Peto MacCallum Ltd.
CONSULTING ENGINEERS

MEMORANDUM
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN
HEBERT CREEK BRIDGE REPLACEMENT, SITE 43-017
HIGHWAY 539, TOWNSHIP OF BADGEROW
GWP NO. 5116-07-00

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Dear Mr. D'Andrea

Memorandum
Preliminary Foundation Investigation and Design
Hebert Creek Bridge Replacement, Site 43-017
Highway 539, Township of Badgerow
GWP No. 5116-07-00

This document summarises the results of a preliminary foundation investigation carried out for the proposed replacement of the existing bridge over Hebert Creek located on Highway 539 in the Township of Badgerow, Ontario. It also provides preliminary foundation engineering comments and recommendations regarding the design and construction of foundations, abutments and approach fill embankments for the Herbert Creek Bridge replacement alternatives.

Highway 539 passes over Hebert Creek at approximate Station 10+153, Highway 539 chainage (ref. Drawing P1 'Hebert Creek Bridge. Preliminary Site Plan' prepared by Stantec Consulting Ltd. in November 2009). The existing bridge constructed in 1951 is a three span structure supported on timber piles with a post-tensioned timber deck 17.1 m long and 8.7 m wide.

The existing road grade on Highway 539 at the bridge location is near elevation 227 as determined on the basis of survey data shown on the drawing referred to above. The existing approach embankments are about 3 m above the water level in the creek (approximate elevation 224).

Site Description and Geology

The structure to be replaced carries two lanes of Highway 539 traffic over Hebert Creek approximately 4 km west of Highway 64. At the location of the bridge, Highway 539 runs in the northwest-southeast direction. A culvert is present some 100 m southeast of the bridge.

Hebert Creek is a tributary of the Sturgeon River. The project site is on the north margin of the Sturgeon River floodplain. The land surface adjacent to the creek is vegetated with grass, shrubs and trees. Site photographs are shown in Appendix A.

The site is located in the Algonquin Highlands physiographic region. The topography is irregular in detail, with about 3 m high earth cuts on the southwest side of the highway and flooded drainage / soft areas to the northeast. Soil cover can vary in thickness significantly over a short distance and is represented by varying deposits of sand, silt, lacustrine clay and glacial till. Frequent bedrock outcrops are common.

The study area is situated within a structural subdivision of the Canadian Precambrian Shield identified as the Grenville Province and forming the southern margin of the shield between Georgian Bay and Labrador. Bedrock predominantly comprises granites and gneisses. The bedrock in the vicinity of the site is at various depths ranging from surface to over 30 m.

Investigation Procedures

The field work for this study was carried out during the period of October 6 to 14, 2009 and comprised three boreholes drilled to depths of 25.3 to 33.0 m at the locations shown on Drawing 1, attached.

The locations of and ground surface elevations at the boreholes were established in the field by PML. Benchmark HCP 101 (elevation 226.916) provided by Stantec and located southeast of the bridge at Sta. 10+260 was used to establish the ground surface elevations.

The boreholes were advanced using continuous flight hollow stem augers, powered by a track-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. NQ diamond rock coring equipment supplemented by NW wash boring techniques was used to extend one borehole through boulders.

Representative soil samples were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open boreholes. Upon completion of drilling, the boreholes were backfilled with bentonite/cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. Recovered soil samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determination. Atterberg limits testing (5) and grain size distribution analyses (14) were performed on selected soil samples. A consolidation test and an unconfined compressive strength test were conducted on a relatively undisturbed Shelby tube cohesive soil sample. The laboratory test results are presented in Figures PC-1 to PC-3, GS-1 to GS-6, C-1 and on the corresponding logs.

Summarised Subsurface Conditions

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations, standard penetration test data, in situ vane and penetrometer undrained shear strength values as well as groundwater observations. The results of laboratory Atterberg limits testing, grain size distribution analyses, an unconfined compressive strength test and natural moisture content determination are also shown on the Record of Borehole sheets.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing 1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the boreholes drilled at the site comprised surficial fill or topsoil overlying cohesionless sandy/silty soils interlayered with a cohesive deposit of silty clay / clayey silt. No bedrock was reached, with the boreholes terminated due to refusal on probable boulders at depths of 25.3 to 33.0 m (elevation 193.8 to 200.7). During drilling, artesian water was encountered at a depth of 19.8 m (elevation 206.1) in borehole E1, with a head of 1.5 m.

The strata encountered are summarised below.

<u>Fill</u>

Pavement fill was present surficially in borehole S1 put down on the existing highway. Composed of sand and gravel (with 75 mm of asphalt), the pavement fill was 175 mm in thickness and compact in relative density. Directly beneath the pavement fill was sand fill. This unit was 1.9 m thick and compact to loose (SPT-‘N’ values decreasing from 17 to 5). The results of grain size distribution analysis performed on a sample of the sand fill are presented in Figure GS-1. The embankment fill had a moisture content of 4%.

Sand and gravel fill with organic inclusions was present surficially in borehole W1. The 800 mm thick sand and gravel fill was very loose in relative density (SPT-‘N’ value of 2) and about 18% in moisture content.

<u>Topsoil</u>

Surficial topsoil was present in borehole E1. The silty topsoil 200 mm thick and penetrated at elevation 225.7.

<u>Sandy/Silty Soils</u>

Underlying the fill or topsoil at depths of 0.2 to 2.2 m (elevation 224.6 to 225.7) were cohesionless soils of various granulometric composition (gravelly sand, sand, silty sand, sandy silt, silt). Containing organics in the upper zone and interlayered with a deep deposit of clayey soils, this stratum was very loose to compact (SPT-'N' values of 0 to 18) becoming dense to very dense at depths of 25.0 to 27.9 m (elevation 198.0 to 201.0). The moisture content of the sandy/silty soils ranged from 9 to 36%, typically from 13 to 30%. The boreholes were terminated in the stratum at depths of 25.3 to 33.0 m (elevation 193.8 to 200.7) due to refusal on probable boulders. It is noteworthy that rock coring was necessary to advance through 6.8 m of cobbles and boulders revealed at 26.2 m depth (elevation 200.6) in borehole S1. A layer with N >100 blows for more than 3 m was encountered above the termination depth of borehole S1.

The results of Atterberg limits testing and grain size distribution analyses conducted on 9 samples of the sandy/silty soils are presented in respective Figures PC-1 and GS-2 to GS-4.

Silty Clay / Clayey Silt

A deposit of silty clay / clayey silt was revealed within the sandy/silty soils at depths of 19.5 to 20.1 m (elevation 205.8 to 206.7) in all the boreholes. This deposit was 5.5 to 6.4 m in thickness and firm to stiff in consistency. The results of in situ vane testing carried out in the clayey soils yielded undisturbed shear strength values in a range of 60 to 84 kPa (soil sensitivity of 3 to 5). An unconfined compression test on the clayey silt sample from borehole E1 gave a shear strength value of 26 kPa (strain at failure of 11%). The deposit was penetrated at depths of 25.0 to 26.5 m (elevation 199.4 to 201.0).

The results of Atterberg limits testing and grain size distribution analyses conducted on 4 samples of the clayey soils are presented in respective Figures PC-2, PC-3 and GS-5, GS-6. The liquid and plastic limits of the silty clay ranged from 36 to 37 and from 21 to 22 respectively, with the plasticity index of 14 to 15. The clayey silt had a liquid limit of 31, plastic limit of 21, thus giving the plasticity index of 10. The moisture content of the cohesive soils varied between 28 and 36%.

Groundwater

In the process of augering, water was detected at 2.1 m depth (elevation 223.8) in borehole E1 and a depth of 0.9 m (elevation 225.1) in borehole W1. In the former borehole, artesian water was encountered at 19.8 m depth (elevation 206.1) during drilling, with a head of 1.5 m. No water was observed in borehole S1.

The water level in Hebert Creek was reported to be at elevation 223.9 on October 1, 2009. The groundwater levels at the site are subject to seasonal fluctuations and precipitation patterns.

Engineering Discussion and Recommendations

Replacement Alternatives

The replacement bridge is envisaged to be a single-span (approximately 12 m) structure. The new abutments will be constructed at stations between the existing abutments and piers. The current plans call for analysis of various options of bridge replacement:

- on existing alignment with a detour on either side
- on a new alignment on either side
- in stages with demolition and reconstruction, one half at a time
- in stages by constructing a portion of the new bridge beside the existing structure, demolishing the old bridge and completing the remainder of the new bridge
- on existing alignment utilising an off-site detour with full road closure

The alternatives on the existing alignment or staged construction will require that the embankments be extended between the existing and the new abutments. These new embankment fills may be about 3 m high. The detour embankments and widenings or construction on a new alignment will also require the construction of 3 m high fills.

The subsurface conditions revealed in the boreholes drilled on the existing and new alignments are similar, with the very loose to compact sandy/silty soils extending to about 20 m depth. The densification of these soils will likely occur during pile driving operations, resulting in down-drag on the existing piles, due to embankment settlement, and in the need to implement a programme of monitoring and jacking of the existing bridge. Therefore, staged construction is not considered practical at this site.

A new embankment constructed east or west of the existing would be subjected to settlements due to the consolidation of the loose to very loose sandy soils at the site. The settlements are estimated to be in the order of 100 mm and would require a preloading period of at least 3 to 6 months to mitigate the drag-down forces on the new piles. If it is required that an on-site detour be constructed, care should be taken to ensure that the new embankments are constructed ahead of driving of the piles to minimize drag-down forces, as indicated.

It is recommended to construct the replacement bridge on the existing alignment using an off-site detour. Since the road grade is planned to be raised 0.7 m or less, the existing embankments will not settle significantly. Near the proposed abutments, however, the grades will be raised about 3 m and, as a consequence, there will be a potential for negative skin friction on new piles from consolidation of the 6 m thick cohesive deposit encountered at depth. The fill should be placed in advance of the pile driving to minimize the development of negative skin friction and lateral loads on the piles.

Foundation Considerations

Based on the available information, design and construction of foundations to support the replacement bridge is considered feasible at the site. A summary of advantages, disadvantages and the recommended foundation type is provided in Table 1.

Cognisant of the relatively low bearing resistance of the native soil and the presence of cohesionless soils and boulders below the creek / groundwater level, it is not considered feasible to employ either spread footings or caissons to support the proposed structure foundations. Construction of spread footings on engineered fill would require excavation below the water level and necessitate groundwater control measures.

Use of end-bearing piles driven into the very dense sandy soils is considered to be the preferred foundation system from a foundation engineering perspective. It is noteworthy that installation of piles may encounter some difficulty due to the presence of cobbles and boulders.

The foundation frost penetration depth at this site is 2.1 m according to OPSD-3090.100. The seismic site coefficient for the conditions at the site is 1.0 – Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC) 2006 Edition – for the anticipated foundation conditions.

