near normally consolidated; therefore small changes in loading can result in significant settlement magnitudes. Therefore, additional increases in loading during construction (such as over-excavation at the subgrade level which would result in increased thicknesses of granular material or increased concrete thicknesses) and the creation of stockpiles of excavated materials should be avoided.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

GROUND WATER AND SURFACE WATER CONTROL – Item No.

Non-Standard Special Provision

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement to allow excavation and foundation construction to be carried out in dry conditions.

The surface water flow could be diverted by pumping from behind a temporary cofferdam(s) and passed through or around the culvert area by means of a temporary pipe. Large/heavy temporary cofferdams should not be constructed above the existing gas pipelines as this could lead to settlement of the pipelines. Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance and weakening of the sensitive clay subgrade soils.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

EXCAVATION SIDE SLOPES – Item No.

Non-Standard Special Provision

Temporary excavations for the culvert replacement will be made through the existing fill and are expected to extend into and terminate within the soft to firm unweathered clay deposit. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. The existing fill would be classified as Type 3 soil and the underlying soft to firm clay would be classified as Type 4 soil, based on the OHSA. According to OHSA, excavations that extend to, or into, Type 4 soils should be made with side slopes no steeper than 3 horizontal to 1 vertical (3H:1V). However, due to the nature of the underlying soft sensitive clay deposit at this site flatter excavation side slopes may be necessary.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

BASAL INSTABILITY OF SHORED EXCAVATIONS – Item No.

Non-Standard Special Provision

The shoring system for the culvert replacement must consider the soft clay deposit at depth and the potential for basal instability of the excavation. A basal instability failure could lead to the flow of sheared/disturbed clay into the excavation, significant ground deformation (settlement and ground slumping) behind the temporary support systems, and possible collapse of the shoring system. Therefore, the shoring system will need to extend below the excavation floor level sufficiently to prevent basal instability and off-loading of materials adjacent to the sheet piles could be required. In addition, the design of the sheeting projection would also need to resist the lateral loading imposed by the clay. This may require a very heavy/strong sheeting section.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

SUBGRADE PROTECTION – Item No.

Non-Standard Special Provision

The subgrade for the culvert foundations will be very susceptible to disturbance from construction traffic and ponded water. Following inspection and approval of the prepared subgrade, a 300 millimetres thick layer of OPSS Granular A shall be placed on the foundation subgrade for a box culvert.

The excavation for the bedding should be made using a smooth bladed bucket and the bedding should be compacted to 95 percent of the material's Standard Proctor maximum dry density using light 'walk behind' compaction equipment in loose lifts not less than 200 millimetres in thickness in accordance with SP105S10.

Construction traffic should not be permitted to travel on the subgrade.

Basis of Payment

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

END OF SECTION

APPENDIX C

Vibration Monitoring Plan
NSP - Vibration Monitoring
Settlement Monitoring Plan
NSP - Settlement Rods

TECHNICAL MEMORANDUM

DATE March 5, 2015 **PROJECT No.** 12-1121-0193-1311

TO Brad Craig, P.Eng.
Dillon Consulting Limited

FROM Daniel Corkery, B.Sc. **EMAIL** dcorkery@golder.com

HIGHWAY 34 CULVERT REPLACEMENT PROJECT
GROUND VIBRATION MONITORING OF THE ENBRIDGE GAS PIPELINES
OTTAWA, ONTARIO

Enbridge Gas Distribution (Enbridge) requires for settlement and vibration monitoring to be carried out as part of the culvert replacement project on Highway 34. There are two operating pipelines in the vicinity of the proposed construction:

- 6 inch diameter west pipe; and,
- 8 inch diameter east pipe.

Monitoring Program

In defining a vibration monitoring specification for this project, the following items need to be considered:

- A vibration monitoring specialist is needed for this program;
- The depth and location of pipelines and installed seismograph is critical relative to the proposed construction activities in providing representative data; and,
- Vibration intensity diminishes with distance and, thus, monitoring should be carried out at the nearest pipeline to the vibration source.

The vibration limits at the pipelines are outlined in Enbridge's document "Third Party Requirements in the Vicinity of Natural Gas Facilities" (October, 2007). A copy of the document, which outlines Enbridge's requirements, is attached to this memorandum. The vibration limits and policies for construction vibrations are outlined in Section 5.0 as follows:

- Section 2.1 – All work in the vicinity of gas pipelines must be approved by Enbridge.
- Section 5.1 – Prior to any pile driving or compaction operations within the vicinity of a gas pipeline, the potential damage to Enbridge Gas Distribution plant will be evaluated to ensure the uninterrupted operation and long-term safety of its underground facilities.
- Section 5.2 – The application must include the following information:
  - Name of project owner, general contractor and relevant sub-trades;
  - A copy of the permits, certificates or other forms required by municipal bylaws;

- Name of design engineer and a copy of plans issued for construction with detailed drawings identifying all affected natural gas facilities;
- The type of piles and equipment used; including the methods of control to prevent the deviation of the piles;
- Geo-technical reports and other pertinent information;
- A copy of the location of other public utilities such as telephone, cable TV, sewer and water mains, electrical services, etc.;
- If required, a technical report with appropriate analysis and prediction of the vibration levels according to the opinion of an independent Engineer specialized in vibration control and analysis;
- A clause stating that the work will be carried out by qualified personnel with appropriate experienced supervision; and,
- A clause stating that all vibration testing results, or other preventative control testing, will be submitted to Enbridge Gas Distribution on a regular basis, or upon request.

- Section 5.4
  - Prior to pile driving and/or compaction work, a site meeting shall be arranged with an authorized representative of the contractor and Enbridge representative to confirm details of the location of Enbridge's facilities and the proposed work.
  - Section 5.4 – The maximum vibration intensity measured at the nearest of the two pipelines shall be as follows:
    - The Peak Particle Velocity (PPV) measured on the pipeline, or at the closest point of the related structure with respect to the work, shall not exceed 50 mm/s.
    - The maximum displacement for the vertical and/or horizontal component corresponding to the above stated vibration intensity shall not exceed 50 mm at any given length of the pipeline in question.
  - The vibration monitoring reports of recorded intensities shall be provided on a regular basis or at the request of Enbridge.
  - If the velocity or displacement limits are exceeded, work in the vicinity shall stop and the cause of the exceedance shall be investigated. The operations shall resume only when the cause and remedy are established and with the approval of Enbridge.

The seismographs and the operation of the instruments shall comply with the attached International Society of Explosive Engineering (ISEE) documents a) Performance Specifications for Blasting Seismographs and a) Field Practice Guidelines for Blasting Seismographs.

Burial of the geophone (velocity transducer) in the ground above the nearest pipeline (at the nearest location to the vibration source) is the most common method of attaching the sensor for ground vibration monitoring. The preferred burial method entails excavating a hole that is no less than three times the height of the sensor, spiking the sensor to the bottom of the hole, and firmly compacting soil around and over the sensor. This allows for proper vibration monitoring while enabling quick movement of the instrument along the pipeline length as the construction proceeds. If level vibration exceeds the limits described above (i.e. 50 mm/s), the nearest of the nearest pipeline should be day lighted and the sensor be mounted on the pipeline to ensure the most accurate measurement of vibrations at the pipeline.

Special Considerations

Vibration monitoring shall be conducted for the work carried out within the following minimum separation distances between the vibration source and the nearest pipeline:

- 6 m for compaction of soils or backfill rated at 10,000 ft-lbs or higher;
- 10 m for pile driving; and,
- 30 m for high-energy dynamic compaction for the rehabilitation of soils.

No operations shall be permitted within a separation distance of 1.5 m from the pipeline unless approved by Enbridge.

Closure

We trust that this memorandum provides sufficient information at this time. If you have any questions or comments regarding this memorandum, please feel free to contact the undersigned.

Yours truly,

GOLDER ASSOCIATES LTD.

Paul McAnuff
Senior Technician

Daniel Corkery, B.Sc.
Senior Blasting Consultant

DJC/PLM/KSL/bg

n:\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\foundations\9 - technical memorandums\12-1121-0193-1311 hwy vibration monitoring plan 2015-03-05.docx

Attachments: Enbridge's Document "Third Party Requirements in the Vicinity of Natural Gas Facilities" dated October 2007
International Society of Explosive Engineering (ISEE) Documents

Third Party Requirements

In the Vicinity of

Natural Gas Facilities

- **General Requirements**
- **Support of Gas Pipelines**
- **Blasting Requirements**
- **Pile Driving or Compaction Requirements**
- **Heavy Equipment Operation in the Vicinity of Gas Pipelines**

October 2007

TABLE OF CONTENTS

TABLE OF CONTENTS

1.0 DEFINITIONS

Terms used in the following Guideline are defined as follows unless otherwise specified:

Company	- Enbridge Gas Distribution Inc. or any of its representatives
LDC	- Local Distribution Company
Contractor or Excavator	- Any individual, partnership, corporation, public agency or other entity that dig, bore, trench, grade excavate or break ground with mechanical equipment or explosives in the vicinity of a gas pipeline or related facility.
Facility	- Defined as any Enbridge Gas Distribution Inc. Company Pipeline (main or service), regulator station or storage facility and their related components
Pile	- Any vertical or slightly slanted structural member introduced or constructed in the soil in order to transmit loads and forces from the superstructure to the subsoil; the structural member can also be used as a component of a retaining wall system
Pile Driving	- The placement of piles carried out by gravity hammer, vibratory hammer, auguring, pressing, screwing or any combinations of the above methods
Surface Blasting	- An operation involving the excavation of rock foundations for various types of structures, grade construction for highways or railroads, canals (trenches) for water supply or collection purposes.
Tunnel Blasting	- Operations involving the piercing of below ground (generally horizontal) opening in rock.
Blaster	- The person or persons responsible for setting the charges and performing the blast.
Applicant	- The owner of the proposed work
Compaction	- Any vibration generating operation which will result in a potential increase of the density of soils or controlled backfill materials. The means to increase the density may be static or dynamic
Engineer, Independent blasting consultant	- A Professional Engineer who is registered as a member of the Professional Engineers of Ontario (PEO) and a holder of Certificate of Authorization (C of A)
Construction Operations	- Activities associated with excavation, blasting, piling or compaction
Vicinity	- A horizontal distance of 30 meters, or less, from any Enbridge Gas Distribution Inc. natural gas facility (above-ground or below-ground)

2.0 GENERAL REQUIREMENTS

2.1 WORK IN THE VICINITY OF GAS PIPELINES

All work in the vicinity of gas pipelines must be approved by Enbridge Gas Distribution (the "Company").

All work within 30.0 metres of an NEB operated pipeline right-of-way must have the approval from Enbridge. This is a requirement of all NEB pipelines, which are under the jurisdiction of the National Energy Board, and follows the NEB Pipeline Crossing Regulations.

A stake out of the gas pipeline must be requested prior to any Construction. Call Ontario One Call at 1-800-400-2255 or 905-709-1717 at least 48 hours in advance of the proposed work.

Mechanical equipment shall not be operated within 0.3 m of the pipeline. Hand Excavation shall be performed when locating and digging within 0.3 m of the pipeline.

Mechanical excavation is not permitted within 3.0 m of the NEB or Vital pipelines without the approval of Enbridge.

Hand held compaction equipment shall be used within 1.0 m of the sides or top of all gas pipelines.

Spoil from excavation shall not be piled on the gas pipeline. This blocks access to the gas pipeline in the event that maintenance or operations activities are required on the pipeline.

The gas pipeline must be inspected for damage before backfilling the excavation.

It is the excavator's responsibility, under Section 18 and 19 of the Energy Act to ensure the gas pipeline(s) is not undermined or endangered in any way.

2.2 SUPPORT OF PIPELINES REQUIRED AT ALL TIMES

It is the responsibility of the Contractor to ensure that existing underground plant is properly supported.

Precautions must be taken to support underground plant at all times and to prevent damage to gas pipelines due to excavation activities. Inadequate support damages underground plant and can result in the escape of natural gas, constituting a hazard to persons and property.

When excavation is necessary over, under, near or parallel to underground Gas plant, the support is the responsibility of the excavator. The methods of support vary from case to case depending on the characteristics of the excavation, adjacent soil and the pipeline material. Failure to provide proper support will render the excavator responsible for all consequential damage or loss. (**Refer to Section 3.0, Support of Gas Pipelines**, for details on supporting the gas pipeline.)

2.3 ENCROACHMENT

Permanent awnings and roof structures are prohibited above gas pipelines within the public right-of-way, or within the Company's right-of-way. Enbridge Gas Distribution will not accept responsibility for any damages to the encroaching structure within the public right-of-way, or within the Company's right-of-way, if it is necessary for the maintenance or operation of the existing underground plant or to install new underground facilities in the future.

2.4 TREE PLANTING

For pipelines regulated by the NEB and Vital Mains (identified as critical pipelines), trees or large shrubs must have a minimum lateral clearance between the edge of the root ball or open bottom container and adjacent edge of the existing pipeline of not less than 2.5 m (8 feet).

