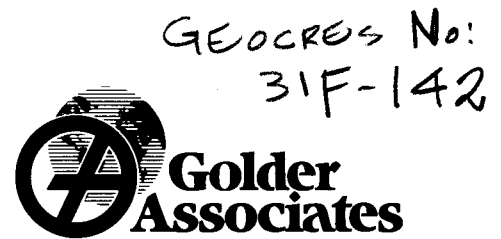


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**FOUNDATION
INVESTIGATION AND DESIGN REPORT
TRANS-CANADA TRAIL
PEDESTRIAN CULVERTS
HIGHWAY 7 TWINNING FROM CARLETON PLACE
TO 3 KM WEST OF JINKINSON ROAD
W.P. 251-99-00**

Submitted to:

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March 2004

021-1155-5



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March 2004

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PART A

**FOUNDATION INVESTIGATION REPORT
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W.P. 251-99-00**

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Lists of Abbreviations and Symbols

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Figure 1 Grain Size Distribution Test Results – Fill

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Marshall Macklin Monaghan (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the twinning of Highway 7 from two to four lanes in the former West Carleton and Goulbourn Townships which are now part of the City of Ottawa, and in Beckwith Township in Lanark County. The sections of Highway 7 included in this assignment extend from Highway 417 westerly 7 km to 3 km west of Jinkinson Road (W.P. 256-99-00), and from 3 km west of Jinkinson Road westerly to Carleton Place (W.P. 251-99-00 and 252-99-00). Foundation investigation services are also required as part of this assignment for the widening of Highway 417 from Highway 417-7 interchange easterly to the CN Rail bridge (W.P. 458-98-00).

Foundation investigation services are required for the following components:

- **W.P. 256-99-00:** New structures at the Highway 417E-7W ramp and Hazeldean Road, a high fill embankment along the Highway 417E-7W ramp, high mast light poles, and overhead signs.
- **W.P. 251-99-00 and 252-99-00:** Five new structures at Appleton Sideroad, Ashton Station Road, Dwyer Hill Road, the Trans-Canada Trail, and Lavallee Creek.
- **W.P. 458-98-00:** Widening of two existing structures (the Carp River bridge and CN Rail overpass) into the existing Highway 417 median area, a 900 m long section of high fill embankment within the Highway 417 median in the vicinity of the CN Rail overpass, and overhead signs.

This report addresses the proposed pedestrian crossings at the Highway 7 – Trans-Canada Trail site.

The terms of reference for the original scope of work and Addenda 1 through 7 issued during the proposal period are outlined in the MTO's Request for Proposal (RFP) and in Golder Associates' Proposal No. P21-1301, dated July 2002. Scope changes (Scope Change No. 1) related to additional borehole investigation work at the abutments of several structures and the high fill embankment on the Highway 417E-7W ramp are outlined in Golder Associates' letters dated November 12, 2002 and November 18, 2002, respectively. Further scope changes (Scope Change No. 2) related to additional borehole investigation work associated with overhead signs, high mast light pole foundations, the high fill embankments at the Hazeldean Road site, and additional investigation work for the south abutment at the Hazeldean Road site, are outlined in Golder Associates' letter dated May 7, 2003.

2.0 SITE DESCRIPTION

The Trans-Canada Trail site is located on the abandoned CP Rail line approximately 900 m east of Ashton Station Road, in the City of Ottawa. The existing Highway 7 is currently grade-separated from the Trans-Canada Trail by a single-span bridge.

The terrain in the vicinity of the site is flat to gently undulating, with the natural ground surface varying from about Elevation 136 m to 138 m, rising eastward. The existing Trans-Canada Trail grade at the site is at about Elevation 137 m to 137.5 m, slightly above the surrounding natural grade. The existing Highway 7 has been constructed on up to 10 m of embankment fill at the site, with the highway grade at about Elevation 146 m over the existing structure.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the Trans-Canada Trail site in November 2002, at which time six boreholes (Boreholes 02-201 to 02-206) were advanced within the limits of the proposed structures, at the locations shown on Drawing 1. All six boreholes were advanced along the north side of the existing Trans-Canada Trail, due to the presence of underground fibre optic cable along the south side. Two boreholes at each structure were extended by coring into the bedrock. Due to the length of each structure and the possible variation in the elevation of the bedrock surface as encountered in Boreholes 02-201 to 02-204, one additional borehole was advanced to refusal at the mid-point of each of the proposed structures.

The boreholes were advanced by hollow stem augers using a bombardier-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to auger and/or sampler refusal which occurred at depths between 0.9 m and 1.2 m below the existing ground surface at the borehole locations. Samples of the overburden were obtained at 0.75 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In four of the six boreholes, about 3 m of bedrock coring was carried out using NQ-size coring equipment. The water level in the open boreholes was observed throughout the drilling operations, and a total of three piezometers were installed (two within the bedrock, and one within the overburden) to monitor the groundwater level(s) at the site.

The field work was supervised on a full-time basis by members of Golder Associates' staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder Associates' laboratory in Ottawa for further examination, and to Golder Associates' laboratory in Mississauga for index and classification testing, consisting of water content determinations and grain size distribution analyses on selected samples.

The borehole locations and ground surface elevations were established by MMM surveyors or were determined by Golder Associates relative to points staked by MMM. 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.

<i>Borehole No.</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
02-201	5,004,437.6	340,464.1	137.5 m
02-202	5,004,420.4	340,445.2	137.4 m
02-203	5,004,400.6	340,423.1	137.3 m
02-204	5,004,381.8	340,402.0	137.2 m
02-205	5,004,391.3	340,412.7	137.2 m
02-206	5,004,429.0	340,454.6	137.4 m

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within two minor physiographic regions, as delineated in *The Physiography of Southern Ontario*¹, that lie within the major physiographic region of the Ottawa-St. Lawrence Lowland. The Highway 7 area between the Highway 417-7 interchange and Carleton Place is part of the Smiths Falls Limestone Plain, while the area along Highway 417 east of the Highway 417-7 interchange is part of the Ottawa Valley Clay Plain. Most of both physiographic regions is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield. The Shield rock generally outcrops to the north of the Ottawa River, and it is also present immediately below the overburden in a localized area between the Hazeldean Fault (approximately the location of the Carp River) and the Ottawa River.

The Smiths Falls Limestone Plain, in which the Trans-Canada Trail site is located, is characterized by shallow overburden deposits overlying limestone bedrock of the Ottawa Formation; this formation consists of grey limestone with some shaly partings and seams.² The shallow overburden soils are typically between 1 m and 3 m in thickness and are commonly comprised of sandy to gravelly till derived from the Precambrian Shield to the north, overlain by glaciofluvial sediments that consist of layered sands and gravels. Large areas of the plain are covered with peat and muck, due to poor drainage as a consequence of the relatively flat topography and shallow depth to bedrock.¹

The Ottawa Valley Clay Plain region, present along Highway 417 from the Highway 417-7 interchange site eastward, is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock.¹ West of the Carp River valley along Highway 417, the upper bedrock consists of limestone of the Ottawa Formation, as described above. Within and immediately east of the Carp River valley, the upper bedrock consists of sandstones and dolostones that have been cut by igneous and metamorphic rocks, controlled by faulting in the vicinity of the Carp River.²

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.

² Belanger, J.R. "Urban Geology of Canada's National Capital Area", in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, six boreholes were advanced within the limits of the two proposed pedestrian culverts. The borehole locations and ground surface elevations are shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 and 2. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the soils encountered immediately below ground surface at this site consist of topsoil and existing fill overlying relatively thin overburden soil consisting of sand to sand and silt. These surficial soils are, in turn, underlain by bedrock that was encountered between 0.9 m and 1.2 m depth (about Elevation 136.1 m to 136.4 m) in the boreholes; the bedrock is comprised of interlayered limestone, sandy dolostone and shale. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections, and stratigraphic profiles and sections of this site are shown on Drawing 1.

4.2.1 Topsoil

Between 100 mm and 150 mm of topsoil was encountered immediately below ground surface in Boreholes 02-201, 02-202 and 02-203.

4.2.2 Fill

Fill, associated with the Trans-Canada Trail construction, was encountered in all of the boreholes advanced as part of this subsurface investigation. The fill ranges from 0.6 m to 0.9 m in thickness, with the base of the fill encountered between Elevations 136.3 m and 136.6 m.