Shallow Foundations

Taking into account the presence of extensive weak deposits across the site, it is not considered feasible to employ conventional spread footings to support the proposed structure foundations. Spread footings may be constructed on engineered fill that would require removal of the very loose organic sandy/silty soils some 2 m below the creek level and necessitate appropriate groundwater control measures which may not be economically viable.

In view of a relatively short and light structure required, use of a crib foundation is considered feasible at the site. A proprietary system of Guardian Bridge Inc. employs prefabricated abutments on helical screw piles of varying diameter. This scheme should be further assessed during the detail design stage. In particular, the crib foundation placed on the loose sands may not be stable and variations in the water level in the creek may cause erosion/scour of the crib fill and/or associated gabion wing walls. These possible constraints may need extensive site-specific design and add to the cost of the system.

Piles

A foundation system consisting of steel H-piles driven into the very dense sandy soils is recommended. Taking account of the anticipated foundation loads and depth to a competent bearing stratum, construction of integral abutments supported on end-bearing piles is considered to be feasible at this site.

It is anticipated that driven piles will encounter practical refusal in the bouldery soils at depths of 26.0 to 30.5 m (elevation 195.4 to 200.0) below existing grade. A pile penetration of 1 to 2 m into the bearing stratum was assumed for adequate refusal. The H-piles should be designed using the following geotechnical axial resistance at ultimate limit states (ULS) for three pile sections:

	Factored Geotechnical Axial Resistance at ULS, kN	Geotechnical Axial Resistance at SLS, kN
HP 310x110	1600	1200
HP 310x132	2100	1400
HP 310x174	2900	1800

The selection of the pile section for the project should consider the pile length and the fact that heavier pile sections are less likely to be damaged by cobbles / boulders during installation.

Drag-down forces of about 200 to 300 kN, caused by embankment settlement at the abutments, should be considered for preliminary design of foundations for detour alternatives unless the embankments are preloaded ahead of the pile driving, as previously indicated.

The approach embankment fill including fill material placed below grade to replace any excavated unsuitable/compressible soils within the limits of the pile foundation should comprise Granular A or Granular B Type II with a maximum nominal size of 75 mm to enable driving of the piles and minimise the potential for damage during pile installation. Granular B Type II is recommended below the water table if required.

The soil adjacent to the upper portion of the abutment piles is expected to comprise granular fill materials over loose to very loose sandy/silty soils. To accommodate movement of the integral abutment, it is recommended that two concentric CSPs that extend at least 3 m below the bottom of the abutment be placed around the pile to create an annular space. The inner CSP of 600 mm diameter should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP filled with loose uniform sand meeting the requirements given in Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

Since the piles will be about 30 m long and the soil cover generally comprises sandy/silty soils interlayered with a clayey deposit, it is considered, based on our extensive experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed.

The piles should be driven to a set of about 20 blows per 25 mm penetration and rising for the last 75 mm of penetration. This should be confirmed by dynamic analysis (such as the Hiley formula) in the process of pile installation.

The H-piles will set into the very dense sandy soils with cobbles / boulders and should be equipped with driving shoes as per OPSD-3000.100 or the Titus ‘H’ Bearing Pile Points, Standard model, in accordance with SP 903S01.

The piles should be installed and monitored in accordance with the requirements of SP 903S01. This should involve confirmation of the founding elevation, alignment, plumbness, uniformity of set and quality of splices, and should be done on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 2.1 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. The lateral resistance for the pile sections recommended is as follows:

Parameters	**Cohesionless Soils**		**Granular Fill**
	Very Loose	**Loose**	
Factored Lateral Resistance at ULS, kN	110	130	150
Lateral Resistance at SLS, kN	25	40	50

If greater resistance is required, batter piles should be installed.

The coefficient of horizontal subgrade reaction, k_s (kN/m^3), should be computed using the following equations to evaluate the point of contraflexture:

k_s = $n_h\, z/b$
n_h = coefficient related to soil density
= 12 MN/m^3 for granular fill
= 1 MN/m^3 for native sand/silt
z = depth, m
b = pile width, m

Abutment Walls

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation, assuming a triangular pressure distribution:

$$p = K(\gamma h + q) + Cp + Cs$$

where K = coefficient of lateral earth pressure (dimensionless)
γ = unit weight of free-draining granular material, kN/m^3
h = depth below final grade, m
q = surcharge load, kPa, if present
Cp = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
Cs = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where Ø = angle of internal friction of retained soil (35º for Granular B Type II)
δ = angle of friction between the soil and wall (23.5º for Granular B Type II)

Free-draining granular material or rockfill should be used as backfill behind the walls. The following parameters are recommended for design:

Parameters	Granular A or Granular B Type II	Rockfill
Angle of Internal Friction, degrees	35	42
Unit Weight, kN/m^3	22.8	18.0
Coefficient of Active Earth Pressure K_a	0.27	0.20
Coefficient of Earth Pressure At-Rest K_o	0.43	0.33
Coefficient of Passive Earth Pressure K_p	3.69	5.04

Refer to MTO Report SO-96-11 for procedures to determine the earth pressure coefficient to be employed for design of integral abutments. The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures. The earth pressure coefficients should be reviewed if the slope of the backfill exceeds 10° to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load (q in the preceding equation).

A weeping tile system (SP 405F03 and OPSD 3190.100) should be installed to minimise the build-up of hydrostatic pressure behind the walls. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

Backfilling adjacent to retaining structures should be carried out in conformance with Ontario Provincial Standards specifications for granular or rock backfill at abutments (OPSD 3101.150 and 3101.200).

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to SP 105S10 for additional information in this regard.

Installation of a temporary roadway protection system behind each abutment will be needed if staged construction of the bridge is adopted. Recommendations for criteria for the temporary roadway protection should be provided during the detailed design phase.

Approach Embankments

It is anticipated that the approach embankments will be constructed with earth borrow, granular material or rockfill. The height of fill embankments will be about 3 m at both approaches. Construction of the fill on a new alignment is considered to be feasible.

The topsoil (200 mm thick in borehole E1) encountered during construction at the abutment locations and along the alignment of the approach fills within 20 m of the abutments should be stripped prior to placement of the embankment fill.

Backfilling adjacent to the structure abutments should be carried out in conformance to Ontario Provincial Standards specifications for granular or rock backfill at abutments (OPSD 3101.150 and 3101.200). As noted earlier, Granular A or Granular B Type II should be employed within the limits of driven piles.

The embankments should be constructed in accordance with OPSD 201.020, 202.010 and SP 206S03. The side slopes of the approach embankments should be inclined no steeper than 2H:1V for earth fill and 1.25H:1V for rockfill. A 2 m wide mid-height berm for erosion control and slope maintenance purposes is not necessary at the site.

Where slope flattening is proposed, a drainage gap should be provided in accordance with OPSD 202.020. Where slopes are flattened to eliminate the need for a guiderail, a granular infilled drainage gap should be provided in accordance with the Northeastern Region Pavement Design Practices and Guidelines. Granular B Type II should be used for the drainage gaps.

It is considered that the approach embankments constructed in accordance with these recommendations will be stable. Some settlement of the road surface should be expected as a result of two mechanisms – consolidation of the native soil below the recently placed fill and 'consolidation' of the new fill.

Settlement of the existing embankment fill due to consolidation of the subgrade soil at both embankments is computed to be within 25 mm and completed within three months following fill placement. Settlement of a new embankment constructed on a detour alignment east or west of the existing would be some 90 mm and completed within six months after placement of the fill.

The backfill placed adjacent to the abutments will be about 3 m thick. The magnitude of 'consolidation' of this fill will be dependent on the workmanship employed by the contractor and, if placed in 200 to 300 mm thick lifts compacted to 100% of the standard Proctor maximum dry density in accordance with the requirements of SP 902S01 and OPSS 501 (Method A), should be in the order of 10 mm at both abutments.

Consequently, the total settlement of the approach fill surface near the abutments should be less than 35 mm on the existing alignment and 100 mm on a detour alignment. The total settlement will be essentially complete within 3 months after fill placement on the existing alignment and 6 months on a new alignment.

Earth fill slopes where employed should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 571 or 572 for time constraints and the type of seed and mulch required.

Excavation and Groundwater Control

Excavation for construction of foundations at the abutment locations is expected to extend through the embankment fill and/or very loose to compact sandy/silty soils to a depth less than 3 m below existing grade. Excavation of these soils should be relatively straightforward.

The fill and loose to compact sand/silt are classified as Type 3 and very loose sand/silt as Type 4 soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Since open cut procedures are governed by soils with the highest number, temporary cut slopes over the full depth of excavation should therefore be inclined at 3 horizontal to 1 vertical.

The stabilised groundwater level is expected to be consistent with the water level in Hebert Creek, near elevation 223.9 in early October 2009. Taking account of the relatively pervious sandy/silty soils at the site, conventional sump pumping techniques are unlikely to be able to handle groundwater seepage if the excavation at the abutment locations extends more than 0.6 m below the water level. A steel sheeting cofferdam will be needed in such a case with the base sealed using a layer of tremie concrete.

The sheet piles should extend to a depth equal to at least 2 times the excavation depth below the creek water level to minimise the potential for bottom heave. The specification should call for a groundwater control specialist and clearly state that control of water is the contractor's responsibility.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

We trust this is sufficient for your immediate purposes.

Yours very truly

Peto MacCallum Ltd.