For all other pipelines, a minimum clearance of 1.2 m (4 feet) horizontally must be maintained between the edge of the root ball or open bottom container and adjacent edge of the existing gas pipeline

In cases where 1.2 m (4 feet) clearance cannot be maintained, a minimum clearance of 0.6 m (2 feet) can be permitted provided a root deflector is installed on the sides of the root ball adjacent to the gas pipeline.

Final location of the trees must be confirmed with Enbridge Gas Distribution to avoid interference with the existing gas pipelines.

Root Deflectors

A root deflector is a mechanical barrier placed between tree roots and pipelines to prevent damage to the pipelines. A root deflector can be made from 1/4-inch rigid plastic, fiberglass or a non-degradable material. As the root tip of a tree travels out from the root ball the tip will contact the barrier, unable to penetrate to the barrier, the root will turn.

Root deflectors must be installed 0.6 meters (2 feet) from the pipeline on the side of the tree facing the pipeline and must extend 1.2 meters (4 feet) from the center of the tree trunk, parallel to the pipeline, at both directions; or the deflector must circle the tree.

Root deflectors usually have a collar to keep the top of the deflector at ground level, and they should extend down to the bottom of the root-ball as shown in Figure 2.4.

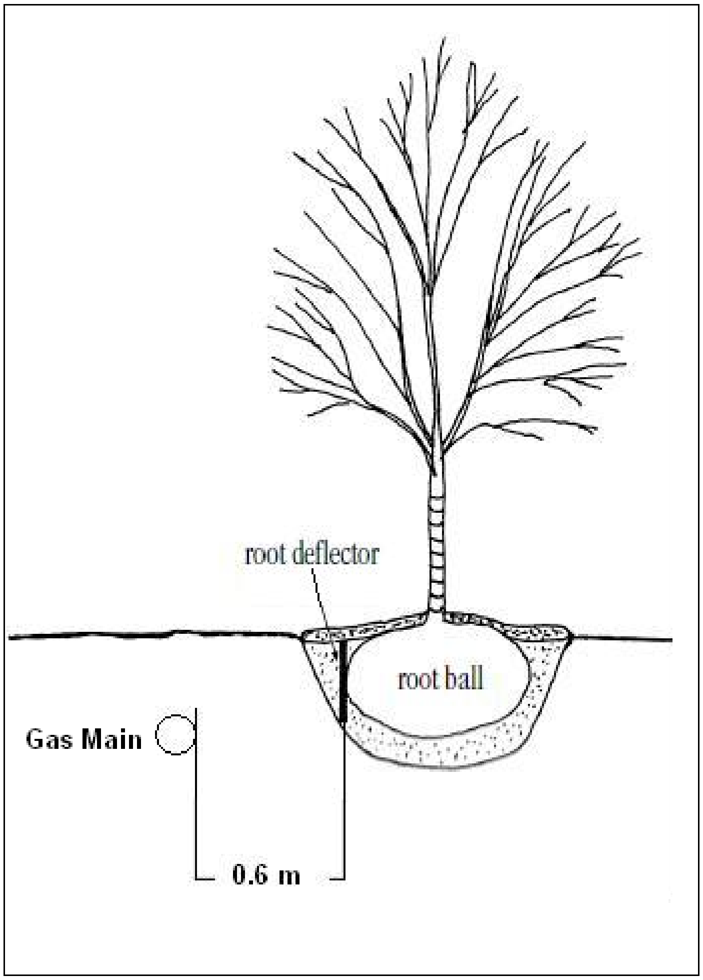


Figure 1
Root Deflector

2.5 MINIMUM CLEARANCE FROM OTHER STRUCTURES

The following clearances must be maintained between the outside wall of the gas pipeline and other underground structures:

Horizontal	-	0.6 m minimum
Vertical	-	0.3 m minimum
Vertical	-	0.6 m minimum for pipelines 16 inches in diameter and larger

Excavations for permanent structures (i.e. pools, root cellars, septic tanks etc.) must be at least 10.0 m from the limit of the existing right-of-way of the NEB pipeline.

Any work performed within 30.0 meters of an NEB pipeline right-of-way must be approved by Enbridge.

2.6 MINIMUM COVER REQUIREMENTS (Table No. 1)

	Location	Minimum cover (m)
Mains	Below traveled surfaces (roads), Road Crossings, General, Rights-of-way (roads)	1.2/0.9 *
	Water crossings	1.5
	Controlled Access Highways crossings, Below base of rails (cased)	1.7
	Rights-of-way (railroads), Drainage, Irrigation Ditches	1.0
Services	Private property	0.3
	Streets and Roads	0.45
	Wet Gas Areas @ Main/Building	1.2 / 0.9

* 1.2m is required for Transmission Lines 0.9m is required for Distribution Lines

2.7 POINTS OF THRUST

Precautions must be taken when working in the immediate vicinity of points of thrust. Points of thrust occur at pipeline fittings such as Elbows (45° or 90°), End Caps, Weld Tees, Reducer Couplings and closed Valves. In the event that the excavation involves exposing a point of thrust, or exposing an area near a point of thrust, specific instructions provided by the Company must be followed. Failure to follow these instructions can result in significant harm to persons and property.

2.8 REPAIR OF DAMAGED PIPE AND PIPE COATING

In all cases where the pipe or the pipe coating is damaged by the construction operation, contact the Company immediately and leave the excavation open until Company personnel have made the necessary repairs.

2.9 BLASTING, PILE DRIVING OR COMPACTION

Blasting, Pile Driving, or Compaction activities in the vicinity of natural gas pipelines requires the prior approval by the owner of the pipeline.
(TSSA Act 2001).

Written notification from the owner of the proposed work (municipality, etc.) shall be submitted to the Manager Distribution Planning. The request shall be submitted a minimum of four (4) weeks prior to blasting, pile driving or compaction to allow sufficient time to ensure the Company requirements are followed. (**Refer to Section 4.0, Blasting Requirements, and Section 5.0, Pile Driving and Compaction Requirements**, for specific responsibilities.)

3.0 SUPPORT OF GAS PIPELINES

3.1 TRENCHING PARALLEL TO GAS PIPELINES

When a trench parallels an existing gas pipeline, support may be required depending on trench depth, pipeline material and soil conditions. (**Refer to Section 3.4, Support of Pipelines Parallel to Trench**, for details.)

3.2 MINIMUM REQUIREMENTS

Support methods specified by the Company are minimum requirements. Excavators shall not depart from these unless a Professional Engineer working for or on behalf of the excavator has designed an alternative method. Any alternative method must ensure support comparable to these specifications and be, in the opinion of the Professional Engineer, consistent with good engineering practices. Where that is the case, the alternative specification shall be documented and approved by the Professional Engineer and sent to the Company's Engineering Department for acceptability.

The following specifications deal with the support of gas pipelines in the vicinity of excavations. Two typical field situations are covered:

- support of gas pipelines **crossing the trench** and
- support of gas pipelines **parallel to the trench**.

3.3 SUPPORT OF PIPELINES CROSSING TRENCH

3.3.1 Temporary Support

Temporary support refers to the support of gas pipelines prior to or at the time of excavation to protect the pipeline from deflection due to its own weight while it is exposed. Temporary support shall remain in place until the backfill material underneath the pipeline is compacted adequately to restore support of pipeline.

Prior to trenching beneath a pipeline or service, temporary support shall be erected for pipelines if the unsupported span of pipeline in the trench exceeds the length indicated in **Table No. 2, page 11**.

When temporary support is required, **Table No. 3, page 11**, below, indicates the required beam for a given span. The beam shall be a continuous length grade No. 1 Spruce-Pine-Fir (S-P-F) or equivalent. For spans exceeding 4.5 m, contact the Company's Engineering Department for approval.

Table No. 2 **Maximum Span Without Support Beam**			
Pipe Size (NPS)	Steel (m)	PE (polyethylene) (m)	CI (cast iron) (m)
½	2.0	1.0	-
¾ - 1¼	2.5	1.25	-
2	3.0	1.5	-
3 to 4	4.5	1.75	1.0
6	6.0	2.0	1.0
8	7.0	2.0	1.0
12	10.0	-	1.0
16	11.5	-	1.0
20	13.0	-	1.0
24	15.0	-	1.0

Table No. 3 **Support Beam Sizes** Given: max. span between Beam Supports						
Pipe Size (NPS)	Steel		PE		Cast Iron	
	≤ 2 m	**≤ 4.5 m**	**≤ 2 m**	**≤ 4.5 m**	**≤ 2 m**	**≤ 4.5 m**
½ - 2	Nil	4 x 6	4 x 4	4 x 6	4 x 4	6 x 8
3 – 6	Nil	Nil	4 x 4	6 x 6	4 x 4	8 x8
8 – 12	Nil	Nil	4 x 4	8 x 8	6 x 6	10 x 10
16 – 24	Nil	Nil	Nil	Nil	8 x 8	12 x 12

The beam shall be placed above the pipeline with the ends of the beam resting on firm undisturbed soil. The beam shall not bear directly on the gas pipeline. The pipeline shall be supported from the beam with rope, chain or equivalent in a manner that will prevent damage to the pipeline and pipeline coating, and eliminate sag. The spacing between the rope, canvas sling or equivalent, shall not exceed 1.0 m (**see Drawing No. 1, page 15, for details**).

Backfill material underneath the exposed pipeline shall be compacted to a minimum of 95% Standard Proctor density. Sand padding shall be placed to a level 150 mm above and below the pipeline. Perform compaction with the loose lift height not exceeding 200 mm or one-quarter of the trench width, whichever is less. Injecting water into the backfill beneath the pipeline is not an acceptable method of compaction.

Mechanical equipment **shall not** be operated within 0.3 m of the pipeline. Hand Excavation shall be performed when locating and digging within 0.3 m of the pipeline. Hand held compaction equipment shall be used within 1.0 m of the sides or top of all gas pipelines.

3.3.2 Cast Iron Pipelines

Any cast iron pipeline NPS 8 or less which is completely exposed crossing a trench for a length greater than 1.0 m must either be replaced or temporarily supported and properly backfilled. Any cast iron pipeline NPS 12 or greater that is completely exposed for greater than 1.0 m must be referred to the Company's Engineering Department for analysis. (**See Drawing No. 1, page 15**, for details)

If the pipeline is to be replaced, the replacement section shall extend to beyond the two 45° lines projected upward from the trench bottom (**see Drawing No. 3, page 16**, for details).

If the pipeline is to be temporarily supported, the spacing of the rope, canvas sling or equivalent, shall be a maximum of 1.0 m. Any exposed joint shall be supported by canvas sling or rope at either side of the joint and at 1.0 m spacings along the pipeline's length (**see Drawing No. 1, page 15**, for details).

3.3.3 Steel and Polyethylene Pipelines

All steel and polyethylene pipelines exposed to a length greater than indicated in Table No. 1 shall be temporarily supported and backfilled as shown in **Drawing No. 2, page 15**, and as outlined in **Section 3.3.1**, Temporary Support.

NOTE: All temporary support on polyethylene pipes must be removed prior to permanent backfill. Adequate support shall remain in place until the backfill material has restored support.

3. 4 SUPPORT OF PIPELINES PARALLEL TO TRENCH

3.4.1 General

Two cases exist for pipelines parallel to an excavation;

i) trench < 1.2 m deep,
ii) trench ≥ 1.2 m deep.

In either instance, the pipeline is not to be exposed unless it is necessary to provide direct support.

Trench wall support is not required for excavations provided the pipeline meets the following criteria:

- depth is less than 1.2 metres,
- the pipeline is at least 0.6 metres from the edge of excavation or is outside the shaded area as indicated in Drawing No. 2 and,
- soil is stable (TYPE 1or 2, refer to **Soil Types, page 30**)

Trench wall support is required for excavations if one of the following conditions exists:

- depth is equal to or greater than 1.2 metres,
- the pipeline is closer to the edge of the excavation than the minimum allowed distance as indicated in **Table No. 4, page 13**
- depth is less than 1.2 metres and the soil is unstable (TYPE 3 or 4, refer to **Soil Types, page 30**)

NOTE: Adequate support shall remain in place until the backfill material has restored support.

Table No. 3 gives minimum distances from the edge of the trench to the pipeline in which the excavation influences pipelines for the given soil types.

Table No. 4 Minimum Allowed Distance from Pipeline to Excavation (m)		
Trench Depth (m)	Soil Types 1 & 2*	Soil Types 3 & 4*
>1.2	0.9	0.9
≥1.5	0.9	0.9
≥1.8	0.9	0.9
≥2.1	0.9	0.9
≥2.4	0.9	0.9
≥2.7	0.9	1.0
≥3.0	0.9	1.5
≥3.3	0.9	1.8
≥3.6	0.9	2.2
≥3.9	0.9	2.5
≥4.2	0.9	3.0
≥4.5	1.0	3.4
≥4.8	1.5	3.8
≥5.1	2.0	4.1
≥5.4	2.5	4.6
≥5.7	3.0	5.0
≥6	3.4	5.5
*as defined in the Occupational Health and Safety Act		

3.4.2 Cast Iron Pipelines

If a cast iron pipeline lies within the 45° line projected upward from the bottom of the trench, the trench shall be suitably shored to support the pipeline. A sliding trench box does not provide adequate support.