The existing fill consists of sand containing trace to some gravel and trace silt; grain size distribution test results obtained from two samples of the fill are shown on Figure 1. The Standard Penetration Test (SPT) "N" values measured within the existing fill vary from 8 to 9 blows per 0.3 m of penetration, indicating that the fill has a loose state of compaction.

4.2.3 Sand to Sand and Silt

Below the topsoil and existing fill is a sand to sand and silt deposit with a total thickness of between 0.1 m and 0.5 m. This deposit directly overlies bedrock, with the base of this deposit encountered between about Elevation 136.1 m and 136.4 m. The deposit ranges in composition from sand containing trace to some silt and trace gravel, to sand and silt containing trace to some

gravel; a grain size distribution test result obtained for one sample from this deposit is shown on Figure 2. Silty clay seams were observed within this deposit in one of the boreholes. Trace quantities of organic matter were also observed in some of the recovered samples.

Measured Standard Penetration Test (SPT) "N" values in this deposit vary from about 6 to 7 blows per 0.3 m of penetration, indicative of a loose state of compaction.

4.2.4 Bedrock

Bedrock, consisting of interlayered limestone, sandy dolostone and shale, underlies the surficial sand to sand and silt deposit at this site. The surface of the bedrock was encountered between Elevation 136.1 m and 136.4 m (at about 0.9 m to 1.2 m depth). The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations. It should be noted that bedrock was cored in four of the boreholes; the surface of the bedrock was inferred in the two remaining boreholes by refusal to split-spoon sampler and/or auger advance.

<i>Proposed Structure</i>	<i>Borehole Number</i>	<i>Ground Surface Elevation</i>	<i>Depth to Bedrock</i>	<i>Bedrock Surface Elevation</i>
Eastbound Highway 7 Culvert	02-201	137.5 m	1.2 m	136.3 m (Cored)
	02-206	137.4 m	1.0 m	136.4 m
	02-202	137.4 m	1.0 m	136.4 m (Cored)
Westbound Highway 7 Culvert	02-203	137.3 m	0.9 m	136.4 m (Cored)
	02-205	137.2 m	1.1 m	136.1 m
	02-204	137.2 m	1.1 m	136.1 m (Cored)

The limestone, sandy dolostone and shale bedrock at the site is a member of the Ottawa Formation. The limestone, which comprises the upper 0.8 m to 1.8 m of the bedrock that was cored in the boreholes, is grey, fresh, weak to medium strong, and thinly- to medium-bedded (refer to the Lithological and Geotechnical Rock Description Terminology sheet that precedes the borehole records for further information). A sandy dolostone layer is present below about Elevation 134.4 m to 135.2 m; the sandy dolostone is grey to dark grey, fresh, weak to medium strong, and thinly- to medium-bedded. A 400 mm to 500 mm thick interbed of black, very thinly- to thinly-bedded, weak shale was encountered within the sandy dolostone, below about 2.6 m to 3.6 m depth (i.e. below Elevation 133.9 m to 134.6 m). Details of the thickness and depth of the limestone, sandy dolostone and shale layer(s) encountered in each borehole are shown on the Record of Drillhole sheets for Boreholes 02-201 to 02-204.

Rock Quality Designation (RQD) values measured on the bedrock core samples recovered from Boreholes 02-201 to 02-204 ranged from 5 to 40 per cent in the upper 1.5 m of the bedrock, indicative of very poor to poor quality rock. In the lower 1.5 m of recovered rock core, the RQD values ranged from 40 to 60 per cent, indicative of poor to fair quality rock. The discontinuities

observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding and stylolite planes, although some vertical to sub-vertical jointing was also observed.

4.3 Groundwater Conditions


Water was encountered between about 0.7 m and 0.9 m depth (about Elevation 136.3 m to 136.7 m) during drilling. Three piezometers were installed within the boreholes drilled at the site: one within the shallow overburden soils, and two within the bedrock. The water level measured in the piezometers varied from Elevation 136.3 m to 137.4 m in March 2003, and from Elevation 136.9 m to 137.1 m in April 2003, as summarized in the following table:


Borehole No.	Screen Interval	March 21, 2003		March 26, 2003		April 15, 2003	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
02-201	Bedrock	0.0 *	137.5	0.1	137.4	0.4	137.1
02-204	Bedrock	0.0 *	137.2	0.7	136.5	0.2	137.0
02-205	Overburden	0.0 *	137.2	0.9	136.3	0.3	136.9

* NOTE: The water in the piezometers was frozen on March 21, 2003.

Based on the above water level measurements, the groundwater level associated with the overburden and bedrock at the site is at about Elevation 137 m during early spring conditions. It should be noted that groundwater levels are expected to fluctuate seasonally.

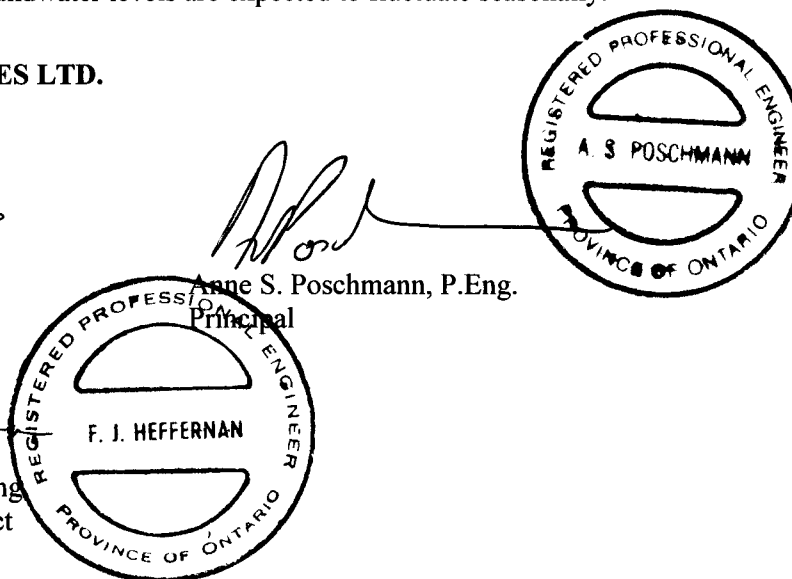
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LCC/ASP/FJH/lcc

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PART B

**FOUNDATION DESIGN REPORT
TRANS-CANADA TRAIL
PEDESTRIAN CULVERTS
HIGHWAY 7 TWINNING FROM CARLETON PLACE
TO 3 KM WEST OF JINKINSON ROAD
W.P. 251-99-00**

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed Trans-Canada Trail pedestrian culvert structures. 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.

It is understood that the existing single-span bridge at the site will be removed, the existing Highway 7 (future westbound lanes) profile will be lowered, and two 4 m x 4 m concrete culverts will be constructed in series to convey the Trans-Canada Trail under the future eastbound and westbound lanes of Highway 7. The existing Trans-Canada Trail grade is at about Elevation 137 m to 137.5 m, and the future Highway 7 eastbound and westbound lanes will be at about Elevation 142.5 m at the structure site. This will involve lowering of the existing Highway 7 (future westbound lanes) embankment by about 3.5 m, and construction of a new embankment up to about 5.5 m in height for the future eastbound lanes.

According to Totten Sims Hubicki's *Structural Planning Report for Highway 7 from Carleton Place to Highway 417*, dated August 2002, open footing, cast-in-place concrete, rigid frame culverts are planned for the site, since these may permit location of the existing fibre optic plant between the footings, without relocation of the plant. It is understood from the TSH report that concrete headwalls and retaining walls are planned to be used at the ends of the concrete culverts to minimize the length of the culverts.

5.2 Culvert and Retaining Wall Foundation Options

The native soils at the site consist of shallow overburden (topsoil, existing fill and surficial sand to sand and silt soils) overlying weak to medium strong, interlayered limestone, sandy dolostone and shale bedrock, the surface of which was encountered in the boreholes between 0.9 m and 1.2 m depth (about Elevation 136.1 m to 136.4 m). The bedrock is suitable for support of the proposed rigid frame culverts and associated concrete headwalls and retaining walls on shallow foundations. Consideration could also be given to the use of concrete box culverts at the site; however, based on the utility locates provided prior to drilling at this site, it is understood that the fibre optic plant is located along the south side of the railway and, therefore, the use of a box culvert would probably

require relocation of the fibre optic plant unless the culvert structures can be shifted northward away from the utility or a smaller culvert can be used. Ultimately, the choice between a rigid frame structure on spread footings or a concrete box structure will have to be determined by the designers, with due consideration given to the cost of relocation of the fibre optic utility.

