



October 2009

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN HIGHWAY 401 AT CORNWALL BRIDGE REPLACEMENT OVER CNR MILE 63.1 OF KINGSTON SUBDIVISION CORNWALL, ONTARIO GWP 237-00-00

Submitted to:

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REPORT



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PART A

**FOUNDATION INVESTIGATION REPORT
BRIDGE REPLACEMENT
OVER CNR MILE 63.1
HIGHWAY 401
CORNWALL, ONTARIO
GWP 237-00-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Genivar on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation associated with the replacement of two bridges along Highway 401 near Cornwall and Prescott, Ontario.

Foundation investigation services are required on this project for the following components:

- **GWP 237-00-00:** Replacement of the existing twin structures at the CNR overhead (EBL & WBL) crossing at Mile 63.1 of the Kingston Subdivision, near Cornwall, Ontario (Sites 31-216/1 & 31-216/2); and,
- **GWP 4600-02-00:** Replacement of the existing rigid frame structure at the abandoned CP Rail crossing near Prescott, Ontario (Site 16-129).

This report addresses the replacement of the existing twin structures near Cornwall, Ontario.

The terms of reference for the original scope of work are outlined in MTO's Request for Proposal (RFP) dated December 2007, and in MTO's Addendum #1 dated December 20, 2007. The work was carried out in accordance with Golder's Quality Control Plan for this project dated May 2008.

A separate foundation investigation has been carried out by Golder Associates for the roadway protection that will be needed for the new construction access ramps associated with the replacement of the existing twin structures. The results of that investigation are reported separately.



2.0 SITE DESCRIPTION

The existing Highway 401 twin structures included in this assignment (GWP 237-00-00) carry Highway 401 over the CN rail line (Mile 63.1 of the Kingston Subdivision) between Highway 401's Boundary Road interchange and Summerstown Road interchange near Cornwall, Ontario.

Through this area, Highway 401 is a four lane divided highway with a rural cross-section. The existing structures are aligned approximately east-west and cross the CNR tracks at a skew of approximately 45 degrees. The highway profile grade over the structures is about Elevation 65.0 m. The existing structures consist of two identical three-span steel girder bridges supported on concrete abutments and piers. The drawings for the construction of the existing structures indicate that the abutments of the bridges are supported on steel piles. The piers of the eastbound lanes (EBL) structure are indicated to also be supported on piles while those of the westbound lanes (WBL) structure are indicated to be supported on spread footings.

The CNR Kingston Subdivision crosses beneath the Highway 401 twin structures at an elevation of about 56 m. The railway has two tracks at this crossing, located about 4.2 metres apart. Provision for a third track on the east side is to be considered in the design of the replacement structures.

The structures were constructed in the late 1950's and early 1960's, with the eastbound structure apparently having been constructed first. The foundation investigation for the existing twin structures was carried out by E. M. Peto Associates Ltd. in 1958. The results of that investigation are provided in a report titled "Department of Highways of Ontario, Highway 401 – C.N.R. Crossing, W.P. 69-57 Township of Charlottenburg" (MTO's GEOCREs No. 31G-137). It appears in fact that the highway was originally to be aligned about 100 m further south, however potentially weak and compressible clay was encountered at that location. The alignment was subsequently shifted to the current location, which was considered to include a 'drumlin ridge'.

The existing approach embankments are about 10 to 11 m high relative to the surrounding ground surface and have approximately 2H:1V side slopes. No signs of embankment instability were observed.

The highway profile at the approaches does not seem to indicate that significant differential settlement of the roadway relative to the bridge has occurred, although the maintenance history at this location is not currently known.



3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the proposed bridge replacement locations between June 16 and September 10, 2008, at which time eighteen boreholes were advanced at the locations shown on Drawings 1 and 2. The boreholes locations were selected as follows:

Highway 401 Eastbound lanes:

- Two boreholes (numbered 08-1A and 08-4A) located near the existing abutments, advanced using a truck mounted drill rig, extending through the embankment fill and some compressible soil on the west side over glacial till and then cored over 3 m into bedrock; and,
- Two boreholes (numbered 08-6 and 08-5) located along the new abutment/retaining wall alignment, advanced using portable drilling equipment on the existing east and west embankment foreslopes, respectively, just outside of the bridge deck footprint, and extending through the embankment fill and any compressible soils and into the dense to very dense glacial till.

Highway 401 Westbound lanes:

- Two boreholes (numbered 08-10A and 08-11A) located near the existing abutments, advanced using a truck mounted drill rig, extending through the embankment fill and glacial till and then cored 3 m into bedrock; and,
- Two boreholes (numbered 08-13 and 08-14) located along the new abutment/retaining wall alignment, advanced using portable drilling equipment on the existing east and west embankment foreslopes, respectively, just outside of the bridge deck footprint, and extending through the embankment fill and into the dense to very dense glacial till.

Median area:

- Four boreholes (numbered 08-2A, 08-3A, 08-9A and 08-12A) in the median, located near the crest of the existing embankment foreslopes adjacent to the existing abutments, advanced using a track mounted drill rig, extending through the embankment fill and any compressible soils and the glacial till, then cored 3 m into bedrock;
- Two boreholes (numbered 08-19 and 08-20) on the existing west and east embankment foreslopes, respectively, located at the proposed abutment / retaining wall locations, advanced using portable drilling equipment, extending through the embankment fill and compressible soils and into the dense to very dense glacial till; and,
- Four boreholes (numbered 08-7, 08-8, 08-15 and 08-16) located in the median about 20 m behind the existing abutment locations, advanced using a track mounted drill rig, and extending through the embankment fill and any compressible soils and into the dense to very dense glacial till.



The boreholes were generally located within 5 m of the foundation location, except for Boreholes 08-6, 08-13, and 08-20, which are located up to about 8 m from the east abutment line; that abutment location was modified subsequent to the investigation being carried out.

The six boreholes located on the existing east and west embankment foreslopes were advanced using portable/manual drilling equipment supplied and operated by OGS Drilling Services of Appleton, Ontario. The boreholes were advanced to depths ranging from 4.3 to 8.8 m below the existing ground surface.

All other boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers on truck-mounted drill rigs (boreholes located on highway) and track-mounted drill rigs (boreholes located in median), supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths ranging from 11.3 to 23.6 m below the existing ground surface.

Soil samples were obtained continuously during the portable drilling, and at intervals of 0.75 m to 1.5 m of depth for all other drilling, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

Eight boreholes, as listed above, were advanced 3.0 m into the bedrock by coring using NQ-Size coring equipment.

In addition, an augerhole (numbered 09-1) was advanced about 6 m south of the proposed south-west RSS wall location, on September 16, 2009, at the location shown on Drawings 1 and 2. The purpose of this augerhole was to further investigate the presence of compressible clay soils within the footprint of the RSS wall. The augerhole was advanced by a member of our technical staff using portable/manual augering equipment, to a depth of about 5 m below the existing ground surface. Within the augerhole, the subsurface conditions and approximate depths to strata changes were visually logged at the time of augering and by examination of the auger cuttings. Grab samples were retrieved from the auger cuttings. In-situ vane testing was carried out where possible in the silty clay to evaluate the undrained shear strength of this soil unit. This vane testing was carried out using a small-size Geonor 'inspection vane'. The augerhole was terminated at 5 m depth, which was the practical limit for the equipment being used.

The water levels in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in boreholes 08-3A, 08-7, 08-9A, and 08-16 to monitor the groundwater levels at the site. The standpipes consist of 50 mm diameter rigid PVC pipe with a 0.7 m long slotted screen section, installed within silica sand backfill and sealed by sections of bentonite pellet backfill. The water levels in the standpipe piezometers were measured on July 18, 2008 and on August 20, 2009.

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

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes and augerhole, supervised the drilling, sampling and in-situ testing operations, logged the boreholes and augerhole, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to our Ottawa geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing, including grain size distribution, water content, and Atterberg limit testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.



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In addition, laboratory point load index and unconfined compressive strength testing was carried out on selected samples of the bedrock core, at Golder's Mississauga laboratory.

The borehole locations and ground surface elevations were determined by Golder personnel at the site using a Trimble R8 GPS unit. The augerhole location was determined using a 'hand held' GPS unit and the ground surface elevation was estimated with respect to the elevation of the nearby rail tracks. The borehole and augerhole 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 Drawings 1 and 2.



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Borehole/ Augerhole No.	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
09-1	South of South- West RSS wall	4992380.7	213525.5	54.1
08-1A	Eastbound lanes	4992408.2	213528.5	65.2
08-2A	Median area	4992419.9	213531.4	65.2
08-3A	Median area	4992439.3	213610.7	64.2
08-4A	Eastbound lanes	4992423.4	213595.0	64.6
08-5	Eastbound lanes	4992409.0	213544.3	58.6
08-6	Eastbound lanes	4992416.5	213580.3	60.0
08-7	Median area	4992417.0	213517.8	65.1
08-8	Median area	4992442.4	213625.6	64.0
08-9A	Median area	4992433.8	213542.8	64.6
08-10A	Westbound lanes	4992447.5	213553.9	64.9
08-11A	Westbound lanes	4992463.9	213624.5	64.1
08-12A	Median area	4992451.4	213620.0	63.7
08-13	Westbound lanes	4992464.5	213610.1	58.8
08-14	Westbound lanes	4992459.5	213579.0	57.7
08-15	Median area	4992430.4	213528.8	64.6
08-16	Median area	4992455.4	213636.4	63.6
08-19	Median area	4992432.3	213559.9	58.2
08-20	Median area	4992442.8	213600.7	60.9



4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

As delineated in *The Physiography of Southern Ontario*¹, the study area for this assignment lies within the physiographic region known as the Lancaster Flats.

The Lancaster Flats extend north from the shores of the St. Lawrence River and are characterized as a lowland in which water-laid overburden deposits of clay and fine sand overlie glacial till. Till drumlins and ridges locally protrude through the shallower soils. The area is notable for its poor drainage as a consequence of the relatively flat topography.¹

Three bedrock formations of the Middle Ordovician can be found in this region. Thus this site may be underlain by limestone bedrock of the Bobcaygeon Formation, interbedded silty dolostone, limestone, shale and quartz sandstone bedrock of the Gull River Formation, and/or interbedded sandstone, shaley limestone and shale bedrock of the Rockcliffe Formation.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, the existing GEOCREs information was supplemented by eighteen boreholes. The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes and augerhole 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 (Appendix A) and on Figures 1 to 15. Five relevant borehole records from the 1958 investigation are also provided in Appendix B.

The borehole locations and ground surface elevations from both the present investigation and MTO's 1958 subsurface investigation (GEOCREs No. 31G-137) are shown on Drawings 1 and 2. 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 summary, the subsurface conditions encountered in the boreholes and augerhole consist of up to about 11 m of embankment fill overlying a discontinuous layer of silty clay (predominantly over the west and south parts of the site), overlying a sandy silt to silty sand till deposit. The overburden soils are underlain by interbedded shale and fossiliferous limestone bedrock, which was encountered between about 18 to 20 m depth below roadway level (at about Elevation 44.5 m to 46.1 m, respectively). The silty clay deposit is, with the exception of one borehole and Augerhole 09-1, generally a stiff weathered deposit. The underlying glacial till would generally be described as a silty sand to sandy silt, with cobbles and boulders, as well as some gravel and clay. The till is of compact to very dense relative density. The groundwater level is within the bottom part of the fill or the upper part of the glacial till.

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.



A more detailed description of the subsurface conditions encountered in the boreholes and augerhole carried out at the site of the proposed twin structures is provided in the following sections, and stratigraphic profiles and sections of this site are shown on Drawings 1 and 2.

4.2.1 Peat / Topsoil / Pavement Structure / Fill

Approximately 0.2 m of peat (possibly fill) were encountered at ground surface at augerhole 09-1, which was located within the lower lying area southwest of the existing structures.

The peat at augerhole 09-1 is underlain by a deposit of organic silty clay, about 0.4 m in thickness. Cobbles and boulders were encountered in the organic silty clay, therefore this may be a fill material. The measured water contents of two samples of the organic silty clay were approximately 45 and 47 percent.

Approximately 0.1 m of topsoil (fill) were encountered at the ground surface at boreholes 08-2A, 08-3A, 08-7, 08-9A, 08-16, and 08-20, which were located in the grassed areas, either in the median or on the embankment foreslopes.

A thin layer of topsoil was also encountered buried beneath the embankment fill at boreholes 08-2A, 08-7, 08-9A, and 08-10A. That layer is approximately 0.1 to 0.3 m thick.

The pavement structure was penetrated by Boreholes 08-1A, 08-4A, 08-10A, and 08-11A and ranges from approximately 0.5 to 1.4 m in thickness. The pavement structure varies in composition, but is generally comprised of asphaltic concrete (i.e., asphalt), overlying Portland cement concrete (i.e., concrete), overlying crushed stone or crushed sand and gravel base material. The combined asphalt and concrete thickness ranges from 0.3 to 0.5 m. These boreholes were located on the right/outside shoulder of Highway 401 (two on the eastbound lanes and two on the westbound lanes).

The embankment fill was fully penetrated at all of the borehole locations and varied in thickness from 3.1 to 11.9 m. Those variable depths reflect, in part, the differing ground surface elevations at the borehole locations (e.g., boreholes at the roadway level versus boreholes on the foreslope). The native ground surface level beneath the embankment fill varies from about elevation 52.8 to 54.9 m on the west side of the tracks and from 54.6 to 59.1 m on the east side of the tracks.

The embankment fill material generally consists of sand and gravel fill overlying silty sand and sandy silt fill. Cobbles and boulders also exist within the fill, as do trace amounts of organic matter. Diamond drilling techniques were required to penetrate the cobbles and boulders in one of the boreholes advanced using portable/manual drilling equipment.

The results of grain size distribution testing carried out on samples of the embankment fill are provided on Figures 1 to 7. The results on those figures have been sorted/reported according to the fill material type in accordance with the descriptions on the Record of Borehole sheets, rather than according to the specific gradation of each sample, recognizing that there are natural variations in the material from the generalized descriptions on the borehole records. The results also do not reflect the cobble, boulder, or full gravel contents of the material, since the samples were retrieved using a 50 mm diameter sampler.



Layers of silty clay or clayey silt embankment fill were encountered at boreholes 08-6, 08-9A, 08-14, 08-15 and 08-20. In particular, Boreholes 08-6 and 08-20, put down through the east embankment foreslope, encountered a thin but distinct layer of silty clay fill with organic matter. The results of Atterberg limit testing carried out on one sample of the silty clay and clayey silt embankment fill in borehole 08-14 gave a liquid limit of about 28 percent and a plasticity index of about 10 percent, indicating a clayey silt of low plasticity.

Standard penetration test 'N' values for the embankment fill ranging from 2 to greater than 100 blows per 0.3 m of penetration indicate it to be very loose to very dense, although the higher 'N' values could reflect the presence of cobbles and boulders, rather than the state of packing of the soil matrix. The N values more typically range from about 15 to 50 blows per 0.3 m.

The measured water content of the fill ranges from approximately 8 to 16 percent, except for one sample of the clayey fill from Borehole 08-14 that had a water content of about 31 percent (same sample on which the Atterberg limit testing was carried out, as discussed above).

4.2.2 Silty Clay to Clay

The embankment fill (and buried native topsoil, where present) at boreholes 08-1A, 08-2A, 08-4A, 08-5, 08-6, 08-7, 08-9A, 08-10A, 08-15, and 08-19, and the organic silty clay at augerhole 09-1 are underlain by a deposit of sensitive silty clay. These boreholes and the augerhole are all located on the south and west parts of the site.

The silty clay was fully penetrated by the all of these boreholes and, on the west side of the tracks, varies from about 0.9 to 3.1 m in thickness, though the deposit is more typically quite uniformly about 2 m thick. On the east side of the tracks, where the clay exists only at boreholes 08-4A and 08-6, the silty clay is only 0.5 and 0.3 m thick, respectively. The silty clay at the augerhole location was not fully penetrated, but proven to a depth of about 5.0 m below ground surface.

The upper portion of the silty clay at borehole 08-1A and augerhole 09-1, and the full thickness of silty clay at all of the other borehole locations, have been weathered to a grey brown colour. Standard penetration tests carried out within the weathered silty clay gave 'N' values ranging from 6 to 31 blows per 0.3 m of penetration, indicating a generally very stiff consistency.

The results of Atterberg limit testing carried out on samples of the weathered silty clay are summarized on Figure 8 and indicate plasticity index values generally ranging from 19 to 37 percent and liquid limit values ranging from 40 to 67 percent, reflecting intermediate to high plasticity (i.e., silty clay to clay). The measured water content of the weathered silty clay ranges from approximately 30 to 53 percent. In a few cases, the measured water contents are at or above the liquid limit.

A layer of un-weathered (i.e., grey in colour) silty clay was encountered at borehole 08-1A and augerhole 09-1, below the upper weathered silty clay. This unweathered silty clay is only about 0.4 metres thick at the borehole location and contains silt and sand layers. The unweathered silty clay was not fully penetrated by the augerhole but was proven for a thickness of about 3.0 m. The results of Atterberg limit testing carried out on two samples of this material are also shown on Figure 8 and gave plasticity index values of 22, and 44 percent and liquid limit values of 41 and 76 percent, indicating an intermediate to high plasticity soil (i.e., silty clay to clay). The measured water contents of the two samples of grey silty clay were approximately 43 and 74 percent, which are in excess of, and at, the measured liquid limit, respectively.



4.2.3 Sandy Silt to Silty Sand Till

At all of the boreholes, the fill materials and/or silty clay are underlain by glacial till. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand and sandy silt with a trace of clay.

The surface of the glacial till varies from about elevation 51.0 to 52.8 m on the west side of the tracks and from about elevation 54.6 to 59.1 m on the east side of the tracks. The glacial till was fully penetrated at eight of the boreholes and varied in thickness from 5.8 to 13.0 metres, extending down to elevations varying from 44.5 to 46.1 m.

Grain size distribution testing was carried out on 23 samples of the till, the results of which are provided on Figures 9 to 13. The results confirm that the till matrix consists of a silty sand and sandy silt with variable amounts of gravel and typically trace amounts of clay. These samples were however retrieved using a 50 mm diameter sampler and therefore the results do not reflect the cobble and boulder content of the deposit.

Standard penetration test 'N' values for this material ranging from 8 to greater than 100 blows per 0.3 m of penetration indicate a loose to very dense state of packing, although the higher 'N' values could reflect the presence of cobbles and boulders, rather than the state of packing of the soil matrix. The N values more typically range from about 20 to 50, indicating a compact to dense state of packing. Refusal to advancement of the sampler was frequently encountered, apparently on cobbles and boulders in the deposit, and in a few instances rotary diamond drilling/coring techniques were required to advance the boreholes within the till.

A zone of layered sandy silt and clayey silt (about 0.8 m thick) was encountered within the till in Borehole 08-9A while a thin sand layer (0.2 m thick) was encountered within the till deposit at Borehole 08-4A. At Borehole 08-11A, a 3.2 m thick layer of silt was encountered about 5 m below the till surface, and a further 0.6 m layer of silt was encountered at the bottom of the till deposit. The results of grain size distribution testing on samples of these strata from Boreholes 08-9A and 08-11A are provided on Figure 14.

The measured water content of the till ranges from approximately 6 to 23 percent, although more generally from about 8 to 13 percent. The water contents of two samples of the silt layers in Borehole 08-11A were 11 and 15 percent.

4.2.4 Refusal and Bedrock

For boreholes 3, 6, 7, and 8 of the 1958 MTO investigation, refusal to casing advancement was encountered at elevations ranging from approximately 45.1 to 45.5 m. Refusal may indicate the bedrock surface; however, it could also represent cobbles and/or boulders within the glacial till.

Bedrock was encountered beneath the glacial till, and cored for about 3 m depth, in boreholes 08-1A, 08-2A, 08-3A, 08-4A, 08-9A, 08-10A, 08-11A, and 08-12A; all of these boreholes are located near the abutments of the existing bridges. Bedrock had also been cored in Borehole 1 from the 1958 MTO investigation.

The following table summarizes the bedrock surface depths and elevations as encountered at the nine borehole locations where bedrock was cored.



Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
08-1A	65.2	20.0	45.2
08-2A	65.2	19.9	45.3
08-3A	64.2	18.1	46.1
08-4A	64.6	19.7	44.9
08-9A	64.6	20.1	44.5
08-10A	64.9	19.5	45.4
08-11A	64.1	18.9	45.2
08-12A	63.7	18.8	44.9
1	57.8*	12.2	45.6

Note: Existing ground surface elevation and depth to bedrock were established at the time the borehole was drilled in 1958.

The bedrock encountered in the boreholes consists of grey interbedded shale and fossiliferous limestone. The bedrock is fresh, medium strong and thinly to medium bedded.

The Rock Quality Designation (RQD) values measured on recovered bedrock core samples were quite variable and ranged from about 0 to 100 percent, indicating a poor to excellent quality rock. However the RQD values were generally found to increase with depth. The discontinuities observed in the rock core are typically horizontal, associated with the bedding planes.

Laboratory point load index testing was carried out, axially, on eight selected specimens from the bedrock core. Laboratory unconfined compressive strength testing was also carried out on two selected specimens of the bedrock core. The results are summarized on Figure 15 and indicate compressive strengths from the point load index testing which range widely from 23 to 127 MPa. The two unconfined compressive strength tests indicate values of about 25 and 49 MPa.

4.3 Groundwater Conditions

The groundwater levels in the piezometers in Boreholes 08-3A, 08-7, 08-16, and 08-9A were measured on July 18, 2008 and on August 20, 2009. The observed groundwater levels are summarized in the table below:

Borehole Number	Existing Ground Surface Elevation (m)	July 18, 2008		August 20, 2009	
		Water Level Depth (m)	Water Level Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)
08-3A	64.2	7.0	57.2	7.5	56.7
08-7	65.1	10.2	54.9	10.4	54.7
08-9A	64.6	10.0	54.6	10.2	54.4
08-16	63.6	6.7	56.9	5.2	58.4

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 S. Lesage, EIT, under the direction of the Project Manager, Mr. Michael I. Cunningham, P.Eng. Mr. Fintan J. Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

Yours truly,

GOLDER ASSOCIATES LTD.

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PART B

**FOUNDATION DESIGN REPORT
BRIDGE REPLACEMENT
OVER CNR TRACKS
HIGHWAY 401
CORNWALL, ONTARIO
GWP 237-00-00**



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the new twin structure and proposed temporary modular bridge that will carry Highway 401 over the CNR Kingston Subdivision near Cornwall, Ontario. 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 to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. 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 current plans for this project are understood to be as follows:

- The existing bridges are to be replaced. A temporary modular bridge is to be constructed within the median to provide a traffic detour during each bridge replacement.
- The new bridges and the modular bridge are to be single span structures. Integral abutment structures have been proposed for the new bridges, but other options are also being considered.
- Retaining walls are to be constructed across the existing embankment foreslopes, to allow for new abutment locations that are closer to the rail line and to thereby allow for shorter bridge spans than the existing structures. The proposed configuration will result in the west retaining wall (and abutment) being located a short distance behind the existing west piers, near the toe of the existing west foreslope. However the east retaining wall (and abutment) is to be constructed along an alignment near the mid-height of the east foreslope; that different position is to allow for a possible future third rail track on the east side of the existing two tracks. For both walls, it is understood that the grade in front of the walls would approximately correspond to the existing track level, and would therefore be at about elevation 56 m. The walls would therefore be up to about 8 m high.

Several arrangements have been proposed for the retaining walls, including:

- Retained Soil Systems (RSS), with the bridges being supported on deep foundations installed behind those walls; and,
- Cast-in-place concrete retaining walls, with the bridges supported directly on those walls; i.e., those walls would form the new bridge abutments.

Retaining walls (i.e., wing walls) are also proposed which would extend approximately 30 m from the ends of the abutments, parallel to the railway, on the north side of the WBL structure and the south side of the EBL structure.



The existing structures consist of two identical three-span steel girder bridges supported on concrete abutments and piers. The drawings for the construction of the existing structures indicate that the abutments of the bridges are supported on steel piles. The piers of the eastbound lanes (EBL) structure are indicated to also be supported on piles while those of the westbound lanes (WBL) structure are indicated to be supported on spread footings.

The existing embankments are about 10 to 11 m high relative to the surrounding natural ground level. It is understood that those embankment heights are to be maintained.

Foundation engineering recommendations for the bridge and retaining wall foundations are provided in Section 6.3.

Foundation engineering recommendations for the temporary modular bridge foundations are provided in Section 6.4.

Foundation engineering recommendations for the RSS walls are provided in Section 6.8.

6.2 Feasibility of Integral Abutments

The site is considered generally feasible for integral abutment design, from a foundation engineering perspective. In accordance with the MTO's document *Integral Abutment Bridges* (report SO-96-01), the primary criteria relating to the ground conditions is the need/ability to support the structure on relatively flexible piles, so as to allow for movement of the structure. As a general guideline, this criteria requires that piled foundations be feasible and that the piles be at least 5 m long. That criteria is satisfied for this site.

However, as discussed in Section 4.2.3, the till is generally compact to dense, which may unacceptably restrict the lateral movement of the piles. The guidelines provided in Section 6.3.1.2 of this report for the resistance of the piles to lateral loading could also be used to evaluate the flexibility of the pile system and the feasibility of having integral abutments. If the foundation system for these ground conditions would not be sufficiently flexible, then consideration could be given to pre-augering holes at the pile locations and then backfilling the holes with loose sand.

Similarly, if the foundations were to be 'perched' behind the new abutment/retaining walls, then the granular backfill to those walls may also provide too much resistance to lateral movement of the piles for integral abutments to be feasible. To provide a foundation system that is more flexible with respect to lateral movement, steel liners could be installed within the backfill (minimum 0.6 m in diameter), and those liners backfilled with loose sand following installation of the piles. Further details for this option are provided in a sample Non-Standard Special Provision (NSSP) included in Appendix C of this report (CSP for Integral Abutments).