If a cast iron pipeline lies within the 45° line projected upward from the trench bottom and the bottom of the trench is below the water table, a field assessment of the situation is required to determine if this pipeline must be replaced.

For cast iron pipelines within the minimum distances given in **Table No. 4, page 13**, above, the support shall be abandoned in place.

If any cast iron pipeline becomes exposed for a length greater than 1.0 m it shall be replaced. Replacement limits shall be determined in the field.

3.4.3 Steel and Polyethylene Pipelines

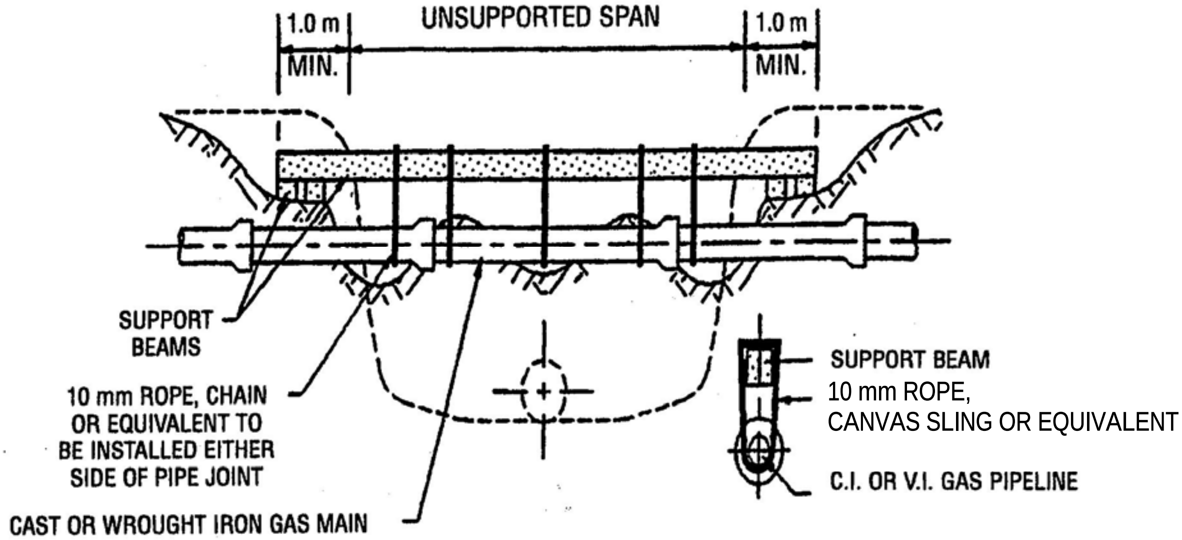
In the case of a steel or polyethylene pipeline within the limits of 3.4.1, and the trench bottom is below the water table, the trench shall be suitably supported as required in 3.4.1.

For steel and polyethylene pipelines within the minimum distances given in **Table No. 4, page 13**, support shall remain in place until backfill material restores support.

Any steel or polyethylene pipeline that is unsupported for a length greater than indicated in **Table No. 2, page 11**, shall require field assessment by the Company.

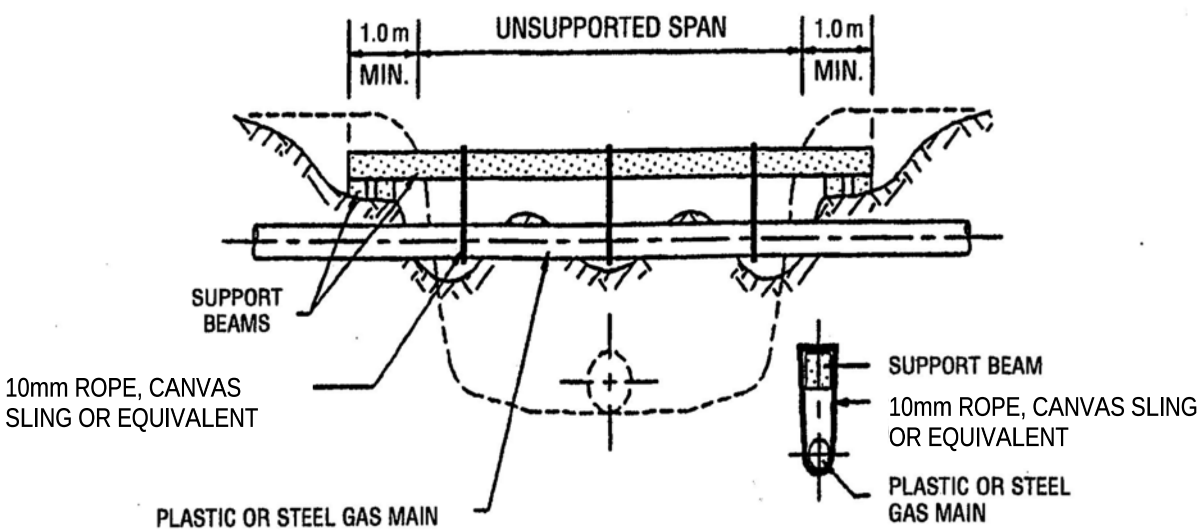
DWG NO. 1: Support of Cast/Wrought Iron Gas pipelines Crossing Excavations

NOTE: BEAM SHALL EXTEND TO 1.0 m BEYOND THE SIDE OF THE TRENCH ON UNDISTRURBED SOIL OR A DISTANCE EQUAL TO THE DEPTH OF THE PROPOSED EXCAVATION, WHICHEVER IS GREATER.



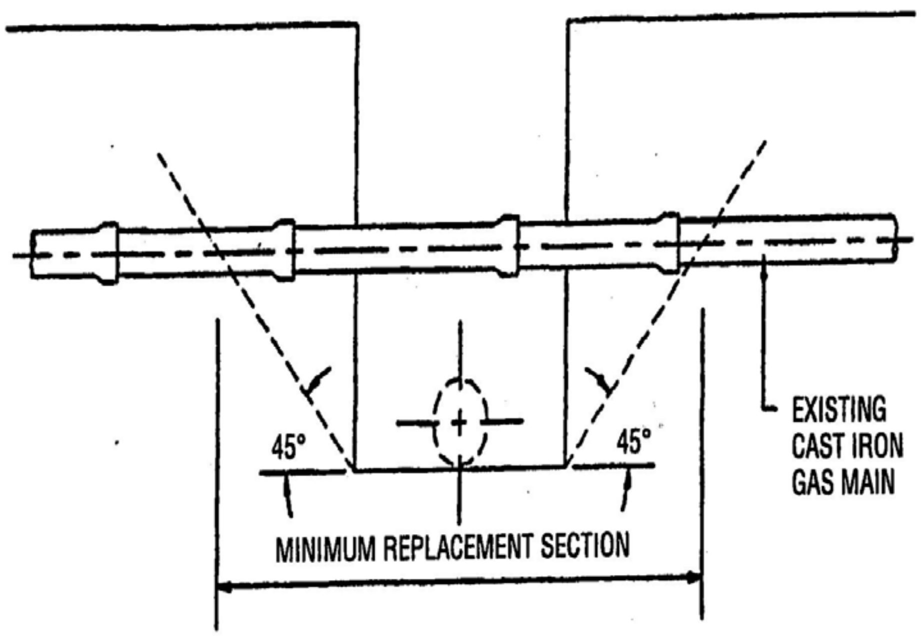
DWG NO. 2: Support of Plastic or Steel Gas Pipelines Crossing Excavations

NOTE: BEAM SHALL EXTEND TO 1.0m BEYOND THE SIDE OF THE TRENCH ON UNDISTURBED SOIL OR A DISTANCE EQUAL TO THE DEPTH OF THE PROPOSED EXCAVATION, WHICHEVER IS GREATER.

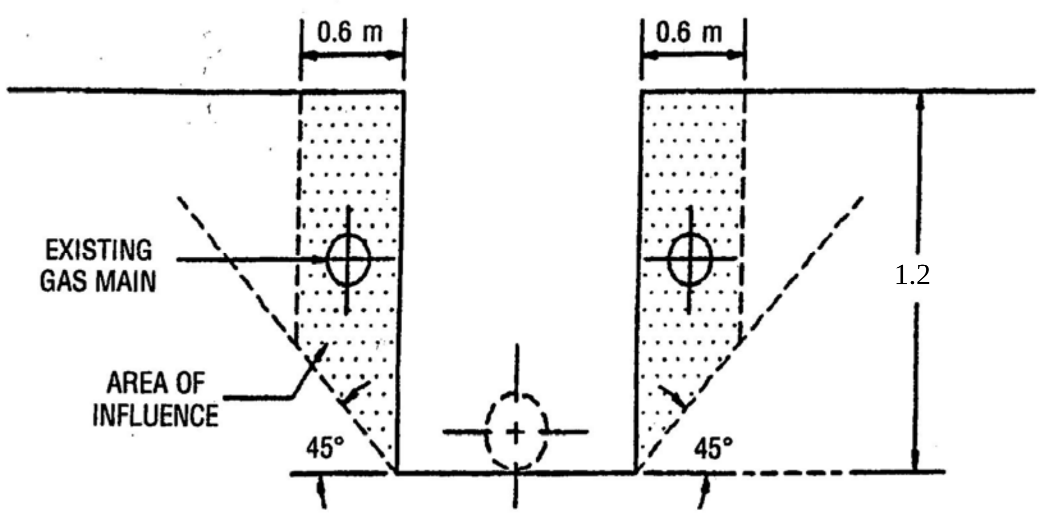


DWG NO. 3: Influence Lines for Gas Pipelines Adjacent to Excavations

CAST IRON CROSSINGS – MINIMUM REPLACEMENT SECTIONS



PARALLEL MAINS – GENERAL



NOTE: IF PIPE IS LOCATED IN THE SHADED AREA, IF SOIL IS UNSTABLE (TYPE 3 or 4), THE TRENCH IS REQUIRED TO BE SUPPORTED

4.0 BLASTING REQUIREMENTS

4.1 POLICY

Prior to any blasting operation in the vicinity of a gas pipeline, the hazard to Enbridge Gas Distribution Inc. plant will be evaluated to ensure the uninterrupted operation and long-term safety of its underground facilities. Responsibility for the design of the blast and any resultant damage is born entirely by the party using the explosives.

A recognized independent blasting consultant shall be retained at the applicants' expense to evaluate and validate the risks for blasting under any of the following conditions:

a) Explosive charge weight per delay in **Table 5, page 22**, is exceeded.

b) Blasting requirements less than 3 meters from Company facilities.

c) Blasting in the vicinity of cast iron and wrought iron pipelines.

d) Any tunnel blasting operation in the vicinity of Company facilities.

e) Surface blasts less than 10 meters from a Company pipeline where the excavation depth of the first blast hole is equal to the depth of the top of the pipeline and subsequent blast hole depths are greater than one half the horizontal distance to the closest portion of the pipeline.

f) Any time if in the opinion of Enbridge Gas Distribution Inc., it is felt the integrity of Company facilities may be affected by the blast.

The Independent Blasting Consultant shall be a Registered Professional Engineer and a holder of a Certificate of Authorization (C of A), specializing in blasting.

A copy of the consultant's report shall be forwarded to Enbridge Gas Distribution Inc. Engineering Department for review.

If in the opinion of Enbridge Gas Distribution Inc. or an independent blasting consultant, blasting cannot be carried out without affecting the facility's integrity, alternatives shall be considered, including the replacement or relocation of the affected facility at the applicants' expense. In these situations, additional time must be allowed to obtain the necessary permits and to complete the necessary construction work.

4.2 NOTIFICATION REQUIREMENTS

4.2.1 Surface Blasting Applications

The written request for surface blasting shall include the following information:

- Name of the owner of the project, general contractor and design engineer.
- Name of the blasting contractor and person in charge of the blast.
- Date for the blasting operation.
- A copy of a construction drawing or sketch drawn to scale indicating:
  - i Details of the proposed drilling and loading pattern for explosives.
  - ii Diameters of drilled holes, relative to Company facilities.
  - iii Location of other public utilities, i.e. Bell, hydro, water etc.
- Number and timing of delays.
- Total explosive weight to be detonated per delay.
- Specifications for the type of explosives to be used.
- Predicted vibration levels anticipated at the pipeline and controls to be used to confirm vibration levels (i.e. Seismographs).
- Potential stabilization of rock face and type of potential stabilization techniques i.e.: rock anchors, shot crete, ribs, etc.
- Geological parameters (Borehole logs or Geological reports) which indicate the design of the blast are acceptable.
- Written confirmation that the blasting operation will be carried out by qualified personnel with appropriate engineering supervision.

4.2.2 Tunnel Blasting Applications

The written request for tunnel blasting shall include all information required in the surface blasting application as set out above in 4.2.1. In addition, the required independent blasting consultant's report shall include:

- Location plans and profile views with construction drawing or sketch, drawn to scale.
- Evaluation of geo-technical data.
- Exact stand-off distances horizontal and direct (radial)

- Type of advancement proposed and type of tunnel method proposed; full face, top of heading and bench, pilot tunnel

- Type of tunnel lining proposed.

