Submitted To AECOM Canada Ltd.
189 Wyld Street Suite 103, North Bay, Ontario P1B 1Z2
On Behalf of the Ontario Ministry of Transportation

Culvert Replacement
Highway 118
Station 22+510 - Twp. of Dysart
GWP 5466-04-00

FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

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Englobe

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Final Foundation Investigation and Design Report

Prepared by:

Alexander Tepylo, P. Eng.
Project Engineer

LICENSED PROFESSIONAL ENGINEER S. HU 90524232 2017-02-03 PROVINCE OF ONTARIO

Sen Hu, P. Eng.
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER M. H. MacKAY PROVINCE OF ONTARIO

Reviewed by:

Michael H. MacKay, M. Eng., P. Eng.
Vice President – Expertise
Pavement Technology & Geotechnical Engineering
MTO Designated Contact

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Client:
AECOM Canada Ltd.
189 Wyld Street, Suite 103
North Bay, Ontario
P1B 1Z2
Attention: **Mr. Jason Wright, P. Eng.**

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1 INTRODUCTION

Englobe Corp. (Englobe), formerly LVM-Merlex, a Division of Englobe Corp., has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a foundation investigation at an existing centerline culvert site. The site is located at Station 22+510 in the Township of Dysart on Highway 118, some 310 m east of Burke's Road.

The foundation investigation location was specified by the MTO in the Terms of Reference for work under Agreement No. 5014-E-0004. The terms of reference for the scope of work are outlined in Englobe's Proposal P-14-199-R2, dated January 15, 2015. The purpose of this investigation was to determine the subsurface conditions in the area of the existing culvert for the contract preparation of the Detailed Design package. Englobe investigated the foundation area by the drilling of boreholes, carrying out in-situ tests, and performing laboratory testing on select samples.

After submission of the Final Foundation Investigation and Design Report (FIDR), Revision 1, on July 8, 2016, Englobe was requested in December 2016 to provide additional comments on potential trenchless technology options for installation of the replacement culvert in consideration of the existing traffic conditions on Highway 118; therefore the Final FIDR has been revised to include possible trenchless construction options for the culvert installation at the site.

2 SITE DESCRIPTION

A Corrugated Steel Pipe (CSP) culvert is located on Highway 118 at Station 22+510 in the Township of Dysart, Ontario. The topography in the area of this site is generally rolling. The existing highway embankment currently supports two undivided lanes of highway, running in a west-east direction. The existing highway, at the culvert location, is constructed on a granular mixed with rock fill embankment some 4.8 m in height (at centerline), with centerline elevation of 377.8 m at the culvert location. At the north slope, the maximum height of the embankment is some 5.8 m. At the south slope, the maximum height of the embankment is some 3.5 m. The existing embankment slopes, in the area of the culvert, have been generally established at an angle of approximately 1.3H:1V at the north slope and at an angle of approximately 2.2H:1V at the south slope. The culvert at this location is a 480 mm diameter Corrugated Steel Pipe (CSP) culvert, some 27.5 m in length. Flow through the culvert is from the south to the north (right to left).

Infrastructure at the culvert location consists of overhead wires to the right (south) side of the highway embankment.

2.1 SITE PHYSIOGRAPHY AND SURFICIAL GEOLOGY

This project is located in the Geomorphic Sub-province known as the Muskoka Ridges and Pockets. The topography on this section of Highway 118 is generally rolling. Layers of earth overlay bedrock. Organic materials were also observed in the region. Within the project area native overburden consists primarily of sands overlying bedrock.

Bedrock in the area, based on Ontario Geologic Survey (OGS) Map MRD-126, consists of migmatitic rocks and gneisses of undetermined protolith.

3 INVESTIGATION PROCEDURES

The fieldwork for this investigation was carried out during the period of May 13th to August 19th, 2015 during which time four (4) sampled boreholes, were advanced. Two (2) boreholes were advanced through the embankment. A single borehole was advanced at each of the inlet (south) and outlet (north) ends of the culvert, respectively.

The field investigation was carried out using a truck and bombardier mounted CME drilling rig equipped with hollow stem augers, standard augers, casing equipment and routine geotechnical sampling equipment. Soil samples were obtained at the borehole locations at regular intervals of depth using the standard 50 mm O.D. split spoon sampler advanced in accordance with the Standard Penetration Test (SPT) procedures (ASTM D-1586). The SPT method involves advancing a 50 mm O.D. split spoon sampler with the force of a 63.5 kg hammer freely dropping 760 mm. The number of blows per 300 mm penetration was recorded as the "N" value. If refusal to further advance of the augers, was encountered within the proposed depth of borehole, the boring was advanced through diamond drilling, using NQ size coring equipment. All samples taken during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and select laboratory testing.

Groundwater conditions in the open boreholes were observed during the advancement of and immediately following, completion of the individual boreholes. A 19 mm diameter standpipe was installed in selected open boreholes prior to backfilling to allow for further monitoring of the shallow groundwater levels. All open boreholes were backfilled upon completion with compacted auger cuttings in the general order they were removed, and where necessary, bentonite pellet backfill was added to the boreholes to bring them up to grade in accordance with requirements of Ontario Regulation 903. At the borehole(s) through the embankment, the upper portion of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface.

The fieldwork for this investigation was under the full time direction of a senior member of the Englobe engineering staff, Mr. Jame Lavigne, who was responsible for locating the boreholes, clearing the borehole locations of underground services, in-situ sampling and testing operations, logging of the boreholes, labeling and preparation of samples for transport to our North Bay laboratory, plus overall drill supervision. All samples received a visual confirmatory inspection in our laboratory. Laboratory testing of select samples included routine testing for natural moisture content determination and particle size analysis. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix 2), with a summary of results presented on the laboratory sheets in Appendix 3 (Figures Nos. L-1 to L-5 and Table No. L-6).

The location of the individual boreholes was determined in the field using highway chainage established by Callon Dietz Inc. (Callon Dietz) and offset relative to highway centerline. The MTO co-ordinates, northing and easting, were then established for the boring locations, using coordinates from MTM Zone 10, NAD 83 CSRS. The borehole elevations are based on coordinating the borehole locations with the Highway survey carried out by Callon Dietz. Elevations contained in this report are referenced to a geodetic datum.

4 SUBSURFACE CONDITIONS

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Records of Borehole Logs (Appendix 2) and on Drawing No. 2 (Appendix 3). Please note that stratigraphic delineation presented on the borehole logs and soil strata plot are the results of non-continuous sampling, response to drilling progress, the results of SPT, plus field observations. Typically such boundaries represent transitions from one zone to another and are not an exact demarcation of specific geological unit. Additional consideration should be given to the fact that subsurface conditions may vary markedly between adjacent boreholes and beyond any specific boring location, and are shown on the drawings for illustration purposes only.

4.1 CULVERT STATION 22+510, TOWNSHIP OF DYSART

A plan and profile illustrating the borehole locations and stratigraphic sequences is shown on Drawing No. 2, Appendix 3. During the course of the exploration program, four (4) sampled boreholes were put down at this site, with Borehole Nos. 1 and 2 advanced through the embankment, Borehole No. 3 advanced adjacent to the culvert inlet, and Borehole No. 4 advanced adjacent to the culvert outlet. At the time of the subsurface investigation, the ground surface elevations at Boreholes Nos. 1 to 4 were recorded at Elevations 378.1, 377.8, 374.9, and 370.1 m, respectively.

4.1.1 Pavement Structure

At Borehole No. 1, was advanced through the embankment where a pavement structure consisting of 125 mm crushed gravel was penetrated. At Borehole No. 2, a layer of asphalt some 50 mm thick was penetrated.

4.1.2 Embankment Fill

Underlying the crushed gravel and asphalt at Borehole Nos. 1 and 2, respectively, a layer of fill consisting of brown sand some gravel to gravelly, trace silt, mixed with rock fill, was penetrated. Cobble and boulder size rock fill were encountered at depths between 1.2 to 4.9 m below ground surface (Elevations 376.6 to 372.9 m). The natural moisture content measured on retrieved samples of this deposit was generally in the order of 2 to 12%. Gradation (sieve) analyses were carried out on three (3) samples of this deposit, the results of which indicated 33 to 34% gravel size particles, 53 to 59% sand size particles, and 8 to 9% silt and clay size particles (Figure No. L-1, Appendix 3). Based on SPT 'N' values of 5 to 22 blows per 300 mm penetration, the compactness/relative density of the granular portion of the deposit was described as loose to compact. This deposit was encountered to depths of 2.6 and 5.6 m below grade at Borehole Nos. 1 and 2, respectively (Elevations 375.5 and 372.2 m, respectively).

4.1.3 Organic Soils

At ground surface at Borehole Nos. 3 and 4, a layer of black silty organic soils was penetrated. This organic soil layer was encountered to approximate depths of 0.2 and 0.1 m below ground surface at Borehole Nos. 3 and 4, respectively (Elevations 374.7 and 370.0 m, respectively).

4.1.4 Sandy Silt

Underlying the granular fill at Borehole No. 1, and underlying the organic soil at Borehole No. 4, a deposit of sandy silt, trace gravel, trace clay was penetrated. Trace grass and rootlets were encountered in this deposit. The natural moisture content measured on samples of this deposit was in the order of 34 to 39%. A gradation (hydrometer) analysis was carried out on one (1) sample of this deposit, and the results of testing indicated 10% gravel size particles, 33% sand size particles, 55% silt size particles, and 2% clay size particles (Figure No. L-2, Appendix 3). Based on a SPT 'N' value of 2 to 14 blows per 300 mm penetration, the compactness/relative density of this deposit was described as very loose to compact. This deposit was encountered to depths of 3.7 and 0.5 m below ground surface at Borehole Nos. 1 and 4, respectively (Elevations 374.4 and 369.6 m).

4.1.5 Silty Sand

Underlying the organic soils at Borehole No. 3, a deposit of silty sand, trace gravel was penetrated. Trace grass and rootlets were encountered in this deposit. The natural moisture content measured on samples of this deposit was in the order of 10 to 19%. A gradation (hydrometer) analysis was carried out on one (1) sample of this deposit, and the results of testing indicated 2% gravel size particles, 61% sand size particles, 37% silt size particles, and 0% clay size particles (Figure No. L-3, Appendix 3). Based on SPT 'N' values of 9 to 34 blows per 300 mm penetration, the compactness/relative density of this deposit was described as loose to dense. This deposit was encountered to a depth of 2.3 m below ground surface at Borehole No. 3 (Elevation 372.6 m).

4.1.6 Sands

Underlying the sandy silt at Borehole Nos. 1 and 4, and underlying the fill at Borehole No. 2, a deposit of brown to grey sand trace to some gravel, some to with silt, trace clay was penetrated. The natural moisture content measured on samples of this deposit was in the order of 12 to 18%. Gradation (sieve) analyses were carried out on three (3) samples of this deposit, and the results of testing indicated 12 to 18% gravel size particles, 58 to 63% sand size particles, and 19 to 30% silt and clay size particles (Figure No. L-4, Appendix 3). Gradation (hydrometer) analyses were carried out on four (4) samples of this deposit, the results of which indicated 2 to 19% gravel size particles, 67 to 87% sand size particles, 10 to 24% silt size particles, and 0 to 3% clay size particles (Figure No. L-4, Appendix 3). Based on SPT 'N' values of 13 to 43 blows per 300 mm penetration, the compactness/relative density of this deposit was described as compact to dense, generally compact. This deposit was encountered to depths of 6.1, 9.1, and 3.7 m below grade at Borehole Nos. 1, 2, and 4, respectively (Elevations 372.0, 368.7, and 366.4 m, respectively).

4.1.7 Gravelly Sands

Underlying the sand at Borehole No. 1, a deposit of grey to brown gravelly sand, some silt was penetrated. The natural moisture content measured on samples of this deposit was in the order of 9%. A gradation (sieve) analysis was carried out on one (1) sample of this deposit, and the results of testing indicated 38% gravel size particles, 46% sand size particles, and 16% silt and clay size particles (Figure No. L-5, Appendix 3). This deposit was encountered to a depth of 6.4 m below grade at Borehole No. 1 (Elevation 371.7 m).

4.1.8 Bedrock

Underlying the gravelly sands at Borehole No. 1, underlying the sands at Borehole Nos. 2 and 4, and underlying the silty sands at Borehole No. 3, bedrock was proven by diamond core drilling. The bedrock was described as black gneiss bedrock. Based on RQD values of 40 to 89% the bedrock was described as poor to good quality. Based on visual review, the bedrock generally showed negligible weathering. Sampling in the bedrock was terminated at depths of 9.6, 12.2, 5.7, and 6.7 m below grade at Borehole Nos. 1 to 4, respectively (Elevations 368.5, 365.6, 369.2, and 363.4 m, respectively). It should be noted that, when encountered, the underlying bedrock surfaces in this area can be very erratic in nature, varying substantially in elevation over short horizontal distances.

4.2 GROUNDWATER DATA

At the time of this investigation (May 13th and 14th, and August 18th to 19th, 2015), surface water was not observed at the culvert.

Measurements of the groundwater table and cave-in levels were undertaken, where possible, in the open boreholes during the advance of the individual borings and upon completion. A standpipe was installed in Borehole Nos. 1 and 3 to obtain post borehole completion water levels. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B).

The water levels were measured at Elevations 372.6, 371.7, 371.3, and 368.9 m at Borehole Nos. 1 to 4, respectively.

The groundwater and surface water levels will fluctuate seasonally/yearly.

5 DISCUSSION AND RECOMMENDATIONS

5.1 GENERAL

A foundation investigation was carried for the proposed replacement of a CSP culvert as identified by the MTO.

The existing culvert, located at Station 22+510, in the Township of Dysart, is a 480 mm diameter CSP culvert some 27.5 m long. The existing culvert invert, at centerline, is estimated at a depth of some 4.8 m (Elevation 373.0 m). The culvert invert at the inlet and outlet are established at elevations 374.3 and 372.0 m, respectively. The existing highway embankment currently supports two undivided lanes of highway, running in a west-east direction. Flow through the culvert is from the right to the left (south to north). Based on data from this foundation investigation, the embankment supporting the existing pavement structure at this site has been constructed using a granular pavement structure overlying sand fills mixed with rock fill. The native material, underlying the embankment fill, generally consisted of compact sandy silts, silty sands and sands.

The type of culvert (concrete, CSP, or High Density Polyethylene (HDPE)) to replace the existing culverts is currently unknown. However, it is understood that the new culvert will be constructed along a similar skew and alignment. It is further understood that the final vertical alignment of the highway will remain essentially the same.

After submission of the Final Foundation Investigation and Design Report (FIDR), Revision 1, on July 8, 2016, Englobe was requested in December 2016 to provide additional comments on potential trenchless technology options for installation of the replacement culvert in consideration of the existing traffic conditions on Highway 118; therefore the Final FIDR has been revised to include possible trenchless construction options for the culvert installation at the site.

5.1.1 Frost Penetration

Generally, culverts within the depth of frost penetration below the pavement structure are included in the pavement structure frost treatment (see OPSD 803.010 and OPSD 803.030). However, closed culverts are not designed in consideration of frost penetration below the culvert. Culverts with footings, (i.e. open culverts, culvert retaining walls, etc.) require the footings to be designed for frost penetration.

