



MERLEX ENGINEERING LTD.

CONSULTING GEOTECHNICAL ENGINEERS

**FINAL
FOUNDATION INVESTIGATION AND
DESIGN REPORT**

**CULVERT SITE NO. 46-209 – Wolf River
Town of Noelville - GWP 330-00-00**

**Highway 64, from 0.3 km South of Highway 528,
Northerly 22.8 km and Highway 535, from the
intersection with Highway 64, northerly 0.65 km**

MEL Ref. No.: 06/11/06161 August 29, 2007

Submitted to:

Earth Tech (Canada) Inc.
189 Wyld Street
North Bay, Ontario
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Geocres No.: 41I-211



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APPENDIX A

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APPENDIX B

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APPENDIX C

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APPENDIX D

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1.0 INTRODUCTION

Merlex Engineering Ltd. (MEL) has been retained by Earth Tech (Canada) Inc., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a foundation investigation at Culvert Site No. 46-209, Town of Noelville, on Highway 64 (GWP 330-00-00). The limits of GWP 330-00-00 are described as: Highway 64 from 0.3 km south of Highway 528, northerly 22.8 km and Highway 535 from the intersection with Highway 64, northerly 0.65 km (see Enclosure No. 2, Key Plan, Appendix A). This project involves the replacement of a 5.33 x 2.60 x 18.6 m SPCSPA at Culvert Site No. 46-209, Wolf River, in the Town of Noelville. The culvert is located about 100 m south of Highway 535 junction at Noelville.

The foundation investigation location was specified by the MTO in the RFP/TPM documentation Agreement No. 5005-E-0070. The terms of reference for the scope of work are outlined in MEL's proposal P-06-052 dated July 18, 2006. The purpose of the investigation was to determine the subsurface conditions in the area of the culvert. MEL investigated the foundation area by the drilling of boreholes, carrying out in-situ tests, and performing laboratory testing on select samples.

2.0 SITE DESCRIPTION

Culvert Site No. 46-209 is situated on Highway 64 in the Town of Noelville. The topography at the site is generally of low relief and the Wolf River flows from west to east at this location. The existing highway embankment is some 15.4 m wide and supports two lanes of traffic, paved shoulder, curb and gutter, and sidewalks on both sides. The height of the embankment above local topography at the culvert location is some 4 m, including 1 m of earth cover over the culvert. The culvert is described as a 5.33 x 2.60 x 18.6 m SPCSPA on Plate No. 1024-64/12-0, WP 88-84-01 surveyed in May 1985. A recent survey (May 2007) established the culvert length as some 32.5 m long, indicating extensions have been added to the ends. The culvert is also



shown on a drawing from Contract No. 65-190, WP 264-62-1. Anecdotal evidence indicates that, prior to the installation of the culvert, a wood bridge supported on a pile foundation was located at this crossing of the Wolf River.

2.1 Site Geology

Bedrock in the area is late Precambrian, felsic anorthosite suite, comprising of granitic to syenitic rocks and derived gneisses (OGS Map 2440). For the project area, the surficial geology maps indicate that surficial conditions generally consist of bedrock at shallow depth (outcrop observed \pm 100 m to the north of nearby church) and overburden soils comprising of glaciolacustrine plain deposits of silts and clays, with frequent organic deposits between the rock outcrops (OGS Map 5003).

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out during the period of March 22 to 24, 2007, and consisted of a total of three (3) sampled boreholes, with accompanying Dynamic Cone Penetration Test (DCPT).

The field investigation was carried out using a Bombardier mounted CME 45B drilling rig equipped with hollow stem augers and standard geotechnical sampling equipment. The boreholes were advanced using 165 mm O.D. continuous flight hollow stem augers and/or 110 mm O.D. continuous flight standard augers. Soil samples were obtained 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 at all borehole locations. In-situ field vane testing, using an MTO "N" size vane, was carried out where appropriate and possible.



Groundwater conditions in the open boreholes were observed during and immediately following completion of the individual boreholes. All open boreholes were backfilled upon completion with the auger cuttings, in the general order they were removed, using reverse augering techniques. Where necessary, the boreholes were capped off with a bentonite (hole plug) grout or patched with a granular fill and cold mix when located on the existing asphalt surface.

