REPORT

April 18, 2012

FOUNDATION INVESTIGATION AND DESIGN REPORT

CULVERT 34 - STA 20+287, PECK TOWNSHIP
HIGHWAY 60 FROM WEST GATE
EASTERLY 24.5 KM TO STATION ROAD
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5551-04-00

Submitted to:

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Golder Associates

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FOUNDATION INVESTIGATION REPORT

CULVERT 34 – STA 20+287, PECK TOWNSHIP

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by HDR Corporation (HDR), on behalf of the Ministry of Transportation, Ontario (MTO), to provide foundation engineering services for the replacement of a culvert at STA 20+287 on Highway 60 in Peck Township, Ontario. The Key Plan showing the general location of this section of Highway 60 and the location of the investigated area are shown on Drawing 1. The purpose of this investigation is to establish the subsurface conditions at the location of the proposed culvert by borehole drilling, rock coring, in situ testing and laboratory testing on selected samples.

The Terms of Reference and Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal dated November 12, 2009. Golder's proposal (P9-1191-0062, dated December 11, 2009) for foundation engineering services associated with this culvert is contained in Section 6.8 of HDR's Technical Proposal that forms part of the Consultant's Agreement (Purchase Order Number 5008-E-0059) for this project. The original Scope of Work was subsequently updated with a Revised Scope of Work (dated September 19, 2011), which forms part of the overall Consultant's Agreement for this project. The work was carried out in accordance with Golder's Supplemental Specialty Quality Control Plan for this project dated June 2010. A drawing showing the alignment for the proposed culvert was provided to Golder by HDR on October 27, 2011.

2.0 SITE DESCRIPTION

The replacement culvert (Culvert 34) will be located at the same station and on the same alignment as the existing culvert at STA 20+287 in Peck Township, approximately 15 km east of the west gate to Algonquin Park. The existing highway grade at the culvert location is at about Elevation 425.6 m, up to about 7.3 m above the surrounding terrain which is at Elevation 418.2 m and 420.4 m at the south and north toe of slope, respectively. The south side slope of the existing embankment is formed at about 2 Horizontal to 1 Vertical (2H:1V) to the top of the culvert and then steeper to the toe of the slope, resulting in an overall local side slope of about 1.7H:1V. The north side slope of the existing embankment is formed at about (2.7H:1V).

In general, the topography in the area of the overall project limits consists of rolling terrain, including densely treed areas, numerous bedrock outcrops and steep valleys. Open water is present beyond the culvert outlet south of Highway 60, discharging into nearby Smoke Lake. At the time of our investigation, the culvert did not have water flowing through it.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the investigation associated with the replacement of the culvert at STA 20+287 was carried out on August 25 and September 6 to 12, 2011, during which time a total of three (3) Boreholes (C34-1 to C34-3) were advanced along the culvert alignment. In addition, seven (7) probe holes were advanced in the immediate vicinity of C34-2 and C34-3 to confirm the depth to refusal, as noted in the Record of Borehole sheets. The locations of, and ground surface elevations at, the boreholes are shown on Drawing 1.

Borehole C34-1, located on the existing highway embankment, was advanced using a track-mounted CME 55 drill rig outfitted with 108 mm inside diameter continuous flight hollow-stem augers, NW casing with wash boring and NQ size core barrel. Boreholes C34-2 and C34-3, located at the south and north toe of slope, respectively, were advanced using portable equipment outfitted with BW casing and thin-wall NQ coring equipment. All equipment was supplied and operated by Landcore Drilling Inc. of Sudbury, Ontario. The boreholes were advanced through the overburden using primarily wash boring methods and through cobbles/boulders using rock coring techniques. Soil samples in Borehole C34-1 were obtained continuously, or at intervals of depths of about 0.75 m and 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler (driven by an automatic hammer), performed in accordance with Standard Penetration Test (SPT) procedure (ASTM D1586) or using a thin-wall NQ core barrel. Boreholes C34-2 and C34-3 were advanced using portable equipment, and the split-spoon sampler was driven by a ½ weight hammer that was lifted manually to the SPT height. The number of blows per 0.3 m of penetration was converted to 'N'-values for the lower energy drive. Samples of the bedrock were obtained using a thin-wall NQ core barrel which fits inside NW or BW casing. All boreholes were backfilled with bentonite upon completion of drilling and coring in accordance with Ontario Reg. 903 (as amended).

As requested by the Ministry of Natural Resources (MNR), both the track-mounted and portable drill rigs were washed and sterilized with a 10 per cent bleach solution prior to being mobilized to site. The drill rigs were subsequently re-sterilized upon every re-entry to the site. These sterilization methods were completed in accordance with our Environmental Protection Plan.

Traffic protection was implemented for the boreholes drilled within the roadway in accordance with the Traffic Protection Plan for this project and MTO Book 7 "Temporary Conditions Manual of the Ontario Traffic Manual" (2001).

The boreholes were advanced to depths ranging between 3.6 m and 12.9 m below the ground surface, which includes coring of bedrock for depths ranging from about 1.7 m to 3.3 m below the surface of the bedrock. The groundwater conditions and water levels in the open boreholes were observed during the drilling operations and are described on the Record of Borehole sheets in Appendix A.

The fieldwork was supervised throughout by members of our technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock core samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Sudbury geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content and grain size distribution) was carried out on selected soil samples. Strength testing (uniaxial compression) was also carried out on selected specimens of the bedrock core. The results of the laboratory testing are presented on the Record of Borehole and Drillhole sheets in Appendix A, and are also included in Appendix B.

The highway was surveyed for station location and the stationing was painted on the asphalt surface by exp (formerly Trow), sub-consultant to HDR, prior to drilling. The as-drilled borehole locations and ground surface elevations were measured and surveyed by members of our technical staff, referenced to the painted stations on the highway. The MTM NAD 83 northing and easting coordinates, ground surface elevations referenced to Geodetic datum and borehole depth at each borehole are presented on the Record of Borehole sheets in Appendix A and are summarized below.

Borehole	Borehole Location		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
C34-1	5044664.9	367267.9	425.6	12.9
C34-2	5044647.5	367276.9	418.2	3.6
C34-3	5044678.9	367251.0	420.4	4.9

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Published literature indicates that the site is located in the McClintock Domain of the Algonquin Terrane, which is located in the Grenville Province (Geology of Ontario; OGS Special Volume 4)[1]. The bedrock of this domain generally consists of metasedimentary gneiss in granulite facies.

Based on terrain mapping (Ontario Geological Survey[2]), the site is located with a bedrock ridge below a ground moraine veneer with a ridged moderate local relief and a dry surface condition.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions, as encountered in the boreholes advanced for this investigation, together with the results of the laboratory tests carried out on selected soil and bedrock core samples, are given on the attached Record of Borehole and Drillhole sheets in Appendix A. Detailed results of the laboratory testing of the soil samples are provided in Appendix B. The stratigraphic boundaries shown on the Record of Borehole and Drillhole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of SPTs and in situ testing. These boundaries, therefore, represent transitions between soil and rock types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The inferred stratigraphy as encountered in the boreholes is shown on Drawing 1. It should be noted that the orientation (i.e. north, south, east, west) stated in the text of the report is typically referenced to project north (i.e. Highway 60 is oriented east - west) and therefore may differ from that shown on the drawings which represents magnetic north.

In general, the subsurface stratigraphy along the culvert alignment consists of topsoil or embankment fill, underlain by a deposit of sand and silt to silt underlain by a deposit of cobbles and boulders, overlying gneiss bedrock.

[1] Geology of Ontario, 1991. Ontario Geological Survey, Special Volume o4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

[2] Southern Ontario Engineering Geology Terrain Study, 1980. Ontario Geological Survey.

4.2.1 Topsoil

Approximately 0.1 m of topsoil was encountered at the ground surface at Elevation 418.2 m and 420.4 m in Boreholes C34-2 and C34-3, respectively.

4.2.2 Fill

Borehole C34-1 was advanced through the existing Highway 60 embankment and penetrated a layer of asphalt 60 mm thick, underlain by an approximately 75 m thick layer of granular fill which, in turn, is underlain by another layer of asphalt 320 mm thick. Underlying the asphalt in Borehole C34-1 and underlying the topsoil in Boreholes C34-2 and C34-3, the boreholes penetrated a deposit of fill consisting of sand to sand and gravel, trace to some silt, trace organics and/or blast rock, with a thickness between 0.3 m and 5.6 m, respectively. In Borehole C34-1, the fill deposit is comprised of a 0.9 m thick layer of sand, underlain by a 3.5 m thick layer of blast rock in a sand and gravel matrix, and a 1.2 m thick layer of sand. The top of the fill was encountered at Elevation 425.0 m (below the lower layer of asphalt) in Borehole C34-1 and at Elevation 418.1 m and 420.3 m in Boreholes C34-2 and C34-3, respectively.

The SPT 'N'-values measured within the fill range between 9 blows and 37 blows per 0.3 m of penetration, indicating a loose to dense relative density. When coring through the blast rock fill between a depth of 2.1 m and 4.0 m, a Total Core Recovery (TCR) of 40 per cent was recorded. In Boreholes C34-2 and C34-3, the split-spoon was noted to bounce upon penetrating the SPT sample depth.

The grain size distribution of one sample of the sand fill is shown on Figure B1 in Appendix B.

The measured water content on two samples of the fill is about 4 per cent and 13 per cent.

4.2.3 Sand and Silt to Silt

A 2.9 m thick deposit of grey sand and silt to silt containing trace to some clay and trace gravel was encountered underlying the fill in Borehole C34-1. The top of the deposit was encountered at a depth of about 6.1 m below the top of the embankment, at Elevation 419.5 m.

The SPT 'N'-values measured within this deposit range between 13 blows and 63 blows per 0.3 m of penetration, indicating a compact to very dense relative density. The split-spoon was noted to be bouncing at the bottom of the lowest sample taken.

The grain size distributions of two samples of this deposit are shown on Figure B2 in Appendix B.

The measured water content on two samples of this deposit is about 17 per cent and 19 per cent.

4.2.4 Cobbles and Boulders

A layer of cobbles and boulders between 0.8 m and 1.2 m thick was encountered underlying the fill or sand and silt to silt deposit in all the boreholes. The top of the cobbles and boulders layer was encountered between the depths of 0.4 m and 9.0 m below ground/pavement surface, at between Elevation 420.0 m and 416.6 m, respectively.

Rock coring techniques were used to advance the boreholes through the layer of cobbles and boulders. In Borehole C34-1, a TCR of 46 per cent was achieved between the depths of 9.0 m and 9.8 m. Split-spoon samples were taken at depths of 1.7 m and 0.8 m in Boreholes C34-2 and C34-3, respectively, and the split-spoon was noted to be bouncing with no sample being recovered. Boulders were penetrated in Borehole C34-2, between the depths of 0.7 m and 1.2 m and between 1.2 m and 1.9 m, with a TCR of 100 per cent and 0 per cent, respectively. Similarly, in Borehole C34-3, a boulder was penetrated between a depth of 0.4 m and 1.6 m with a TCR of 0 per cent. A total of seven probe holes were advanced by hand methods at both the north and the south sides of the embankment near the ends of the existing culvert to depths ranging from 0.1 m to 1.0 m below ground surface at which depths the split-spoon encountered refusal conditions (i.e. bouncing) on inferred cobbles and boulders.

4.2.5 Bedrock

Bedrock was encountered in all of the boreholes at depths ranging from 1.6 m to 9.8 m below the ground/pavement surface, corresponding to between Elevation 418.8 m and 415.8 m.