It is noted however that the abutment height and skew of this bridge may exceed the 6 m and 35 degree maximum permissible values for integral abutment design specified in MTO's document *Integral Abutment Bridges* (report SO-96-01), and therefore fully integral abutments may not be feasible from that perspective.

The use of semi-integral abutments would however be feasible, from a foundation engineering perspective, based on the guidance provided in MTO's document *Semi-Integral Abutment Bridges* (report BO-99-03).



6.3 Bridge and Retaining Wall Foundation Options

The following options have been considered for the foundations of the new bridges and retaining walls:

- Deep foundations (driven steel H-piles or cast-in-place concrete caissons) which derive their support from end-bearing on the bedrock surface at depth, or possibly within the lower portion of the glacial till; or,
- Shallow foundations (i.e., spread footings) bearing on or within the glacial till.

If the proposed wing walls are to be of cast-in-place reinforced concrete construction, then the same options and design recommendations are applicable to these foundations as well.

Geotechnical recommendations for the design of the foundations for the bridge abutments and retaining walls are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Table 1 following the text of this report. Support of the bridge foundations on steel H-piles is considered to be the preferred option, from a foundation engineering perspective.

Guidelines for the design of possible RSS walls, for either the wing walls and for the new retaining walls parallel the tracks, are provided separately in Section 6.8 of this report.

6.3.1 Steel H-Pile Foundations

Steel H-piles driven to found on the limestone bedrock may be used for support of the bridge abutments or cast-in-place concrete retaining walls.

The pile caps should be provided with a minimum of 1.7 m of soil cover for frost protection. Since the grade in front of the new abutment/walls will be at about track level (approximately elevation 56 m) and the bedrock surface is at about Elevation 45 to 46 m, the pile length will be about 9 to 10 m. Alternatively, if the bridges were to be supported on piles driven behind the retaining walls, then the piles would be about 17 to 18 m long. The following table summarizes the anticipated pile toe elevations and founding stratum for each foundation.

Location	Anticipated Toe Elevation (m)	Founding Stratum
WBL Structure West Abutment	45.1	Bedrock
WBL Structure East Abutment	45.6	Bedrock
EBL Structure West Abutment	45.4	Bedrock
EBL Structure East Abutment	45.5	Bedrock

The glacial till that overlies the bedrock at this site consists of gravel, cobbles, and boulders in a matrix of sandy silt and silty sand. That till deposit is considered to be quite bouldery. The piles should therefore be provided with Titus-type bearing points or equivalent to protect the pile tips during driving through the bouldery overburden.



The piles should be designed to be founded on bedrock. However some of the piles could have difficulty penetrating to depth and could “hang up” at shallower depth in the glacial till. In that case pre-drilling of the overburden could be considered. Alternatively a reduced capacity may apply to these piles, as discussed below. A preliminary assessment of the proportion of the piles that might encounter this condition can be made based on the drilling of the boreholes advanced to date at this site (including the aforementioned separate investigation for the shoring design for the construction access ramps). Approximately one third of the boreholes have needed to be advanced past boulders using rotary diamond drilling/coring techniques. This one third factor may therefore represent the proportion of piles that may hang up in the overburden.

6.3.1.1 Axial Geotechnical Resistance

The following factored axial resistances at Ultimate Limit States (ULS) may be assumed for design of piles that are successfully driven to found on the bedrock:

Pile Size	Factored ULS Resistance (kN)
HP 310 x 110	2,000
HP 360 x 132	2,400
HP 360 x 152	2,750

The above values represent structural limitations for the piles rather than geotechnical limitations.

SLS resistances do not apply to piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. ULS conditions will govern for this foundation type, providing the piles are successfully driven to bedrock.

As discussed previously, it is expected that some of the piles may not fully penetrate the bouldery glacial till to reach the bedrock surface; these piles could “hang up” at shallower depth in the glacial till. In that case, predrilling of the glacial till could be considered. An NSSP should be included in the contract documents to address this issue and a sample version is included in Appendix C (Boulders/Cobbles During Pile Installation).

Alternatively, the piles may need to be designed for a reduced capacity. The ULS factored axial resistance of these piles will depend on the depth to which they penetrate and the set that is achieved. As a preliminary guideline, for HP 310 x 110 piles founded within the glacial till, a ULS factored geotechnical resistance of 1,800 kN may be used. The axial resistance at SLS for 25 millimetres of settlement would likely be in the order of 1,600 kN. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The set criteria should be established using the Hiley formula, using a resistance factor of 0.5 on the factored axial resistance. For this situation, the piles should be driven in accordance with Standard SS 103-11, using an ultimate capacity of 3,600 kN per pile.

Should this situation occur, of a reduced capacity needing to be used, additional piles could be driven to achieve the required combined overall foundation capacity. Consideration could also be given to using this lower capacity for general design purposes, and thereby limit the potential need for additional piles should refusal in the glacial till occur.



Pile installation should be in accordance with SP903S01. The drawings should incorporate the appropriate note stating that the piles should be equipped with bearing points and should be driven to bedrock (where possible). For piles driven to refusal on bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

It is understood that portions of the existing structures are in poor condition. Vibration monitoring should therefore be carried out during pile installation to ensure that the vibration levels at the existing structure(s) are maintained below tolerable levels.

A maximum peak particle velocity of 100 mm/s is recommended at existing abutments and bridge retaining walls. The piles further from the existing structures should be driven first, in order to check the vibration level at the existing structures and, if necessary, alter the pile driving criteria for the remaining piles. A Non Standard Special Provision for vibration monitoring should be included in the contract documents and a sample has been included in Appendix C of this report (Vibration Monitoring).

The existing piles may be difficult to remove and will probably need to be left in-place. The potential for interference between the new and existing piles should be considered and avoided. With the current foundation design, interference is considered to be a concern only at the west abutment of the eastbound lane structure. It could be necessary to adjust the locations of some of the new piles once the existing pile cap has been removed and the locations of the existing piles can be determined.

For the west abutment of the EBL structure, where silty clay is present, the extension (forward) of the embankments up to the new abutment and retaining wall alignments will raise the effective stress level in the silty clay and could generate downdrag forces on the piles. Some limited compression of the deposit is expected (see later discussion in Section 6.7.3), and is estimated at about 25 mm. The elastic shortening of the piles could be less than that value and therefore the differential settlements would be sufficient to generate downdrag forces.

Downdrag forces are not considered to be an issue for the other foundation areas (WBL structure or east abutment of EBL structure) since the clay is either thinner or absent from these areas.

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual as well as the US Transportation Research Board's report "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the silty clay deposit.

The unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the silty clay and overlying embankment fill is estimated to be about 120 kN, for piles supporting the new retaining walls and abutments. Note: If abutments 'perched' behind the new retaining walls were to be considered, a revised downdrag load would need to be provided, based on the longer length of pile, or the piles would need to be sleeved through the embankment fill (such as would be required for integral abutment design).

The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the CHBDC.



6.3.1.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. Alternatively, the resistance to lateral loading will have to be derived from the soil in front of the piles, and it may be assumed that this resistance will be nearly the same for vertical and inclined piles as indicated in Section C6.8.7.2 of the Commentary to the CHBDC.

The SLS geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the equations given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction, as given below;
 z is the depth (m); and,
 B is the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: k_h is the coefficient of horizontal subgrade reaction;
 s_u is the undrained shear strength of the soil (kPa); and,
 B is the pile diameter/width (m).

The following values of n_h and s_u may be assumed in the structural analysis.

Location	Elevation (m)	Soil Type	γ (kN/m ³)	Friction Angle (degrees)	n_h (MN/m ³)	s_u (kPa)
WBL Structure West Abutment	PCL ¹ – 58.2	New Compacted Fill	21.5	35	6.5	-
	53.6 – 58.2	Existing Embankment Fill	20	30	3	-
	52.7 – 53.6	Silty Clay	17.5	0	-	100
	Below 52.7	Glacial Till	21	35	11	-
WBL Structure East Abutment	PCL – 58.8	New Compacted Fill	21.5	35	6.5	-
	55.8 – 58.8	Existing Embankment Fill	20	30	3	-
	Below 55.8	Glacial Till	21	35	11	-
EBL Structure West Abutment	PCL – 58.6	New Compacted Fill	21.5	35	6.5	-
	53.7 – 58.6	Existing Embankment Fill	20	30	3	-
	51.4 – 53.7	Silty Clay	17.5	0	-	100
	Below 51.4	Glacial Till ²	21	35	11	-
EBL Structure East Abutment	PCL – 60.0	New Compacted Fill	21.5	35	6.5	-
	56.0 – 60.0	Existing Embankment Fill	20	30	3	-
	55.7 – 56.0	Silty Clay	17.5	0	-	100
	Below 55.7	Glacial Till	21	35	11	-

Note 1: PCL = Pile cap level.

Note 2: Clay thickens to the south. By the end of the wing wall, Augerhole 09-1 shows till surface lower than elevation 49.1 m.



Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

For establishing the ULS factored *structural* resistance, the shear force and bending moment distribution in the piles under factored loading can be established using the procedures and parameters given above for evaluating the SLS response of the pile.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.8.7 of the *Commentary to the CHBDC*.

For individual piles in cohesive soils (i.e., silty clay and clay) the ULS lateral resistance is assumed to vary linearly with a magnitude of $2S_u$ at the surface of the deposit and to a magnitude of $9S_u$ at a depth equal to three pile diameters below the underside of the pile cap (where S_u is the undrained shear strength of 100 kPa). Below a depth equal to 3 pile diameters, and to the bottom of the deposit, the lateral resistance is assumed to be constant at $9S_u$.

The ULS lateral passive resistance from the glacial till (or compacted engineered fill) may be assumed to act over the pile shaft to a depth equal to six pile diameters below the underside of the pile cap (except where the silty clay thickness exceeds that depth) and the resistance per unit length of pile may be calculated as:

Above the water table:

$$P_p(z) = 3 d K_p \gamma z$$

Below the water table:

$$P_p(z) = 3dK_p \gamma D_w + 3dK_p (z - D_w) (\gamma - \gamma_w)$$

- Where:
- $P_p(z)$ is the ULS lateral resistance at depth 'z' below ground surface (kN/m);
 - γ is average unit weight of overlying soil, use 20 kN/m^3 ;
 - K_p is the coefficient of passive earth pressure, use 3.7;
 - D_w is the depth to groundwater table below ground surface(m), assume is at track level;
 - γ_w is the unit weight of water, use 9.8 kN/m^3 ; and,
 - d is the pile diameter (m).



The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the face of the pile group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; in accordance with the *CHBDC*, a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.

For *preliminary* design purposes, the ULS *geotechnical* resistance can also be estimated using the “Assessed Horizontal Passive Resistance Values for Various Pile Types” provided in the *Commentary* to the *CHBDC*. On that basis, a maximum lateral resistance of 110 kN at ULS (unfactored), and a maximum lateral resistance of 40 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310 x 110 piles.

6.3.1.3 Frost Protection

The pile caps should be provided with a minimum of 1.7 m of soil cover for frost protection.

6.3.2 Caisson Foundations

Caissons founded on or socketed into the bedrock may be used for support of the bridge abutments or cast-in-place concrete retaining walls.

The use of a liner or casing will be required in order to advance the caissons with minimal loss of ground. Additionally, it will be difficult to clean the bedrock surface, even with the use of liners, unless the liner is socketed into the bedrock; the sandy till could flow under the casings, at the interface with the bedrock. It may therefore be more practical to socket the caissons into the rock, rather than found on the bedrock surface.

The bedrock at the site is moderately strong. If socketing of the caissons into the bedrock is required, the sockets will have to be advanced by rock coring or churn drilling.

Casing installation through the bouldery glacial till may also be difficult. Churn drilling techniques could be required.

6.3.2.1 Axial Geotechnical Resistance

Caissons founded on the surface of the shale and limestone bedrock, or socketed nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 3 MPa should be used. This ULS resistance considers the poor rock quality (RQD values) recorded for the upper portion of the bedrock as well as the shale interbeds within the otherwise fairly strong limestone.

SLS resistances do not apply to caissons founded on or socketed in the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Extension of the approach embankments, up to the new abutment and retaining wall alignment, will raise the effective stress level in the silty clay (west foundations of EBL structure only), leading to some consolidation of the deposit. This condition will result in downdrag forces on caissons, as discussed previously in Section 6.3.1.1. The unfactored downdrag load acting on a single 0.9 m or 1.5 m diameter caisson over its length is estimated to be 280 and 470 kN, respectively (based on an underside of pile cap level at about elevation 54.5 m). The structural capacity of the caissons must be checked for the factored dead and downdrag loads in



accordance with Section 6.8.4 of the *CHBDC*. The assumptions and methods used in assessing that downdrag force are the same as those described in Section 6.3.1.1 of this report with respect to steel H-piles.

6.3.2.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.3.1.2.

6.3.2.3 Frost Protection

The pile (caisson) caps should be provided with a minimum of 1.7 m of soil cover for frost protection.

6.3.3 Shallow Foundations

Shallow foundations on the glacial till may be considered for the support of the new abutments and the retaining walls.

The existing embankment fills and the clay deposit (where present) are not considered suitable for the support of heavy foundations loads and therefore any shallow foundations would need to be supported below these materials. Shallow foundations would therefore only be an option for elements founded at the lower level (i.e., at track level) and not for foundations perched above/within the new fill (i.e., those founded above and behind the new retaining walls).

As discussed subsequently in Section 6.3.3.3, shallow foundations will need to be provided with at least 1.7 m of earth cover for frost protection purposes. For the foundations on the west side of the tracks, the surface of the glacial till is in the range of about elevation 51.4 m to 52.7 m, which is below the founding level that would be dictated by the required 1.7 m of earth cover needed for frost protection purposes. Existing fill materials and/or silty clay would therefore need to be excavated and the structure founded at greater depth, on the surface of the glacial till. For foundations on the east side of the tracks, where the silty clay is essentially absent, the glacial till is present at the founding level dictated by the earth cover requirements.

The following founding levels are therefore envisaged:

Foundation	Available Founding Elevation (m)	Founding Strata	Remarks
WBL Structure West Abutment	52.8 – 52.7	Glacial Till	Need to subexcavate fill materials and silty clay to reach glacial till.
WBL Structure East Abutment	55.8 – 57.2	Glacial Till	Founding level to be lowered as required to provide 1.7m of earth cover.
EBL Structure West Abutment	51.4 - 52.7	Glacial Till	Need to subexcavate fill materials and silty clay to reach glacial till. Note: Clay thickens to the south. Beyond the end of the wing wall, till surface is below elevation 49.1 metres.
EBL Structure East Abutment	55.7 – 57.2	Glacial Till	Founding level to be lowered as required to provide 1.7m of earth cover.



For the west foundations, the deeper excavations needed to reach the surface of the glacial till would require track protection (i.e., excavation shoring), as discussed subsequently in Section 6.9.3 of this report. The till surface is particularly deep at the south end of the south-west wing wall (versus beneath the abutment wall), and therefore shallow foundations for this wall would not be preferred (versus using piled foundations or an RSS wall, for which subexcavation would not be needed).

It must be confirmed during construction that the soils at the base of the excavation are consistent with those anticipated. 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.

Variations in the thickness of the silty clay deposit that overlies the glacial till should be anticipated and provision should be made in the contract for extending the footing excavation deeper as may be needed to reach the till founding stratum. Provision should be made for either deepening the footings and/or foundations or for subexcavation and replacement with mass concrete, should a deeper glacial till level be encountered.

Should it be chosen to support the wing walls on shallow foundations but the bridge abutments on deep foundations, then larger settlements would occur for the retaining wall footings versus the bridge abutments, and those settlements would be entirely differential. Therefore, in that case, the walls should be provided with an articulated joint with the bridge abutments.

6.3.3.1 Limits States Factored Geotechnical Resistance and Reaction

Spread footings placed on undisturbed till may be designed based on a factored geotechnical resistance at ULS of 500 kPa. The geotechnical resistance at SLS may be taken as 300 kPa (based on 25 mm of settlement).

These are preliminary values and will need to be confirmed based on the actual footing size, geometry, and founding level. 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 of the load should be taken into account in accordance with the Canadian Highway Bridge Design Code (CHBDC).

6.3.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and subsoils should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.50 for cast-in-place concrete footings constructed on undisturbed till. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3.3.3 Frost Protection

The footings should be provided with a minimum of 1.7 m of soil cover for frost protection.



6.4 Foundations of Temporary Modular Bridge

The foundations of the temporary modular bridge will be located in the median area, between the existing EBL and WBL structures.

It is understood that the foundations will, at least in part, be supported on the retaining walls which will form the new west and east abutments of the new WBL and EBL bridges; the retaining walls will be continuous from the north side of the WBL bridge to the south side of the EBL bridge, through the median area. Where the modular bridge can be supported on those walls, the recommendations provided in Section 6.3 (for shallow and deep foundation options for the bridges and retaining walls) are equally applicable to those portions of the walls which support the temporary modular bridge.

Other portions of the modular bridge foundations will need to be supported separately. It is understood that separate foundations are required because the temporary modular bridge cannot be supported on skewed foundations, whereas the abutment/retaining wall is to be aligned along an approximately 45 degree skew to the highway centreline. The locations of those separate foundations are controlled by the width of the modular bridge and the skew of the abutment/retaining wall. It is understood that these foundations, which will consist of single supports (i.e., essentially isolated piers) will be located about 7 metres behind the new abutment/retaining walls. The support for each modular bridge abutment will therefore consist of individual support points for each side of the abutment - one support on the abutment/retaining wall and one on a separate support. The separate support points will be located above the existing foreslopes and about 5 m in front of the existing abutments at the northwest and southeast ends of the modular bridge.

It is considered that, for the current proposed design, the support for the modular bridge foundations should consist entirely of piled foundations. That recommendation is based on the following:

- 1) One support point for each foundation will bear on the new abutment/retaining walls, which are also proposed to be supported on piles, and this arrangement would therefore provide a consistent founding condition (thereby avoid possible twisting of the structure due to differential settlements of the foundations).
- 2) The modular bridge bearing level will be several metres above the existing foreslope level, and it appears that the embankment fills in the foreslopes consist of a loose to compact mixture of silty sand, sandy silt, and silty clay, which range from a generally loose to compact state. Organic matter also exists within the foreslope fill. The fill materials in the embankment foreslopes do not therefore appear to have been placed in a controlled manner, as compacted engineered fill. These are not therefore convenient or ideal conditions for supporting the modular bridge on shallow foundations on the foreslopes. Subexcavation of the embankment fill material would be required, which would likely require shoring of the adjacent embankment to avoid undermining the adjacent bridges, embankments, and the roadway.

Piled foundations are therefore preferred. Further guidelines on the design of piled foundations for this bridge are provided in Section 6.4.1 (below).

If shallow foundations were to be desired, then it is considered that the bridge span would need to be lengthened such that the bridge could be supported on the embankment fill *behind* the existing abutments (and not on the fills within the foreslopes). The embankment fills behind the existing abutments appear to be in a more compact state and to have been placed under more controlled conditions. Therefore, if the span of the temporary modular



bridge could be lengthened, then it could be feasible to support the bridge on a pad of compacted engineered fill supported on the existing granular embankment fills behind the current abutments. Further discussion on the feasibility of this option and recommendations on the design are provided in Section 6.4.2 (below).

A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options for the modular bridge is presented in Table 2 following the text of this report. Support of the modular bridge foundations entirely on steel H-piles is considered to be the preferred option, from a foundation engineering perspective.

6.4.1 Steel H-Pile Foundations

The guidelines provided previously in Section 6.3.1 for the design of steel H-piles for the bridge and retaining wall foundations apply equally to the foundations of the temporary modular bridge. The only differences relate to the position and longer length of the piles that will support the separate bearing supports – i.e., the supports which are not located along the abutment/retaining walls.

Those foundations are located above the existing foreslopes. Construction of the retaining/abutment walls and filling over the foreslopes (in accordance with Section 6.7.1 of this report) will need to precede the pile installation.

The pile caps should be provided with a minimum of 1.7 m of soil cover for frost protection.

It is understood that the founding levels for the west and east supports would be at about elevations 44.5 m and 45.8 m respectively. The piles would therefore be about 16 to 18 m long.

The guidelines provided in Section 6.3.1.1 for the axial resistance of H-pile foundations apply equally to these piles.

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. Alternatively, as discussed in Section 6.3.1.2, the resistance to lateral loading could be derived from the soil in front of the piles.

The following values of n_h and s_u may be assumed in the structural analysis.

Location	Elevation (m)	Soil Type	γ (kN/m ³)	Friction Angle (degrees)	n_h (MN/m ³)	s_u (kPa)
West Support	PCL – 58.2	New Compacted Fill	21.5	35	6.5	-
	53.6 – 58.2	Existing Embankment Fill	20	30	3	-
	52.7 – 53.6	Silty Clay	17.5	0	-	100
	Below 52.7	Glacial Till	21	35	11	-
East Support	PCL – 60.9	New Compacted Fill	21.5	35	6.5	-
	57.2 – 60.9	Existing Embankment Fill	20	30	3	-
	Below 57.2	Glacial Till	21	35	-	-
					11	-

Note: PCL = Pile cap level.

Other than the above recommendations specific to the modular bridge foundations, the other recommendations provided in Section 6.3.1.2 apply equally to the lateral resistance of these foundations.



6.4.2 Shallow Foundations

If the span of the temporary modular bridge can be lengthened, so that the supports would be located in the median area behind the existing abutments, then it is potentially feasible to support the bridge on spread footing foundations.

To avoid reductions in the bearing resistance associated with stability of the embankment foreslope, it is recommended that, as a preliminary guideline, the front face of the foundations should be located at least 5 m behind from the crest of the median slope. This set-back distance is a preliminary recommendation and would need to be confirmed once the overall load from the foundations is known.

The boreholes put down in the median area generally encountered compact granular fill materials. However a surficial layer of more random fill material was locally encountered in some of the boreholes. For example, Borehole 08-15 encountered about 1.5 m of silty clay fill and Borehole 08-9A encountered about 3 m of loose sandy silt. It is therefore proposed that any surficial topsoil, organic fill materials, or *silty clay* fill should be subexcavated from beneath the foundation areas and from the full zone of influence of the foundations (defined below). It is considered that sandy silt or other granular fills could remain in-place beneath the new foundations provided that the subgrade would be proof rolled and that the foundations would be supported on a minimum 2 m thick layer of compacted engineered fill (Granular A).

The zone of influence used to establish the subexcavation limits and engineered fill placement is considered to be defined by a theoretical surface extending down and out from the edge of the foundations at a slope of 1H:1V (horizontal:vertical). From a constructability/practical perspective, it is recommended that this zone be extended out a further 1 m each way from the foundation edge (i.e., to form a minimum 2 m overall top width, centered on the wall alignment).

The subgrade beneath these limits should first be proof rolled and compacted to at least 95 % of the subgrade material's standard Proctor maximum dry density, prior to placing the engineered fill.

It would need to be confirmed during construction that the soils at the base of the excavation are consistent with those anticipated. 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.

That zone of influence should then be filled using compacted engineered fill consisting of OPSS Granular 'B' Type II or Granular 'A', placed in maximum 300 mm thick lifts, and compacted to at least 95% of its standard Proctor maximum dry density.

A sketch showing the approximate arrangement of the granular pad is provided on Figure 16.

It is expected that the founding elevation of the engineered fill pad would be at about elevation 60 to 61 m (based on a 2 m height for the ballast wall and foundations, and a further 2 m height for the granular pad). The footings of the temporary modular bridge, supported on at least 2 m of Granular A fill, may then be designed to a factored ULS geotechnical resistance of 900 kPa and an SLS geotechnical resistance of 350 kPa.

Although it is expected that the foundation settlements for this arrangement would not be excessive, the actual total settlements are difficult to predict, since they relate to compression of the existing embankment fills. As a



preliminary guideline, it is expected that the settlements of foundations sized using the above SLS bearing resistance value could be in the range of 25 to 50 mm. That estimate could potentially be refined once the total loading on the foundations and the size of the foundations are known.

Resistance to lateral forces as provided by sliding resistance between the concrete footings and Granular A pad should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.55 for cast-in-place concrete footings constructed on the granular fill. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.5 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the *CHBDC* may be taken as 1.0, consistent with Soil Profile Type I.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any retaining walls 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 freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutment stems and retaining walls:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) 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 MTO's Special Provision 105S10. 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 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 6.9.3. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. 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 1.7 m behind the back of the abutment 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	0.27
At rest, K_o	0.43	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 abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- 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 Cornwall is 0.2. Based on experience, for the subsurface conditions at this site, no significant amplification of the ground motion is expected. 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*, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.3$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.1$).

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	0.39	0.30	0.30
Non-yielding wall	0.62	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 0.20. This corresponds to displacements of up to approximately 50 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 depth, d, (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/m³), as given previously;
- d is the depth below the top of the wall (m); and,
- H is the total height of the wall (m).

6.7 Approach Embankment and Retaining Wall Design and Construction

The embankments are to be extended forward to the new abutment and retaining wall alignments. Extensive filling over the existing embankment foreslopes will therefore be required, with the grade being raised (behind the new abutments) by up to about 5 to 6 m at the new west abutment and by about 3 m at the east abutment. Overall, the new abutment and/or retaining walls will be up to about 8 to 9 m high.

Wing walls will also be provided extending north and south of both the east and west abutments, parallel to the rail tracks, extending approximately 30 m from the ends of the new abutment walls. The filling behind those walls will widen the embankment footprint in this area (of the junction between the existing embankment side-slopes and fore-slopes), although it is understood that the overall embankment width, behind the existing abutments, will not change significantly.

Both cast-in-place concrete walls and RSS wall systems have been considered for the new retaining walls and wing walls.



6.7.1 Subgrade Preparation and Embankment Construction

Based on the borehole results, the subgrade soils will consist of existing embankment fill materials and stiff weathered silty clay overlying silty sand to sandy silt till.