The field work for this investigation was under the full time direction of a senior member of our engineering staff, 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, Atterberg Limits determination, grain size analysis (sieve and/or hydrometer), and specific gravity testing. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix B), with a summary of select results presented on the laboratory sheets in Appendix C (Figure L-1).

The location of the individual boreholes were surveyed in the field using highway chainage and offset relative to highway centerline.



4.0 SUBSURFACE CONDITIONS

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Record of Borehole Logs (Appendix B) and on Figure No. 1 (Appendix C). 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, and the results of SPT and DCPT, 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.

4.1 Culvert Site No. 46-209, Station 12+669, Town of Noelville

A plan and profile showing the borehole locations and stratigraphic sequences is shown on Figure No. 1. During the course of the exploration program, three (3) sampled boreholes (Borehole Nos. 1A, 2, and 3) were put down at this site. Borehole No. 3 was advanced from the surface of the existing highway embankment along the east shoulder, as was DCPT No. 3A. Borehole Nos. 1A and 2 were advanced at the west and east ends of the existing culvert, respectively. Borehole No. 1 was advanced adjacent to Borehole No. 1A, through peat with cobble and boulder fill, in order to allow driving of the DCPT.

Borehole Nos. 1A and 2, advanced at the culvert inlet and outlet locations, revealed a thin surficial layer of black silty organics overlying a peat deposit. The peat was a dark brown fibrous peat mixed with cobbles and boulders at Borehole No. 1A and rockfill in the upper portion at Borehole No. 2. The fibrous peat extended to depths ranging between 2.4 and 2.0 m below grade at Borehole Nos. 1A and 2, respectively. The lower portion of the peat in Borehole No. 2 contained layers/seams of dark grey organic clay and some fine sands.



Underlying the peat stratum, a deposit of dark grey organic clay was penetrated to depths of 5.0 and 5.5 m below grade at Borehole Nos. 1A and 2, respectively. The organic clay contained a fine fiber thatching sporadically throughout the samples and occasional coarser (± 5 mm diameter) inclusions. Atterberg limits testing carried out on the deposit returned values of 61 to 94%, with plastic limits ranging from 30 to 56%, and natural moisture contents ranging from 48 to 84%. Several tests to establish the liquid limit from an oven dried state were carried out and indicated a decrease in the liquid limit of 14 to 28%. In-situ shear strength tests carried out on this deposit returned values of 28 to 44 kPa, indicating a firm consistency. Particle size analysis carried out on this deposit indicated 0% gravel size particles, 8 to 11% sand size particles, 65 to 71% silt size particles (based on 0.002 mm diameter grain size split between silt and clay), and 21 to 24% clay size particles. This deposit has been classified based on the Atterberg Limits testing and it is considered, based on past experience, that this organic clay deposit is normally consolidated.

At the location of Borehole No. 3, advanced from the top of the embankment, ± 75 mm of asphalt underlain by ± 400 mm of crushed gravel was penetrated overlying a fine to medium sand fill. Gradation analysis on two samples of the fill indicated 1 to 11% gravel size particles, 69 to 72% sand size particles, and 17 to 30% silt and clay size particles. Underlying some 3.9 m of embankment fill, the grey organic clay was also penetrated. At this location, the natural moisture content of the samples ranged between 63 and 83%. The in-situ shear strength values were higher in this deposit, due to the pre-loading associated with the embankment fill, and returned values ranging between 84 and 104 kPa. The lower part of this stratum contained numerous pockets/seams of fine sands, which returned lower moisture content values. This deposit extended to a depth of approximately 7.3 m below grade (elevation 205.9 m).



A consolidation test (see Figures L-3 and L-4 in Appendix C) was carried out on Sample No. 7 (Borehole No. 3) from a depth of 6.5 m indicating the deposit was marginally overconsolidated with a p_c value of 100 kPa and an existing effective overburden pressure (p_o) of 85 kPa.

Underlying the organic clay deposit, a grey silty clay was penetrated. Atterberg Limits testing was carried out on this deposit and returned liquid limit value ranging from 23 to 46%, with plastic limits ranging from 18 to 23%, and natural moisture contents ranging from 31 to 49%. Based on in-situ field vane testing, which returned values ranging from 26 to 40 kPa, the consistency of this deposit is described as firm. This clay deposit extended to depths ranging between 9.2 and 12.4 m below grade at the borehole locations.