Based on a review of the bedrock core samples, the bedrock generally consists of fine to coarse grained, fresh to highly weathered, pinkish grey gneiss, as presented in the Record of Drillhole sheets in Appendix A. Photographs of the retrieved bedrock core samples are shown on Figure B3.

The TCR for the bedrock core samples ranges from about 51 per cent to 100 per cent. For the core samples obtained from Boreholes C34-1 and C34-3, the Solid Core Recovery (SCR) ranges from about 37 per cent to 85 per cent and the Rock Quality Designation (RQD) ranges from about 37 per cent to 100 per cent, indicating that generally the rock is of poor to excellent quality according to Table 3.10 in the Canadian Foundation Engineering Manual (CFEM, 2006). In Borehole C34-2, the SCR and RQD is 0 per cent.

Laboratory Unconfined Compression Strength (UCS) testing was carried out on two core samples of the bedrock. The UCS values which are presented on the Record of Drillhole Sheets in Appendix A and summarized below, indicate that the bedrock is very strong (R5, 100 MPa < UCS < 250 MPa) as per Table 3.5 of CFEM (2006).

Borehole	Elevation (m)	UCS (MPa)
C34-1	414.3	134
C34-3	416.8	140

4.2.6 Groundwater Conditions

The unstabilized water level in Boreholes C34-1 and C34-3 was measured at depths of 6.1 m and 0.2 m below pavement/ground surface, corresponding to Elevation 419.5 m and 420.2 m, respectively. Borehole C34-2 was dry upon completion of drilling. The groundwater level in the area is subject to seasonal fluctuations and variations due to precipitation events.

5.0 CLOSURE

The field drilling program was carried out under the supervision of Mr. Ed Savard and Mr. Matt Thibeault EIT, under the overall direction of Mr. Evan Childerhose, P.Eng. This report was prepared by Mr. Evan Childerhose, P.Eng, and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and Principal with Golder, conducted an independent quality control review of the report.

Report Signature Page

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PART B

FOUNDATION DESIGN REPORT

CULVERT 34 – STA 20+287, PECK TOWNSHIP

HIGHWAY 60 FROM WEST GATE

EASTERLY 24.5 KM TO STATION ROAD

MINISTRY OF TRANSPORTATION, ONTARIO

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the factual geotechnical data obtained during the subsurface investigation and recommendations on the foundation aspects of design of the proposed works. The recommendations provided are intended for the guidance of the design engineer. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

6.1 General

We understand from HDR that the culvert to be constructed at STA 20+287 in Peck Township will have a width and height of 1.8 m. The existing culvert is about 40 m long and we understand the new culvert will be constructed to approximately the same length, at the same location (i.e. skewed to the highway centreline) and at the same grade elevation. The embankment at the culvert location is currently 6.1 m high, and some minor widening on the south side will be required to maintain a 4.0 m wide travelling lane. We understand that the invert elevations of the new culvert will be the same as the invert elevations of the existing culvert, and that head walls and wing walls will not be required. Further, we understand that staged construction methods are preferred and that portable traffic signals and a temporary detour will be used to accommodate construction of the new culvert.

The subsoils along the culvert alignment generally consist of topsoil and fill materials, underlain by a sand and silt to silt deposit and a deposit of cobbles and boulders, overlying bedrock. The bedrock surface was encountered between Elevation 418.8 m and 415.8 m. Details of the subsurface conditions along Culvert 34 are presented in Section 4.2 and shown in stratigraphic profile on Drawing 1 following the text of this report.

6.2 Stability

Limit equilibrium slope stability analyses were carried out for three embankment configurations at the Culvert 34 site using the commercially available program GeoStudio 2007 (Version 7.17), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions. This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available, and is based on deep-seated, global failure surfaces that would affect the operation of the roadway. The stability analyses were carried out to check that the target minimum FoS was achieved for the proposed embankment height and geometry at the culvert location.

The analyses assume that all topsoil beneath the culvert alignment will be removed prior to construction. Further, the analyses assume that the embankment in the excavated area adjacent to the culvert will be reconstructed using Granular ‘B’ Type II fill. The analyses were also carried out for the alternative of backfilling the excavation using rock fill consistent with the specifications in SP 206S03 (Rock Embankments) for size of rock fill adjacent to structures although it is recommended that Granular ‘B’ Type II be used as backfill to the culvert excavation. Depending on the height and location of the detour, which may determine the maximum side slopes to which it will be constructed, the detour embankment may be constructed of granular fill or rock fill.

For the native granular soils and fill, effective stress parameters were employed in the analyses assuming drained conditions. The effective stress parameters (effective friction angle and effective cohesion) for the granular soils were estimated from empirical correlations using the results of in situ SPT, in conjunction with engineering judgement based on experience in similar soil conditions. Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types for the re-instated embankment over the culvert.

Soil Type	Unit Weight (kN/m^3)	Angle of Internal Friction
New Granular 'A' or Granular 'B' Type II Fill	21	35^o
New Rock Fill	19	40^o
Existing Sand Fill and Rock Fill in a Sand and Gravel Matrix (compact to dense)	20	38^o
Existing Sand and Gravel Fill (loose to compact)	19	30^o
Sand and Silt to Silt (compact to very dense)	19	28^o
Cobbles and Boulders	19	35^o

Currently, the existing embankment overall side slopes are about 1.7H:1V or slightly shallower (see Section 2.0 for description of existing embankment geometry). We assume that the embankment in the immediate area of the culvert will be reconstructed of Granular 'B' Type II and hence will have side slopes no steeper than 2H:1V above the culvert and 1.7H:1V for rock fill in the immediate area adjacent to the culvert.

The stability analysis carried out for the proposed embankment at the culvert location indicates that after reconstruction of the embankment using Granular 'B' Type II fill (or rock fill) to the same side slopes as the existing embankment, the embankment will have a FoS of 1.3 or greater for deep-seated, global failure surfaces that would impact the operation of the roadway. Figure 1 shows the final embankment geometry adjacent to the culvert, stratigraphy and parameters used in the analyses and the results of the stability analyses for an embankment constructed of Granular 'B' Type II material, including the minimum FoS centroid. A similar FoS was achieved for the embankment constructed out of rock fill with the same side slopes.

For a final embankment geometry of side slopes at 2H:1V and Granular 'B' Type II construction at/over the culvert, a FoS greater than 1.3 was also achieved.

6.3 Settlement

Analyses to estimate the magnitude of the expected settlement along the culvert alignment were carried out using hand and spreadsheet calculations. The estimated settlement below the culvert consists of the immediate settlement of the native granular soils due to embankment loading. In addition, the estimated settlement of the overall embankment at this site also includes the component of settlement due to the self-weight compression of the embankment backfill material. The thickness of the foundation soils and the height of the embankment vary along the proposed culvert crossing and as such, the settlement along the length of the culvert will similarly vary.

The immediate compression of the native cohesionless deposits was modelled by estimating an elastic modulus of deformation based on the SPT 'N' values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). The compact to very dense sand and silt to silt layer was assigned an Elastic Modulus (E') of 10 MPa and the cobbles and boulders layer was assigned an E' of 30 MPa.

6.3.1 Embankment Fill Types

Different embankment fill materials (i.e. granular fill and rock fill) provide relative advantages and disadvantages in terms of weight (i.e. driving force and applied load to founding subsoils/bedrock), construction cost and time, achievable side slope geometry and ease of construction/availability. A combination of sand or sand and gravel fill and rock fill in a sand and gravel matrix was used for construction of the existing embankments.

The main advantage of using granular fill (i.e. sand and gravel, Granular 'A' or Granular 'B' Type II) for embankment construction is the ease of construction and the negligible amount of post-construction settlements that will occur within the fill embankment itself above the water level. However, this option will require a larger volume of fill and wider right-of-way because the side slopes (2H:1V) will be flatter than for rock fill slopes. For this project, the final embankment cross-section needs to match the existing approximately 2H:1V to 2.7H:1V side slopes. For this project, acceptable granular fill is considered to be well-graded, locally available and/or imported, granular material such as Granular 'B' Type II.

The main advantage of constructing embankments using rock fill is the ability to achieve steeper side slopes (1.25H:1V) thereby reducing the overall quantity of material required for the project as well as reducing the width of the right-of-way required. However, since the final embankment side slopes need to match the existing slopes at approximately 2H:1V to 2.7H:1V, the embankment profile using rock fill would require the same volume as that of a granular fill embankment. The main disadvantage of using rock fill is that some post-construction settlement of the rock fill itself will occur, mostly within about the first year post-construction.

6.3.1.1 Settlement of Embankment Fill

It is recommended that the embankment be reconstructed at the culvert location using SP 110S13 (Aggregates) Granular 'B' Type II material. The magnitude of compression settlement from the properly compacted embankment fill is expected to occur during construction.

The total immediate settlement of the native foundation soils along the culvert alignment due to the embankment widening (after reconstruction of the embankment over the culvert) is estimated to be less than 25 mm due to the compact to dense nature of the subsoils, given that there is not anticipated to be any grade raise (i.e. no net increase in loading).

Therefore, the total post-construction settlement at the roadway level is expected to be less than 25 mm. Further, differential settlement between the existing highway embankment and the reconstructed embankment over the culvert is expected to be less than 25 mm if Granular 'A' or Granular 'B' Type II, or other acceptable granular material, is used as backfill.

If rock fill is used for the reconstruction of the embankment over the culvert, there will be settlement due to compression within the rock fill itself under self-weight, in addition to the settlement of the underlying foundation soils, as described above. It is estimated, based on MTO's "Post-Construction Rock Fill Settlement and Guidelines for Estimating Rock Fill Quantity" dated April 2010, that approximately 50 mm of settlement will occur after backfilling over the culvert to the roadway profile grade. Since this settlement will occur post-construction and will be differential between the existing highway embankment and the reconstructed embankment over the culvert, we do not recommend the use of rock fill to backfill over the culvert.

Further, due to the minimal amount of settlement estimated that will occur below the new culvert after construction, horizontal strain is not anticipated to be a significant structural design factor.

6.4 Geotechnical Resistance

The invert of the culvert is expected to be the same as the existing invert level at Elevation 420.1 m and 418.1 m at the inlet (north side) and outlet (south side), respectively. If a pre-cast concrete box culvert is used it will be embedded 0.5 m below this depth resulting in a subgrade at Elevation 419.6 m and 417.6 m at the inlet and outlet sides, respectively. The subsoils at these elevations consist of cobbles and boulders near the ends of the culvert and compact to very dense sand and silt to silt along the length of the culvert under the existing embankment. If a pre-cast open footing culvert is used the footings will be founded 1.8 m below the existing invert elevation or on bedrock, whichever is shallower. At the inlet and outlet the footings will be founded on bedrock at Elevation 418.8 m and 416.3 m, respectively. In the middle of the culvert, the footings will be founded on the sand and silt to silt.

A factored geotechnical axial resistance at Ultimate Limits States (ULS) of 500 kPa is recommended for design for an assumed 1.8 m wide concrete box culvert founded on a properly prepared subgrade and bedding (see Section 6.6.1) overlying the native cohesionless soils, as described above. The geotechnical reaction at Serviceability Limit States (SLS) for 25 mm settlement may be taken as 200 kPa.

For a pre-cast concrete open footing culvert with 0.9 m wide footings, a ULS of 500 kPa is recommended for design for the footings on the sand and silt to silt deposit near the midpoint of the culvert. The geotechnical reaction at SLS may be taken as 200 kPa. For the footings founded on bedrock near the inlet and outlet, the factored geotechnical axial resistance at ULS is estimated to be 1000 kPa and governs the design.