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Enclosure(s):
Table 1 – Summary of Advantages, Disadvantages and Recommended Foundation
Table 2 – Gradation Specification for Sand Fill in Pre-Augered Holes at Integral Abutments
Figures PC-1 to PC-3 – Plasticity Charts
Figures GS-1 to GS-7 – Results of Grain Size Distribution Analyses
Figure C-1 – Consolidation Test Results
Explanation of Terms Used in Report
Record of Borehole Sheets
Drawing 1 – Borehole Locations and Soil Strata
Appendix A – Site Photographs

TABLE 1

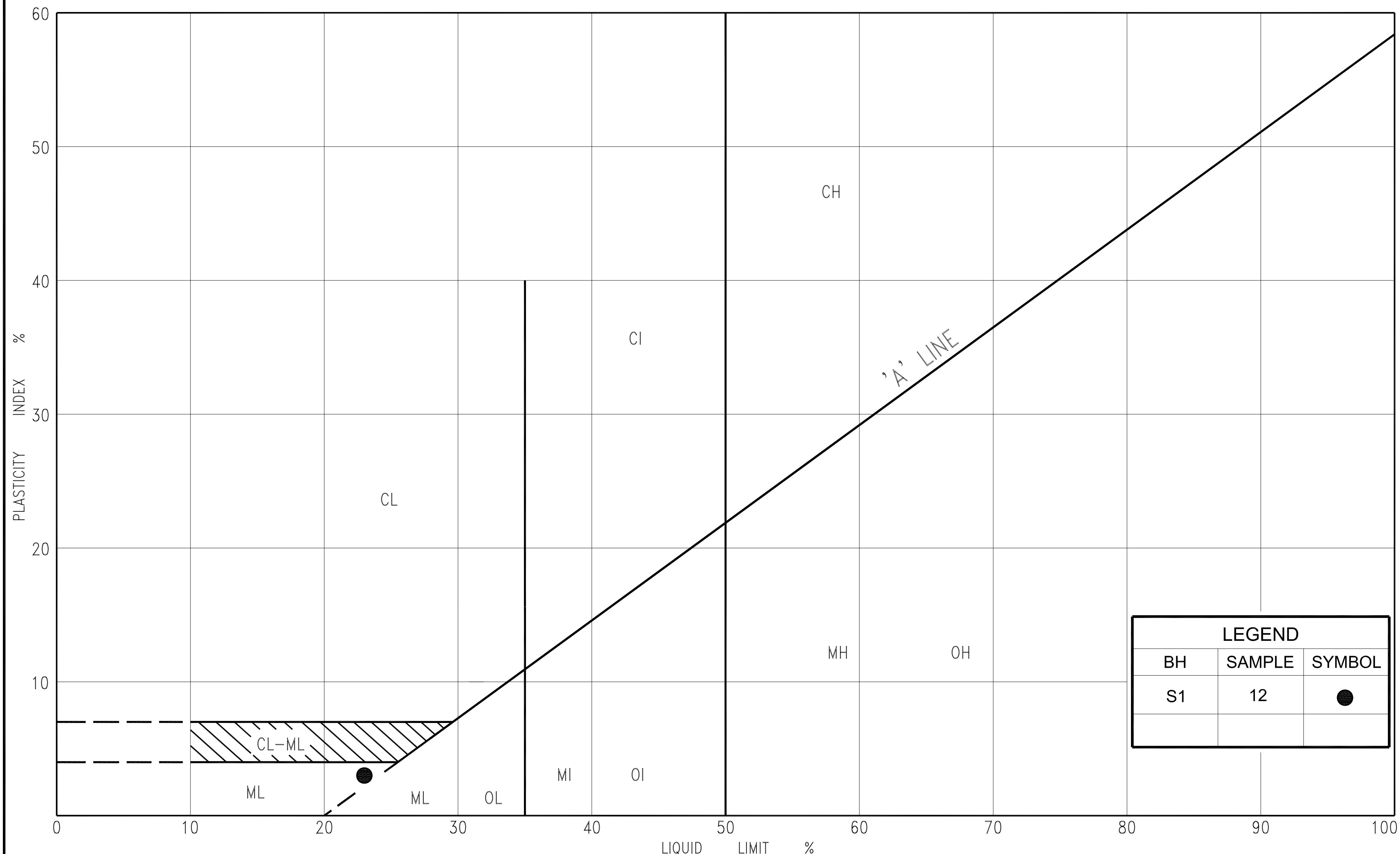
SUMMARY OF ADVANTAGES, DISADVANTAGES AND RECOMMENDED FOUNDATION

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS	RISKS / CONSEQUENCES	RECOMMENDED FOUNDATION TYPE
Spread footings on native soil	• Ease of construction relative to piles / caissons	• Low bearing resistance necessitates large footings • Large amount of settlements anticipated • High groundwater level • Dewatering required	• Lower cost than for piles / caissons	• Large footings may not be feasible • Post-construction settlements	Driven piles
Spread footings on engineered fill pad	• Ease of construction relative to piles / caissons	• Low bearing resistance relative to piles / caissons • Large amount of settlements anticipated • High groundwater level • Dewatering required	• Lower cost than for piles / caissons	• Groundwater control measures needed for excavation • Post-construction settlements	
Crib foundation	• Fast installation due to prefabrication	• Stability concerns • Deep foundation support required (helical screw piles) • Susceptibility to erosion from creek flow may require sheet pilling	• High cost due to site conditions	• Potential erosion may wash away foundation support • Instability of crib may cause abutment failure	
Driven piles	• Higher capacity than for footings • Construction of integral abutments possible	• Heavy pile driving equipment necessary	• Higher cost than for footings	• Special care required during pile installation near existing bridge	
Caissons	• Higher capacity than for footings	• Need to advance through cohesionless soils and boulders below creek / groundwater level • Need to employ tremie techniques to place concrete	• High cost relative to other alternatives	• Installation considered not feasible due to groundwater conditions and loose sandy soils	

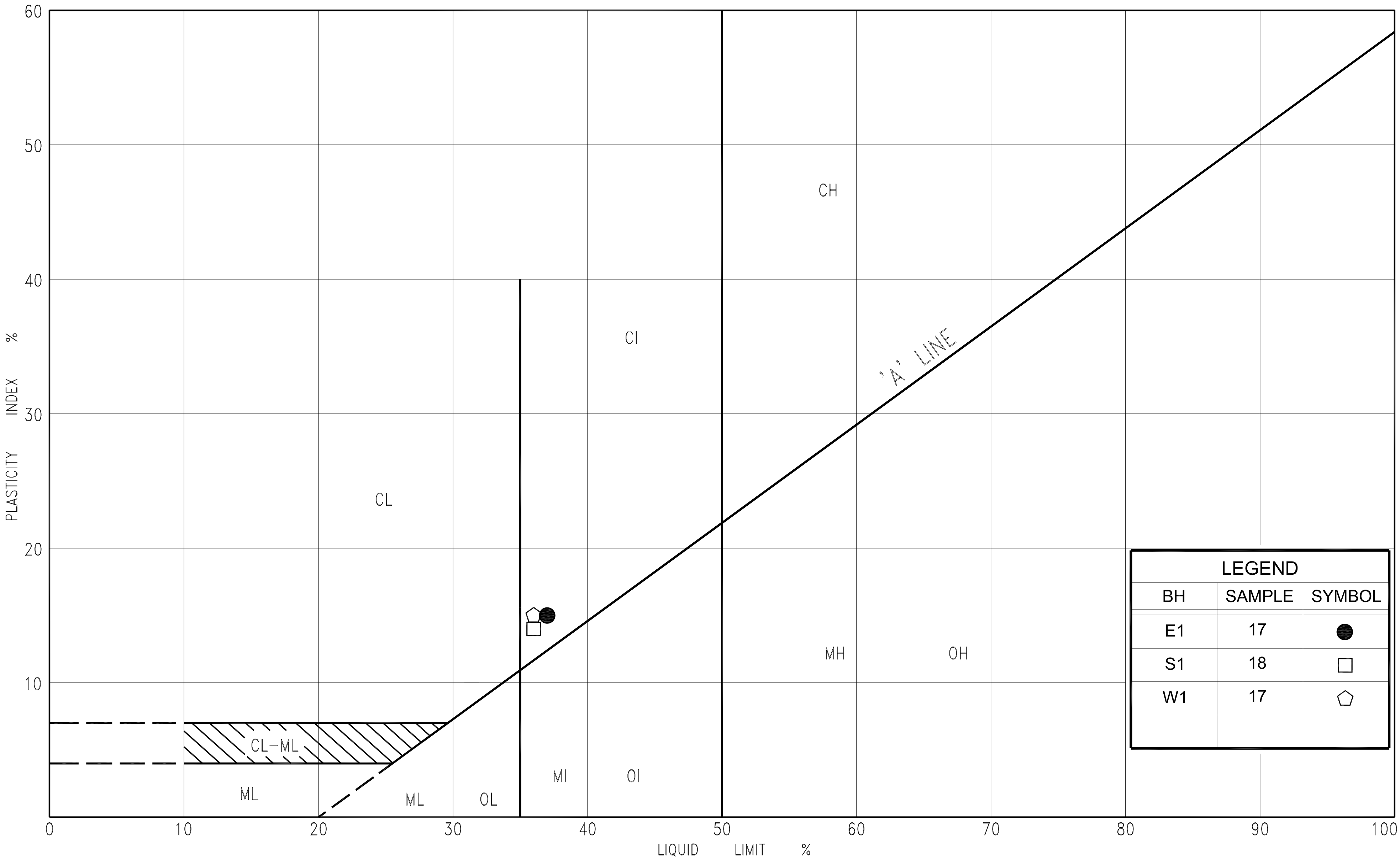
TABLE 2

Gradation Specification for Sand Fill in Pre-Augered Holes at Integral Abutments

MTO SIEVE DESIGNATION		PERCENTAGE PASSING BY MASS
2 mm	#10	100
600 μm	#30	80 – 100
425 μm	#40	40 – 80
250 μm	#60	5 – 25
150 μm	#100	0 – 6



Ministry of Transportation Ontario	PLASTICITY CHART	FIG No. PC-1
	SILT, some clay, trace sand	HWY: 539
		W.P. No. 5116-07-00