Underlying the grey silty clay deposit, a stratum of fine sand was penetrated at Borehole Nos. 1A and 3. Gradation analysis carried out on a sample from Borehole No. 3 returned 0% gravel size particles, 96% sand size particles, 4% silt and clay size particles. Based on the SPT values, which ranged from 2 to 18 blows per 300 mm penetration, the compactness of the deposit is described as very loose to compact. The sand deposit extended to depths of 12.4 and 14.2 m below grade, where sampling was terminated in Borehole Nos. 1A and 3, respectively. At the location of Borehole No. 2, a grey silt with some fine sand was penetrated underlying the clay. Particle size analysis carried out on the sample indicated 0% gravel size particles, 19% sand size particles, 60% silt size particles, and 21% clay size particles. Atterberg Limits testing carried out on a sample returned liquid limits of 67%, with plastic limits of 57%, and a natural moisture content of 169%. Based on the Atterberg Limits test results, the silt is classified as an inorganic silt of high compressibility. Based on the SPT values, which ranged between 3 to 8 blows per 300 mm penetration, the silt deposit is described as in a very loose to loose state of compactness. The silt deposit extended to a depth of 11.8 m below grade, where sampling was terminated.



Adjacent to Borehole No. 1A, a DCPT was advanced from a depth of some 2.4 m below grade to refusal at 11.4 m. At this location it was necessary to pre-drill through cobbles and boulders to a 2.4 m depth. At the location of Borehole No. 2, a DCPT was driven from grade and met refusal at a depth of 18.1 m. From the top of the existing embankment, at the location of Borehole No. 3, a DCPT was driven from the bottom of the borehole and met refusal at a depth of 27.4 m (elevation 185.4 m). Adjacent to Borehole No. 3, a DCPT was driven from grade and met refusal at 20.7 m below grade (elevation 192.5 m).

4.2 Groundwater Conditions

At the location of Borehole Nos. 1A and 2, put down adjacent to the culvert inlet and outlet, groundwater levels varied between 0.2 to 0.5 m below existing grade. At the time of advancing the boreholes the river level rose quickly, due to precipitation overnight, and both boreholes were flooded the last morning of the field investigation.

Borehole No. 3, advanced through the embankment, encountered groundwater during drilling at approximately elevation 210.0 m. Groundwater levels will fluctuate seasonally.

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5.0 DESIGN COMMENTS AND RECOMMENDATIONS

5.1 General

The existing culvert, located in the Town of Noelville, is identified as Culvert Site No. 46-209 and is described on Plate No. 1024-64-12-0 for WP 88-84-01 as a 5.33 x 2.60 x 18.60 m SPCSPA. At this location, the Wolf River flows from west to east under Highway 64, which is oriented in a north south direction. The height of the embankment above local topography, at the culvert location, is about 4 m including 1 m earth cover over the culvert. The embankment platform is some 15.4 m wide and supports two lanes of traffic (north south direction), paved shoulders with curb and gutter, and sidewalks on both the left and right sides (see Photo 3). Two catch basins are located directly above the culvert, at left and right curbs, and they discharge vertically down directly into the culvert. During our investigation we received anecdotal information that the culvert ends were extended some 20 years ago and this information is confirmed by recent survey data, which now identifies the culvert as being 30.52 m long. We were also informed that prior to the installation of the culvert, a wood bridge supported on a piled foundation was at this location. This wood bridge structure had to be present prior to 1962, since drawings for Contract No. 65-190, WP 264-62-1 show a pipe arch culvert at this location.

The 1985 profile shows a distinct sag at the culvert location. The profile alignment was compared with the 1965 alignment and it was calculated that the 1985 profile was some 750 mm lower in elevation. The existing (2007) centreline profile is essentially the same as the 1985 profile elevations. Present plans do not call for changes to the vertical alignment at this location.

It is understood that due to hydraulic insufficiencies it has been decided to replace the culvert with a larger concrete box structure. The hydraulic capacity and size of the replacement has yet to be determined. Presently under consideration is replacing the existing culvert with precast



concrete box units. It is proposed to construct a culvert with a longer length (without wing walls), such that replacement operation could be carried out under a 24 hour continuous operation. This would not require staging and would minimize the duration of construction impact to the public in this urban area.