The geotechnical resistances given above are for loads that will be applied perpendicular to the surface of the base of the culvert. Where loads are not applied perpendicular to the base of the culvert, inclination of the loads should be taken into account in accordance with Section 6.7.4 and Section C6.7.4 of the Canadian Highway Bridge Code (CHBDC) and its Commentary.

6.4.1 Resistance to Lateral Loads/Sliding Resistance

Resistance to lateral forces/sliding resistance between the base of a concrete box culvert and the granular fill/bedding should be calculated in accordance with Section 6.7.5 of the CHBDC. The following summarizes the coefficient of friction for the interface materials for both precast and cast-in-place concrete.

Interface Materials	Coefficient of Friction (tan δ or tan Φ')
Precast Concrete Culvert on Compacted Granular 'A' or Granular 'B' Type II Bedding	0.45
Cast-in-Place Concrete Culvert on Compacted Granular 'A' or Granular 'B' Type II Bedding	0.58

6.4.2 Frost Protection

Spread footings for an open footing concrete culvert in the Huntsville area should be provided with a minimum of 1.8 m of conventional soil cover for frost protection, as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario). The soil used for frost protection should be comprised of SP 110S13 (Aggregates) Granular 'A' or Granular 'B' Type I or Type II material. Spread footings founded directly on bedrock do not require frost protection.

If a pre-cast concrete box culvert is used, frost penetration could also extend to 1.8 m below the invert in the event of non-flowing (dry) conditions in the winter. As the subgrade soils below the culvert base slab are comprised of frost-susceptible sand and silt to silt in places, it would be prudent that these soils be sub-excavated to a depth of 1.8 m below the culvert invert and replaced with non frost-susceptible material such as SP 110S13 Granular 'B' Type II. The cobbles and boulders layer encountered near the culvert ends is not considered to be a frost-susceptible material and as such, this layer does not require to be excavated for the purpose of frost protection for the box culvert.

6.5 Lateral Earth Pressures

The lateral earth pressures acting on the side walls of the culvert (we understand that head walls and/or wing walls will not be required at this site) will depend on the type and method of placement of backfill materials, the nature of soils/embankment fill behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the culvert walls.

The following recommendations are made concerning the design of the box culvert.

- Select, free-draining granular fill meeting the specifications of SP 110S13 (Aggregates) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the No. 200 (0.075 mm) sieve should be used as backfill behind the culvert. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Backfill should be placed with a maximum of 200 mm loose lift thickness. Other aspects of the granular backfill requirements for concrete culverts should be in accordance with OPSD 803.010 (Backfill and Cover for Concrete Culverts).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the culvert, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For a box culvert, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the culvert (in accordance with Figure C6.20(a) of the Commentary to the CHBDC).
- For a box culvert, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of granular fill:

	Granular Fill
Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.31
At rest, K_o	0.47

If the culvert allows for lateral yielding, active earth pressures may be used in the geotechnical design of the structure. If the culvert does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume a restrained structure, may be taken as presented in Table C6.6 of the Commentary to the CHBDC.

6.6 Construction Considerations

6.6.1 Bedding and Backfill Above Base of Culvert

The precast box culvert should be constructed in accordance with SP 422S01 (Precast Concrete Box Culverts). The box culvert should be constructed on a minimum 300 mm thick layer of SP 110S13 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II material for bedding purposes. As the excavation is to be un-watered to allow for construction of the culvert in dry conditions (see Section 6.6.3), a minimum 75 mm thick un-compacted levelling pad consisting of concrete fine aggregate meeting the grading requirements specified in OPSS 1002 (Aggregates for Concrete) should be provided as shown on OPSD 803.010 (Backfill and Cover for Concrete Culverts).

In dry conditions, the bedding should be placed in lifts not exceeding 200 mm in loose thickness, and compacted to at least 95 per cent of the Standard Proctor maximum dry density (SPMDD) of the material, as specified in OPSS 501 (Compacting). The structural design of the culvert should take into consideration the conditions for bedding placement and compaction in accordance with the requirements of Section 7.8.3.6 of the CHBDC. For culverts where the invert level is located at or below the groundwater table, the structural design should assume that the bedding material will only achieve 80 per cent of the SPMDD during placement unless the excavation is dewatered (see Section 6.6.4).

For open footing culverts, the footings may be placed directly on the properly prepared native sand and silt to silt as long as all softened/loosened soils within the footprint of the footings at the founding level should be removed and replaced with mass concrete in accordance with OPSS 902 (Excavation and Backfilling for Structures). Where mass concrete is used to level the founding area, it should be of the same compressive strength as will be used for the actual footing. If bedrock excavation is required to level the founding area, it should be carried out using controlled blasting techniques (i.e. line drilling, pre-shearing or cushion blasting) in order to minimize shattering and over-break resulting from blast damage to the rock mass.

Backfill to the culvert walls should consist of granular fill meeting the specifications for OPSS Granular ‘B’ Type II (but with less than 5 per cent passing the 200 sieve). The backfill should be placed in lifts not exceeding 200 mm loose thickness and compacted to 95 per cent SPMDD. The fill depth during placement should be maintained equal on both sides of the culvert with one side not exceeding the other by more than 500 mm. Granular fill materials and placement should be carried out in accordance with the requirements as outlined in SP 206S03 (Earth Excavation, Grading; Earth Embankment).

The culverts should be designed for the full overburden stress and appropriate live loads, assuming a fill unit weight of 22 kN/m^3 for Granular ‘A’ and 21 kN/m^3 for Granular ‘B’ Type II backfill above and surrounding the culvert.

Prior to placement of the roadway granular subbase and base courses, the final lift of embankment fill should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

6.6.2 Erosion Protection

Provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert location. In order to prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall should be provided at the upstream end of the culvert. If a clay seal is adopted, the clay material should meet the requirements of OPSS 1205 (Clay Seal), and the seal should extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high water level including along the embankment slope. Alternatively, a clay blanket may be constructed, extending upstream three (3) times the culvert height and along the adjacent slopes to a height of two (2) times the culvert height or the high water level, whichever is greater. If a concrete cut-off wall is adopted, it is to be designed based on hydraulic and structural considerations, as appropriate.

The requirements for, and design of, erosion protection measures for the inlet and outlet of the culvert should be assessed by the hydraulics design engineer. As a minimum, rip-rap treatment for the outlet of the culvert should be consistent with the standard presented in OPSD 810.010 (Rip-Rap Treatment). Erosion protection for the inlet of the culverts should follow the standard presented in OPSD 810.010 (Rip-Rap Treatment) similar to the outlet but with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket, including the creek side slopes and fill slope over the culvert.

6.6.3 Subgrade Preparation and Control of Groundwater and Surface Water

All topsoil and softened/loosened soils should be stripped from below the culvert and embankment areas, prior to placement of bedding or new fill.

Excavation for frost protection and to allow placement of bedding material, the precast box units and the embankment backfill will be required to approximately 9 m below the existing highway grade. This excavation to Elevation 416.8 m at the midpoint of the culvert is about 3 m below the interface between the embankment and underlying native sand and silt to silt. At the base of the excavation, groundwater flow into the excavation can be expected to occur due to the relatively permeable subsoils and water level observed at the culvert location. Therefore, control of surface water and groundwater will be necessary at the culvert site to allow for excavation and construction to be carried out in dry conditions.

Groundwater control will be required at the culvert location, as the foundation excavation is expected to extend up to 2.7 m below the groundwater level near the midpoint of the culvert where the deepest excavation is required for frost protection. At the culvert ends, the excavation will extend less than 1 m below the groundwater level. Where the excavation will be advanced through, or into, water-bearing cohesionless soil deposits, appropriate unwatering will be required to maintain the water level below the founding level for the culvert during construction. Since the depth of the excavation below the water level is less than about 3 m, it is anticipated that groundwater inflow may be controlled locally by pumping from properly filtered sumps below the base of the excavation. Alternatively, Granular 'B' Type II backfilled below the middle section of the culvert could be placed sub-aqueously in accordance with OPSS 209 (Construction Specifications for Embankments Over Swamps and Compressible Soils) provided that less than 2 m of sub-aqueous filling occurs and that dewatering is still carried out to lower the groundwater level to at least below the culvert foundation and bedding (i.e. Elevation 418.3 m), approximately 1 m below the groundwater level.

An NSSP should also be included in the Contract to alert the contractor to the potential issues associated with unwatering of the soils at this culvert site and that the excavation must be unwatered and kept stable during construction; a sample NSSP is included in Appendix C.

Surface water should be directed away from the excavation area to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade.

6.6.4 Construction Staging, Detour, Excavations and Temporary Shoring

We understand that staged construction is being considered at this site for replacement of the culvert. However, given the presence of rock fill within the embankment and cobbles and boulders at/below the invert of the culvert, it may not be possible to install conventional shoring through these deposits to facilitate construction. We further understand that due to environmental considerations regarding the watercourse, that altering the current slope geometry (i.e. embankment toes) is not desirable.

We understand that staged construction with a roadway detour approach is proposed to allow for construction of the culvert as depicted on Figure 2. In this case, the existing embankment will be widened to the south by steepening the existing side slope to approximately 1.5H:1V and maintaining the toe of slope at the same location. The detour on the north side of the embankment will be constructed over a portion of the replaced culvert at a lower grade than the existing highway. In this case, a 5 m long temporary culvert extension will be installed to accommodate the detour, with side slopes note not to extend beyond the highway right-of-way (ROW).

Stability analysis of the temporary conditions for Stages 1 and 2 (as referenced on Figure 2) is shown on Figures 3a and 3b for the south and north side of the temporary embankment, respectively, and indicate a FoS greater than 1.3 against deep-seated failure.

For a detour embankment on the north side of the highway approximately 1 m above the top of the culvert, 1.5H:1V side slopes will have a FoS greater than 1.3 m. However, we understand the grade difference between the existing highway and the detour may be too great, and a higher detour embankment is being proposed. For a detour embankment 2.5 m above the top of the culvert, we understand that there is not enough space available to achieve 1.5H:1V side slopes within the right-of-way. The embankment will not be stable at the proposed side slopes at 1.25H:1V if constructed with Granular 'B' Type II, with respect to shallow surficial failures near the crest of the embankment. For stability, we recommend that the core of the detour embankment be constructed out of Granular 'B' Type II with 1.5H:1V side slopes and that rock fill be placed on the north side of the detour embankment to achieve the required maximum side slope of 1.25H:1V. The rock fill should be placed such that there is a minimum width of 2 m of rock fill at the base of the detour embankment above the top of the culvert. This 2 m base will ensure that the rock fill wedge will be practical from a constructability standpoint. A temporary detour embankment constructed with the 2 m wide base will have a FoS greater than 1.3 as shown on Figure 3c.

Excavations through the existing embankment fill to the founding level should be made no steeper than 1.5H:1V in all directions. The widening portion of the south side of the embankment should be constructed using SP 110S13 (Aggregates) Granular 'B' Type II material. The new fill should be keyed into the existing embankment side slope as per the requirements of OPSD 208.010 (Benching of Earth Slopes) to minimize differential settlement between the existing embankment slopes and the newly placed embankment fill. The Granular 'B' Type II backfill over the culvert should be similarly keyed into the existing embankment. If the detour embankment is constructed with rock fill, this rock fill needs to be removed and replaced with Granular 'B' Type II material to match the final embankment side slopes. Further, the rock fill wedge placed over the granular fill for the detour should be similarly keyed in.