- The use of preventative blasting techniques such as line drilling, cushion blasting, etc.

- Other pertinent information specific to tunneling techniques.

To assist with the preparation of the written request, locates to determine the location of the pipeline can be requested, or mark-ups of drawings can be obtained by contacting the Manager Distribution Planning, Enbridge Gas Distribution. Lists of Regional addresses and phone numbers are outlined at Appendix A.

4.3 EVALUATION BY ENBRIDGE GAS DISTRIBUTION

Enbridge Gas Distribution will conduct a record search on the facilities in the vicinity of the blast to determine the material, location and maintenance history.

Enbridge Gas Distribution will evaluate the impact of the blast on the facilities, assessing the charge weight to be detonated in relation to the stand off distance. If, in the opinion of Enbridge Gas Distribution, a hazardous condition may result if the charges are fired as outlined in the application, the applicant shall be notified in writing. The applicant shall not commence operations and shall retain the services of an independent blasting consultant to evaluate and validate the application. A copy of the required consultants' report shall be forwarded to Enbridge Gas Distribution Engineering Department for approval.

Enbridge Gas Distribution shall conduct a leak survey (flame ionization unit) of the pipeline prior, during and after the blasting and independently of its normal leak-monitoring program to establish satisfactorily that the pipeline is not leaking.

Enbridge Gas Distribution shall prepare a contingency plan to respond in the event that isolation of the pipeline becomes necessary. Blasting operations shall not commence until all Enbridge Gas Distribution procedures have been implemented and the applicant has received written notification of it.

Enbridge Gas Distribution shall locate all control valves within the vicinity of the approved blast area. Check all valves involved in the contingency plan to ensure accessibility and proper operability.

In the event a third party is affected as a result of the blasting operations, all expenses associated therewith incurred by Enbridge Gas Distribution shall also be at the applicant's expense

4.4 GROUND WATER MONITORING

Where there is a potential for damage to nearby wells, the blaster shall conduct an evaluation designed and implemented to minimize adverse impacts on potentially affected wells. Generally, all water wells within 100 meters of proposed blasting locations should be monitored for quality and quantity prior to construction.

Blasting in a watercourse requires Department of Fisheries and Oceans (DFO) authorization.

4.5 GUIDELINES FOR BLASTING

The information provided in this section is not to be construed as an exhaustive list of performance specifications, but rather a guide for conducting blasting in the vicinity of Enbridge Gas Distribution pipelines. The applicant is responsible for ensuring that all blasting work is performed in a good and workmanlike manner in accordance with all applicable laws, codes, by-laws, and regulations.

The contractor shall be liable for and indemnify Enbridge Gas Distribution in relation to any and all damage directly or indirectly caused or arising as a result of blasting operations carried out by the applicant, its employees, contractors or those for whom the applicant is responsible at law.

Prior to blasting operations, a site meeting shall be arranged with an authorized representative of the applicant and an Enbridge Gas Distribution representative to confirm details of the location of Company facilities and the proposed blast.

Enbridge Gas Distribution pipelines shall not be excavated prior to blasting. If excavation is unavoidable, then the pipeline shall be properly supported according to current Enbridge Gas Distribution requirements as outlined in this booklet. The applicant shall take suitable precautions to protect the exposed pipeline from fly-rock. Blasting mats shall be used to minimize the risk of fly-rock.

Explosives shall be of a type that will not propagate between holes nor desensitize due to compression pressures. No explosives shall be left in the drill hole overnight.

For surface blasts located at distances of 10 meters or less from a pipeline and when the excavation of the first blast hole has attained a depth equal to the top of the buried natural gas pipeline, the vertical depth of subsequent blast holes shall be restricted to one half of the horizontal distance to the closest portion of the natural gas pipeline. The required independent blasting consultants' report shall specifically address the impact of these conditions. This condition is not applicable for tunnel blasting operations.

Horizontal stand-off distances for surface blasting and directs stand-off distances for tunnel blasting of less than 3 meters are not permitted.

If the applicant insists that blasting is necessary, the required independent blasting consultants report shall evaluate and validate the proposal.

The applicant shall comply with the Ontario Provincial Standard Specification - OPSS 120 - General Specification for the Use of Explosives, in addition to these Enbridge Gas Distribution blasting requirements.

Monitoring of blasting vibrations with a portable seismograph capable of producing on site print outs in the vicinity of Company facilities is mandatory to confirm that predicted vibration levels are respected. At the completion of the blasting operation, a copy of the seismographic report shall be provided to Enbridge Gas Distribution.

Table 5, page 22, shall be used to guide explosive charge weights. Peak Particle Velocity (PPV) shall be limited to 50 mm/sec and maximum amplitude shall be limited to 0.1524 mm.

4.6 POST BLASTING OPERATION

Upon completion of daily blasting operations and within 30 days after the final blasting, Enbridge Gas Distribution shall conduct a leak survey (flame ionization) of the pipeline at the applicants' expense. Leak survey shall also be completed at the end of each day of blasting. Damage that has resulted from the blast will be repaired at the applicants' expense. A summary of all blasting operations including blasting logs, vibration control, seismograph reports and other pertinent information shall be provided to Enbridge Gas Distribution by the applicant at the completion of blasting operations.

TABLE NO 5 Stand-off Distance for Blasting Near Polyethylene and Steel Facilities		
STAND-OFF DISTANCE FROM FACILITY (m)		**MAXIMUM ALLOWABLE EXPLOSIVE CHARGE WEIGHT PER DELAY (kg)**
3.00		0.18
4.00		0.33
5.00		0.51
6.00		0.73
7.00		1.00
8.00		1.31
9.00		1.65
10.00		2.04
12.00		2.94
14.00		4.00
16.00		5.22
18.00		6.61
20.00		8.16
22.00		9.87
24.00		11.75
26.00		13.79
28.00		16.00
30.00		18.36

The chart above is based on a Peak Particle Velocity (PPV) of 50 mm/sec. No greater velocity shall be allowed. Maximum amplitude shall be limited to 0.1524 mm.

5.0 PILE DRIVING OR COMPACTION REQUIREMENTS

5.1 POLICY

Prior to any pile driving or compaction operations within the vicinity of a gas pipeline, the potential damage to Enbridge Gas Distribution plant will be evaluated to ensure the uninterrupted operation and long-term safety of its underground facilities. Any resultant damage caused either directly or indirectly to the gas plant will be borne entirely by the Contractor undertaking the proposed work.

If, in the opinion of Enbridge Gas Distribution, the particular pile driving or compaction operation cannot be carried out without affecting the pipeline or facility integrity, the following alternatives, or contingencies, may be implemented:

- a review of the particular situation by an independent consultant including a risk analysis and a prevention program;
- change in the construction methods;
- replacement or relocation of the pipeline/facility.

All costs incurred will be covered by the Contractor undertaking the proposed work with final approval being granted by Enbridge Gas Distribution.

5.2 PILE DRIVING OR COMPACTION APPLICATION

The application must include the following information:

- Name of project owner, general contractor and relevant sub-trades;
- A copy of the permits, certificates or other forms required by municipal bylaws;
- Name of design engineer and a copy of plans issued for construction with detailed drawings identifying all affected natural gas facilities;
- The type of piles and equipment used; including the methods of control to prevent the deviation of the piles;
- Geo-technical reports and other pertinent information;
- A copy of the location of other public utilities such as telephone, cable TV, sewer and water mains, electrical services, etc.;

- If required, a technical report with appropriate analysis and prediction of the vibration levels according to the opinion of an independent Engineer specialized in vibration control and analysis;

- A clause stating that the work will be carried out by qualified personnel with appropriate experienced supervision;

- A clause stating that all vibration testing results, or other preventative control testing, will be submitted to Enbridge Gas Distribution on a regular basis, or upon request.

To help with the preparation of the written request, locates to determine the location of the pipeline can be requested by calling "Ontario One Call" listed in Regional Contact List on Appendix A, and appropriate markups of drawings can be obtained by contacting "Distribution Planning" listed in Regional Contact List on Appendix A.

5.3 EVALUATION BY ENBRIDGE GAS DISTRIBUTION

Enbridge Gas Distribution shall conduct a record search on the natural gas facilities in the vicinity of the proposed work to identify their materials, location and maintenance history.

Enbridge Gas Distribution shall assess the impact of the proposed operation on the pipeline or related facility versus the stand-off distance. If it is determined that the proposed operation and/or method of work may be detrimental, the Contractor must retain the services of an independent Engineer. This Engineer must be specialized in vibration control, analysis and soil movement in order to evaluate and validate the proposed method of work and operation.

Enbridge Gas Distribution shall conduct leak surveys (flame ionization unit) of the pipelines and other related natural gas facilities prior, during and after the start of work. Leak surveys shall be conducted at any time during the project notwithstanding any delays or costs incurred by the Contractor responsible for proposed work.

Enbridge Gas Distribution shall prepare a contingency plan in case the isolation of the line or shut down of the related facility becomes necessary. This may not be possible without affecting a large number of customers and all operations may be suspended until Company investigations are completed notwithstanding any delays or costs incurred by the Contractor responsible for proposed work.

Enbridge Gas Distribution shall locate all control valves within the vicinity of the approved location and check all valves involved in the contingency plan to ensure accessibility and proper operability.

Enbridge Gas Distribution shall be responsible for isolating the area of the pipeline in the direct vicinity of the operations as required. The Contractor will be responsible for all Company costs during piling operations.

In the event a third party is affected as a result of the pile driving and/or compaction operations, all expenses associated therewith incurred by Enbridge Gas Distribution shall also be at the Contractor's expense.

5.4 GUIDELINE FOR PILE DRIVING OR COMPACTION

The information provided in this section is to be viewed as a guideline only and is not intended to remove Contractor responsibility for damages caused by the piling and/or compaction operations. The contractor is responsible for ensuring that all pile driving and/ or compaction work is performed in a good and workmanlike manner in accordance with all applicable laws, codes, by-laws and regulations.

Prior to pile driving and/or compaction work, a site meeting shall be arranged with an authorized representative of the Contractor and an Enbridge Gas Distribution representative to confirm details of the location of Company facilities and the proposed work.

The pipeline should not be excavated prior to the piling or compaction operation. If the particular situation warrants the excavation of the pipeline, then it must be properly supported in accordance with **Section 3.0 Standard Procedures**.

If in the assessment of Enbridge Gas Distribution, the soil cover is deemed to be insufficient, Enbridge Gas Distribution shall require that a protective ramp be constructed and maintained above the pipeline in accordance with Company guidelines. Construction vehicles or equipment will not be allowed to pass over a pipeline without the authorization of a Company representative.

The following situations will require the opinion of an independent Engineer. This Engineer must be specialized in vibration control, analysis and soil movement in order to evaluate and validate the proposed method of work and operation.

a) Compaction of soils or backfill rated at 10,000 ft-lbs or higher at a stand-off distance of 6 meters or less from the pipeline

b) Pile driving at a stand-off distance of 10 meters or less from the pipeline or other natural gas facility.

c) High-energy dynamic compaction for the rehabilitation of soils at a distance of 30 meters or less from the pipeline.

d) Soil types fitting the description of Type 4 soil as defined in Article 226 of the Occupational Health and Safety Act and Regulations for Construction Projects (**Refer to Section 5.6 Soil Types, page 30**).

For all these situations, monitoring of vibrations, with the appropriate number of seismographs, is mandatory. The seismographs shall be the portable types with the capability of producing on site printouts. This control will confirm the intensity of the vibrations generated by the pile driving or compaction work as projected. Furthermore, reports of recorded intensities shall be provided on a regular basis or at the request of Enbridge Gas Distribution.

Should a situation with low energy compaction operations with a soil cover of less than 1.5 meters above the pipeline at a stand-off distance of 3 meters or less from a pipeline be encountered, Enbridge Gas Distribution may require the opinion of an independent Engineer.

In addition, if a Type 3 soil (**refer to Section 5.6 Soil Types, page 30**) is present on site, Enbridge Gas Distribution may, again, require the opinion of an independent Engineer.

For the start of the construction operations, the equipment and method used for pile driving shall comply with the guidelines presented in **Figure 2, page 28**, and **Table 6, page 29**, which identify the maximum vibration intensities expected from pile driving in dry and wet sand and clay. These guidelines can be replaced by actual vibration testing (portable seismograph) on site.

The Peak Particle Velocity (PPV) measured on the pipeline, or at the closest point of the related structure with respect to the work, shall not exceed 50 mm/s. Furthermore, the maximum displacement for the vertical and/or horizontal component corresponding to the above stated vibration intensity shall not exceed 50 mm at any given length of the pipeline in question.

For all operations, if the Peak Particle Velocity (PPV) and/or the displacement limit are surpassed, all operations must stop notwithstanding any delays or costs incurred by the contractor or owner of the proposed work. Enbridge Gas Distribution will require that the cause of these higher vibrations or displacement be investigated. The operations shall resume only when the cause and remedy are established and with the approval of Enbridge Gas Distribution's Engineering Department.