5.2 Foundation Consideration

The existing 5.3 x 2.6 SPCSPA is founded on a deposit of organic clay, which extends over a depth of some 3.4 m at the location of Borehole No. 3 (elevation 209.3 to 205.9 m). The thickness of the deposit is somewhat less at the inlet (2.6 m at Borehole No. 1A) and about the same thickness at the outlet (3.5 m at Borehole No. 2). The organic clay deposit has been preloaded for over 40 years with some 4 m of embankment fill at the culvert location. This preload has resulted in compression of the organic clay. This is evidenced by the higher in-situ shear strength values of 84 to 104 kPa at Borehole No. 3A advanced through the embankment versus the lower values obtained at Borehole Nos. 1A and 2 (ranging from 28 to 44 kPa), which were advanced at the inlet and outlet locations beyond the area of influence of the embankment fill.

The difference in centreline profile elevations, between the 1965 and 1985 surveys, indicates that the magnitude of settlement is possibly in the order of 750 mm. Recent survey data (May 2007) indicates that the present centreline elevation is the same as the 1985 survey data. This similarity in centreline elevations over the last 22 years indicates that both primary and essentially all secondary consolidation, associated with the embankment fill load, has occurred. Provided there is no net load increase along this embankment then, theoretically, there will be no further settlement associated with consolidation of the compressible organic clay stratum or with the underlying firm silty clay and silt deposits.



Hydraulic considerations will require a larger opening for a concrete box structure. The mass of a precast concrete box unit is greater than a SPCSPA per metre of cross-sectional length, however, when the mass of the pipe arch and surrounding backfill soil that would be displaced by the wider precast box unit is taken into consideration, the mass per unit length may be similar. As such, it may be possible, with the existing design concept, to achieve the goal of zero net load increase on the underlying clay strata.

As noted above, the founding organic clay, which is slightly overconsolidated, is sensitive to disturbance during construction. However it is judged feasible to support rigid precast concrete box units of equivalent mass provided subgrade conditions are maintained in a stable state and the subgrade strength is preserved during construction.

5.3 Dewatering and Excavation

It is imperative that a properly designed groundwater control system be installed and operational well in advance of excavation and installation of the precast box units, such that an unwatered excavation is maintained. The water level in the creek was at elevation 210.9 m on May 11, 2007 and, due to precipitation, Borehole Nos. 1A and 2 were flooded to a depth of 300 mm above grade on the final day of drilling. The presence of rockfill/cobbles and boulders in the fill/peat mix, which is present at the inlet and outlet, will have to be addressed if the contractor intends to install a cut-off/cofferdam wall down to the lower, relatively impermeable, clay deposits. Flow through the embankment granular fills must also be controlled, such that a stable, unwatered excavation is maintained during excavation and construction. Ultimately, the method of dewatering will be the choice of the contractor however, the importance of maintaining the subgrade in an unwatered stable condition during excavation and subgrade preparation/construction cannot be overstressed.



All excavations greater than 1.2 m in depth must be sloped or shored in accordance with the Occupational Health and Safety Act (OHSA) Regulation for construction projects. Temporary open excavations, above the prevailing water table, will be stable at a temporary angle of 1H:1V. Below the prevailing water table, slopes of excavations will have to be flattened to 2H:1V, depending upon the method of dewatering employed. Excavations must be maintained in an unwatered condition during foundation construction and every reasonable effort must be made to prevent disturbance to the founding subgrade.

If a continuous 24 hour operation cannot be carried out, it is possible that roadway protection along the highway centreline over the culvert will be required. Sheet piling is considered appropriate for highway protection during construction. A sufficiently robust sheet pile section must be used to minimize difficulties due to possible cobble/boulder size obstructions in the embankment fill. Over the actual culvert location the sheeting would be carried down to the top of the culvert behind a suitably sized and connected anchorage system. A suitably sized whaler could be provided and tied into the full depth piling and bracing at the structure edge in order to provide support at the top and bottom of the sheeting over the top of the culvert. The full depth piling sections will have to extend an adequate depth into the underlying firm clay deposits to maintain stability of the sheeting and the base of the excavation. Depending on the section properties and penetration depth of the sheeting and walers section properties, a system of rakers or ground anchors may be required. Once one half of the box culvert is complete, a system of dead man type anchors could be installed to tie back the sheeting as the highway fill is brought up to grade. When the excavation on the other side reaches the anchor depth, a support whaler can be placed and tensioned as specified in the contractor's approved sheeting design.