All excavations must be carried out in accordance with Ontario Regulation 213 "Ontario Occupational Health and Safety Act for Construction Projects" (as amended by Ontario Regulation 443). The fill and native soils are considered to be Type 3 soil, but should be excavated to slopes no steeper than 1.5H:1V to allow for proper benching and keying-in of new fill. In addition, provisions for traffic control measures should be included in the Contract Documents to maintain the safe operation of the existing Highway 60 and any associated side roads or detours during excavation operations, where applicable.

If temporary excavation support systems are still required, such as below the existing culvert, they should be designed and constructed in accordance with OPSS 539 (Temporary Protection Systems). Temporary excavation support systems should be designed to Performance Level 2 for any excavation adjacent to existing roadways.

7.0 CLOSURE

This report was prepared by Mr. Evan Childerhose, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Coyne, P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge M. A. Costa, P.Eng., Golder's Designated MTO Contact for this project and a Principal with Golder, reviewed the technical aspects of and conducted an independent quality control review of the report.

Report Signature Page

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EC/SEMC/JMAC/lb/cl

n:\active\2009\1190 sudbury\1191\09-1191-0062 hdr itrans hwy 60 culverts\7000 reporting\final report\09-1191-0062 final rpt 12apr18 final fidr hwy 60 culvert 34.docx

REFERENCES

Bowles, J.E., 1984. Physical and Geotechnical Properties of Soils, Second Edition. McGraw Hill Book Company, New York.

Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, Fourth Edition.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6-06, 2006. CSA Special Publication, S6.1-06. Canadian Standard Association.

Kulhawy, F.H. and Mayne, P.W., 1990. Manual on Estimating Soil Properties for Foundation Design. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

Ministry of Transportation, Ontario, 2001. Ontario Traffic Manual, Book 7. Temporary Conditions, Field Edition.

Ministry of Transportation Ontario, 2010. Post-Construction Rock Fill Settlement and Guidelines for Estimating Rock Fill Quantity, April 12, 2010.

STANDARDS:

ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects

Ontario Regulation 443/09 Amendment to Ontario Regulation 213

Contract Design Estimating and Documentation (CDED):

SP 110S13 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

SP 206S03 Earth Excavation, Grading; Earth Embankment; Rock Excavation, Grading; Rock Embankment

SP 422S01 Precast Concrete Box Culvert

Ontario Provincial Standard Drawings:

OPSD 208.010 Benching of Earth Slopes. November 2008.

OPSD 803.010 Backfill and Cover for Concrete Culverts With Spans less than or equal to 3.0 m. November 2010.

OPSD 810.010 Rip-Rap Treatment for Sewer and Culvert Outlets. November 2007.

Ontario Provincial Standard Specifications:

OPSS 209	Construction Specification for Embankments Over Swamps and Compressible Soils. April 2009.
OPSS 501	Construction Specification for Compacting. November 2010.
OPSS 539	Construction Specification for Temporary Protection Systems. November 2009.
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS 1002	Material Specification for Aggregates – Concrete. April 2011.
OPSS 1205	Material Specification for Clay Seal. November 2009.

Ontario Water Resources Act:

Ontario Regulation 903/90	Wells
Ontario Regulation 468/10	Amendment to Ontario Regulation 903

Commercial Software:

GeoStudio (Version 7.17) by Geo-Slope International Ltd.

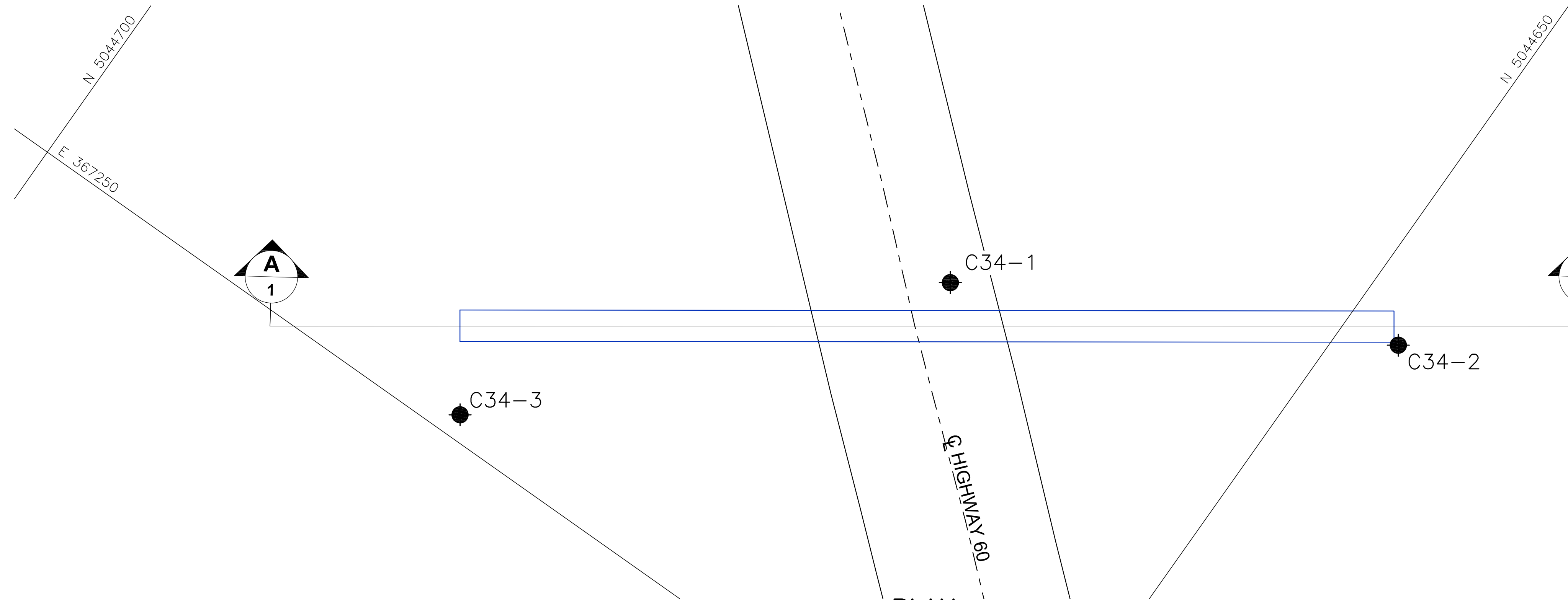
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 5551–04–00

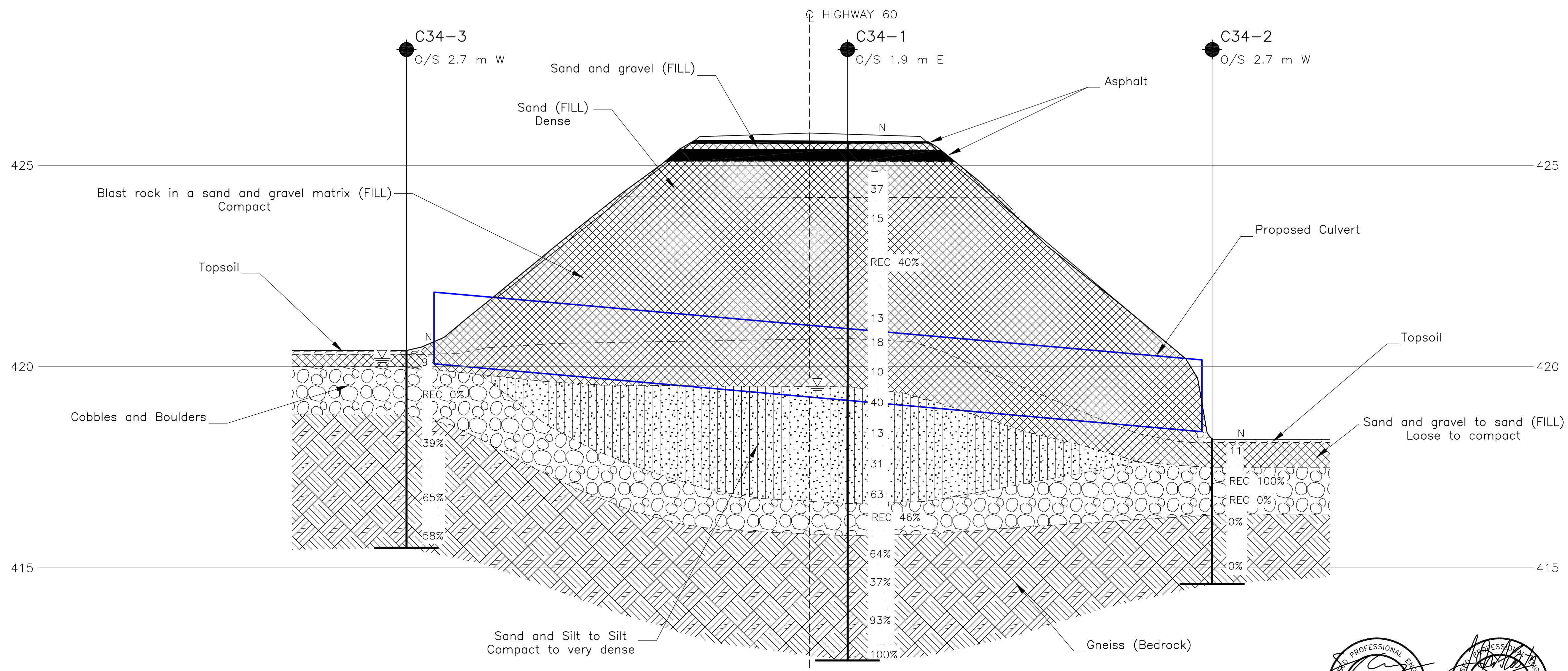
SHEET

HIGHWAY 60
CULVERT AT STA. 20+287
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
SUDBURY, ONTARIO, CANADA

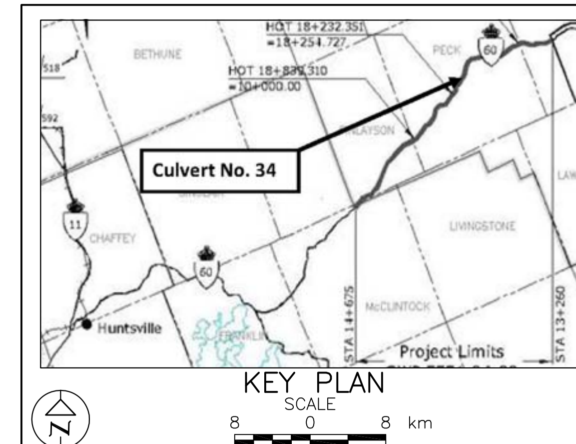


PLAN
SCALE
2.5 0 2.5 5 m



A–A' 1 PROFILE ALONG CULVERT
HIGHWAY 60

HORIZONTAL SCALE
2.5 0 2.5 5 m
1.25 0 1.25 2.5 m
VERTICAL SCALE



KEY PLAN
SCALE
8 0 8 km

LEGEND

- Borehole – Current Investigation
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% — Rock Quality Designation (RQD)
- REC — Recovery
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
C34–1	425.6	5044664.9	367267.9
C34–2	418.2	5044647.5	367276.9
C34–3	420.4	5044678.9	367251.0

NOTES

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

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

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

REFERENCE

Base plans provided in digital format by HDR, PDF file no. 2011–07–14 Option G at Culvert 34.pdf, received JULY 14, 2011. Original ground / culvert profile based on PDF received from HDR on OCT. 27, 2011.