Should any subsequent recordings indicate vibration intensities or displacements above the prescribed limits all operations shall immediately stop. Enbridge Gas Distribution shall require that the work be carried out according to methods it judges to be acceptable to the integrity of the pipeline or related structure notwithstanding any delays or costs incurred by the Contractor responsible for the proposed work.

No operations shall be permitted within a standoff distance of 1.5 meters from the pipeline or other natural gas facility unless approved by Enbridge Gas Distribution.

Auguring of the soil up to the base of the pipeline may be required in order to avoid deviation of the piles within a distance of 1.5 m from the pipeline.

All operations must comply with the Provincial Occupational Health and Safety Act and Regulations for Construction Projects as well as all applicable Company specifications, standards and guidelines.

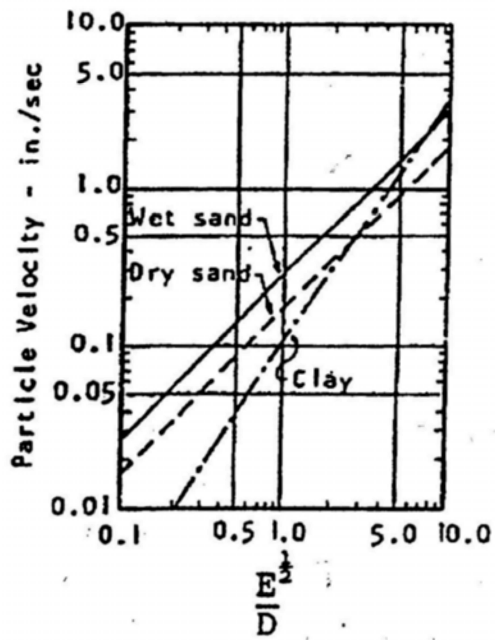
Leak surveys (flame ionization) shall be conducted at any time following the higher vibration intensities or displacements notwithstanding any delays or costs incurred by the contractor or authority responsible for the proposed work.

5.5 POST PILING OR COMPACTION OPERATIONS

A summary of all operations including pile driving and compaction logs, vibration control, seismographs and other pertinent information shall be provided to Enbridge Gas Distribution by the Contractor responsible for the proposed work no later than 5 business days after work has been completed.

On completion of the daily operations, and approximately 30 days after the end of the operations, Enbridge Gas Distribution shall conduct a leak survey (flame ionization) of the pipeline. The resulting damages will be repaired at the expense of the Contractor responsible for the proposed work.

GROUND VIBRATIONS FROM PILE DRIVING
(Figure 2)



NOTE: E is the rated energy of the pile hammer in ft-lbs. D is the distance of the pile tip from point of reference in ft.

Maximum vibration intensities expected from pile driving in wet sand, dry sand, and clay

GROUND VIBRATIONS FROM PILE DRIVING AND THE EFFECT OF GROUND VIBRATIONS (after Liu and Wiss, 1974)

Table No. 6

MAXIMUM VIBRATION INTENSITIES EXPECTED FROM PILE DRIVING IN DRY AND WET SAND AND CLAY

Particle Velocity in/s

E/D	DRY SAND	WET SAND	CLAY
0.10	0.02	0.03	------
0.22	0.04	0.06	0.01
0.30	0.05	0.08	0.02
0.40	0.07	0.11	0.04
0.50	0.08	0.13	0.04
0.60	0.10	0.18	0.05
0.70	0.11	0.20	0.06
0.80	0.13	0.23	0.08
0.90	0.16	0.27	0.09
1.00	0.18	0.29	0.10
2.00	0.33	0.59	0.30
3.00	0.56	0.88	0.58
4.00	0.70	1.10	0.89
5.00	0.88	1.40	1.10
6.00	1.05	1.85	1.80 Acceptable
7.00	1.10	2.01	2.01 *Unacceptable*
8.00	1.40	2.30	2.40
9.00	1.75	2.80	3.10
10.00	1.85	2.90	3.40

Particle Velocity mm/s

E/D	DRY SAND	WET SAND	CLAY
0.10	0.43	0.74	------
0.22	0.97	1.50	0.25
0.30	1.27	1.27	0.43
0.40	1.75	2.80	0.66
0.50	2.06	3.30	1.02
0.60	2.54	4.57	1.27
0.70	2.80	5.08	1.52
0.80	3.30	5.84	1.96
0.90	4.06	6.86	2.29
1.00	4.57	7.37	2.54
2.00	8.38	14.99	7.62
3.00	14.22	22.35	14.73
4.00	17.78	27.94	22.61
5.00	22.35	35.56	27.94
6.00	26.67	46.99	45.72 Acceptable
7.00	27.94	50.80	50.80 *Unacceptable*
8.00	35.56	58.42	60.96
9.00	44.45	71.12	78.74
10.00	46.99	73.66	86.36

5.6 SOIL TYPES

(Occupational Health and Safety Act

And Regulations for Construction Projects)

(1) For the purposes of this Part, soil shall be classified as Type 1, 2, 3, or 4 in accordance with the descriptions set out in this section.

(2) Type 1 Soil

a) is hard, very dense and only able to be penetrated with difficulty by a small sharp object;
b) has a low natural moisture content and a high degree of internal strength;
c) has no signs of water seepage; and
d) can be excavated only by mechanical equipment.

(3) Type 2 Soil

a) is very stiff, dense and can be penetrated with moderate difficulty by a small sharp object;
b) has a low to medium natural moisture content and a medium degree of internal strength; and
c) has a damp appearance after it is excavated.

(4) Type 3 Soil

a) is stiff to firm and compact to loose in consistency or is previously excavated soil;
b) exhibits signs of surface cracking;
c) exhibits signs of water seepage;
d) if it is dry, may run easily into a well-defined conical pile; and
e) has a low degree of internal strength.

(5) Type 4 Soil

a) is soft to very soft and very loose in consistency, very sensitive and upon disturbance is significantly reduced in natural strength;
b) runs easily or flows, unless it is completely supported before excavating procedures;
c) has almost no internal strength;
d) is wet or muddy, and
e) exerts substantial fluid pressure on its supporting system.

6.0 HEAVY EQUIPMENT OPERATION IN THE VICINITY OF GAS PIPELINES

6.1 GENERAL

This information is presented as a guideline to cover precautions necessary when heavy construction equipment (gross weight greater than 10 tonnes) is to be operated in the vicinity of buried pipelines where no pavement exists or where grading operations are taking place.

Prior to any crossing, the location of the gas plant must first be located by an Enbridge Gas Distribution representative.

The excavator/constructor is responsible for confirming the location and depth of the gas plant by having test holes excavated as necessary with respect to the local conditions but not more than 50 m intervals.

6.2 EQUIPMENT MOVING ACROSS THE PIPELINE

Crossing locations for heavy equipment are to be kept a minimum.

The crossing locations shall be determined between the Enbridge Gas Distribution representative and the excavator/constructor. The crossing location shall be based on the following:

- Nature of the construction operations
- The types and number of equipment involved
- Pipeline material and depth

Once the predetermined crossing locations have been established, heavy equipment must be restricted to crossing at these locations only. It is the responsibility of the excavator/constructor to inform their personnel of the crossing location restrictions.

Gas plants shall be protected from possible damage at crossing locations at all times. The protection can be provided by constructing berms over the staked lines unless minimum cover of twice the pipe diameter or 1.0 m (whichever is greater) has been verified.

Equipment shall be operated at “dead slow “ speeds when crossing pipelines to minimize impact loading.

6.3 EQUIPMENT MOVING ALONG THE PIPELINE

Heavy equipment may be operated parallel to existing pipelines provided that a minimum offset of 1.0 m is maintained on pipeline sizes less than NPS 12 and 2.0 m on pipelines NPS 12 and larger unless otherwise directed by Enbridge Gas Distribution.

Only lightweight rubber tired equipment shall be operated directly over existing gas pipelines unless a minimum pipe cover of twice the pipe diameter or 1.0 m (whichever is greater) can be verified.

When working directly over existing gas pipelines, all equipment movements shall be transverse to the staked location rather than parallel to it.

6.4 COMPACTION EQUIPMENT RESTRICTIONS

Mechanical equipment shall not be operated within 0.3 m of the pipeline.

Hand held compaction equipment shall be used within 1.0 m of the sides or top of all gas pipelines.

Heavier compaction equipment may be used once the pipe cover equals the greater of twice the diameter or 1.0 m.

6.5 GENERAL VEHICLE EXTERNAL LOADING RESTRICTIONS

For most vehicles, other than heavy construction equipment discussed above, external loading will not be factor because the standard Enbridge Gas Distribution pipeline cover requirements provide sufficient protection.

In cases where extreme loading is likely to occur, the following table provides vehicle load restrictions based on the depth of cover of pipe. If the loads exceed these, or if there are additional concerns, the contact name listed in the permit application should be contacted to specify required precautions and/or perform any loading calculation.

Since the depth of cover is important, if the depth is questionable, the pipeline should be located by hand. During wet weather conditions, increasing the amount of cover should be considered due to the rutting over the main.

Table No. 7		
Weight / Axle Maximum Allowable Load (kg)		
Cast Iron (CI)	Steel (ST)	Plastic (PE)
12,000	12,000	7,000

Vehicle Load Restrictions Based on Minimum Depth of 0.6 m.

ENBRIDGE

APPENDIX "A"

REGIONAL CONTACT LIST

ENBRIDGE GAS DISTRIBUTION	Markups	mark-ups@enbridge.com
500 Consumers Road	Mail to:	Distribution Planning
North York, ON M2J 1P8	Ontario One Call Locates:	1 (800) 400-2255
	Damage Prevention:	1 (866) 922-3622
	Emergency:	**1 (866) 763-5427**
ENBRIDGE GAS STORAGE		
P. O. Box 520	Ontario One Call Locates:	1 (800) 400-2255
3595 Tecumseh Road	Engineering Dept.:	1 (519) 862-6015
Mooretown ON N0N 1M0		
	Emergency:	**1 (800) 255-1431**
GAZIFÈRE		
706 Boulevard Greber,	Locates:	1 (800) 663-9228
Gatineau QC	Planning Dept.:	1 (819) 771-8321 X-2449
J8V 3P8		
	Emergency:	**1 (819) 771-8321**
ST. LAWRENCE GAS		
COMPANY LTD.	Locates:	1 (315) 769-3511
33 Stearns Street,	Planning Dept.:	1 (315) 769-3516 x 174
P.O. Box 270		
Massena, NY. 13662	**Emergency:**	**1 (315) 769-3511**

International Society of
Explosives Engineers

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ISEE
Field Practice Guidelines For Blasting Seismographs
2009 Edition

ISEE
Performance Specifications For Blasting Seismographs
2011 Edition

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30325 Bainbridge Road
Cleveland, OH 44139

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Mark Svinkin, Vibraconsult
Robert Turnbull, Instantel
Randall Wheeler, White Industrial Seismology
Board Liaison, John Wiegand, Vibronics, Inc.

**This list represents the membership at the time the Committee was balloted on the final text of this edition. Since that time, changes in the membership may have occurred. A key to classifications is found at the back of the document.*

Committee Scope: This Committee shall have primary responsibility for documents on the manufacture, transportation, storage, and use of explosives and related materials. This Committee does not have responsibility for documents on consumer and display fireworks, model and high power rockets and motors, and pyrotechnic special effects.

Table of Contents

Disclaimer: These field practice recommendations are intended to serve as general guidelines, and cannot describe all types of field conditions. It is incumbent on the operator to evaluate these conditions and to obtain good coupling between monitoring instrument and the surface to be monitored. In all cases, the operator should describe the field conditions and setup procedures in the permanent record of each blast.

Preface: Blasting seismographs are used to establish compliance with Federal, state and local regulations and evaluate explosive performance. Laws and regulations have been established to prevent damage to property and injury to people. The disposition of the rules is strongly dependant on the accuracy of ground vibration and air overpressure data. In terms of explosive performance the same holds true. One goal of the ISEE Standards Committee is to ensure consistent recording of ground vibrations and air overpressure between all blasting seismographs.

ISEE Field Practice Guidelines For Blasting Seismographs

TM
International Society of Explosives Engineers
30325 Bainbridge Road
Cleveland, OH 44139

International Society of Explosives Engineers

ISEE Field Practice Guidelines
For
Blasting Seismographs
2009 Edition

This edition of *ISEE Field Practice Guidelines for Blasting Seismographs* was revised by the ISEE Standards Committee on February 4, 2008 and supersedes all previous editions. It was approved by the Society's Board of Directors in its role of Secretariat of the Standards at its February 5, 2009 meeting.

Origin and Development of ISEE Field Practice Guidelines for Blasting Seismographs

In 1994, questions were raised about the accuracy, reproducibility and defensibility of data from blasting seismographs. To address this issue, the International Society of Explosives Engineers (ISEE) established a Seismograph Standards Subcommittee at its annual conference held in February 1995. The committee was comprised of seismograph manufacturers, researchers, regulatory personnel and seismograph users.