At this site, the frost penetration depth below cleared pavement surfaces is approximately 1.8 m. The culvert at this location is not located within the depth of frost penetration below the pavement surface, as such, will not require frost treatments.

5.2 FOUNDATION CONSIDERATIONS

The founding native sands present below the existing embankment are considered adequate for support of a culvert and for a conventional highway embankment of this height.

Geotechnical bearing resistance should not be a major issue provided the natural bearing surface is not disturbed during construction and groundwater is controlled throughout construction, as discussed in Section 5.6.

Based on the characteristics of the native sand subgrade present below the culverts, the response of the existing embankment, and a founding elevation and culvert size similar to that of the existing culvert (i.e. 480 mm diameter), a factored bearing resistance at ULS of 450 kPa can be used for a closed culvert (i.e. precast concrete frame box culvert or CSP culvert). In consideration of the width of the culvert, depth of overburden, and response of the existing embankment, a geotechnical reaction at SLS of 150 kPa, established at a depth of 1.5 m, founded on the compact native sands and silts, and silts) can be used for design, in consideration of 25 mm settlement.

5.2.1 Slope Stability

The maximum height of the embankment above the stream bed at this location is some 4.8 m at centreline, and up to some 5.8 m at the north side of the embankment. A stability analysis, using the GEO-SLOPE computer program, Slope/W (GeoStudio 2007, version 7.17, Geo-Slope International Ltd.), was carried out at this location with the existing embankment slopes of some 1.3H:1.0V in granular and rock fills. For the purposes of these analyses, the materials were modeled using the following parameters;

PARAMETER	MATERIAL			
	EMBANKMENT FILL ABOVE ELEVATION 376.5 m	EMBANKMENT FILL BELOW ELEVATION 376.5 m	GRANULAR FILL	ROCK FILL
Unit Weight (kN/m^3)	19.0	20.0	20.0	22.0
Effective Friction Angle (degrees)	32	35	32	43
PARAMETER	**MATERIAL**			
	SANDY SILT	**SILTY SAND**	**SAND**	**GRAVELLY SAND**
Unit Weight (kN/m^3)	19.0	18.0	19.0	20.0
Effective Friction Angle (degrees)	32	30	32	35

The unit weights and friction angles for the slope calculations are based on general representative values for the various soil types, obtained through laboratory testing and tactile analysis. The groundwater levels used for the analyses are shown on Figure Nos. F-1 to F-4, Appendix 5. The results of the analyses indicate factors of safety in the order of 1.0 on the existing embankment slopes, against minor surficial slippage on the embankment (see Figure No. S-1, Appendix 5). The factor of safety against long term deep seated failures is in the order of 1.1 with the existing slopes (see Figure No. S-2, Appendix 5). It is recommended that the finished slopes are established at angles of 1.25H:1V or flatter in rock fill, or 2H:1V in granular, earth, and mixed fills (such as those encountered in the embankment at this site). The factor of safety of embankment slopes established at angles of 2H:1V are in the order of 1.4 (see Figure Nos. S-3, Appendix 5). The highway Right of Way at the culvert location is wide enough to allow construction of embankment slopes at an angle of 2H:1V. Constructing the embankment slopes at an angle of 2H:1V would require widening at the embankment toe and extending the culvert by approximately 4 m. Alternatively, the toe of the embankment can be cut back and replace with rock fill. This would allow the existing slopes to be maintained, if treated to prevent surficial erosion. The factor of safety of embankment slopes established at angles of 2H:1V are in the order of 1.4 (see Figure Nos. S-4, Appendix 5). Reconstructing the embankment toe with rock fill would require widening at the embankment toe and extending the culvert by approximately 2 m.

Proprietary slope stabilization systems, such as mechanically stabilized earth or soil nailing systems could also be considered as an alternative to constructing the slopes at shallower angles.

Lower factors of safety will occur during excavation and backfilling as discussed in Section 5.5. Short term stability should not be an issue if construction is carried out as described herein.

5.3 CULVERT DESIGN, BEDDING, AND EMBEDMENT

The embankment generally consists of granular fills mixed with rock fill. The results of this investigation indicate that, below the culvert invert, the native subgrade soils at Borehole Nos. 1 to 4 consisted of generally compact sandy silts, silty sands, and sands. A review of the condition of the pavement surface, at the culvert locations, revealed that the embankment appears to have performed well. The existing embankment has preloaded the soils at the culvert locations and since there will be no change in the height of the embankment, and therefore no increases in embankment load, no appreciable settlement of the embankment is anticipated. As such, installing the culverts on a camber will not be required at this site.

5.3.1 Rigid Concrete Culvert

Concrete pipes can be considered for culvert replacement at this site. A Class B Bedding for the concrete pipes shall consist of Granular A with a thickness of 300 mm. Alternatively, specifically if construction is carried out under wet conditions, a bedding and levelling course consisting of 19 mm clear stone per OPSS.PROV 1004 should be used, which would aid in dewatering operations. During backfilling, the material of bedding and cover shall be placed in uniform layers not exceeding uncompacted thickness of 200 mm. The elevation difference of backfilling for bedding and cover on either side of the rigid pipe shall be limited to a maximum 200 mm per OPSS.PROV 401. Cover material for concrete pipes can consist of Granular A and placed to the dimensions as shown on OPSD 802.031. If circular concrete pipes are used, compaction of the haunch is critical and should be constructed in accordance with OPSS.PROV 501.

Considering the size of the culvert, a precast concrete rigid frame box culvert is not considered appropriate for this site.

The inlet and outlet stream bed shall be protected with a rip-rap (R-50 size as per OPSS.PROV 1004) apron. The apron shall be 3 m in length, a minimum 400 mm thick and extend across the stream bed to 3 m beyond the outside edges of the culvert. Clay seals are generally used only where significant head differences exist between the inlet and outlet of the culverts to prevent flow through the bedding/embedment granulars. In consideration of the culvert size and anticipated flow, clay seals are not considered necessary at this location, provided embedment/bedding materials are properly compacted in the haunch area and rip rap over a Class II geotextile is placed around the inlet end of the culvert.

5.3.2 Flexible Culvert

Flexible culverts (i.e. CSP/SPCSP/HDPE) can also be considered for culvert replacement at this site. If flexible pipes are used for replacement, embedment material should consist of Granular B Type I per OPS.PROV 1010 provided the maximum size of stone inclusions is limited to 25 mm or less in size and placed in accordance with OPSD 802.010 for a Type 3 soil. The material in the haunch area must be compacted to 100% Standard Proctor Maximum Dry Density prior to placing the remainder of the embedment material. During backfilling, the embedment (bedding and cover) material shall be placed in uniform layers not exceeding uncompacted thickness of 200 mm. The elevation difference of the embedment fill on either side of the flexible pipe must be limited to a maximum 200 mm per OPSS.PROV 401. The backfill should be placed to a minimum depth of 900 mm above the crown of the pipe before power tractors or rolling equipment can be used for compacting per OPSS.PROV 401.

In consideration of the culvert size and anticipated flow, clay seals are not considered necessary at this location, provided embedment/bedding materials are properly compacted in the haunch area and rip rap over a Class II geotextile is placed around the inlet end of the culvert. The inlet and outlet stream bed shall be protected with a rip-rap (R-50 size as per OPSS.PROV 1004) apron. The apron shall be 3 m in length, a minimum 400 mm thick and extend across the stream bed to 3 m beyond the outside edges of the culvert.

5.4 CULVERT INSTALLATION AND CONSTRUCTION STAGING CONSIDERATIONS

At the centerline of highway, the invert elevation of the existing culvert is at 373.0 m, with the top of the embankment at elevation 377.8 m. The culvert inverts at the inlet and outlet are established at Elevations 374.3 and 372.0 m, respectively. As such, the embankment at this location is some 4.8 m in height above the culvert invert at the centerline. Therefore, a minimum 5.1 m deep excavation (i.e. to Elevation 372.7 m) will be required (at centerline) in consideration of a 300 mm thick layer of bedding/embedment material. At the outlet end of the culvert, the excavations will be required to depths of some 5.8 m (Elevation 371.7 m). The present platform width at this location is some 12 m as can be seen on the cross section on Drawing No. 2. The platform width at this location, as is, will not be sufficient to carry out an open excavation using staged construction unless local lowering of the grade and/or sliver widening is undertaken. In general, an open cut excavation can be considered if the platform is temporarily lowered by some 1.8 m (at centerline). If this lowering cannot be accommodated then consideration can be given to a combination of lowering and widening or to constructing a temporary vertical wall for use as a protection system.

5.4.1 Staged Construction

As noted, the platform at this location, as is, is of insufficient width to carry out an open excavation using staged construction, unless temporarily lowering the vertical alignment is carried out. The culvert is located on a horizontal curve which may also limit using a staged construction. To carry out an open cut excavation, locally lowering the grade to allow for staged construction using staged sequencing and limiting traffic flow to one lane would be required (see Figure No. SK-3, Appendix 5).

A possible staging plan for a continuous open cut excavation under a 24/7 traffic control operation, as shown on Figure No. SK-3, Appendix 5, is as follows:

- Locally lower the grade at the culvert to an elevation of approximately 376.0 m.
- Limit traffic to a single lane on the left, with a minimum platform width of 6 m, under 24/7 traffic control.
- Open cut excavate, to the right, and install approximately 13 m of new culvert.
- Reconstruct the embankment on the right, with a minimum platform width of 6 m for traffic.
- Divert the single lane of traffic to the right and continue open excavation to install the remainder of the culvert on the left.
- As the width of the platform increases on the right, the vertical alignment can be raised, and the traffic can revert back to two lanes when sufficient width permits.

It should be noted that additional subsurface information may be required if widening beyond the existing embankment toe is required due to possible variation of subsurface conditions.

5.4.2 Protection System

As noted above, consideration could be given to constructing a vertical wall, along centerline, for use as a temporary protection system.

The installation of a protection system for use in the culvert replacement operation will require penetration through some 5.0 m of cohesionless fills mixed with rock fill. The embankment fill is generally underlain by compact sands, and as cobble and boulder size rock pieces were encountered in the embankment fills. Considering the presence of rock fill in the embankment, advancing a temporary retaining system (i.e. driven sheet piles) through the rock fill may be problematic. A notice to Contractor indicating the presence of the cobble/boulder size rock fill in the embankment has been included in Appendix 5. Several approaches to constructing a protection system are described below in the following. See Table A, Appendix 5, for advantages and disadvantages for the different type of protection system considered for this site. The conceptual shoring location is illustrated on Figure No. SK-4, Appendix 5.

One method to construct a protection system would be to penetrate the rock fill in the embankment with H piles (soldier piles) extending into the underlying sands and/or into bedrock and install lagging. Pre-drilling may likely be required to advance the H piles through the rock fill and into the underlying the bedrock. The H piles would be installed at an interval of 2.5 to 3 m apart and the lagging would be installed as the excavation progresses. A waler and raker system or tie back anchor system would have to be installed as the excavation advances. The contractor must be prepared to address large pieces of rock fill and control groundwater as the excavation progresses, without compromising the adjacent active lane of traffic.

The resistance (R) for grouted anchors, located outside the active failure wedge, in cohesionless soils can be estimated from the following equation as supplied in the Canadian Foundation Manual (4th Edition):

$R = \sigma'_z * A_s * L_s * \alpha_g$ Where: σ'_z = effective vertical stress at the midpoint of the load carrying length

As = effective unit surface area of the anchor

Ls = effective embedment length of the anchor

α_g = anchorage coefficient use 1.0 for granular backfill

Unless the pull-out resistance (capacity) of the anchor is proven with a load test program, the allowable anchor load (as suggested by the Canadian Foundation Engineering Manual, 4th Edition), is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3. Alternatively, proprietary anchor systems can be used.

Alternatively, a caisson wall or drilled micropile system with an intermediate support system of reinforced shotcrete, to act as lagging, could be considered as alternatives for roadway protection at this site. One method of constructing this system would be to drill in micropiles, advancing to either side of the culvert below the invert several meters into the compact to dense sands or probably into the bedrock, depending upon the size and capacity of the micropiles. Over the actual culvert location, the piles would be carried down to top of culvert grade followed by bracing, with a suitably sized waler and anchorage system, tied into the full depth piling at the culvert sides, in order to provide support at the top of the piling over the culvert barrel. Depending on the section properties of the retaining structure, walers and bracing struts or ground anchor support systems will probably be required. As the excavation progresses downward in 1 to 1.2 m lifts, a reinforced shotcrete, tied into the piles, is applied. Once one half of the box culvert construction is complete, a system of buried anchors could be installed to tie back the micropiles as the highway fill is brought up to grade. When the excavation on the opposite side reaches the anchor depths, a support waler, if required, can be placed and tensioned to support the shotcrete as specified in the contractor's approved shoring design. However these shoring system are generally more costly, as such are not recommended at this site.

The contractor's shoring/protection system design must be carried out by a geostructural engineer with appropriate experience.

A table outlining the possible temporary excavation protection/flexible retaining systems and their relative advantages, disadvantages and costs, as well as comments on the viability of the methods is provided in Table A, Appendix 5. A conceptual shoring location is illustrated on Figure No. SK-4, Appendix 5.

The protection system can be designed using the lateral earth pressure parameters as outlines in Section 5.5.

Considering the cohesionless nature of the embankment fills (granular pavement structure overlying a granular fill and rock fill mix) a rectangular apparent pressure distribution over the height of the cut would be appropriate for design of the temporary shoring. The width of the apparent rectangular pressure distribution, over the height of excavation, can be considered equal to $0.65*K_a*\gamma*H$, where:

K_a = active earth pressure,

γ = unit weight, and

H = height of wall above the base of excavation.

The temporary protection system should be designed and constructed to comply with OPSS 539. In consideration of the location of the protection system and traffic volume, a Performance Level 2 is considered appropriate.

5.4.3 Trenchless/Tunnelling Techniques

The boreholes through the embankment indicate that the embankment fills consisted primarily of sands, trace silt, with varying gravel content. Cobble and boulder size obstructions were encountered within the existing embankment fills between Elevations 372.9 and 376.6 m in Borehole No. 2, adjacent to the existing culvert. The embankment is approximately 4.8 m in height at the centreline of highway.

A trenchless construction approach for the proposed 480 mm diameter culvert replacement would eliminate the need for open cuts and/or roadway protection systems, and associated traffic delays on existing highway during construction. Several trenchless technologies are feasible for the project as outlined in the table below.

The following table summarizes the general advantages and disadvantages of the different trenchless techniques for potential consideration at this site.