LEGEND

BH	SAMPLE	SYMBOL
E1	17	●
S1	18	□
W1	17	⬠

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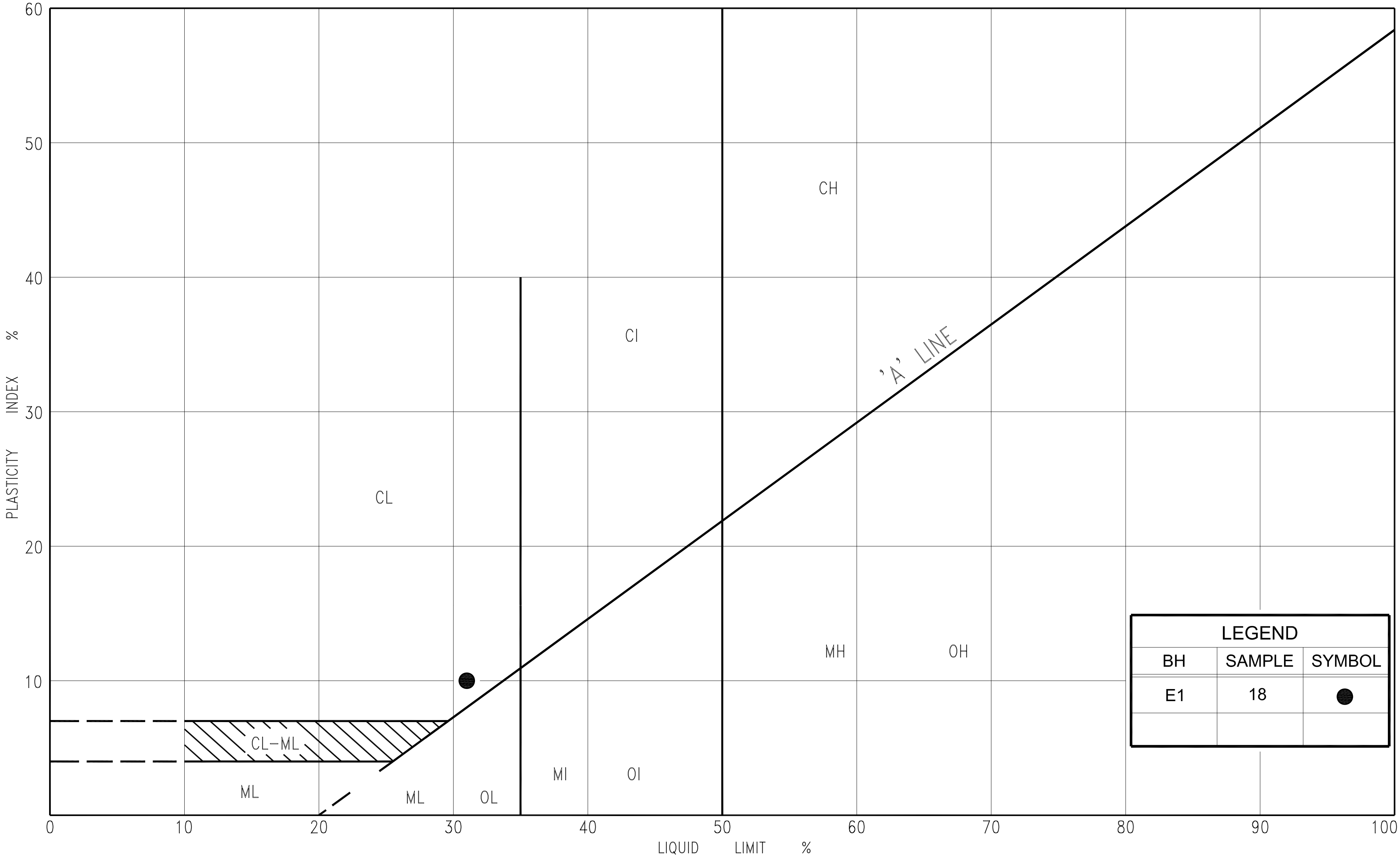
PLASTICITY CHART

SILTY CLAY

FIG No.	PC-2
HWY:	539
W.P. No.	5116-07-00



LEGEND

BH	SAMPLE	SYMBOL
E1	18	●

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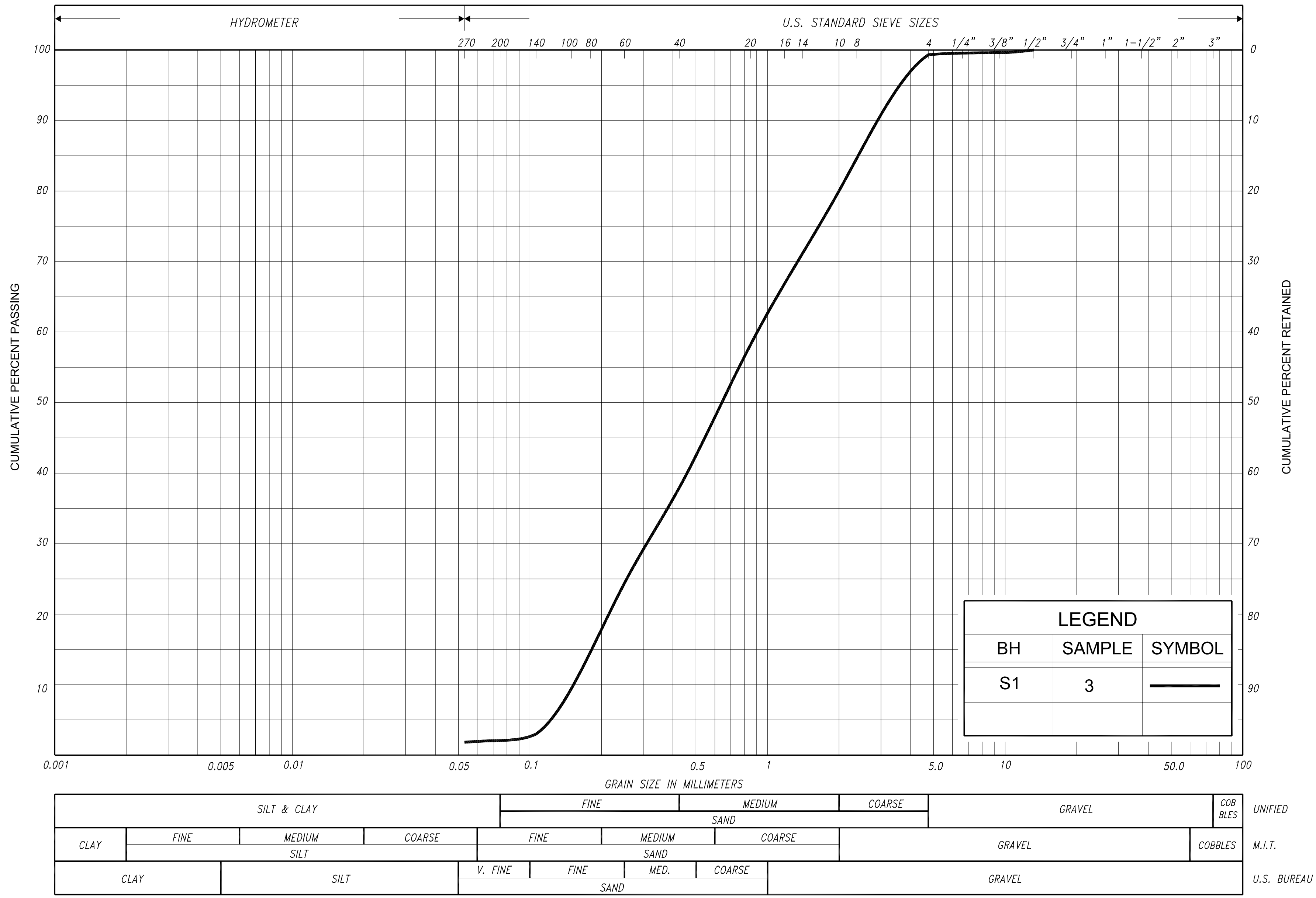
PLASTICITY CHART

CLAYEY SILT, trace sand

FIG No.	PC-3
HWY:	539
W.P. No.	5116-07-00



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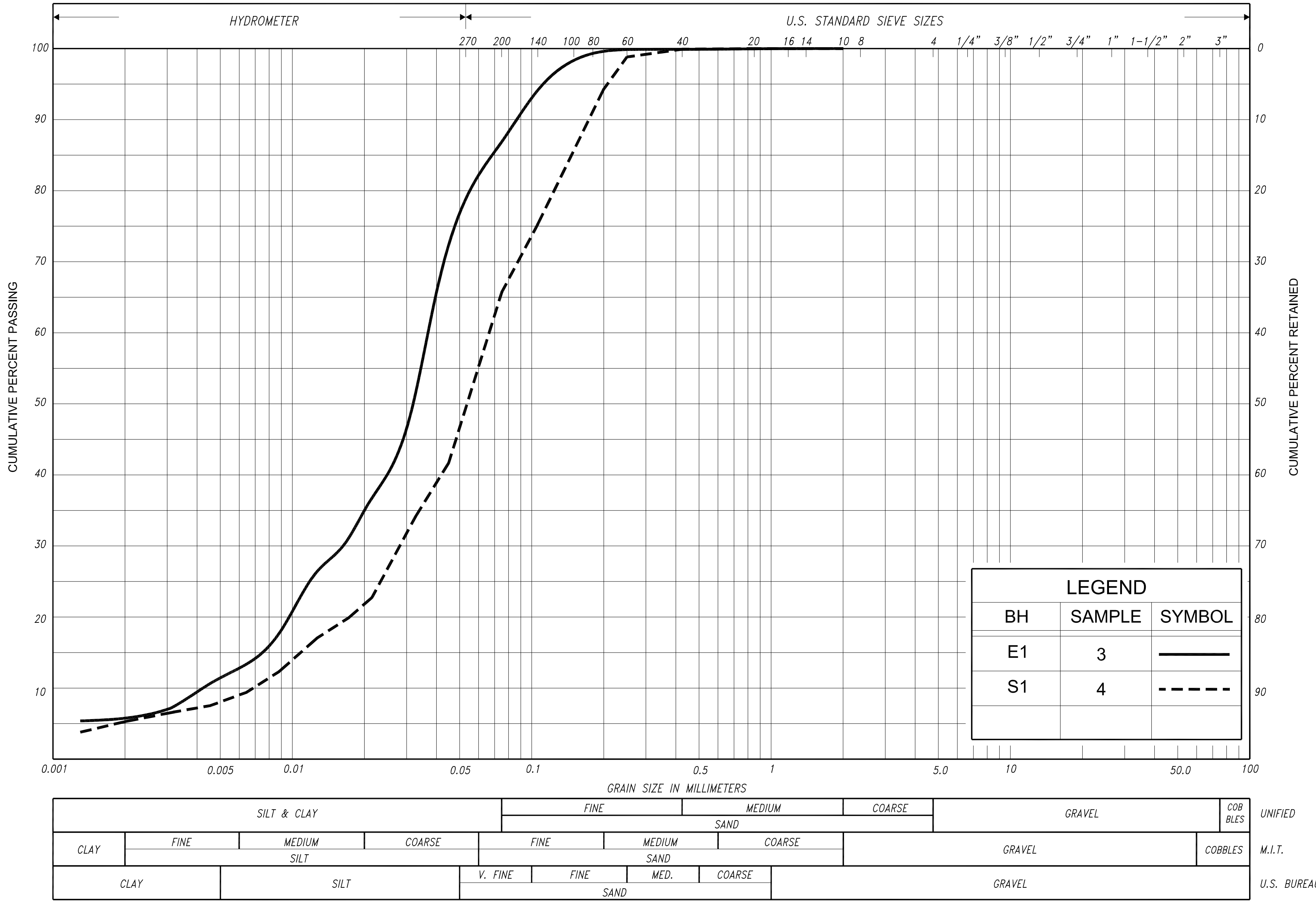
GRAIN SIZE DISTRIBUTION

SAND, trace silt, trace gravel
(FILL)

FIG No.	GS-1
HWY:	539
W.P. No.	5116-07-00



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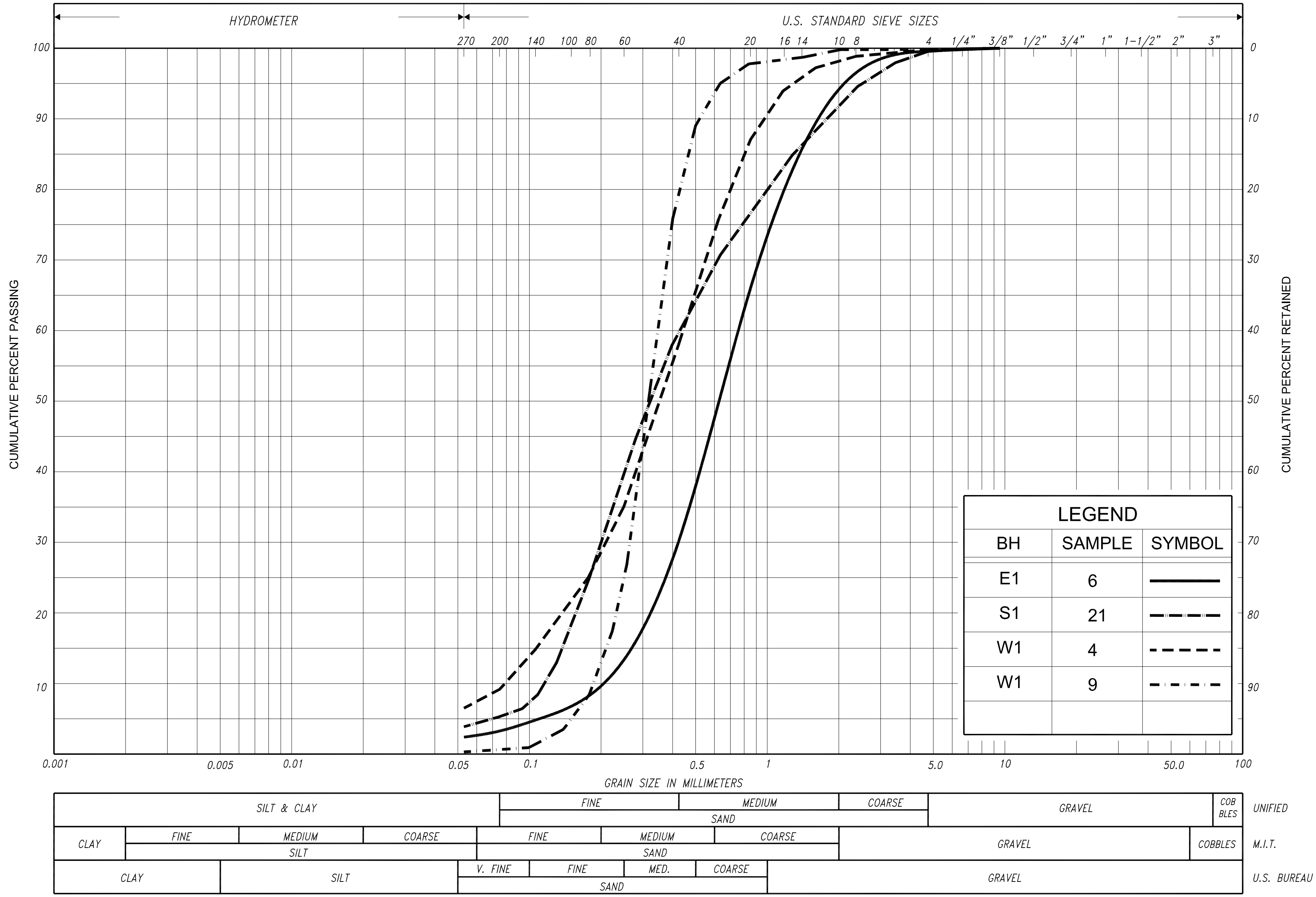
Ontario

GRAIN SIZE DISTRIBUTION

SILT / SANDY SILT, trace clay

FIG No.	GS-2
HWY:	539
W.P. No.	5116-07-00



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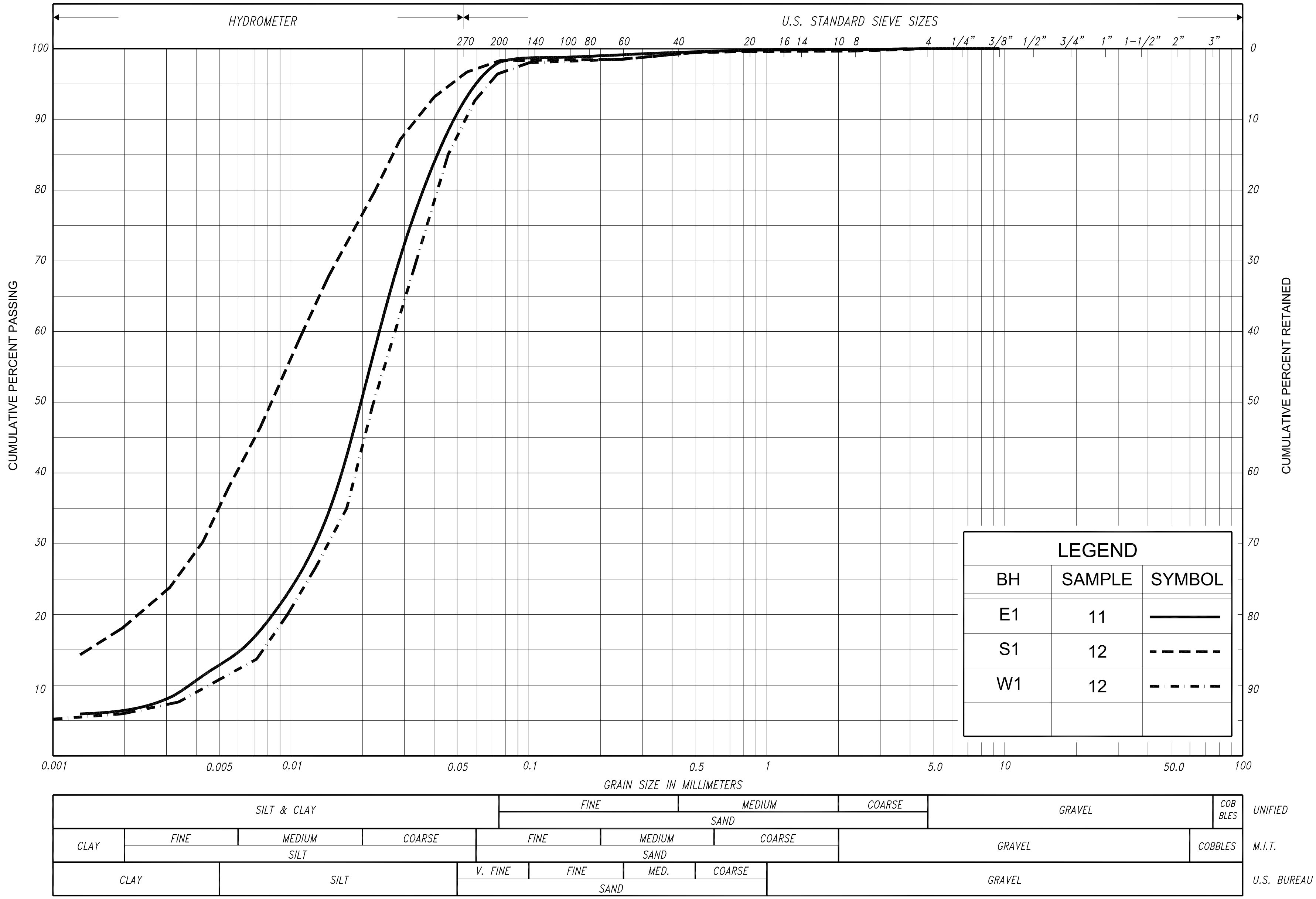
GRAIN SIZE DISTRIBUTION

SAND, trace silt, trace gravel

FIG No.	GS-3
HWY:	539
W.P. No.	5116-07-00



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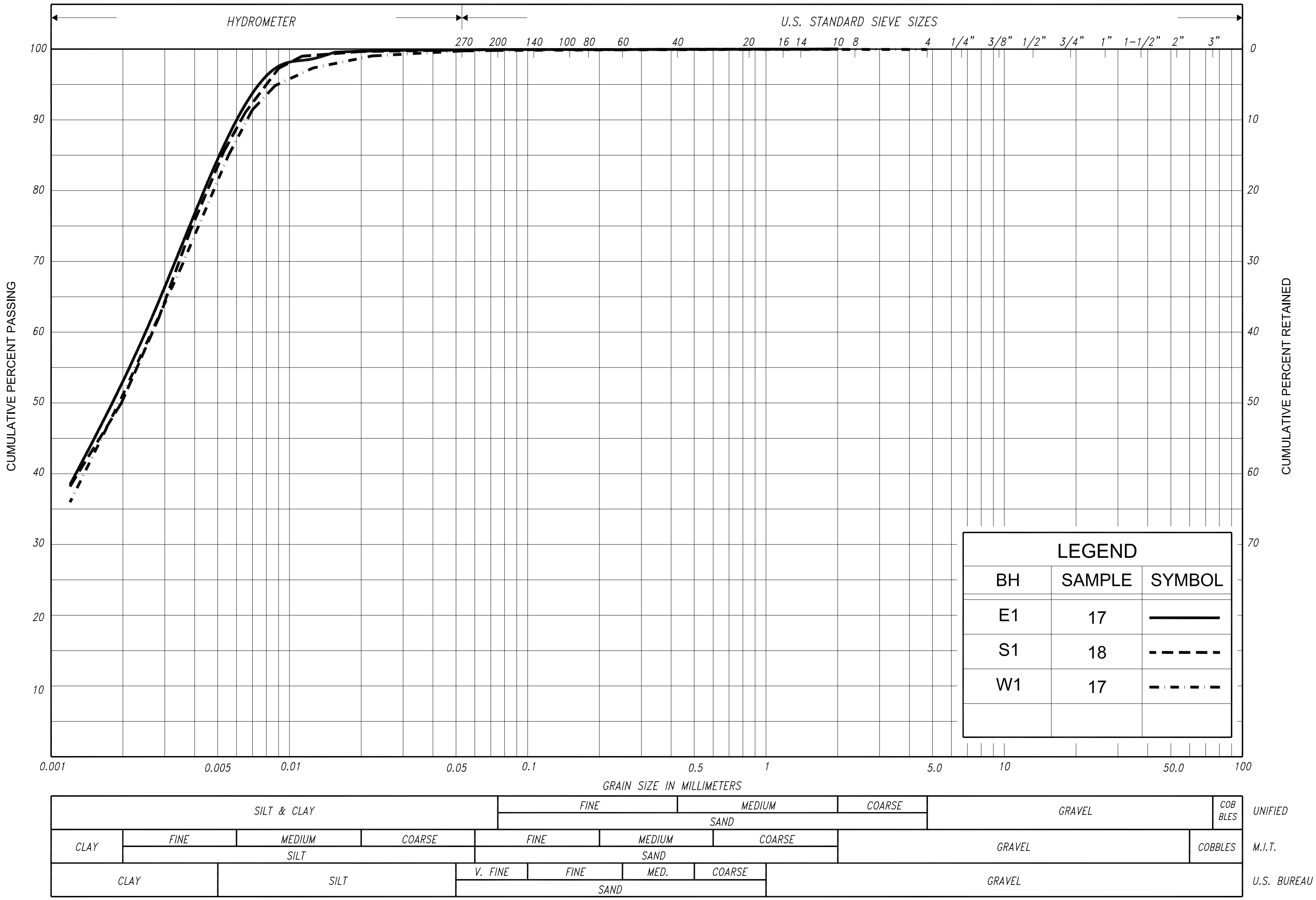
GRAIN SIZE DISTRIBUTION

