

FOUNDATION INVESTIGATION  
AND DESIGN REPORT

PROPOSED REHABILITATION OF  
STRUCTURAL CULVERT SITE NO. 30-676/C  
TOWNSHIP OF RAMA  
HIGHWAY 12 FROM GAMEBRIDGE TO RAMA ROAD 25

W.P. 365-98-00  
Agreement # 2004-E-0070



I.E.  
Group

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## **PART A – FOUNDATION INVESTIGATION**

### **1.0 INTRODUCTION**

This report presents the results of a foundation investigation carried out in December 2008 and February 2009 by Infrastructure Engineering Group Inc. (IEG) on behalf of Morrison Hershfield Limited (Morrison Hershfield).

This assignment involves the rehabilitation of approximately 24 km of Highway 12, from Rama Road to Gamebridge. The original scope of the rehabilitation is based on addressing the immediate and short term deficiencies identified in the Ministry's Highway Assessment Report for W.O. #03-20019 (February 2005). The scope of work may include extension or replacement of seven (7) non-structural culverts and four (4) structural culverts.

Foundation investigation and recommendations are required for the design and construction of culvert replacements and/or extension as part of the improvement of Highway 12. Seven (7) non-structural culverts and four (4) structural culverts are to be investigated. The scope of work was subsequently changed to include rehabilitation/replacement of non-structural Culvert C03, and rehabilitation of structural Culvert C28, and no work to be done on structural Culverts C14, C15 and C25.

This report covers the site of Structural Culvert No. 30-676/C, also described as C28 in this report, and in the culvert summary as Culv 28.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes and, based on the findings, to provide geotechnical recommendations for the foundation elements.

Based on the information presented in the Preliminary Drawings provided by Morrison Hershfield, and verbal discussion with the project team, it is understood this culvert will be rehabilitated and will not be replaced. The rehabilitation will consist of concrete repairs and installation of tie-back anchors near the toe of the culvert walls.

Authorization to complete this assignment was given by Mr. Stanley Ma, P. Eng., of Morrison Hershfield, the TPM Consultant who is completing this assignment for MTO under Agreement # 2004-E-0070.

## **2.0 SITE DESCRIPTION**

### **2.1 Site Location**

The project alignment starts in Gamebridge, at Station 10+000 and extends northerly to approximately Station 19+200 just south of County Road 169, then extends north westerly to approximately Station 24+800 just before Side Road 15, then extends westerly to Station 34+000 just east of Rama Road 25. For the purpose of description, standard MTO conventional description will be used, i.e. a site north pointing in the direction of increasing chainage. When facing the direction of increasing chainage, the right hand side is referred to as east, and the left hand side is referred to as west. Any directions with clarifications in brackets (e.g., north-west) are given with reference to the true north direction.

Structure 30-676/C is located on Highway 12, approximately 16.5 km north of the south limit of this Contract at Gamebridge (Station 10+000), located at Station 26+514. Photographs of this culvert site are presented in Appendix "D". The Culvert Summary provided by Morrison Hershfield indicates that the existing structure is a reinforced concrete, rigid frame open footing culvert with a span of 3.66 m, a height of 1.8 m, a length of 20.6 m (3.66 m span by 1.53 m height in accordance with ETR Plate No. 205-12/55-0), with an overfill height of approximately 0.9 m. The culvert opening dimensions were obtained from the Culvert Summary provided by Morrison Hershfield and compared with the ETR drawings provided in the RFP.

This culvert is located within a drainage valley in which the stream easterly (northerly). The approach embankments were built on both the east and west sides of the culvert, with a maximum height of approximately 2.9 m. The embankment slopes are typically 3H:1V or flatter and are grass covered. No signs of embankment slope instability were observed at the time of this foundation investigation.

There are no headwalls for this culvert and the ends of the culvert protrude beyond the road embankment. The water levels were observed above the bottom of the creek, at an approximate Elevation of 222.6 m on December 17, 2008, likely reflecting a normal flow condition.

Photographs taken on March 1, 2002, as shown in Appendix B of the Highway Assessment Study Report indicate that water level was slightly lower than those observed during the field work.

### **2.2 Physiography and Topography**

The project alignment except for the extreme western portion is located within the Simcoe Lowlands physiographic region (Chapman and Putnam, 1984). This area was previously flooded by glacial Lake Algonquin. The portion of the alignment located east of the Atherley Narrows (narrows between Lakes Couchiching and Simcoe) is comprised of an elevated, drumlinized till plain comprised primarily of undifferentiated sand to sandy silt (Chapman and Putnam, 1984). The character of local topography and soils in proximity to the highway corridor elsewhere are predominantly comprised of clay plain with interspersed elongated drumlins comprised of

calcareous till (kame moraine) (Chapman and Putnam, 1984). There is a large patch of peat/muck located on the east shore of Lake Simcoe associated with several of the wetland features located along the lakeshore. There is also a section of Carden limestone plain located north of the Talbot River at the south end of the study area. This area is characterized as limestone overlaid with a very shallow overburden (Chapman and Putnam, 1984).

The topography of the study area is primarily flat with scattered drumlin features. The area slopes gently down towards Lake Simcoe. There are numerous headwater areas of small size that traverse the ROW of Highway 12. Movement of shallow ground water is confined by the tight till and clay soils and would follow surficial topography towards Lake Simcoe.

There are six provincially significant wetlands (PSW) located in part within the project alignment. From west to east, they include the Orillia Filtration Swamp, Victoria Point Wetland, Atherley Wetlands, Mud Lake Wetland, Barnstable Bay Wetland, and the Lagoon City Wetland.

The asphalt pavement surface over the existing culvert is near Elevation 224.9 m while the ground surface at the base of the embankment at the stream bed is at approximate Elevation 221.9 m.

### **3.0 INVESTIGATION PROCEDURES**

#### **3.1 Field Investigation**

Between December 3, 2008 and February 18, 2009, a CME 55 truck-mounted drill rig was supplied by London Soil Test Ltd. and used on site for drilling and Standard Penetration Testing (SPT, following the procedures of ASTM D 1586). Two (2) boreholes (Boreholes C28-2 and C28-3) were drilled and sampled to obtain data for foundation design of the proposed rehabilitation work and potential culvert replacement. Rock coring was carried out on February 18, 2009 in Borehole C28-3 to provide geotechnical data as per the requirements of our proposal for this work. Hand-drilled boreholes cannot be completed at the location of Borehole C28-1 and C28-4 due to inundation of the area, and a series of dynamic cone penetration tests were carried out instead. The locations of the boreholes are shown on Drawing 1.

The culvert borehole numbering system was established from the Culvert Summary spreadsheet provided by Morrison Hershfield. The subject Culvert was identified as Culv 28, with a Structure Number 30-676/C as presented in the Culvert Summary. The boreholes for this culvert are numbered C28-1 to C28-4 accordingly.

The boreholes were numbered C28-1 to 28-4 for the subject culvert and the depths of sampling were as follows:

<b>Borehole No.</b>	<b>Depth of Sampling (m)</b>
C28-1 (DCP only)	3.76
C28-2	7.32
C28-3 (with rock coring)	9.27
C28-4 (DCP only)	4.39

The sampled boreholes were drilled using continuous flight solid stem or hollow stem augers. Soil samples were retrieved at selected intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT). Samples were generally taken at intervals of depth of 0.75 m to the maximum depth of exploration.

The undrained shear strength was obtained by shear vane test, with the sensitivity measured. Field pocket penetrometer was used on the retrieved SPT samples, where applicable, to determine the undrained shear strength of the cohesive soil deposits. These undrained shear strengths are used to supplement the properties of the cohesive soils. It is noted that the measured shear strength value on the retrieved SPT samples would be slightly lower than the actual value due to sampling disturbance.

Rock cores were retrieved using NQ core assembly (47.6 mm ID). The rock core samples were identified in the field and physical index properties were determined by visual examination and also by measurement of rock quality designations (RQD's) and rock core recovery. All rock cores were placed in wooden core boxes and transported to our laboratory for further examination, to confirm the field logging, and laboratory testing.

Seepage and water levels were noted in each borehole during and at the completion of drilling and sampling. All boreholes were grouted with a bentonite/cement mix at completion of sampling in accordance with Ontario Regulation 903.

Our field engineer, Mr. Ralph Billings, P. Eng., supervised the fieldwork and worked under the direction of the project engineer, Mr. Eric Chung, P. Eng. Our field staff cleared the location of buried utilities and logged the boreholes. The soil samples obtained were placed in labeled containers and transported to IEG's London laboratory for further examination and laboratory testing.

The stations, offsets and ground surface elevations at the as drilled borehole locations were provided by Morrison Hershfield to IEG for the purpose of this report.

The results of the drilling, sampling, in-situ testing and groundwater observations are summarized on the Record of Borehole sheets and enclosed in Appendix "A".

### **3.2 Laboratory Analysis**

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all retrieved soil samples. In addition, grain size analyses, Atterberg Limit tests and unit weight tests were performed on selected samples.

A section of the rock core (at 8.47 m depth from Borehole C28-3) was selected for unconfined compressive strength testing in accordance with ASTM 2938. The testing was performed by Trow Associates Inc. of Brampton and the results are presented as Figure 8 in Appendix B.

The results of the laboratory testing are presented on the Record of Borehole sheets (Appendix “A”), and Laboratory Test Results (Figures 1 to 8, Appendix “B”).

## **4.0 SUBSURFACE CONDITIONS**

### **4.1 General Subsurface Conditions**

Reference is made to the Record of Borehole sheets (Appendix “A”) and Laboratory Test Results (Appendix “B”) for detailed subsurface soil and groundwater conditions encountered in the boreholes. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, consequently, represent transitions between soil types rather than exact planes of geological change. The soil profiles depicting the subsurface conditions on Drawings 1 and 2 will vary between and beyond the borehole locations.

In general, the subsurface deposits at the site consist of loose embankment fill, placed on very soft silty clay or loose to compact sand and silt to sandy silt deposits to depths of 5.03 m (Elevation 219.71 m at Borehole C28-2) and 5.79 m (Elevation 219.00 m at Borehole C28-3). These very soft/loose to compact deposits were underlain by a compact to very dense silty sand stratum to depths of 7.32 m (Elevation 217.42 m at Borehole C28-2) and 7.32 m (Elevation 217.47 m at Borehole C28-3). The silty sand was further underlain by limestone bedrock.