Any surficial topsoil, organic matter and softened / loosened soils should be stripped from within the limits of the new embankment filling, including the existing embankment sideslope and the new footprint. All subgrade soils should be proof-rolled prior to fill placement.

Construction of the embankment should be in accordance with SP206S03. Embankment fill should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent 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 percent 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.

The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.7.2 Approach Embankment and Bridge Retaining Wall Stability

With appropriate subgrade preparation and proper placement of earth or granular soils, the up to 11 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical, founded on the native stiff weathered silty clay (where present) and glacial till, will have a factor of safety greater than 1.3 against deep-seated slope instability.

Similarly, the proposed abutment retaining wall and adjacent wing walls, up to 9 m in height, founded on the native soils or engineered fill, after removal of the existing fill materials and surficial topsoil, will have a factor of safety greater than 1.3 against deep seated slope instability.

RSS walls, up to 9 m in height, founded on the native soils or on engineered fill on the native soils after removal of the existing fill materials and surficial topsoil, will have a factor of safety greater than 1.3 against deep seated slope instability. The *internal* stability of RSS walls should however be checked by the supplier.

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the retaining walls and embankment side slopes will have factors of safety of greater than 1.1 against deep-seated slope instability based on an acceleration of 0.1g. The results do however indicate that some shallow sloughing (with factors of safety less than 1.1) could occur of the embankment side slopes during seismic loading. That sloughing would not however impair the short term use of the structure and is mainly a maintenance/repair issue. The potential for sloughing could be reduced by providing well vegetated side slopes.



The slope stability analyses for the above embankment and retaining wall configurations were carried out using the following parameters:

Material	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Earth or Granular Embankment Fill	21	32°	
Existing Fill	21	32°	
Weathered Silty Clay	17.5		100
Till	22	Impenetrable by failure surface	

6.7.3 Embankment Settlement

Settlement of the approach embankment extensions, adjacent to the abutments, and any RSS walls will occur due to compression of the new embankment fill itself, as well as compression of the existing fill materials and the discontinuous silty clay. Settlements due to compression of the underlying glacial till should be negligible in magnitude.

Provided that the new embankment fill material consists of Select Subgrade Material or clean earth fill, the settlement due to compression of the embankment fill itself is expected to be less than about 25 mm. The use of granular fill for the new embankment construction would reduce the magnitude of *post-construction* settlement (likely to less than half that value), since the majority of the settlement of granular fills will occur during construction.

The new embankment fill materials will be (partially) underlain by the existing embankment fill materials that form the current foreslopes. These existing fill materials can generally be left in place beneath the embankment widening provided some modest settlement (i.e., less than 15 mm) of the subgrade can be tolerated. However the subgrade surface should be proof rolled and compacted to 95 percent of the standard Proctor maximum dry density.

Some additional subgrade settlements can be expected over the south portion of the west embankment (and retaining wall) where up to about 2 to 3 m of generally stiff weathered silty clay are present. At the very end of the retaining wall (i.e., wing wall), the clay is up to about 5 m thick although the wall tapers to nil by this location. Beneath most of the areas to be filled, the full thickness of the deposit has been weathered.

The silty clay deposit will be stressed below its estimated preconsolidation pressure and therefore the settlements are not expected to exceed 25 mm in magnitude. Furthermore, the coefficient of consolidation of the weathered silty clay, typically being a fissured soil and being stressed within its re-compression limits, is expected to be relatively high. The deposit is also generally fairly thin and only very minimal filling is planned over the thicker portions. Therefore the subgrade settlements resulting from compression of the silty clay would be expected to occur quite rapidly (within 1 to 2 months), likely almost entirely during embankment construction, such that the post-construction settlements of the embankment surface would not be expected to noticeably exceed the compression of the embankment fill itself.

The subgrade settlements of the east embankment, where the silty clay is absent, should be negligible.



6.8 Retained Soil Systems

It is understood that retained soil systems (RSS) are being considered for the wing walls, which will extend approximately 30 metres north and south of the abutments, parallel to the rail tracks. It is further understood that, although cast-in-place concrete walls are currently proposed to form the new abutments and retaining walls, aligned parallel to the tracks, consideration has also been given to using an RSS wall system to form those retaining walls.

From a foundation engineering perspective, it is considered that both cast-in-place reinforced concrete retaining walls or RSS walls may be used for these walls. The choice of retaining wall system will depend on the desired appearance, the anticipated costs, performance and on other considerations such as constructability. Furthermore, for the southwest wall where the clay is up to about 5 m thick, the use of an RSS wall avoids the potential need for significant subexcavation and track protection, as would be required if a reinforced concrete wall supported on shallow foundations was proposed.

As discussed previously, settlements at this location should be less than the allowable limits for RSS walls. RSS walls are therefore feasible from that perspective.

It is understood that the Site Performance Rating is classified as High and the Appearance Criteria is also classified as High for this site.

Given the relatively greater flexibility of an RSS system versus a cast-in-place concrete wall, subexcavation of the silty clay (as discussed in Section 6.3.3 for bridge and retaining wall shallow foundations) is not considered necessary. The RSS wall and its foundations can therefore be supported on a minimum 0.3 m thick compacted Granular 'A' or Granular 'B' Type II levelling pad constructed on the surface of the stiff weathered silty clay (where present) or glacial till. For these conditions, RSS wall foundations can be evaluated based on a factored ULS bearing resistance of 250 kPa and an SLS resistance of 200 kPa (based on 25 mm of settlement).

The design founding levels for the granular pad, consistent with the granular pad requirements described above and design grading, would be as follows:

Foundation	Founding Elevation (m)	Founding Strata	Remarks
WBL Structure West Wing Wall	55.3	Fill	Subexcavation of fill required to reach surface of the glacial till, at elevation 52.8 m.
WBL Structure East Wing Wall	55.3	Glacial Till	
EBL Structure West Wing Wall	55.3	Fill	Subexcavation of fill required to reach surface of the weathered silty clay, at elevation 53.5 to 53.7 m. Subexcavation of the silty clay not required.
EBL Structure East Wing Wall	55.3	Glacial Till	



As noted in the above table, the founding levels of the RSS walls adjacent to the west abutment walls would be above the native subgrade surface and within the existing fill materials, at least in the area closest to the ends of the abutments. Those fill materials consist of a loose to compact mixture of silty sand, sandy silt, and silty clay, which range from a generally loose to compact state, and locally contain organic matter. The fill materials in the embankment foreslopes do not therefore appear to have been placed in a controlled manner, as compacted engineered fill. It is therefore proposed that any fill materials be subexcavated from beneath the RSS walls and from the full zone of influence of the foundations. That zone of influence is considered to be defined by a theoretical surface extending down and out from the edge of the foundations at a slope of 1H:1V (horizontal:vertical). From a constructability/practical perspective, it is recommended that this zone be extended out a further 1 m each way from the wall/foundation edge (i.e., to form a minimum 2 m overall top width, centered on the wall alignment). That same zone of influence should then be filled using compacted engineered fill consisting of OPSS Granular 'B' Type II or Granular 'A', placed in maximum 300 mm thick lifts, and compacted to at least 95% of its standard Proctor maximum dry density. A sketch of this arrangement is provided on Figure 17.

The subgrade levels shown in the above table needed to subexcavate the fill materials are based on boreholes advanced near the ends of the proposed abutments, and through the west foreslopes, plus Augerhole 09-1 at the end of the southwest wing wall. The required subgrade level (i.e., subexcavation depth) may therefore be expected to vary along the length of the walls, particularly with distance away from the abutment wall. It would therefore need to be confirmed during construction that the soils at the base of the excavation are consistent with those anticipated. 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.

For the east walls, where the surface of the glacial till is higher, it is expected that the founding level would be within the glacial till, which would be an acceptable bearing material.

The reinforcing strips for a RSS wall are typically about 0.8 times the height of the wall in length and therefore some subexcavation of the existing embankment fill may be required in order to install the RSS reinforcing strips and granular fill. It is expected that temporary excavation support measures would be required to ensure the stability of the existing embankment side slopes during the removal of the existing embankment fill materials.

The resistance to lateral forces/sliding for the wall should be calculated in accordance with Section 6.7.5 of the *CHBDC*, using the following parameters:

Interface and Loading Condition	Parameter
Granular 'A' pad – clay subgrade: short term loading	Undrained cohesion = 100 kPa
Granular 'A' pad – clay subgrade: long term loading	Effective friction angle of 30 degrees
Granular 'A' pad – till subgrade: short or long term loading	Effective friction angle of 35 degrees

The internal stability of the RSS wall should be checked by the RSS supplier/designer.



6.9 Design and Construction Considerations

6.9.1 Excavations

The excavations for the construction of abutments or cast-in-place retaining wall foundations will extend through the existing fill materials and the stiff silty clay (west side), and potentially into the glacial till. Excavations 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. These soils are classified as Type 3 soils according to the OHSA and therefore excavations should be made with side slopes no steeper than 1 horizontal to 1 vertical.

If shallow foundations are to be used, the excavations for the west foundations could extend up to 5 m below track level (if/where the silty clay is to be subexcavated). Rail track protection (i.e., temporary shoring) will be required for this excavation; track protection was apparently also required for construction of the existing piers.

It is expected that track protection will not be needed for the east foundations, which will be located much further from the existing tracks and where (significant) subexcavation of silty clay is not needed.

Roadway protection, installed parallel to the highway alignment and extending back from the abutment areas, along both sides of the median, will also likely be required to accommodate the construction access and staging (to build access ramps).

6.9.2 Subgrade Protection

If shallow foundations are chosen to support the bridge or retaining walls on the glacial till, the till subgrade may be wet and sensitive to disturbance. In that case, the subgrade should be protected with a working mat of lean concrete that is placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed either with a note on the General Arrangement drawing, or with a NSSP. It is recommended that MTO's Special Provision SP902S01 be included in the Contract Documents, requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction.

6.9.3 Temporary Excavation Shoring

As discussed above, railtrack protection will be required for the construction of the west foundations. Roadway protection will also likely be required to accommodate the construction access and staging. As mentioned previously, a separate foundation investigation has been carried out for the roadway protection that will be needed for the new construction access ramps. The results of that investigation are reported separately.

The temporary excavation shoring (i.e., roadway and railtrack protection) should be designed and installed in accordance with MTO Special Provision 105S19. The roadway protection should be designed to Performance Level 2 as defined in SP 105S19. For the rail track protection, the requirements for the excavation support and the performance level should be determined based on consultation with the railway, in view of the tolerance of the railway to accept movement.

It is understood that the design of the shoring will be entirely the responsibility of the contractor. To the expected depths of excavation, it is not expected that basal heaving or basal instability will be a concern. The shoring will



have to be designed to resist lateral earth pressures that are controlled by the flexibility of the shoring and its method of support. Conceptually, it is expected that the rail track protection might consist of steel sheet piling driven through the silty clay and into the glacial till, with lateral support provided by means of internal braces. The roadway protection could conceivably consist soldier piles and lagging, with the lateral restraint provided by means of either rakers supported on footings or piles within the excavation, or using tie-backs grouted into the soil or bedrock behind the shoring. Cantilevering of the shoring might also be feasible, provided the retained height is no more than about 3 m.

For the rail track protection, the potential for interference with the existing pier foundations (or the rail track protection from the original construction) will need to be evaluated.

6.9.4 Reinstatement of Construction Access Ramps

Based on the borehole results from the present investigation and the separate investigation for the shoring for the access ramps, the soils excavated to create the ramps will consist of existing sand and gravel to sandy silt embankment fill materials. Layers/zones of silty clay fill (in some cases with organic matter) also exist.

Re-instatement of the access ramps should be in accordance with SP206S03. Fill should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent 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 percent 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.

It should generally be possible to re-use the excavated embankment fill, which has a water content of about 8 to 16 percent, as backfill to the access ramps, once bridge construction is complete. However, portions of the embankment fill with high water contents (i.e., clayey material) should not be used as backfill since these materials may be too wet to compact.

6.9.5 Groundwater and Surface Water Control

The groundwater level is considered to be in the range of elevation 55 to 57 m. Some ground inflow to the excavation could therefore be experienced, particularly if subexcavation of the silty clay is carried out to reach the glacial till. However, for the soil conditions at this site, only a modest amount of groundwater flow is expected for the foundation excavations. It is anticipated that adequate groundwater control can be affected through the use of pumping from properly filtered sumps in the excavations.

Surficial drainage may be also required around the perimeter of the excavation due to the interference of the foundation excavations with the existing drainage ditches and pipes.



6.9.6 Obstructions

Numerous boulders were encountered in the boreholes advanced through the till during the present investigation, and during the separate investigation carried out for the construction access ramp shoring. The presence of boulders in the glacial till could affect the installation of the driven steel H-piles. Pre-drilling of the pile locations could be required. Provision should be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions. Further discussion on this issue is provided in Section 6.3 of this report.



7.0 CLOSURE

This report was prepared by Ms. Kim S. Lesage, EIT, under the direction of the Project Manager, Mr. Michael Cunningham, P.Eng., Mr. Fintan J. Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

Yours truly,

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FOUNDATION INVESTIGATION - GWP 237-00-00

**Table 1: Evaluation of New Bridge Foundations/Construction Alternatives
Highway 401 Bridge Replacement Over CN Rail Line
GWP 237-00-00**

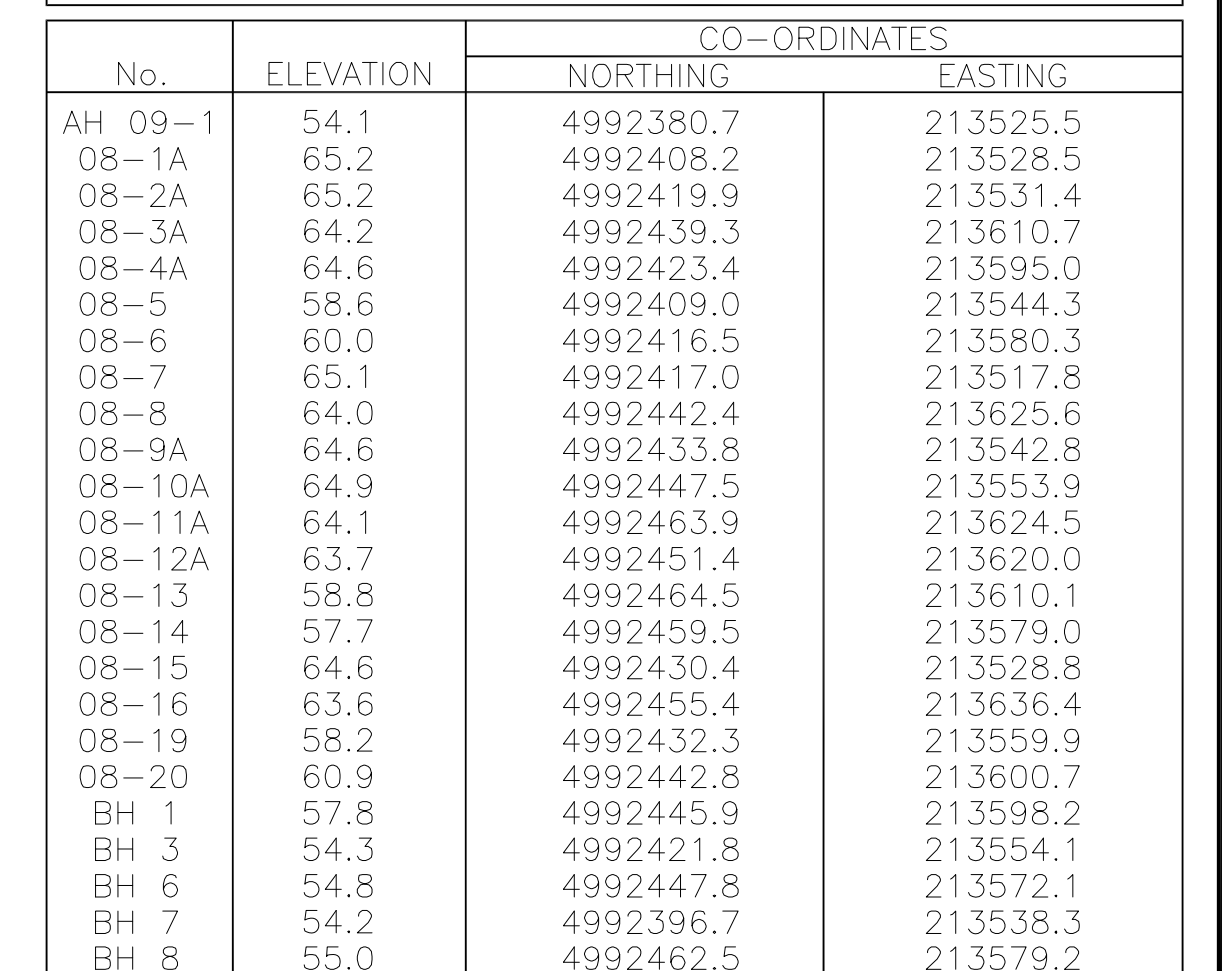
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on glacial till	<ul style="list-style-type: none"> Feasible. 	<ul style="list-style-type: none"> Relatively simple construction. 	<ul style="list-style-type: none"> Requires excavation up to 5m deep beside railway track on the west side. 	<ul style="list-style-type: none"> Likely least expensive option, except for shoring cost. 	<ul style="list-style-type: none"> Possible additional subexcavation needed, due to variations in till surface elevation. Risk of settlement of rail line due to excavation on west side.
Steel H-piles	<ul style="list-style-type: none"> Feasible for support of all foundation elements. 	<ul style="list-style-type: none"> High bearing resistance. Negligible settlement. 	<ul style="list-style-type: none"> Possibility of encountering cobbles or boulders during pile driving, and needing to pre-drill pile locations, use lower pile capacity, and/or drive additional piles. Negative skin friction ("down drag") loads must be considered in design. 	<ul style="list-style-type: none"> Less expensive than rock-socketed caisson option. 	<ul style="list-style-type: none"> Risk of having to pre-drill or drive additional piles. Risk of damage to piles due to boulders in till.
Cast-in-place concrete caissons founded or socketed into rock	<ul style="list-style-type: none"> Feasible for support of all foundation elements. 	<ul style="list-style-type: none"> High resistance. Negligible settlement. 	<ul style="list-style-type: none"> Permanent casings required to construct caissons. High likelihood of encountering cobbles or boulders during drilled shaft installation. If rock socket required, coring or churn drilling will be required to form socket in medium strong bedrock. Negative skin friction ("down drag") loads must be considered in design. 	<ul style="list-style-type: none"> Likely most expensive option. 	<ul style="list-style-type: none"> Risk of construction difficulties due to boulders in glacial till. Possible loss of ground associated with liner installation and socket construction.



FOUNDATION INVESTIGATION - GWP 237-00-00

**Table 2: Evaluation of Modular Bridge Foundations/Construction Alternatives
Highway 401 Bridge Replacement Over CN Rail Line
GWP 237-00-00**

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Spread footings supported on engineered fill pad on embankment fill.	<ul style="list-style-type: none"> Feasible, provided bridge span can be lengthened, so that the foundations can be supported at least 5 m behind the median slope crest. 	<ul style="list-style-type: none"> Relatively simple construction. 	<ul style="list-style-type: none"> Requires excavation up to 4m deep beside existing roadway, potentially requiring additional roadway protection. Requires longer modular bridge. Settlements of potentially 25 to 50 mm. 	<ul style="list-style-type: none"> Likely least expensive option, not including shoring cost, and cost of longer bridge. 	<ul style="list-style-type: none"> Possible additional subexcavation needed, due to variations in embankment fill quality.
Steel H-piles	<ul style="list-style-type: none"> Feasible for support of all foundation elements. 	<ul style="list-style-type: none"> High bearing resistance. Negligible settlement. Can use shorter bridge. 	<ul style="list-style-type: none"> Possibility of encountering cobbles or boulders during pile driving, and needing to pre-drill pile locations, use lower pile capacity, and/or drive additional piles. Negative skin friction ("down drag") loads must be considered in design. 	<ul style="list-style-type: none"> Foundations would be more expensive. 	<ul style="list-style-type: none"> Risk of having to pre-drill or drive additional piles. Risk of damage to piles due to boulders in till.



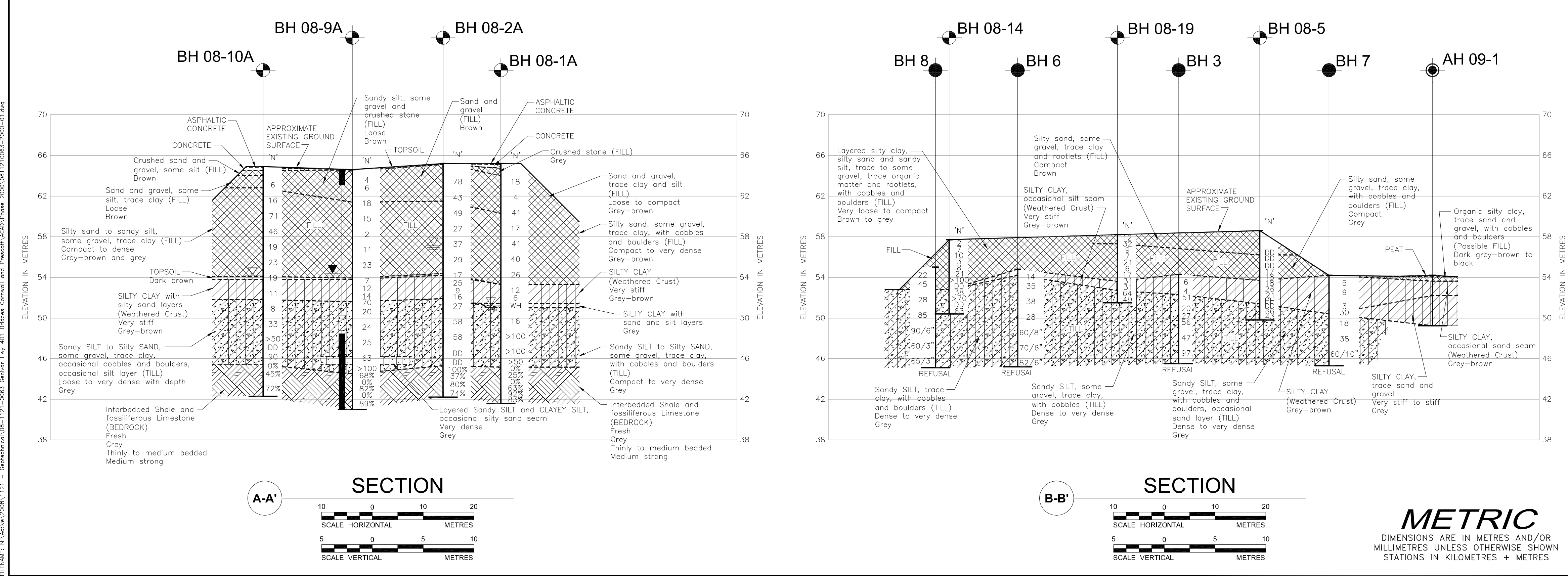
NOTES

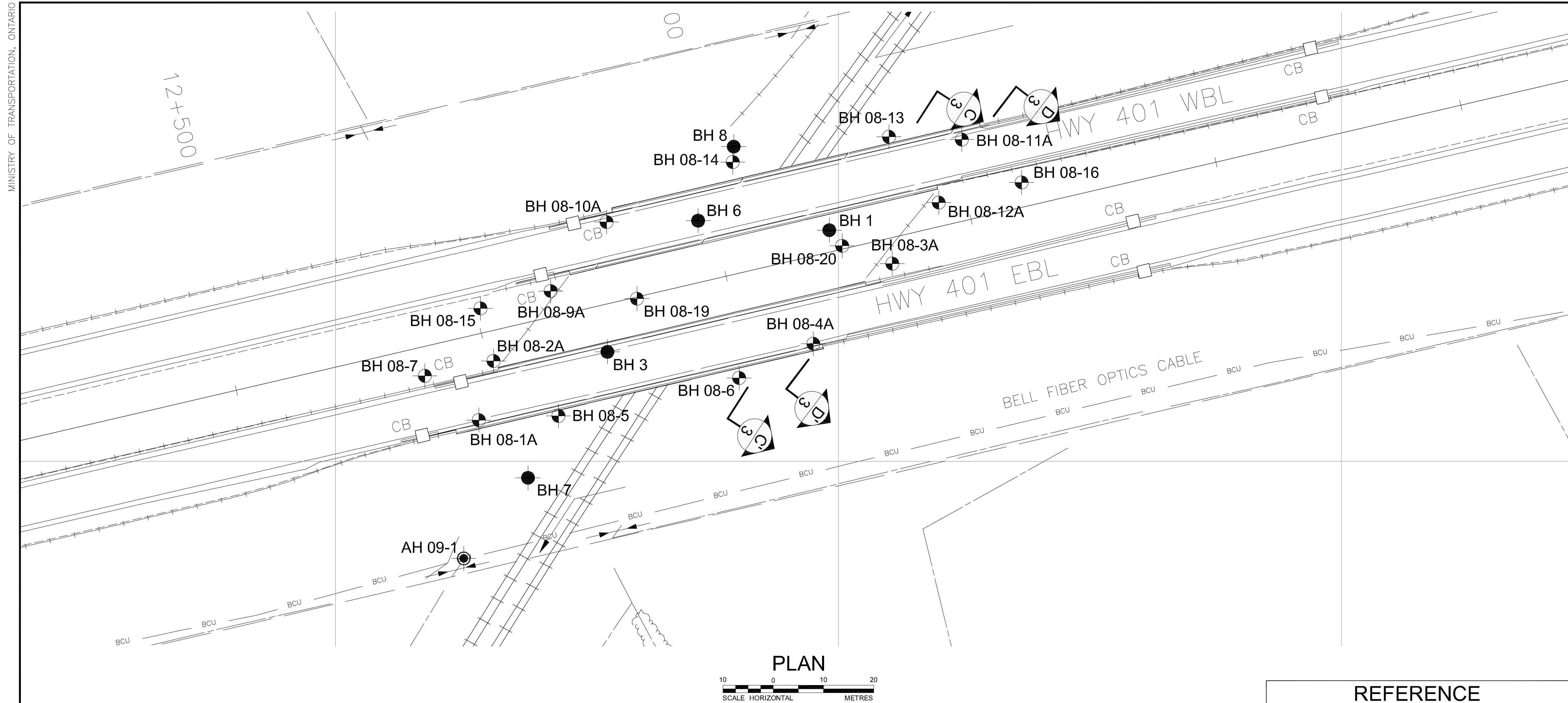
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

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.