In 1997, the Committee became the Blast Vibrations and Seismograph Section. The Guidelines were drafted and approved by the Section in December of 1999. The Section completed two standards in the year 2000: 1) ISEE Field Practice Guidelines for Blasting Seismographs; and 2) Performance Specifications for Blasting Seismographs.

In 2002, the Society established the ISEE Standards Committee. A review of the ISEE Field Practice Guidelines and the Performance Specifications for Blasting Seismographs fell within the scope of the Committee. Work began on a review of the Field Practice Guidelines in January of 2006 and was completed in February of 2008 with this edition.

One of the goals of the ISEE Standards Committee is to develop uniform and technically appropriate standards for blasting seismographs. The intent is to improve accuracy and consistency in ground and air vibration measurements. Blasting seismograph performance is affected by how the blasting seismograph is built and how it is placed in the field.

The ISEE Standards Committee takes on the role of keeping the standards up to date. These standards can be obtained by contacting the International Society of Explosives Engineers located at 30325 Bainbridge Road, Cleveland, Ohio 44139 or by visiting our website at www.isee.org.

Part I. General Guidelines

Blasting seismographs are deployed in the field to record the levels of blast-induced ground vibration and air overpressure. Accuracy of the recordings is essential. These guidelines define the user's responsibilities when deploying blasting seismographs in the field and assume that the blasting seismographs conform to the ISEE "Performance Specifications for Blasting Seismographs".

1. Read the instruction manual and be familiar with the operation of the instrument. Every seismograph comes with an instruction manual. Users are responsible for reading the appropriate sections and understanding the proper operation of the instrument before monitoring a blast.

2. Seismograph calibration. Annual calibration of the seismograph is recommended.

3. Keep proper blasting seismograph records. A user's log should note: the user's name, date, time, place and other pertinent data.

4. Document the location of the seismograph. This includes the name of the structure and where the seismograph was placed on the property relative to the structure. Any person should be able to locate and identify the exact monitoring location at a future date.

5. Know and record the distance to the blast. The horizontal distance from the seismograph to the blast should be known to at least two significant digits. For example, a blast within 1000 meters or feet would be measured to the nearest tens of meters or feet respectively and a blast within 10,000 meters or feet would be measured to the nearest hundreds of feet or meters respectively. Where elevation changes exceed 2.5h:1v, slant distances or true distance should be used.

6. Record the blast. When seismographs are deployed in the field, the time spent deploying the unit justifies recording an event. As practical, set the trigger levels low enough to record each blast.

7. Record the full time history waveform. Summary or single peak value recording options available on many seismographs should not be used for monitoring blast-generated vibrations. Operating modes that report peak velocities over a specified time interval are not recommended when recording blast-induced vibrations.

8. Set the sampling rate. The blasting seismograph should be programmed to record the entire blast event in enough detail to accurately reproduce the vibration trace. In general the sample rate should be at least 1000 samples per second.

9. Know the data processing time of the seismograph. Some units take up to 5 minutes to process and print data. If another blast occurs within this time the second blast may be missed.

10. Know the memory or record capacity of the seismograph. Enough memory must be available to store the event. The full waveform should be saved for future reference in either digital or analog form.

11. Know the nature of the report that is required. For example, provide a hard copy in the field, keep digital data as a permanent record or both. If an event is to be printed in the field, a printer with paper is needed.

12. Allow ample time for proper setup of the seismograph. Many errors occur when seismographs are hurriedly set-up. Generally, more than 15 minutes for set-up should be allowed from the time the user arrives at the monitoring location until the blast.

13. Know the temperature. Seismographs have varying manufacturer specified operating temperatures.

14. Secure cables. Suspended or freely moving cables from the wind or other extraneous sources can produce false triggers due to microphonics.

Part II. Ground Vibration Monitoring

Placement and coupling of the vibration sensor are the two most important factors to ensure accurate ground vibration recordings.

A. Sensor Placement

The sensor should be placed on or in the ground on the side of the structure towards the blast. A structure can be a house, pipeline, telephone pole, etc. Measurements on driveways, walkways, and slabs are to be avoided where possible.

1. Location relative to the structure. Sensor placement should ensure that the data obtained adequately represents the ground-borne vibration levels received at the structure. The sensor should be placed within 3.05 meters (10 feet) of the structure or less than 10% of the distance from the blast, whichever is less.

2. Soil density evaluation. The soil should be undisturbed or compacted fill. Loose fill material, unconsolidated soils, flower-bed mulch or other unusual mediums may have an adverse influence on the recording accuracy.

3. The sensor must be nearly level.

4. The longitudinal channel should be pointing directly at the blast and the bearing should be recorded.

5. Where access to a structure and/or property is not available, the sensor should be placed closer to the blast in undisturbed soil.

B. Sensor coupling

If the acceleration exceeds 1.96 m/s^2 (0.2 g), decoupling of the sensor may occur. Depending on the anticipated acceleration levels spiking, burial, or sandbagging of the geophone to the ground may be appropriate.

1. If the acceleration is expected to be:
   a. less than 1.96 m/s^2 (0.2 g), no burial or attachment is necessary
   b. between 1.96 m/s^2 (0.2 g), and 9.81 m/s^2 (1.0 g), burial or attachment is preferred. Spiking may be acceptable.
   c. greater than 9.81 m/s^2 (1.0 g) , burial or firm attachment is required (RI 8506).

The following table exemplifies the particle velocities and frequencies where accelerations are 1.96 m/s^2 (0.2 g) and 9.81 m/s^2 (1.0 g).

Frequency, Hz	4	10	15	20	25	30	40	50	100	200
Particle Velocity mm/s (in/s) at 1.96 m/s^2 (0.2 g)	78.0 (3.07)	31.2 (1.23)	20.8 (0.82)	15.6 (0.61)	12.5 (0.49)	10.4 (0.41)	7.8 (0.31)	6.2 (0.25)	3.1 (0.12)	1.6 (0.06)
Particle Velocity mm/s (in/s) at 9.81 m/s^2 (1.0 g)	390 (15.4)	156 (6.14)	104 (4.10)	78.0 (3.07)	62.4 (2.46)	52.0 (2.05)	39.0 (1.54)	31.2 (1.23)	15.6 (0.61)	7.8 (0.31)

2. Burial or attachment methods.
   a. The preferred burial method is excavating a hole that is no less than three times the height of the sensor (ANSI S2.47), spiking the sensor to the bottom of the hole, and firmly compacting soil around and over the sensor.

   b. Attachment to bedrock is achieved by bolting, clamping or adhering the sensor to the rock surface.

   c. The sensor may be attached to the foundation of the structure if it is located within +/- 0.305 meters (1-foot) of ground level (RI 8969). This should only be used if burial, spiking or sandbagging is not practical.

3. Other sensor placement methods.
   a. Shallow burial is anything less than described at 2a above.

   b. Spiking entails removing the sod, with minimal disturbance of the soil and firmly pressing the sensor with the attached spike(s) into the ground.

c. Sand bagging requires removing the sod with minimal disturbance to the soil and placing the sensor on the bare spot with a sand bag over top. Sand bags should be large and loosely filled with about 4.55 kilograms (10 pounds) of sand. When placed over the sensor the sandbag profile should be as low and wide as possible with a maximum amount of firm contact with the ground.

d. A combination of both spiking and sandbagging gives even greater assurance that good coupling is obtained.

C. Programming considerations
Site conditions dictate certain actions when programming the seismograph.

1. Ground vibration trigger level. The trigger level should be programmed low enough to trigger the unit from blast vibrations and high enough to minimize the occurrence of false events. The level should be slightly above the expected background vibrations for the area. A good starting level is 1.3 mm/s (0.05 in/s).

2. Dynamic range and resolution. If the seismograph is not equipped with an auto-range function, the user should estimate the expected vibration level and set the appropriate range. The resolution of the printed waveform should allow verification of whether or not the event was a blast.

3. Recording duration - Set the record time for 2 seconds longer than the blast duration plus 1 second for each 335 meters (1100 feet) from the blast.

Part III Air Overpressure Monitoring

Placement of the microphone relative to the structure is the most important factor.

A. Microphone placement
The microphone should be placed along the side of the structure, nearest the blast.

1. The microphone should be mounted near the geophone with the manufacturer's wind screen attached.

2. The microphone may be placed at any height above the ground. (ISEE 2005)

3. If practical, the microphone should not be shielded from the blast by nearby buildings, vehicles or other large barriers. If such shielding cannot be avoided, the horizontal distance between the microphone and shielding object should be greater than the height of the shielding object above the microphone.

4. If placed too close to a structure, the airblast may reflect from the house surface and record higher amplitudes. Structure response noise may also be recorded. Reflection can be minimized by placing the microphone near a corner of the structure. (RI 8508)

5. The orientation of the microphone is not critical for air overpressure frequencies below 1,000 Hz (RI 8508).

B. Programming considerations

Site conditions dictate certain actions when programming the seismograph to record air overpressure.

1. Trigger level. When only an air overpressure measurement is desired, the trigger level should be low enough to trigger the unit from the air overpressure and high enough to minimize the occurrence of false events. The level should be slightly above the expected background noise for the area. A good starting level is 20 Pa (0.20 millibars or 120 dB).

2. Recording duration. When only recording air overpressure, set the recording time for at least 2 seconds more than the blast duration. When ground vibrations and air overpressure measurements are desired on the same record, follow the guidelines for ground vibration programming (Part II C.3).

References:

1. American National Standards Institute, Vibration of Buildings – Guidelines for the Measurement of Vibrations and Evaluation of Their Effects on Buildings. ANSI S2.47-1990, R1997.

2. Eltschlager, K. K., Wheeler, R. M. Microphone Height Effects on Blast-Induced Air Overpressure Measurements, 31st Annual Conference on Explosives and Blasting Technique, International Society of Explosives Engineers, 2005.

3. International Society of Explosives Engineers, ISEE Performance Specifications for Blasting Seismographs, 2000.

4. Siskind, D. E., Stagg, M. S., Kopp, J. W., Dowding, C. H. Structure Response and Damage by Ground Vibration From Mine Blasting. US Bureau of Mines Report of Investigations RI 8507, 1980.

5. Siskind, D. E., Stagg, M. S. Blast Vibration Measurements Near and On Structure Foundations, US Bureau of Mines Report of Investigations RI 8969, 1985.

6. Stachura, V. J., Siskind, D. E., Engler, A. J., Airblast Instrumentation and Measurement for Surface Mine Blasting, US Bureau of Mines Report of Investigations RI 8508, 1981.

ISEE Performance Specifications For Blasting Seismographs 2011 Edition

International Society of Explosives Engineers
30325 Bainbridge Road
Cleveland, OH 44139

International Society of Explosives Engineers (ISEE)
Standards Committee

Chairman: Kenneth K. Eltschlager

Committee Members: Douglas Bartley, Steven DelloRusso, Alastair Grogan, Alan Richards, Douglas Rudenko, Mark Svinkin, Robert Turnbull, Randall Wheeler

Disclaimer: These performance specifications are intended to provide design guidelines for blasting seismograph manufacturers. It is incumbent on the blasting seismograph operator to evaluate field conditions, identify the appropriate field criteria and select the proper blasting seismograph for the field application. The operator is responsible for documenting the field conditions and setup procedures in the permanent record for each blast.

Preface: Blasting seismographs are used to establish compliance with regulations that have been established to prevent damage to public and private property. The disposition of the rules is strongly dependant on the accuracy of ground vibration and air overpressure data. One goal of the ISEE Standards Committee is to ensure consistent recording of ground vibrations and air overpressure between all blasting seismographs.

Part I. General Guidelines

Blasting seismographs are deployed in the field to record the levels of blast-induced ground vibration and air overpressure. Accuracy of the recordings is essential. These guidelines define the manufacturers' responsibilities when building blasting seismographs for outdoor field use to measure ground vibrations and air overpressures that will be suitable for comparison to limiting criteria presented in United States Bureau of Mines RI 8507 and RI 8485 which often form the basis of regulations for blast vibrations. Blasting seismographs should be deployed in the field according to the ISEE "Field Practice Guidelines for Blasting Seismographs" (ISEE 2009). The following specifications are considered minimums.

Digital sampling rate……………	1000 samples/sec or greater, per channel
Operating temperature range……	10 to 120F (-12 to 49C)
Electrical cross-talk …………….	Less than 2% of the input signal appears on any other channel

Part II. Ground Vibrations Measurement

Ground vibration sensor response characteristics should conform to the following minimum values:

Frequency range………….	2 to 250 Hz, within zero to -3 dB of an ideal flat response
Accuracy………………….	±5 pct or ±0.5 mm/sec (±0.02 in/sec), whichever is larger, between 4 and 125 Hz.
Phase response………..….	Phase shift between 2.5 Hz to 250 Hz shall not cause an error of more than 10% to the maximum absolute value of two superimposed harmonic vibrations.
Cross-talk response……….	Less than 5% of the excited axis indication on either of the mutually perpendicular channels when excited at the natural frequency of the sensor or at 10 Hz for sensors with a natural frequency greater than 250 Hz.
Density of sensor **...............**	< 2405 kg/m^3 (150 lbs/ft^3) (should be reported for user consideration).