METHOD	ADVANTAGES	DISADVANTAGES
Horizontal Directional Drilling	• Can be used in most ground conditions • Generally does **not** require staging pits therefore minimal ground water control required • Alignment can be adjusted to avoid obstructions • New pipe size within the practical construction limits between 140 and 1200 mm	• Site grades may require longer bore or staging construction shafts • Larger drilling equipment may be required • Requires drilling fluid to maintain the bore, which could result in heave
Symmetrix Drilling	• Can be advanced through bedrock, and most overburden types • has been used to advance casings through many rock fills in Scandinavian countries • New pipe size within the practical construction limits between 140 and 1200 mm	• May require staging pits for horizontal drilling • Contractor availability; not in general use in Ontario at this time
Jack and Bore	• Good contractor availability • Good for shorter tunnel lengths (less than 100 m) • Good gradient control • New pipe size within the practical construction limits between 200 and 1500 mm	• Requires construction shafts • Groundwater control may be required for the bore and construction shafts • Elevated potential for ground subsidence • Larger boring diameter required to allow removal occasional cobbles/boulders • Not well suited for use in rock fills or if there is a high concentration of large obstructions
Pipe Ramming	• Minimal groundwater control required along the installation route (unless required to remove obstruction/old pipe) • Can penetrate soils containing cobbles/boulders if obstruction less than casing diameter • New pipe size within the practical construction limit of 2 m	• Installation problems can occur in soils with cobble/boulders • Requires staging construction shafts • Groundwater control may be required at construction shafts • Possible ground displacement/heaving in the soils above the crown

As noted, obstructions due to the presence of cobble/boulder sized rock fragments were encountered within the embankment fills. These obstructions could limit the feasibility of some of trenchless installation methods at this site. As such, it is recommended that additional probes be advanced along the proposed alignment of the replacement culvert if trenchless technologies are used to confirm the constructability of the proposed construction method.

To date, there are no known projects in Ontario where trenchless methods have been successfully used to install casing through soils having high concentrations of obstructions. However, the Symmetrix System, as summarized above, has proven to be successful in advancing horizontal holes up to 1,200 mm diameter through rock fill. This technology has been used by Bermingham Foundation Solutions for deep foundation construction in Ontario and for over 8 years in Scandinavian countries, and as such, it could be considered for potential use at this site.

Pipe Ramming could also be considered for advancing a heavily-reinforced casing through fills that include obstructions. However, to advance the pipe, the casing diameter must be large enough to allow hand mining operations to be carried out at the face to remove large pieces of rock that cannot be swallowed into the advancing casing. Generally, a minimum 1.2 m diameter is required to have sufficient room to hand mine large rock.

Jack and Bore is a common trenchless construction method of advancing a culvert. However, considering the presence of cobble and boulder sized obstructions, Jack and Bore is not considered to be a suitable method for culvert installation at this site and hence, it will not be discussed further.

It is understood that the preferred method of trenchless culvert replacement is to install a new culvert along the same alignment using the Pipe Ramming methods (e.g. the “culvert swallowing replacement” method to swallow and crush the existing culvert using specialized equipment) or by Pipe Bursting. The pipe swallowing method involves ramming a larger size steel casing around the existing culvert, following which the existing culvert is then removed using the specialized equipment. Pipe Bursting involves ramming a bursting tool to split the existing culvert, while pulling a new culvert of the same diameter to replace the existing. Pipe Bursting can be difficult through the corrugated steel culvert and may result in significant vibrations and possible displacement/heaving of the soils above the crown of culvert; however the Pipe Bursting method may be possible, depending on the equipment and methodology proposed by the Contractor.

Staging construction shafts will be required for Pipe Ramming operations. Groundwater levels were encountered between Elevations 371.3 and 368.9 m at Borehole Nos. 3 and 4 located adjacent to the inlet and the outlet of existing culvert, respectively, during the site investigation period. Based on the groundwater levels encountered at Elevations 372.6 and 371.7 m at Borehole Nos. 1 and 2, respectively, located through the existing embankment during the site investigation, construction dewatering may be required for the proposed excavations at construction shafts per Section 5.6. It should be noted that the ground water/surface water levels will fluctuate seasonally/yearly. The trenchless construction should meet requirements of OPSS 415, 416 and/or 450, depending on the proposed trenchless technique.

5.5 LATERAL EARTH PRESSURES

Lateral earth pressures should be computed in accordance with the Canadian Highway Bridge Design Code (CHBDC). The parameters for bedding, cover, embedment and backfill materials are based on compaction levels of 100% Standard Proctor Maximum Dry Density (SPMDD). The design parameters for the bedding/embedment and backfill materials are as follows:

PARAMETER	GRANULAR A	GRANULAR B TYPE I	EMBANKMENT FILL ABOVE ELEVATION 376.5 m	EMBANKMENT FILL BELOW ELEVATION 376.5 m	SANDY SILT
Unit Weight (kN/m^3)	22.8	21.2	19.0	20.0	19.0
Angle of Internal Friction	34°	31°	32°	35°	32°
Coefficient of Active Earth Pressure (K_a)	0.28	0.32	0.31	0.27	0.31
Coefficient of Passive Earth Pressure (K_p)	3.54	3.12	3.23	3.69	3.23
Coefficient of Earth Pressure at Rest (K_o)	0.44	0.48	0.47	0.42	0.47
PARAMETER	**SILTY SAND**	**SAND**	**GRAVELLY SAND**		
Unit Weight (kN/m^3)	18.0	19.0	20.0		
Angle of Internal Friction	30°	32°	35°		
Coefficient of Active Earth Pressure (K_a)	0.28	0.31	0.27		
Coefficient of Passive Earth Pressure (K_p)	3.57	3.23	3.69		
Coefficient of Earth Pressure at Rest (K_o)	0.44	0.47	0.42		

For rigid structures, such as a precast concrete culvert, deflection cannot occur, as such the "at-rest" condition (K_o) applies. For flexible structures, such as CSP/HDPE culverts, deflection can occur, as such the "active" condition (K_a) applies.

5.6 EXCAVATION, DEWATERING, AND EMBANKMENT RECONSTRUCTION

All temporary excavations greater than 1.2 m in depth must, at a minimum, be sloped or shored in accordance with the Occupational Health and Safety Act Regulations for Construction Projects. The embankment material, above the water table, is considered a Type 3 soil as defined in the Occupational Health and Safety Act and Regulations for Construction Projects. Temporary open excavations above the groundwater table, could be cut back at an angle of 1H:1V, provided they are monitored continuously; however, below the groundwater table, the side slopes in fill an/or native materials will have to be cut back to an angle of 2H:1V, possibly shallower, dependent upon the Contractors' chosen method of controlling the groundwater.

The excavation backfill above the culvert bedding/cover should consist, at a minimum, of a granular fill meeting OPSS for Select Subgrade Material (SSM).

Final (permanent) embankment side slopes in granular fills should be established to match the existing slopes or as per OPSD 200.010. Final slopes should be treated with a seed and mulch to prevent ravelling.

Bedrock was not encountered at the borehole locations within the anticipated depth of excavation, therefore bedrock excavation and/or blasting operations are not anticipated.

Excavations must be maintained in a dewatered condition during excavation and foundation construction, and every reasonable effort must be made to prevent disturbing (piping/boiling) at the founding subgrade. Groundwater control, in accordance with OPSS 517 and 518, will be required to maintain a stable subgrade during culvert installation.

At the time of investigation, no surface water was encountered at the culvert location. Groundwater was encountered at Elevations of 372.6, 371.7, 371.3, and 368.9 m at Borehole Nos. 1 to 4, respectively. Excavations to minimum elevations 374.1 and 371.7 at the culvert inlet and outlet, respectively, will likely be required to install the culvert and bedding. As such dewatering may not be required during excavation and culvert installation. However, it should be noted that the ground water/surface water levels will fluctuate seasonally/yearly.

During construction, installation of filtered sumps and pumping from the base of the excavation will, at a minimum, be required to maintain the excavation in a dewatered condition during subgrade preparation and culvert installation. The effectiveness of this method of groundwater control would be limited to conditions where the prevailing groundwater table is less than some 1 m above the final excavation depth. If the excavation must penetrate to a greater depth below the prevailing groundwater table a more effective groundwater control method, such as a vacuum well point system, or sheet pile cut-off wall, should be considered by the contractor to maintain a stable excavation base.

A cofferdam, constructed of earth fill, sand bags, or water filled bag (i.e. aquadam) can be considered at this site. Steel sheet piles may also be considered for controlling stream flow, however, the shallow bedrock at the inlet (i.e. at depth of 2.3 m below ground surface at Borehole No. 3), may limit the penetration of a steel sheet pile type cofferdam. For base design, sheet piles should extend a minimum depth below base of proposed excavation equal to the height of water above the base of excavation. By-pass pumping can be carried out to divert the stream flow at the time of construction. It is recommended that by-pass pumping, through a temporary culvert installed through the embankment, be carried out to divert the stream flow past the work area isolated with the cofferdam system.

Ultimately, the method of excavation, dewatering, and stream flow diversion will be the choice of the contractor; however the importance of maintaining the subgrade in a dewatered stable condition during excavation and construction operations cannot be stressed enough.

5.7 CONSTRUCTION CONCERNS

Considering the nature of the granular mixed with rock fill embankment, issues with driving closed sheeting may develop. The Contractor must be prepared to excavate and advance protection systems through the mixed embankment fill including cobble and boulder size rock fill for the proposed open cut and/or trenchless construction.

The Contractor must also be prepared to deal with seasonal and yearly fluctuations of ground/surface water.

6 STATEMENT OF LIMITATIONS

The design recommendations given in this geotechnical report are applicable only to the project described in the text and only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions may however, vary from those assumed, in which case changes and modifications may be required to our geotechnical recommendations. We recommend, therefore, that we be retained and provided the opportunity during the design stage to review the design drawings, site survey information, proposed elevations, etc. to verify that they are consistent with our recommendations or the assumptions made in our analysis. It is further recommended that we be retained to review the final design drawings and specifications relative to the geotechnical recommendations.

If, during construction, conditions in the field vary from those assumed at the design stage, an engineer from this office must be notified immediately.

Proper subgrade preparation, groundwater control, compaction, etc. are all critical aspects of the bearing capacity of native soils. It must be noted that different aspects of the geotechnical design are based on the assumption that Englobe will be retained during site preparation and construction of the proposed works to ensure that both the geotechnical site characteristics and the construction operations/techniques are consistent with our recommendations. Should Englobe not be involved during the full construction phase, our liability is strictly limited to the factual information contained herein only.

The comments in this report are intended solely for the guidance of the design engineer and address the geotechnical conditions only. The number of boreholes required to determine the localized conditions between boreholes directly affecting construction costs, equipment, scheduling, etc. would in fact be greater than what has been carried out for design purposes. Therefore, contractors bidding on this project or undertaking this work should make their own interpretations of the factual borehole results and carry out further work as they deem necessary to assess the scope of the project.

Section 5 of this reported is intended for the use of the client and the design team only and is not intended to be included in the tender documents. Inclusion of the factual information (Sections 1 to 5 inclusive) in the tender documents is furnished merely for the general information of bidders and is not in any way warranted or guaranteed by or on behalf of the owner or the owner's consultants and its subconsultants or the consultants' or subconsultants' employees, and neither the owner nor its consultants or its employees shall be liable for any representations negligent or otherwise contained in the documents.

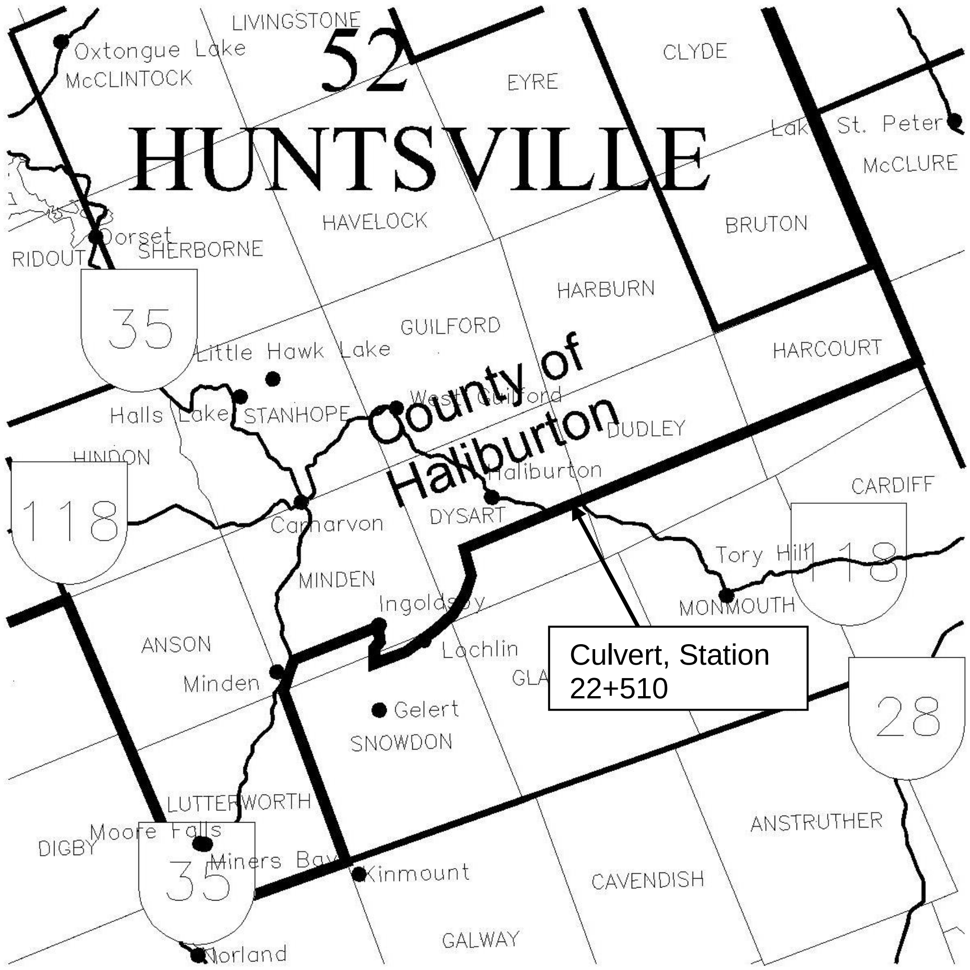
Appendix 1 Key Plan

Drawing No. 1	Key Plan

MACRO KEY PLAN

NOT TO SCALE

Drawing No.1



FOUNDATION INVESTIGATION AND DESIGN REPORT

GWP 5466-04-00

Highway 118

Station 22+510 Culvert

Township of Dysart

Reference No: 15/04/15020-F16 February 2017

Englobe

Appendix 2 Subsurface Data

Enclosure No. 1	List of Abbreviations and Symbols
Enclosure Nos. 2 to 5	Record of Borehole Sheet

LIST OF ABBREVIATIONS & DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

1. ABBREVIATIONS

AS	Auger Sample
CS	Chunk Sample
DS	Denison type sample
FS	Foil Sample
NFP	No Further Progress
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
RC	Rock core with size & percentage of recovery
SS	Split Spoon
ST	Slotted Tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash Sample
WH	Sampler advanced by static weight of hammer and/or rods
Rec	% recovery from individual run of rock core
RQD	Rock quality designation (%)

2. PENETRATION RESISTANCE/"N"

Dynamic Cone Penetration Test (DCPT):

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as ——•——•——•——•

Standard Penetration Test (SPT) or "N" Values

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	*Relative Density*
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	*Consistency*
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

3. SOIL DESCRIPTION (Cont'd)

c) *Bedrock:*

RQD (%)	*Classification*
Less than 25	Very poor quality
25 to 50	Poor quality
50 to 75	Fair quality
75 to 90	Good quality
90 to 100	Excellent quality

d) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

+ 3.2 - Field Vane test in borehole. The number denotes the sensitivity to remoulding.