As an alternative to concrete retaining walls supported on the bedrock, consideration could be given to the use of retained soil system (RSS) walls, supported on the overburden soils or the bedrock at the site.

Recommendations for spread footings for the proposed rigid frame culverts and associated concrete headwalls / retaining walls, for box culverts, and for RSS walls are presented in the following sections; a summary comparison of the advantages, disadvantages, relative costs and risks associated with each of these foundation options is presented in Table 1 following the text of this report. Recommendations are not provided for deep foundations, since these are not a feasible foundation option at this site due to the shallow depth to bedrock below the culvert invert level.

5.3 Spread Footings and Box Culverts

The proposed rigid frame culverts and any associated concrete cantilever wing walls / retaining walls may be supported on spread footings placed on the properly prepared bedrock surface. Box culverts, if adopted, may also be supported on the bedrock or on a granular pad placed on the bedrock if the proposed grade and base slab thickness permit. Given the proposed trail grade of Elevation 137 m and the bedrock surface elevation, this last option is probably not appropriate. The surface of the bedrock was encountered in the boreholes between Elevations 136.1 m and 136.4 m, as summarized in the following table.

<i>Proposed Pedestrian Culvert</i>	<i>Borehole Numbers</i>	<i>Depth to Bedrock</i>	<i>Bedrock Surface Elevation</i>
Eastbound Highway 7	02-201, 02-202 and 02-206	1.0 m to 1.2 m	136.3 m to 136.4 m
Westbound Highway 7	02-203, 02-204 and 02-205	0.9 m to 1.1 m	136.1 m to 136.4 m

Based on the borehole results, there is a slight variability in the bedrock surface within the limits of each of the proposed pedestrian culverts. In addition, the upper portion of the bedrock is fractured (RQD values of 5 to 40 per cent, as encountered in Boreholes 02-201 to 02-204), and it will be necessary to subexcavate and remove any loosened portions of the bedrock exposed at the base of the footing excavation prior to construction of the culvert foundations. MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction, to ensure that all loose and/or fractured rock has been removed from the foundation areas. For

design, the following founding level options for spread footings or for a concrete box may be considered:

1. A founding level of Elevation 136.4 m may be assumed for design of both culverts. In this case, the bedrock surface would have to be exposed and cleaned, and then mass concrete would be placed to raise the grade to the founding level. Provision should be made in the Contract Documents for mass concrete placement to accommodate variations in the bedrock surface. The benefit of this approach is that excavation into the weak to medium strong bedrock is avoided.
2. Alternatively, a founding level of Elevation 136.1 m may be assumed for design of both culverts. In this case, excavation of the higher portions of the bedrock will be required within the foundation footprints. Based on the borehole results, subexcavation of up to about 0.3 m of bedrock will be required in some foundation areas. It is noted that the bedrock is weak to medium strong (corresponding to unconfined compressive strengths in the range of 5 MPa to 50 MPa), making excavation relatively difficult particularly where only small depths are needed. Bedrock excavation could be carried out using hoe ramming techniques; however, line drilling and pre-shearing techniques, if properly executed and inspected, should provide better control over the configuration of the founding surface.
3. As a third option, an intermediate founding level may be assumed for design. In this case, a combination of bedrock subexcavation and mass concrete placement will be required.

Of the above founding options, it is considered that spread footings or a concrete box placed on the bedrock surface or on mass concrete at about Elevation 136.4 m represents the most feasible and economical option from a foundation standpoint. This option minimizes bedrock excavation, which could be time-consuming due to the strength of the bedrock and the difficulty in obtaining a clean cut surface. However, the cost effectiveness of each of the foundation alternatives should be considered in the overall design.

It is noted that excavations to expose the bedrock surface for footing or box culvert construction will extend through water-bearing sand to sand and silt soils. A suitable dewatering scheme will be required in order to maintain a dry and stable excavation.

5.3.1 Geotechnical Resistance

Spread footings or a box structure placed on the properly prepared limestone bedrock or on mass concrete may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 3,000 kPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock and mass concrete are considered to be unyielding materials; as such, ULS conditions will govern the design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination and eccentricity of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings or box structure and the limestone bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \phi'$, may be taken as 0.70 for cast-in-place concrete constructed on the bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. If dowelling into bedrock is adopted at this site, a Special Provision should be included in the Contract Documents to specify the installation, materials and testing of the dowels.

5.3.3 Frost Protection

For spread footings or a concrete box structure founded on the properly prepared limestone bedrock at this site, frost susceptibility is not an issue.

5.4 Retained Soil System (RSS) Walls

A mechanically-reinforced soil retaining wall system (retained soil system or RSS wall) consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material.

The use of RSS walls is considered an appropriate alternative to concrete walls for the proposed retaining walls at the ends of the pedestrian culverts. This is particularly the case if the existing single-span bridge and walls on Highway 7 are removed in an open-cut excavation (in which case much of the embankment fill in the proposed RSS wall area would be removed). Based on the proposed Highway 7 grade of approximately Elevation 142.5 m in the immediate vicinity of the culverts, the highway embankments will be about 5.5 m high at the site. It is anticipated, based on standard embankment geometry, that RSS walls at this site would be up to about 4 m high.

For a typical RSS wall, the front facing panels are supported on a strip footing placed at shallow depth below the ground surface in front of the wall. This footing must be founded below any topsoil, loose fill or unsuitable native soils. For an assumed footing width of 0.6 m, and assuming that the footing is placed on the properly prepared surficial sand to sand and silt deposit, a factored geotechnical resistance at ULS of 40 kPa may be used for design. Given the relatively shallow depth to bedrock, if a higher geotechnical resistance is required consideration could be given to placement of the facing footing directly on the bedrock, or on a compacted Granular "A" pad constructed on the bedrock. A factored geotechnical resistance at ULS of 3,000 kPa or 80 kPa may be used for footings placed directly on the bedrock or for footings placed on a compacted Granular "A" pad on the bedrock, respectively.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the wall, the following factored geotechnical resistances at ULS may be used for design of RSS walls founded below any topsoil or loose fill on the properly prepared surficial sand to sand and silt deposit:

<i>Wall Height</i>	<i>Assumed Footing Width</i>	<i>Factored Geotechnical Resistance at ULS</i>
2 m	1.3 m	80 kPa
4 m	2.7 m	120 kPa

The settlement of the soils underlying the RSS walls will be less than 25 mm for the ULS conditions given above; therefore, the factored geotechnical resistance at ULS will govern the design of RSS walls at this site. The majority of the settlement of the RSS walls will occur during construction since the founding soils are essentially granular, overlying bedrock at a shallow depth.

As an alternative to founding on the surficial sand to sand and silt deposit, the RSS walls could be supported on the bedrock. The advantage of this option is that less settlement of the founding soils would occur; however, it is noted that excavations to expose the bedrock surface will extend through water-bearing sand to sand and silt soils, and groundwater control would be required to maintain a dry and stable excavation. The design founding levels and geotechnical resistance for this option are discussed in Sections 5.3 and 5.3.1.

The resistance to lateral forces / sliding resistance between the compacted Granular "A" of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*, using a coefficient of friction, $\tan \delta$, as given in the table below. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

<i>Subgrade</i>	Coefficient of Friction ($\tan \phi'$)
Loose surficial sand to sand and silt	0.4
Limestone bedrock	0.57

The internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier / designer. In this regard, the internal stability must also be checked for seismic loading. The Factor of Safety related to global stability under static loading for properly designed and constructed RSS walls at this site is greater than 1.3.

For RSS walls founded on soil, the liquefaction potential of the soils below the RSS wall under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, a factor of safety of greater than 1.1 against liquefaction for an earthquake of magnitude 7.5 is obtained for the surficial sand to sand and silt soils below the water table. Pseudo-static stability analysis indicates that the ground surface acceleration due to the design earthquake event does not result in global instability of the RSS wall.

5.5 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the proposed culvert walls and headwalls / retaining walls. For rigid frame or box culverts, it is assumed that lateral yielding will not be allowed and, as such, at-rest earth pressures will apply; for any associated headwalls or retaining walls where lateral yielding may be allowed, active earth pressures will apply. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls; adjustment must be made for sloping ground surface behind the headwalls or retaining walls adjacent to the ends of the culverts.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B', Type II (but with less than 5 per cent passing the 200 sieve) should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 taper should be in accordance with OPSD 3501.00 and 3504.00.
- 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 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06; heavy compaction equipment should not be used within a lateral distance behind the structure equal to the current height of the fill above the base of the structure. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case I in Figure C6.9.1(I) of the *Commentary to the CHBDC*) 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 in Figure C6.9.1(I) of the *Commentary to the CHBDC*).
- 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	0.35
At rest, K_o	0.50

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

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

- 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 National Building Code of Canada, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio for Ottawa is 0.18. Based on experience, for the subsurface conditions at this site, a 10 to 20 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.18g to between 0.2g and 0.22g. 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 the proposed rigid frame culverts which do not allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.33$). For structures which allow lateral yielding, such as wingwalls associated with the culverts, k_h , is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.11$). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3 k_h$, $k_v = 0$, and $k_v = -2/3 k_h$.
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. 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.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.40	0.32	0.36
Non-yielding wall	0.80	0.63	0.71

- 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 55 mm 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:

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

where $\sigma_h(d)$ is the lateral earth pressure at a given depth (kPa);
 K_a is the static active earth pressure coefficient;
 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 total height of the wall (m).

5.6 Highway Embankment Design and Construction

The twinning of Highway 7 at the Trans-Canada Trail site will involve lowering of the existing Highway 7 (future westbound lanes) embankment by about 3.5 m, and construction of a new embankment up to about 5.5 m in height for the future eastbound lanes. Based on the borehole results, the new eastbound highway embankment subgrade soils will consist of loose to compact, surficial sand to sand and silt.

5.6.1 Subgrade Preparation and Embankment Construction

Any topsoil, organic matter and softened / loosened soils should be stripped from below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement. Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

For the future westbound lanes (i.e. existing Highway 7), the use of granular fill is recommended for any widening of the existing embankment, and for backfill following removal of the existing single-span bridge structure, in order to minimize differential settlement between the new embankment areas and the existing embankment fill. The majority of settlement of granular fills will occur during construction. The new embankment fill should be keyed by benching into the existing embankment side slopes and into the temporary cut slopes, to reduce the impact of differential settlement. Benching should be carried out in accordance with OPSD 208.01.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended. It is noted that ditching alongside the embankment may extend below the existing groundwater level at the site. The cuts should be inspected after completion to check for evidence of water seepage which could affect the surficial stability. It is recommended that remedial measures, such as a granular blanket, be placed where seepage is present.

5.6.2 Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, approach embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety of greater than 1.3 against deep-seated slope instability.

The liquefaction potential of the soils below the embankment under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the *CHBDC Commentary*,

which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, a factor of safety of greater than 1.1 against liquefaction for an earthquake of magnitude 6.2 is obtained for the surficial sand to sand and silt soils below the water table.

Although the site soils are not considered to be liquefiable, there will still be some deformation of the soils under seismic loading conditions. Pseudo-static methods of slope stability analysis indicate a yield acceleration of approximately 0.2g is required to reduce the factor of safety against slope instability to 1.0. Using this result and the simplified Newmark method, embankment deformations as a result of the design earthquake event are anticipated to be less than 25 mm. This magnitude of deformation will not cause deep-seated global instability, but localized embankment toe failures or surficial slippages could occur, which would result in steepening of the embankment side slopes. The maintenance issues associated with these types of deformations should be considered in the life cycle costing when assessing the relative costs of the works.

5.6.3 Embankment Settlement

Settlement of the Highway 7 embankments at the site will occur due to compression of the new embankment fill itself, as well as compression of the relatively thin, cohesionless overburden soils. Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the embankment fill itself is expected to be less than 25 mm. As discussed in Section 5.6.1, the use of granular fill for the new embankment construction would reduce this magnitude, since the majority of settlement of granular fills will occur during construction.

Settlement analyses for the foundation soils were carried out using the commercially available computer program Unisettle. The immediate compression of the loose sand to sand and silt deposit was modelled using an elastic deformation modulus based on correlations with the measured SPT "N" values, as summarized in the following table:

<i>Soil Unit</i>	<i>Bulk Unit Weight</i>	<i>Elastic Modulus</i>
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m ³	–
Loose sand to sand and silt	19 – 20 kN/m ³	10 – 20 MPa

Provided that proper subgrade preparation is carried out, the settlement of the cohesionless foundation soils for the immediate approach embankments along Highway 7 is expected to be less than 15 mm, as a result of the construction of approach embankments up to 5.5 m in height. This compression is expected to occur during construction.

5.7 Excavation and Groundwater / Surface Water Control

Excavations for construction of spread footings or box culverts will typically extend through about 0.9 m to 1.2 m of loose sand fill and loose, surficial sand to sand and silt soil. The surface of the limestone bedrock is between Elevations 136.1 m and 136.4 m. The groundwater level at the site is at about Elevation 136.5 m, slightly above the surface of the bedrock.

Based on the shallow depth to bedrock and the relatively open site conditions, it is anticipated that temporary open-cut excavations will be suitable for most of the construction areas. Assuming that the construction is staged such that the future eastbound lanes are constructed first, then traffic is rerouted onto the eastbound lanes, it is likely that the removal of the existing single-span bridge on the existing Highway 7 (future westbound lanes) could also be carried out in an open-cut excavation. Removal in open-cut excavation is preferable from the point of view of embankment reconstruction, and is also likely to be more economical than installation of a temporary shoring system at the site. However, recommendations for temporary excavation support as well as open-cut excavation are provided in the following subsections.

5.7.1 Temporary Open-Cut Excavations

Excavations for construction of spread footings or box culverts, or for removal of the existing single-span bridge, should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and water-bearing surficial sand to sand and silt soils are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through the existing highway embankment fill and the overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) assuming that the overburden soils are dewatered. Some sloughing of the side slopes at this inclination is possible, particularly if full dewatering is not achieved.

The groundwater level at the site is generally at or less than about 1 m below the natural ground surface, and excavations to expose the bedrock surface will require groundwater control. The shallow depth to bedrock at the site will dictate the type of groundwater control system that may be used. It is likely that open-cut excavations with sufficient sumping will adequately control the groundwater; in this case, however, the excavation side slopes will probably have to be maintained at about 3H:1V.

5.7.2 Temporary Excavation Support for Removal of Existing Bridge

If space and/or construction staging restrict the use of an open-cut excavation for removal of the existing single span bridge, a temporary excavation support system could be constructed to support the excavation through the existing Highway 7 embankment. Based on the subsurface

conditions at the site and the likely excavation geometry, it is anticipated that a soldier pile and lagging system using rakers to provide lateral support would be suitable. It may be necessary to socket the soldier piles into the bedrock to provide sufficient lateral resistance at the pile toe once the excavation is advanced.

The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. Assuming that the construction is staged such that traffic is diverted onto the new Highway 7 eastbound lanes during the removal of the existing bridge, the lateral movement of the temporary shoring system should meet Performance Level 3 as specified in SP 539S01. If traffic must remain on the existing Highway 7 (future westbound lanes) while the existing structure removed in sections, then the lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 539S01.