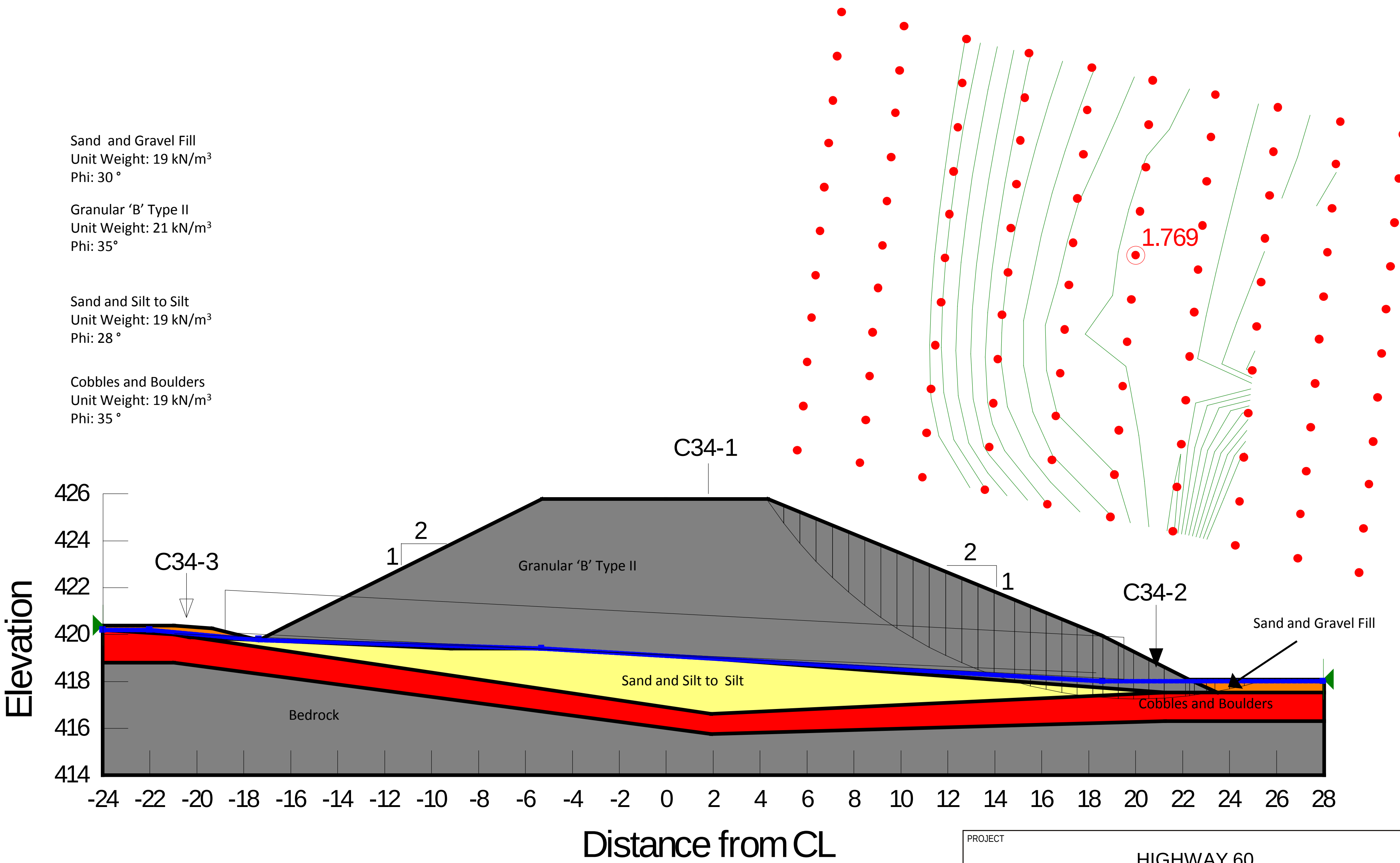
LICENSED PROFESSIONAL ENGINEER S.E.M. COYNE 90471921 APR 18, 2012 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER J.M.A.COSTA APR 18, 2012 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No. 31E–314

HWY. 60		PROJECT NO. 09–1191–0062	DIST.
SUBM'D. EC	CHKD.	DATE: APR 2012	SITE:
DRAWN: JJL	CHKD. SEMC	APPD. JMAC	DWG. 1



PROJECT

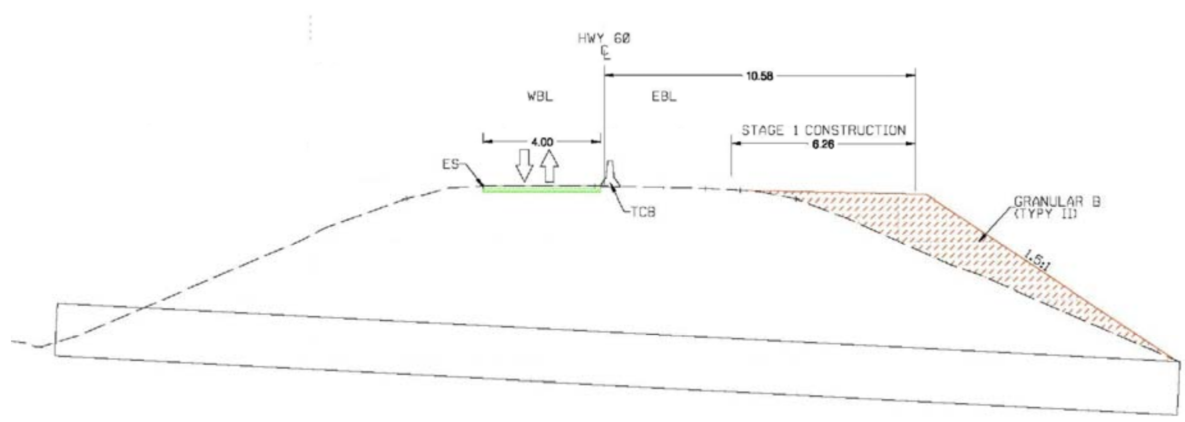
HIGHWAY 60
CULVERT AT STA 20+287

TITLE

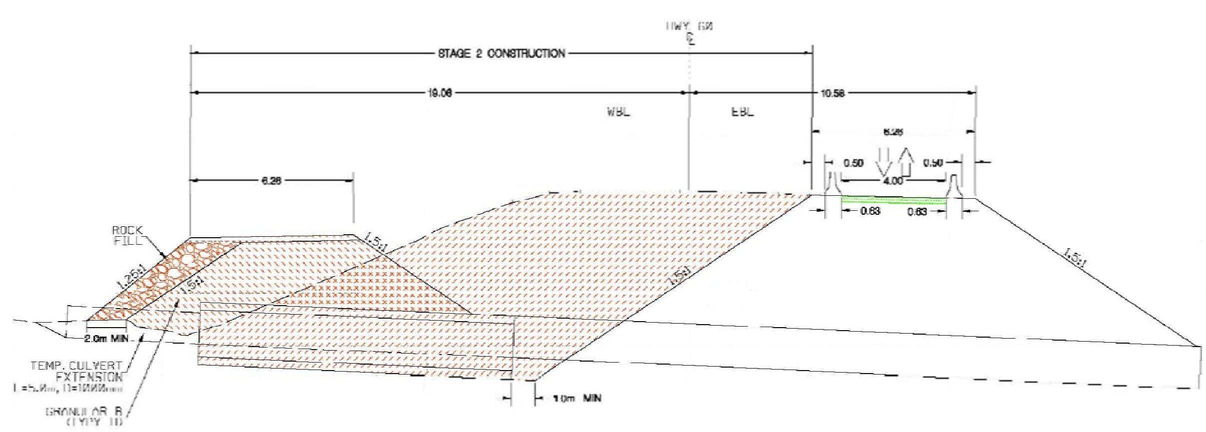
STABILITY ANALYSIS
FINAL EMBANKMENT CONFIGURATION

Golder Associates

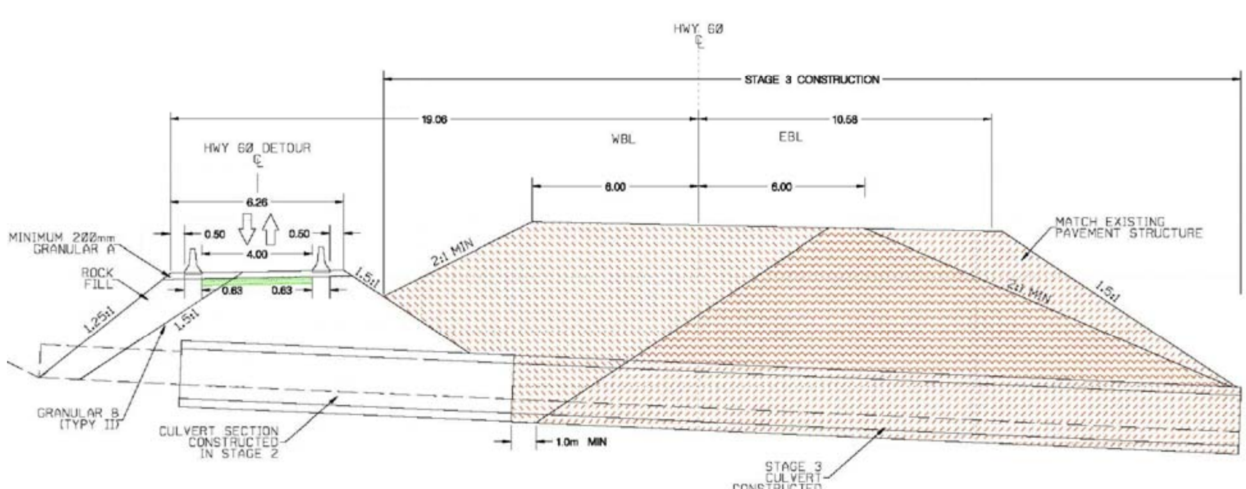
PROJECT No. 09-1191-0062			FILE No. ----	
DESIGN	EC	APR 2012	SCALE AS SHOWN	REV.
CADD	--		**Figure 1**	
CHECK	SEMC	APR 2012		
REVIEW				



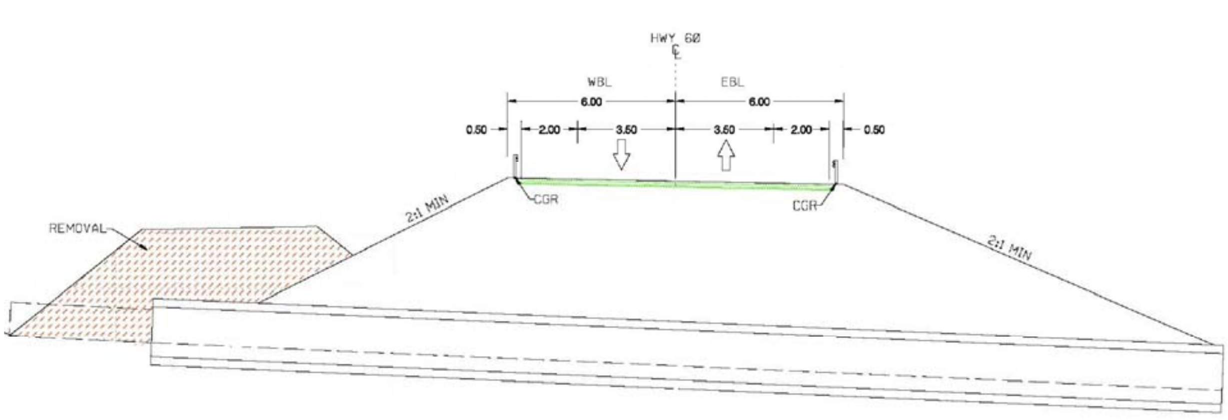
STAGE 1:
—Widen south side of embankment to 1.5H:1V using Granular 'B' Type II.



STAGE 2:
—Excavate north side of embankment at 1.5H1V.
—Replace north portion of culvert.
—Build detour embankment on replaced section of culvert as shown.