D - Laboratory Vane Test

¨ - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

e) *Soil Moisture:*

Moisture	*Described as*
Dry	Below optimum moisture content
Moist	Near optimum moisture content
Wet	Above optimum moisture content

4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

Terminology for cobbles and boulders is based on auger response and field observations:

Occasional	Obstructions encountered in borehole, however advance is not impeded
Numerous	Obstructions are essentially continuous over drilled length

SAMPLE DESCRIPTION NOTES:

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as 'fill'. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.

2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.

3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled 'possibly' or 'probable' etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients' request, through a series of closely positioned unsampled auger probes combined with core drilling.

4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

ENCLOSURE NO.:2 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 1

Englobe

REFERENCE 15/04/15020-F16 DATUM Geodetic LOCATION N 4989026.8 E 386951.0 - Dysart Twp., Station 22+500 ORIGINATED BY JL

PROJECT GWP 5466-04-00, Highway 118 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 2015 May 13 DATE (Completed) 2015 May 13 TIME (Completed) 3:45:00 PM CHECKED BY MAM

ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
378.1	Ground Surface						
0.0	125 mm Crushed Gravel EMBANKMENT FILL- sand, some gravel to gravelly , trace silt Cobble/boulder size rock fill encountered between depths from 1.6 m to 2.3 m brown, moist (loose/compact) trace organic material encounted at depth of 2.6 m	1	SS	17	378	○	33 59 (8)
		2	SS	22	377	○	
		3	SS	30/51mm			
375.5		4	SS	5	376	○	
2.6	sandy SILT - trace gravel, trace clay (compact)	5	SS	14	375	○	10 33 55 2
374.4 3.7	SAND - some gravel, some silt, trace clay brown (compact)	6	SS	19	374	○	18 63 (19)
		7	SS	23	373	○	19 67 14 0
372.0 6.1	gravelly SAND - some silt grey to brown	8	SS	43/152mm	372	○	38 46 (16)
371.7 6.4	Auger Refual Start Rock Coring Bedrock - black gneiss Poor to fair quality	9	RC	Rec= 97% RQD= 40%	371		
		10	RC	Rec= 100% RQD= 66%	370, 369		
368.5 9.6	End of Sampling End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 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

STRATA PLOT, GROUND WATER CONDITIONS

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+3, ×3: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 15/5/14 2:45:00 PM	3.5	-
2) 15/8/19 4:30:00 PM	5.5	-
3) 16/8/15	5.9	-

MEL-GEO 15020 - BOREHOL LOGS - F16.GPJ MEL-GEO.GDT 17/1/23

Englobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@englobecorp.com

ENCLOSURE NO.:3 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 2

Englobe

REFERENCE 15/04/15020-F16 DATUM Geodetic LOCATION N 4989024.8 E 386965.4 - Dysart Twp., Station 22+511.5 ORIGINATED BY JL

PROJECT GWP 5466-04-00, Highway 118 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 2015 May 14 DATE (Completed) 2015 May 14 TIME (Completed) 12:00:00 PM CHECKED BY MAM

ELEV / DEPTH	DESCRIPTION (see Enclosure No. 1)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	NATURAL MOISTURE CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
377.8	Ground Surface						
0.0	50 mm Asphalt EMBANKMENT FILL- sand, some gravel to gravelly, trace silt Cobble/boulder size rock fill encountered between depths of 1.2 m to 4.9 m brown, moist (loose/compact) 300 mm diameter boulder encounted at depths from 4.9 m to 5.2 m (very dense/dense)	1	SS	20		o	
		2	SS	13		o	
		3	SS	13		o	39 53 (8)
		4	SS	13		o	
		5	SS	6		o	34 57 (9)
		6	SS	11			
		7	SS	25/0mm		o	
372.2 / 5.6	SAND - trace to some gravel, with silt, trace clay grey to brown (compact/dense)				▽ (approx. 6.1 m); cave-in (approx. 7.1 m)		
		8	SS	43		o	6 67 23 4
		9	SS	27		o	16 61 (23)
368.7 / 9.1	Auger Refual Start Rock Coring Bedrock - black gneiss Fair to good quality	10	RC	Rec= 92% RQD= 74%			
		11	RC	Rec= 95% RQD= 89%			
365.6 / 12.2	End of Sampling End of Borehole						

Elevation scale: 377, 376, 375, 374, 373, 372, 371, 370, 369, 368, 367, 366

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 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

COMMENTS

+3, ×3: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

	Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	15/5/14 2:40:00 PM	6.1	7.1
2)		-	-
3)		-	-

The stratification lines represent approximate boundaries. The transition may be gradual.

MEL-GEO 15020 - BOREHOL LOGS - F16.GPJ MEL-GEO.GDT 17/1/23

Englobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@englobecorp.com

ENCLOSURE NO.:4 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 3

Englobe

REFERENCE 15/04/15020-F16 DATUM Geodetic LOCATION N 4989013.2 E 386952.8 - Dysart Twp., Station 22+510.5 ORIGINATED BY JL

PROJECT GWP 5466-04-00, Highway 118 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 2015 August 18 DATE (Completed) 2015 August 18 TIME (Completed) 12:05:00 PM CHECKED BY MAM

ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
374.9	Ground Surface					
374.7 0.0	ORGANIC SOIL - silty, black					
0.2	silty SAND - trace gravel, trace grass, rootlets, brown to grey (loose/dense) wet	1	SS	9	~19	
		2	SS	34	~15	2 61 37 0
		3	SS	50/279 mm	~16	
372.6 2.3	Auger Refual, Start Rock Coring, Bedrock - black gneiss, Fair to good quality	4	RC	Rec= 85% RQD= 66%		
		5	RC	Rec= 95% RQD= 76%		
369.2 5.7	End of Sampling, End of Borehole					

Elevation scale: 374, 373, 372, 371, 370

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 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

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+3, ×3: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 15/8/18 12:10:00 PM	1.2	-
2) 15/8/19 4:30:00 PM	3.6	-
3) 16/8/15	5.7	-

ENCLOSURE NO.:5 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 4

Englobe

REFERENCE 15/04/15020-F16 DATUM Geodetic LOCATION N 4989035.2 E 386974.0 - Dysart Twp., Station 22+510 ORIGINATED BY JL

PROJECT GWP 5466-04-00, Highway 118 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 2015 August 19 DATE (Completed) 2015 August 19 TIME (Completed) 4:30:00 PM CHECKED BY MAM

ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
370.1	Ground Surface							
370.0 0.1	ORGANIC SOIL - silty, black	1	SS	2		370		
369.6 0.5	sandy SILT - trace gravel, trace grass, rootlets brown (very loose)							
	SAND - trace to some gravel, with silt, trace clay trace rootlets, decayed wood grey to brown (compact) wet	2	SS	16	∇	369		
		3	SS	23				2 69 29 0
		4	SS	15		368		2 87 10 1
		5	SS	13		367		12 58 (30)
366.4 3.7	Auger Refual Start Rock Coring Bedrock - black gneiss Good quality	6	RC	Rec= 98% RQD= 83%		366		
		7	RC	Rec= 97% RQD= 78%		365, 364		
363.4 6.7	End of Sampling End of Borehole							

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (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

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+3, ×3 : Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 15/8/19 4:30:00 PM	1.2	-
2)	-	-
3)	-	-

MEL-GEO 15020 - BOREHOL LOGS - F16.GPJ MEL-GEO.GDT 17/1/23

Englobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@englobecorp.com

Appendix 3 Borehole Plan and Lab Data

Drawing No. 2:	Borehole Location and Soil Strata
Figure Nos. L-1 to L-5:	Grain Size Distribution Curves
Table No. L-6:	Lab Test Summary Sheet

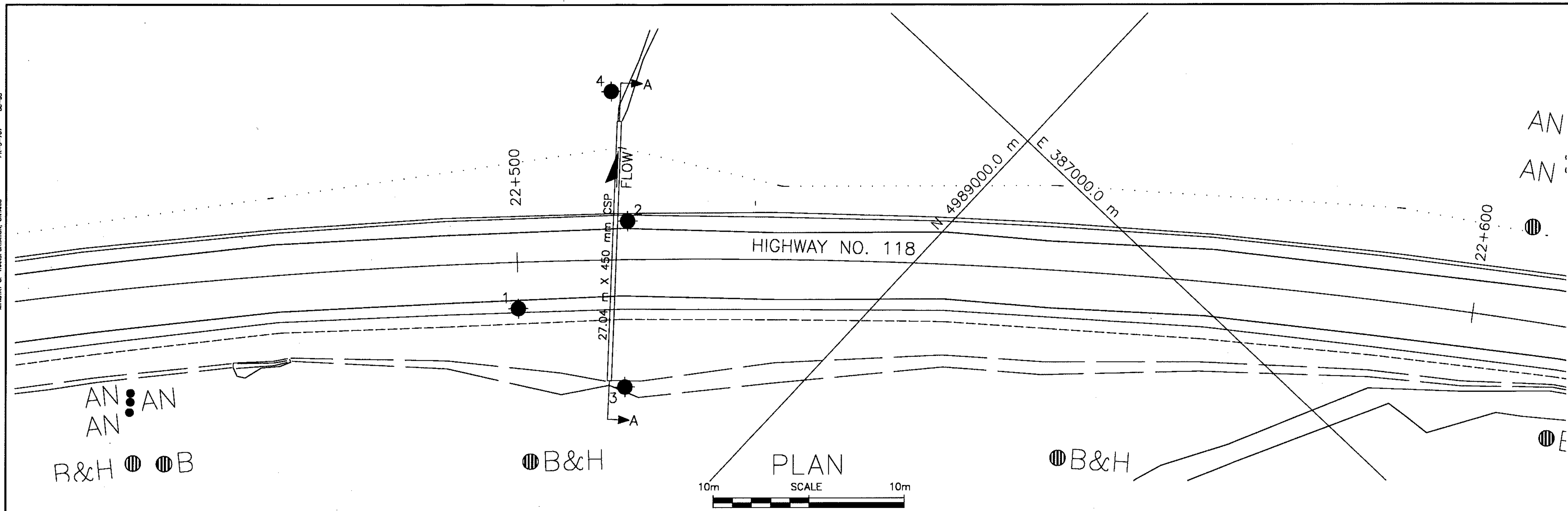
Englobe

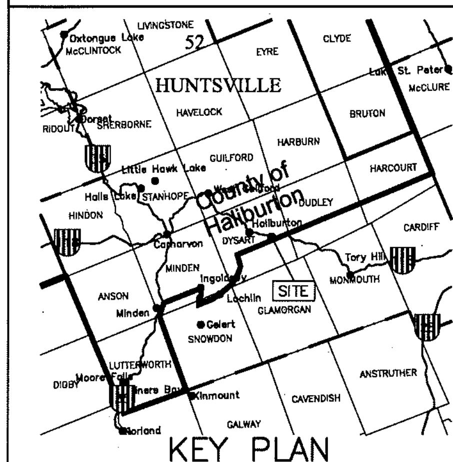
DISTRICT
CONT. No.
GWP No. 5466-04-00

HWY 118
CULVERT AT STN: 22+510
TOWNSHIP OF DYSART
BOREHOLE LOCATIONS
AND SOIL STRATIGRAPHY

SHEET 2

Englobe



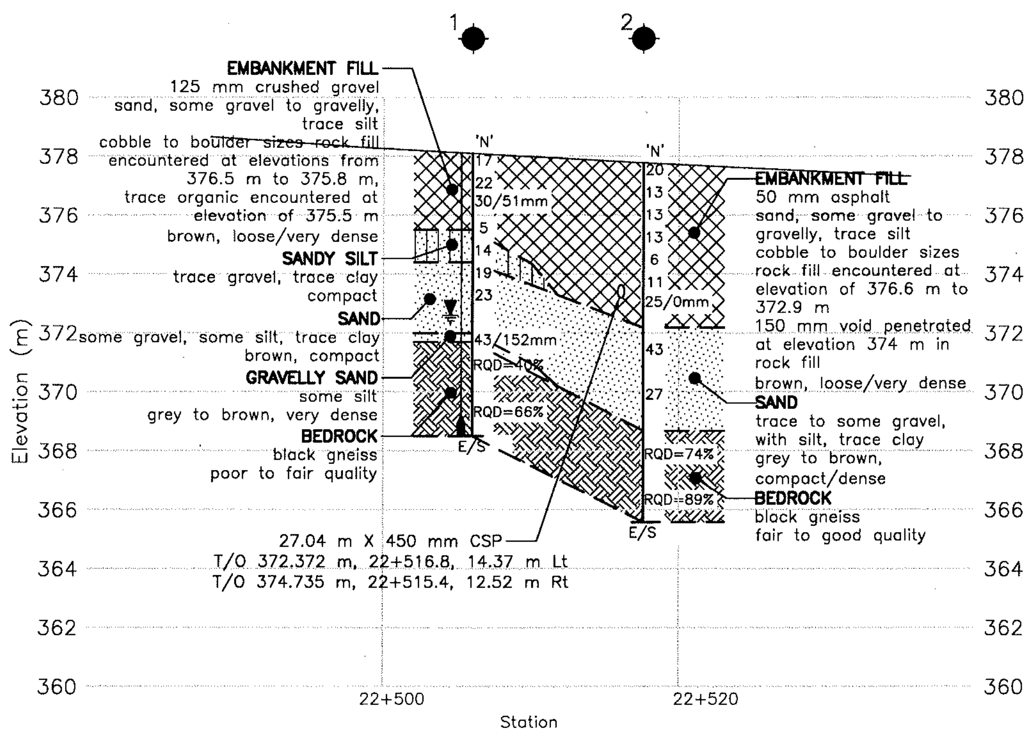


KEY PLAN
N.T.S.