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

NO.	DATE	BY	REVISION		
Geocres No. 31G-232					
HWY. 401		PROJECT NO.08-1121-0063		DIST.	
SUBM'D. K.L.	CHKD. M.I.C.		DATE: AUGUST 2009	SITE:	
DRAWN: J.M.	CHKD. F.J.H.		APPD.	DWG. 1	



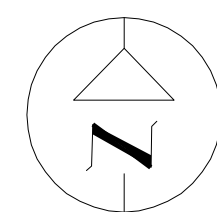
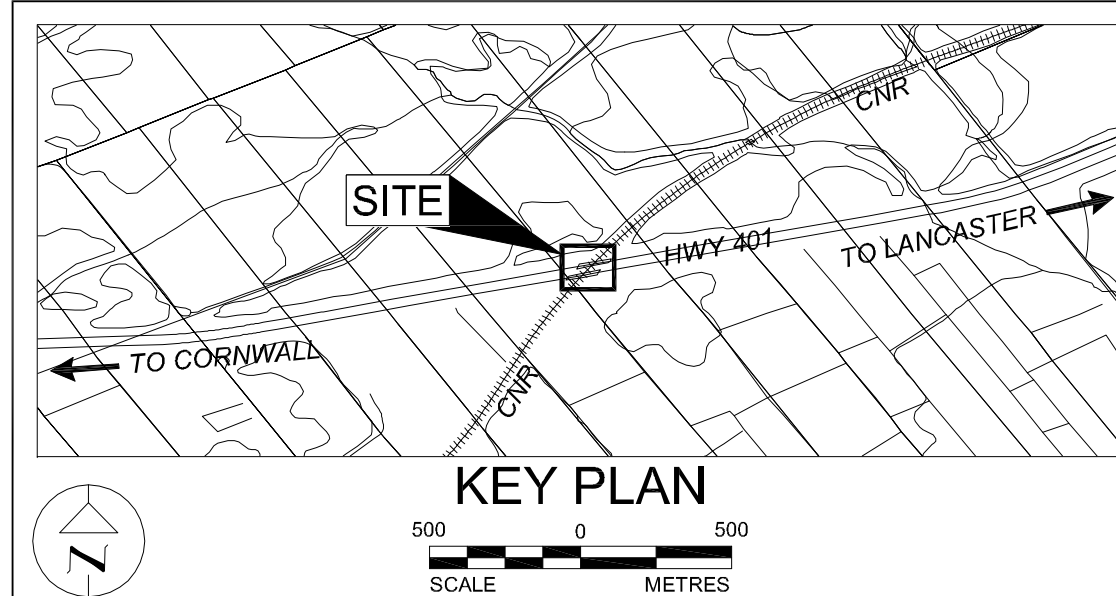


PLAN

SCALE HORIZONTAL
10 0 10 20
METRES

REFERENCE

Base plan supplied by the Genivar Consulting Group

CONT No. -
WP No. 237-00-00HIGHWAY 401 - CORNWALL BRIDGE
REPLACEMENT OVER CNR MILE 63.1
BOREHOLE LOCATIONS AND SOIL STRATASHEET
2Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA

LEGEND

- Borehole — Current Golder Associates Ltd. Investigation
- Augerhole — Current Golder Associates Ltd. Investigation
- Borehole — Previous MTO Investigation
Geocres No. 31G-137
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on August 20, 2009
- WL upon completion of drilling
- C-1 Location of cross-section
- Seal
- Piezometer

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
AH 09-1	54.1	4992380.7	213525.5
08-1A	65.2	4992408.2	213528.5
08-2A	65.2	4992419.9	213531.4
08-3A	64.2	4992439.3	213610.7
08-4A	64.6	4992423.4	213595.0
08-5	58.6	4992409.0	213544.3
08-6	60.0	4992416.5	213580.3
08-7	65.1	4992417.0	213517.8
08-8	64.0	4992442.4	213625.6
08-9A	64.6	4992433.8	213542.8
08-10A	64.9	4992447.5	213553.9
08-11A	64.1	4992463.9	213624.5
08-12A	63.7	4992451.4	213620.0
08-13	58.8	4992464.5	213610.1
08-14	57.7	4992459.5	213579.0
08-15	64.6	4992430.4	213528.8
08-16	63.6	4992455.4	213636.4
08-19	58.2	4992432.3	213559.9
08-20	60.9	4992442.8	213600.7
BH 1	57.8	4992445.9	213598.2
BH 3	54.3	4992421.8	213554.1
BH 6	54.8	4992447.8	213572.1
BH 7	54.2	4992396.7	213538.3
BH 8	55.0	4992462.5	213579.2

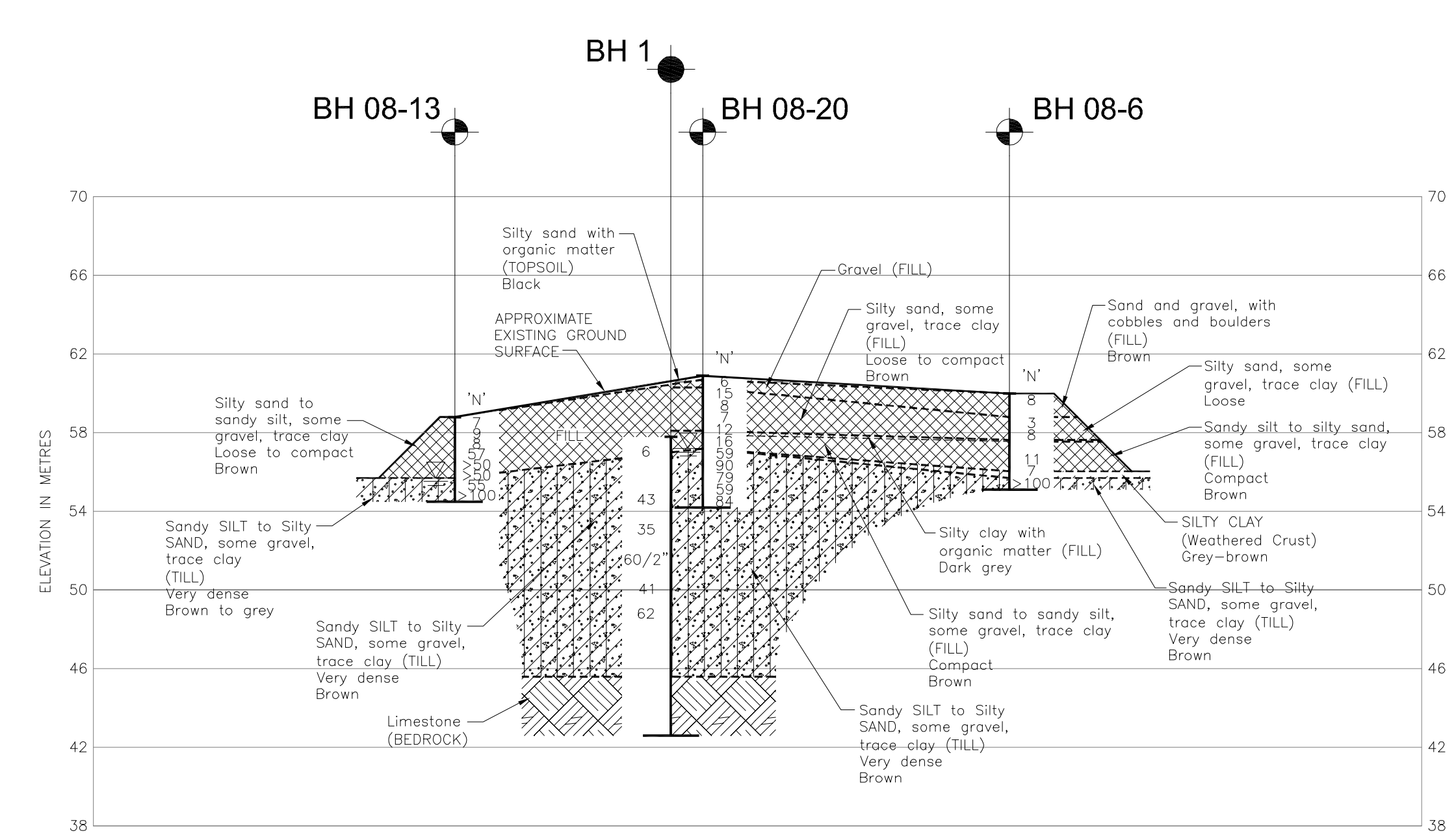
NOTES

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.

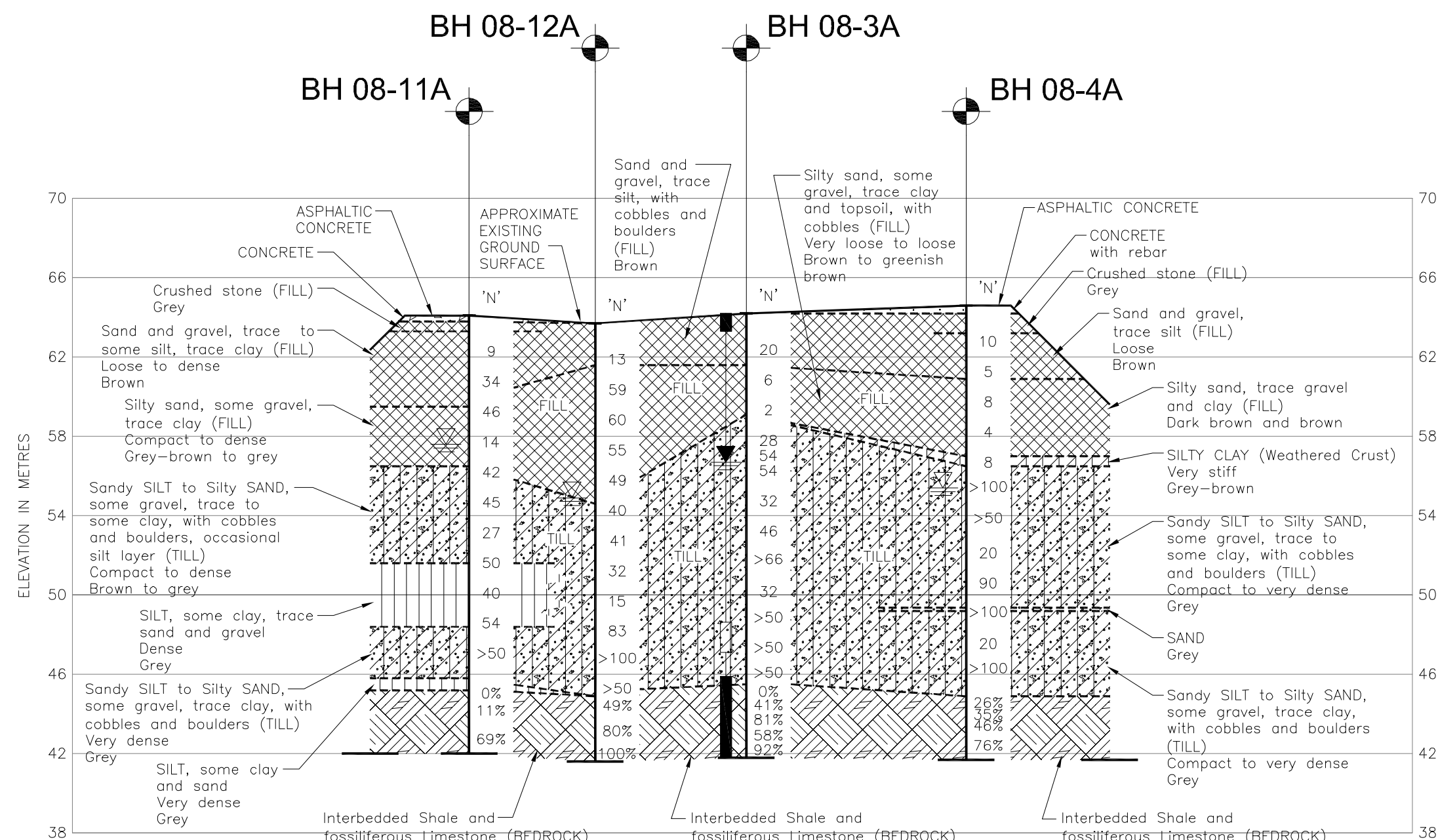
This drawing is for subsurface information only. The proposed details are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

NO.	DATE	BY	REVISION
Geocres No. 31G-232			
HWY. 401		PROJECT NO. 08-1121-0063	DIST.
SUBM'D: K.L.	CHKD: M.I.C.	DATE: AUGUST 2008	SITE:
DRAWN: J.M.	CHKD: F.J.H.	APPD.	DWG. 2



SECTION

C-C'

SCALE HORIZONTAL
10 0 10 20
METRES
SCALE VERTICAL
5 0 5 10
METRES

SECTION

D-D'

SCALE HORIZONTAL
10 0 10 20
METRES
SCALE VERTICAL
5 0 5 10
METRES

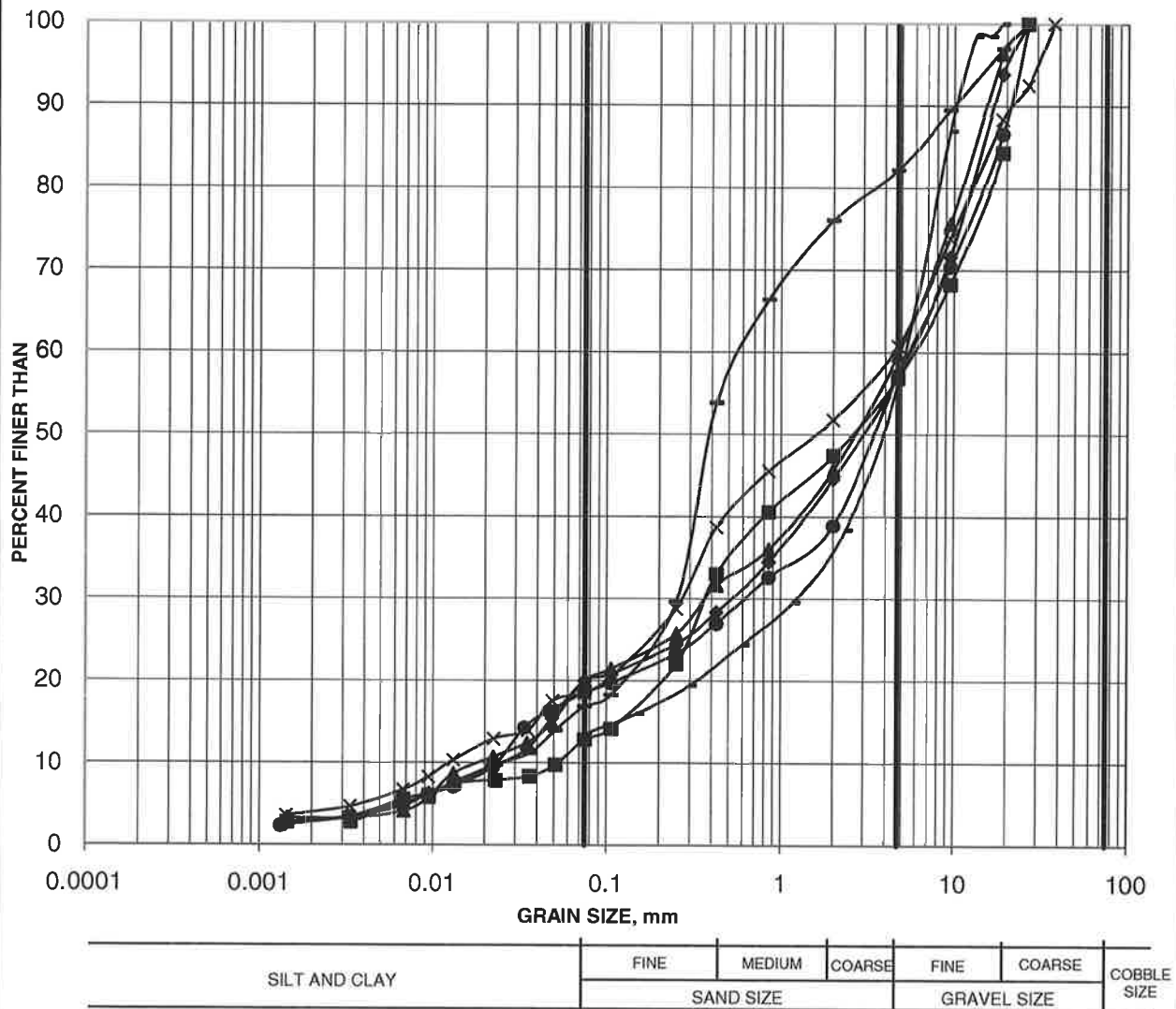
METRIC

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

GRAIN SIZE DISTRIBUTION

FIGURE 1

SAND AND GRAVEL FILL

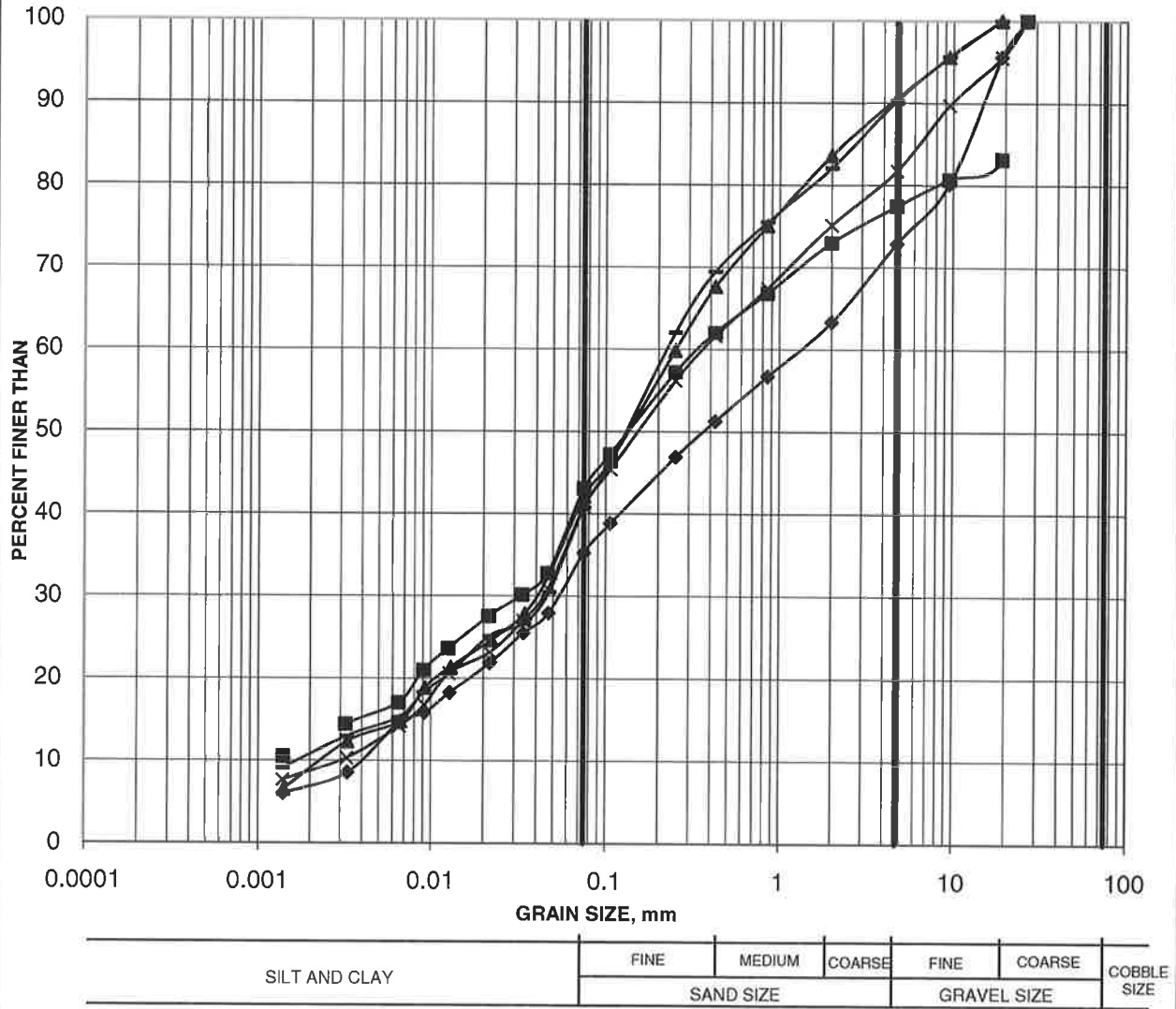


Borehole	Sample	Depth (m)
08-1A	2	1.52-2.13
08-2A	2	3.05-3.66
08-3A	1	0.00-1.52
08-4A	1	0.00-1.52
08-10A	2	1.52-2.13
08-11A	1	1.52-2.13
08-12A	2	1.52-2.13

GRAIN SIZE DISTRIBUTION

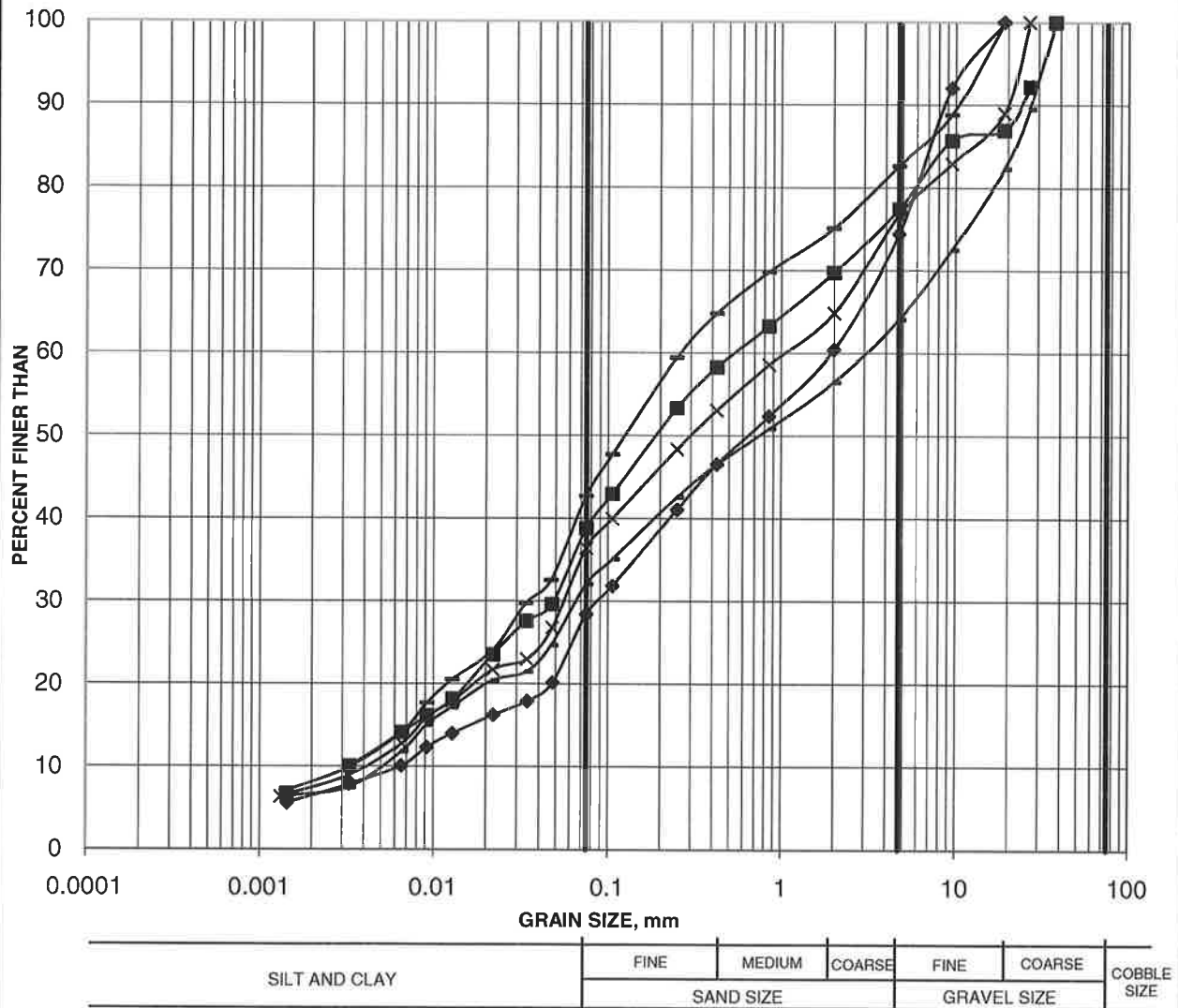
FIGURE 2

SILTY SAND FILL



Borehole	Sample	Depth (m)
08-1A	5	6.10-6.71
08-2A	4	6.10-6.71
08-3A	4	4.57-5.18
08-4A	5	6.10-6.71
08-7	6	9.15-9.76