#### **4.1.1 Pavement, Fill**

Boreholes C28-2 and C28-3, located at the edge of existing pavement in the shoulder areas, encountered 360 mm shoulder gravel. Underlying the shoulder gravel is the embankment fill material that extended to depths of 3.05 to 3.35 m, respective Elevations of 221.69 m and 221.44 m at Boreholes C28-2 and C28-3. The fill beneath the shoulder gravel consists of a mixture of silt, sand and gravel with cobbles and silty clay lumps.

Two (2) grain size distribution analyses of the embankment fill is shown on Figure 1 of Appendix “B”. The results of an Atterberg Limit test are provided in Figure 2.

Standard penetration tests yielded “N”-values from 4 to 12 blows per 0.3 m. This fill is brown to dark brown in colour and the measured natural moisture contents range from 10 to 42%. The



higher moisture contents reflect the presence of organic matters and clay lumps. Based on one sample, the unit weight of the fill was measured to be  $22.1 \text{ kN/m}^3$ .

Based on the above field and laboratory test results, together with visual and tactile examination, the fill beneath the shoulder gravel consists of a mixture of silt, sand and gravel with cobbles and silty clay lumps and has a loose to compact compactness condition.

#### **4.1.2 Silty Clay**

A 0.76 m thick layer of brown to grey silty clay was contacted below the embankment fill at Borehole C28-2 and extended to a depths of 3.81 m below the existing ground surface (at Elevations 220.93 m). The silty clay also contains organic inclusions.

A single grain size analysis was performed and the results are presented on Figure 3 of Appendix "B". The same sample was tested for Atterberg Limits and the results in Figure 4 of Appendix "B" and summarized below:

Liquid Limit ( $W_L$ )	25%
Plastic Limit ( $W_P$ )	14%
Plasticity Index ( $I_p$ )	11%
Natural Moisture Content ( $W$ )	49%

A standard penetration test yielded an "N"-value of 1 blow per 0.3 m. The low N-value could also be attributed by disturbance from the drilling operations. Based on the above field and laboratory test results, together with visual and tactile examination, the silty clay deposit is considered to have a very soft consistency and can be classified as a clay of low plasticity (CL).

#### **4.1.3 Sand and Silt**

The silty clay at Borehole C28-2 and the embankment fill at C28-3 were underlain by a sand and silt deposit that extended to depths of 5.03 to 5.79 m, at Elevations of 219.71 m and 219.00 m in Boreholes C28-2 and C28-3, respectively.

Three (3) grain size analyses were performed and the results are presented on Figure 5 of Appendix "B". One (1) sample was tested and exhibited the following Atterberg Limits. These results are shown in Figure 6 and summarized below:

##### **CL-ML, Sample at 5.33 m from Borehole C28-3**

Liquid Limit ( $W_L$ )	16%
Plastic Limit ( $W_P$ )	10%
Plasticity Index ( $I_p$ )	6%
Natural Moisture Content ( $W$ )	9%

The natural moisture contents were in the range of 9 to 24%, indicative of wet to saturated moisture condition. The results of the grain size and Atterberg Limit tests indicate that the sand and silt is generally non-plastic to slightly plastic and can be classified as a SM-ML material with occasional clayey pockets (CL-ML).

Standard penetration tests yielded “N”-values between 2 and 12 blows per 0.3 m, indicative of very loose to compact compactness condition.

#### **4.1.4 Silty Sand (Till-like)**

The sand and silt to sandy silt were underlain by a silty sand deposit which has a till-like structure.

A single grain size analysis was performed and the results are presented on Figure 7 of Appendix “B”.

The natural moisture contents of the silty sand were in the range of 8 to 16%, indicative of damp to moist moisture condition. Standard penetration tests yielded “N”-values 20 and over 100 blows per 0.3 m. Based on the above field and laboratory test results, together with visual and tactile examination, the silty sand till-like deposit exhibited a compact to very dense compactness condition.

#### **4.1.5 Limestone Bedrock**

The silty sand till-like stratum was further underlain by a grey to tan limestone bedrock at depths of 7.32 m (Elevation 217.42 m at Borehole C28-2) and 7.32 m (Elevation 217.47 m at Borehole C28-3). The appearance of the rock core sample is fossiliferous with sections that are coralliferous, with close to wide bedding planes.

Recovery of the rock core sample was at 100%. Rock Quality Designation (RQD) varied from 62 to 81%, indicative of a fair to good quality.

A single uniaxial compressive strength determination carried out on a section of rock core samples yielded a result of 128 MPa. The uniaxial compressive strength test report is presented in Figure 8.

### **4.2 Groundwater Conditions**

The groundwater condition was monitored during and upon completion of sampling. On completion of drilling, groundwater was observed in Boreholes C28-2 and C28-3 at depths of 2.3 and 2.1 m, corresponding to Elevations 222.44 and 222.69 m.

The water levels were observed above the bottom of the creek, at an approximate Elevation of 222.6 m on December 17, 2008, likely reflecting a normal flow condition.

Photographs taken on March 1, 2002, as shown in Appendix B of the Highway Assessment Study Report indicate that water level was slightly lower than those observed during the field work.

It should be noted that the groundwater level will fluctuate seasonally and in response to weather events. Under adverse conditions, water could be perched within the embankment fill. It is reasonable to assume that groundwater could be similar to the water level in the creek during high flow conditions.

## **PART B – FOUNDATION DESIGN**

### **5.0 DISCUSSION AND RECOMMENDATIONS**

#### **5.1 General**

This section of the report provides our recommendations on the geotechnical aspects of foundation design of the proposed replacement/extension of Structural Culvert No. 30-676/C, based on our interpretation of the factual information obtained during this investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

Structure 30-676/C is located on Highway 12, approximately 16.5 km north of the south limit of this Contract at Gamebridge (Station 10+000), located at Station 26+514. Photographs of this culvert site are presented in Appendix “D”. The Culvert Summary provided by Morrison Hershfield indicate that the existing structure is a reinforced concrete, rigid frame open footing culvert with a span of 3.66 m, a height of 1.8 m, a length of 20.6 m (3.66 m span by 1.53 m height in accordance with ETR Plate No. 205-12/55-0), with an overfill height of approximately 0.9 m. The culvert opening dimensions were obtained from the Culvert Summary provided by Morrison Hershfield and compared with the ETR drawings provided in the RFP. The culvert summary also indicated that this Culvert consists of 4 sections:

- East Section (exterior extension), 4.95 m length;
- East Intermediate Section (original), 9.0 m length;
- West Intermediate Section, 1.74 m long;
- West Section (exterior extension), 4.95 m length.

Details of the varying height of the culverts in these sections were not provided in the Highway assessment report, nor on the culvert summary or preliminary design drawings. It should be pointed out that the original culvert likely consist of the 9.0 m length with a height of 1.53 m as shown on the ETR Plate, and the remaining sections are assumed to be 1.8 m.

This culvert is located within a drainage valley in which the stream easterly (northerly). The approach embankments were built on both the east and west sides of the culvert, with a maximum height of approximately 2.9 m. The embankment slopes are typically 3H:1V or flatter and are grass covered. No signs of embankment slope instability were observed at the time of this foundation investigation.

There are no headwalls for this culvert and the ends of the culvert protrude beyond the road embankment. The water levels were observed above the bottom of the creek, at an approximate Elevation of 222.6 m on December 17, 2008, likely reflecting a normal flow condition.

Photographs taken on March 1, 2002, as shown in Appendix B of the Highway Assessment Study Report indicate that water level was slightly lower than those observed during the field work.

In general, the subsurface deposits at the site consist of loose embankment fill, placed on very soft silty clay or loose to compact sand and silt to sandy silt deposits to depths of 5.03 m (Elevation 219.71 m at Borehole C28-2) and 5.79 m (Elevation 219.00 m at Borehole C28-3). These very soft/loose to compact deposits were underlain by a compact to very dense silty sand stratum to depths of 7.32 m (Elevation 217.42 m at Borehole C28-2) and 7.32 m (Elevation 217.47 m at Borehole C28-3). The silty sand was further underlain by limestone bedrock.

Based on the information presented in the Culvert Summary provided by Morrison Hershfield, and verbal discussion with the project team, it is understood this culvert will be rehabilitated and not replaced. The details of culvert rehabilitation/replacement are still under development at the time of preparing this report. At the time of preparing this draft report, Morrison Hershfield is also considering rehabilitation of the culvert by providing lateral support near the bottom of the culvert walls to prevent kick-in of the culvert walls.

The inverts of the culverts will be placed between Elevations 222.0 m (inlet) and 221.8 m similar to the existing invert elevations.

## **5.2 Culvert Rehabilitation/Replacement Type, Design and Construction**

It is understood that the vertical and horizontal alignment of Highway 12 will remain relatively similar to the present condition. The pavement in this section will not be widened since the existing road platform has a paved width of 3.5 m or more per lane. Based on on-going discussion with Morrison Hershfield, considerations are being given to rehabilitate the existing culvert or replacement with a precast concrete box culvert. The following table summarizes the design and construction considerations, the advantage and disadvantage of each culvert rehabilitation or replacement strategy from a geotechnical perspective.