LEGEND

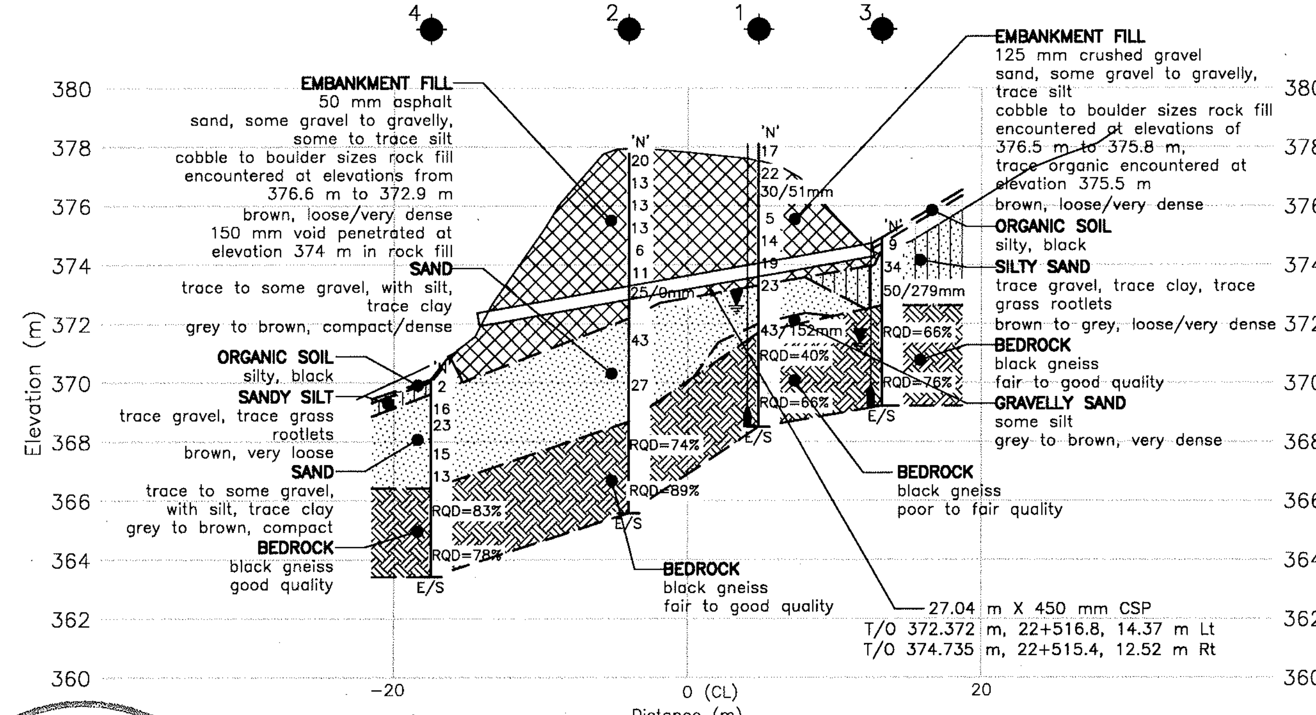
●	Borehole
N	Blows/0.3 m (Std Pen Test, 475 J/blow)
▼	Water Level at Time of Investigation
E/S	End of Sampling
▮	Piezometer

BOREHOLE No.	ELEVATION	O/S	NORTHING	EASTING
1	378.1	4.8m Rt	4989026.8	386951.0
2	377.8	4.0m Lt	4989024.8	386965.4
3	374.9	13.2m Rt	4989013.2	386952.8
4	370.1	17.5m Lt	4989035.2	386974.0



C/L PROFILE HWY 118

SCALE
HOR 10 5 0 10 20m
VERT 5 2.5 0 5 10m



CROSS SECTION A–A

SCALE
HOR 10 5 0 10 20m
VERT 5 2.5 0 5 10m

NOTES:

The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.

The thickness of fill directly below the culvert has been assumed at 300 mm on the cross section.

Base plan and alignment provided in digital format by Callon Dietz on August 4, 2015

Coordinates based on MTM Zone 10 NAD83 CSRS

GEOCRES No. 31E-357

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

LICENSED PROFESSIONAL ENGINEER
S. HU
90524232
2017-02-03
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
M. H. MacKAY
PROVINCE OF ONTARIO

2017-02-03

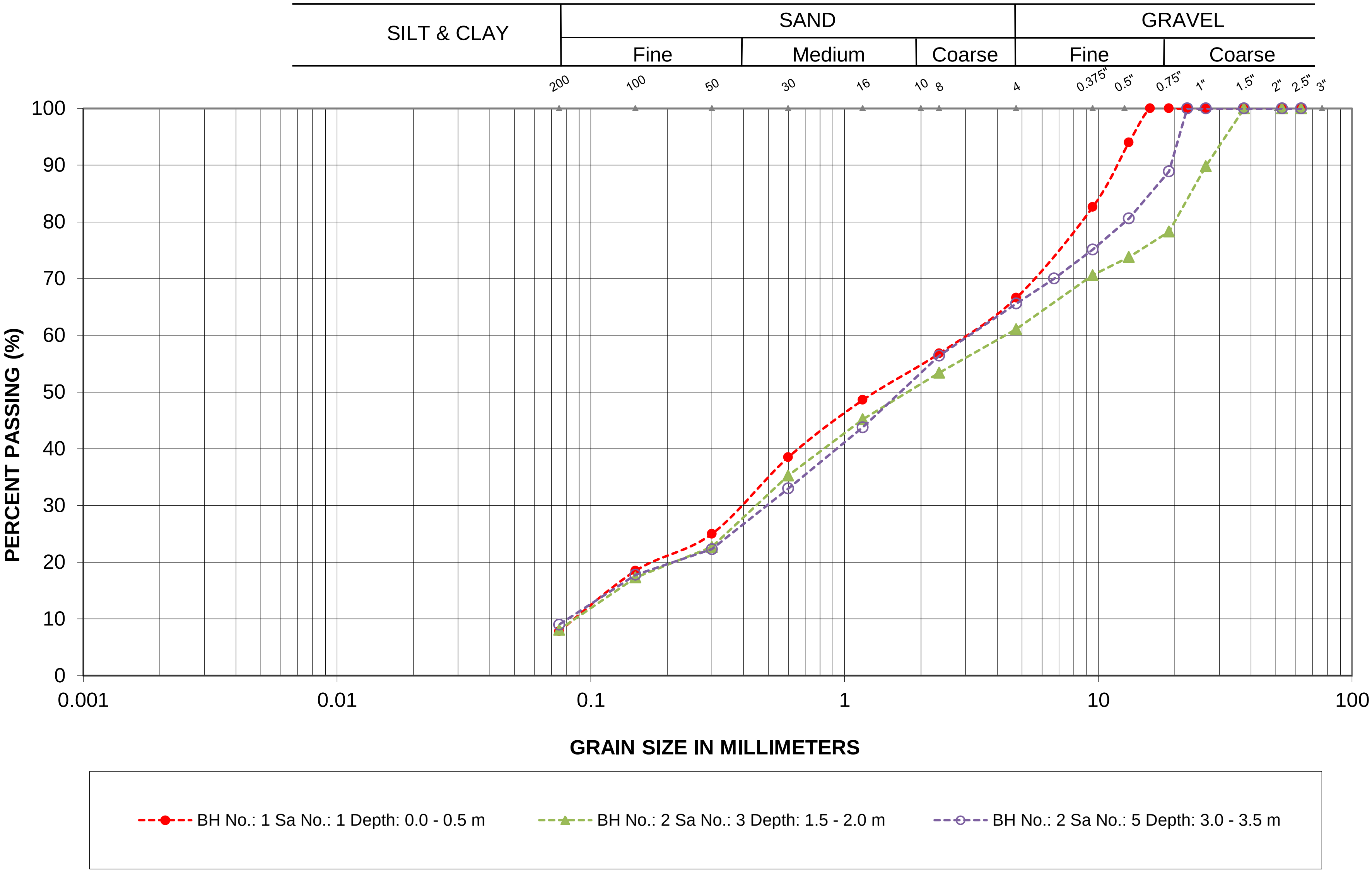
REVISIONS			
JAN/16	DM	DRAFT	
APR/16	DM	FINAL	
FEB/17	DM	FINAL R2	
		DESCRIPTION	

DESIGN	CHK	CODE	LOAD	DATE FEB/17
DRAWN DM	CHK SH	SITE	STRUCT	SCHEME	DWG 2

CAD FILE LOCATION AND NAME: O:\2015\15020 - PAV & FDN, Hwy 60 & 118, 5014-E-0004 (AECOM)\FOUNDATION\Drawings\F16\15020 - 22+510 - Revised Chainage (17-01-23).dwg
MODIFIED: 2/3/2017 1:59:21 PM BY: GRASRY
DATE PLOTTED: 2/3/2017 2:02:04 PM BY: RYAN GRASSER

Reference No.: 15/04/15020-F16
Date: November 2015

GRAIN SIZE ANALYSIS



EMBANKMENT FILL

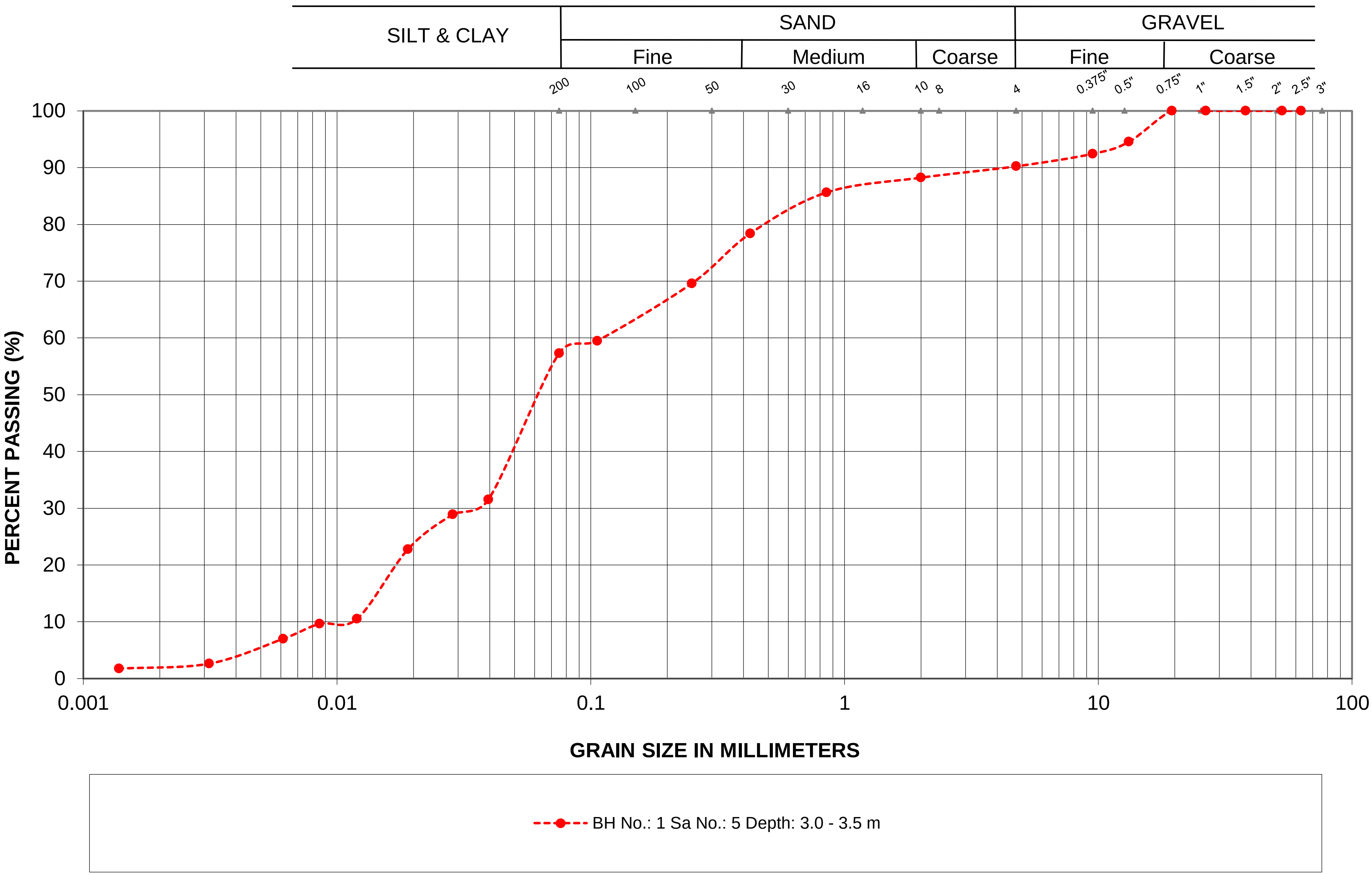
LOCATION: Hwy 118, Station 22+510
TWP of Dysart

Englobe Corp.

FIGURE L-1

Reference No.: 15/04/15020-F16
Date: November 2015

GRAIN SIZE ANALYSIS



SANDY SILT

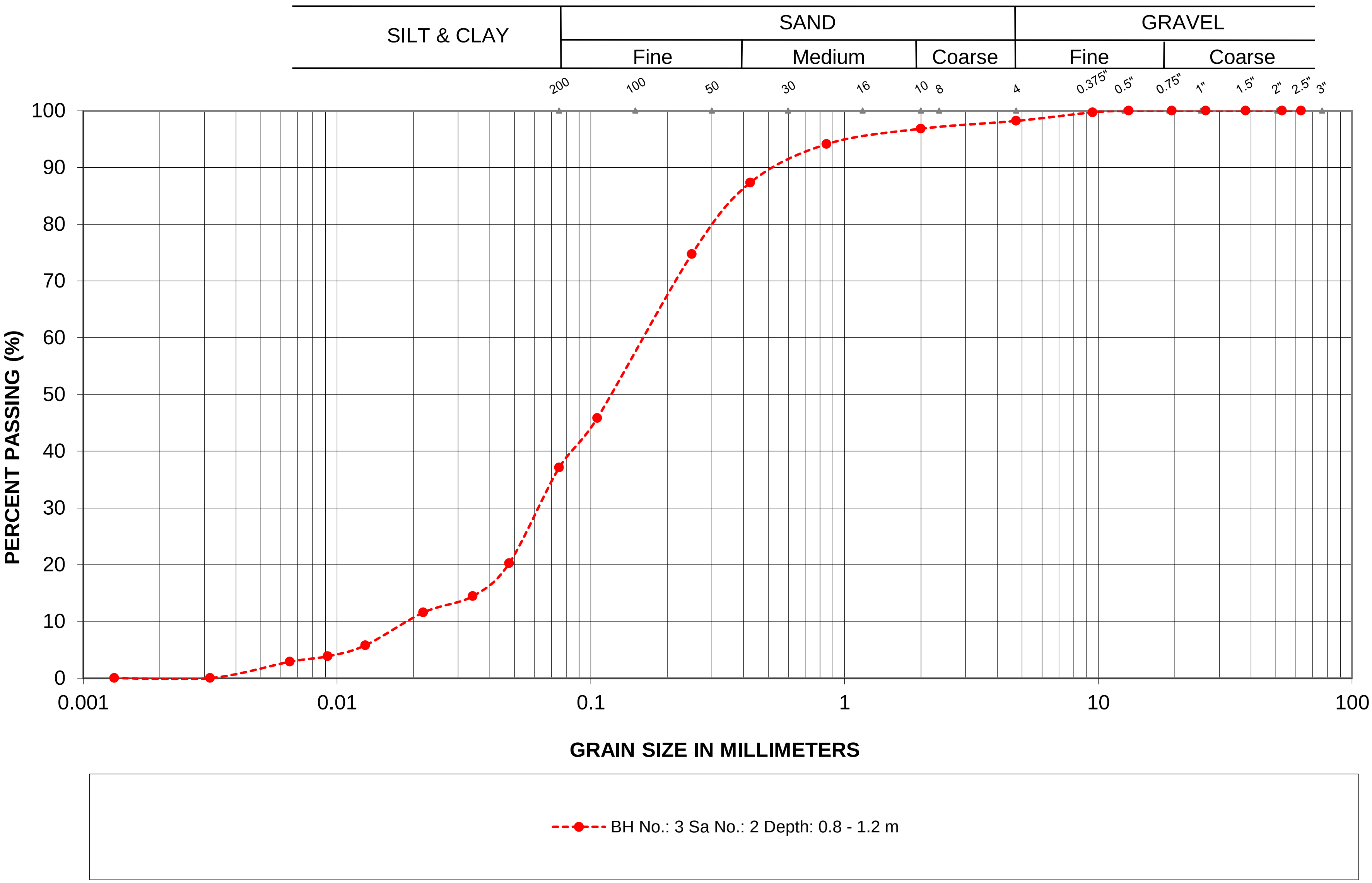
LOCATION: Hwy 118, Station 22+510
TWP of Dysart

Englobe Corp.