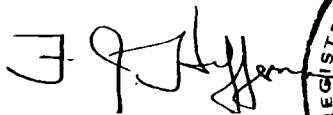
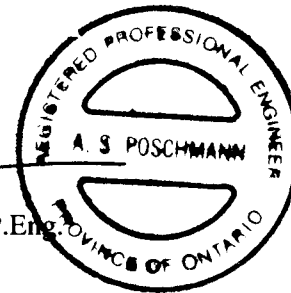
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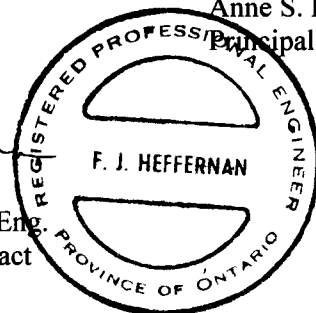
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TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
TRANS-CANADA TRAIL PEDESTRIAN CULVERTS

	<i>Foundation Option</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks and Consequences</i>
CULVERTS	Rigid frame with spread footings founded on bedrock	<ul style="list-style-type: none"> No risk of settlement If higher founding level (Elevation 136.4 m) selected, placement of mass concrete is readily constructable May permit fibre optic plant to remain in place between or outside footings 	<ul style="list-style-type: none"> If lower founding level (Elevation 136.1 m) selected, excavation into medium strong limestone bedrock would be required Appropriate dewatering required during excavation and construction 	More expensive (in-situ construction)	Low risk
	Concrete box founded on bedrock	<ul style="list-style-type: none"> No risk of settlement If higher founding level (Elevation 136.4 m) selected, placement of mass concrete or granular pad to provide level bearing surface is readily constructable 	<ul style="list-style-type: none"> If lower founding level (Elevation 136.1 m) selected, excavation into medium strong limestone bedrock would be required Appropriate dewatering required during excavation and construction Relocation of the fibre optic plant may be required, if it is not possible to shift structure northward or use a smaller culvert. 	Less expensive (pre-cast construction)	Low risk
WALLS	Retained soil system (RSS) wall founded on soil	<ul style="list-style-type: none"> Minimizes excavation depth/width and need for groundwater control during construction Minimal excavation required to construct WBL walls if existing bridge structure is removed in open-cut excavation 	<ul style="list-style-type: none"> Possibly larger excavation width compared to concrete retaining wall More settlement (although still minimal) than for concrete retaining wall founded on bedrock 	Less expensive than concrete wall	Risk of some settlement
	Retained soil system (RSS) wall founded on bedrock	<ul style="list-style-type: none"> No risk of settlement of founding material 	<ul style="list-style-type: none"> Possibly larger excavation width compared to concrete retaining wall Appropriate dewatering and bedrock surface preparation required during excavation and construction 	Less expensive than concrete wall, but more expensive than RSS wall founded on soil	Low risk
	Concrete retaining wall with footing founded on bedrock	<ul style="list-style-type: none"> No risk of settlement Potentially smaller excavation width (especially within existing Highway 7 embankment) compared to RSS wall 	<ul style="list-style-type: none"> Appropriate dewatering and bedrock surface preparation required during excavation and construction 	More expensive than RSS wall	Low risk

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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

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

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.) drive open sampler for a distance of 300 mm (12 in.)

(b) Cohesive Soils

Consistency	c_u, 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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G_s)
DS	direct shear test
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 (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to 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
F	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 effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

- Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of 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 - 2 mm
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 to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and 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° angle is horizontal.

Description and Notes

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

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	



PROJECT 021-1155-5		RECORD OF BOREHOLE No 02-201		1 OF 1	METRIC
W.P. 251-99-00	LOCATION N 5004437.6, E 340464.1			ORIGINATED BY P.A.H.	
DIST HWY 7	BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger			COMPILED BY M.I.C.	
DATUM Geodetic	DATE Nov. 11, 2002			CHECKED BY L.C.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L	WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES							
137.5	Ground Surface											
0.0	Topsoil											
0.1	Sand, trace gravel and silt (Fill) Loose Brown Moist		1	AS	-		137					9 82 (9)
136.6	Sand, trace silt, trace organic matter Dark brown Moist		2	SS	7							
136.3	Sand and silt, some gravel with silty clay seams Loose Grey brown Moist						136					
1.2	Interlayered LIMESTONE, SANDY DOLOSTONE and SHALE (BEDROCK) Fresh Weak to medium strong Very thinly to medium bedded Grey to black						135					
	Bedrock cored between 1.2 m and 4.3 m depth. For Bedrock coring details refer to Record of Drillhole 02-101						134					
133.2	End of Borehole											
4.3	Notes: 1. Water level in piezometer at 0.7 m depth (Elev. 136.7m) on Nov. 11, 2002. 2. Water level in piezometer measured frozen at ground surface (Elev. 137.5m) on March 21, 2003, 0.1m depth (Elev. 137.4m) on March 26, 2003, and at 0.4m depth (Elev. 137.1m) on April 15, 2003.											

PROJECT: 021-1155-5

RECORD OF DRILLHOLE: 02-201

SHEET 1 OF 1

LOCATION: N 5004437.6, E 340464.1

DRILLING DATE: Nov. 11, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE-FAULT				SM-SMOOTH				FL-FLEXURED				BC-BROKEN CORE				DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)					CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK									
									SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY		B-BEDDING									
									VN-VEIN		S-SLICKENSIDED		PL-PLANAR		C-CURVED											
RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY																				
TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	K, cm/sec	10 ⁻⁸	10 ⁻⁵	10 ⁻²																	
		Refer to previous page		136.22																						
2	Rotary Drill NQ Core	LIMESTONE (BEDROCK) Fresh Weak to medium strong Thinly to medium-bedded Grey		1.24	1		100																			
3		SANDY DOLOSTONE (BEDROCK) Fresh Weak to medium strong Thinly to medium-bedded Dark grey		134.41 3.05	2	100																				
		SHALE (BEDROCK) Fresh Weak Very thinly to thinly bedded		133.89 3.57																						
4		Black SANDY DOLOSTONE (BEDROCK) Fresh Weak to medium strong Medium-bedded Grey		133.41 4.05 133.17 4.29																						
5		End of Borehole																								
6																										
7																										
8																										
9																										
10																										
11																										

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: L.C.C.

MISS. ROCK ROCK1155-5.GPJ GLDR CAN.GDT 1/8/03

PROJECT 021-1155-5		RECORD OF BOREHOLE No 02-202		1 OF 1 METRIC	
W.P. 251-99-00		LOCATION N 5004420.4, E 340445.2		ORIGINATED BY P.A.H.	
DIST HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger		COMPILED BY M.I.C.	
DATUM Geodetic		DATE Nov. 11, 2002		CHECKED BY L.C.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								20 40 60 80 100					W _p W W _L				
137.4	Ground Surface																
0.0	Topsoil																
0.1	Sand, trace gravel and silt (Fill) Brown Moist		1	AS	-												
136.5	Sand and silt, trace gravel Loose to compact Grey-brown Moist		2	SS	6/0 15												
1.0	Interlayered LIMESTONE, SANDY DOLOSTONE and SHALE (BEDROCK) Fresh Weak to medium strong Very thinly to medium-bedded Grey to black Bedrock cored between 1.0 m and 4.2 m depth. For bedrock coring details refer to Record of Drillhole 02-102																
133.2	End of borehole																
4.2	Notes: * Split-spoon bouncing after 6 blows / 0.15m of penetration. 1. Water encountered at about 0.9m depth (Elev. 136.5m) during overburden drilling.																

PROJECT: 021-1155-5

RECORD OF DRILLHOLE: 02-202

SHEET 1 OF 1

LOCATION: N 5004420.4, E 340445.2

DRILLING DATE: Nov. 11, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE-FAULT										SM-SMOOTH			FL-FLEXURED			BC-BROKEN CORE			DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									CL-CLEAVAGE			J-JOINT			R-ROUGH			UE-UNEVEN			MB-MECH. BREAK								
									SH-SHEAR			P-POLISHED			ST-STEPPED			W-WAVY			B-BEDDING								
									VN-VEIN			S-SLICKENSIDED			PL-PLANAR			C-CURVED											
RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		TYPE AND SURFACE DESCRIPTION		10 ⁻⁵		10 ⁻⁴		10 ⁻³													
TOTAL CORE %		SOLID CORE %		5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80 85 90 95 100		0 10 20 30 40 50 60 70 80 90 100		0 10 20 30 40 50 60 70 80 90 100		0 10 20 30 40 50 60 70 80 90 100		0 10 20 30 40 50 60 70 80 90 100		0 10 20 30 40 50 60 70 80 90 100		0 10 20 30 40 50 60 70 80 90 100													
2	Rotary Drill NQ Core	Refer to previous page		136.35																									
		LIMESTONE (BEDROCK) Fresh Weak to medium strong Thinly to medium-bedded Grey		1.04																									
3	Rotary Drill NQ Core	SANDY DOLOSTONE (BEDROCK) Fresh Weak to medium strong Thinly to medium-bedded Dark grey		134.72																									
		SHALE (BEDROCK) Fresh Weak Very thinly to thinly bedded Black		2.67																									
4	Rotary Drill NQ Core	SANDY DOLOSTONE (BEDROCK) Fresh Weak to medium strong Medium-bedded Grey		134.16																									
				3.23																									
5	Rotary Drill NQ Core			133.73																									
				3.66																									
6	Rotary Drill NQ Core			133.17																									
				4.22																									
7	Rotary Drill NQ Core																												
8	Rotary Drill NQ Core																												
9	Rotary Drill NQ Core																												
10	Rotary Drill NQ Core																												
11	Rotary Drill NQ Core																												

MISS. ROCK ROCK1155-5.GPJ GLDR CAN.GDT 1/8/03

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: L.C.C.