SILT, trace to some clay, trace sand

FIG No.	GS-4
HWY:	539
W.P. No.	5116-07-00



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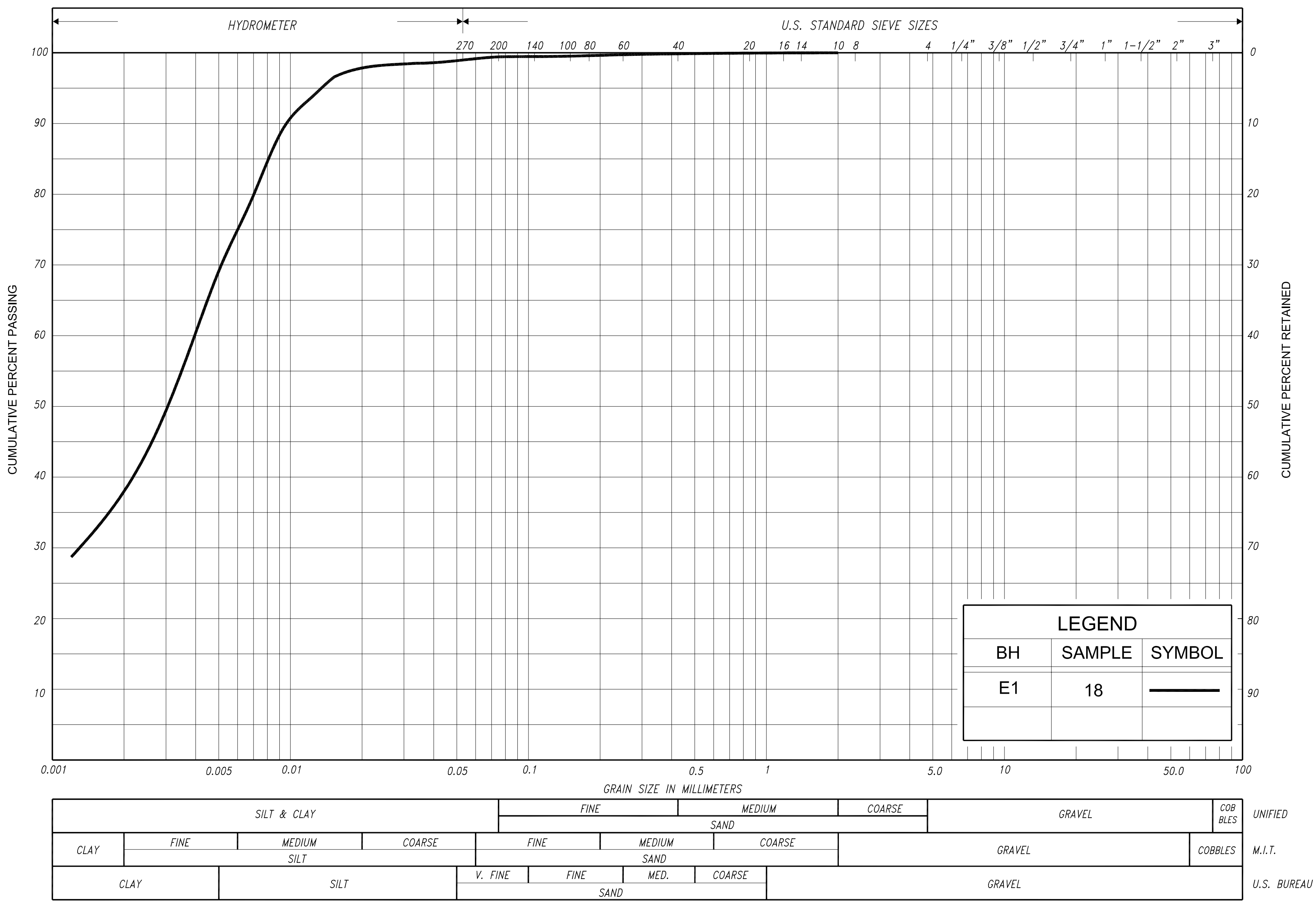
GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIG No.	GS-5
HWY:	539
W.P. No.	5116-07-00



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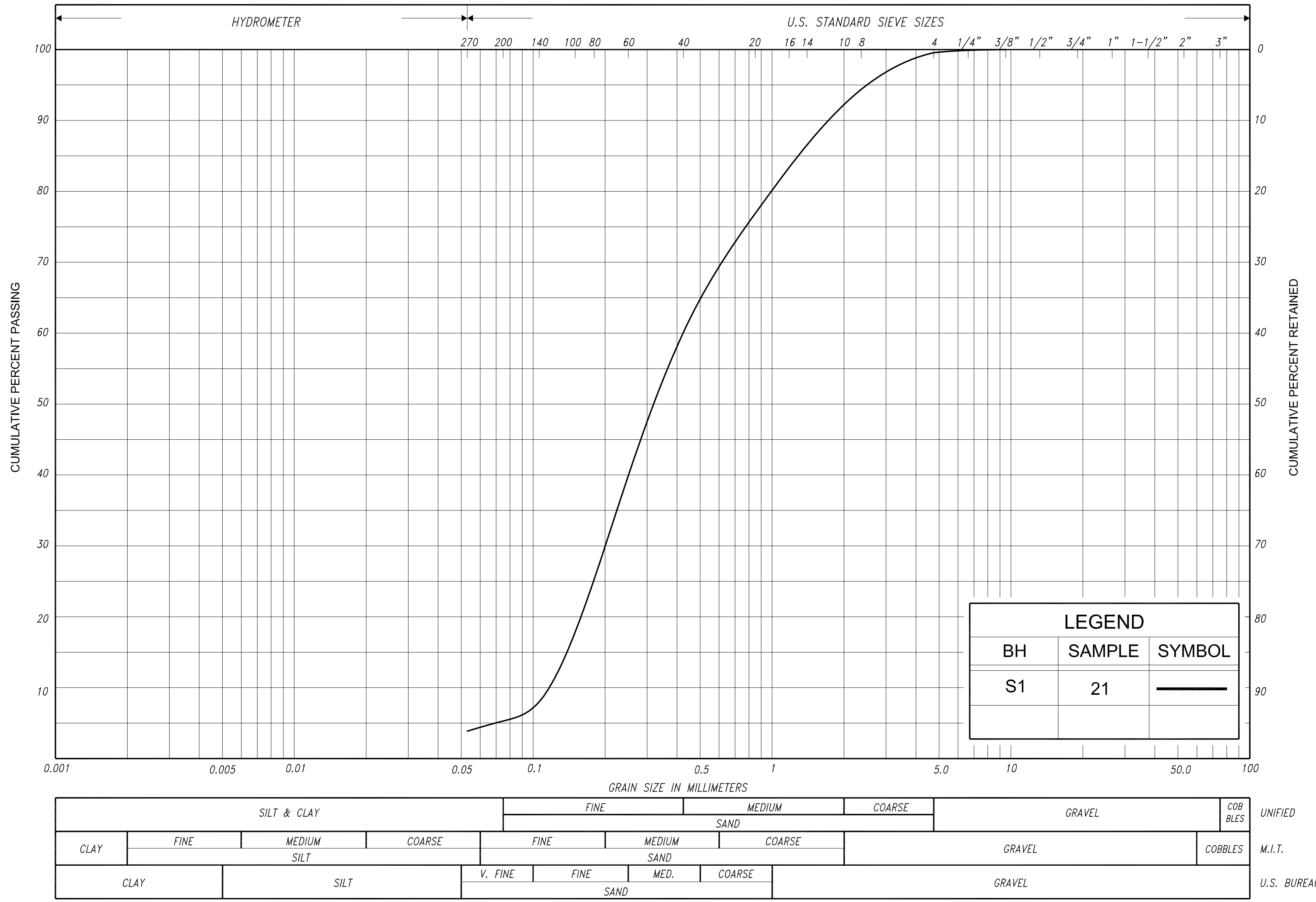
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand

FIG No.	GS-6
HWY:	539
W.P. No.	5116-07-00



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Ontario

GRAIN SIZE DISTRIBUTION

SAND, trace silt, trace gravel

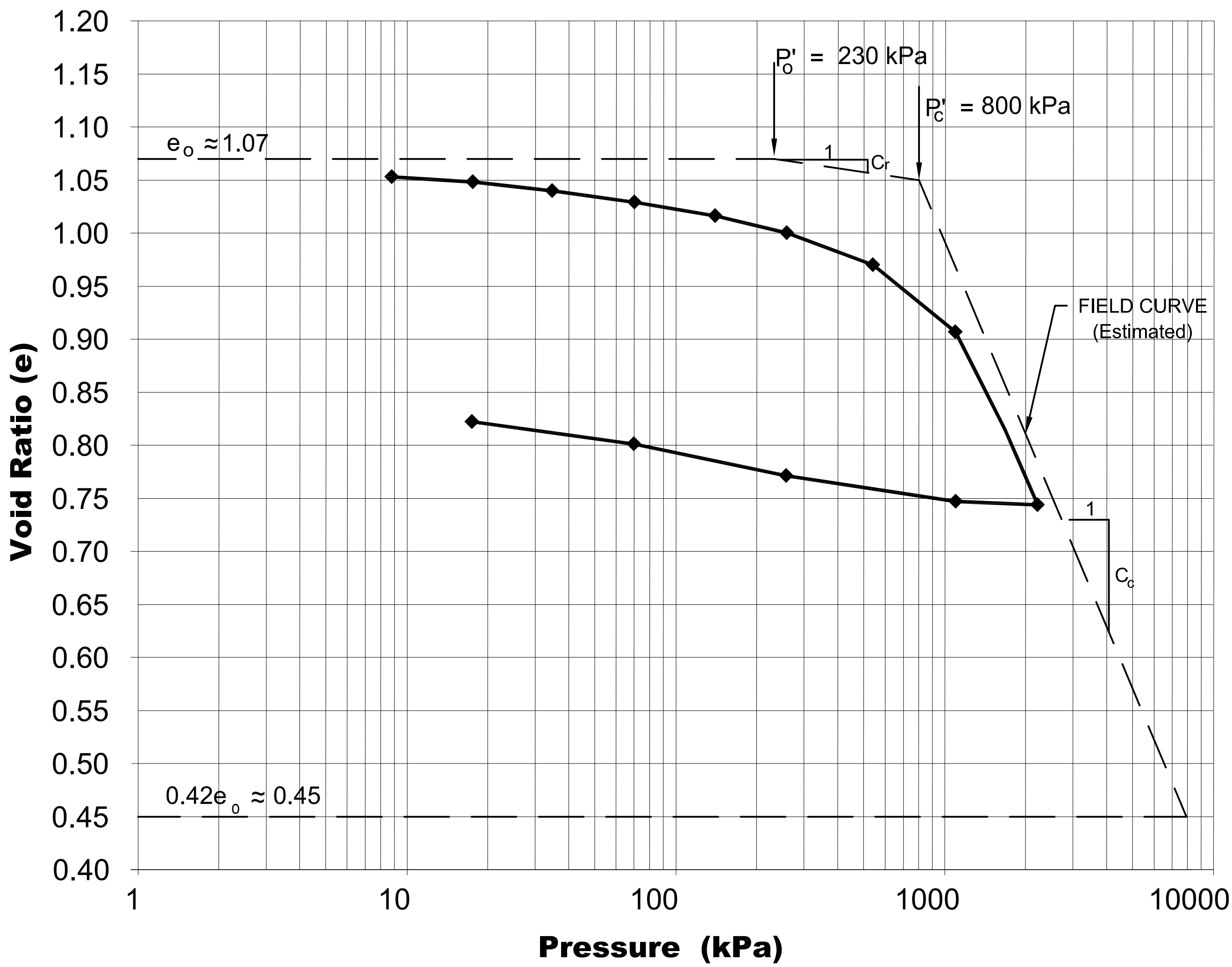
FIG No.	GS-7
HWY:	539
W.P. No.	5116-07-00

Laboratory Consolidation Test Results

Highway 539
Hebert Creek Bridge Replacement
District 54, Ontario

Location Sta.10+147, 11.0 m Lt.
Borehole E1, Sample 18, Depth 24.4 - 25.0 m

Void Ratio versus Log of Pressure



SOIL TYPE: CLAYEY SILT, trace sand				
$e_o \approx 1.07$	P'_o = 230 kPa	W_L = 31	FIGURE No:	C-1
W_o = 40%	P'_c = 800 kPa	W_P = 21	HIGHWAY:	539
γ = 18.7 kN/m^3	C_c = 0.60	PI = 10	TOWNSHIP:	BADGEROW
	C_r = 0.04		G.W.P.	5116-07-00

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 'A' 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/0.3 m)	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:

R Q D (%)	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
F V	FIELD VANE		

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO

n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

RECORD OF BOREHOLE No E1 1 of 3 METRIC

G.W.P. 5116-07-00 LOCATION Hwy 539 Sta. 10+147, o/s 11.0m Lt. CL ORIGINATED BY F.P.

DIST 54 HWY 539 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.S.B.

DATUM Geodetic DATE October 06 & 07, 2009 CHECKED BY C.N.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
225.9 / 0.0	Ground Surface
225.7 / 0.2	Topsoil
225.3 / 0.6	Silty sand, trace gravel organic inclusions
	Very loose Brown Moist
224.5 / 1.4	Gravelly sand Loose Brown Wet
222.2 / 3.7	Silt trace sand, trace clay layers of silty sand Loose to very loose Grey Wet; layers of peat
215.8 / 10.1	Sand trace silt, trace gravel Loose to very loose Brown Wet; Grey; Compact
	Silt trace sand, trace clay Loose to very loose Grey Wet
	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	ELEVATION (approx.)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	2			
2	SS	4			
3	SS	7		(Non-plastic)	0 13 81 6
4	SS	2			
5	SS	2			
6	SS	8			1 96 (3)
7	SS	6			
8	SS	4			
9	SS	1			
10	SS	11			
11	SS	9			0 2 91 7
12	SS	3			
13	SS	6			

GROUND WATER CONDITIONS: ∇* at approx. El. 224.1

ELEVATION SCALE: 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211

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

+7, ×5: Numbers refer to Sensitivity — 20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd. Foundation Design

RECORD OF BOREHOLE No E1

METRIC

G.W.P. 5116-07-00 LOCATION Hwy 539 Sta. 10+147, o/s 11.0m Lt. CL ORIGINATED BY F.P.

DIST 54 HWY 539 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.S.B.

DATUM Geodetic DATE October 06 & 07, 2009 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
210.9	Silt, trace sand, trace clay. Compact to loose, Grey, Wet (Cont'd)	14	SS	12	
		15	SS	13	
		16	SS	9	
205.8 / 20.1	Silty clay. Stiff, Grey, Wet	17	SS	1	0 0 47 53
			FV		
202.6 / 23.3	Clayey silt, trace sand. Firm to stiff, Grey, Wet	18	TW	PH	0 1 61 38
			FV		
199.4 / 26.5	Silt, trace sand, layers of sand. Compact to dense, Grey, Wet	19	SS	32	
		20	SS	50/13cm	
197.3 / 28.6	End of borehole. Refusal on probable boulder. Sample 20: Sampler bouncing				

Cont'd

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (210 to 198); 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.

Field vane values: +5 near elev. 204; +3 near elev. 201.

$+^7$, $\times^5$: Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No E1

METRIC

G.W.P. 5116-07-00 LOCATION Hwy 539 Sta. 10+147, o/s 11.0m Lt. CL ORIGINATED BY F.P.

DIST 54 HWY 539 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.S.B.

DATUM Geodetic DATE October 06 & 07, 2009 CHECKED BY C.N.

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O 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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
195.9											
	* 2009 10 06 Water level observed during drilling Artesian well										

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
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RECORD OF BOREHOLE No S1

METRIC

G.W.P. 5116-07-00 LOCATION Hwy 539 Sta. 10+167, o/s 3.0m Lt. CL ORIGINATED BY F.P.

DIST 54 HWY 539 BOREHOLE TYPE C. F. H. S. A. + NQ Diamond Coring COMPILED BY N.S.B.

DATUM Geodetic DATE October 08, 13 & 14, 2009 CHECKED BY C.N.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
226.8	Ground Surface
226.5 / 0.0–0.3	75mm asphalt over 175mm sand and gravel. Compact Brown Moist (PAVEMENT FILL)
	Sand, trace silt, trace gravel. Compact to loose Brown Moist (FILL)
224.6 / 2.2	Sandy silt, trace clay, organic inclusions. Very loose Brown Wet; Loose to compact Grey
221.0 / 5.8	Sand, trace silt. Loose Grey Wet
216.7 / 10.1	Silt, trace sand, trace to some clay. Loose Grey Wet
	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	Note
1	SS	17		
2	SS	7		
3	SS	5	1 97 (2)	
4	SS	1	0 34 61 5	(Non-plastic)
5	SS	3		
6	SS	5		
7	SS	18		
8	SS	6		
9	SS	6		
10	SS	7		
11	SS	7		
12	SS	6	0 2 80 18	
13	SS	9		
14	SS	5		

Elevation scale: 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212

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

+7, ×5 : Numbers refer to Sensitivity; 20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

Peto MacCallum Ltd.

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RECORD OF BOREHOLE No S1

METRIC

G.W.P. 5116-07-00 LOCATION Hwy 539 Sta. 10+167, o/s 3.0m Lt. CL ORIGINATED BY F.P.

DIST 54 HWY 539 BOREHOLE TYPE C. F. H. S. A. + NQ Diamond Coring COMPILED BY N.S.B.

DATUM Geodetic DATE October 08, 13 & 14, 2009 CHECKED BY C.N.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
211.8	Silt, trace sand trace to some clay; Compact to loose, Grey, Wet	15	SS	12	211			
		16	SS	17	210			
		17	SS	7	209 / 208			
206.7 / 20.1	Silty clay; Stiff, Grey, Wet				207 / 206			
		18	SS	WH**	205		Wp–w	0 0 49 51
			FV			+ 4		
					204 / 203			
		19	TW	PM	202			
			FV			+ 3		
200.6 / 26.2	Sand trace silt, trace gravel cobbles and boulders; Dense to very dense, Grey, Wet	20	RC NQ	-	201 / 200			
		21	SS	52	199			1 93 (6)
		22	RC NQ	-	198			
	Cont'd	23	SS	50/3cm	197			

+7, ×5: Numbers refer to Sensitivity

20 / 15–o–5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No S1

METRIC

G.W.P. 5116-07-00 LOCATION Hwy 539 Sta. 10+167, o/s 3.0m Lt. CL ORIGINATED BY F.P.

DIST 54 HWY 539 BOREHOLE TYPE C. F. H. S. A. + NQ Diamond Coring COMPILED BY N.S.B.

DATUM Geodetic DATE October 08, 13 & 14, 2009 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O 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	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
196.8											
	Sand trace silt, trace gravel cobbles and boulders Very dense Grey Wet		24	RC NQ	-						
							196				
			25	RC NQ	-						
							195				
							194				
193.8 33.0			26	SS	50/5cm						
	End of borehole Refusal on probable boulder Samples 23 and 26: Sampler bouncing * Borehole charged with drilling water WH** denotes penetration due to weight of rods and hammer C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers										

$+^7$, $\times^5$: Numbers refer to Sensitivity

20
15—○—5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No W1

METRIC

G.W.P. 5116-07-00 LOCATION Hwy 539 Sta. 10+144, o/s 11.0m Rt. CL ORIGINATED BY F.P.

DIST 54 HWY 539 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.S.B.

DATUM Geodetic DATE October 07 and 08, 2009 CHECKED BY C.N.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
226.0	Ground Surface
0.0	Sand and gravel, organic inclusions; Very loose, Wet (FILL)
225.2 / 0.8	Sandy silt, organic inclusions; Very loose Grey, Wet
223.9 / 2.1	Sand, trace silt; Very loose to loose, Brown, Wet; Grey; trace gravel, Compact
214.9 / 11.1	Silt, trace sand, trace clay; Compact to very loose, Grey, Wet
	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	2		
2	SS	2	225	
3	SS	1	224	
4	SS	3	223	0 91 (9)
5	SS	1		
6	SS	6	222	
7	SS	6	221	
			220	
8	SS	7	219	
9	SS	7	218	0 99 (1)
			217	
10	SS	15	216	
11	SS	15	215	
			214	
12	SS	4	213	0 3 91 6 (Non-plastic)
13	SS	WH**	212	

GROUND WATER CONDITIONS: ∇* (at approx. El. 225.3)

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

+7, ×5: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No W1

METRIC

G.W.P. 5116-07-00 LOCATION Hwy 539 Sta. 10+144, o/s 11.0m Rt. CL ORIGINATED BY F.P.
DIST 54 HWY 539 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY N.S.B.
DATUM Geodetic DATE October 07 and 08, 2009 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
211.0	Silt, trace sand, trace clay. Compact to loose, Grey, Wet (Cont'd)	14	SS	11			o	
					210			
		15	SS	8	209		o	
					208			
		16	SS	14	207		o	
206.5 / 19.5	Silty clay. Stiff, Grey, Wet				206			
					205			
		17	SS	WH			w_P — w — w_L	0 0 49 51
			FV		204	+ (4)		
					203			
					202			
		18	TW	PM			o	
201.0 / 25.0	Sandy silt, trace gravel. Very dense, Grey, Wet	19	SS	50/3cm	201			
200.7 / 25.3	End of borehole. Refusal on probable boulder							

Sample 19: Sampler bouncing

* 2009 10 08

∇ Water level observed during drilling

WH** denotes penetration due to weight of rods and hammer

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; UNIT WEIGHT γ kN/m³

+7, ×5 : Numbers refer to Sensitivity   20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

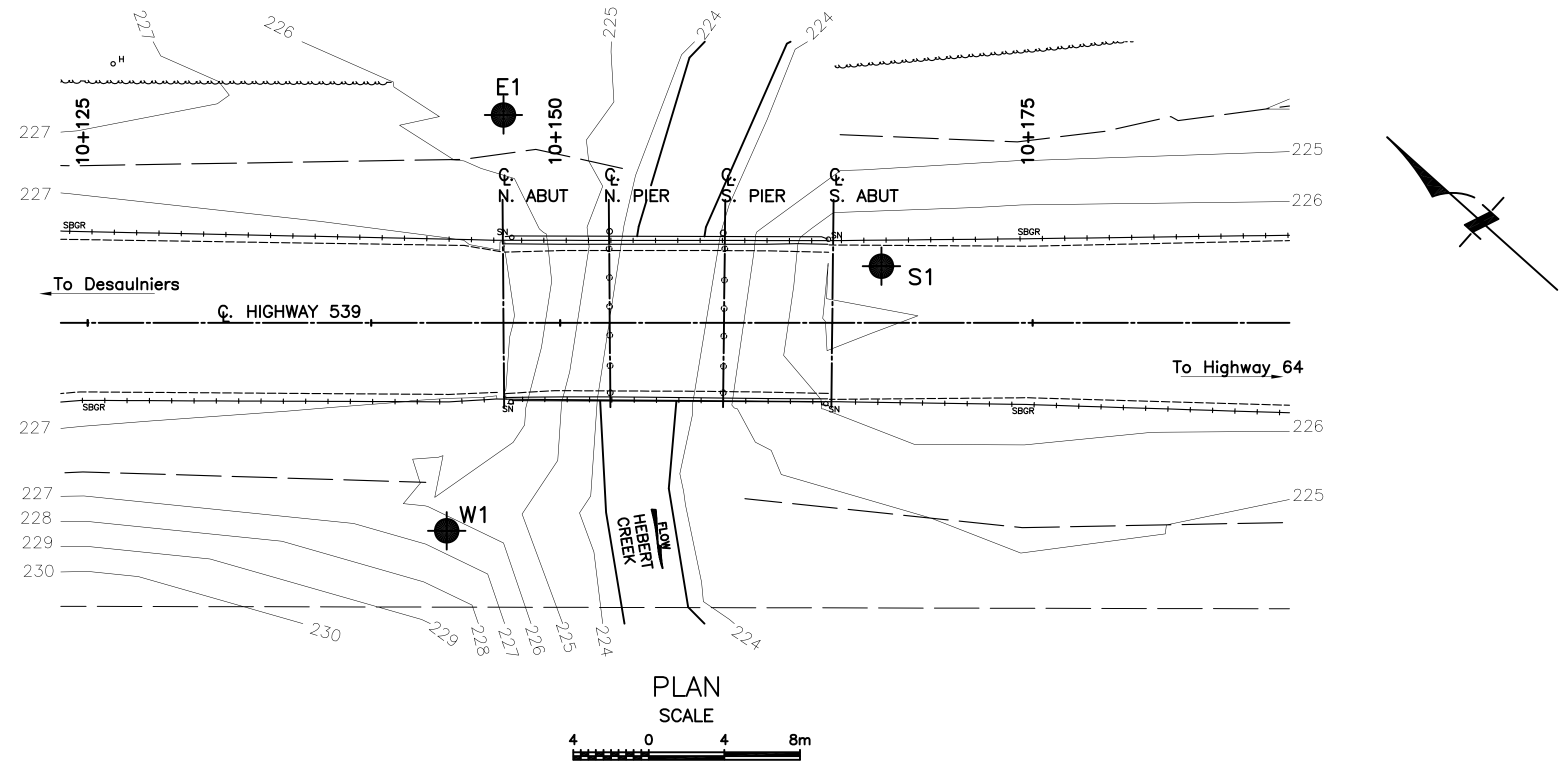
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

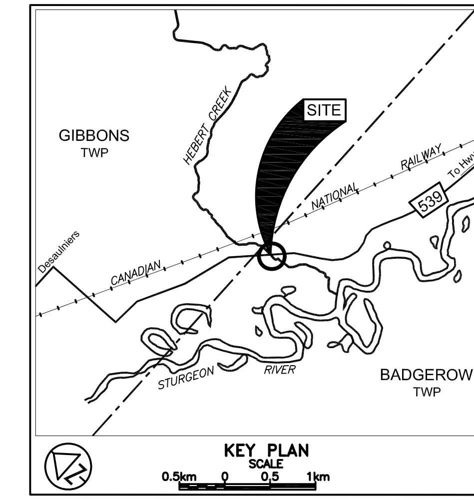
CONT No
GWP No 5116-07-00

HEBERT CREEK BRIDGE REPLACEMENT
HIGHWAY 539
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS

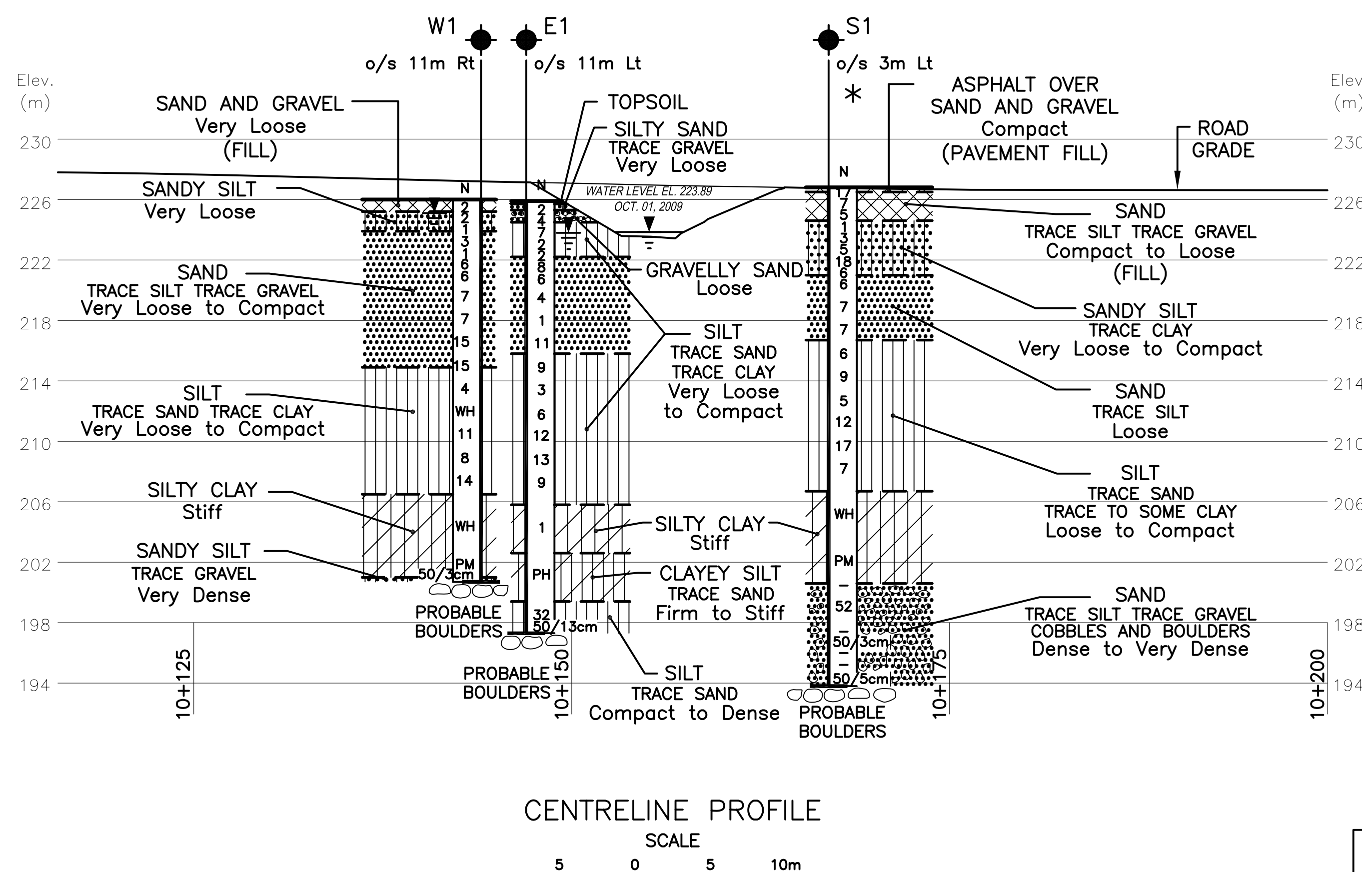




LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- W L at time of investigation: October 2009
- Water level not established
- Head ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	STATION	OFFSET
E1	225.9	10+147	11.0m Lt. CL
S1	226.8	10+167	3.0m Lt. CL
W1	226.0	10+144	11.0m Rt. CL



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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 41I-258

HWY No 69			DIST 54
SUBM'D GD	CHECKED GD	DATE APR. 30, 2010	SITE 43-017
DRAWN NA	CHECKED CN	APPROVED BRG	DWG 1

REF No. STANTEC Drawing: 165000723-HebertCreek-GA.dwg; dated November 2009

NOTE:

1. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

APPENDIX A

Site Photographs

Photograph 1: Looking northwest to Hebert Creek bridge. (October 2009)

Photograph 2: Facing northwest – southwest side of southeast approach. West detour would be on left side of photograph. (October 2009)

Photograph 3: Facing northwest – northeast side of southeast approach (take note of flooded drainage ditch). East detour would be at centre of photograph. (October 2009)

Photograph 4: Looking southeast from northwest end of bridge deck. East detour would be on left side of photograph. (October 2009)

Photograph 5: Looking southeast from northwest end of bridge deck. West detour would be at centre of photograph. (October 2009)

Photograph 6: Facing southeast – southwest side of northwest approach. Note earth cut on right side of photograph. (October 2009)

Photograph 7: Looking northeast from southeast end of bridge deck. (October 2009)

Photograph 8: Looking south from location of borehole E1. (October 2009)