FIGURE L-2

Reference No.: 15/04/15020-F16
Date: November 2015

GRAIN SIZE ANALYSIS



SILTY SAND

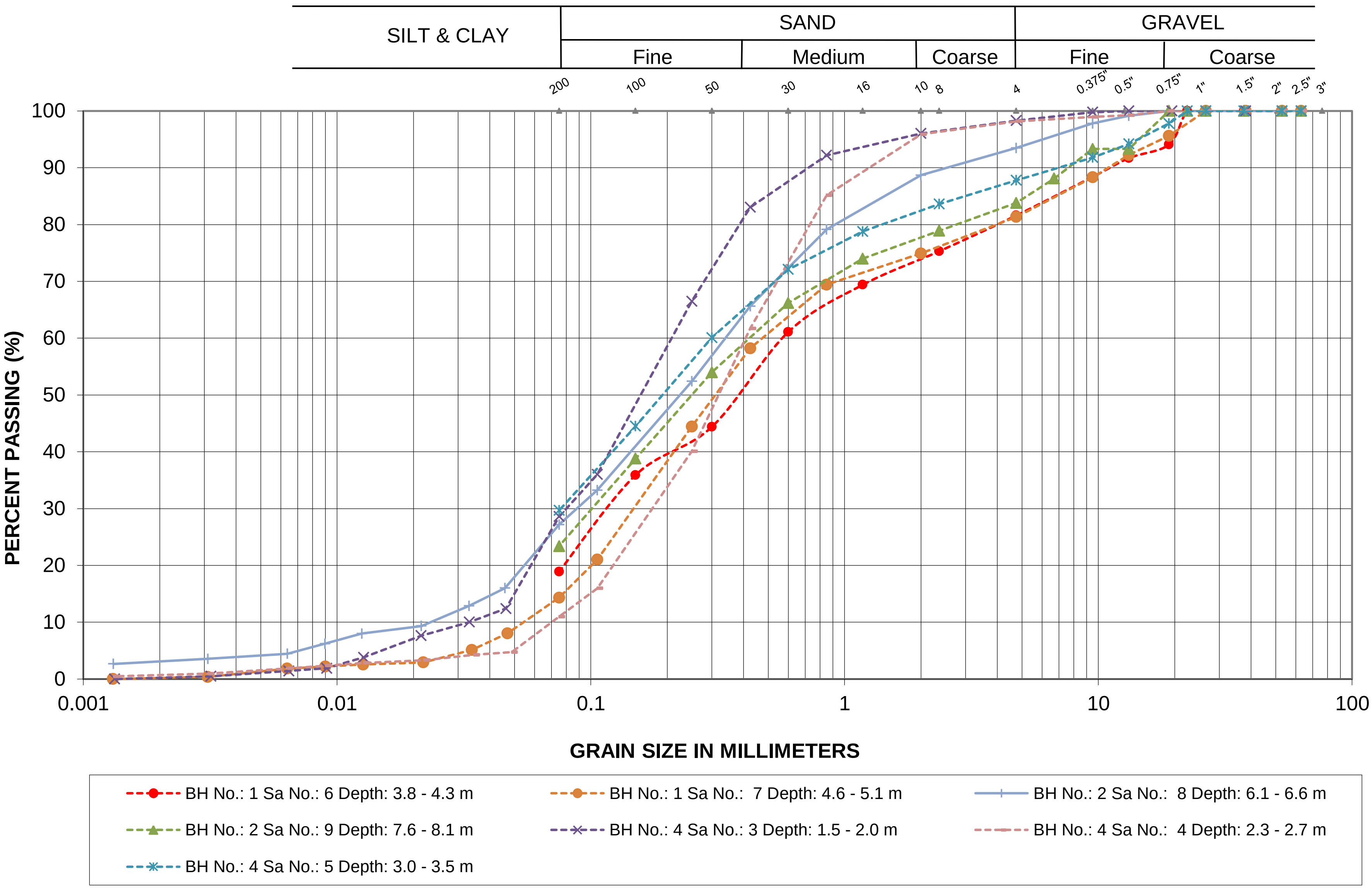
LOCATION: Hwy 118, Station 22+510
TWP of Dysart

Englobe Corp.

FIGURE L-3

Reference No.: 15/04/15020-F16
Date: November 2015

GRAIN SIZE ANALYSIS



SAND

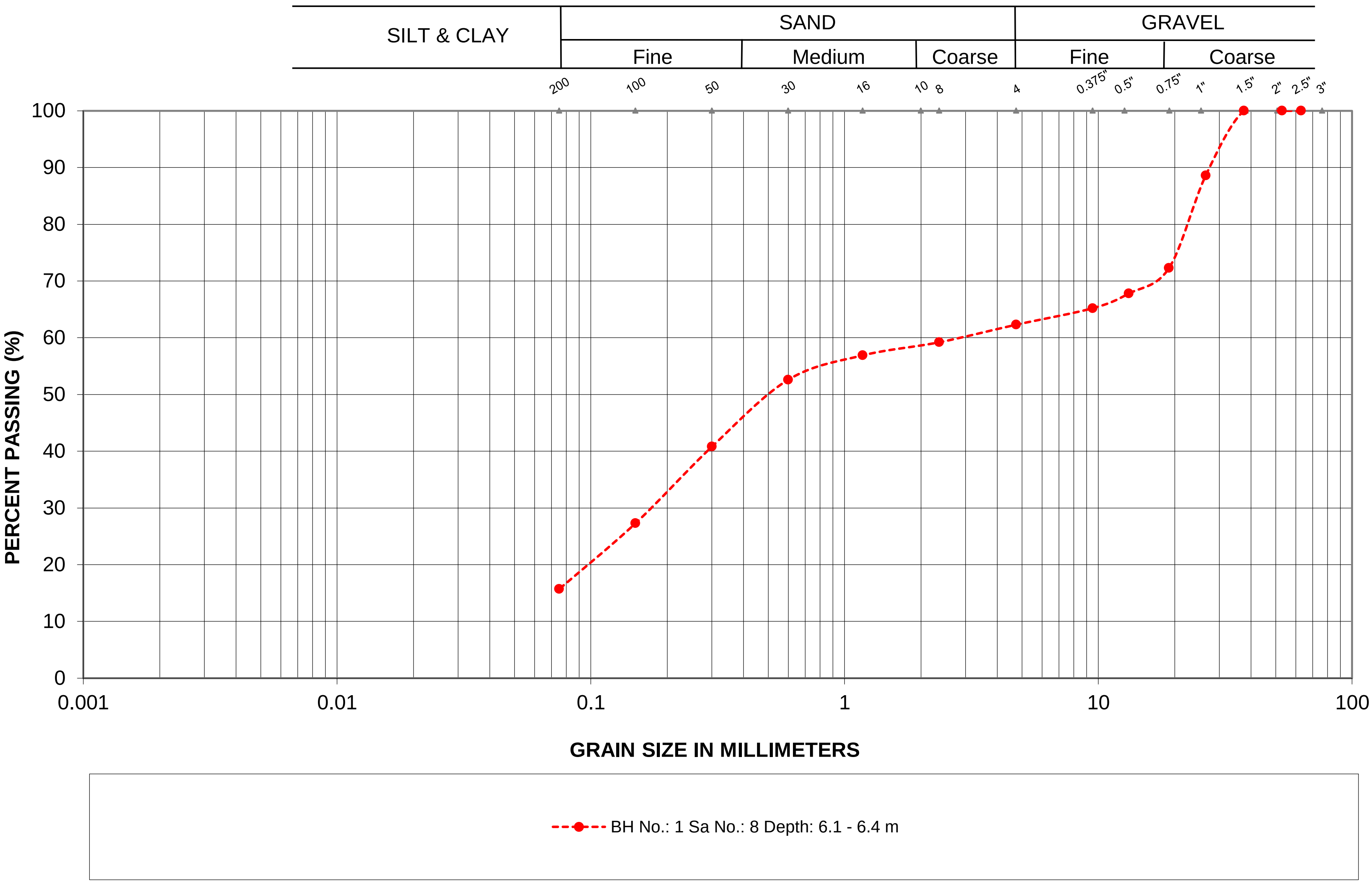
LOCATION: Hwy 118, Station 22+510
TWP of Dysart

Englobe Corp.

FIGURE L-4

Reference No.: 15/04/15020-F16
Date: November 2015

GRAIN SIZE ANALYSIS



GRAVELLY SAND

LOCATION: Hwy 118, Station 22+510
TWP of Dysart

Englobe Corp.

FIGURE L-5

Laboratory Tests - Summary Sheet

Englobe

Borehole No.	Sample No.	Depth	Grain Size Analysis: Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)	NMC	Atterberg Limits: LL (%)	PL (%)	IP (%)	SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
1	1	0.0	33	59	8		4.6				17			
	2	0.8					5.5				22			
	3	1.5									30/51mm			
	4	2.3					12.0				5			
	5	3.1	10	33	55	2	38.6				14			Non-Plastic (NP)
	6	3.8	18	63	19		12.8				19			
	7	4.6					12.6				23			
	8	6.1	38	46	16		8.9				43/152mm			
	9	6.4												Rec= 97%, RQD= 40%
	10	7.9												Rec= 100%, RQD= 66%
2	1	0.0					2.9				20			
	2	0.8					4.6				13			
	3	1.5	39	53	8		4.1				13			
	4	2.3					6.2				13			
	5	3.1	34	57	9		4.6				6			
	6	3.8									11			
	7	4.6					2.4				25/0mm			
	8	6.1					12.1				43			
	9	7.6	16	61	23		11.6				27			
	10	9.1												Rec= 92%, RQD= 74%
	11	10.7												Rec= 95%, RQD= 89%

Laboratory Tests - Summary Sheet

Englobe

Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
3	1	0.0					19.4				9			
	2	0.8	2	61	37	0	10.4				34			
	3	1.5					12.5				50/279 mm			
	4	2.3												Rec= 85%, RQD= 66%
	5	4.1												Rec= 95%, RQD= 76%
4	1	0.0					33.5				2			
	2	0.8					14.2				16			
	3	1.9	2	69	29	0	18.3				23			
	4	2.3					17.1				15			
	5	3.1	12	58	30		16.0				13			
	6	3.7												Rec= 98%, RQD= 83%
	7	5.2												Rec= 97%, RQD= 78%

Appendix 4 Photo Essay

Enclosure No. 6: Photo Essay

Embankment at Culvert Location – Looking West, south side of embankment	Photo: 1

Embankment at Culvert Location – Looking East, north side of embankment	Photo: 2

Project: Hwy 118 – Culvert, Station 22+510, Township of Dysart	Photos Provided By: Englobe Date: August 2015

Culvert Inlet – Looking South	Photo: 3

View of Embankment at Culvert Outlet – Looking North	Photo: 4

Project: Hwy 118 – Culvert, Station 22+510, Township of Dysart	Photos Provided By: Englobe Date: August 2015

Rock Cores – Borehole 2 (left) and Borehole 3 (right)	Photos: 5 and 6

Project: Hwy 118 – Culvert, Station 22+510, Township of Dysart	Photos Provided By: Englobe Date: August 2015

Rock Cores – Borehole 4	Photos: 7
	
Project: Hwy 118 – Culvert, Station 22+510, Township of Dysart	Photos Provided By: Englobe Date: August 2015

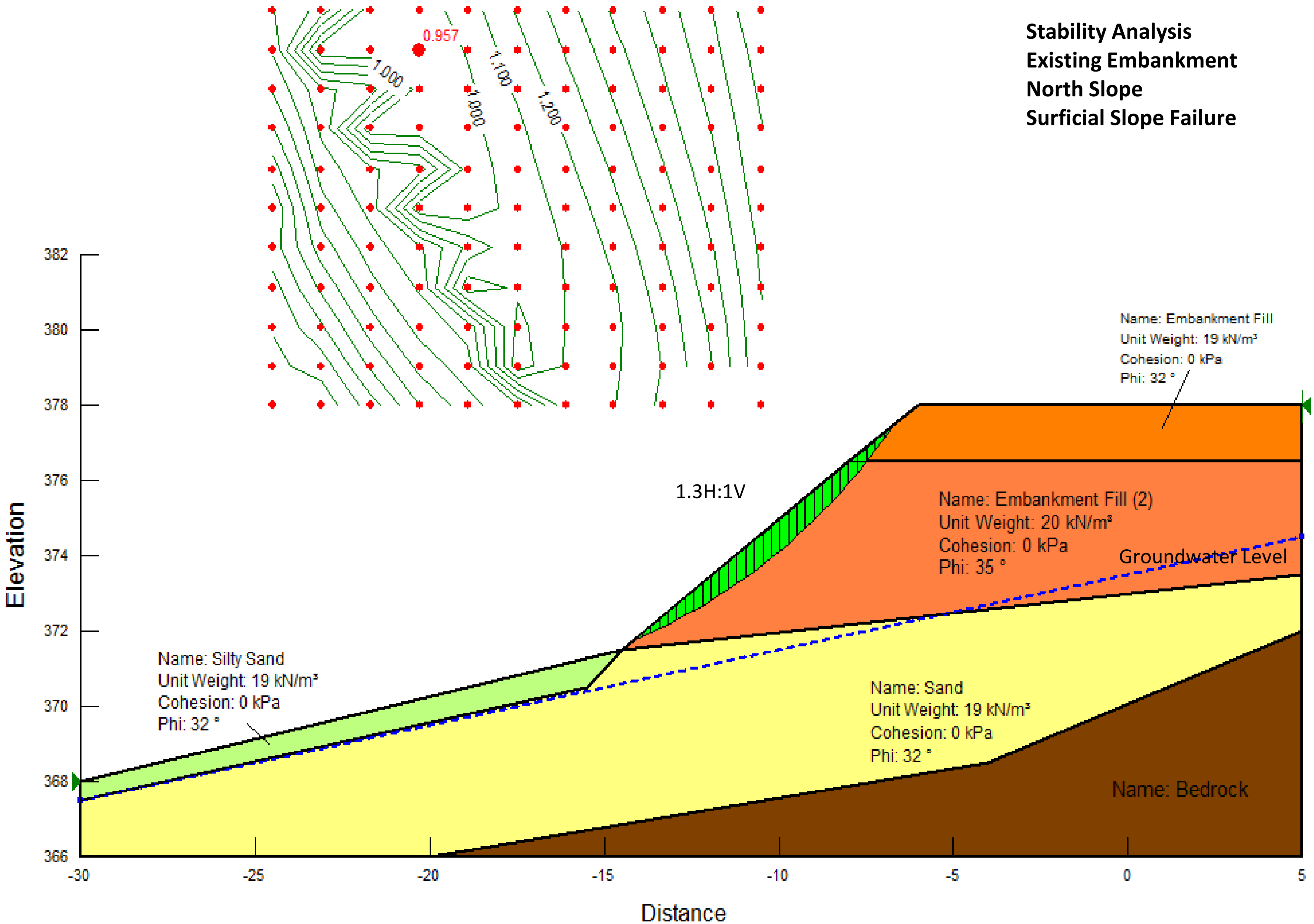
Appendix 5 Design Data

Figure Nos. S-1 to S-4:	Slope Stability
Table A:	Comparison of Shoring Alternatives
Figure No. SK-3:	Conceptual Staging Plan
Figure No. SK-4:	Conceptual Shoring Locations
Figure No. SK-5	Conceptual Shoring Sections
	Notice to Contractor