5.4 Subgrade Preparation

The results of Borehole No. 3 indicate that a granular bedding surrounds the existing culvert with the invert at, or very close, to the native organic clay stratum. The elevation of the replacement box units should be at a similar elevation to minimize, excavation into the organic clay stratum. Excavation to prepare the subgrade should be carried out with a flat bladed grade-all type bucket or, if a bucket with teeth is used, a flat plate should be attached across the teeth to prevent gouging the subgrade. Prior to placement of the culvert bedding, a Class II non-woven geotextile with a Factor of Safety (FOS) of 50 - 100 should be placed across the full width of the subgrade behind the box units and to 1.0 m beyond the side walls of the box to act as a separator. To further reduce the risk of differential settlement of the box units, we recommend a layer of BX1200 geogrid as produced by Terrafix (or equivalent design) over the geotextile, followed by 450 mm of bedding material. Considering the wet subgrade conditions, we recommend that a 19 mm clear stone be used as bedding material.

5.5 Design Bearing Pressure

Based on the above noted natural soil conditions and founding elevation, we have determined, below the existing embankment, a factored bearing resistance value at ULS of 150 kPa. As noted in the previous, the marginally overconsolidated organic clay deposit is very susceptible to changes in loading. Based on our understanding of the past response of the deposit to the embankment dead load associated with 4 m (height) of fill, for a geotechnical reaction at SLS of 25 mm total settlement, a net ultimate bearing pressure increase of 10 kPa is applicable. If a geotechnical reaction at SLS of 50 mm total settlement is considered, the net ultimate bearing pressure increase of 20 kPa would be appropriate.

These settlements would result in a minor sag along the longitudinal alignment of the culvert. This development can be compensated for by installing the culvert with a slight (± 50 mm)



camber. To address concerns raised by MTO structural section dealing with future grade raises plate type connections should be installed between the concrete box units. During culvert replacement, the river flow will have to be diverted by pumping through a temporary culvert. Pumping capacity, with adequate backup system, will have to be installed. Replacement under a 24 hour continuous operation will reduce the risk that a storm event will occur during the culvert replacement stage of the project.

If the above noted constraints cannot be met during the detailed design, then consideration will have to be given to the use of a deep (piled) foundation. Alternatively, considering the 40 year plus performance of the existing SPCSPA, replacement with a more flexible structural plate arch, with a galvanized treatment to resist the corrosive nature of the groundwater, could also be considered. A nesting of plastic (HDPE) pipes may also be a possible solution to achieving a zero net load increase.

5.6 Deep Foundation

As discussed, the loads to be supported are relatively low and within the range that can be accommodated by a timber pile foundation. A No. 14 timber pile, 15 m in length, with a butt diameter of 350 mm and tip diameter of 230 mm can supply a factored axial resistance at ULS of 160 kN and an axial resistance at SLS of 120 kN per pile at a 15 m depth. The pile toes will require protection against possible damage due to obstructions, which may be encountered during driving, specifically in the area of the inlet.

If greater pile capacity is required, a steel H pile, driven to practical refusal on bedrock, would be an appropriate selection. If a higher capacity pile is required, further geotechnical information below the 20 m depth investigated during this program must be obtained prior to finalizing design for this type of deep foundation.



5.7 Lateral Earth Pressure

A free draining granular material, such as OPSS Form 1010 Granular B Type I, is recommended as appropriate backfill to the culvert in order to insure the prevention of hydrostatic pressure build-up. The embankment fill should be placed in a balanced manner on the outer sides of the box units.