STAGE 3:
—Replace remainder of culvert.
—Reconstruct side slope to original embankment on south side at min. 2H:1V using Granular 'B' Type II.
—Rebuild north side slope to 2H:1V.



STAGE 4:
—Shift traffic to existing alignment then remove temporary detour and the culvert extension.

REFERENCES:

1. SCHEMATICS TAKEN FROM PROPOSED STAGING FIGURES RECEIVED FROM HDR ON MAR. 30, 2012. RECOMMENDATIONS FOR THE CONFIGURATION OF THE TEMPORARY DETOUR ARE AS PER GOLDER'S EMAIL ON APRIL 3, 2012

Golder Associates
Sudbury, Ontario, Canada

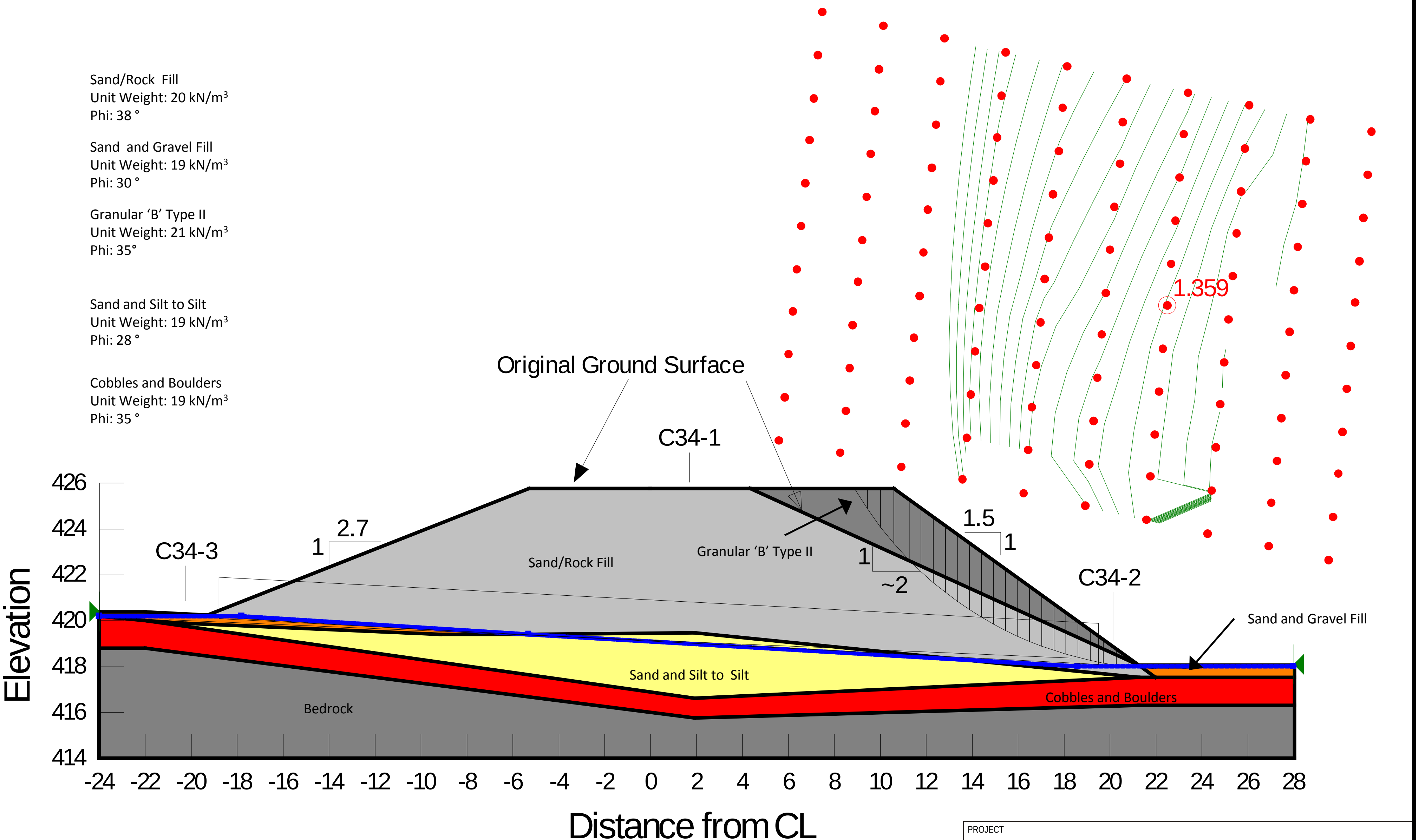
SCALE	NTS
DATE	APR 2012
DESIGN	
CAD	JJL
CHECK	SEMC
REVIEW	JMAC
FILE No.	0911910062AA001.dwg
PROJECT No.	09-1191-0062
REV.	

TITLE

HIGHWAY 60
CULVERT AT STA 20+287

CULVERT STAGING DETAILS

FIGURE
2



PROJECT

HIGHWAY 60
CULVERT AT STA 20+287

TITLE

STABILITY ANALYSIS
TEMPORARY CONDITION – STAGE 1

Golder Associates

PROJECT No. 09-1191-0062			FILE No. ----	
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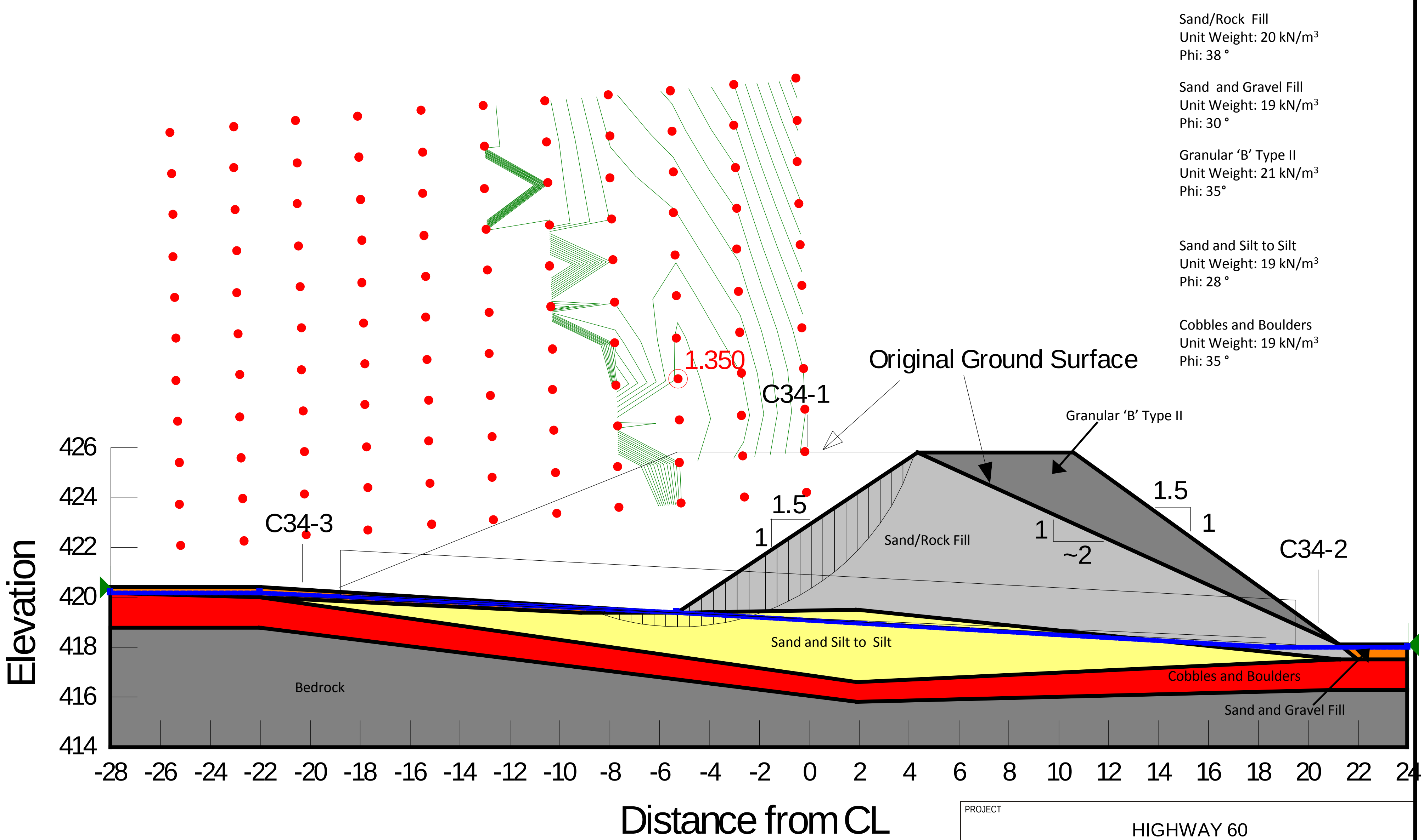
CADD	--			
CHECK	SEMC	APR 2012	**Figure 3a**	
REVIEW				



PROJECT

HIGHWAY 60
CULVERT AT STA 20+287

TITLE

STABILITY ANALYSIS
TEMPORARY CONDITION – STAGE 2

Golder Associates

PROJECT No. 09-1191-0062			FILE No. ----	
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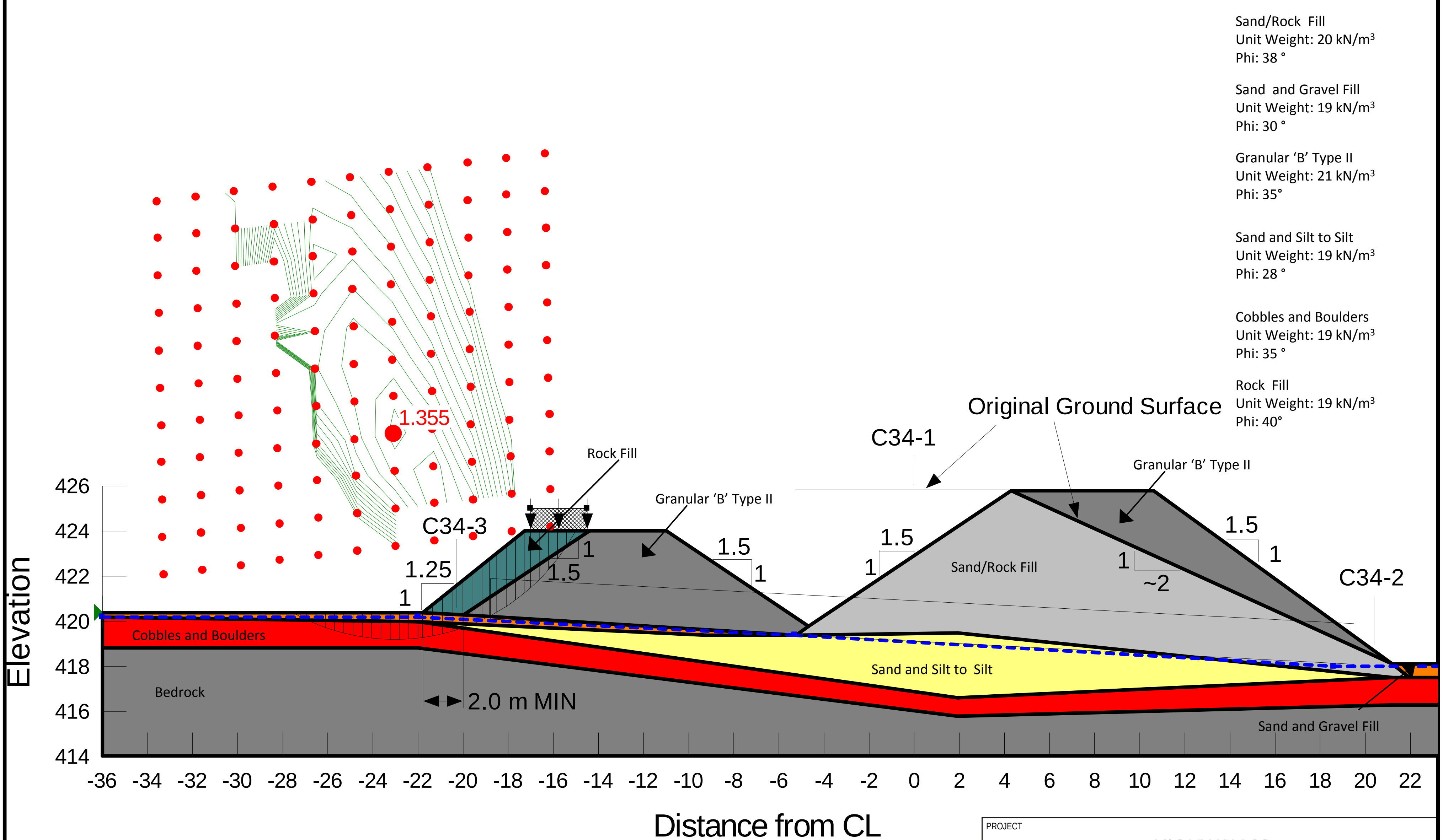
CADD	--			
CHECK	SEMC	APR 2012	**Figure 3b**	
REVIEW				