PROJECT 021-1155-5		RECORD OF BOREHOLE No 02-203		1 OF 1	METRIC
W.P. 251-99-00		LOCATION N 5004400.6, E 340423.1		ORIGINATED BY P.A.H.	
DIST HWY 7		BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger		COMPILED BY M.I.C.	
DATUM Geodetic		DATE Nov. 11, 2002		CHECKED BY L.C.C.	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							WATER CONTENT (%) 20 40 60 80 100
137.3	Ground Surface					▽	137							3 88 (9)	
0.0	Topsoil														
0.2	Sand, trace gravel and silt (Fill) Loose Brown Moist		1	SS	8										
136.5			2	SS	7										
136.4	Sand and silt, trace gravel, trace organic material Loose Dark brown Wet						136								
0.9	Interlayered LIMESTONE, SANDY DOLOSTONE and SHALE (BEDROCK) Fresh Weak to medium strong Very thinly to medium-bedded Grey to black Bedrock cored between 0.9 m and 3.9 m depth. For bedrock coring details refer to Record of Drillhole 02-103							135							
									134						
133.4	End of borehole														
3.9	Note: Water encountered at about 0.6m depth (Elev. 136.7m) during overburden drilling.														

MISS_MTO 021-1155-5.GPJ ON_MOT_GDT 22/7/03

PROJECT: 021-1155-5

RECORD OF DRILLHOLE: 02-203

SHEET 1 OF 1

LOCATION: N 5004400.6, E 340423.1

DRILLING DATE: Nov. 11, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR/FX-FRACTURE-FAULT CL-CLEAVAGE J-JOINT SM-SMOOTH SH-SHEAR P-POLISHED R-ROUGH FL-FLEXURED VN-VEIN S-SLICKENSIDED PL-PLANAR UE-UNEVEN C-CURVED										BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION		
								RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec								
								TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS		TYPE AND SURFACE DESCRIPTION								
								00000	00000			00000	00000	00000	00000	00000	00000				00000	00000
1		Refer to previous page		136.37																		
	Rotary Drill NQ Core	LIMESTONE (BEDROCK)		0.94																		
		Fresh																				
		Weak to medium strong																				
		Thinly to medium-bedded																				
		Grey																				
2		SANDY DOLOSTONE (BEDROCK)		134.99																		
		Fresh		2.32																		
		Weak to medium strong																				
		Thinly to medium-bedded																				
		Dark grey		134.57																		
3	SHALE, (BEDROCK)		2.74																			
	Fresh																					
	Weak		134.05																			
	Very thinly to thinly bedded		3.26																			
	Black																					
	SANDY DOLOSTONE (BEDROCK) with shale interbeds																					
	Fresh																					
	Weak to medium strong																					
	Thinly to medium-bedded																					
	Grey																					
	End of Borehole																					
4				133.37																		
				3.94																		
5																						
6																						
7																						
8																						
9																						
10																						

MISS. ROCK ROCK1155-5.GPJ GLDR. CAN GDT. 1/8/03

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: L.C.C.

PROJECT		2021-1155-5		RECORD OF BOREHOLE No 02-204		1 OF 1		METRIC												
W.P.		251-99-00		LOCATION		N 5004381.8, E 340402.0		ORIGINATED BY P.A.H.												
DIST		HWY 7		BOREHOLE TYPE		CME 55 Bombardier, 108mm I.D. Hollow Stem Auger		COMPILED BY M.I.C.												
DATUM		Geodetic		DATE		Nov. 11, 2002		CHECKED BY L.C.C.												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ			GR SA SI CL		
137.2	0.0	Ground Surface Sand, some gravel, trace silt (Fill) Loose Brown Moist		1	SS	9		137	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED			w _p w w _L			kN/m ³					
136.6	0.8	Sand and silt, trace organics Loose Dark brown		2	SS	2/0.15		136				o			6 41 41 12					
136.1	1.1	Sand and silt, trace gravel Loose Grey-brown Wet Interlayered LIMESTONE, SANDY DOLOSTONE and SHALE (BEDROCK) Fresh Weak to medium strong Very thinly to medium-bedded Grey to black						135												
		Bedrock cored between 1.1 m and 4.3 m depth. For bedrock coring details refer to Record of Drillhole 02-104						134												
132.9	4.3	End of borehole						133												
<p>Notes:</p> <p>* Split-spoon bouncing after 2 blows / 0.15m of penetration.</p> <p>1. Water encountered at 0.6m depth (Elev. 136.6m) during overburden drilling.</p> <p>2. Water level in piezometer frozen at ground surface (Elev. 137.2m) on March 21, 2003, and measured at 0.7m depth (Elev. 136.5m) on March 26, 2003, and at 0.2m depth (Elev. 137.0m) on April 15, 2003.</p>																				

PROJECT: 021-1155-5

RECORD OF DRILLHOLE: 02-204

SHEET 1 OF 1

LOCATION: N 5004381.8, E 340402.0

DRILLING DATE: Nov. 11, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Bombardier

DRILLING CONTRACTOR: Marathon

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE-F-FAULT										SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
									CL-CLEAVAGE		J-JOINT		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK									
									SH-SHEAR		P-POLISHED		ST-STEPPED		W-WAVY		B-BEDDING									
									VN-VEIN		S-SLICKENSIDED		PL-PLANAR		C-CURVED											
RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K _s cm/sec																		
TOTAL CORE %	SOLID CORE %					TYPE AND SURFACE DESCRIPTION																				
8888	8888																									

		Refer to previous page		136.05																		
		LIMESTONE (BEDROCK)		1.14																		
		Fresh																				
		Weak to medium strong																				
		Thinly to medium-bedded																				
		Grey																				
2		SANDY DOLOSTONE (BEDROCK)		135.21	1		100															
		Fresh		1.98																		
		Weak to medium strong																				
		Thinly bedded																				
		Dark grey																				
		SHALE (BEDROCK)		134.60																		
		Fresh		2.59																		
		Weak																				
3		Very thinly to thinly bedded		134.14																		
		Black		3.05																		
		SANDY DOLOSTONE (BEDROCK)																				
		Fresh																				
		Weak to medium strong		133.72	2		100															
		Thinly to medium-bedded		3.47																		
		Grey																				
		LIMESTONE (BEDROCK) with shale																				
		seams																				
		Fresh																				
		Weak to medium strong		132.92																		
		Thinly to medium-bedded		4.27																		
		Grey																				
		End of Borehole																				
5																						
6																						
7																						
8																						
9																						
10																						
11																						

DEPTH SCALE





1 : 50



LOGGED: P.A.H.

CHECKED: L.C.C.

MISS. ROCK ROCK1155-5.GPJ GLDR CAN.GDT 1/8/03

PROJECT 021-1155-5			RECORD OF BOREHOLE No 02-205			1 OF 1			METRIC										
W.P. 251-99-00			LOCATION N 5004391.3, E 340412.7			ORIGINATED BY P.A.H.													
DIST HWY 7			BOREHOLE TYPE CME 55 Bombardier, 108mm I.D. Hollow Stem Auger			COMPILED BY M.I.C.													
DATUM Geodetic			DATE Nov. 11, 2002			CHECKED BY L.C.C.													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ					
137.2 0.0	Ground Surface Sand, some gravel, trace silt (Fill) Grey-brown Moist		1	AS	-		137	20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			10 20 30 W _p W W _L			kN/m ³			GR SA SI CL		
136.3																			
136.1	Sand and silt, some gravel Brown		2	AS	-														
1.1	End of Borehole Refusal to Auger Penetration Notes: 1. Water level in open borehole at 0.9m depth (Elev. 136.3m) on completion of drilling. 2. Water level in piezometer frozen at ground surface (Elev. 137.2m) on March 21, 2003, and measured at 0.9m depth (Elev. 136.3m) on March 26, 2003, and at 0.3m depth (Elev. 136.9m) on April 15, 2003.																		