Englobe

Reference No.: 15/04/15020-F16
Date: February 2017

Stability Analysis
Existing Embankment
North Slope
Surficial Slope Failure



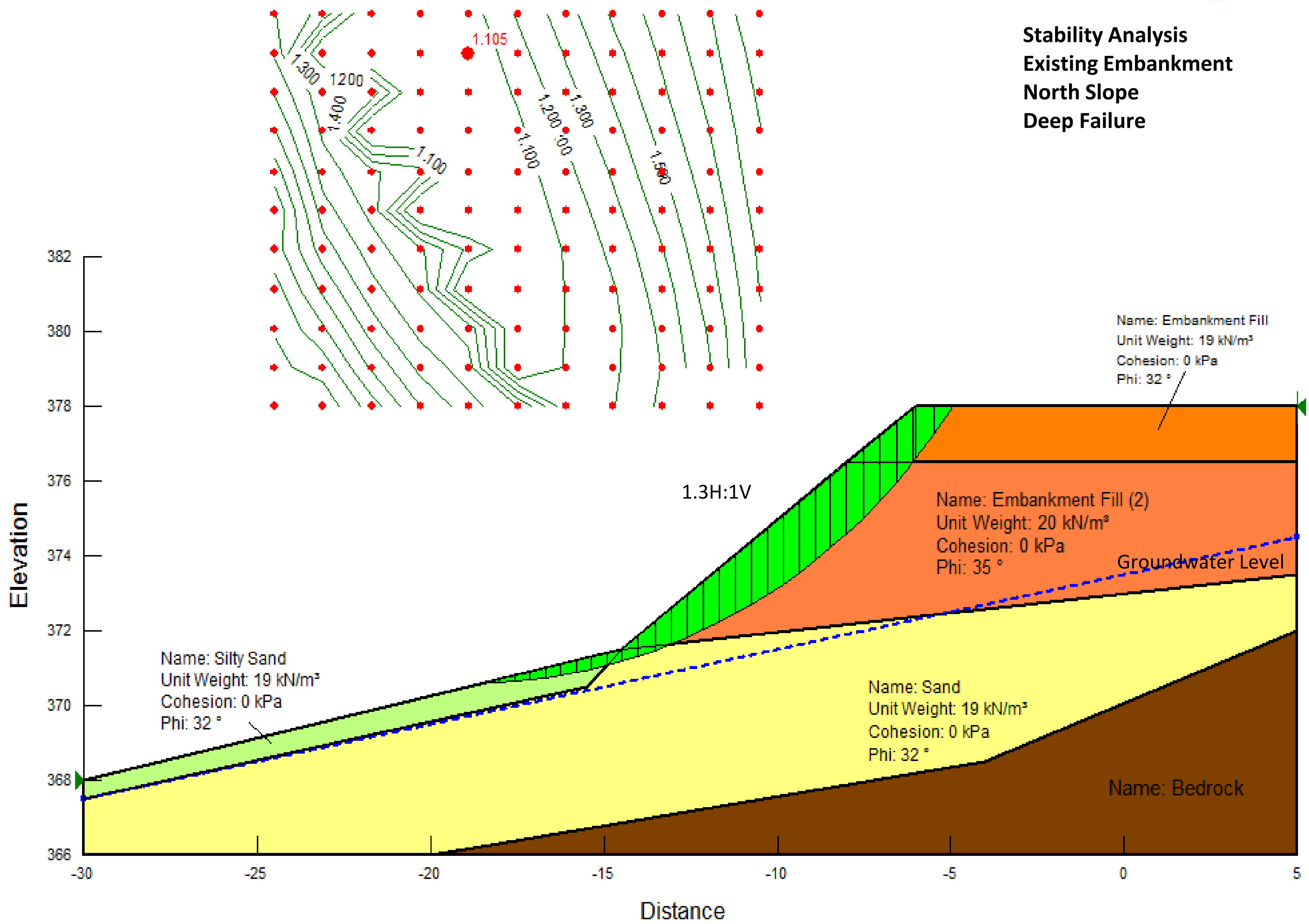
North Slope
Culvert Station 22+510

Project: G.W.P 5466-04-00
Location: Hwy 118, Township of Dysart

Figure No. S-1

Reference No.: 15/04/15020-F16
Date: February 2017

Stability Analysis
Existing Embankment
North Slope
Deep Failure



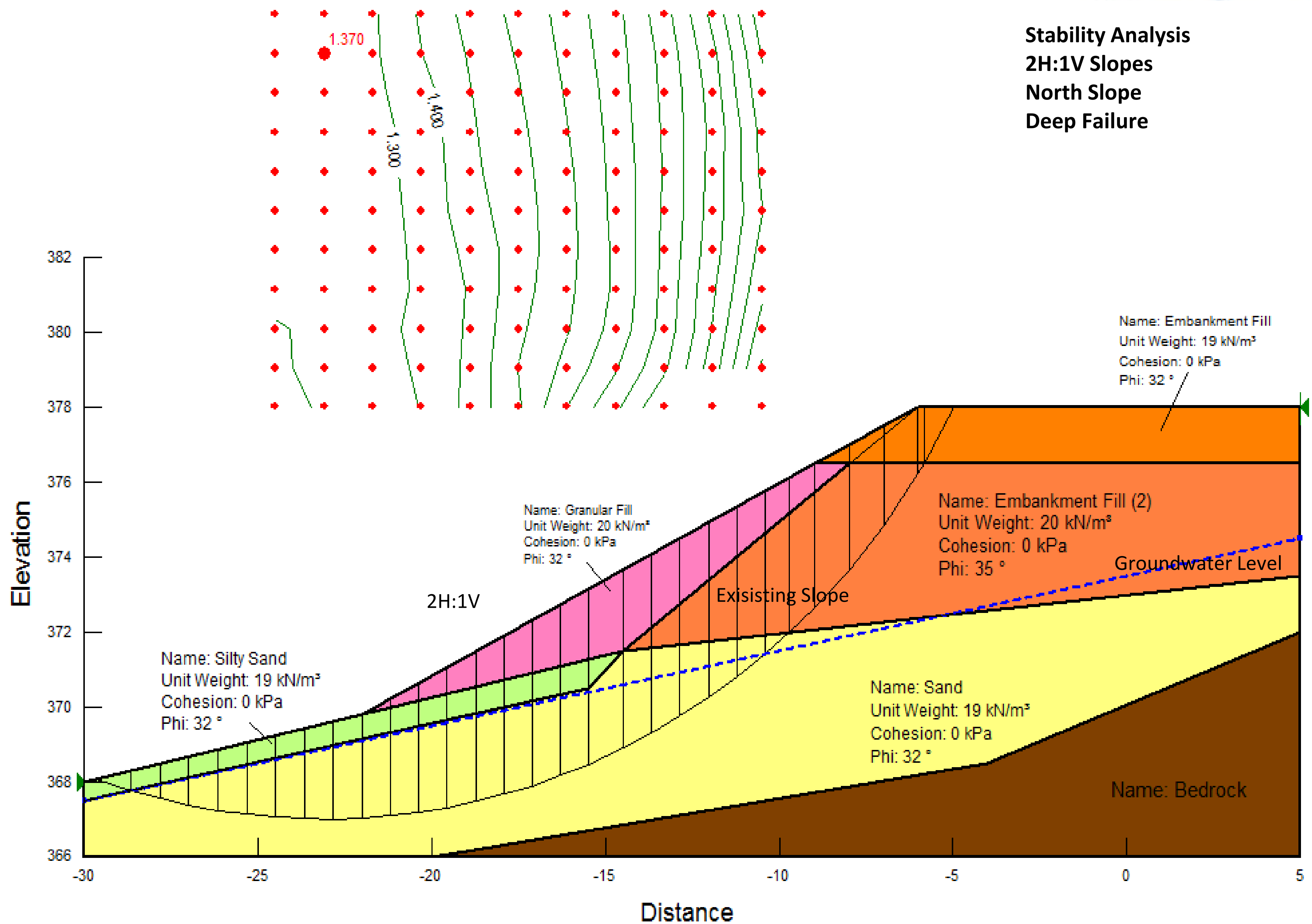
South Slope
Culvert Station 22+510

Project: G.W.P 5466-04-00
Location: Hwy 118, Township of Dysart

Figure No. S-2

Reference No.: 15/04/15020-F16
Date: February 2017

Stability Analysis
2H:1V Slopes
North Slope
Deep Failure



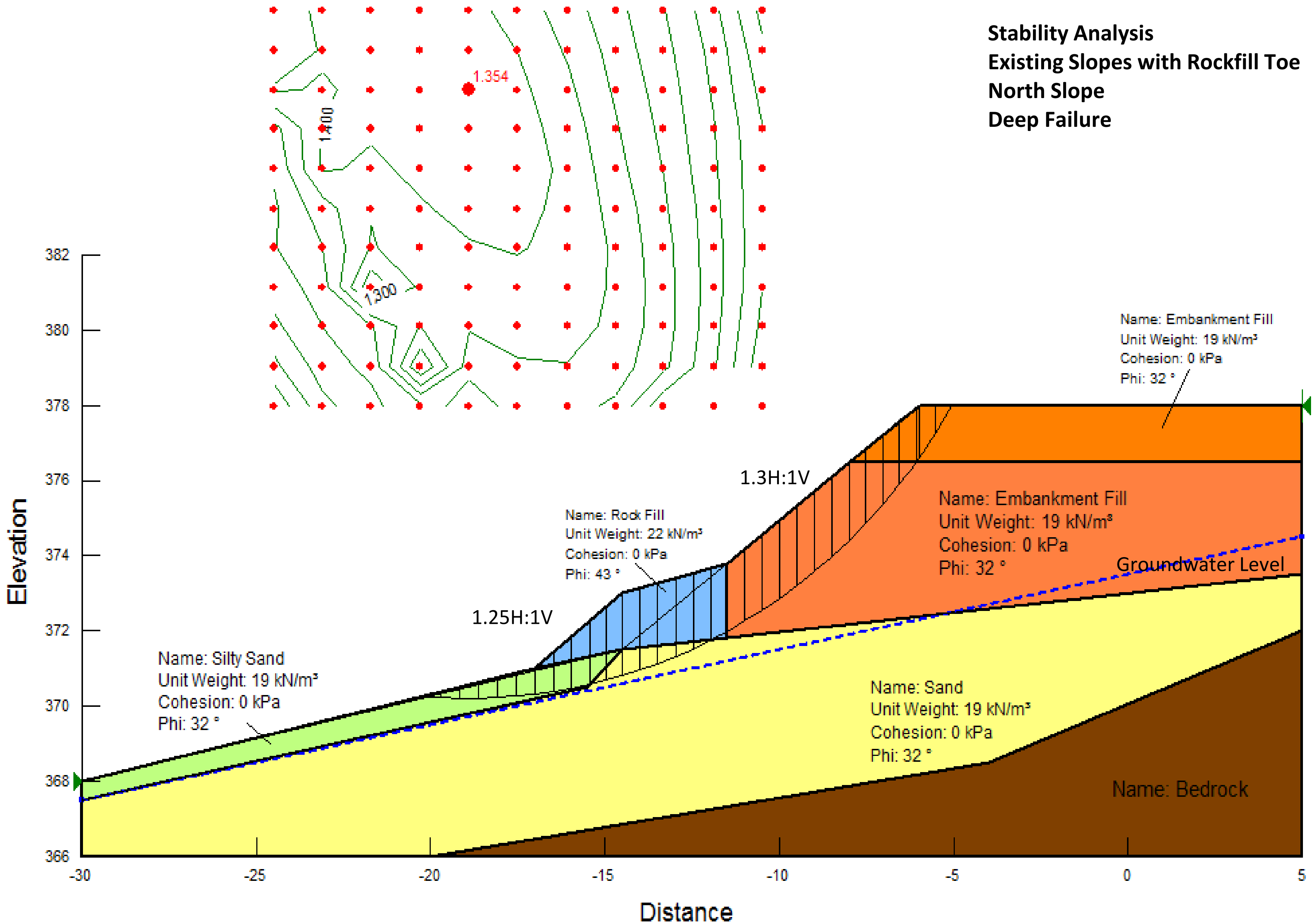
South Slope
Culvert Station 22+510

Project: G.W.P 5466-04-00
Location: Hwy 118, Township of Dysart

Figure No. S-3

Reference No.: 15/04/15020-F16
Date: February 2017

Stability Analysis
Existing Slopes with Rockfill Toe
North Slope
Deep Failure



South Slope
Culvert Station 22+510

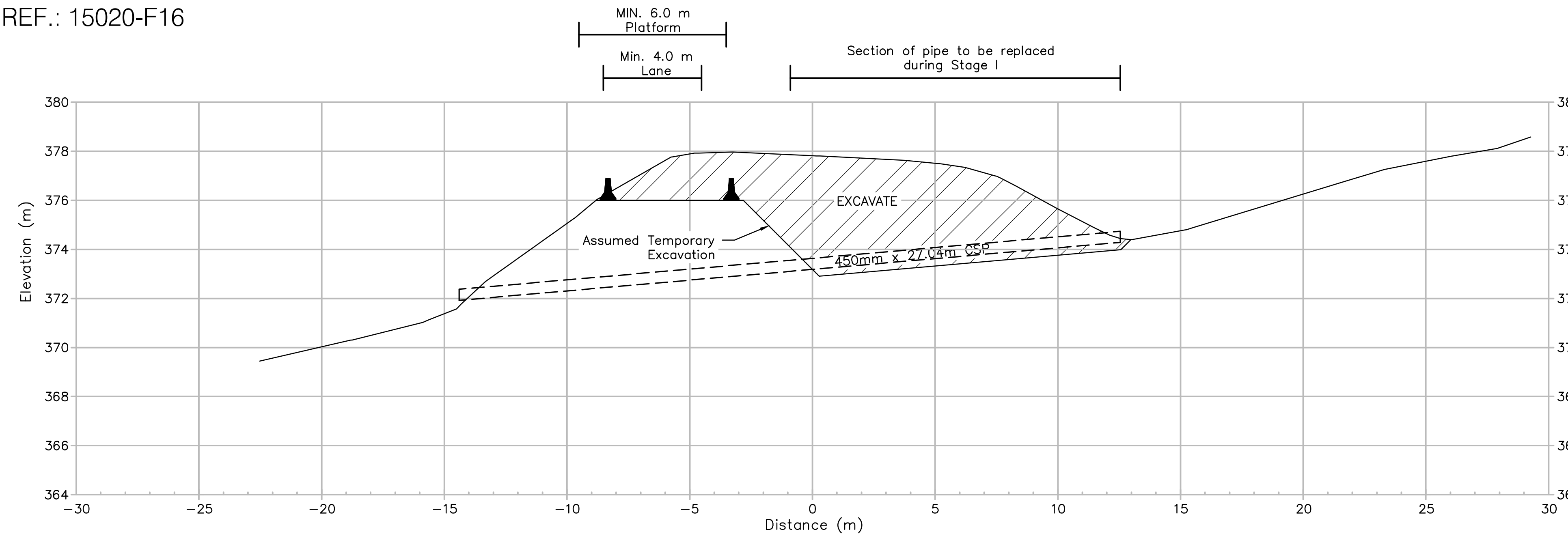
Project: G.W.P 5466-04-00
Location: Hwy 118, Township of Dysart