Lateral earth pressures should be computed in accordance with the Canadian Highway Bridge Design Code (CHBDC). The design parameters are as follows:

	<u>Granular A</u>	<u>Granular B Type I</u>	<u>Rock Fill</u>
Angle of Internal Friction (degrees)	35	30	43
Unit weight (KN/m ³)	22	20	18.5
Active earth pressure (Ka)	0.27	0.33	0.19
At-rest earth pressure (Ko)	0.43	0.50	0.32

For rigid structures, such as precast concrete box units, deflection cannot occur, as such the “at-rest” condition (Ko) applies.

5.8 Constructability

The existing Wolf River Culvert is a 5.33 x 2.60 x 30.52 m SPCSPA supported on a firm to stiff organic silty clay however, beyond the embankment footprint the clay deposit is of a lower consistency. Care must be exercised during excavation of the subgrade material to minimize disturbance (ie. use grade-all type bucket to prevent gouging the subgrade) and the subgrade is to be reinforced with a geogrid. Effective groundwater control is necessary to allow open excavations and maintain a stable supporting subgrade. It is strongly suggested that,



considering the ample platform width, a continuous 24 hour operation be used for culvert replacement.



6.0 CLOSURE

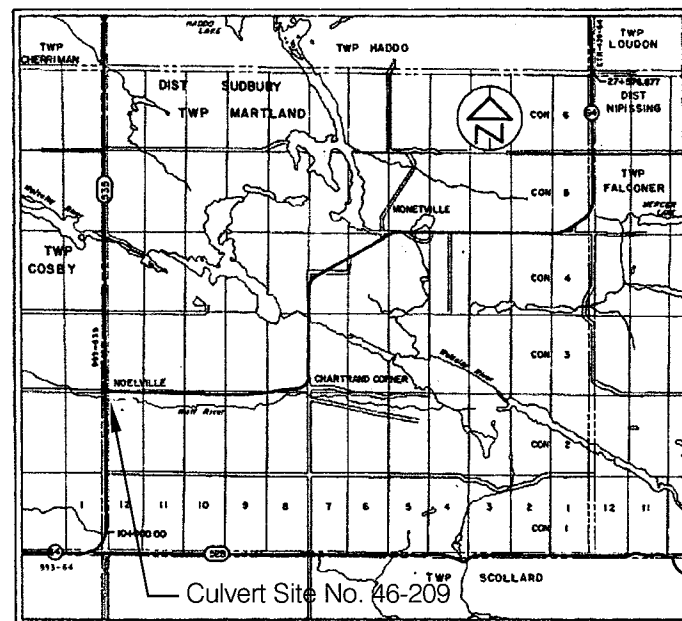
Information provided in this report is valid only at the locations described above. Any assumptions of continuity of soil stratigraphy between boreholes, as shown on the enclosed cross-sections, is intended as an aid for design purposes only and does not constitute a statement of existing conditions for contractual or construction purposes.

Details of the investigation, the material analysis and recommendation in this report are considered to be complete. However, should any questions arise, please do not hesitate to contact the undersigned.