PROJECT

HIGHWAY 60
CULVERT AT STA 20+287

TITLE

STABILITY ANALYSIS
TEMPORARY DETOUR – STAGE 2

Golder Associates

PROJECT No. 09-1191-0062			FILE No. ----	
DESIGN	EC	APR 2012	SCALE AS SHOWN	REV.
CADD	--			
CHECK	SEMC	APR 2012	**Figure 3c**	
REVIEW				

APPENDIX A

Record of Boreholes and Drillholes

LIST OF SYMBOLS

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

1. GENERAL

π	3.1416
in x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	Factor of Safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
v	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

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

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

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

(b) Cohesive Soils
Consistency

	c_u, s_u **kPa**	c_u, s_u **psf**
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

Percent by Weight	**Modifier**	**Example**
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of Major discontinuities

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock Mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separation) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separation such as fractures, bedding planes and foliation planes or mechanically induced fractures 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	$\perp$	- Perpendicular To
FO	- Foliation / Schistosity	$\parallel$	- Parallel To
CL	- Cleavage	P	- Polished
SH	- Shear Plane / Zone	K	- Slickensided
VN	- Vein	SM	- Smooth
F	- Fault	R	- Rough
CO	- Contact	ST	- Stepped
J	- Joint	PL	- Planar
FR	- Fracture	U	- Undulating
MF	- Mechanical Fracture	C	- Curved

RECORD OF BOREHOLE No C34-1

 METRIC

PROJECT 09-1191-0062
W.P. 5551-04-00
DIST ___ HWY 60
DATUM Geodetic

LOCATION N 5044664.9; E 367267.9
BOREHOLE TYPE 108 mm I.D. Hollow Stem Augers, NW Casing, Wash Boring (Auto Hammer)
DATE August 25, 2011

ORIGINATED BY EHS
COMPILED BY EC
CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
425.6	GROUND SURFACE							
0.0	ASPHALT (60 mm)	1	AS	-				
	Sand and gravel (FILL)							
425.1 / 0.5	ASPHALT (320 mm)	2	AS	-		425		
	Sand, some gravel, trace to some silt (FILL) Dense Brown Moist	3	SS	37			o	19 70 (11)
424.2 / 1.4	Blast rock, in a sand and gravel matrix (FILL) Compact Brown and grey Moist	4	SS	15		424		
			RC	REC 40%		423		
	Spoon attempted at 4.0 m depth: No recovery.	-	SS	13		422		
420.7 / 4.9	Sand, some gravel, trace to some silt (FILL) Compact Brown Wet	5	SS	18		421	o	
		6	SS	10		420		
419.5 / 6.1	SAND and SILT to SILT, trace to some clay, trace gravel Compact to very dense Grey Wet	7	SS	40	∇	419		
		8	SS	13			o	1 58 37 4
		9	SS	31		418		
		10	SS	63		417	o	0 12 75 13
416.6 / 9.0	COBBLES and BOULDERS (as recovered in core barrel)	-	RC	REC 46%		416		
415.8 / 9.8	GNEISS (BEDROCK) Bedrock cored from 9.8 m depth to 12.9 m depth. For coring details see Record of Drillhole C34-1.	1	RC	REC 100%				RQD = 64%
		2	RC	REC 100%		415		RQD = 37%
		3	RC	REC 100%		414		RQD = 93%
412.7		4	RC	REC 100%		413		RQD = 100%
12.9	END OF BOREHOLE Note: 1. Water level at a depth of 6.1 m below ground surface (Elev. 419.5 m) upon completion of drilling.							

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100
Shear Strength kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100
Plastic Limit W_P, Natural Moisture Content W, Liquid Limit W_L; Water Content (%) 20 40 60
Unit Weight γ kN/m^3

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

PROJECT: 09-1191-0062

RECORD OF DRILLHOLE: C34-1

LOCATION: N 5044664.9 ;E 367267.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: August 25, 2011

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN - Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE	415.8			
10	NW; NQ Coring; August 25, 2011	GNEISS Fine to medium grained Fresh to slightly weathered Very strong Pinkish grey Sand infilling observed in broken rock zone between 10.4 m and 10.8 m depth.	9.8	1	JFOR JFOR JFOR JFOR JFOR JFOR BR	
11				2	JIR JFOR JFOR JFOR	UCS = 134 MPa
12				3	JFOR JFOR JFOR JIR JFOR JFOR JFOR	
			412.7 12.9	4	JFOR JFOR JFOR	
13		END OF DRILLHOLE				
14						
15						
16						
17						
18						
19						

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: EHS
CHECKED: SEMC

RECORD OF BOREHOLE No C34-2

 METRIC

PROJECT 09-1191-0062
W.P. 5551-04-00
DIST ___ HWY 60
DATUM Geodetic

LOCATION N 5044647.5; E 367276.9
BOREHOLE TYPE Portable Equipment, BW Casing, Wash Boring
DATE September 6-8, 2011

ORIGINATED BY MT
COMPILED BY EC
CHECKED BY SEMC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
418.2	GROUND SURFACE					
0.0	TOPSOIL					
0.1	Sand and gravel, trace to some silt, trace organics (FILL) Compact Brown and black Wet Spoon bouncing at 0.4 m depth.	1	SS	11	418	
417.5 / 0.7	COBBLES and BOULDERS No recovery in core barrel between 1.2 m and 1.9 m depth Spoon attempted at 1.7 m depth.		RC	REC 100%	417	
			RC	REC 0%		
416.3 / 1.9	GNEISS (BEDROCK) Bedrock cored from 1.9 m depth to 3.6 m depth. For coring details see Record of Drillhole C34-2.	1	RC	REC 100%	416	RQD = 0%
		2	RC	REC 51%	415	RQD = 0%
414.6 / 3.6	END OF BOREHOLE					

Column headings also include: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m^3; GR SA SI CL.

Note:

1. Borehole dry upon completion of drilling.

2. Split Spoon sample obtained by driving with a 1/2 weight hammer. SPT 'N' value has been adjusted to the inferred value that would be obtained using a standard weight hammer.

3. Additional three probe holes were advanced within a 1 m distance of this borehole and encountered refusal (i.e. spoon bouncing) between 0.1 m and 1.0 m depth.

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

PROJECT: 09-1191-0062

RECORD OF DRILLHOLE: C34-2

SHEET 1 OF 1

LOCATION: N 5044647.5 ;E 367276.9

DRILLING DATE: September 8, 2011

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Landcore Drilling Inc.

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate
BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage
PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular
PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break
BR - Broken Rock
NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	FLUSH	COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX METRES	DISCONTINUITY DATA: B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY k, cm/s	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE		416.3																	
2	BW; NQ Coring, September 8, 2011	GNEISS Coarse grained Slightly to highly weathered Pinkish grey		1.9	1																
3					2																
				414.6 3.6																	
		END OF DRILLHOLE																			

Depth scale: 2, 3, 4, 5, 6, 7, 8, 9, 10, 11

MTO-RCK 001 09-1191-0062 HWY 60 HDR.GPJ GAL-MISS.GDT 17/11/11 DATA INPUT:

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: MT

CHECKED: SEMC

Golder Associates

RECORD OF BOREHOLE No C34-3

 METRIC

PROJECT 09-1191-0062
W.P. 5551-04-00
LOCATION N 5044678.9; E 367251.0
ORIGINATED BY MT/EHS
DIST HWY 60
BOREHOLE TYPE Portable Equipment, BW Casing, Wash Boring
COMPILED BY EC
DATUM Geodetic
DATE September 7, 9 and 12, 2011
CHECKED BY SEMC

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
420.4	GROUND SURFACE						
0.0	TOPSOIL	1	SS	9	∇		
420.0 0.4	Sand and gravel, trace silt, trace organics (FILL) Loose Brown Wet Spoon bouncing at 0.4 m depth.		RC	REC 0%		420 419	
418.8 1.6	COBBLES and BOULDERS Spoon attempted at 0.8 m depth, spoon bouncing. No recovery in core barrel between 0.4 m and 1.6 m depth.	1	RC	REC 93%		418	RQD = 39%
	GNEISS (BEDROCK) Bedrock cored from 1.6 m depth to 4.9 m depth. For coring details see Record of Drillhole C34-3.	2	RC	REC 100%		417	RQD = 65%
		3	RC	REC 100%		416	RQD = 58%
415.5 4.9	END OF BOREHOLE						

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

Note:

1. Water level at a depth of 0.2 m below ground surface (Elev. 420.2 m) upon completion of drilling.

2. Split Spoon sample obtained by driving with a 1/2 weight hammer. SPT 'N' value has been adjusted to the inferred value that would be obtained using a standard weight hammer.

3. Additional four probe holes were advanced within a 2 m distance of this borehole and encountered refusal (i.e. spoon bouncing) between 0.15 m and 0.4 m depth.

SUD-MTO 001 09-1191-0062 HWY 60 HDR.GPJ GAL-MISS.GDT 17/11/11 DATA INPUT:

PROJECT: 09-1191-0062

RECORD OF DRILLHOLE: C34-3

LOCATION: N 5044678.9 ;E 367251.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 12, 2011

DRILL RIG: CME 55

DRILLING CONTRACTOR: Landcore Drilling Inc.