Figure No. S-4

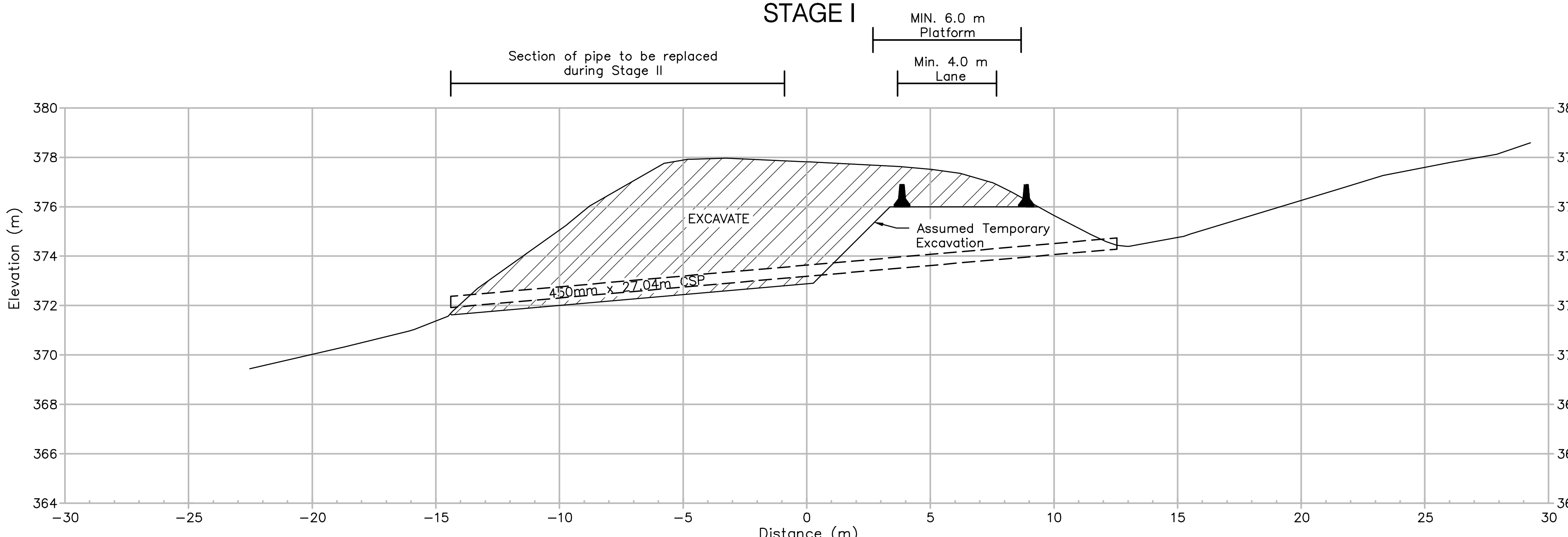
Table A – Comparison of Shoring Alternatives

Method	Depth Range (m)	Advantages	Disadvantages	Remarks	Estimated Costs
Wood Sheeting	1.5 – 5	-Low cost, -Easily installed in good ground conditions	-Limited by soil conditions, -Limited depth of installation, -Low strength, -discontinuous	Not considered due to rock fill embankment	$ 650/m^2
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Not considered due to rock fill embankment	$ 650/m^2
Pre-cast concrete panels	3 – 10	-Durable -Assists in minimizing seepage	-Limited depths -Can be damaged during installation -Limited by soil conditions (i.e. obstructions)	Not considered due to ground conditions and higher cost	
Soldier piles	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Recommended for protection system at this site	$ 725/m^2 Predrilling $ 1,500/m^2
Tangent/ Secant/ Staggered Drilled Piles	10 – 18	-Readily available -Adaptable to various ground conditions	-Possible ground loss and/or seepage -Poor alignment tolerance	Considered as alternative for excavations requiring a protection system at this site	
Concrete Diaphragm	10 – 30	-High Strength -Durable -Can be permanent	-High cost -Requires specialized equipment/control	Not Considered due to ground conditions and higher costs	
Micropiles with reinforced shotcrete face		-Can be installed in various ground conditions -High strength -Good tolerance	-High Cost -Requires specialized equipment	Considered as alternative for protection system; higher cost.	$ 1,200 – 1,500/m^2

REF.: 15020-F16



STAGE I



STAGE II

METRIC

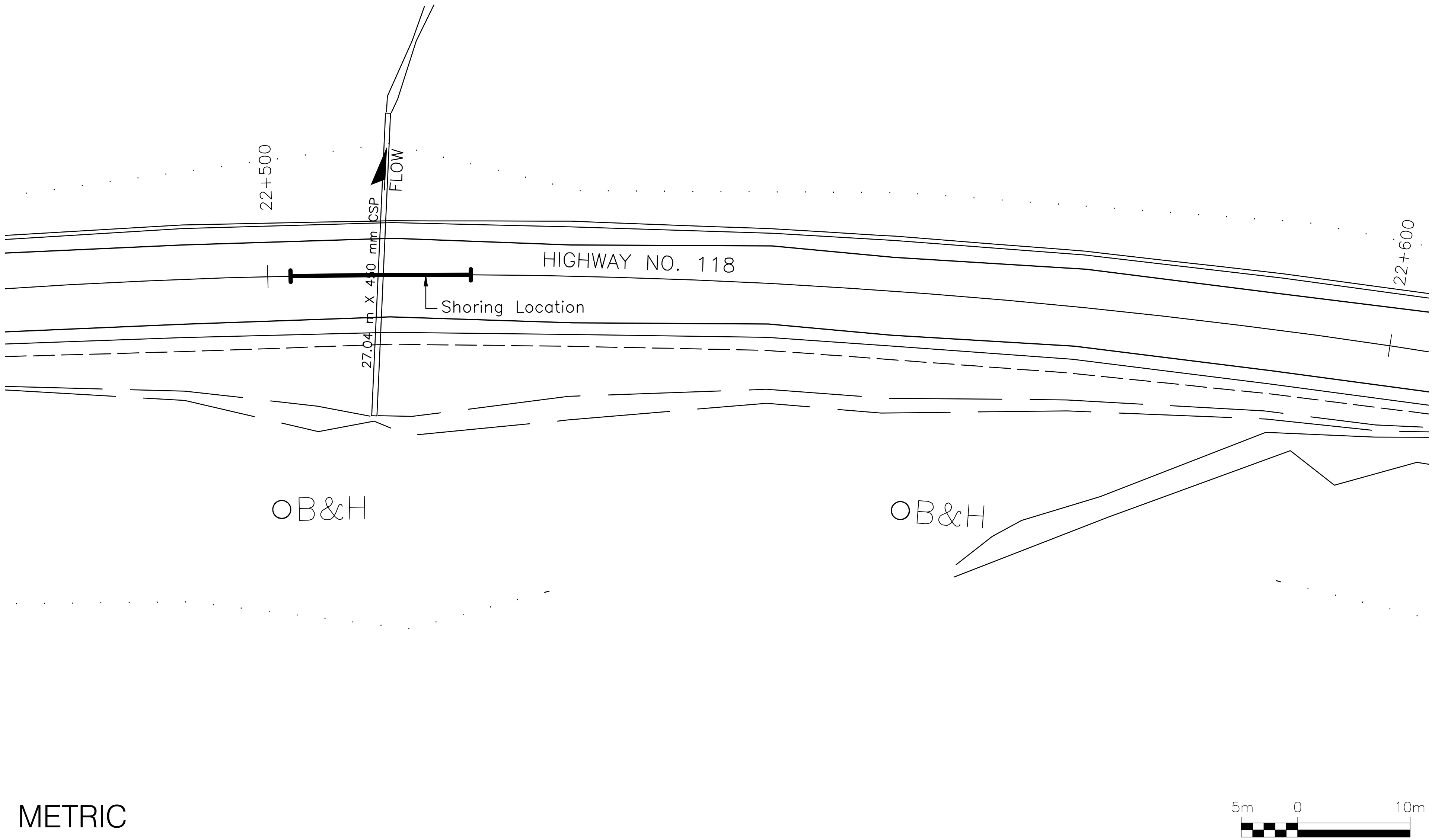
Dimensions are in meters and/or millimeters unless otherwise shown. Stations are in kilometers + meters.

2.5m 0 5m

Highway 118, Township of Dysart - Culvert at Station 22+510
Conceptual Shoring Location Section

FIGURE SK-3

REF.: 15020-F16



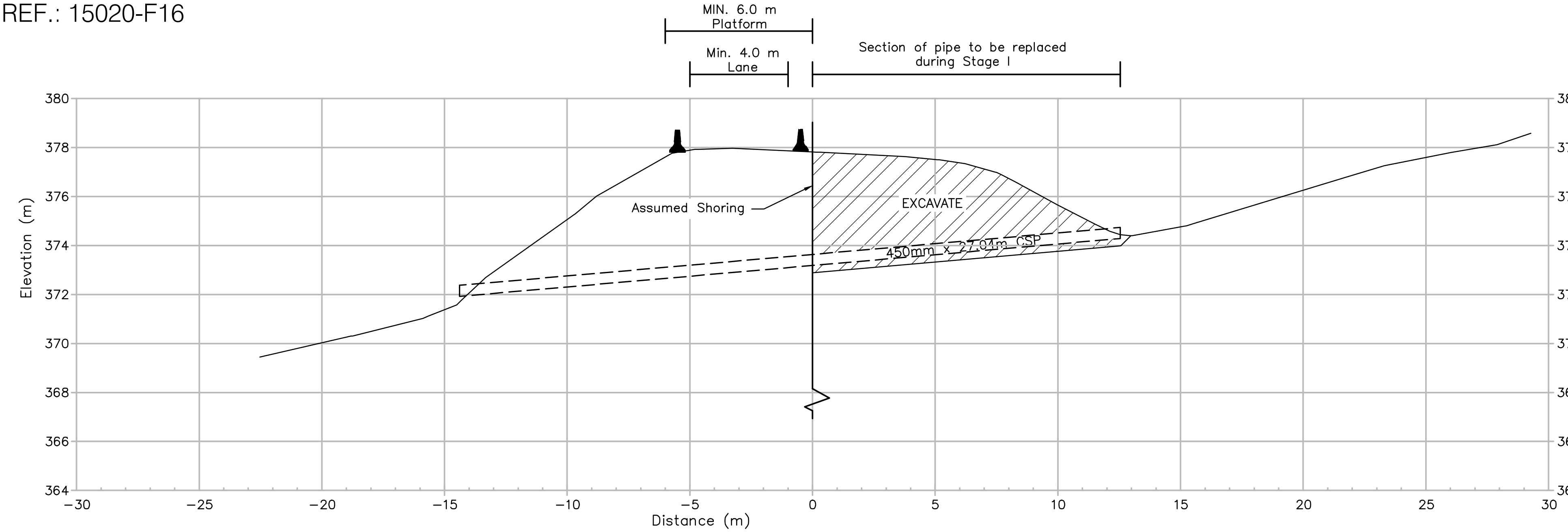
METRIC

Dimensions are in meters and/or millimeters unless otherwise shown. Stations are in kilometers + meters.

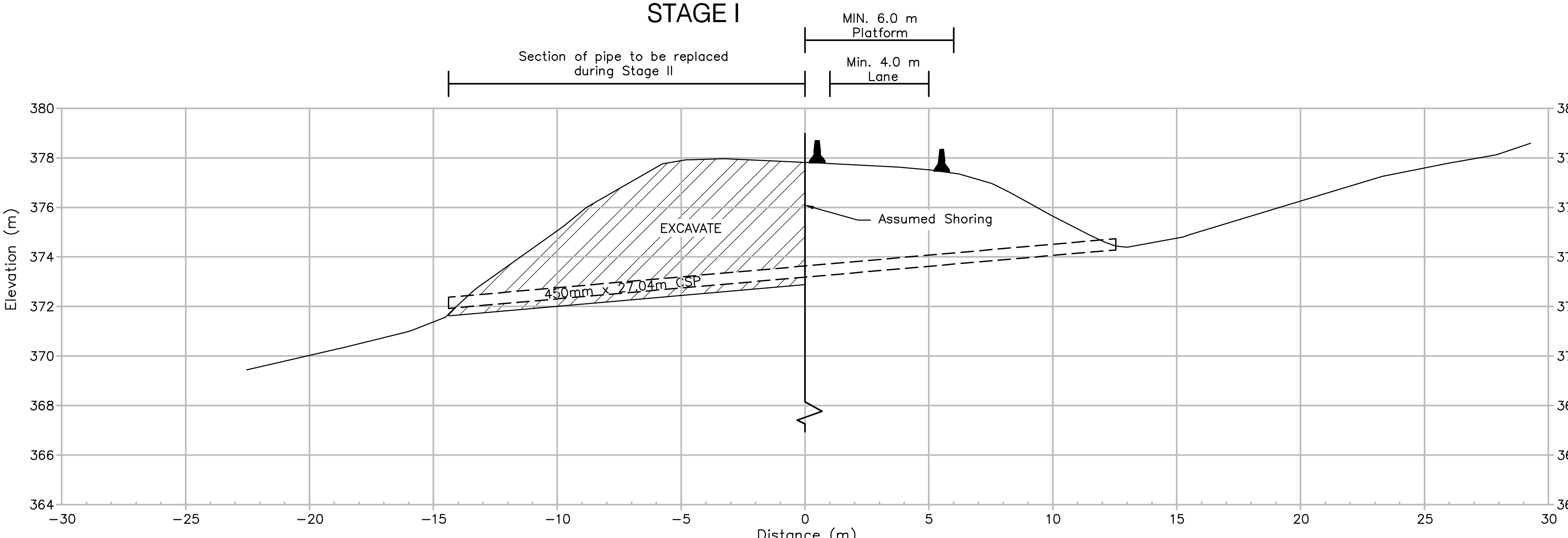
Highway 118, Township of Dysart - Culvert at Station 22+510
Conceptual Shoring Location Plan

FIGURE SK-4

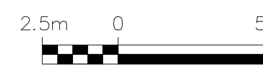
REF.: 15020-F16



STAGE I



STAGE II



METRIC

Dimensions are in meters and/or millimeters unless otherwise shown. Stations are in kilometers + meters.

Highway 118, Township of Dysart - Culvert at Station 22+510 Conceptual Shoring Location Section	FIGURE SK-5

<u>NOTICE TO CONTRACTOR</u> – Obstructions in Fills

Special Provision

The Contractor is notified that, during foundation field investigations for the Structural Culvert at Station 22+510, Township of Dysart, on Highway 118, cobble/boulder sized rock fill was encountered in the embankment fills. The contractor shall take into account these materials when designing and installing the dewatering, shoring protection systems, and/or trenchless construction.