PROJECT <u>021-1155-5</u>		RECORD OF BOREHOLE No 02-206		1 OF 1	METRIC
W.P. <u>251-99-00</u>	LOCATION <u>N 5004429.0, E 340454.6</u>	ORIGINATED BY <u>P.A.H.</u>			
DIST <u>HWY 7</u>	BOREHOLE TYPE <u>CME 55 Bombardier, 108mm I.D. Hollow Stem Auger</u>	COMPILED BY <u>M.I.C.</u>			
DATUM <u>Geodetic</u>	DATE <u>Nov. 11, 2002</u>	CHECKED BY <u>L.C.C.</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT										UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)						
137.4	Ground Surface																		
0.0	Sand, trace gravel and silt (Fill) Loose Brown Moist		1	SS	8														
			2	AS	-														
136.5	Sand and silt, trace gravel and organics Grey-brown End of Borehole Refusal to Auger Penetration		3	AS	-														
1.0	Note: Water level in open borehole at 0.7 m depth (Elev. 136.7m) on completion of drilling.																		

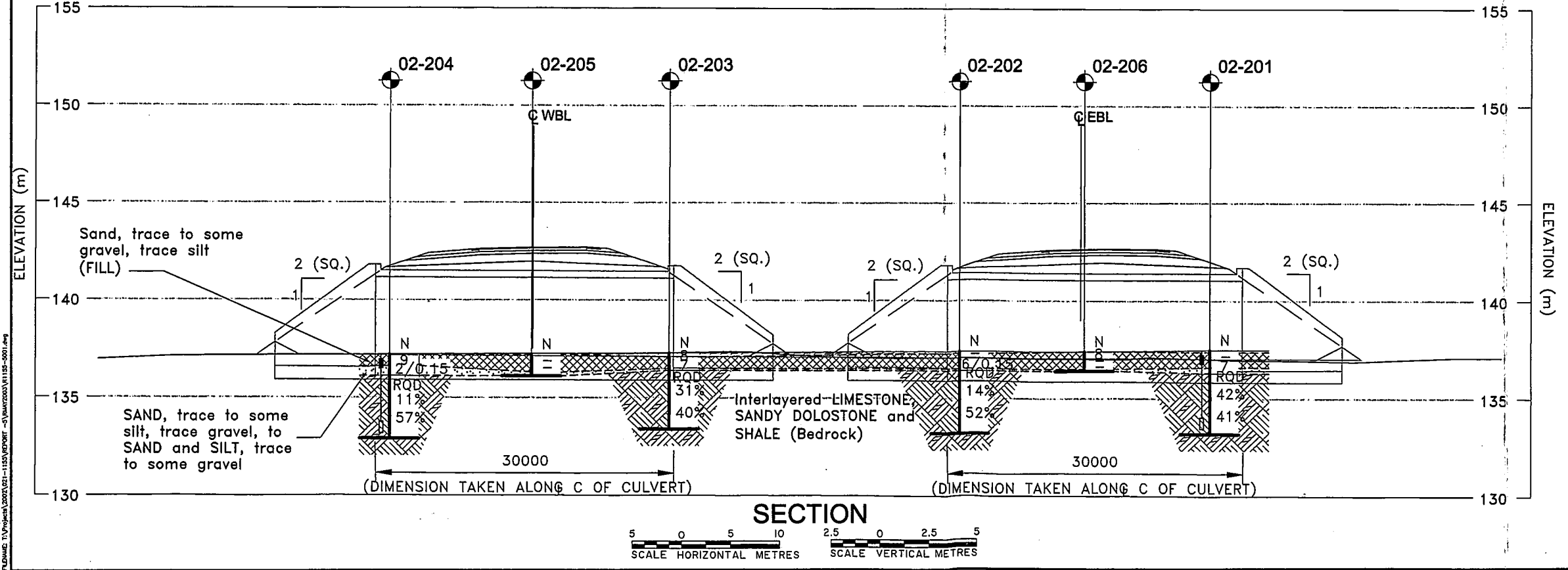
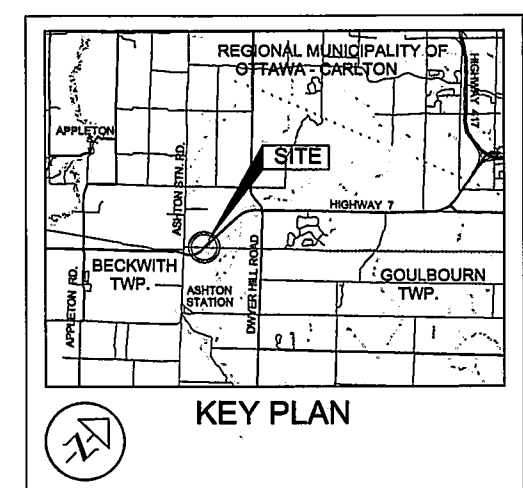
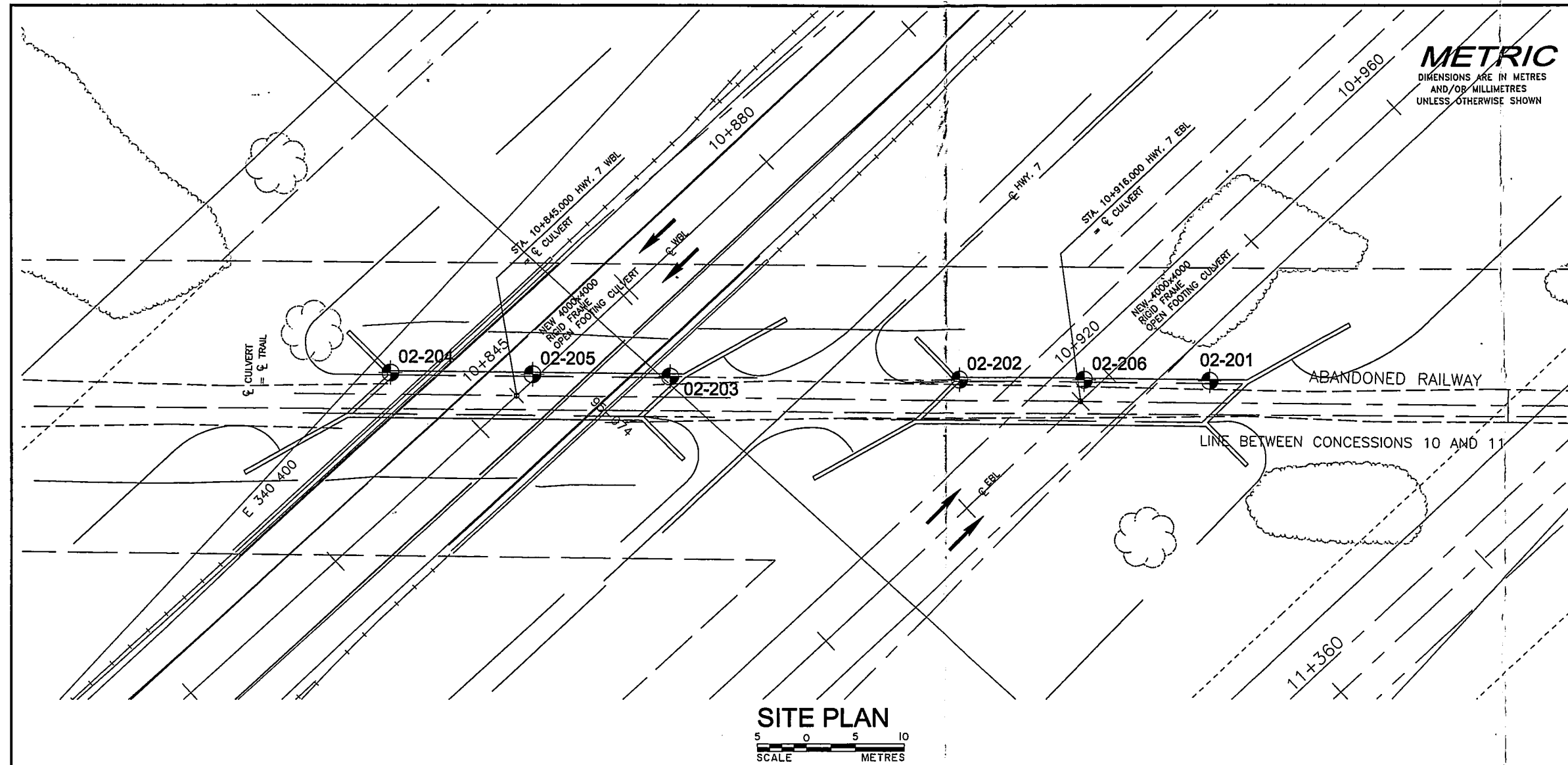
Note:
Water level in open borehole at 0.7 m depth (Elev. 136.7m) on completion of drilling.