Part III. Air Overpressure Measurement

Air overpressure microphones should conform to the following minimum values:

Frequency range…………………	2 to 250 Hz, -3 dB at 2 and 250 Hz, ±1dB
Accuracy…………………………	±1 dB between 4 and 125 Hz.
Microphone seismic sensitivity….	Microphone response to a mechanical vibration of 50 mm/s (2 in/s) at 30 Hz, from any angle, must be less than 40 dB below the maximum microphone output, or 106 dB whichever is lower.

Part IV. Calibration

To ensure proper operation, blasting seismographs should be calibrated annually by a facility authorized by the manufacturer.

Frequency………………...	Annually
Traceability………………..	Calibration equipment accuracy must be traceable to National Institute Standards and Testing, National Research Council or equivalent.
Certificate …………………	Issued with each calibration and signed by the authorized service representative.
Documentation……………	List the frequencies tested along with input and output values at each frequency. Provide documentation of measured frequency response characteristics.
Ground Vibration Sensor …..	Calibration must be of the assembled sensor. Component calibrations of individual sensors are not appropriate.

Part V. Measurement Practices

In addition to the Performance Specifications described above, blasting seismograph setup or installation in the field is crucial for accurate defensible data acquisition. These measurement practices are specified in the ISEE Field Practice Guidelines for Blasting Seismographs (2009).

Furthermore, some blasting seismograph field needs are specific to an operator, an application, or a region. For example, blasting seismograph use in arctic-type conditions may require good performance at low temperatures or for close-in construction blasting extended frequency ranges might be necessary.

It is the responsibility of the operator to confirm that the blasting seismograph selected for measurement of ground vibrations and air overpressure in conditions not specifically covered by this standard, has performance characteristics to record data consistent with the tolerances described herein.

References:

1. American National Standards Institute, Characteristics to be Specified for Seismic Transducers. ANSI S2.46-1989, R-2005.

2. Deutsches Institut fur Normung (DIN), Mechanical Vibration and Shock Measurement, DIN-45669-1, 1995.

3. International Society of Explosives Engineers. ISEE Field Practice Guidelines for Blasting Seismographs, 2009.

4. Siskind, D. E., Stachura, V. J., Stagg, M. S., Kopp, J. W. Structure Response and Damage Produced by Airblast From Surface Blasting. US Bureau of Mines Report of Investigations 8485, 1980.

5. Siskind, D. E., Stagg, M. S., Kopp, J. W., Dowding, C. H. Structure Response and Damage by Ground Vibration From Mine Blasting. US Bureau of Mines Report of Investigations 8507, 1980.

6. Stachura, V. J., Siskind, D. E., Engler, A. J., Airblast Instrumentation and Measurement for Surface Mine Blasting, US Bureau of Mines Report of Investigations 8508, 1981.

7. Stagg, M. S., Engler, A. J., Measurement of Blast –Induced Ground Vibrations and Seismograph Calibration, US Bureau of Mines Report of Investigations 8506, 1980.

Special Provision

1.0 SCOPE

1.1 General

This special provision describes requirements for vibration monitoring during sheet piling and compaction operations.

The purpose of the vibration monitoring is to directly monitor ground vibrations adjacent to two Enbridge high pressure gas pipelines during culvert replacement works on Highway 34. The two operating pipelines in the vicinity of the proposed construction are as follows:

- 6 inch diameter west pipe;
- 8 inch diameter east pipe.

1.2 General Procedure

To ensure the most accurate measurement of vibrations at the pipeline, the geophone (velocity transducer) must be mounted directly on the nearest day lighted pipeline (at the nearest location to the vibration source). The geophone must be levelled and properly coupled to the pipeline to provide an accurate reflection of the pipeline movement.

1.3 Location

Vibration monitoring shall be conducted for the work carried out within the following minimum separation distances between the vibration source and the nearest pipeline:

- 6 m for compaction of soils or backfill rated at 10,000 ft-lbs or higher;
- 10 m for pile driving;
- 30 m for high-energy dynamic compaction for the rehabilitation of soils.

2.0 REFERENCES

This specification refers to the following standards, specifications, or publications:

Enbridge Gas Distribution Inc., 2007. “Third Party Requirements in the Vicinity of Natural Gas Facilities”. 33 pp.
International Society of Explosive Engineering (ISEE), 2011. “Performance Specifications for Blasting Seismographs”. 4 pp.
International Society of Explosive Engineering (ISEE), 2009. “Field Practice Guidelines for Blasting Seismographs”. 12 pp.

3.0 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Quality Verification Engineer (QVE) means an Engineer with a minimum of five (5) years experience in the field of vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and shall issue certificate(s) of conformance.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Vibration Monitoring Plan

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibrations monitoring specialist;
- Proposed instrumentation;
- Proposed location of instruments;
- Proposed frequency of readings;
- Proposed methods for adjusting compaction methods if readings show vibrations exceeding tolerable levels.

4.2 Reporting

4.2.1 Installation Records

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- Vibration monitoring equipment (type, serial number and calibration date);
- Ground vibration monitoring pipeline ID, location, easting, northing;
- Distance between the vibration sensor and the nearest construction vibration source;
- Dates of installation and datum readings;
- Installation notes / sketches;

4.2.2 Monitoring Records

The Contractor shall take readings during the sheet piling and compaction operations. The readings should be taken and recorded during the entire length of the sheet piling and compaction operations carried out within the minimum separation distances between the vibration source and the nearest pipeline.

Sheet piling and compaction operations should begin in the area furthest from the monitored pipeline(s) to assess the vibration level at the pipeline. If necessary, the contractor must alter the compaction procedures for the remaining works. The revised procedure shall be submitted to the Contract Administrator for approval prior to the remaining compaction operations.

The vibration monitoring reports of recorded intensities shall be provided on a regular basis to Enbridge or at the request of Enbridge. The monitoring report must contain a graph displaying Peak Particle Velocity (PPV) against time in order to confirm that no exceedance of the ground vibration limit has occurred.

4.2.3 Criteria for Assessment of Induced Vibrations

The PPV measured on the pipeline, or at the closest point of the pipeline with respect to the work, shall not exceed 50 mm/s.

The maximum displacement for the vertical and/or horizontal component corresponding to the above stated vibration intensity shall not exceed 50 mm at any given length of the pipeline in question.

5.0 MATERIALS

5.1 General

The Contractor shall supply all materials and equipment required for the installation of the ground vibration monitoring.

5.2 Monitoring Equipment

The seismographs and the operation of the instruments shall comply with the International Society of Explosive Engineering (ISEE) documents:

- Performance Specifications for Blasting Seismographs;
- Field Practice Guidelines for Blasting Seismographs.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.1 General

The geophone (velocity transducer) must be mounted directly on the nearest day lighted pipeline (at the nearest location to the vibration source). The geophone must be levelled. This may require a shallow sand pad (1 cm – 2 cm) so base of geophone contacts ground/pipeline surface evenly. The instruments internal sensor check will confirm if the geophone is performing within its specified tolerances. In order to provide proper coupling of the geophone to the pipeline, it can be a) buried by and firmly compacting soil around and over the sensor, or b) covering the geophone with a sandbag (roughly 5 kg to 10 kg). Failure to provide adequate coupling may result in inaccurate ground vibration measurements.

7.2 Monitoring Frequency

Monitoring of the sheet pile and compaction operations will be completed for work within the separation distances to the nearest pipeline as described in Section 1.3.

The results shall be submitted to the Contract Administrator after each piling or compaction operation prior to continuing with the subsequent compacting operation. As a minimum, the compaction location must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the sheet piling and with the next compaction operation with readings taken during each piling or compaction operation. The results of subsequent operations should be submitted to the Contract Administrator after each has been carried out.

If the readings are not within the limits stated above, the Contractor must alter the sheet piling or compaction procedures until the vibrations are within acceptable levels. The above process must be repeated for each compaction operation. The operations shall resume only when the cause and remedy are established and with the approval of Enbridge.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

Payment at the lump sum Contract price for the above tender item shall be full compensation for all labour, Equipment and Material for completion of the work.

Golder Associates

TECHNICAL MEMORANDUM

DATE March 26, 2015 **PROJECT No.** 12-1121-0193-1310

TO Brad Craig, P.Eng.
Dillon Consulting Limited

FROM Kim Lesage, P.Eng. **EMAIL** Kim_Lesage@golder.com

SETTLEMENT RODS
HIGHWAY 34 CULVERT – SITE NO. 27-266C
TOWNSHIP OF PRESCOTT AND RUSSELL, ONTARIO
W.P. 4110-11-01

This memo provides recommendations for monitoring of settlement during the construction work for the replacement of the culvert at the crossing of Unnamed Creek and Highway 34, just north of the intersection of Nixon Road and the Highway 417 westbound off-ramp in the township of Prescott and Russell, Ontario, W.P. 4110-11-01.

The culvert replacement project also includes a localized minor widening of the existing embankment near the eastern end of the new culvert, which is adjacent to two Enbridge Gas Distribution (Enbridge) high pressure gas mains. Settlement monitoring is required to ensure that the pipes do not settle more than the tolerance value provided by Enbridge (i.e., 50 mm).

A total of four settlement rods should be installed, between the two existing gas mains, at the locations show on Drawing 1. A settlement rod installation detail is also included on Drawing 1.

A non-standard special provision for the settlement rods is attached which outlines the requirements for supply and installation of the settlement rods and subsequent data collection prior to, during and after construction of the widened embankment and replacement of the culvert.

Any significant measured settlement could indicate that a response and corrective measure is needed. The following protocol is therefore recommended:

- If a maximum value of 15 mm (Review Level) relative to the baseline readings is reached, the Contractor shall review or modify the method, rate or sequence of construction to mitigate further ground displacement. If this Review Level is exceeded, the Contractor shall immediately notify the Contract Administrator (CA) and review and discuss response actions. The Contractor shall submit a plan of action to prevent Alert Levels from being reached. All construction work shall be continued such that the Alert Level is not reached.

- If a maximum value of 25 mm (Alert Level) relative to the baseline readings is reached, the Contractor shall cease construction operations, inform the CA and execute pre-planned measures to secure the site, to mitigate further movements. No construction shall take place until the CA deems it is safe to proceed.

We trust that this information is sufficient for your needs. Please contact the undersigned should you have any questions.

Yours truly,

Kim Lesage, P.Eng.
Geotechnical Engineer

Fin Heffernan, P.Eng.
Designated MTO Contact

KSL/FJH/bg
n:\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\foundations\9 - technical memorandums\highway 34 culvert\12-1121-0193 tech memo rev-1 26mar2015.docx

Attachments: Drawing 1 – Culvert Replacement, Unnamed Creek Culvert at Highway 34 Settlement Monitoring Locations and Installation Detail
Non-Standard Special Provision – Settlement Rods

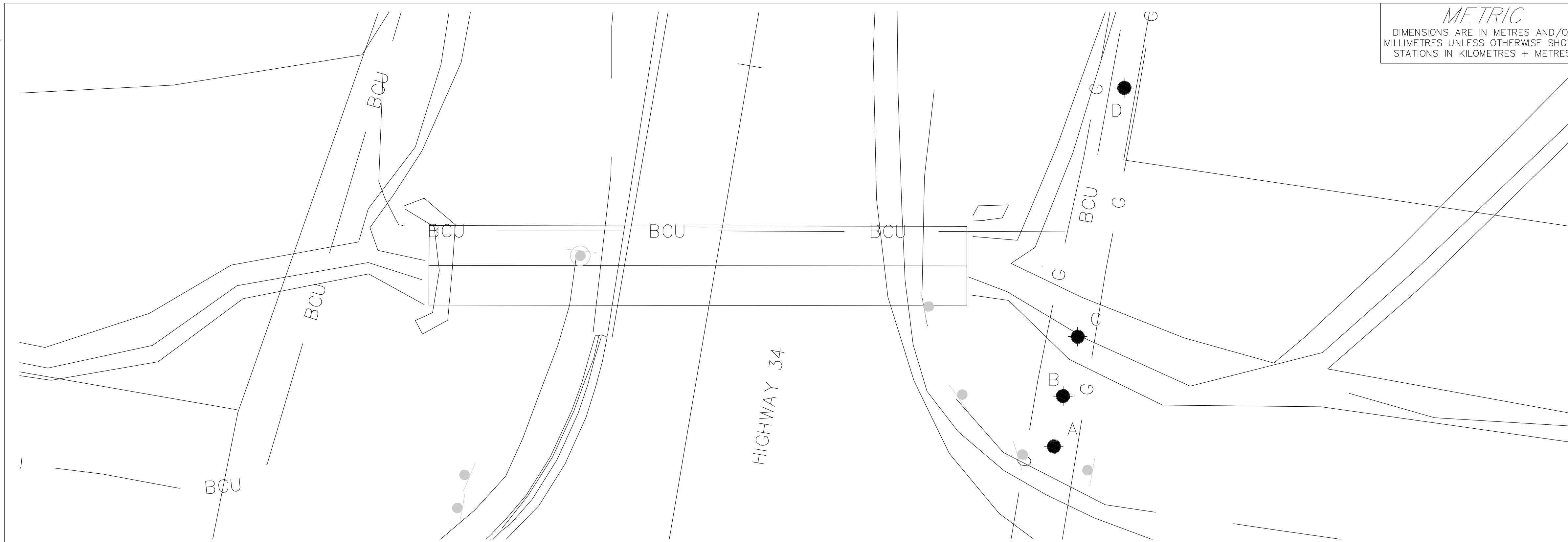
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No. 2015–4011
WP No. 4108–11–00