DATUM: Geodetic

JN - Joint, FLT - Fault, SHR- Shear, VN - Vein, CJ - Conjugate, BD- Bedding, FO- Foliation, CO- Contact, OR- Orthogonal, CL - Cleavage, PL - Planar, CU- Curved, UN- Undulating, ST - Stepped, IR - Irregular, PO- Polished, K - Slickensided, SM- Smooth, Ro - Rough, MB- Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		REFER TO PREVIOUS PAGE					
2	BW	GNEISS Coarse grained Fresh to slightly weathered Very strong Pinkish grey		418.8 1.6	1	JFOR, JFOR, JFOR, JIR, JFOR, JFOR, JFOR, JFOR, JIR, JFOR, JFOR, JIR, JIR, JIR, JFOR, JFOR	
3	NQ Coring September 12, 2011	Broken rock zone between 3.0 m and 3.05 m depth.			2	JFOR, JFOR, JFOR, JFOR, JFOR, JFOR, JFOR, JIR, JIR	UCS = 140 MPa
4				415.5	3	JFOR, JFOR, JFOR, JFOR, JFOR	
5		END OF DRILLHOLE		4.9			
6							
7							
8							
9							
10							
11							

Column headers also include: COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.25 METRES; B Angle; DIP w.r.t. CORE AXIS; Jr; Ja; Jn; HYDRAULIC CONDUCTIVITY k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

DEPTH SCALE

1 : 50

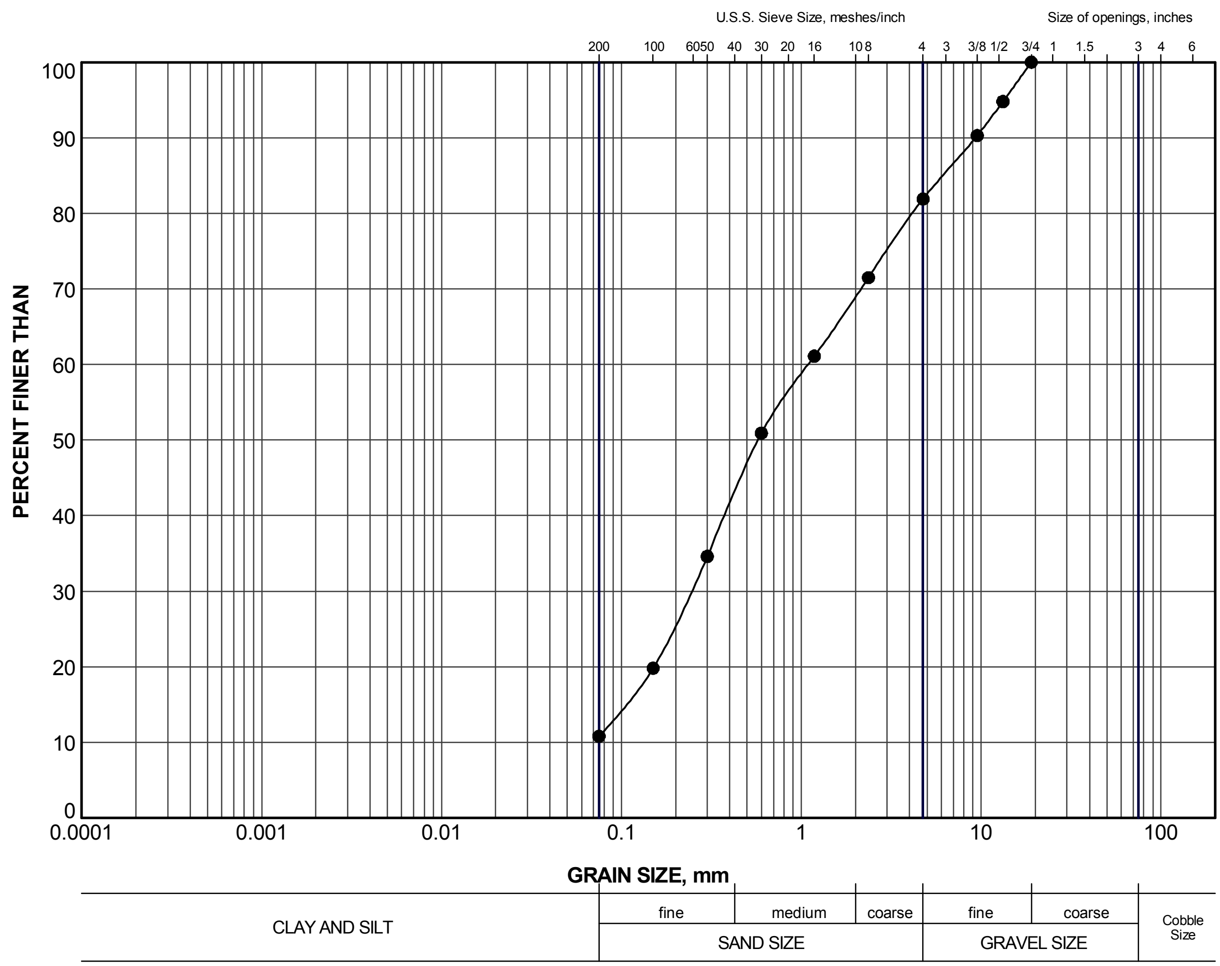
Golder Associates

LOGGED: MT/EHS

CHECKED: SEMC

APPENDIX B

Laboratory Test Results

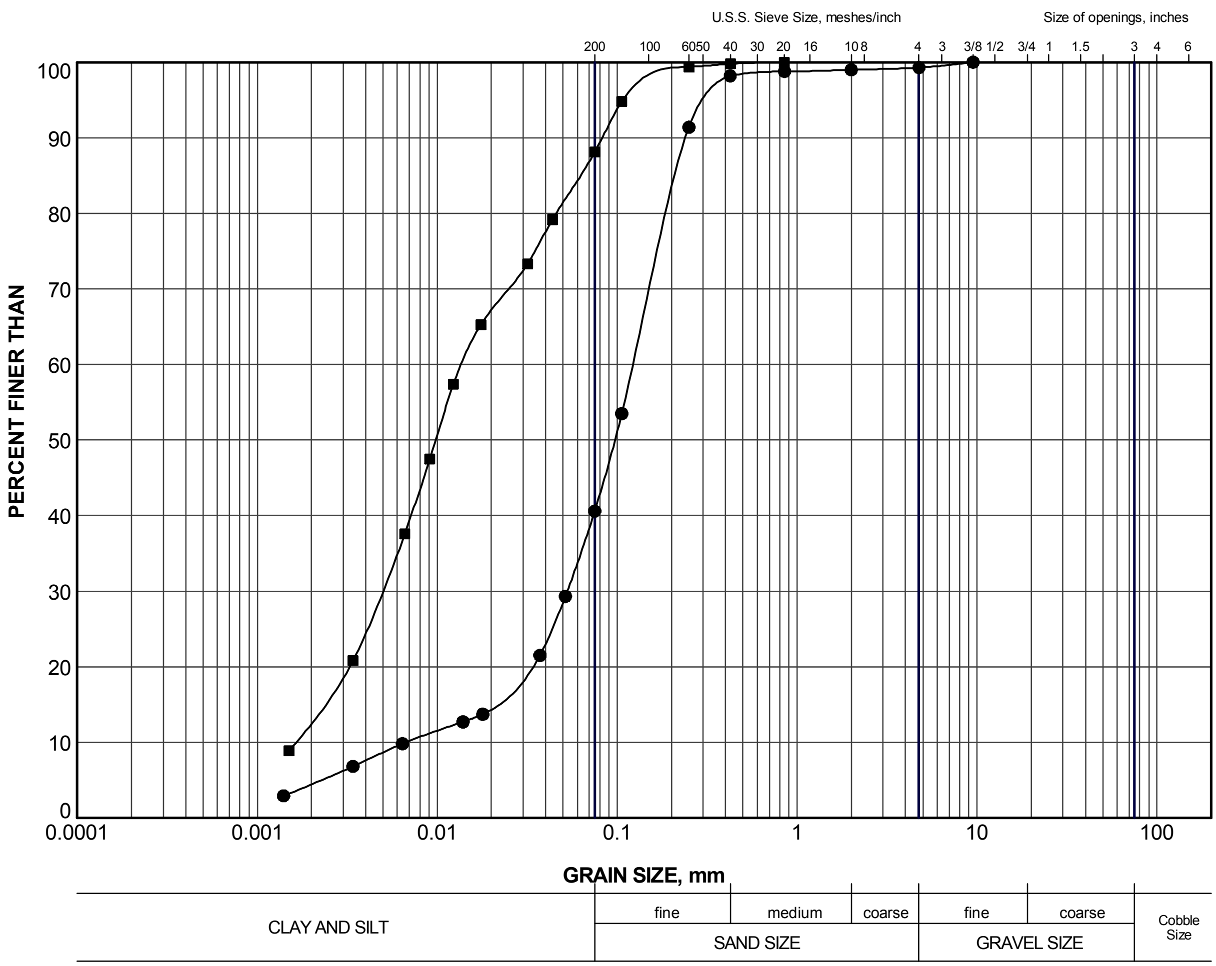


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C34-1	3	424.5

PROJECT	HIGHWAY 60 CULVERT AT STA 20+287			
TITLE	**GRAIN SIZE DISTRIBUTION** SAND (FILL)			
Golder Associates SUDBURY, ONTARIO	PROJECT No.	09-1191-0062		FILE No. 09-1191-0062 HWY 60 HDR.GPJ
	DRAWN	JJL	Nov 2011	SCALE N/A REV.
	CHECK	SEMC	Nov 2011	**FIGURE B1**
	APPR	JMAC	Nov 2011	

LDN_MTO_NEW GLDR_LDN.GDT



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	C34-1	8	418.4
■	C34-1	10	416.9

PROJECT

HIGHWAY 60
CULVERT AT STA 20+287

TITLE

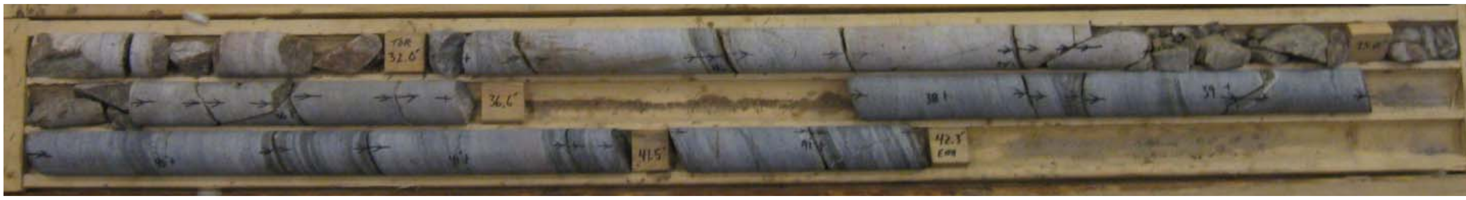
GRAIN SIZE DISTRIBUTION

SAND AND SILT TO SILT

PROJECT No.	09-1191-0062	FILE No. 09-1191-0062 HWY 60 HDR.GPJ	
DRAWN	JJL	Nov 2011	SCALE N/A	REV.
CHECK	SEMC	Nov 2011	FIGURE B2
APPR	JMAC	Nov 2011	

Golder Associates
SUDBURY, ONTARIO

LDN_MTO_NEW GLDR_LDN.GDT



C34-1: 9.8 m – 12.9 m

C34-2: 1.9 m – 3.6 m

C34-3: 1.6 m – 4.9 m

PROJECT
CULVERT AT STA 20+287, PECK TOWNSHIP
HIGHWAY 60

TITLE
BEDROCK CORE (Boreholes C34-1 to C34-3)

Golder Associates

PROJECT No.	09-1191-0062		FILE No. ----
DESIGN	EC	APR 2012	SCALE AS SHOWN REV.
CADD	--		
CHECK	SEMC	APR 2012	**FIGURE B3**
REVIEW	JMAC	APR 2012	

APPENDIX C

Non-Standard Special Provisions

GROUNDWATER CONTROL - Item No.

Non-Standard Special Provision

Construction of the new culvert will require excavations to extend below the groundwater level at the site. Cohesionless soils comprising sand and silt, silt or cobbles and boulders that are present below the groundwater table will slough, run, boil or cave into the excavation unless appropriate groundwater controls are in place. The Contractor is to design and install an appropriate dewatering system to enable construction in dry conditions, to prevent disturbance to the founding soils.

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

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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