MISS MTO 021-1155-5-GPJ ON MOT.GDT 22/7/03

DIST. HWY. 7
CONT No.
WP No. 251-99-00

TRANS-CANADA TRAIL
PEDESTRIAN CULVERTS
BOREHOLE LOCATIONS & SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND

Borehole
 Seal
 Piezometer
N Standard Penetration Test value
16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 l/blow)
100% Rock Quality Designation (RQD)
 WL in piezometer, April 15, 2003
 WL upon completion of drilling

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
02-201	137.5	5004437.6	340464.1
02-202	137.4	5004420.4	340445.2
02-203	137.3	5004400.6	340423.1
02-204	137.2	5004381.8	340402.0
02-205	137.2	5004391.3	340412.7
02-206	137.4	5004429.0	340454.6

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

REFERENCE
This drawing was prepared using the electronic "GENERAL ARRANGEMENT" file provided by MTO Eastern Region on Nov. 27, 2002.

NO.	DATE	BY	REVISION

Geocres No.
HWY. 7
SUBM'D. LCC
DRAWN: MHW

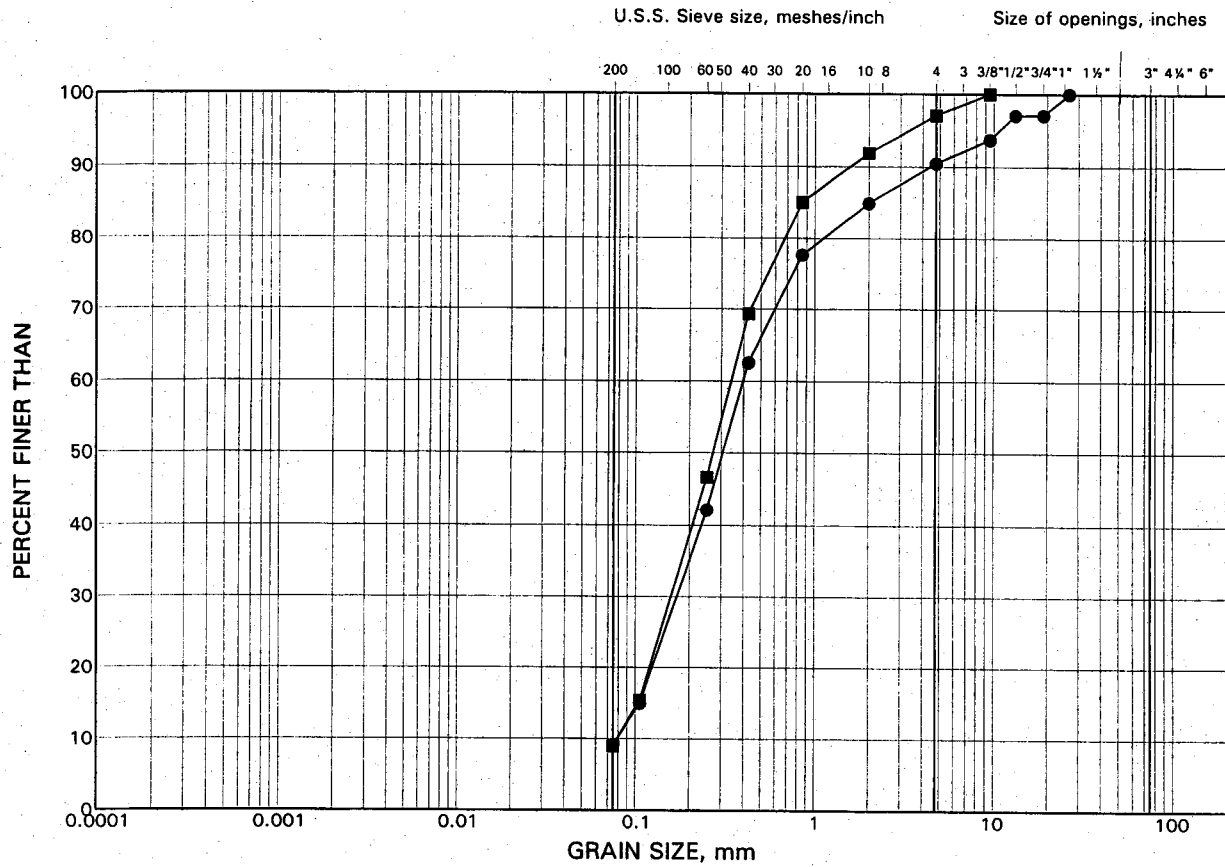
PROJECT NO. 021-1155-5
CHKD. LCC
DATE: APRIL 2003
APPD. ASP

DIST.
SITE:
DWG. 1

GRAIN SIZE DISTRIBUTION TEST RESULTS

Fill

FIGURE 1



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

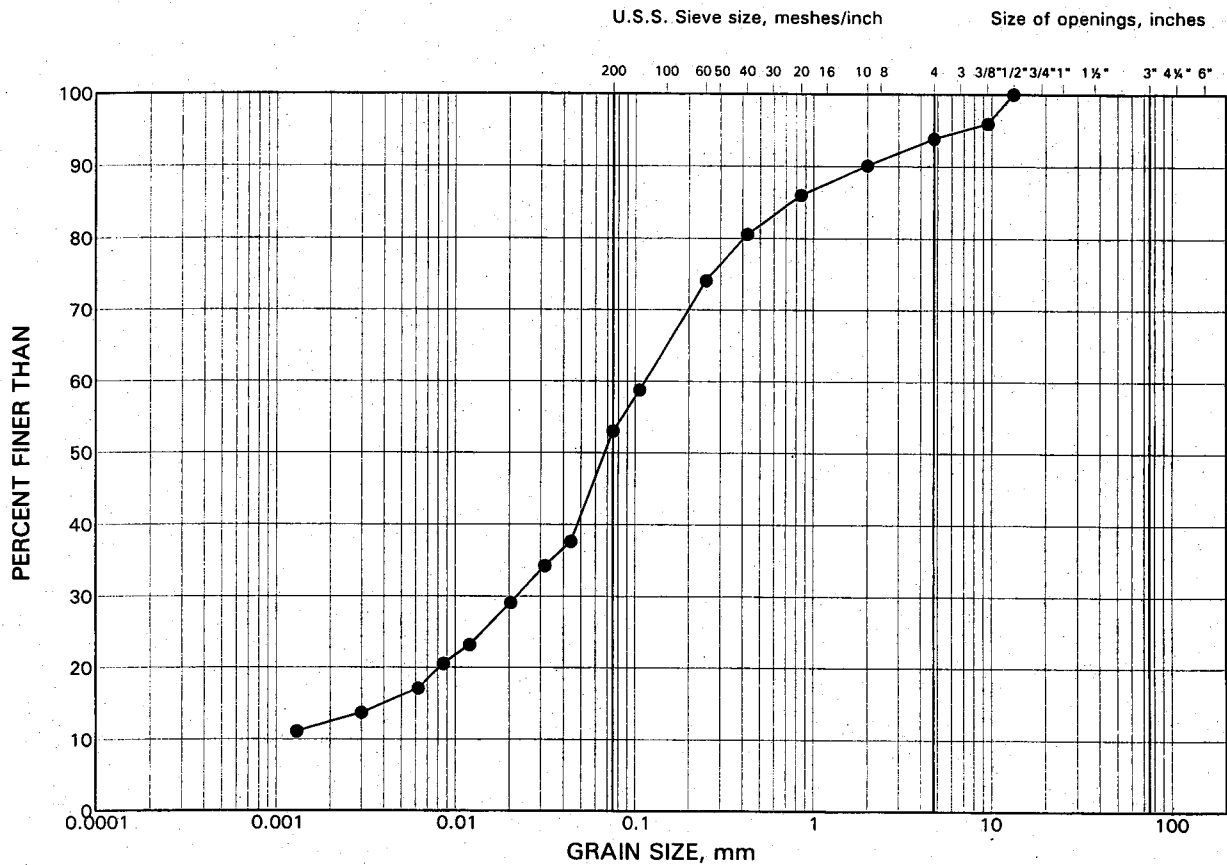
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	02-201	1	137.0
■	02-203	1	136.9

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand and Silt

FIGURE 2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	02-204	2B	136.3