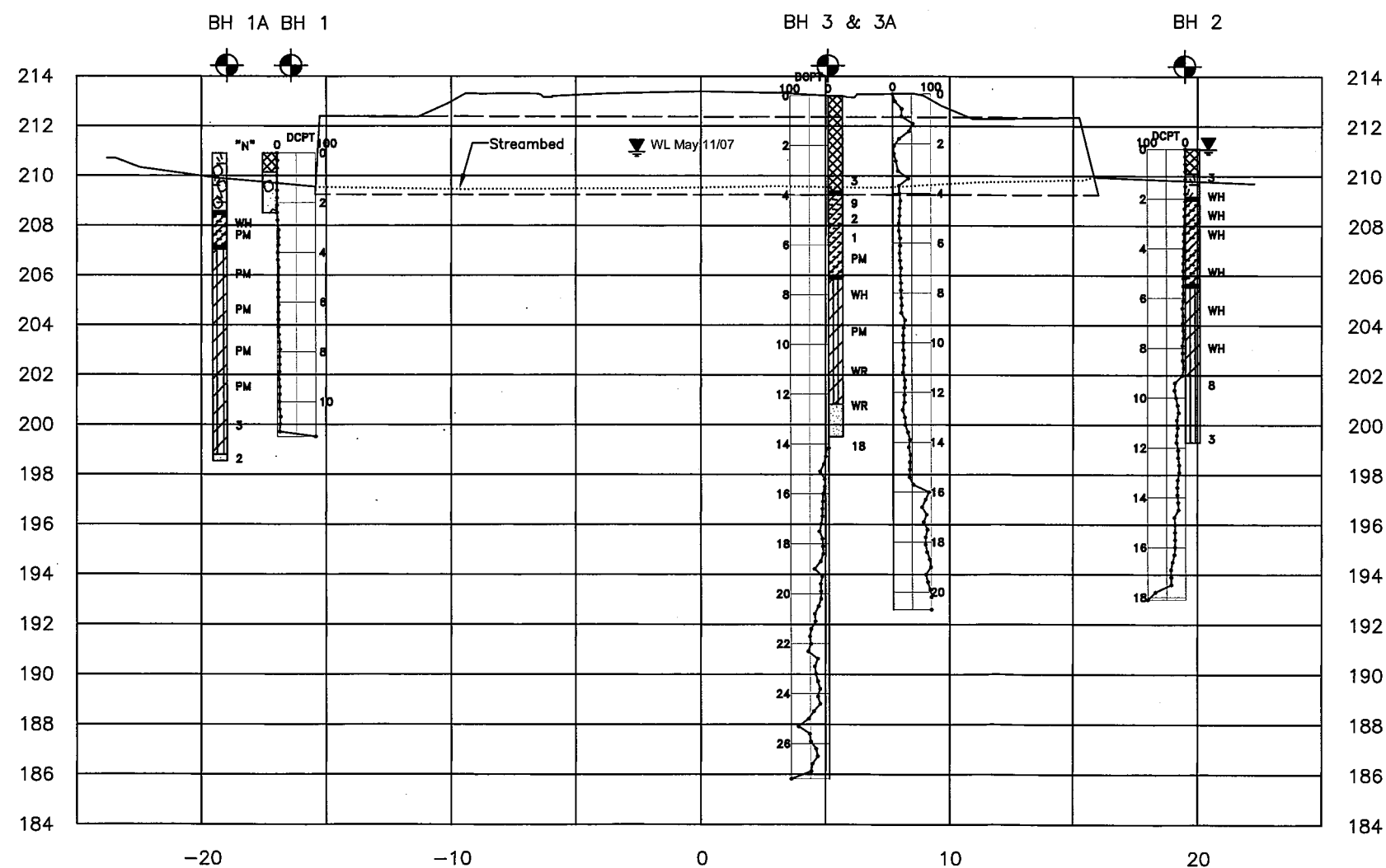
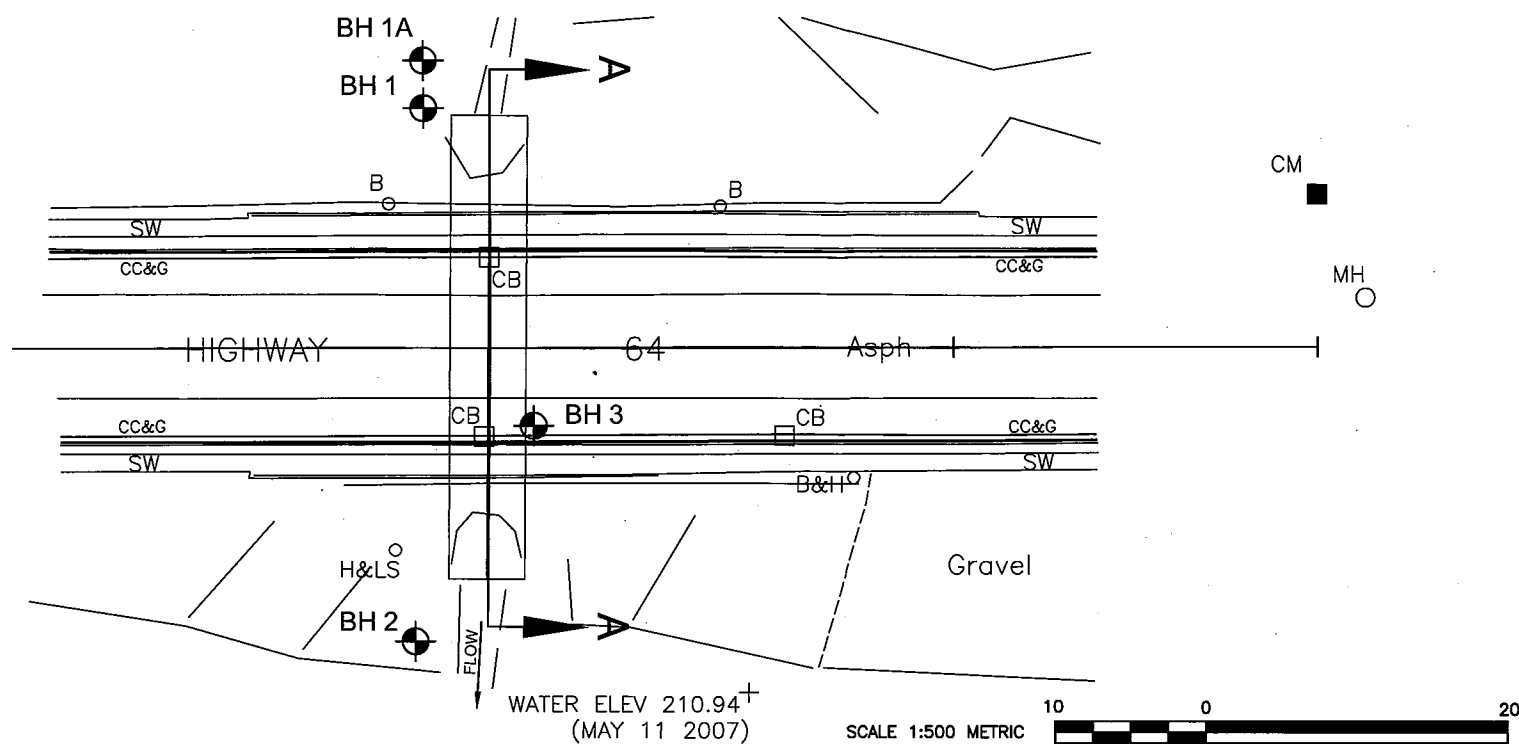
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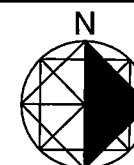
KEY PLAN