HIGHWAY 34 CULVERT
SETTLEMENT MONITORING LOCATIONS AND INSTALLATION DETAIL

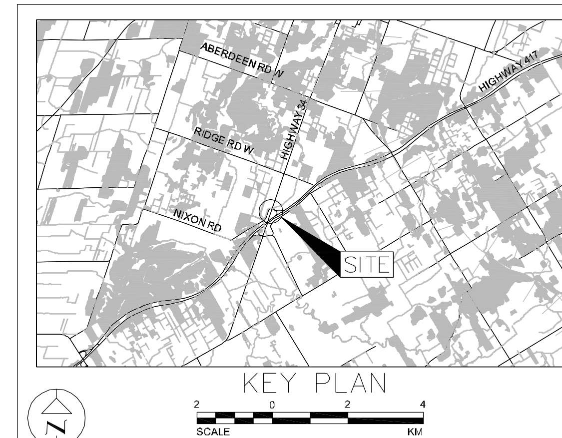
SHEET 15

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

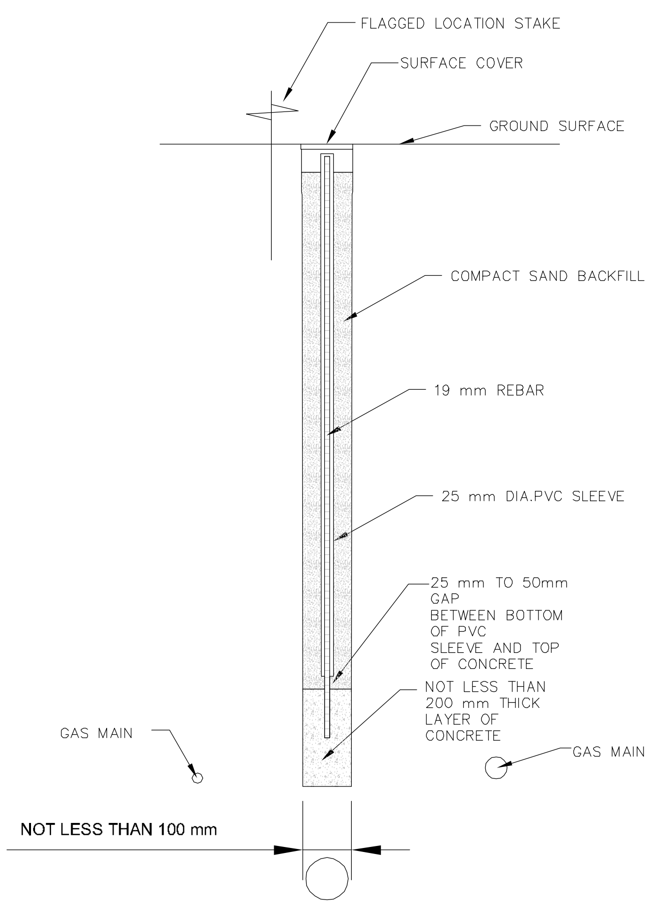
2 0 2 4 m



KEY PLAN

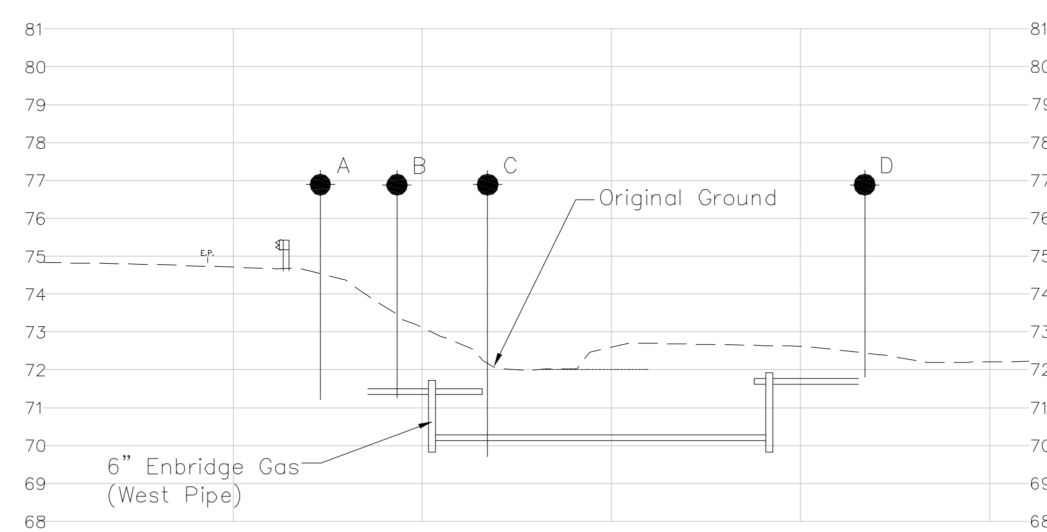
LEGEND

Proposed Settlement Rod



SETTLEMENT ROD
INSTALLATION DETAIL

NOT TO SCALE



PROFILE
SECTION ALONG WEST (6") GAS PIPELINE

HORIZ. SCALE
2 0 2 4 m
2 0 2 4 m
VERT. SCALE

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plan provided in digital format by Dillon, drawing file no. Contract B –BasePlan.dwg, received March 19, 2015.

NO.	DATE	BY	REVISION

Geocres No. 31G–247

HWY. 34		PROJECT NO. 12–1121–0193	DIST.
SUBM'D. KSL	CHKD. KSL	DATE: 4/17/2015	SITE: 27–266/C
DRAWN: JM/ABD	CHKD. KSL	APPD. FJH	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

PLOT DATE: April 17, 2015
FILENAME: N:\Active\2012\1121 - Geotechnical\12-1121-0193 Dillon Mega 3 Eastern Region\Spatial_IM\CAD\-A4- Unnamed Creek Culvert Ph 1310\1211210193AA003.dwg

<u>SETTLEMENT RODS – Item No.</u>

Special Provision

1.0 SCOPE

1.01 General

This special provision contains the requirements for the supply and installation of settlement rods and data collection during construction.

The purpose of the settlement rods is to directly monitor settlements of the soils adjacent to two Enbridge high pressure gas pipelines in areas where existing embankments are widened. Settlement is to be measured by survey of the top of the rod with reference to stable, non-settling benchmarks such as the nearby Highway 417 bridge.

1.02 General Procedure

Rods shall be installed in pre-excavated holes in the ground, at least 100 mm in inner diameter, with the bottom of the rods concreted in-place at the elevation of the pipelines.

Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod.

The holes shall be backfilled with uniform sand, around the rods and sleeves.

Where the rods are located within the roadway width, the rods and sleeves shall be cut down to just below subgrade level and covered with a flush mount surface cover.

1.03 Location

At total of 4 settlement rods are to be installed at the locations displayed on Drawing 1 - Highway 34 Culvert Replacement – Settlement Monitoring Locations and Installation Detail. These should be placed in the ground, between the two gas mains. A typical installation detail is also given on Drawing 1.

2.0 REFERENCES – Not Used

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 REPORTING

4.01.01 Installation Records

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- Settlement rod location, easting, northing;
- Elevation of top of rod;
- Distance between bottom of hole and top of rod;
- Dates of installation and datum readings;
- Installation notes / sketches;
- Description of settlement rod and backfill.

4.01.02 Monitoring Records

The party responsible for monitoring the settlement rods shall record and report the readings to the Contract Administrator within 24 hours of completion of the survey. Each report shall include all survey data collected in tabular and graphical format as plots of time versus settlement in comparison to survey data collected prior to commencement of the work.

4.01.03 Criteria for Assessment of Pipe Subsidence

Based on the monitoring of ground movement as specified in Subsections 4.0, the following represents trigger levels that define magnitude of movement and corresponding action:

- Review Level: If a maximum value of 15 mm relative to the baseline readings is reached, the Contractor shall immediately notify the CA.

- Alert Level: If a maximum value of 25 mm relative to the baseline readings is reached, the Contractor shall immediately notify the CA.

The Contractor shall avoid damaging instrumentation during construction. Instrumentation that is damaged as a result of the Contractor's operation shall be repaired or replaced by the Contractor within one business day. The costs for replacement/repair shall be borne by the Contractor.

5.0 MATERIALS

5.01 General

The Contractor shall supply all materials and equipment required for the installation of the settlement rods.

5.02 Rod

The Contractor shall supply a steel rod with an outside diameter of at least 19 mm.

The top of the rod shall be capped in such a way that a single survey point can be clearly identified and returned to.

5.03 Concrete

The Contractor shall supply concrete, to concrete the rod in-place within each pre-excavated hole.

5.04 Friction Reducing Sleeve

The Contractor shall supply a PVC pipe, friction reducing sleeve with an internal diameter of at least 25 mm.

5.05 Sand

The Contractor shall supply uniform sand to backfill the hole once the rod and friction reducing sleeve are in place.

5.06 Monitoring Equipment

An experienced registered surveyor, retained by the Contractor, to provide the datum readings, shall survey the elevation of the top of the settlement points. The surveyor shall provide suitable equipment capable of surveying settlement rod elevations to an accuracy of +/- 2 mm or better.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.01 INSTALLATION

7.01.01 General

The Contractor shall install settlement rods as detailed elsewhere in the Contract, in addition to what is stated or emphasized below.

7.01.02 Holes

Rods shall be placed in pre-excavated holes, at least 100 mm in inner diameter.

7.01.03 Rod

The rod shall be cemented in-place at the bottom of the hole, up to at least 200 mm from the bottom of the hole.

7.01.04 Friction Reducing Sleeve

The friction reducing sleeve shall extend over the entire length of the rod, with a 25 to 50 mm gap between the bottom of the sleeve and the top of the concrete.

The settlement rod shall be in the centre of the sleeve

7.01.05 Backfill

The annulus between the ground and the friction reducing sleeve shall be filled with sand to a level no higher than the top of the sleeve.

7.01.06 Installation Details

The elevation, easting and northing of the top of the rod shall be surveyed by the Contractor.

The total distance from the bottom of the hole to the top of the rod shall be measured and recorded to an accuracy of +/- 2 mm or better.

The contractor is responsible for preventing damage to the settlement rods during the embankment widening process. If the rod is damaged or the location/inclination of the rod is altered during the filling or other construction activities, the rods and protective casing shall be replaced and surveyed before resuming the filling.

7.02 MONITORING

The settlement rods shall be monitored by a licensed surveyor, under the direction of the Contract Administrator. The Contractor shall meet with the Contract Administrator and staff Responsible for the on-going monitoring immediately after installation of the instruments and before completing the embankment widening. This meeting is referred to as the "hand-over" meeting.

At the meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments and all equipment to be supplied by the Contractor.

Monitoring by others for the baseline readings shall commence within two working days after the "hand over" meeting and prior to placing embankment widening fill. The monitoring shall continue on a schedule described in Section 4.2 throughout the completion of the embankment widening and for approximately six months following the completion of construction or as dictated by the instrumentation readings.

7.02.01 Baseline Readings

Monitoring of the settlement rods shall commence within seven (7) working days after the "hand over" meeting as described elsewhere in the Contract Documents.

Prior to the start of the embankment widening, a minimum of three baseline readings must be obtained. Anomalous readings which cannot be repeated are to be discarded and the average of the remaining readings used as a datum.

7.02.02 Monitoring Frequency

Each settlement rod shall be monitored at the following minimum frequencies:

PERIOD	MINIMUM FREQUENCY
During construction of the embankment widening	Three times daily
Up to 3 months after completion of embankment widening	Once weekly
4 to 6 months after completion of embankment widening	Once monthly

Anomalous readings should be flagged, checked and discarded, if necessary. The reason for the anomalous reading should be identified and corrected, if possible. Damaged settlement rods shall be reported to the Contract Administrator.

The monitoring data should be reviewed and analysed DAILY in order to assess performance of the gas pipelines, and to determine if adjustment of the monitoring schedule or construction methodology or schedule is necessary.

7.03 REMOVAL

After completion of the settlement monitoring period, the settlement rods should be removed to at least 0.3 m below the subgrade by excavating and cutting of the rods and casings. The voids resulting from the removal of the settlement rods should be backfilled with compacted granular.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT

Measurement is by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the number of settlement rods placed. The unit of measurement is each.

10.0 BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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