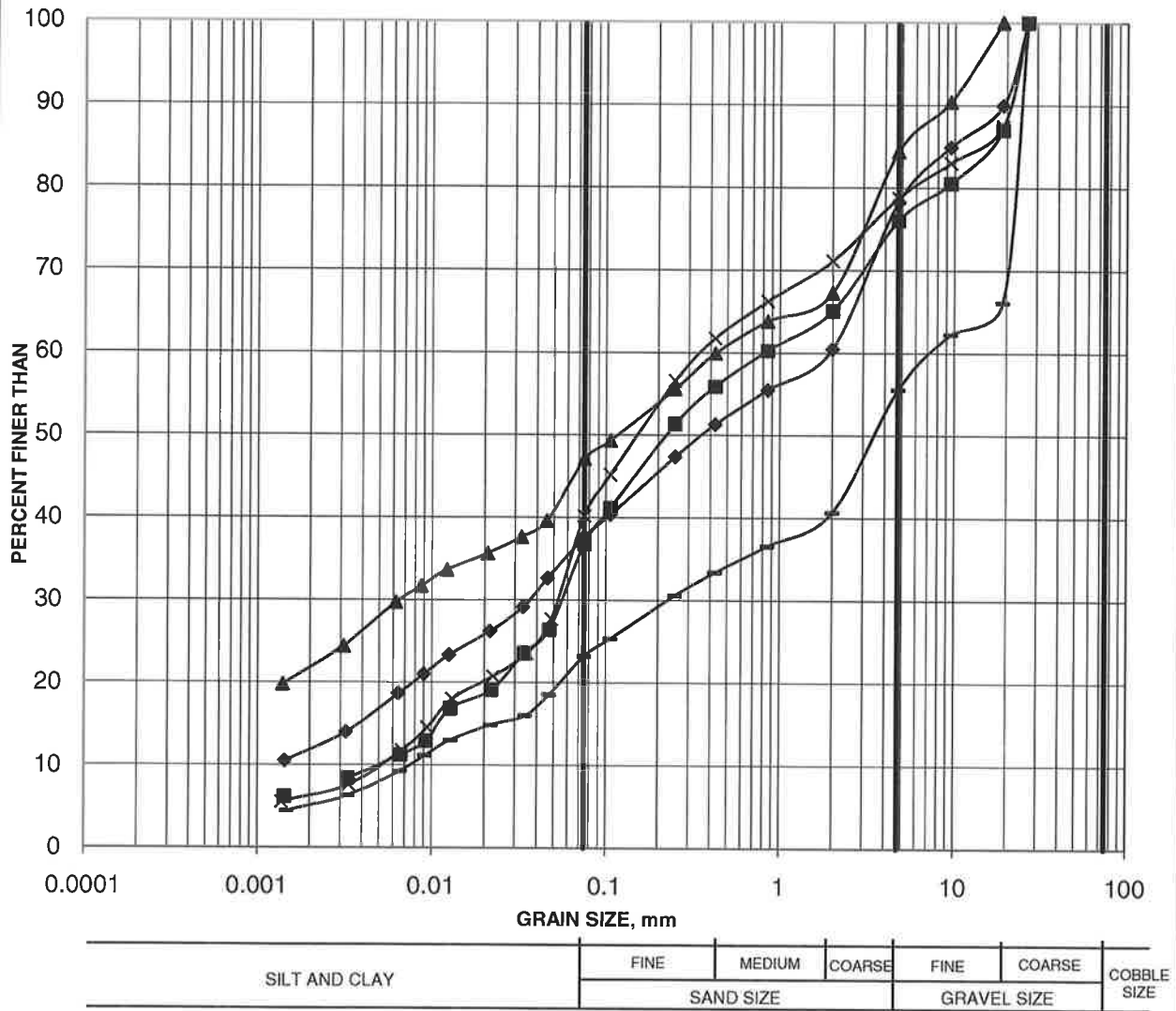
SILTY SAND FILL



GRAIN SIZE DISTRIBUTION

FIGURE 4

SILTY SAND FILL

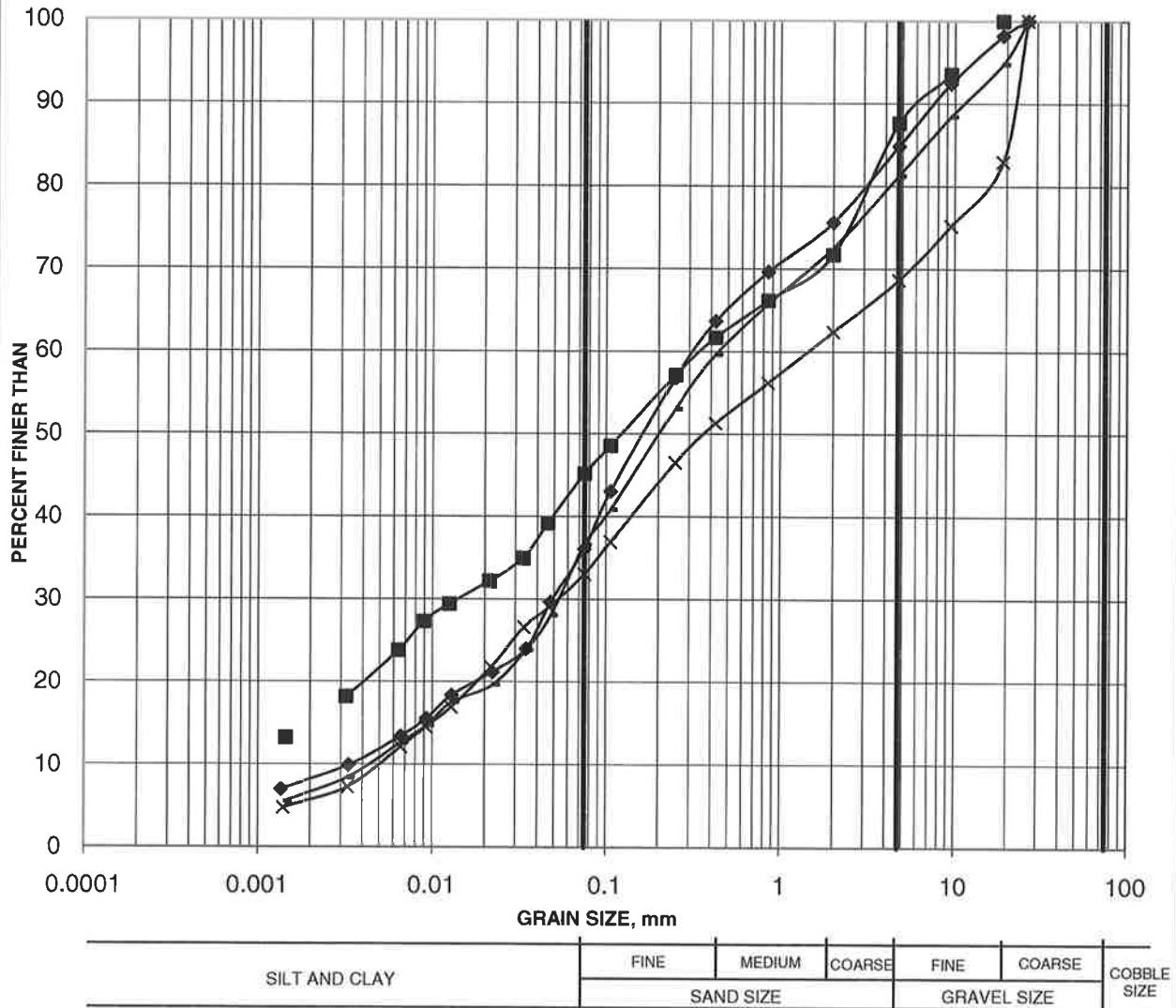


Borehole	Sample	Depth (m)
08-12A	4	4.57-5.18
08-13	2	0.61-1.22
08-14	2	0.61-1.22
08-14	5	2.44-3.05
08-14	6	3.05-3.66

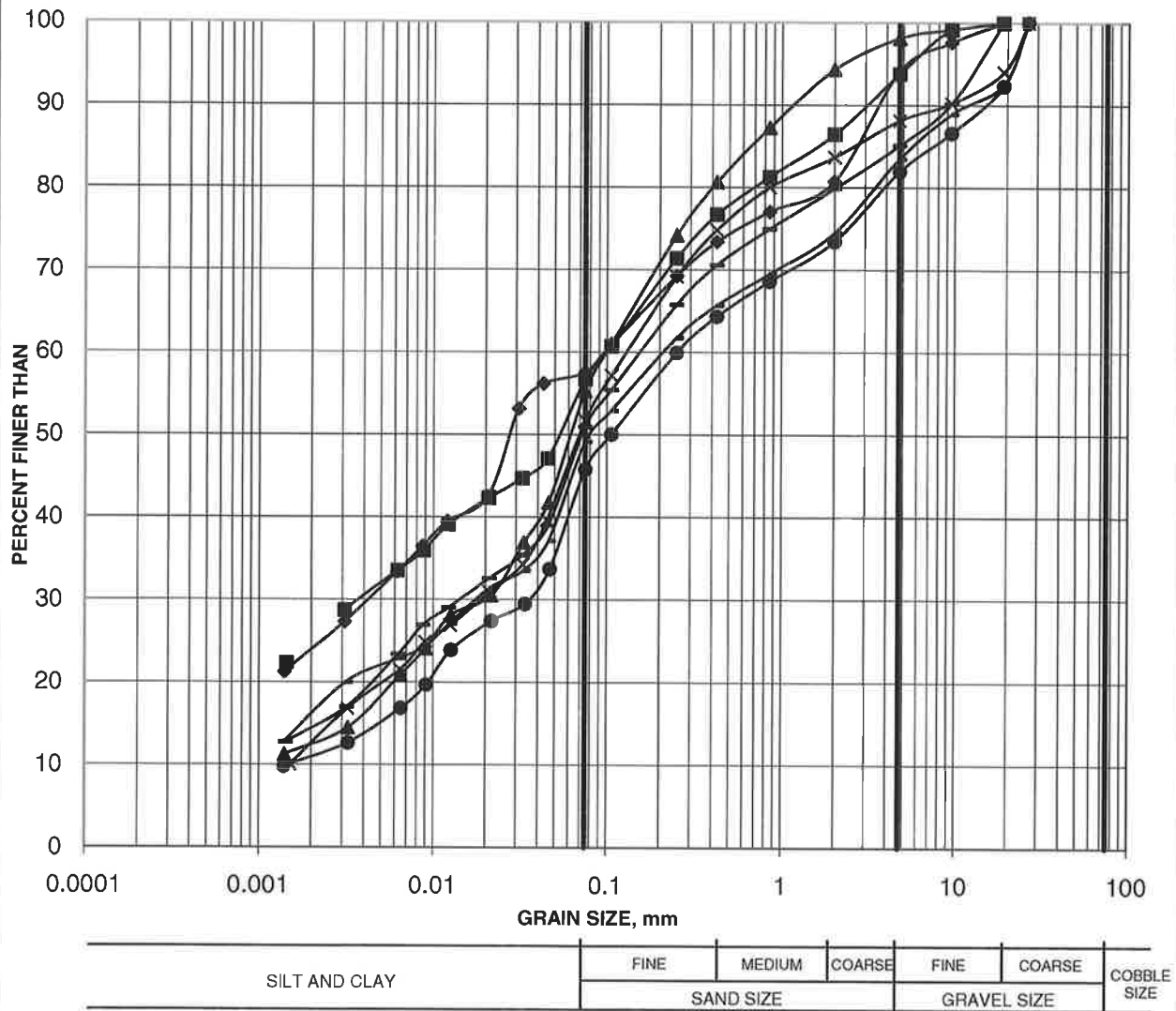
GRAIN SIZE DISTRIBUTION

FIGURE 5

SILTY SAND FILL



SILTY SAND TO SANDY SILT FILL

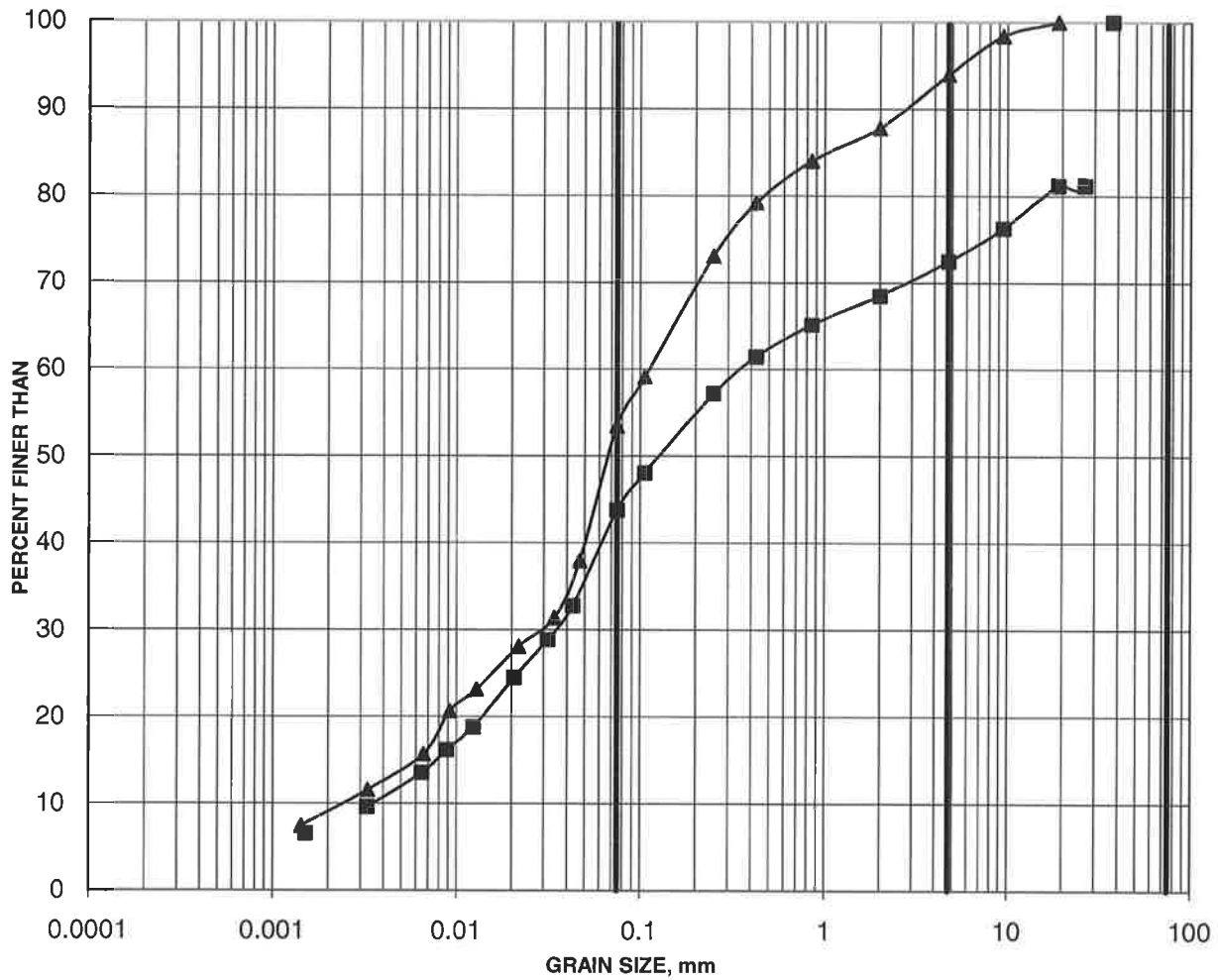


Borehole	Sample	Depth (m)
08-6	2	1.22-1.83
08-6	5	3.20-3.66
08-9A	3	3.05-3.66
08-10A	3	3.05-3.66
08-14	3	1.22-1.83
08-15	3	3.05-3.66
08-15	7	7.62-8.23

GRAIN SIZE DISTRIBUTION

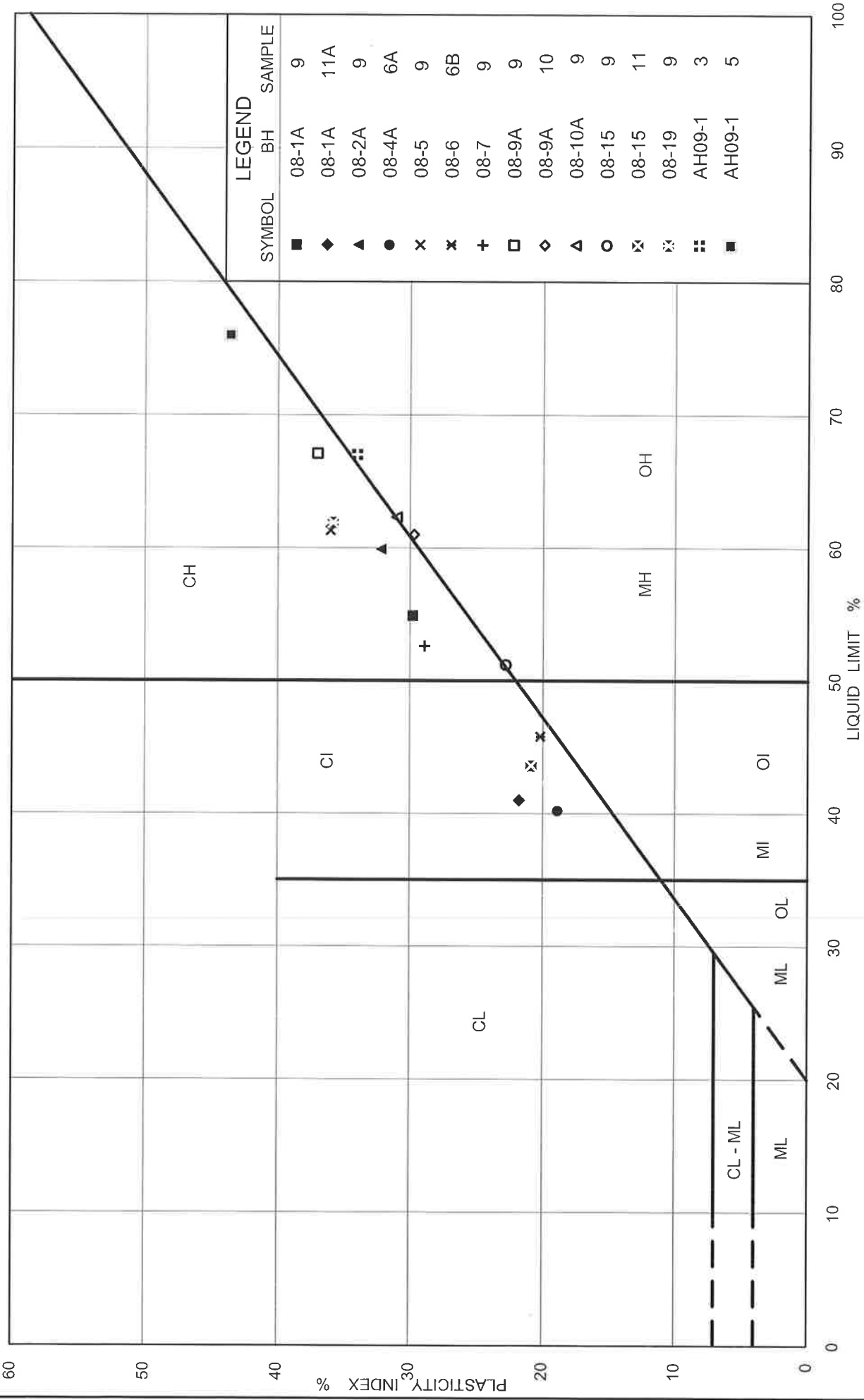
FIGURE 7

SANDY SILT FILL



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

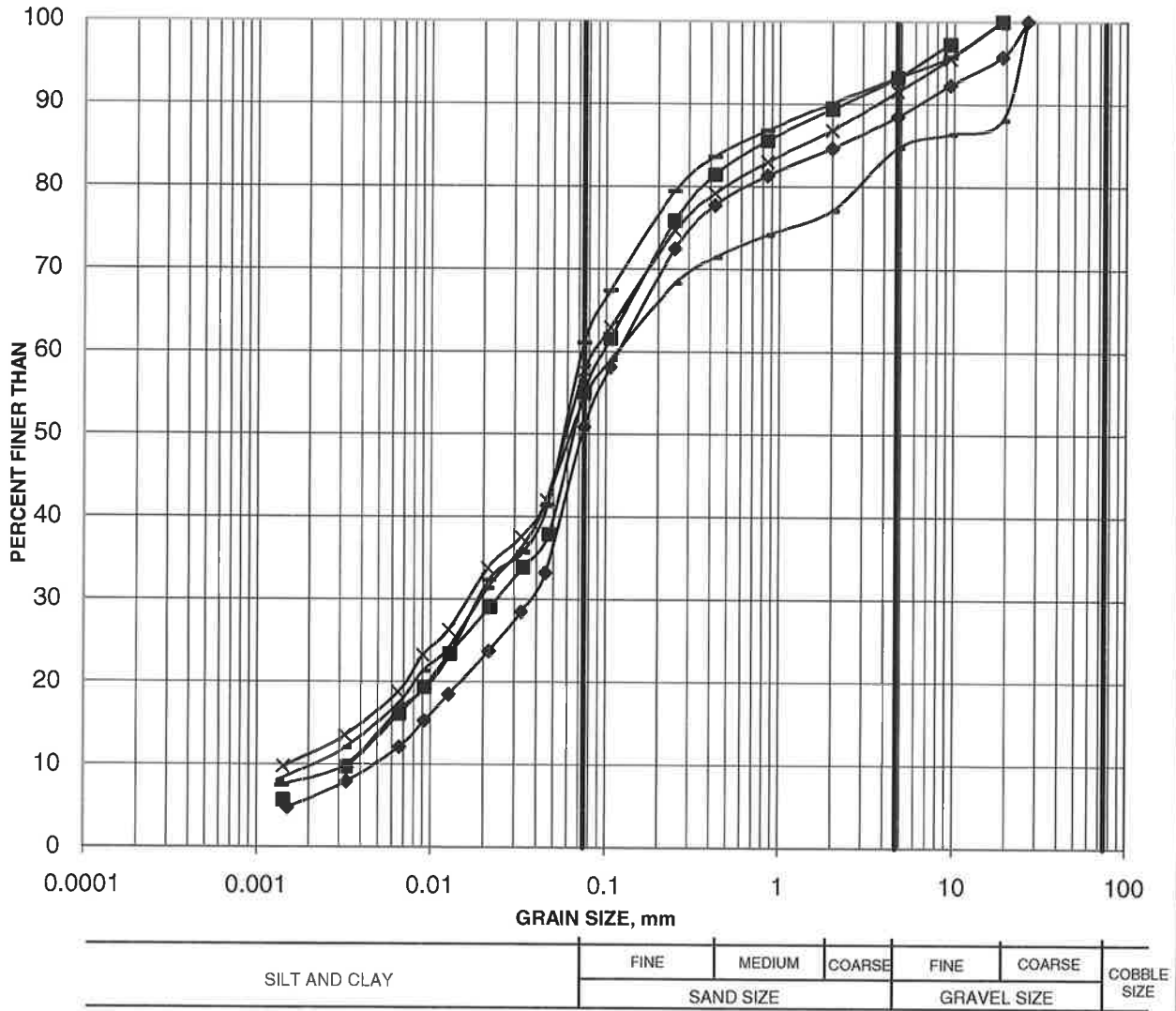
Borehole	Sample	Depth (m)
08-13	08-20	5
		6
		2.44-2.74
		3.05-3.66



GRAIN SIZE DISTRIBUTION

FIGURE 9

SANDY SILT TILL

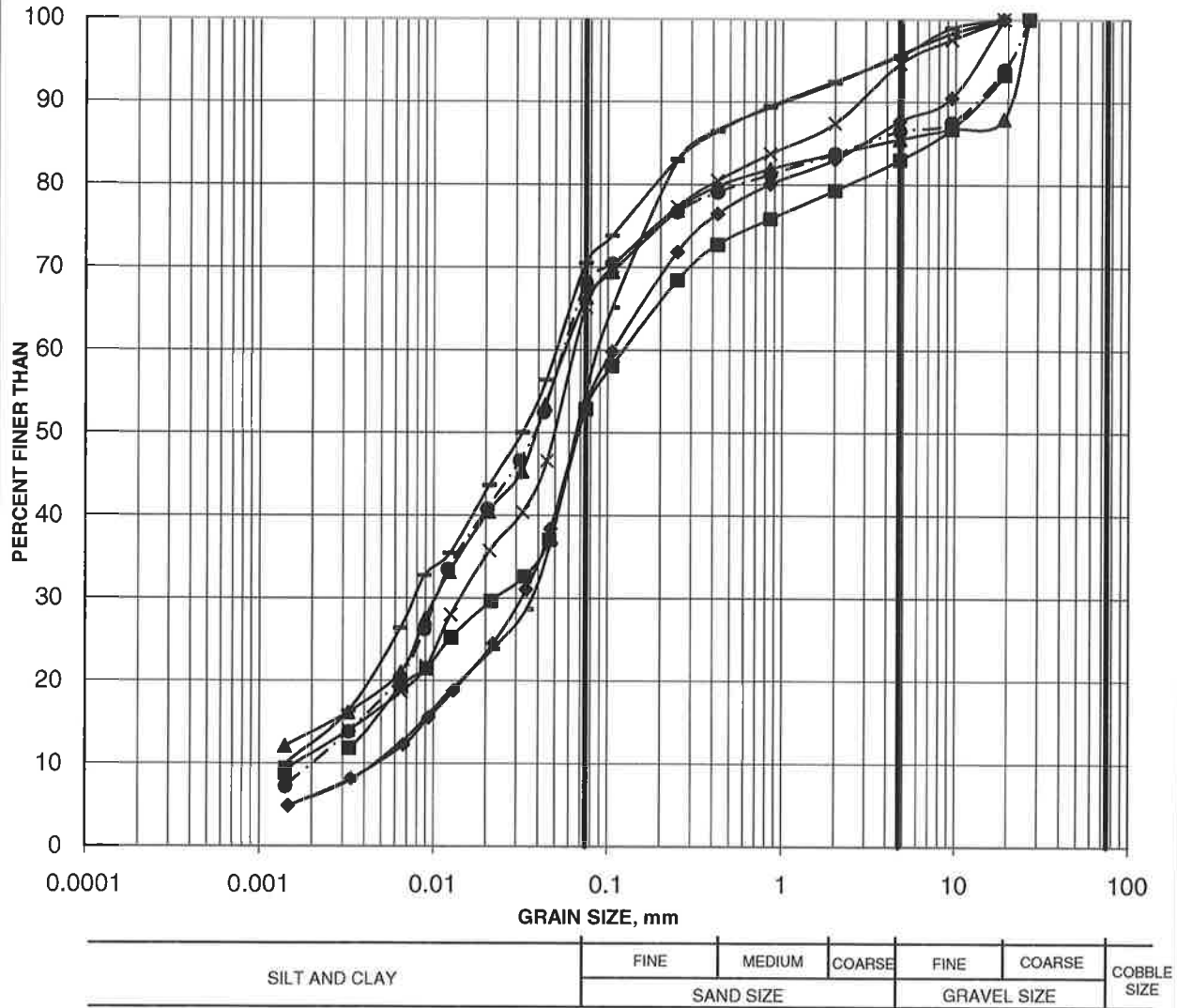


Borehole	Sample	Depth (m)
08-8	9	10.67-11.28
08-12A	12	16.77-17.01
08-15	12	13.72-14.33
08-16	8	10.67-11.28
08-20	11	6.10-6.71