**Table 1**  
**Culvert Type, Design and Construction Consideration**

<b>Option</b>	<b>Culvert Type and Construction Methodology</b>	<b>Advantage</b>	<b>Disadvantage</b>
A	Construct new slab and curb to provide lateral support	No geotechnical concerns, allow temporary flow through inside existing culvert, little to no interruptions to traffic, minor requirement for soil dewatering	Shorter life expectancy (say 15 to 20 years, under development and to be evaluated), working inside confined space with a large volume of soil to be removed by hand digging

Option	Culvert Type and Construction Methodology	Advantage	Disadvantage
B	Provide lateral load support near the bottom of the culvert walls to prevent kick-in of the walls, with Lateral struts, soil nails or anchors on whalers	General - allow temporary flow through inside existing culvert, little to no interruptions to traffic, no requirement for soil dewatering Struts - Cheapest alternative, easy to install and maintain Soil Nails or Anchors - Easy to install, very little debris to cleanup, moderate costs	General - short life expectancy (say 10 years, under development and to be evaluated), working inside confined space Strut - hydraulic issues to be satisfied, potential ponding inside culvert under low flow conditions, may not meet fisheries requirement Soil Anchors - higher costs than struts
C	New precast box culvert beside the existing culvert	Allow flow through within existing culvert, quicker installation, less interruption to traffic	May require property acquisition due to limited ROW, possible future erosion of the re-routed bank, grouting of existing culvert required
D	Over-built on top of and beside the existing culvert (open footing)	Allow flow through within existing culvert, no property acquisition issue, less disturbance compared to Option D	Longer construction period, maintenance issues if existing culvert not demolished
E	Remove existing and replace with new precast concrete box culvert at same location	May be able to reuse existing frost taper if present, no additional erosion control measures, no property acquisition issue	Require flow through beneath existing road with live traffic, potential difficulty when removing existing footing, requirement to backfill existing footing areas after removal

From a geotechnical foundations perspective, Alternative A and B using rehabilitation strategies will eliminate a lot of the geotechnical concerns in dealing with dewatering, loose or soft soil conditions, constructability, and become a structural rehabilitation solution. Based on the information provided in the preliminary design drawings provided by Morrison Hershfield, the selected rehabilitation strategy consists of concrete repairs and installation of soil anchors.

Soil anchors alternatives are design-build items carried out by specialty sub-contractors using soil parameters given in Section 5.3. There will be constructability issues to deal with when working in a confined space environment and the Provincial Occupational Health and Safety requirements must be strictly adhered to. It has been confirmed by a specialty contractor that Chance Helical Anchors can be installed using portable equipment.

### 5.3 Lateral Earth Pressures and Soil Anchor Capacity

The lateral earth pressures acting on the culvert walls, headwalls (wing walls) and retaining walls (armour stone, gabion etc.) will depend on the type and method of placement of the backfill materials and on the subsequent lateral movement of the structure whether it is restrained or unrestrained. The lateral earth pressures to be used in the design should be computed in accordance with Section 6.9 of the CAN/CSA-S6-06.

Granular backfill should be constructed behind the culvert walls, headwalls (wing walls) and retaining walls as per OPSD-3121.150, with particular attention to the frost taper requirement. The granular backfill should conform to OPSS 1010 for either Granular "A" or Granular "B"

Type III. To maintain free draining characteristics in granular fill materials, the maximum percentage passing the No. 200 sieve (75 µm) should be limited to 5%.

The backfill should be constructed as per OPSS 902 and 501, and SSP902S01. A perforated subdrain should be installed behind the walls with a positive outlet or wall drains as per OPSD-3190.100 to drain the granular fill above the stream water level. Alternatively, the culvert walls could be designed to resist hydrostatic pressure.

The lateral earth pressure,  $P_h$ , acting on the culvert walls, headwalls (wing walls) or retaining walls may be computed using the equivalent fluid pressures presented in Clause 6.9.2.3 of the CAN/CSA-S6-06, or employing the following equation based on unfactored earth pressure distributions:

$$P_h = K (\gamma h + q)$$

Where:

$K$  = earth pressure coefficient, use value from table below

$\gamma$  = unit weight of soil, = 21.2 kN/m<sup>3</sup> for Granular "B"  
= 22.8 kN/m<sup>3</sup> for Granular "A"

$h$  = depth below top of wall, m

$q$  = live load surcharge pressure, equivalent fill height of 0.8 m as per Clause 6.9.5 of CHBDC and CAN/CSA-S6-06

Wall Type	Earth Pressure Coefficient (K)	
	Granular "A" $\phi = 35^\circ$	Granular "B" $\phi = 30 \text{ to } 35^\circ$
Restrained Wall ( $K_o$ )	0.43	0.50 to 0.43
Unrestrained Wall ( $K_a$ )	0.27	0.33 to 0.27

The submerged unit weight of the backfill should be used for any submerged portion of the granular backfill when calculating the lateral earth pressure.

The above parameters are based on a horizontal back slope (not exceeding 5 degrees) behind the headwalls. A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the headwalls and retaining walls in accordance with Clause 6.9.3 of the CAN/CSA-S6-06.

The sliding resistance of the cast-in-place footings should be checked. The unfactored horizontal resistance (Clause 6.7.5, CAN/CSA-S6-06) against sliding between concrete and undisturbed, silty clay and sandy silt founding soil can be calculated using a coefficient of friction (friction factor) of 0.40 may be used for concrete on clay soils as per Table 24.4 CFEM 4<sup>th</sup> Edition, 2006.

For a precast concrete culvert, the friction factor and adhesion should be reduced by a factor of 0.67.

Vibratory equipment for use behind the culvert walls, headwalls (wing walls) and retaining walls should be restricted in size as per current MTO practices, and should conform to OPSS 501 and SSP105S10.

Soil anchors can be installed to provide allowable anchor loads of between 30 to 35 kN/m, as determined by Morrison Hershfield. However, it will be very difficult to construct conventional soil anchors or nails due to excessive cave-ins within the saturated sand and silt zones, and installation within a confined area. It has been confirmed with a specialty contractor that Chance Helical anchors could be used due to its modular (sectional) construction, and can provide a working load (SLS) of 60 to 80 kN per anchor.

#### **5.4 Temporary Flow Through**

It is noted that a “Permit To Take Water” (PTTW, Regulation 387/04) will be required from the MOE (Ministry of Environment) when the total quantity of water to be handled exceeds 50,000 litres/day while employing temporary pumping of water, flow passages through culverts, stream diversion or dam and pump method as groundwater control measures (unwatering). It may take up to 90 days for MOE to review an application and issue a permit.



## 6.0 STATEMENT OF LIMITATION

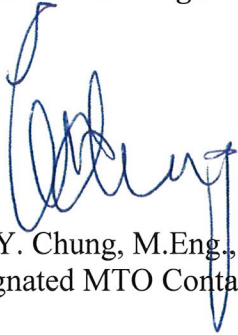
We recommend that once the details of the proposed structure are finalized, our recommendations should be reviewed for their specific applicability.

The Limitations of Report, as Quoted in Appendix "C", is an integral part of this report.

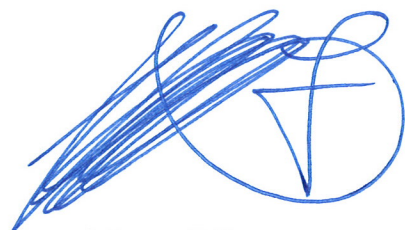
We trust that we have completed the assignment within the Terms of Reference for this project. If there are any questions concerning this report, please do not hesitate to contact our office.

Yours truly,

**Infrastructure Engineering Group Inc.**



Eric Y. Chung, M.Eng., P.Eng.  
Designated MTO Contact



Joseph Law, P.Eng.  
Project Manager



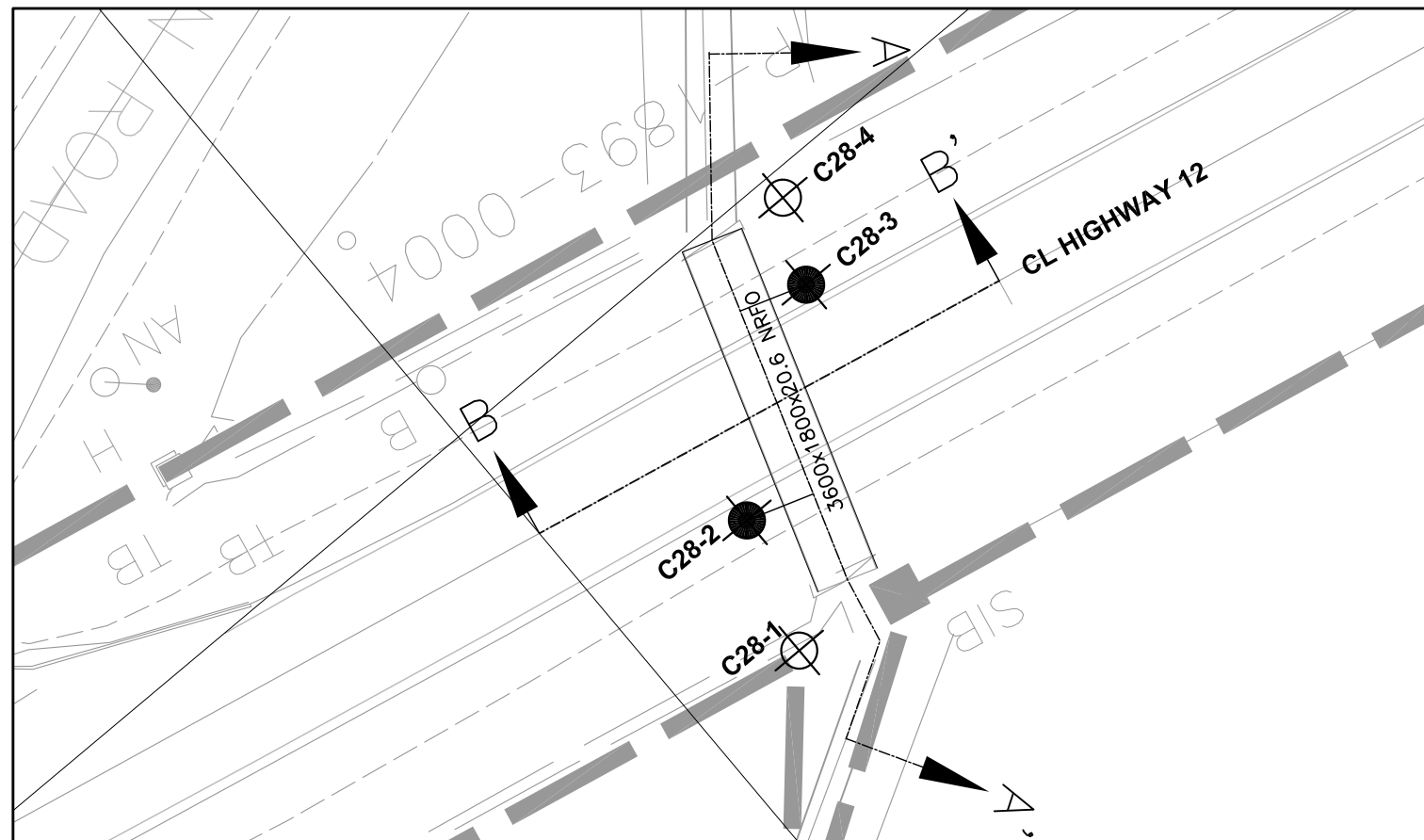
Tom O'Dwyer, P. Eng.  
Quality Review Engineer