Cross Section A-A
Cross Section @12+669.1

SCALE 1:250 METRIC

CONT No 0
GWP No 330-00-00



HWY 64 - Wolf River Culvert
- Town of Noelville

Site No. 46-209
Section 12+669.1

BOREHOLE LOCATIONS & SOIL STRATA

Figure
1



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Consulting Geotechnical Engineers

STRATIGRAPHY LEGEND

	TOPSOIL		SAND		SILTY SAND/ SANDY SILT
	PEAT		SAND & GRAVEL		SILT
	FILL		CLAY		SILTY CLAY
	SILTY CLAY		TILL		COB/BLD MIXED WITH PEAT
	ORGANIC CLAY		ORGANIC SILTS		

LEGEND

- Borehole and Dynamic Cone Penetration Test
- Borehole
- Dynamic Cone Penetration Test
- N Blows/0.3 m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3 m (60° Cone, 475 J/blow)
- Water Level at Time of Investigation
- Auger Refusal at Elevation

Borehole No.	Co-ordinates		Elevation
	Station	Offset	
Borehole No.1	12+664.8	16.4 m Lt	211.0
Borehole No.1A	12+664.8	19.0 m Lt	211.0
Borehole No.2	12+664.3	19.5 m Rt	211.1
Borehole No.3	12+672	5.1 m Rt	213.2
Borehole No.3A	12+673	5.1 m Rt	213.2

NOTE 1:
The boundaries between soil strata have been established at the borehole locations only. The boundaries between boreholes are assumed based on borehole data.

MEL Ref.: 06161F - Borehole Location Plan - MEL

DATE	BY	DESCRIPTION
14/05/07	DVL	REV 0 - Cross Section and Borehole Plan
15/05/07	DVL	REV 1- Minor Changes to Borehole Samples
28/08/07	DVL	REV 3 - Added Key Plan
00/00/00	DVL	REV 4-

HWY No. 64 - Wolf Creek Culvert			DIST
SUBM'D		DATE 14/05/07	SITE 46-209
DRAWN DVL	CHK MAM	DATE 14/05/07	FIG 1