GRAIN SIZE DISTRIBUTION

FIGURE 10

SANDY SILT TILL

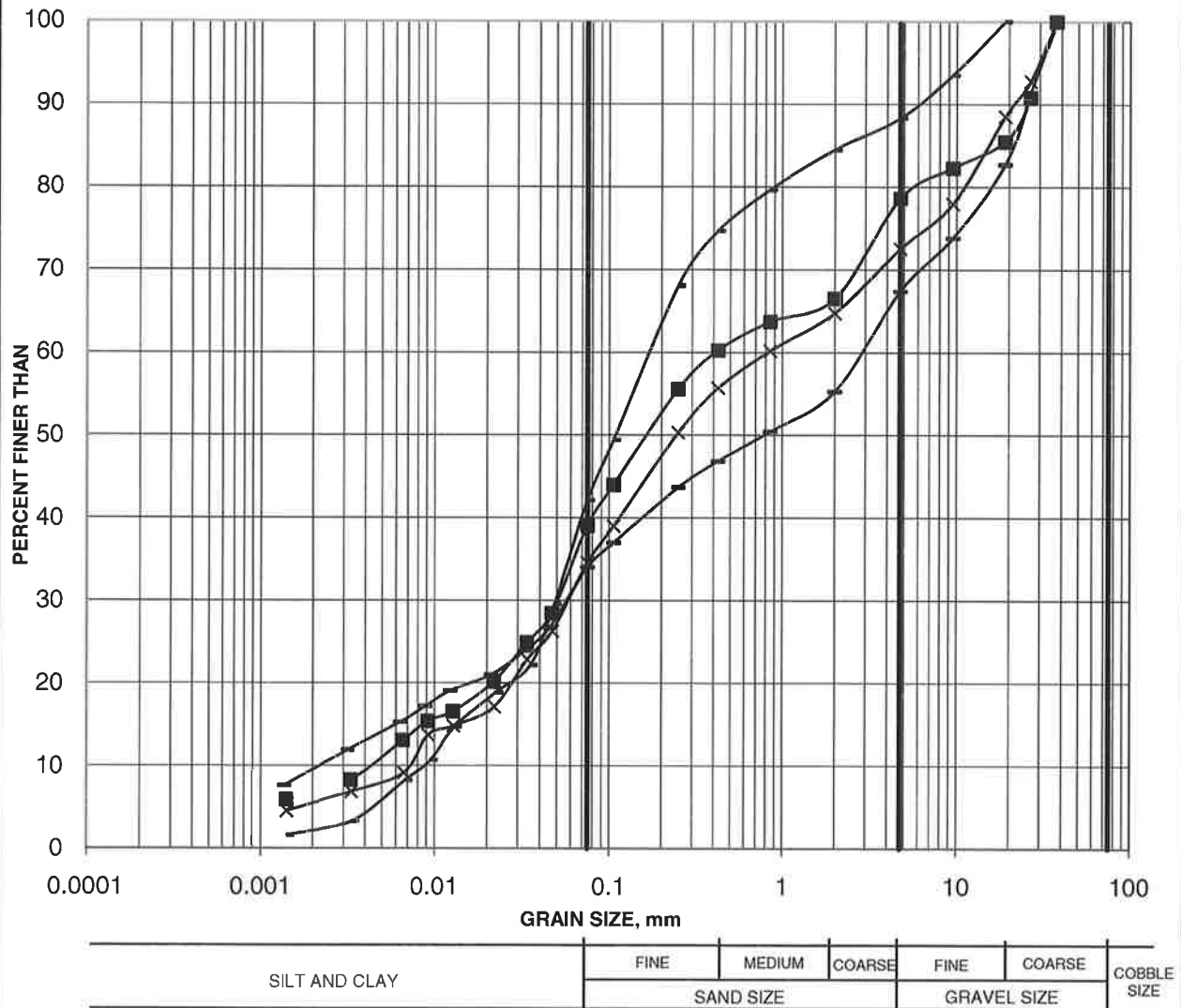


Borehole	Sample	Depth (m)
08-1A	14	18.29-18.73
08-3A	8	9.15-9.76
08-3A	11	13.72-14.33
08-4A	11B	15.40-15.68
08-6	8	4.42-4.88
08-10A	11	15.24-15.85
08-11A	7	10.67-11.28

GRAIN SIZE DISTRIBUTION

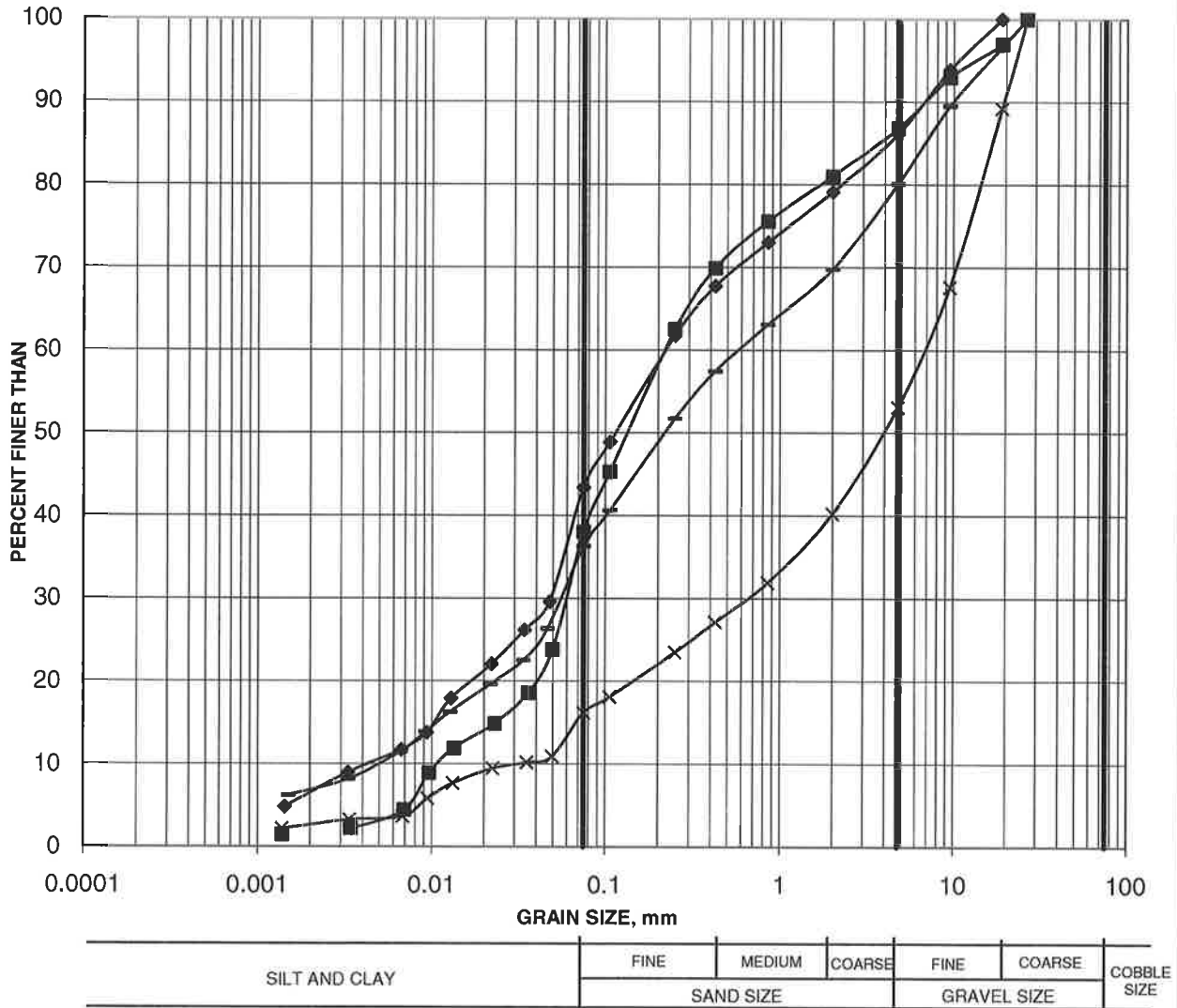
FIGURE 11

SILTY SAND TILL



Borehole	Sample	Depth (m)
08-4A	9	12.20-12.80
08-7	10	13.72-14.33
08-8	6	6.89-7.32
08-9A	11	12.80-13.41

SILTY SAND TILL

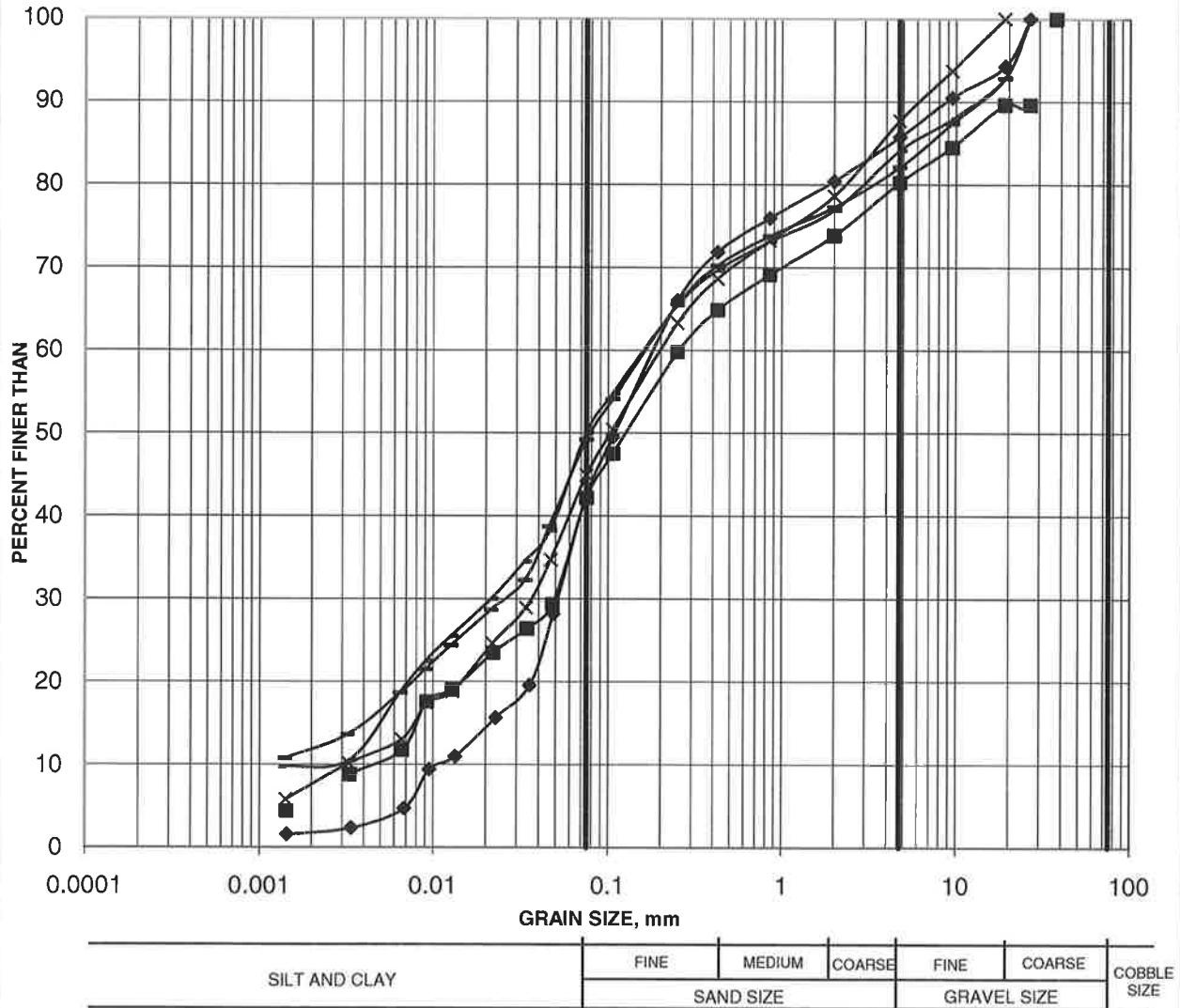


Borehole	Sample	Depth (m)
08-13	7	3.05-3.66
08-13	8B	3.96-4.27
08-16	5	6.10-6.71
08-20	9	4.88-5.49

GRAIN SIZE DISTRIBUTION

FIGURE 13

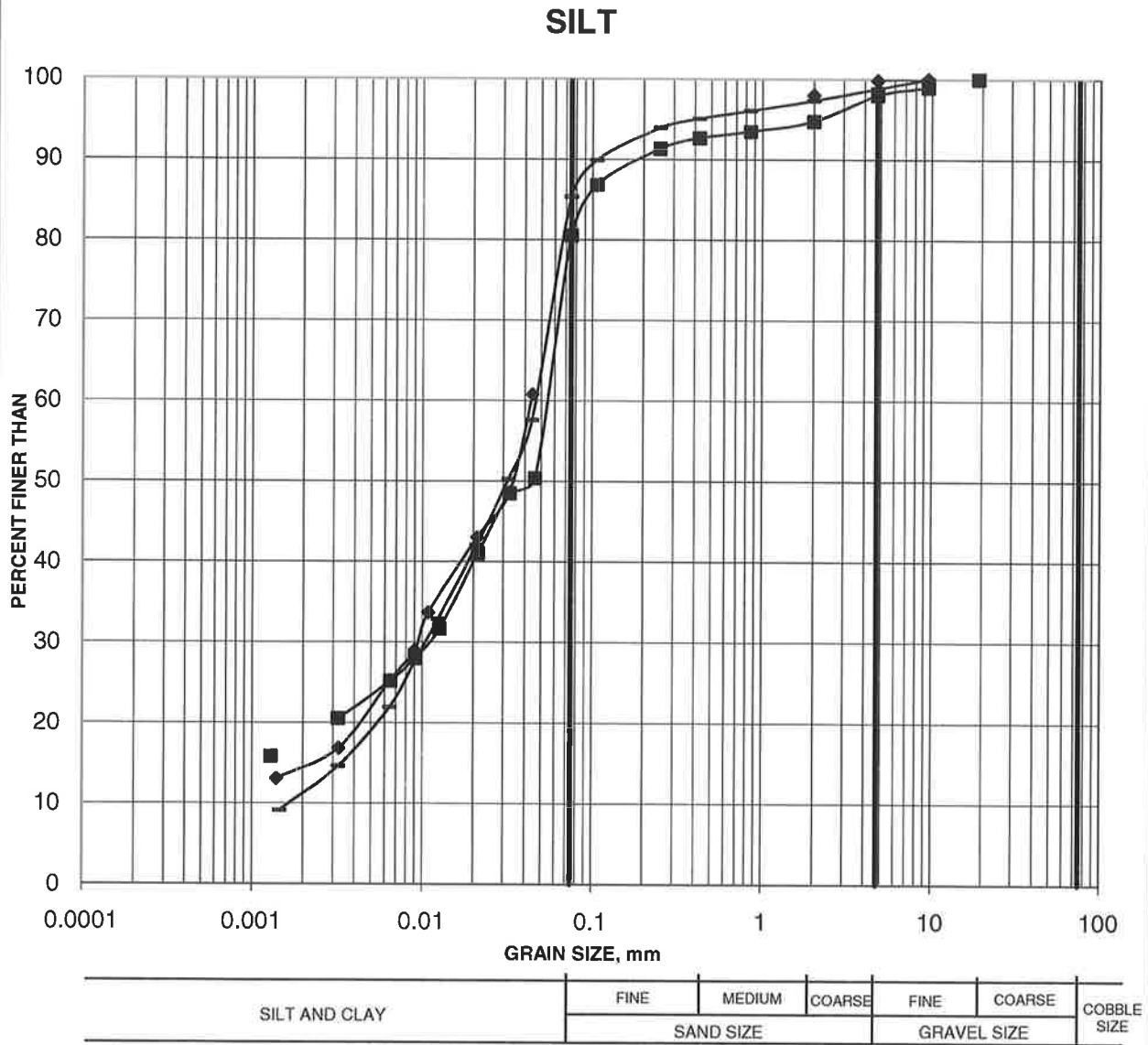
SILTY SAND TO SANDY SILT TILL



Borehole	Sample	Depth (m)
08-1A	12	15.24-15.85
08-2A	11	13.72-14.33
08-2A	13	16.77-17.38
08-3A	13	16.77-17.00
08-12A	7	9.15-9.76

GRAIN SIZE DISTRIBUTION

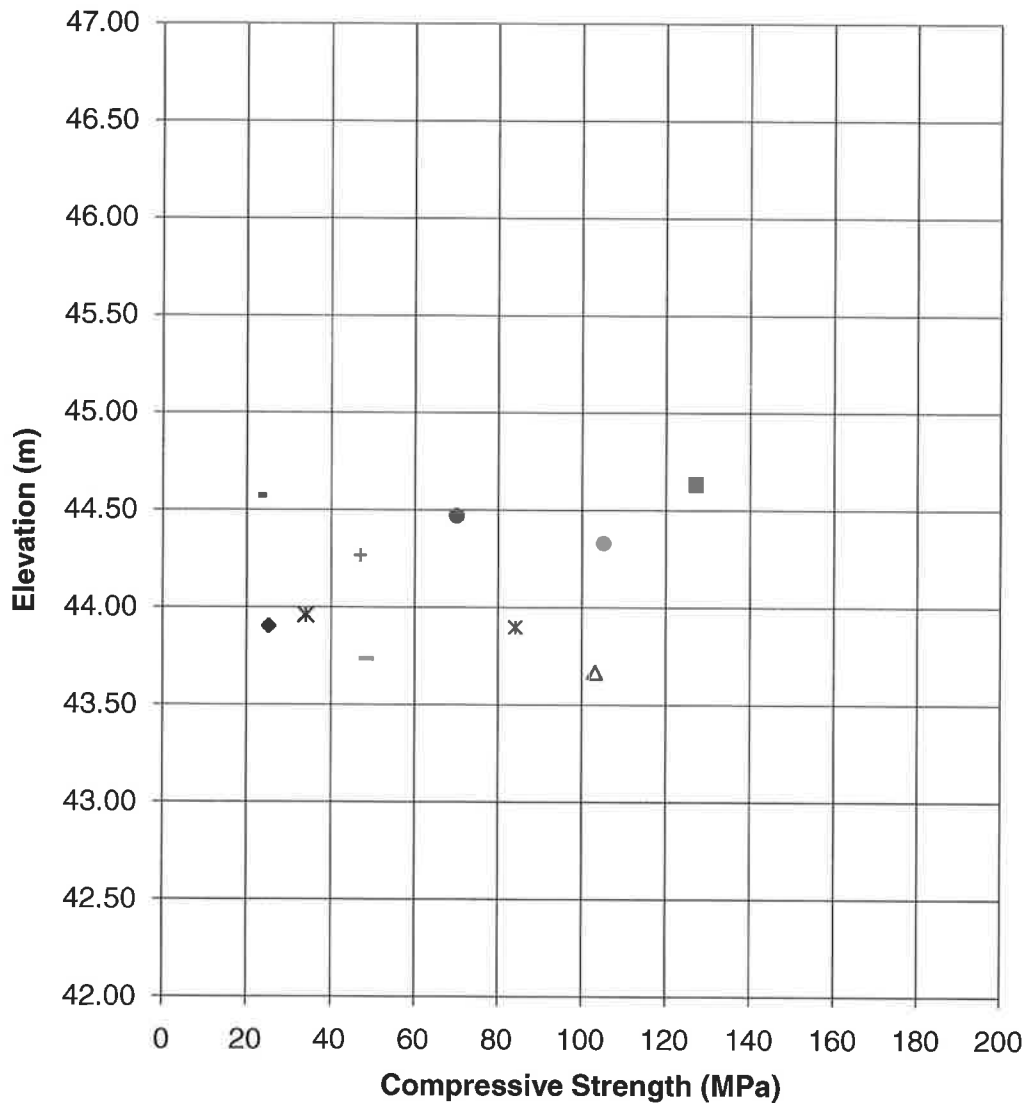
FIGURE 14



Borehole	Sample	Depth (m)
08-9A	15	18.29-18.90
08-11A	9	13.72-14.33
08-11A	12	18.29-18.88

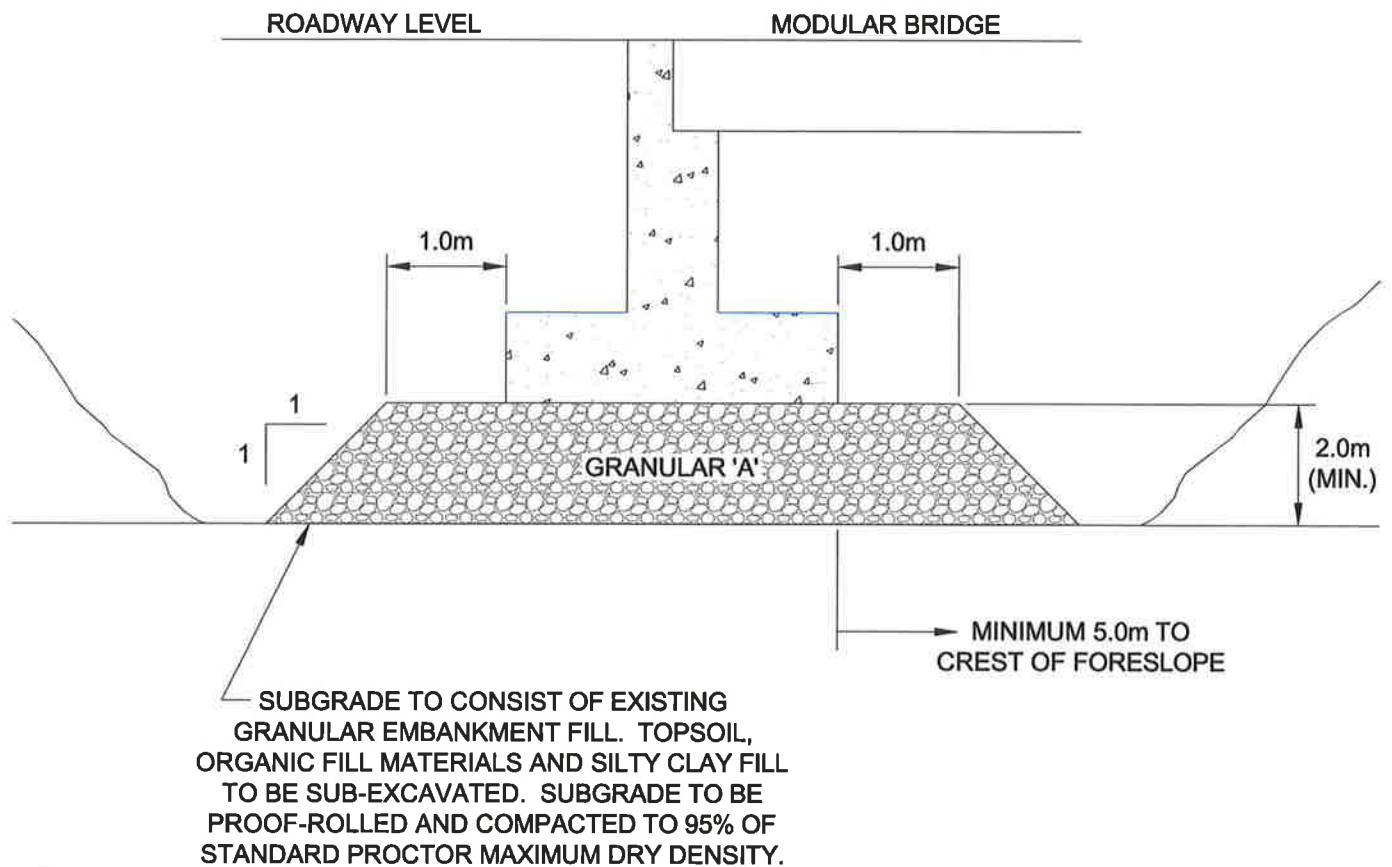
SUMMARY OF LABORATORY COMPRESSIVE STRENGTH MEASUREMENTS

FIGURE 15



- ▲BH 08-1A Point Load Test
- BH 08-2A Point Load Test
- ✕BH 08-3A Point Load Test
- BH 08-4A Point Load Test
- ✕BH 08-9A Point Load Test
- BH 08-10A Point Load Test
- +BH 08-11A Point Load Test
- BH 08-12A Point Load Test
- BH 08-3A Unconfined Compression Test
- ◆BH 08-9A Unconfined Compression Test