Ministry of Transportation/Morrison Hershfield Limited  
W.P. 365-98-00  
Rehabilitation of Highway 12 from Rama Road to Gamebridge  
Agreement # 2004-E-0070

08-1-IEG6-30-676/C  
Final Report  
Drawing 1  
November 6, 2009

Drawings 1 & 2  
Borehole Locations  
And  
Soil Strata



## BOREHOLE LOCATION PLAN

### METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No xxxx-xxxx  
WP No GWP 365-98-00

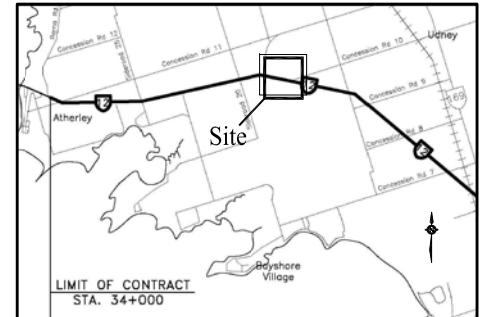
Structural Culvert 30-676/C  
Highway 12  
BOREHOLE LOCATION PLAN & PROFILE



SHEET  
1

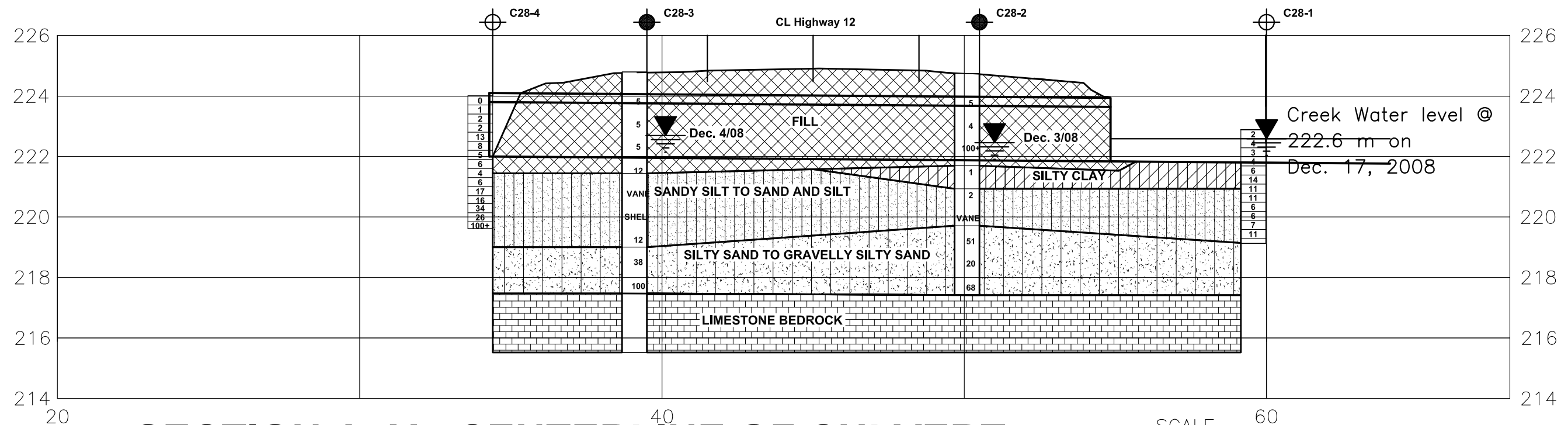
**I.E. Group** Infrastructure Engineering Group Inc.  
Pavement & Construction Materials Consulting Engineers  
GTA • Kitchener • London • Windsor

KEYPLAN NTS

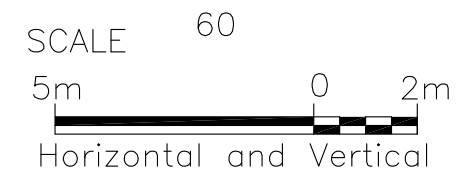


### LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- Blows/0.3m (Std Pen Test, 475 J/blow)
- Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation
- Standpipe



## SECTION A-A' - CENTERLINE OF CULVERT



### NOTES

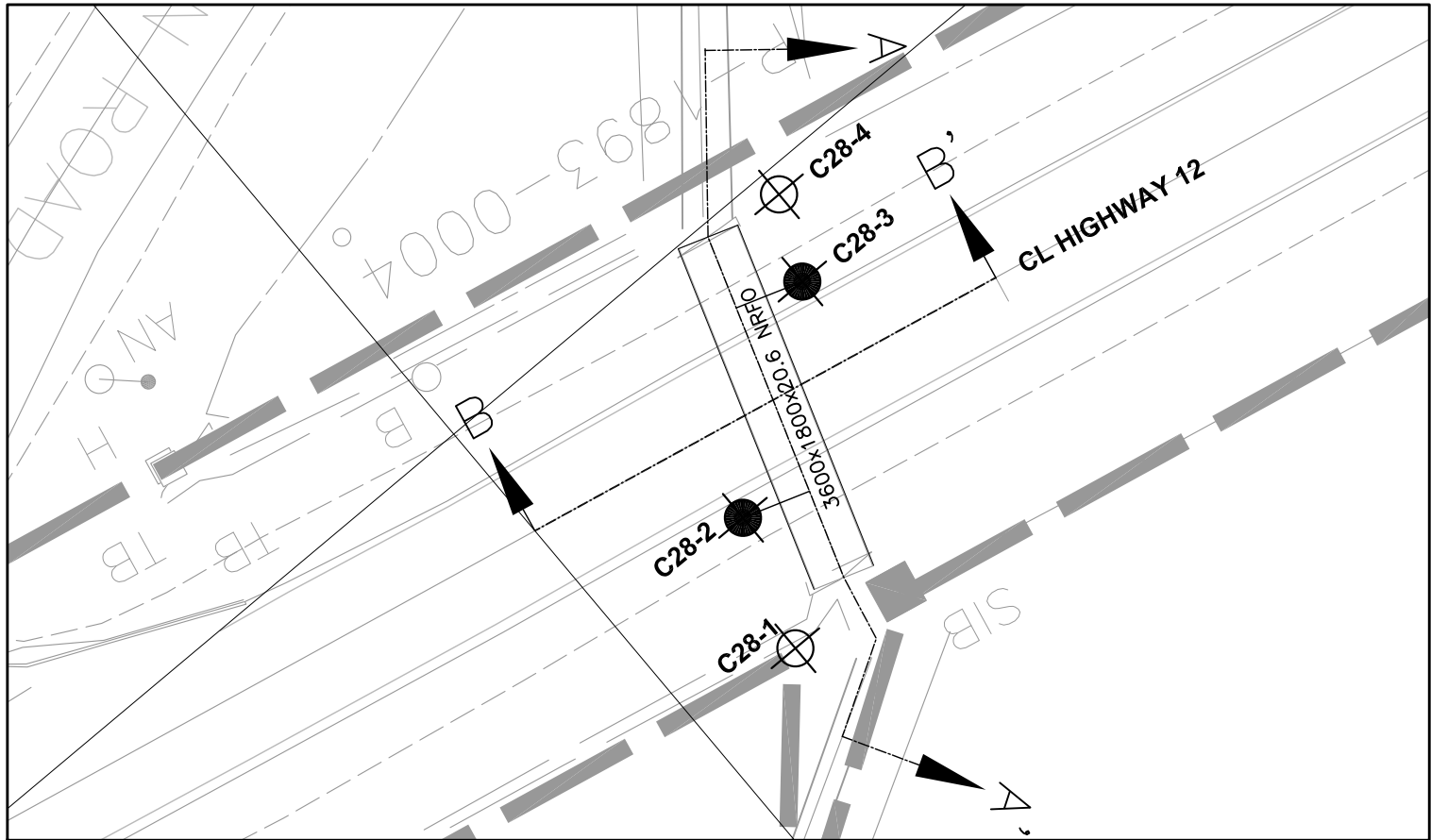
- THE COMPLETE FOUNDATION INVESTIGATION AND DESIGN REPORT FOR THIS PROJECT AND OTHER RELATED DOCUMENTS MAY BE EXAMINED AT THE ENGINEERING MATERIALS OFFICE, DOWNSVIEW. INFORMATION CONTAINED IN THIS REPORT AND RELATED DOCUMENTS ARE SPECIFICALLY EXCLUDED IN ACCORDANCE WITH THE CONDITIONS OF SECTION GC2.01 of OPS GEN. COND.
- THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES AND BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.
- SUBGRADE ELEVATION OF THE EXISTING FOOTING NOT KNOWN AND IS ESTIMATED TO BE AT 1.6m BELOW THE CREEK BED.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

BOREHOLE NO.	ELEVATION	UTM CO-ORDINATES	
		NORTH	EAST
C28-1	222.44	4940578	323607
C28-2	224.74	4940586	323609
C28-3	224.79	4940594	323620
C28-4	223.99	4940599	323623

REVISIONS			
	05/09/09	J.L.	Final Report
	18/05/09	J.L.	Draft
	DATE	BY	DISCRIPTION

Geocres : 31D-488					
HWY No.		HWY 12		DIST	CENTRAL
SUBM'D	J.L.	CHECKED	E.C.	DATE 25/03/09	SITE 30-676/C
DRAWN	J.L.	CHECKED	J.L.	APPROVED E.C.	DWG 1

MINISTRY OF TRANSPORTATION, ONTARIO



BOREHOLE LOCATION PLAN

METRIC

DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No xxxx-xxxx  
WP No GWP 365-98-00