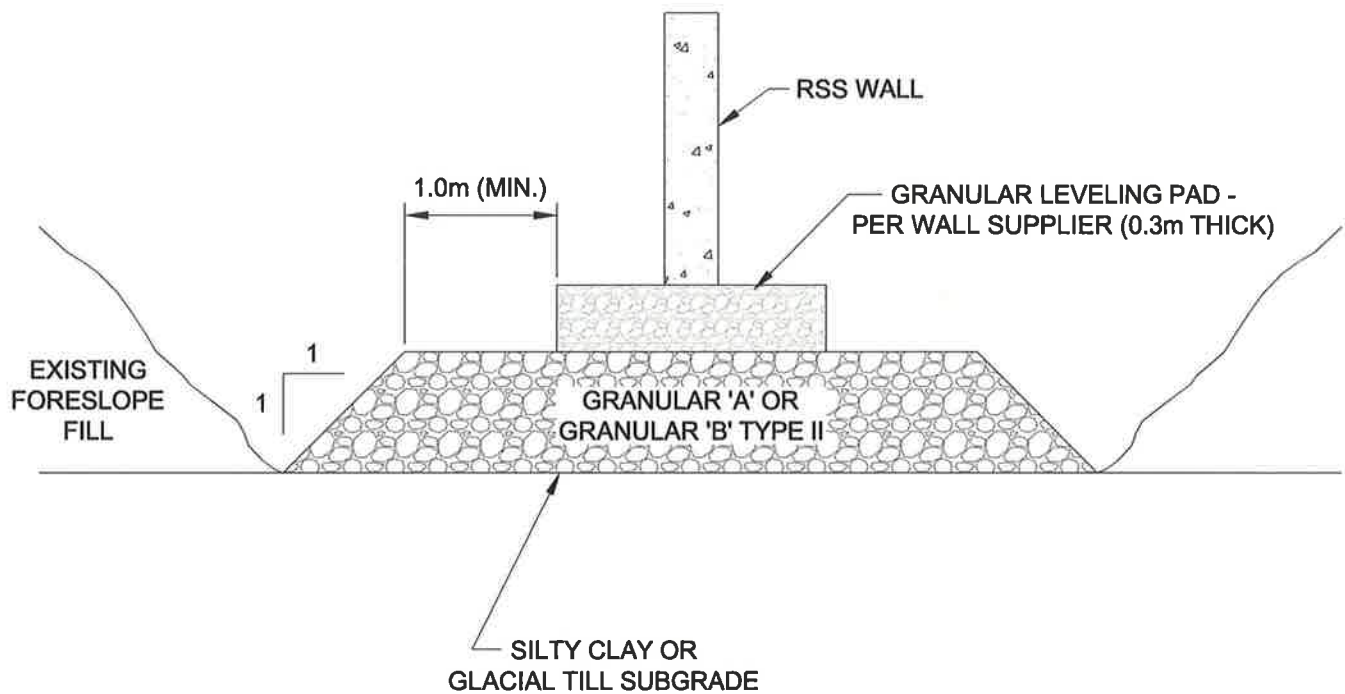
PLOT DATE: October 24, 2009
FILENAME: N:\Active\2008\1121 - Geotechnical\08-1121-0063 Genivar Hwy 401 Bridges Cornwall and Prescott\ACAD\Phase 2000\0811210063-2000-16.dwg



NOT TO SCALE



PLOT DATE: October 24, 2009
FILENAME: N:\Active\2008\1121 - Geotechnical\08-1121-0063 Genivar Hwy 401 Bridges Cornwall and Prescott\ACAD\Phase 2000\0811210063-2000-17.dwg



NOT TO SCALE



APPENDIX A

List of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Record of Borehole and Drillhole 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		III. SOIL DESCRIPTION					
AS	Auger sample	(a)	Cohesionless Soils				
BS	Block sample		Density Index (Relative Density)	N Blows/300 mm Or Blows/ft.			
CS	Chunk sample						
DO	Drive open						
DS	Denison type sample						
FS	Foil sample						
RC	Rock core		Very loose	0 to 4			
SC	Soil core		Loose	4 to 10			
ST	Slotted tube		Compact	10 to 30			
TO	Thin-walled, open		Dense	30 to 50			
TP	Thin-walled, piston		Very dense	over 50			
WS	Wash sample	(b)	Cohesive Soils				
DT	Dual Tube sample		Consistency	C _u or S _u			
II. PENETRATION RESISTANCE							
Standard Penetration Resistance (SPT), N:					Kpa	Psf	
The number of blows by a 63.5 kg. (140 lb.)					Very soft	0 to 12	0 to 250
hammer dropped 760 mm (30 in.) required					Soft	12 to 25	250 to 500
to drive a 50 mm (2 in.) drive open					Firm	25 to 50	500 to 1,000
Sampler for a distance of 300 mm (12 in.)					Stiff	50 to 100	1,000 to 2,000
DD- Diamond Drilling					Very stiff	100 to 200	2,000 to 4,000
Dynamic Penetration Resistance; N _d :					Hard	Over 200	Over 4,000
The number of blows by a 63.5 kg (140 lb.)					IV. SOIL TESTS		
hammer dropped 760 mm (30 in.) to drive					w	water content	
Uncased a 50 mm (2 in.) diameter, 60 ⁰ cone					w _p	plastic limited	
attached to “A” size drill rods for a distance					w _l	liquid limit	
of 300 mm (12 in.).					C	consolidaiton (oedometer) test	
PH:	Sampler advanced by hydraulic pressure				CHEM	chemical analysis (refer to text)	
PM:	Sampler advanced by manual pressure				CID	consolidated isotropically drained triaxial test ¹	
WH:	Sampler advanced by static weight of hammer				CIU	consolidated isotropically undrained triaxial test	
WR:	Sampler advanced by weight of sampler and rod					with porewater pressure measurement ¹	
Peizo-Cone Penetration Test (CPT):		D _R			relative density (specific gravity, G _s)		
An electronic cone penetrometer with		DS			direct shear test		
a 60 ⁰ conical tip and a projected end area		M			sieve analysis for particle size		
of 10 cm ² pushed through ground		MH			combined sieve and hydrometer (H) analysis		
at a penetration rate of 2 cm/s. Measurements		MPC			modified Proctor compaction test		
of tip resistance (Q _t), porewater pressure		SPC			standard Proctor compaction test		
(PWP) and friction along a sleeve are recorded		OC			organic content test		
Electronically at 25 mm penetration intervals.		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
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 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 ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
$*$	Density symbol is p . Unit weight symbol is γ where $\gamma = pg$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_L	liquid limit
w_p	plastic limit
I_p	plasticity Index = $(w - w_p)/I_p$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p)/I_p$
I_c	consistency index = $(w - w_p)/I_p$
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_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	Overconsolidation ratio = σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p \tau_r$	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_i	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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

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

GRAIN SIZE

<u>Term</u>	<u>Size*</u>
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.

O:\ Templates\Rock Description Terminology

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 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 90⁰ 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	Ca-	Calcite
FO-	Foliation/Schistosity	P-	Polished
CL -	Cleavage	S-	Slickensided
SH -	Shear Plane/Zone	SM-	Smooth
VN-	Vein	R-	Ridged/Rough
F -	Fault	ST-	Stepped
CO-	Contact	PL-	Planar
J -	Joint	FL-	Flexured
FR-	Fracture	UE-	Uneven
MF -	Mechanical	W-	Wavy
A-	Angular	C-	Curved
BP-	Bedding Plane	H-	Hackly
BL-	Blast Induced	SL-	Sludge Coated
	Parallel To	TCA-	To Core Axis
	Perpendicular To	STR-	Stress Induced

PROJECT 08-1121-0063		RECORD OF BOREHOLE No AH 09-1		1 OF 1 METRIC												
G.W.P. 237-00-00		LOCATION N 4992380.7 E 213525.5		ORIGINATED BY R.I.												
DIST HWY 401		BOREHOLE TYPE Portable Hand Auger		COMPILED BY J.M.												
DATUM Geodetic		DATE Sept. 16, 2009		CHECKED BY												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			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	
54.1	GROUND SURFACE						54	20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L			25 50 75 W _p W W _L	20 40 60 80 100 25 50 75	
0.0	PEAT						54									
0.2	Organic silty clay, trace sand and gravel, with cobbles and boulders (Possible FILL)		1	A.S.			54									
53.5	Dark grey-brown to black SILTY CLAY, occasional sand seam (Weathered Crust)		2	A.S.			54									
0.6	Grey-brown		3	A.S.			53									
52.1							53									
2.0	SILTY CLAY, trace sand and gravel Very stiff to stiff Grey		4	A.S.			52									
							51									
			5	A.S.			51									
							50									
49.1			6	A.S.			50									
5.0	End of Borehole															
Note: 1. Water level in open borehole at 0.1 m depth (Elev. 54.0 m) upon completion of drilling on Sept. 16, 2009. 2. Elevation approximate.																

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

Continued Next Page

+³, ×³ Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT 08-1121-0063-2000		RECORD OF BOREHOLE No 08-1A				2 OF 2		METRIC							
G.W.P. 237-00-00		LOCATION N 4992408.2; E 213528.5				ORIGINATED BY D.G.									
DIST HWY 401		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem				COMPILED BY J.M.									
DATUM Geodetic		DATE June 16, 2008				CHECKED BY K.L.									
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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
	— CONTINUED FROM PREVIOUS PAGE —							20 40 60 80 100							
	Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Compact to very dense Grey Wet		12	SS	16		50								20 38 36 6
							49								
			13	SS	>100		48								
							47								
			14	SS	>100		46								
45.2			15	SS	>50		45								
20.0	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong Bedrock cored between 20.0 m and 23.6 m depth. For bedrock coring details refer to Record of Drillhole 08-1A.		16	NQ RC	REC 100%		44								RQD = 0%
			17	NQ RC	REC 100%		43								RQD = 25%
			18	NQ RC	REC 57%		42								RQD = 0%
			19	NQ RC	REC 100%										RQD = 63%
			20	NQ RC	REC 100%										RQD = 92%
			21	NQ RC	REC 100%										RQD = 83%
41.6	End of Borehole														
23.6	Note: Water level in open borehole at 14.0 m depth (Elev. 51.2) upon completion of drilling on June 17, 2008														

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

PROJECT: 08-1121-0063-2000

RECORD OF DRILLHOLE: 08-1A

SHEET 1 OF 1

LOCATION: N 4992408.2; E 213528.5

DRILLING DATE: June 16, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No	PENETRATION RATE (mm/min)	FLUSH % RETURN	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POST LOG INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)	ELEV.					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					
TOTAL CORE %	SOLID CORE %	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³														
20	Rotary Drill NQ Core	Continued from Record of Borehole 08-1A		45.20																	
		Interbedded Shale and fossiliferous		20.00	1																
		Limestone (BEDROCK)			2																
21		Fresh			3																
		Grey			4																
		Thinly to medium bedded			5																
22		Medium strong			6																
23																					
24		End of Drillhole		41.60																	
				23.60																	
25																					
26																					
27																					
28																					
29																					
30																					
31																					
32																					
33																					
34																					
35																					

DEPTH SCALE

1 : 75



LOGGED: D.G.

CHECKED: K.L.

MIS-RCK-001_0811210063-2000 (ROCK) GPJ GAL-MISS GDT 10/24/09 JM

RECORD OF BOREHOLE No 08-2A

1 OF 2 **METRIC**

PROJECT 08-1121-0063-2000

G.W.P. 237-00-00

LOCATION N 4992419.9; E 213531.4

ORIGINATED BY D.J.S.

DIST HWY 401

BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem

COMPILED BY J.M.

DATUM Geodetic

DATE June 24, 2008

CHECKED BY K.L.



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 (%)		
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					25 50 75					
65.2	GROUND SURFACE						65										
0.0	TOPSOIL						64										
0.1	Sand and gravel (FILL)						63										
	Brown		1	SS	78		62										
							61										
61.5			2	SS	43		60										
3.7	Silty sand, some gravel, trace clay, with cobbles (FILL)						59										
	Compact to very dense						58										
	Brown and dark brown						57										
	Moist to wet						56										
			3	SS	49		55										
							54										
			4	SS	27		53										
							52										
			5	SS	37		51										
			6	SS	29												
54.4																	
11.0	Silty clay (TOPSOIL)		7	SS	17												
53.8	Dark grey																
11.4	Moist																
	SILTY CLAY (Weathered Crust)																
	Very stiff		8	SS	25												
	Grey-green to grey-brown																
	Moist																
	SILTY CLAY, occasional sand seam with depth (Weathered Crust)		9	SS	9												
	Very stiff																
	Grey-brown																
	Moist to wet																
51.9			10	SS	16												
13.3	Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL)																
	Compact to very dense		11	SS	27												
	Brown to grey																
	Wet																

Continued Next Page

+ 3, × 3

Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT 08-1121-0063-2000			RECORD OF BOREHOLE No 08-2A			2 OF 2 METRIC									
G.W.P. 237-00-00			LOCATION N 4992419.9; E 213531.4			ORIGINATED BY D.J.S.									
DIST HWY 401			BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem			COMPILED BY J.M.									
DATUM Geodetic			DATE June 24, 2008			CHECKED BY K.L.									
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							
--- CONTINUED FROM PREVIOUS PAGE ---															
45.3	Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Compact to very dense Brown to grey Wet		12	SS	58										
			13	SS	58										
			14	NQ RC	DD										
15	NQ RC	DD													
19.9	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong Bedrock cored between 19.9 m and 23.0 m depth. For bedrock coring details refer to Record of Drillhole 08-2A.		16	NQ RC	REC 100%										
17			NQ RC	REC 96%											
18			NQ RC	REC 100%											
19			NQ RC	REC 100%											
42.2	End of Borehole														
23.0	Note: Water level in open borehole at 8.1 m depth (Elev. 57.1) upon completion of drilling on June 25, 2008														

PROJECT: 08-1121-0063-2000

RECORD OF DRILLHOLE: 08-2A

SHEET 1 OF 1

LOCATION: N 4992419.9; E 213531.4


DRILLING DATE: June 24, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

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 %	DIP w.r.t. CORE AXIS			TYPE AND SURFACE DESCRIPTION	10 ⁻⁶		10 ⁻⁵	10 ⁻⁴	10 ⁻³											
		Continued from Record of Borehole 08-2A		45.30 19.00																										
20	Rotary Drill NO Core	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong			1																									
21					2																									
22					3																									
23					4																									
23		End of Drillhole		42.20 23.00																										
24																														
25																														
26																														
27																														
28																														
29																														
30																														
31																														
32																														
33																														
34																														

DEPTH SCALE

1 : 75



LOGGED: D.J.S.

CHECKED: K.L.

MIS-RCK-001_0811210063-2000 (ROCK) GPJ GAL-MISS GDT_10/24/09 JIM

PROJECT 08-1121-0063-2000		RECORD OF BOREHOLE No 08-3A		1 OF 2 METRIC									
G.W.P. 237-00-00		LOCATION N 4992439.3; E 213610.7		ORIGINATED BY H.M.									
DIST _____ HWY 401		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem		COMPILED BY J.M.									
DATUM Geodetic		DATE June 24, 2008		CHECKED BY K.L.									
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES						SHEAR STRENGTH kPa
64.2	GROUND SURFACE						20 40 60 80 100						
64.0	TOPSOIL						20 40 60 80 100						
61.6	Sand and gravel, trace silt, with cobbles and boulders (FILL) Compact Brown Moist		1	A.S.								43 43 (14)	
			2	SS	20								
61.6	Silty sand, some gravel, trace clay and topsoil (FILL) Very loose to loose Brown to greenish brown Moist		3	SS	6								
			4	SS	2							9 49 33 9	
59.1	Sandy SILT to Silty SAND, some gravel, trace to some clay, with cobbles and boulders (TILL) Compact to very dense Brown to grey Moist to wet		5	SS	28								
			6	SS	54								
			7	SS	54								
			8	SS	32							4 30 54 12	
			9	SS	46								
			10	SS	>66								
			11	SS	32							4 41 49 6	

Continued Next Page

+ 3, x 3

Numbers refer to Sensitivity



○ 3% STRAIN AT FAILURE

PROJECT		RECORD OF BOREHOLE No 08-3A				2 OF 2 METRIC								
08-1121-0063-2000														
G.W.P. 237-00-00		LOCATION N 4992439.3; E 213610.7				ORIGINATED BY H.M.								
DIST HWY 401		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem				COMPILED BY J.M.								
DATUM Geodetic		DATE June 24, 2008				CHECKED BY K.L.								
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						
--- CONTINUED FROM PREVIOUS PAGE ---														
45.5	Sandy SILT to Silty SAND, some gravel, trace to some clay, with cobbles and boulders (TILL) Compact to very dense Brown to grey Moist to wet		12	SS	>50		49							
			13	SS	>50		48							
			14	SS	>50		47							
18.7	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong Bedrock cored between 18.7 m and 22.4 m depth. For bedrock coring details refer to Record of Drillhole 08-3A.		15	NQ RC	REC 83%		45							RQD = 0%
			16	NQ RC	REC 74%		44							RQD = 41%
			17	NQ RC	REC 100%		43							RQD = 81%
			18	NQ RC	REC 100%		42							RQD = 58%
41.8			19	NQ RC	REC 100%									RQD = 92%
22.4	End of Borehole Note: Water level in well screen at 7.5 m depth (Elev. 56.7) on Aug. 20, 2009.													

MIS-MTO 001 0811210063-2000 GPJ GAL-MASS GDT 10/24/09

Continued Next Page

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

PROJECT 08-1121-0063-2000		RECORD OF BOREHOLE No 08-4A		2 OF 2 METRIC															
G.W.P. 237-00-00		LOCATION N 4992423.4; E 213595.0		ORIGINATED BY D.G.															
DIST HWY 401		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem		COMPILED BY J.M.															
DATUM Geodetic		DATE June 25, 2008		CHECKED BY K.L.															
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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			WATER CONTENT (%) W _P W W _L			γ kN/m ³	GR	SA	SI	CL	
49.4	— CONTINUED FROM PREVIOUS PAGE —																		
15.4	SAND Grey Wet Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Compact to very dense Grey Wet		11	SS	>100		49									4	25	58	13
			12	SS	20		48												
			13	SS	>100		47												
44.9							46												
19.7	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong Bedrock cored between 19.7 m and 22.9 m depth. For bedrock coring details refer to Record of Drillhole 08-4A.		14	NQ RC	REC 81%		45												RQD = 26%
			15	NQ RC	REC 100%		44												RQD = 35%
			16	NQ RC	REC 100%		43												RQD = 46%
			17	NQ RC	REC 100%		42												RQD = 76%
41.7																			
22.9	End of Borehole Note: Water level in open borehole at 9.2 m depth (Elev. 55.4) upon completion of drilling on June 26, 2008																		

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Marathon Drilling Co. Ltd.

MIS-RCK 001 0811210053-2000 (ROCK) GPJ GAL-MISS GDT 10/24/09 JM

CHECKED: K.L.

PROJECT		08-1121-0063-2000		RECORD OF BOREHOLE No 08-5		1 OF 1 METRIC													
G.W.P.		237-00-00		LOCATION		N 4992409.0; E 213544.3													
DIST		HWY 401		BOREHOLE TYPE		Portable Drill													
DATUM		Geodetic		DATE		Sept. 4, 2008													
						ORIGINATED BY D.G./K.L.													
						COMPILED BY J.M.													
						CHECKED BY M.I.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		
58.6 0.0	GROUND SURFACE Sand and gravel, with cobbles, boulders and blast rock (FILL) Brown Dry																		

PROJECT		08-1121-0063-2000		RECORD OF BOREHOLE No 08-6		1 OF 1 METRIC									
G.W.P.		237-00-00		LOCATION		N 4992416.5; E 213580.3									
DIST		HWY 401		BOREHOLE TYPE		Portable Drill									
DATUM		Geodetic		DATE		July 17, 2008									
				ORIGINATED BY		D.G.									
				COMPILED BY		J.M.									
				CHECKED BY		K.L.									
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 (%)						
60.0 0.0	GROUND SURFACE Sand and gravel, with cobbles and boulders (FILL) Brown Moist	STRAT PLOT	1	SS	8										
58.8 1.2	Silty sand, some gravel, trace clay (FILL) Loose Brown		2	SS	3										7 36 32 25
57.6 2.4	Silty clay with organic matter (FILL) Dark grey Sandy silt to silty sand, some gravel, trace clay (FILL) Compact Brown		3	SS	8										
			4	SS	PM										
			5	SS	11										12 36 39 13
56.0 55.7	SILTY CLAY (Weathered Crust) Grey-brown		6	SS	7										
4.3	Sandy SILT to Silty SAND, some gravel, trace clay (TILL)		7	SS											
55.1 4.9	Very dense Brown End of Borehole		8	SS	>100										12 35 47 6

PROJECT		08-1121-0063-2000		RECORD OF BOREHOLE No 08-7		1 OF 2 METRIC							
G.W.P.		237-00-00		LOCATION		N 4992417.0; E 213517.8							
DIST		HWY 401		BOREHOLE TYPE		Power Auger 108mm I.D. Hollow Stem							
DATUM		Geodetic		DATE		June 26, 2008							
						ORIGINATED BY J.J.							
						COMPILED BY J.M.							
						CHECKED BY K.L.							
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	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL	
65.1	GROUND SURFACE						65						
0.0	TOPSOIL												
0.1	Sand and gravel, trace silt (FILL) Compact Brown Moist		1	SS	22		64						
63.0							63						
2.1	Silty sand, some gravel, trace clay, with cobbles and boulders (FILL) Compact to very dense Grey-brown Moist		2	SS	26		62					22 39 31 8	
							61						
			3	SS	>70		60						
							59						
			4	SS	>50		58						
							57						
			5	SS	80		56					27 37 29 7	
			6	SS	65		55						
54.4							54						
10.8	Silty clay (TOPSOIL) Moist SILTY CLAY, occasional sand seam (Weathered Crust) Very stiff Dark grey to grey-brown Moist		7	SS	17		53						
			8	SS	11		52						
			9	SS	12		51						
52.4													
12.7	Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Dense Grey Wet		10	SS	33							28 37 30 5	
50.8													
14.3													

Continued Next Page

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

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

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

PROJECT 08-1121-0063-2000		RECORD OF BOREHOLE No 08-8		1 OF 1 METRIC	
G.W.P. 237-00-00		LOCATION N 4992442.4; E 213625.6		ORIGINATED BY H.M.	
DIST HWY 401		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem		COMPILED BY J.M.	
DATUM Geodetic		DATE July 2, 2008		CHECKED BY K.L.	

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT LIMIT CONTENT CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W _p W W _L	25 50 75		
64.0	GROUND SURFACE												
0.0	Silty sand, some gravel, trace clay, with cobbles (FILL) Compact Brown Moist		1	A.S.									25 46 22 7
			2	SS	24								
			3	SS	22								
			4	SS	21								
58.8													
5.2	Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Very dense Brown to grey Moist to wet		5	SS	>50								11 47 39 3
			6	SS	77								
			7	SS	86								
			8	SS	62								
			9	SS	49								7 37 49 7
52.7													
11.3	End of Borehole												
	Note: Water level in open borehole at 8.8 m depth (Elev. 55.2) upon completion of drilling on July 2, 2008												

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

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

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT		RECORD OF BOREHOLE No 08-9A				2 OF 2 METRIC							
08-1121-0063-2000													
G.W.P. 237-00-00		LOCATION		N 4992433.8; E 213542.8		ORIGINATED BY D.J.S.							
DIST HWY 401		BOREHOLE TYPE		Power Auger 108mm I.D. Hollow Stem		COMPILED BY J.M.							
DATUM Geodetic		DATE		June 16, 2008		CHECKED BY K.L.							
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
	--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100	○ UNCONFINED + FIELD VANE	W _P W W _L			
								20 40 60 80 100	● QUICK TRIAXIAL × REMOULDED		25 50 75	kN/m ³	
46.3	Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles (TILL) Compact to very dense Brown to grey Wet		13	SS	24		49						
			14	SS	25		48						
							47						
18.4	Layered Sandy SILT and CLAYEY SILT, occasional silty sand seam Very dense Grey Wet		15	SS	63		46						2 17 63 18
19.2	Sandy SILT to Silty SAND, some gravel, clay and cobbles (TILL) Very dense Grey Wet						45						
20.1	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong Bedrock cored between 20.1 m and 23.6 m depth. For bedrock coring details refer to Record of Drillhole 08-9A.		17	NQ RC	REC 100%		44						RQD = 68%
			18	NQ RC	REC 100%								RQD = 0%
			19	NQ RC	REC 100%		43						RQD = 82%
			20	NQ RC	REC 100%								RQD = 0%
			21	NQ RC	REC 100%		42						RQD = 89%
41.0	End of Borehole												
23.6	Note: Water level in well screen at 10.2 m depth (Elev. 54.4) on Aug. 20, 2009.												

PROJECT: 08-1121-0063-2000

RECORD OF DRILLHOLE: 08-9A

SHEET 1 OF 1

LOCATION: N 4992433.8; E 213542.8


DRILLING DATE: June 16, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling Co, Ltd.

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.O.D. %	FRACT INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec																				
TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION		10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³																	
		Continued from Record of Borehole 08-9A		44.50																						
21	Rotary Drill NQ Core	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong		20.10																						
				1																						
				2																						
22				3																						
				4																						
23																										
				41.00																						
24		End of Drillhole		23.00																						
25		Note: Water level in well screen at 10.2 m depth (Elev. 54.4) on Aug. 20, 2009.																								
26																										
27																										
28																										
29																										
30																										
31																										
32																										
33																										
34																										
35																										

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

Continued Next Page

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

PROJECT 08-1121-0063-2000

RECORD OF BOREHOLE No 08-10A

2 OF 2 **METRIC**

G.W.P. 237-00-00

LOCATION N 4992447.5; E 213553.9

ORIGINATED BY P.A.H.

DIST HWY 401

BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem

COMPILED BY J.M.