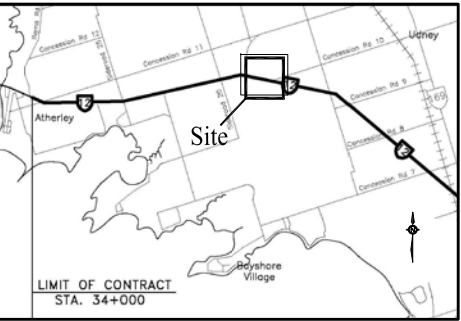


Structural Culvert 30-676/C  
Highway 12  
BOREHOLE LOCATION PLAN & PROFILE

SHEET  
2

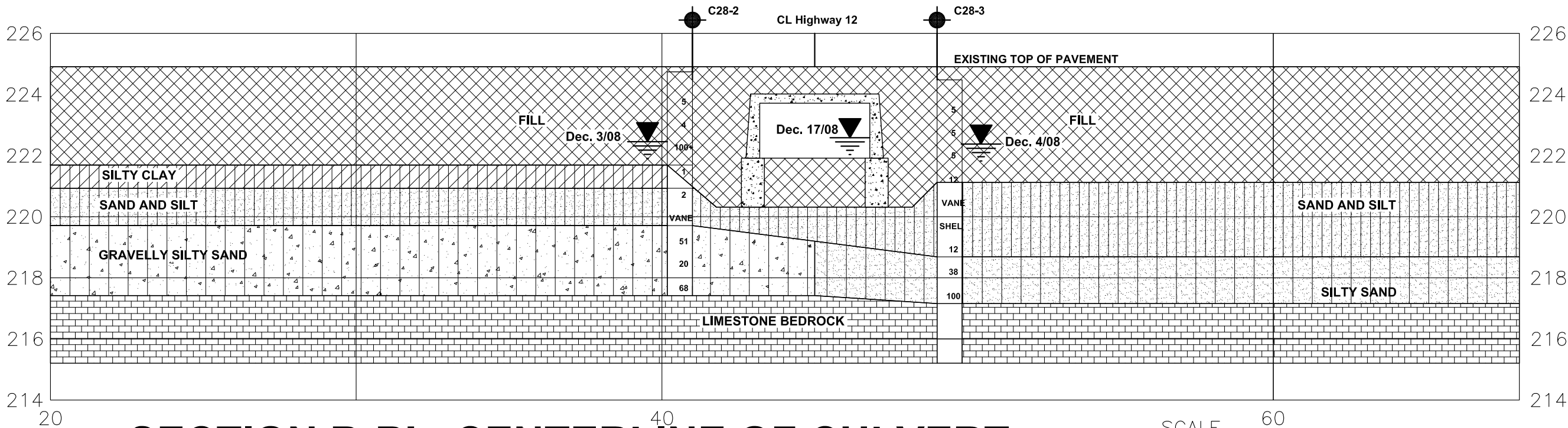
I.E. Group  
Infrastructure Engineering Group Inc.  
Pavement & Construction Materials Consulting Engineers  
GTA • Kitchener • London • Windsor

KEYPLAN NTS



LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- Blows/0.3m (Std Pen Test, 475 J/blow)
- Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation
- Standpipe



SECTION B-B' - CENTERLINE OF CULVERT

SCALE



Horizontal and Vertical

- NOTES
- THE COMPLETE FOUNDATION INVESTIGATION AND DESIGN REPORT FOR THIS PROJECT AND OTHER RELATED DOCUMENTS MAY BE EXAMINED AT THE ENGINEERING MATERIALS OFFICE, DOWNSVIEW. INFORMATION CONTAINED IN THIS REPORT AND RELATED DOCUMENTS ARE SPECIFICALLY EXCLUDED IN ACCORDANCE WITH THE CONDITIONS OF SECTION GC2.01 of OPS GEN. COND.
  - THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES AND BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.
  - SUBGRADE ELEVATION OF THE EXISTING FOOTING NOT KNOWN AND IS ESTIMATED TO BE AT 1.6m BELOW THE CREEK BED.
  - THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

BOREHOLE NO.	ELEVATION	UTM CO-ORDINATES	
		NORTH	EAST
C28-1	222.44	4940578	323607
C28-2	224.74	4940586	323609
C28-3	224.79	4940594	323620
C28-4	223.99	4940599	323623

REVISIONS					
DATE	BY	DATE	BY	DISCRPTION	
05/09/09	J.L.			Final Report	
Geocres : 31D-488					
HWY No.	HWY 12			DIST	CENTRAL
SUBM'D	J.L.	CHECKED	E.C.	DATE	05/09/09
DRAWN	J.L.	CHECKED	J.L.	APPROVED	E.C.
				DWG	2

## Appendix A

### Explanation of Terms Used in Report

#### Record of Borehole Sheet

#### Boreholes C28-1 TO C28-4

## EXPLANATION OF TERMS USED IN REPORT

**N VALUE:** THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{N}$

**DYNAMIC CONE PENETRATION TEST:** CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 1" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T.W. ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T.W. ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_r$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_c$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_c}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1. %	VOID RATIO	$e_{min}$	1. %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1. %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1. %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kn/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kn/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$i_p$	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{i_p}$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kn/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{i_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1. %	VOID RATIO IN LOOSEST STATE	j	kn/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kn/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

## METRIC

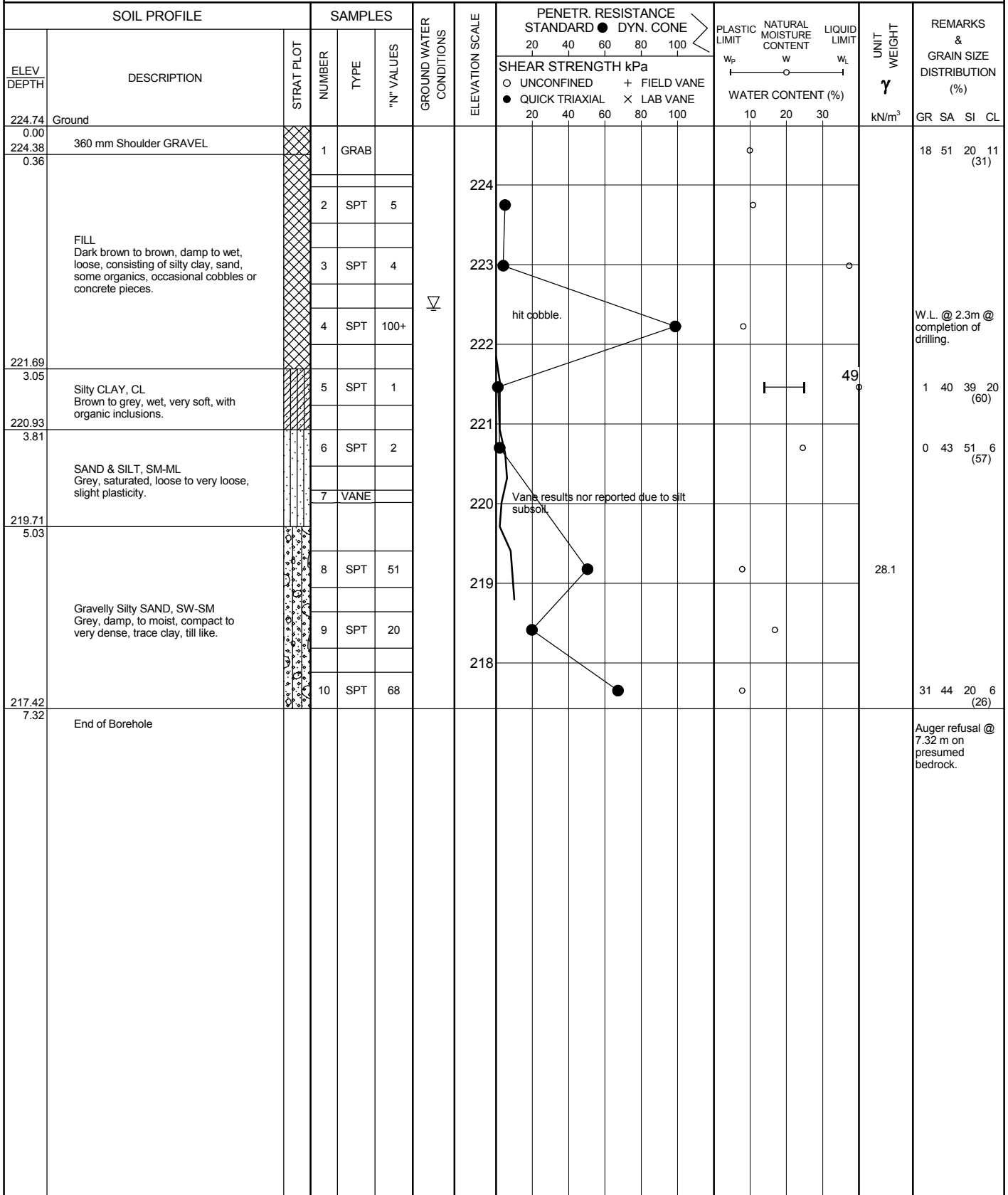
[illegible]

# RECORD OF BOREHOLE No C28-2

1 OF 1

METRIC

W.P. WP 365-98-00 LOCATION Northring - 4940586, Easting - 323609 ORIGINATED BY RB  
 DIST Central Region HWY Highway 12 BOREHOLE TYPE S/S Augering 110 mm Dia. COMPILED BY JL  
 DATUM Geodetic DATE 12.03.08 - 12.03.08 CHECKED BY EC



JOE MTO 08-1-IEG6 CULVERTS.GPJ ONTARIO MOT.GDT 05/01/09

+ 3, × 3: Numbers refer to Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS



# RECORD OF BOREHOLE No C28-3

1 OF 1

METRIC

W.P. WP 365-98-00 LOCATION Northing - 4940594, Easting - 323620 ORIGINATED BY RB  
 DIST Central Region HWY Highway 12 BOREHOLE TYPE H/S Augering 110 mm Dia. COMPILED BY JL  
 DATUM Geodetic DATE 12.04.08 - 12.18.08 CHECKED BY EC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE STANDARD ● DYN. CONE		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
224.79	Ground							20 40 60 80 100	10 20 30						
0.00															
224.43	360 mm Shoulder GRAVEL		1	GRAB											
0.36															
			2	SPT	5		224							31 33 26 9 (36)	
			3	SPT	5		223						22.1	W.L. @ 2.1m @ completion of drilling.	
			4	SPT	5		222						42		
221.44			5	SPT	12		221								
3.35															
			6	VANE											
			7	SH			220							0 30 67 3 (70)	
			8	SPT	12		219							14 42 31 14 (45)	
219.00															
5.79			9	SPT	38		218								
			10	SPT	100+		217								
217.47															
7.32			11	CORE	NQWL		216								
			12	CORE	NQWL										

JOE MTO 08-I-IEG6 CULVERTS.GPJ ONTARIO MOT.GDT 05/01/09

+ 3, X 3: Numbers refer to  
Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

# RECORD OF BOREHOLE No C28-4

1 OF 1

**METRIC**

W.P. WP 365-98-00 LOCATION Northing - 4940599, Easting - 323623 ORIGINATED BY RB  
 DIST Central Region HWY Highway 12 BOREHOLE TYPE Dynamic Cone COMPILED BY JL  
 DATUM Geodetic DATE 02.18.09 - 02.18.09 CHECKED BY EC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	PENETR. RESISTANCE		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	STANDARD ● DYN. CONE					
223.99 0.00	Ground						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100 20 40 60 80 100	10 20 30				31.75 Kg (70lbs.) hammer used for driving dynamic cone. Nc values corrected for standard 63 kg (140 lbs.) hammer.
219.60 4.39	End of borehole.												Dynamic cone refusal @ 4.39 m on presumed bedrock.

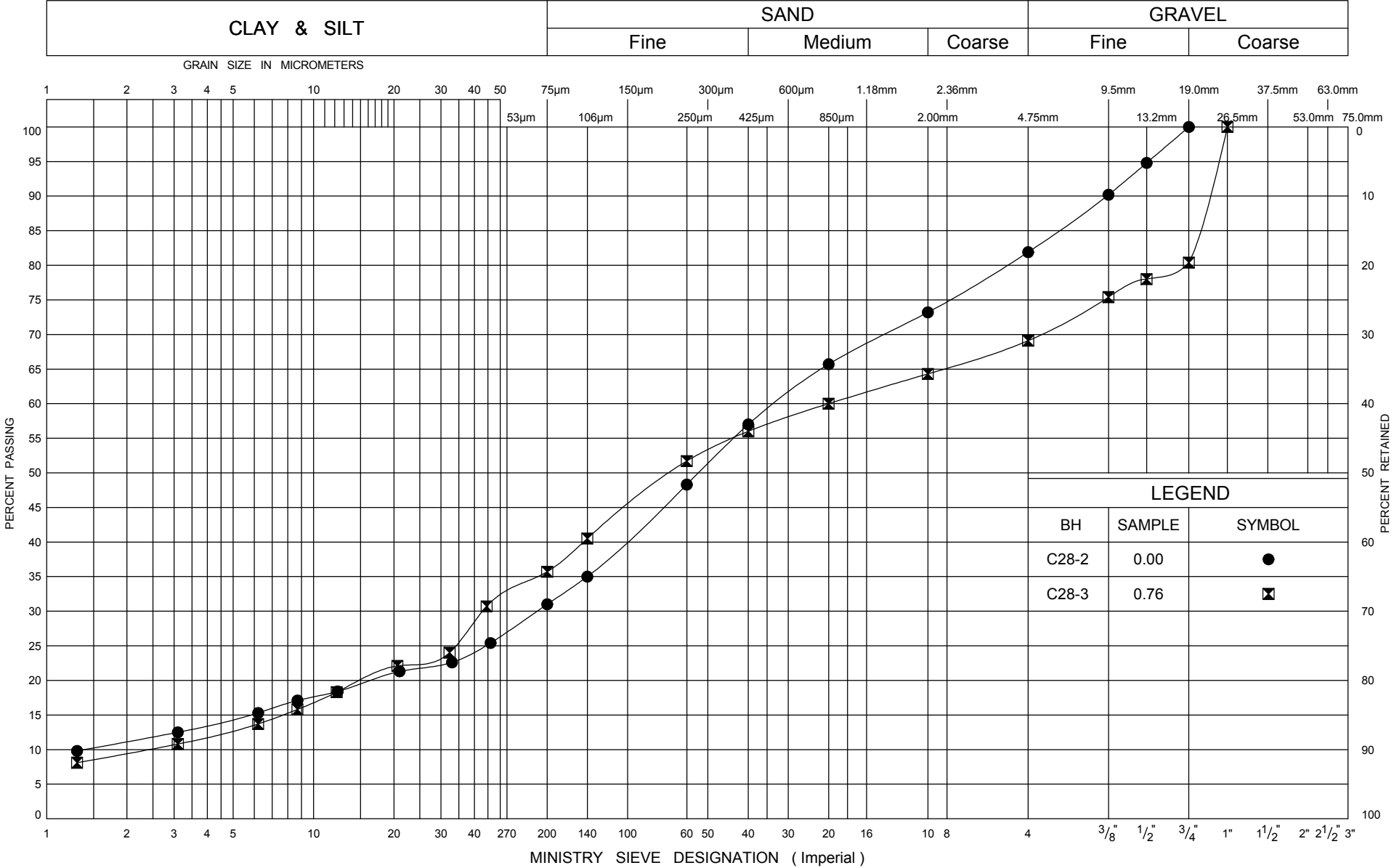
JOE MTO 08-1-IEG6 CULVERTS.GPJ ONTARIO MOT.GDT 05/01/09

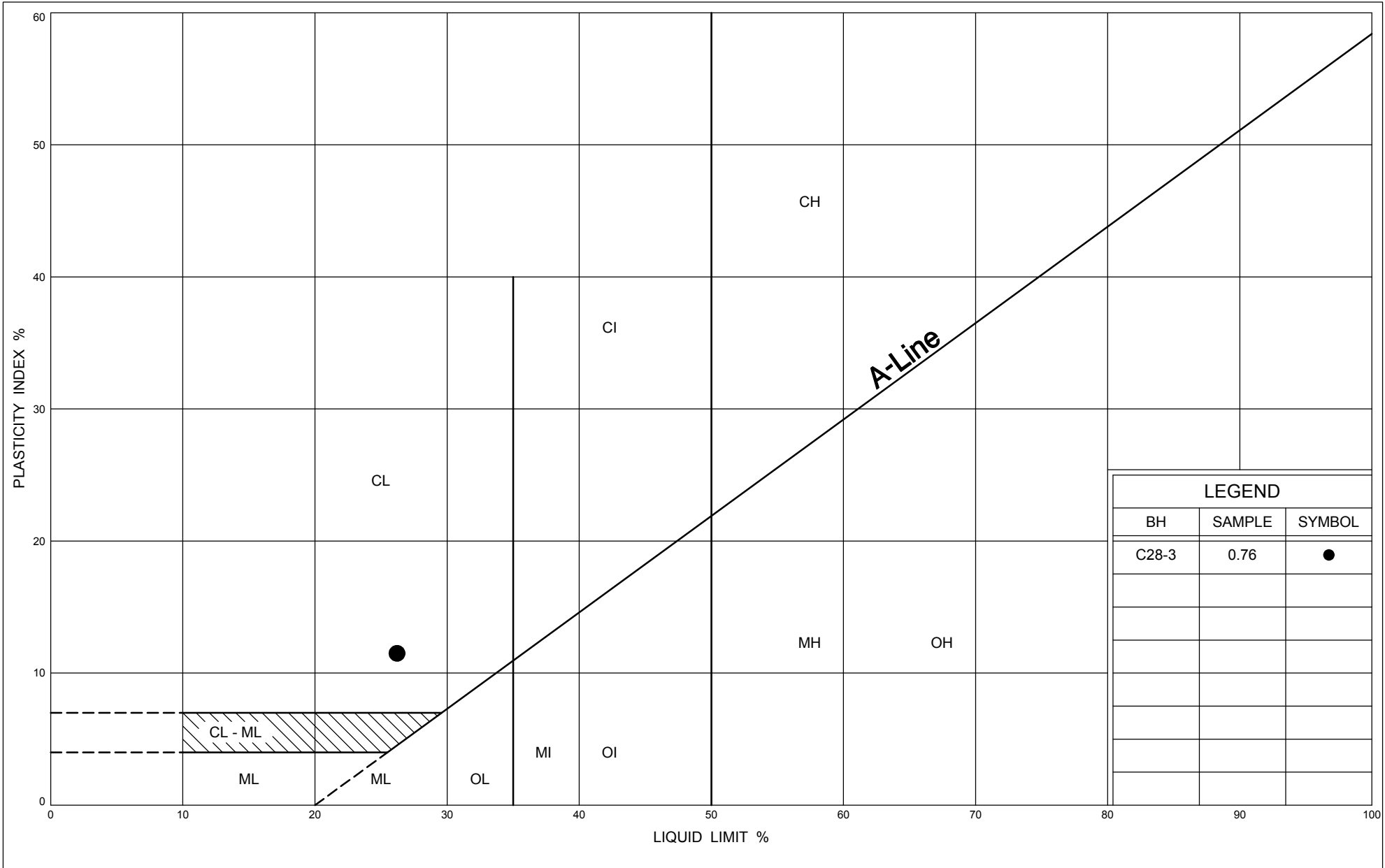
## Appendix B

### Laboratory Test Results

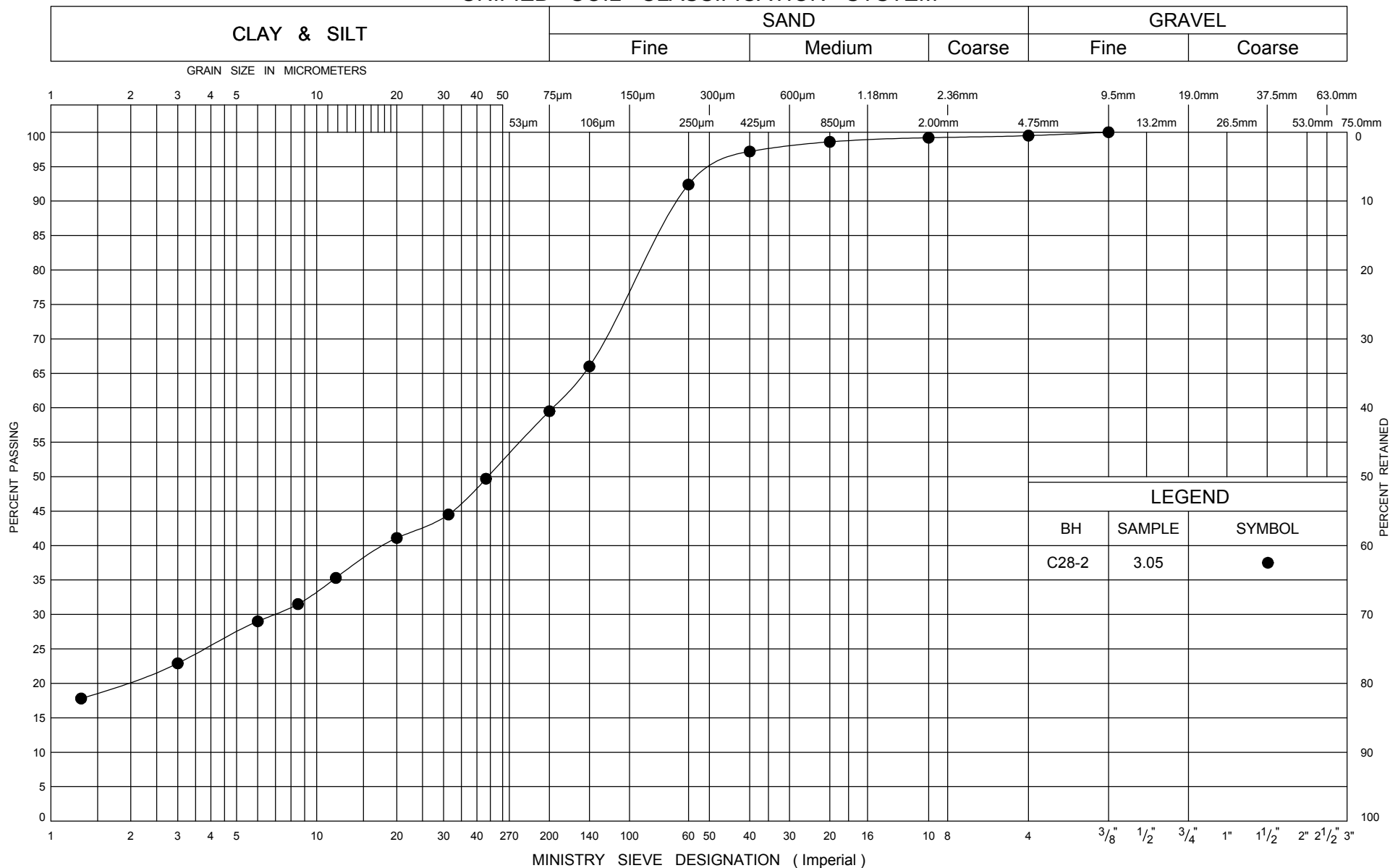
Grain Size Distribution	Figures 1, 3, 5 and 7
Plasticity Chart	Figures 2, 4 and 6
Rock Core Report	Figure 8

UNIFIED SOIL CLASSIFICATION SYSTEM





## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

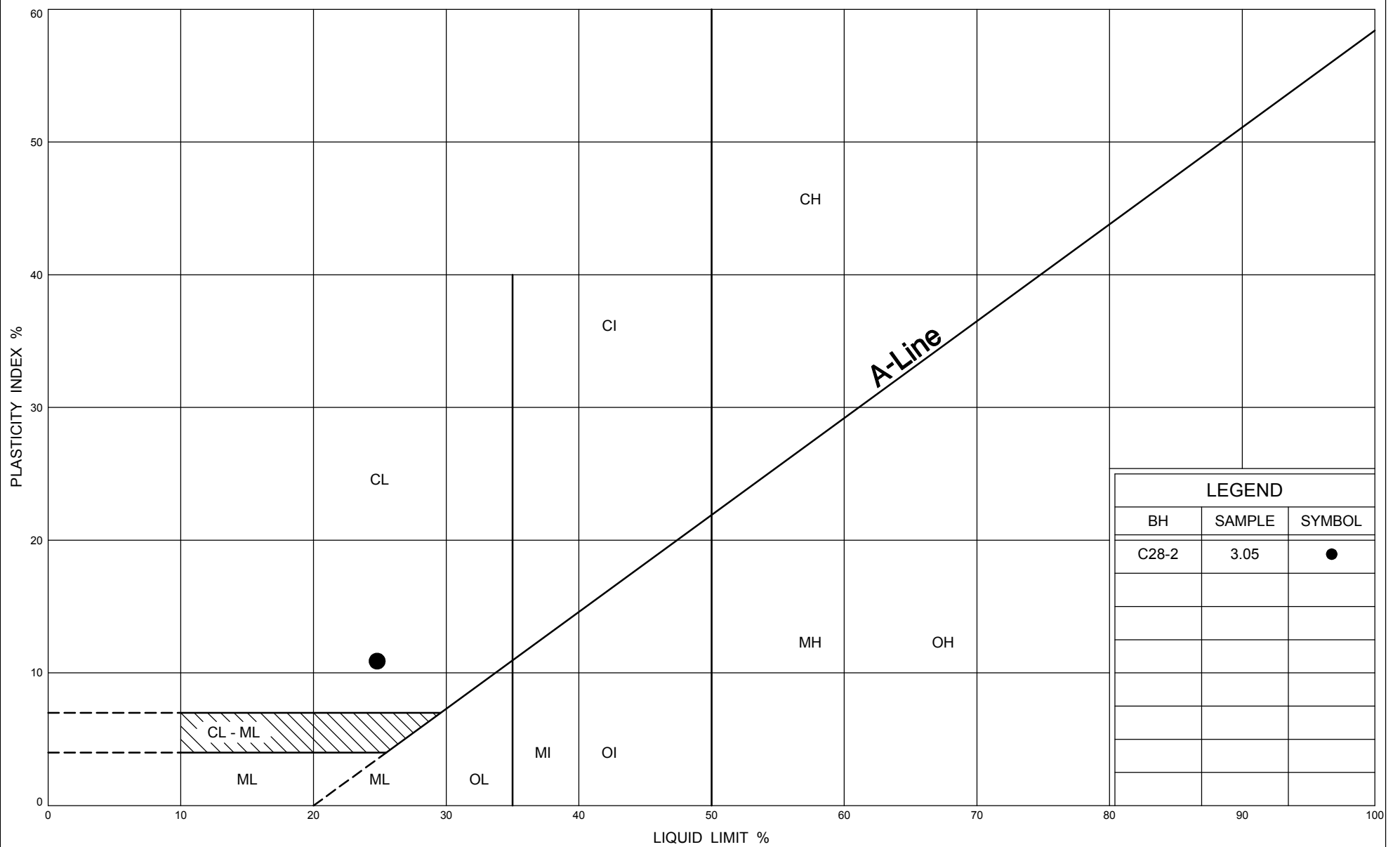
## GRAIN SIZE DISTRIBUTION

### SILTY CLAY, CL

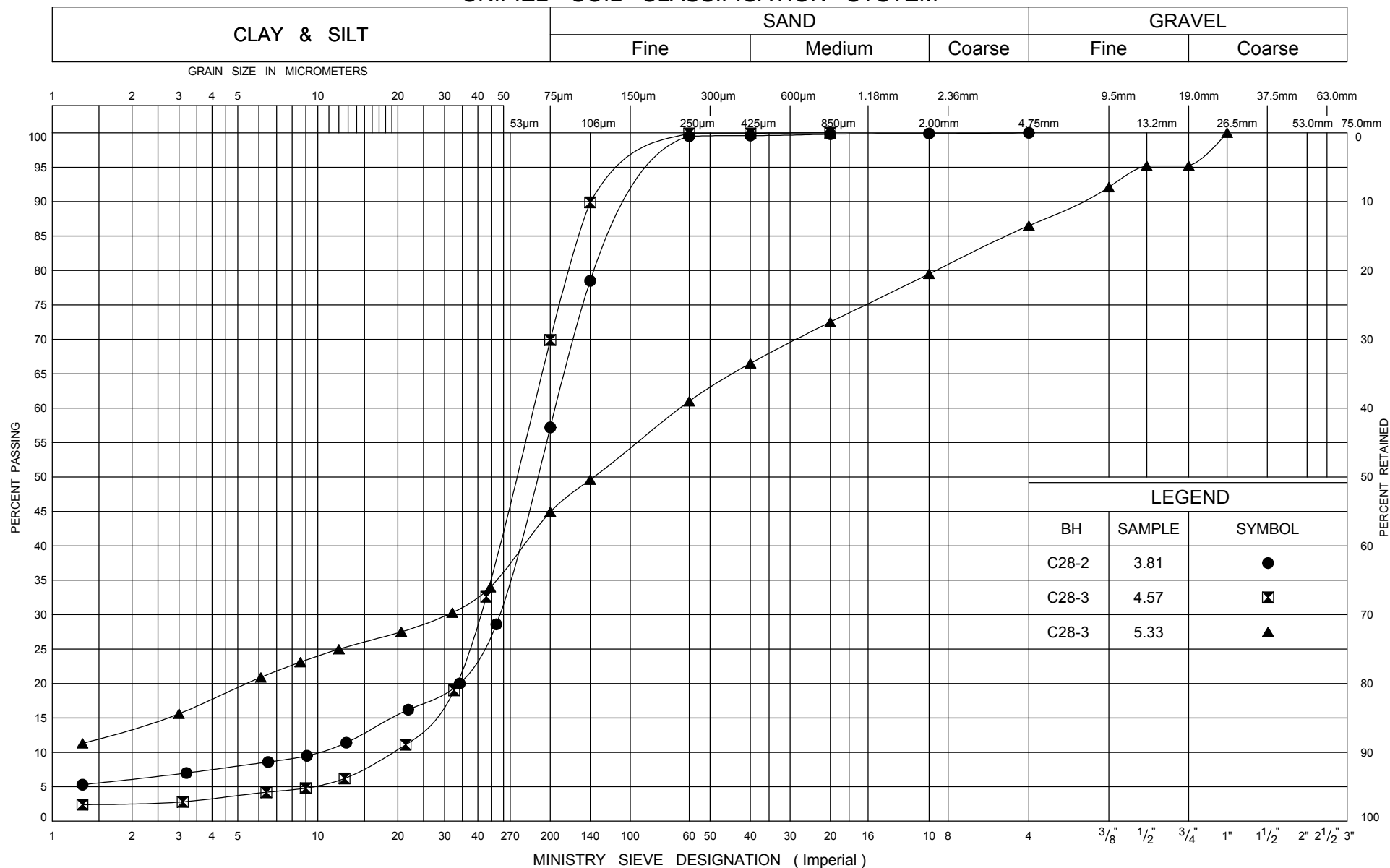
FIG No 3

GWP 365-98-00

Highway 12, Rama Road to Gamebridge



## UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

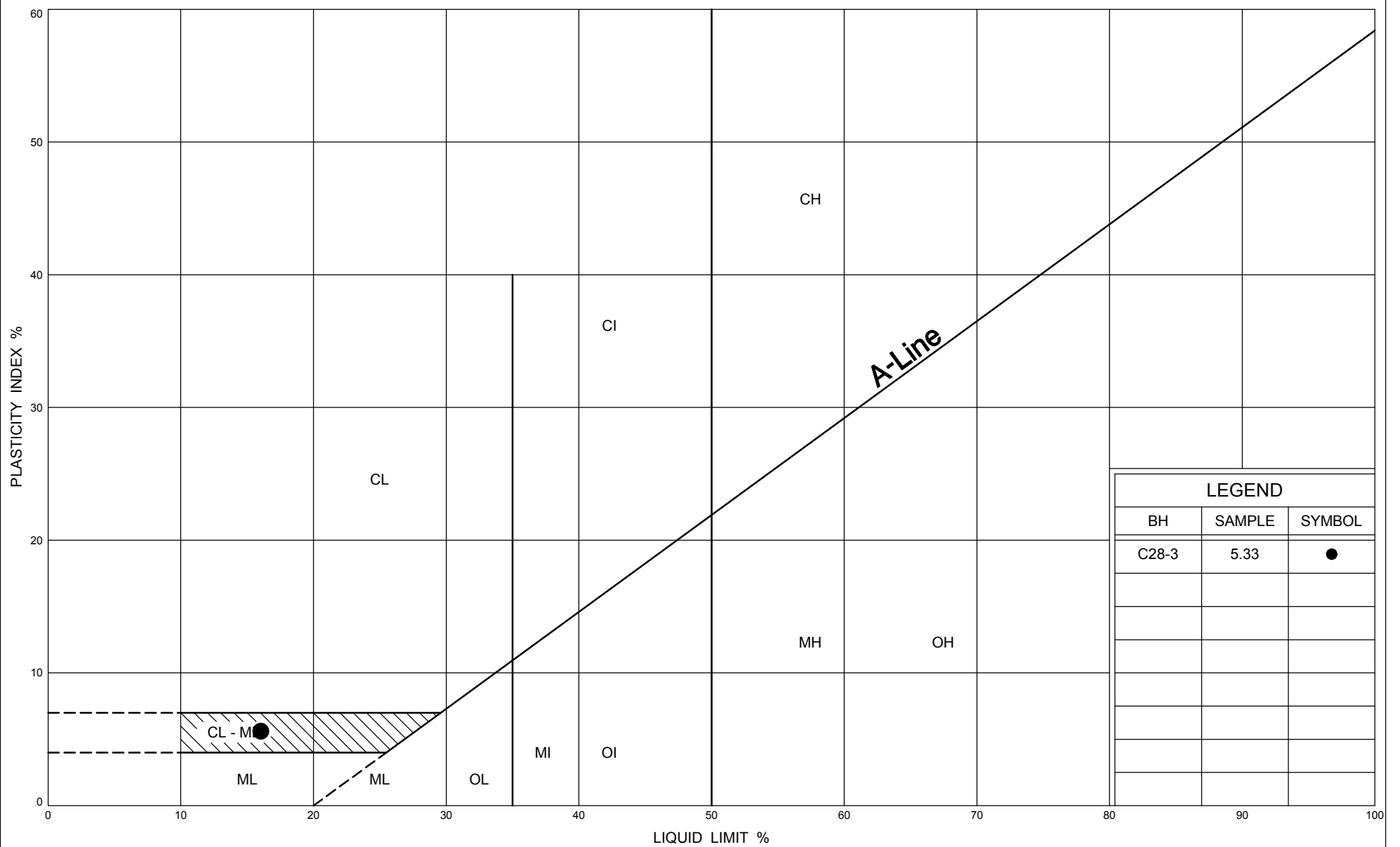
GRAIN SIZE DISTRIBUTION  
SILTY SAND, SAND AND SILT, SANDY SILT, SM TO ML

FIG No 5

GWP 365-98-00

Highway 12, Rama Road to Gamebridge





Ministry of  
Transportation

# PLASTICITY CHART

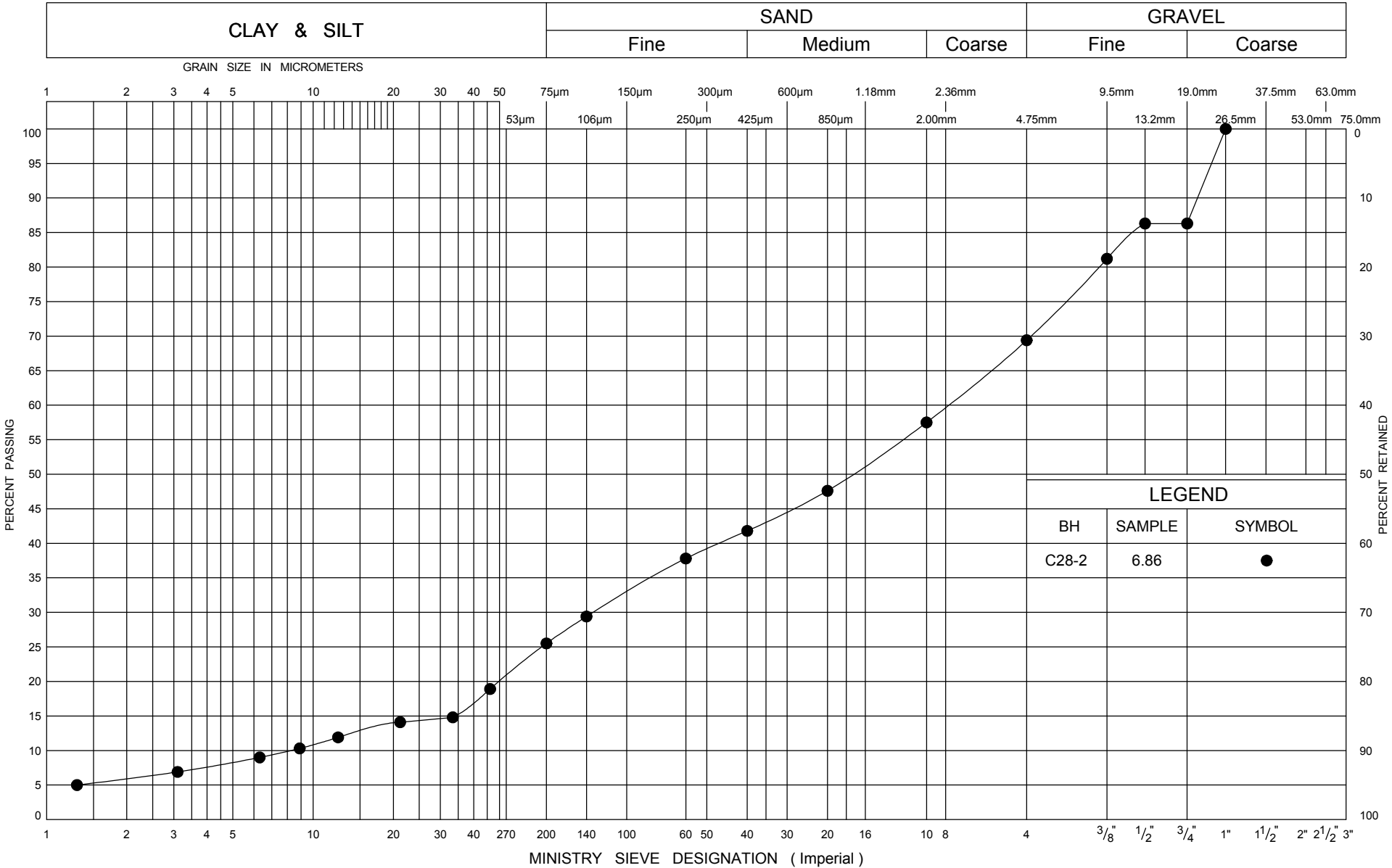
## SILTY SAND, SAND AND SILT, SANDY SILT, SM TO ML

FIG No 6

GWP 365-98-00

Highway 12, Rama Road to Gamebridge

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION  
GRAVELLY SILTY SAND, SW-SM

FIG No 7  
GWP 365-98-00  
Highway 12, Rama Road to Gamebridge




## Rock Core Test Report

**Project No.:** LAGM00289085C

**Project Name:** Hwy 12 – 03/20019

<b>Core No.</b>	BH C28-3
<b>Location</b>	27'10"-28'6"
<b>Date Cored</b>	
<b>Date Tested</b>	April 7, 2009
<b>Height - (mm)</b>	126.5
<b>Average Diameter - (mm)</b>	46.2
<b>Corrected Compressive Strength - (MPa)</b>	128.6

  
Testing Laboratory Representative Signature  
Ammanuel Yousif

  
Date

Ministry of Transportation/Morrison Hershfield Limited  
W.P. 365-98-00  
Rehabilitation of Highway 12 from Rama Road to Gamebridge  
Agreement # 2004-E-0070

08-1-IEG6-30-676/C  
Final Report  
Appendix C  
November 6, 2009

## Appendix C

### Limitations of Report

## **APPENDIX C**

### **LIMITATIONS OF REPORT**

The conclusions and recommendations given in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Soils Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

The benchmark and elevations mentioned in this report were obtained strictly for use in the geotechnical design of the project and by this office only, and should not be used by any other parties for any other purposes.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Infrastructure Engineering Group Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

This report does not reflect the environmental issues or concerns unless otherwise stated in the report.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, IEG recommends that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Ministry of Transportation/Morrison Hershfield Limited  
W.P. 365-98-00  
Rehabilitation of Highway 12 from Rama Road to Gamebridge  
Agreement # 2004-E-0070

08-1-IEG6-30-676/C  
Final Report  
Appendix D  
November 6, 2009

## Appendix D

### Site Photographs



C28 - Station 26+514 - Looking North



C28 - Station 26+514 - Downstream



C28 - Station 26+514 - Upstream

SITE PHOTOGRAPHS

SITE NO.:30-676/C



C28 Photo 1 Roadway Above Culvert / North Approach Roadway



C28 Photo 3 Culvert Barrel from West



C28 Photo 2 West Elevation



C28 Photo 4 Embankment Erosion at Northwest Corner