DATUM Geodetic

DATE July 2, 2008

CHECKED BY K.L.

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							WATER CONTENT (%)
--- CONTINUED FROM PREVIOUS PAGE ---															
45.4	Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles and boulders, occasional silt layer (TILL) Loose to very dense with depth Grey Wet		11	SS	33		49							13 19 58 10	
19.5	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong Bedrock cored between 19.5 m and 22.6 m depth. For bedrock coring details refer to Record of Drillhole 08-10A.		15	NQ RC	REC 94%		45							RQD = 0%	
															RQD = 45%
42.3			17	NQ RC	REC 100%		43							RQD = 72%	
22.6	End of Borehole														
	Note: Water level in open borehole at 10.7 m depth (Elev. 54.2) upon completion of drilling on July 3, 2008														

MIS-MTO 001 0811210063-2000.GPJ GAL-MISS GDT 10/24/08

PROJECT: 08-1121-0063-2000

RECORD OF DRILLHOLE: 08-10A

SHEET 1 OF 1

LOCATION: N 4992447.5; E 213553.9

DRILLING DATE: July 2, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling Co., Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN NO.	PENETRATION RATE (mm/min)	FLUSH % RETURN	FR/FX-FRACTURE F-FAULT CL-CLEAVAGE J-JOINT SH-SHEAR P-POLISHED VN-VEIN S-SLICKENSIDED PL-PLANAR	SM-SMOOTH R-ROUGH ST-STEPPED C-CURVED	FL-FLEXURED UE-UNEVEN W-WAVY	BC-BROKEN CORE MB-MECH. BREAK B-BEDDING	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole 08-10A		45.40									
		Interbedded Shale and fossiliferous Limestone (BEDROCK)		19.50	1								
20		Fresh Grey											
		Thinly to medium bedded			2								
21		Medium strong											
22					3								
		End of Drillhole		42.30									
23				22.60									
24													
25													
26													
27													
28													
29													
30													
31													
32													
33													
34													

DEPTH SCALE

1 : 75



LOGGED: P.A.H.

CHECKED: K.L.

MIS-RCK-001_0811210063-2000 (ROCK) GPJ GAL-MISS GDT 10/24/08 JM

PROJECT <u>08-1121-0063-2000</u>		RECORD OF BOREHOLE No 08-11A		1 OF 2 METRIC	
G.W.P. <u>237-00-00</u>		LOCATION <u>N 4992463.9; E 213624.5</u>		ORIGINATED BY <u>D.G.</u>	
DIST <u>HWY 401</u>		BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem</u>		COMPILED BY <u>J.M.</u>	
DATUM <u>Geodetic</u>		DATE <u>July 2, 2008</u>		CHECKED BY <u>K.L.</u>	

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	W _p	W	W _L	GR		
64.1	GROUND SURFACE													
0.0	ASPHALTIC CONCRETE													
0.3	CONCRETE													
63.3	Crushed stone (FILL)													
	Grey													
0.8	Sand and gravel, trace to some silt, trace clay (FILL)													
	Loose to dense													
	Brown													
	Moist													
			1	SS	9									40 40 17 3
			2	SS	34									
59.5	Silty sand, some gravel, trace clay (FILL)													
4.6	Compact to dense													
	Grey-brown to grey													
	Moist													
			3	SS	46									17 40 35 8
			4	SS	14									
56.5	Sandy SILT to Silty SAND, some gravel, trace to some clay, with cobbles and boulders, occasional silt layer (TILL)													
7.6	Compact to dense													
	Brown to grey													
	Moist													
			5	SS	42									
			6	SS	45									
			7	SS	27									14 19 53 14
51.6	SILT, some clay, trace sand and gravel													
12.5	Dense													
	Grey													
	Wet													
			8	SS	50									
			9	SS	40									1 14 73 12

Continued Next Page

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

MIS-MTO 001 0811210063-2000.GPJ GAL-MISS.GDT 10/24/09

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

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

PROJECT: 08-1121-0063-2000

RECORD OF DRILLHOLE: 08-11A

SHEET 1 OF 1

LOCATION: N 4992463.9; E 213624.5

DRILLING DATE: July 2, 2008

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Marathon Drilling Co, Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (m/min)	FLUSH % RETURN	COLOUR (m/min)	F/F/F X-FRACTURE F-FAULT				SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL COR. LOSS INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)	ELEV.					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					
TOTAL CORE %	SOLID CORE %	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION																		
10	Rotary Drill NQ Core	Continued from Record of Borehole 08-11A		45.20																	
		Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong		18.90	1																
20				2																	
21				3																	
22		End of Drillhole		42.00																	
23				22.10																	
24																					
25																					
26																					
27																					
28																					
29																					
30																					
31																					
32																					
33																					

DEPTH SCALE

1 : 75



LOGGED: D.G.

CHECKED: K.L.

MIS-RCK 001 0811210063-2000 (ROCK) GPJ GAL-MISS GDT 10/24/09 JM



PROJECT <u>08-1121-0063-2000</u>		RECORD OF BOREHOLE No 08-12A		1 OF 2 METRIC	
G.W.P. <u>237-00-00</u>		LOCATION <u>N 4992451.4; E 213620.0</u>		ORIGINATED BY <u>J.D./H.D.</u>	
DIST <u>HWY 401</u>		BOREHOLE TYPE <u>Power Auger 108mm I.D. Hollow Stem</u>		COMPILED BY <u>J.M.</u>	
DATUM <u>Geodetic</u>		DATE <u>June 16, 2008</u>		CHECKED BY <u>K.L.</u>	

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	UNCONFINED	FIELD VANE					
63.7	GROUND SURFACE														
0.0	Sand and gravel, trace silt (FILL) Brown		1	A.S.			63								
			2	SS	13		62								43 39 15 3
61.6	Silty sand, some gravel, trace clay, with cobbles (FILL) Compact to very dense Brown Moist to wet						61								
			3	SS	59		60								
			4	SS	60		59								24 39 30 7
							58								
			5	SS	55		57								
			6	SS	49		56								
							55								
54.6	Sandy SILT to Silty SAND, some gravel, trace to some clay, with cobbles and boulders (TILL) Compact to very dense Grey-brown to grey Wet		7	SS	40		54								14 43 41 2
			8	SS	41		53								
							52								
			9	SS	32		51								
							50								
			10	SS	15		49								



Continued Next Page

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

MIS-MTO 001 0811210063-2000.GPJ GAL-MISS GDT 10/24/09

PROJECT 08-1121-0063-2000		RECORD OF BOREHOLE No 08-12A				2 OF 2 METRIC									
G.W.P. 237-00-00		LOCATION N 4992451.4; E 213620.0				ORIGINATED BY J.D./H.D.									
DIST HWY 401		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem				COMPILED BY J.M.									
DATUM Geodetic		DATE June 16, 2008				CHECKED BY K.L.									
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa							WATER CONTENT (%)
--- CONTINUED FROM PREVIOUS PAGE ---															
44.9 18.8	Sandy SILT to Silty SAND, some gravel, trace to some clay, with cobbles and boulders (TILL) Compact to very dense Grey-brown to grey Wet		11	SS	83										
			12	SS	>100										
			13	SS	>50										
			14	NQ RC	REC 87%										
41.6 22.1	Interbedded Shale and fossiliferous Limestone (BEDROCK) Fresh Grey Thinly to medium bedded Medium strong Bedrock cored between 18.8 m and 22.1 m depth. For bedrock coring details refer to Record of Drillhole 08-12A.		15	NQ RC	REC 96%										
			16	NQ RC	REC 100%										
			End of Borehole												
Note: Water level in open borehole at 8.8 m depth (Elev. 54.9) upon completion of drilling on June 16, 2008															

PROJECT		08-1121-0063-2000		RECORD OF BOREHOLE No 08-13		1 OF 1 METRIC									
G.W.P.		237-00-00		LOCATION		N 4992464.5; E 213610.1									
DIST		HWY 401		BOREHOLE TYPE		Portable Drill									
DATUM		Geodetic		DATE		July 18, 2008									
						ORIGINATED BY D.G.									
						COMPILED BY J.M.									
						CHECKED BY K.L.									
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
58.8 0.0	GROUND SURFACE Silty sand to sandy silt, some gravel, trace clay (FILL) Loose to compact Brown Moist	X	1	SS	7	▽	58								21 39 34 6
			2	SS	9		57								
			3	SS	8										
			4	SS	57										
			5	SS	>50										
			6	SS	>50		56								
55.8 3.1	Sandy SILT to Silty SAND, some gravel, trace clay (TILL) Very dense Brown to grey Wet		7	SS	55		55								27 29 37 7
54.5 4.3	End of Borehole Note: Water level in open borehole at 3.1 m depth (Elev. 55.7) upon completion of drilling on July 18, 2008		8	SS	>100										13 49 36 2
														47 36 14 3	

PROJECT 08-1121-0063-2000		RECORD OF BOREHOLE No 08-14		1 OF 1 METRIC																			
G.W.P. 237-00-00		LOCATION N 4992459.5; E 213579.0		ORIGINATED BY K.L.																			
DIST _____ HWY 401		BOREHOLE TYPE Portable Drill		COMPILED BY J.M.																			
DATUM Geodetic		DATE Sept. 10, 2008		CHECKED BY M.I.C.																			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			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																		
57.7	GROUND SURFACE																						
0.0	Layered silty clay, silty sand and sandy silt, trace to some gravel, trace organic matter and rootlets, with cobbles and boulders (FILL). Very loose to compact. Brown to grey.		1	SS	2																		
			2	SS	7																		
			3	SS	10																		
			4	SS	3																		
			5	SS	8																		
			6	SS	21																		
			7	SS	>100																		
			8	AW RC	DD																		
			9	AW RC	DD																		
52.8																							
4.9	Sandy SILT, trace to some gravel, trace clay, with cobbles and boulders (TILL). Dense to very dense. Grey. Wet.		10	SS	38																		
			11	SS	>70																		
			12	AW RC	DD																		
			13	AW RC	DD																		
			14	SS	63																		
50.4																							
7.3	End of Borehole																						

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

PROJECT		08-1121-0063-2000		RECORD OF BOREHOLE No 08-15		1 OF 1 METRIC											
G.W.P.		237-00-00		LOCATION		N 4992430.4; E 213528.8											
DIST		HWY 401		BOREHOLE TYPE		Power Auger 108mm I.D. Hollow Stem											
DATUM		Geodetic		DATE		July 2, 2008											
						ORIGINATED BY J.D.											
						COMPILED BY J.M.											
						CHECKED BY K.L.											
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	W _P	W	W _L	γ	GR	SA	SI	CL	
64.6 0.0	GROUND SURFACE Silty clay, some gravel, trace sand and organic matter (FILL) Brown		1	A.S.			64										
63.1 1.5	Silty sand to sandy silt, some gravel, trace to some clay, with cobbles and boulders (FILL) Very loose to dense Grey-brown Moist to wet		2	SS	3		63										
			3	SS	14		62										
			4	SS	19		61										
			5	SS	4		60										
			6	SS	>100		59										
			7	SS	6		58										
			8	SS	50		57										
54.9 9.7	SILTY CLAY (Weathered Crust) Very stiff Grey-brown Wet		9	SS	15		56										
			10	SS	15		55										
			11	SS	12		54										
51.8 12.8	Sandy SILT to Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Dense Dark grey Wet		12	SS	33		53										
50.3 14.3	End of Borehole						52										
							51										

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

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

PROJECT 08-1121-0063-2000		RECORD OF BOREHOLE No 08-16		1 OF 1 METRIC	
G.W.P. 237-00-00		LOCATION N 4992455.4; E 213636.4		ORIGINATED BY H.M.	
DIST HWY 401		BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem		COMPILED BY J.M.	
DATUM Geodetic		DATE June 26, 2008		CHECKED BY K.L.	

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 ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED					
63.6	GROUND SURFACE													
0.0	TOPSOIL													
0.1	Silty sand, some gravel, trace clay, with cobbles (FILL) Compact Grey-brown Moist		1	A.S.										
			2	SS	11									32 35 27 6
			3	SS	25									
			4	SS	11									
58.1														
5.5	Sandy SILT to Silty SAND, some gravel, trace clay (TILL) Compact to very dense Brown to grey Moist to wet		5	SS	56									13 44 37 6
			6	SS	46									
			7	SS	27									
			8	SS	40									7 32 53 8
52.3														
11.3	End of Borehole													
	Note: Water level in well screen at 5.2 m depth (Elev. 58.4) on Aug. 20, 2009.													

MIS-MTO 001 0811210063-2000 GPJ GAL-MISS GDT 10/24/09

PROJECT		08-1121-0063-2000		RECORD OF BOREHOLE No 08-19		1 OF 1 METRIC								
G.W.P.		237-00-00		LOCATION		N 4992432.3; E 213559.5								
DIST		HWY 401		BOREHOLE TYPE		Portable Drill								
DATUM		Geodetic		DATE		Sept. 8, 2008								
				ORIGINATED BY		K.L.								
				COMPILED BY		J.M.								
				CHECKED BY		M.I.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						
58.2	GROUND SURFACE						58							
0.0	Silty sand, some gravel, trace clay and rootlets (FILL) Compact Brown Moist		1	SS	12									
57.3			2	SS	32									
0.9	Silty sand, some gravel, trace clay, with cobbles and boulders (FILL) Loose to compact Grey-brown Wet		3	SS	9									
			4	SS	7									
			5	SS	21									
			6	SS	6									
			7	SS	17									
53.6			8	SS	31									
4.6	SILTY CLAY, occasional silt seam (Weathered Crust) Very stiff Grey-brown Wet		9	SS	31									
52.7			10	SS	64									
5.5	Sandy SILT, some gravel, trace clay, with cobbles (TILL) Dense to very dense Grey Wet		11	SS	49									
51.5														
6.7	End of Borehole													

PROJECT		08-1121-0063-2000		RECORD OF BOREHOLE No 08-20		1 OF 1 METRIC								
G.W.P.		237-00-00		LOCATION		N 4992442.8; E 213600.7								
DIST		HWY 401		BOREHOLE TYPE		Portable Drill								
DATUM		Geodetic		DATE		July 16, 2008								
						ORIGINATED BY D.G.								
						COMPILED BY J.M.								
						CHECKED BY K.L.								
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
60.9	GROUND SURFACE													
0.0	Silty sand with organic matter (TOPSOIL)		1	SS	6									
0.2	Black Moist													
60.3	Gravel (FILL)		2	SS	15									
0.6	Silty sand, some gravel, trace clay (FILL)													
	Loose to compact		3	SS	8									
	Brown Moist		4	SS	7									
58.1	SILTY CLAY with organic matter (FILL)		5	SS	12									
3.0	Dark grey Moist		6	SS	16									
57.2	Silty sand to sandy silt, some gravel, trace clay (FILL)		7	SS	59									
3.7	Compact Brown Moist		8	SS	90									
	Sandy SILT to Silty SAND, some gravel, trace clay (TILL)		9	SS	79									
	Very dense Brown Wet		10	SS	59									
			11	SS	84									
54.2	End of Borehole													
6.7	Note: Water level in open borehole at 3.7 m depth (Elev. 57.2) upon completion of drilling on July 16, 2008													



APPENDIX B

Borehole Records





1958 Investigation, Geocres No. 31G-137

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Highway 401 - G.N.R. Crossing Job No. 57149B
Client Dep't. of Highways of Ontario Casing BX (2-1/2" diam.)
Datum Projections Compiled By R.H.H.

Borehole No. 1
Boring Date April 18th - 19th, 1958
Checked By E.H.H.

SAMPLE CONDITION

-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL



SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Logged	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Organic silty loam.	Black		0' 0" 189.6					
Silty fine sand, grits, some organic matter.	Yellowish-Brown	Loose.			1	S.S. G		WET
Silty fine sand with many rocks and boulders.			5' 0" 184.6			DRILLED WITH BX CASING FROM 4' TO 7 1/2'		W.L. = 4' 4" APR 19 1958 23' HOLE, 22' CASING, BAILED 15' NIGHT BEFORE W.T. = 4' 9" MAY 3 1958
Fine sand, grits and pebbles.	Mixed Brown	Dense.	10' 0" 178.8		2	S.S. 43		WET W.L. = 10' 4" APR 21 50' HOLE, 27' CASING
Till: Silty fine sand, grits and pebbles.	Light Grey	Dense	15' 0"		3	S.S. 35		MOIST NAT. M.C. = 76%
As above.	Light Grey	Dense	20' 0" 169.6		4	S.S. 60 1/2		
Silty fine sand, grits and pebbles.	Light Grey	Dense	25' 0"		5	S.S. 41		QUITE MOIST NAT. M.C. = 15.3
Silty fine to medium sand, grits and pebbles.	Light Brownish-Grey				6	W.S. -		
Silty very fine sand, grits and angular rock fragments.	Dark Grey	Very Dense	30' 0"		7	S.S. 62		MOIST
Either bedrock containing sand layers, or a series of large boulders.			35' 0" 154.6					
Dolomitic, some coarse grained limestone.	Dark Grey		40' 0"		8	BXT R.C. -		CORE RECOVERY = $\frac{8}{60} = 13.3\%$
Fossiliferous limestone, some shale seams.	Dark Grey		45' 0"		9	BXT R.C. -		RECOVERY = $\frac{27}{60} = 36.7\%$
Fossiliferous limestone, some shale seams, some very hard limestone.	Dark Grey Light Grey		50' 0" 139.6		10	BXT R.C. -		RECOVERY = $\frac{36}{60} = 60.0\%$
HOLE TERMINATED								NOTE: POOR RECOVERY DUE IN PART TO GRIND OF THE ROCK CORE, THE ROCK DOES CONT SAND SEAMS.

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Highway 401 - C.N.R. Crossing Job No. 57149B
Client Dept. of Highways of Ontario Casing 4" Pipe & BX Casing
Datum Geodetic Compiled By M.M.

Borehole No. 3
Boring Date April 7th - 8th, 1958
Checked By E.M.L.

SAMPLE CONDITION




















-  UNDISTURBED
-  FAIR
-  DISTURBED
-  LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- B.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Organic silty loam.	black.		0' 0" 178.2					W.L. AT SURFACE, APR 8 1958. NO CASE FLOWING OVER. VERY SLOWLY, MAY 3.
Very nuggetty silty clay.	Greyish-Brown	Firm.	5' 0"		1  S.S.	6		NAT. M.C.=48.0% SLIGHTLY WETTER THAN PLASTIC LIMIT.
As above, occasional fine sand seams.	Greyish-Brown	Soft	6' 6" 171.7		2  S.S.	4		AS ABOVE.
Silty fine sand till.	BROWN GREY	Dense	17' 1"		3  S.S.	51		QUITE MOIST.
Very stony till			10' 6"		4  S.S.	20		NAT. M.C.=10.0% WET.
Clayey and silty fine sand, grits and pebbles.	Grey	Compact			5  S.S.	27		HOLE COULD NOT BE BORED BELOW THE 12 FT. DEPTH DUE TO RAPID INGRESS OF WATER.
As above.	Grey	Compact	15' 0"		6  S.S.	56		QUITE MOIST.
Silty fine sand, grits and rock fragments.	Grey	Dense			7  S.S.	47		" " NAT. M.C.=8.6%
			20' 0"					
As above.	Dark Grey	Very dense	25' 0"		8  S.S.	97		MOIST.
			28' 0" 149.2					
								REFUSAL PROBABLY BEDROCK.

e. m. peto associates ltd.





SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name: MILITARY - CUBAN Job No.: 79-08
Client: DIRECTORATE OF NATIONAL INTELLIGENCE Closing: NOV 1964
Datum: GEOGRAPHIC Compiled By: J. J.

Borehole No.
Boring Date .. APRIL 29, 1958.
Checked By .. J. M. P.

SAMPLE CONDITION

	UNDISTURBED
	FAIR
	DISTURBED
	LOST

SAMPLE TYPE

S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

V. T. IN SITU VANE SHEAR TEST
Q/u UNCONFINED COMPRESSIVE STRENGTH
W. L. WATER LEVEL IN CASING
W. T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No and Condition	Sample Type	No. of Blows per Ft.	WATER LEVEL, SOIL MOISTURE & REMARKS
Organic silty loam.	black		0' 0" 179.7					HOLE CAVED IN ON MAY 3, 1952 BUT W.T. BELIEVED TO BE AT SURFACE.
Medium sand.	Yellowish-brown		2' 0" 177.7					
Silty very fine sand, grits and pebbles.	Pale brown	Compact	2' 6"		1	S.S.	14	QUITE MOIST
Silty fine sand, grits and pebbles, some of medium sand.	Pale Brown	Dense	5' 0"		2	S.S.	35	MOIST. NAT. M.C. = 10.6 %
			9' 6"					
Silty fine to medium sand, grits and pebbles.	Gray	Dense			3	S.S.	38	SAMPLE LOST, WASH SAMPLE RETAINED.
			15' 0"					
As above, numerous grits and large angular rock fragments.	Gray	Compact			4	S.S.	28	WET
			20' 0"					
As above.	Gray	Very dense			5	S.S.	60 1/2	WET. NAT. M.C. = 6.8 %
			25' 0"					
Silty fine to medium sand, grits and pebbles.	Gray	Very dense			6	S.S.	70 1/2	SAMPLE LOST, WASH SAMPLE RETAINED.
			30' 0"					
As above.	Gray	Very dense			7	S.S.	82 1/2	WET.
			31' 6" 148.2					
								REFUSAL, PROBABLY BEDROCK.

SOIL ENGINEERING SERVICE - TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401 - C.N.R. Crossing Job No. 57149B

Borehole No. 7

Client: Dept. of Highways of Ontario, Casing, BK (24" diam.)

Boring Date Mar 15 1956

Date Geologic Compiled By M. L.

Checked By 234

SAMPLE CONDITION

SAMPLE TYPE

ABBREVIATIONS

 UNDISTURBED

S. S. 2nd STANDARD SPLIT TUBE SAMPLE

V.T. IN SITU VANE SHEAR TEST

 FAIR

S. L. SPLIT BARREL WITH LINERS

Q/u UNCONFINED COMPRESSIVE STRENGTH

☒ DISTURBED

S.F. THIN-WALLED SHELBY TUBE SAMPLE

W. L. - WATER LEVEL IN CASING

LOST

W. S. WASH SAMPLE

W. T. GROUND WATER TABLE IN SOIL

[illegible]

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name Highway 401 - C.N.R. Crossing Job No. 57195 Borehole No. 8
Client Depts. of Highways of Ontario Casing BX (2-1/2" diam.) Boring Date April 21st, - 22nd, 1958
Datum Geodetic Compiled By E.M. P. Checked By E.M. P.

SAMPLE CONDITION

UNDISTURBED
 FAIR
 DISTURBED
 LOST

SAMPLE TYPE

S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

ABBREVIATIONS

V.T. IN SITU VANE SHEAR TEST
Q/U UNCONFINED COMPRESSIVE STRENGTH
W.L. WATER LEVEL IN CASING
W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
Organic silty loam.	Black		0' 0" 180.4					HOLE BAILED TO 28' UPON COMPLETION. WT AT SURFACE WITHIN ONE HOUR, APR. 22, 1958. WT=0.4% MAY 3, 1958.
Silty fine sand, grits and rock fragments.	Brown	Compact	179.8		1	S.S.	22	MOIST.
As above, many large angular rock fragments.	Grey	Dense	5' 0"		2	S.S.	45	SLIGHTLY MOIST. NAT MC=11.1
Stony till.								
Silty fine to medium sand, grits and pebbles.	Gray	Compact	10' 0" 179.8		3	S.S. W.S.	28	DRILLED WITH BX CASING FROM 8' TO 10' FT.
Silty fine to medium sand, many grits and angular rock fragments.	Grey	Very dense	15' 0"		4	S.S.	85	
Silty very fine sand, grits and pebbles.	Dark Grey	Very dense	20' 0" 157.4		5	S.S.	90/6	MOIST.
Fine to medium sand.	Light Grey	Very dense	25' 0" 148.0		6	S.S.	60/3	SATURATED.
As above.	Light Grey	Very dense	30' 0" 148.0		7	S.S.	65/3	"
								REFUSAL PROBABLY BEDROCK.



APPENDIX C

Non Standard Special Provisions

Special Provision

SCOPE

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

SUBMISSION AND DESIGN REQUIREMENTS

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administer, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

Table 1 – Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 µm	#30	80% to 100%
425 µm	#40	40% to 80%
250 µm	#60	5% to 25%
150 µm	#100	0% to 6%

CONSTRUCTION

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Form concrete levelling pad and place CSPs and spacers.
2. Construct concrete levelling pads.
3. Place loose sand into 600 diameter CSP.
4. Install piles by driving to bedrock.
5. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeter of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

CSP FOR INTEGRAL ABUTMENTS – Item No.

Special Provision

The CSP at each pile shall be constructed to the following tolerances:

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of CSP from pile centroid	+/- 25 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

BASIS OF PAYMENT

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

VIBRATION MONITORING - Item No.

Special Provision

Scope

This special provision describes requirements for vibration monitoring during pile installation works.

Definitions

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and 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.

Submission Requirements

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 piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

Monitoring

The Contractor shall take readings during driving of each pile. The readings should be taken and recorded during the entire length of driving and during seating of the pile on the bedrock.

The pile(s) furthest from the monitored structure or utility should be driven first to assess the vibration level at the existing structures. If necessary, the contractor must alter the pile driving procedures for the remaining piles. The revised procedure shall be submitted to the Contract Administrator for approval prior to driving the remaining piles.

The measured vibrations shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next piles with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations are within acceptable levels. The above process must be repeated for each pile.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

BOULDERS/COBBLES DURING PILE INSTALLATION - Item No.

Special Provision

The overburden soils at the site include embankment fill and sandy silt to silty sand till containing cobbles and boulders.

Appropriate equipment and procedures will be required to penetrate/remove cobbles/boulders that are encountered during pile driving.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION