

**FOUNDATION INVESTIGATION REPORT
PROPOSED CULVERT INSTALLATIONS UNDER
HIGHWAY 401 AND BURNHAM STREET
PORT HOPE TO COBOURG, ONTARIO
G.W.P. 4073-01-00**

Prepared For:

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DRAWING No.

BOREHOLE LOCATION PLANS AND SUBSURFACE PROFILES

1, 2 AND 3

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1. INTRODUCTION

Shaheen & Peaker Limited (S&P) was retained by UMA Engineering Limited (UMA) to conduct a foundation investigation at the sites of proposed culvert installations under Highway 401 and Burnham Street, from Port Hope to Cobourg, Ontario (see the Key Plan, in Drawing Nos. 1, 2 and 3). This project is part of the proposed six-laning of Highway 401 from County Road 28 to 700 m east of Burnham Street, for MTO Eastern Region under Agreement No. 4005-A-000257.

The purpose of the investigation was to obtain the subsurface conditions at the site and, based on the findings, to make recommendations from a geotechnical viewpoint for the design and construction of the culverts.

The findings of this geotechnical investigation, together with our recommendations and comments, are presented in this report. The anticipated construction conditions are also discussed, but only to the extent that they may influence design decisions. The construction methods discussed express our opinion only and are not intended to direct the contractors how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all factors that may have an effect upon construction.

2. SITE DESCRIPTION AND PHYSIOGRAPHY

As part of the Highway 401 improvements from County Road 28 to Burnham Street in Port Hope and Cobourg, three new culverts are proposed under Highway 401 and Burnham Street. The first culvert (designated as Culvert 1 for the purposes of this report) is proposed to be constructed under the embankment of Highway 401, at about Station 12+880 just west of Theatre Road. This culvert is designed to be 600 mm in diameter and 74 m in length. Approximately 1.5 km east of the first culvert, a second culvert (designated as Culvert 2 for the purposes of this report) of 900 mm in diameter and about 71 m in length is proposed to be constructed under the Highway 401 embankment approximately at Station 14+360. Further to the east (about 2.2 km), a third culvert (designated as Culvert 3 for the purposes of this report), 900 mm in diameter and 67 m in length, is proposed to be constructed under the existing Burnham Street embankment at Station 10+047 (Burnham Street) just south of Highway 401.

The existing embankments at the proposed culvert locations vary in height from about 1.5 to more than 6 m, and the side slopes and ditch line areas are covered with grass and occasional small shrubs.

The study area is located within the Physiographic Region known as the 'Iroquois Plain'. The plain generally consists of drumlins and sand plains (Ref. Chapman and Putnam, 1984). Glacio-lacustrine lake plain deposits of silt and clay with gently rolling terrain characterize the soils of the immediate area. Characteristics of the soil types are imperfect drainage, smooth to gently sloping topography, free of stones. The upper lacustrine silt and clay were deposited on clayey glacial tills underlain by sandy tills.

The lowest bedrock in the general area consists of Precambrian rock, with upper layers of limestone. These limestone layers are made up of the Trenton Group bedrock formations and were deposited during the Middle Ordovician Period, during the Paleozoic seas, some 480 million years ago.

3. INVESTIGATION PROCEDURES

The fieldwork for this project was performed in two stages. The first stage was carried out during the period of March 18 to 24, 2004 to investigate Culverts 1 and 3 which were the only two culverts proposed initially. The fieldwork at this stage consisted of drilling and sampling a total of 6 boreholes. Later on, an additional culvert (Culvert 2) was included in the scope of work of this project, and a second stage was conducted on December 9, 2004 to investigate the additional culvert and consisted of drilling and sampling a total of 2 boreholes. The fieldwork for this investigation consists of the following:

Culvert 1 – 3 boreholes (Boreholes JB1, JB2 and JB3);
Culvert 2 – 2 boreholes (Boreholes JB7 and JB8);
Culvert 3 – 3 boreholes (Boreholes JB4, JB5 and JB6).

As for the proposed culverts at Highway 401 (i.e. Culverts 1 and 2), the boreholes were drilled on top of the embankment at the outside shoulder and/or the inside shoulder locations. At Culvert 3 location, the boreholes were drilled at the bottom of the embankment near the proposed culvert ends and at the crest of the Burnham Street embankment.

The plan locations of the boreholes and the soil strata at the three proposed culvert locations are shown on Drawing Nos. 1, 2 and 3.

The boreholes were advanced using truck and track mounted drill rigs owned and operated by Eastern Soil Investigation Limited, under the full time supervision of geotechnical engineers from S&P.

The depth of the boreholes ranged from 6.6 m to 12.2 m. Sampling in the boreholes was conducted at frequent intervals of depth by the Standard Penetration Test (SPT) method, as specified in ASTM D1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split-barrel (split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposit. Where the consistency of the soil permitted in the cohesive deposits, the undrained shear strength of the soil was measured in-situ by means of field vane tests using an MTO-Type Field Vane.

Water level observations in the open boreholes were made during drilling and at the completion of each borehole. In addition, a piezometer was installed in Borehole JB6. This piezometer enabled us to monitor groundwater levels over a prolonged period of time without interference from surface water.

At the completion of drilling, all boreholes drilled were grouted and sealed using a cement/bentonite mixture. In Borehole JB6 where a piezometer was installed, sand was used as backfill from the bottom of the open portion of the borehole to about 0.3 m above the slotted portion of the pipe and then the borehole was sealed with bentonite seal and grouted up to the ground surface. At the end of the field investigation period, the piezometer was decommissioned by grouting with a cement/bentonite mixture, in accordance with MTO procedures and MOE Regulation 903.

The borehole locations and elevations were determined by surveyors retained by UMA, who provided us with coordinates and geodetic elevations.

The results of drilling, in-situ testing and water level measurements are summarized on the Record of Borehole Sheets in Appendix A.

A laboratory testing programme, consisting of natural moisture content, Atterberg Limits tests and grain-size analyses, was performed on selected soil samples. The results of the laboratory tests are given on the appropriate Record of Borehole Sheets and also in Appendix B.

4. SUBSURFACE CONDITIONS

4.1 CULVERT 1 – STATION 12+880 (HIGHWAY 401)

At this location, the finished grade (i.e. top of pavement) of the Highway 401 Eastbound Lanes (EBL) and Westbound Lanes (WBL) is about El. 118 m, and the existing embankment is about 5 m in height.

Three boreholes were drilled for this culvert – Boreholes JB1 and JB2 were put down at the outside and inside shoulders of the existing Highway 401 EBL respectively, while Borehole JB3 was drilled at the outside shoulder of the existing WBL. These three boreholes were drilled to depths of 9.6 and 9.9 m below the ground surface. The boreholes indicate, in general, below some pavement and embankment fills which extends to depths of about 3.5 to 4.4 m below the existing road grade, the presence of a clayey silt till deposit.

The location plan of the boreholes and the stratigraphic soil profile along the proposed culvert alignment are presented in Drawing No. 1. Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The individual strata encountered in the boreholes are briefly described in the following paragraphs.

4.1.1 FILL

4.1.1.1 PAVEMENT FILL

The boreholes were drilled on top of the paved shoulders, and they encountered about 150 to 170 mm asphaltic concrete underlain by a granular pavement fill which consists of mainly sand and gravel extending to depths ranging from 1.5 to 2.1 m below the existing grade or to elevations between 115.8 and 116.3 m. In Borehole JB2, this granular fill was found to contain some sandy silt pockets.

4.1.1.2 EMBANKMENT FILL

4.1.1.2.1 Silty Sand/Sandy Silt Embankment Fill

The pavement fill in Boreholes JB1 and JB2 was underlain by silty sand/sandy silt fill materials which extend to depths of 3.0 and 4.3 m or to El. 114.8 and 113.5 m, respectively. The silty sand fill in Borehole JB1 was found to contain some gravel, while in Borehole JB2 some clay was encountered in the sandy silt fill material. The grain-size distribution of a sample from the sandy silt fill material in Borehole JB2 is given in Figure 1-1 in Appendix B, which indicates 0% gravel, 42% sand, 40% silt and 18% clay size particles. Standard Penetration tests performed in these silty sand to sandy silt fill materials yielded N-values of 6 to 18 blows/0.3 m but generally between 10 and 15, indicating that the fill has probably received some limited systematic compaction, but not uniformly, when it was first placed.

4.1.1.2.2 Clayey Silt Embankment Fill

Below the silty sand fill in Borehole JB1 and the granular pavement fill in Borehole JB3, a clayey silt fill deposit was contacted at depths of 3.0 and 2.1 m respectively. This clayey silt fill material was found to extend to depths/elevations of 4.4 m/El. 113.4 m in Borehole JB1 and 3.5 m/114.4 m in Borehole JB3, overlying the clayey silt till deposit. The clayey silt fill in Borehole JB3 was found to contain occasional gravel, while in Borehole JB1 the clayey silt

fill deposit was found to be rather sandy. One grain-size distribution test was performed on a sample from the clayey silt fill deposit in Borehole JB1, and the results are presented in Figure 1-2 in Appendix B. The results show 0% gravel, 45% sand, 32% silt and 23% clay size particles. Standard Penetration tests performed in the clayey silt fill yielded N-values of 12 to 17 blows/0.3 m indicating that the fill has probably received some systematic compaction when it was first placed.

4.1.2 CLAYEY SILT TILL

Below the embankment fill materials, all three boreholes contacted a major clayey silt till deposit at depths/elevations ranging from 3.5 m/EI. 114.4 m in Borehole JB3 to 4.4 m/113.4 m in Borehole JB1. The clayey silt till deposit was found to extend to at least the termination depths of the boreholes (9.6 m in Boreholes JB1 and JB3, and 9.9 m in Borehole JB2) and probably deeper.

This clayey silt till consists of a heterogeneous mixture of clayey silt with sand and traces of gravel. The presence of cobbles and boulders can always be expected in the glacial till deposits, owing to their mode of deposition. The results of grain-size analyses performed on three selected samples are presented in Figure 1-3 in Appendix B. These indicate the following particle size distribution.

Gravel:	0 – 3%
Sand:	33 – 44%
Silt:	36 – 48%
Clay:	19 – 25%

Atterberg limits tests carried out in the laboratory on one sample from the deposit yielded the following index values.

Liquid Limit:	28%
Plastic Limit:	21%
Plasticity Index:	7%

As presented in Figure 1-4 in Appendix B, these values indicate a clayey soil of low plasticity. The measured natural moisture contents of the sample recovered from this deposit range from 13 to 27% indicating a moist condition.

In these boreholes, the upper 3.0 to 3.5 m (generally above EI. 110 m) of the clayey silt till is relatively more competent with measured N-values of between 11 and 48 blows/0.3 m. Below this upper crust zone, the measured N-values range from 5 to 7 blows/0.3 m. The measured undrained shear strengths (by means of a field vane) in the lower weaker zone range from 25 to 40 kPa. The measured N-values indicate that the consistency of the upper clayey silt till deposit can be described as stiff to hard, while below about EI. 110 m,

the clayey silt till deposit becomes weaker with a firm consistency, based on the recorded N-values and field vane test results.

4.1.3 GROUNDWATER CONDITIONS

Groundwater levels in the boreholes were observed in the open boreholes during the drilling and at the completion of each borehole. The recorded values are detailed on the individual Record of Borehole Sheets presented in Appendix A.

Based on these observations and the change of the soil colour from brown to grey, the permanent groundwater table at the site can be expected to be at about El. 113.0 ± 1.0 m. In addition, a perched water condition can occur due to the accumulation of water in the pavement and/or embankment fill overlying the clayey silt till. It should be pointed out that the groundwater table is subject to seasonal fluctuations and in response to major weather events.

4.2 CULVERT 2 – STATION 14+360 (HIGHWAY 401)

The second culvert is located at Station 14+360 about 2.2 km west of Burnham Street Interchange. At this location, the finished grade (i.e. top of pavement) of the Highway 401 EBL and WBL is about El. 113 m, and the existing embankment is about 1.5 m in height.

Two boreholes were drilled for this proposed culvert – Boreholes JB7 and JB8 were put down at the outside gravel shoulders of the existing Highway 401 EBL and WBL, respectively. The boreholes were drilled to depths of 9.3 and 7.9 m below the ground surface. These boreholes indicate, in general, below some pavement and embankment fills which extend to depths of 2.1 and 1.7 m below the existing road grade, the presence of a generally stiff to hard silty clay deposit which is underlain by a clayey silt to silty clay till deposit at depths of 5.2 and 4.4 m, respectively. The clayey silt to silty clay till deposit is further underlain by a generally cohesionless silty sand to sandy silt till deposit located below the depths of 6.4 and 8.9 m below the ground surface.

The location plan of the boreholes and the stratigraphic soil profile along the proposed culvert alignment are presented in Drawing No. 2. Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The individual strata encountered in the boreholes are briefly described in the following paragraphs.

4.2.1 FILL

4.2.1.1 GRANULAR PAVEMENT FILL

Boreholes JB7 and JB8 were drilled on the gravel shoulders from the top of the existing embankment, and both boreholes contacted a 0.6 m thick sand and gravel layer.

4.2.1.2 EMBANKMENT FILL – CLAYEY SILT TO SILT CLAY FILL

The granular pavement fill in these boreholes is underlain by a clayey silt to silty clay fill which extends to 2.1 m below the ground surface or to El. 110.9 m in Borehole JB7 and to 1.7 m below the ground surface or to El. 111.4 m in Borehole JB8. This clayey silt to silty clay fill is basically a cohesive deposit, and based on the measured 'N' values which range from 5 to 7 blows/0.3 m, the clayey fill is considered to have received little or no systematic compaction when it was first placed.

The measured natural moisture contents of samples recovered from this clayey embankment fill material range from 12 to 22% indicating a moist condition.

4.2.2 SILTY CLAY

Below the embankment fill materials, Boreholes JB7 and JB8 contacted a silty clay deposit which extends to depths/elevations of 5.2 m/El. 107.8 m and 4.4 m/El. 108.7 m, respectively.

The silty clay deposit was found to contain traces to some sand and occasional gravel. The results of grain-size analyses performed on two selected samples are presented in Figure 2-1 in Appendix B. These indicate the following particle size distribution.

Gravel:	0 – 1%
Sand:	13%
Silt:	44%
Clay:	42 – 43%

Atterberg limits tests carried out in the laboratory on two samples from the deposit yielded the following index values.

Liquid Limit:	44 – 52%
Plastic Limit:	18%
Plasticity Index:	26 – 34%

As presented in Figure 2-2 in Appendix B, these values are characteristic of clayey soils of generally medium to high plasticity. The measured natural moisture contents range from 19 to 37%, giving liquidity indices of 0.15 to 0.56. These values indicate that this silty clay deposit is probably overconsolidated.

Standard Penetration tests performed in this clayey deposit yielded N-values ranging from 5 to 57 blows/0.3 m. In Borehole JB7 where N-values of 5 and 7 were recorded in the upper 1.5 m zone (above El. 109.5 m), the undrained in-situ shear strengths measured by field vane tests values ranged from 46 to in excess of 100 kPa. Based on these, the consistency of the silty clay deposit in the upper zones of Borehole JB7 can be described as generally

firm to stiff. Below the upper weaker zone in Borehole JB7 and throughout Borehole JB8, the consistency of the silty clay deposit can be described as very stiff to hard.

4.2.3 CLAYEY SILT TO SILTY CLAY TILL

Underlying the silty clay deposit, a clayey silt to silty clay till deposit was contacted in Borehole JB8, while in Borehole JB7 a relatively more clayey till deposit – silty clay till was contacted. This clayey silt to silty clay till deposit was found to contain some sandy silt till layers and extended to 8.9 m/EI. 104.1 m in Borehole JB7 and 6.4 m/EI. 106.7 m in Borehole JB8.

The glacial till deposit consists of a heterogeneous mixture of clayey silt to silty clay with some sand and gravel. The presence of cobbles and boulders can always be expected in the glacial till deposits, owing to their mode of deposition.

The measured natural moisture contents of samples recovered from this cohesive deposit range between 7 and 20%. Standard Penetration tests performed in these deposits gave N-values ranging from 9 to 48 blows for 0.3 m penetration, indicating a stiff to hard consistency.

4.2.4 SILTY SAND TO SANDY SILT TILL

The clayey till deposit is further underlain by a silty sand to sand silt till deposit which extends to the remaining depths of the exploration – 9.3 m in Borehole JB7 and 7.9 m in Borehole JB8, and possibly deeper.

Due to their mode of deposition, the presence of cobbles and boulders can always be expected in the glacial till deposits.

The measured natural moisture contents of samples recovered from this basically granular (i.e. non-cohesive) deposit range between 5 and 8% indicating a damp to moist condition. Standard Penetration tests performed in this deposit gave N-values in excess of 50 blows for 0.3 m penetration indicating a very dense relative density.

4.2.5 GROUNDWATER CONDITIONS

Groundwater levels in the boreholes were observed in the open boreholes during the drilling and at the completion of each borehole. The recorded values are detailed on the individual Record of Borehole Sheets presented in Appendix A.

Based on these observations and the change of the soil colour from brown to grey, the permanent groundwater table at the site can be expected to be at about EI. 108.0 ± 1.0 m.

The high water level recorded in Borehole JB7 (i.e. at 1.5 m below the ground surface or at El. 111.5 m) is likely to be due to a perched water condition owing to the accumulation of surface water in the fill overlying the practically impervious silty clay deposit. This indicates that a similar perched water condition can also occur at Borehole JB8 location and across the embankment at this site. It should be pointed out that the groundwater table is subject to seasonal fluctuations and in response to major weather events.

4.3 CULVERT 3 – STATION 10+047 (BURNHAM STREET)

The third culvert is located at Burnham Street Station 10+047 just south of the Highway 401 underpass bridge. At this location, the finished grade (i.e. top of pavement) of Burnham Street is about El. 109.7 m, and the height of the existing embankment is about 5 m on the west side and more than 6 m on the east side of Burnham Street.

Three boreholes were drilled for this culvert – Boreholes JB4 and JB6 were put down near the proposed culvert ends on the west and east sides of Burnham Street, respectively, beyond the toe of the embankment, while Borehole JB5 was drilled from the west shoulder of the Burnham Street Embankment. The boreholes were extended to depths ranging from 6.6 to 12.2 m below the ground surface. These boreholes indicate, in general, below some topsoil and embankment fill/backfill, which extend to depths between about 0.25 and 6.1 m below the existing grade, the presence of a firm to very stiff clayey silt deposit which is underlain by the compact to very dense silty sand to sandy silt till deposit at elevations ranging from El. 101.2 to 102.3 m. In Borehole JB6, which extended to a lower elevation, the silty sand to sandy silt till deposit is further underlain by a stiff to very stiff clayey silt till.

The location plan of the boreholes and the stratigraphic soil profile along the proposed culvert alignment are presented in Drawing No. 3. Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The individual strata encountered in the boreholes are briefly described in the following paragraphs.

4.3.1 FILL

4.3.1.1 PAVEMENT FILL

Borehole JB5 was drilled on top of the paved shoulder of the road, and it encountered about 130 mm asphaltic concrete underlain by an approximately 0.7 m thick granular pavement fill consisting of sand and gravel, extending to a depth of about 0.8 m below the existing grade.

4.3.1.2 EMBANKMENT FILL/BACKFILL – CLAYEY SILT FILL

The pavement fill in Borehole JB5 was underlain by a clayey silt embankment fill which extends to 5.3 m below the ground surface or to El. 104.4 m. In Borehole JB4, below a

0.18 m thick surficial topsoil layer, a clayey silt fill was contacted extending to 1.5 m below the ground surface or to El. 103.4 m. This clayey silt fill was found to contain traces of gravel and sand, and occasional topsoil pockets. The measured natural moisture contents of samples recovered from this clayey fill material range from 12 to 21%.

Measured N-values recorded in the clayey silt embankment fill in Borehole JB5 range from 15 to 32 blows/0.3 m indicating that the embankment fill has probably received systematic compaction when it was first placed. However, in Borehole JB4 Standard Penetration tests performed in the clayey silt backfill yielded N-values of 8 and 3, and these values indicate little or no compaction effort for the clayey silt backfill placed.

4.3.2 TOPSOIL

Boreholes drilled off the road (Boreholes JB4 and JB6) contacted a topsoil layer (approximately 0.18 to 0.25 m in thickness) at the ground surface. In Borehole JB5, under the embankment fill, an approximately 0.8 m thick old topsoil layer was encountered extending to a depth/elevation of 6.1 m/El. 103.6 m.

4.3.3 CLAYEY SILT

Underlying the topsoil (Borehole JB6), the old topsoil beneath the embankment fill (Borehole JB5) and the clayey silt fill (Borehole JB4), the boreholes encountered a surficial clayey silt deposit at elevations ranging from 103.2 to 103.6 m. The thickness of this surficial deposit in the boreholes was found to range from about 1.2 to 2.2 m and the deposit extended to elevations from 101.2 to 102.3 m.

This deposit was found to contain traces of gravel. Atterberg limits tests carried out in the laboratory on one sample from the deposit yielded the following index values.

Liquid Limit:	27%
Plastic Limit:	13%
Plasticity Index:	14%

As presented in Figure 3-1 in Appendix B, these values are characteristic of clayey soils of low plasticity. The natural moisture content of the sample was measured to be about 20%, giving liquidity index of about 0.5. This value indicates that this clayey silt deposit is probably overconsolidated. The remaining measured moisture content values range from 15 to 26%.

This deposit is considered to have a firm to very stiff consistency as indicated by the measured 'N' values between 7 and 30 blows/0.3 m, but typically stiff to very stiff.

4.3.4 SILTY SAND TO SANDY SILT TILL

Below the clayey silt deposit, all three boreholes encountered a silty sand to sandy silt till at elevations ranging from El. 101.2 m in Borehole JB4 to El. 102.3 m in Borehole JB5. This glacial till deposit was found to extend at least to the termination depths of 6.6 m (El. 98.3 m) in Borehole JB4 and 12.2 m (El. 97.5 m) in JB5. In Borehole JB6, at the depth of 7.5 m below the ground surface or at El. 95.9 m, a clayey silt till was contacted underlying the silty sand to sandy silt till.

This silty sand to sandy silt till consists of a heterogeneous mixture of silt and sand with traces of gravel and some clay. The presence of cobbles and boulders can always be expected in the glacial till deposits, owing to their mode of deposition. The results of the grain-size analyses performed on four selected samples from the deposit are presented in Figure 3-2 in Appendix B. These indicate the following particle size distribution.

Gravel:	2 – 15%
Sand:	37 – 50%
Silt:	19 – 35%
Clay:	12 – 20%

Atterberg limits tests carried out in the laboratory on two samples from the deposit yielded the following index values (see Figure 3-3 in Appendix B).

Liquid Limit:	16 – 18%
Plastic Limit:	12 – 13%
Plasticity Index:	3 – 6%

This deposit is a basically granular (i.e. non-cohesive deposit). As can be seen from the grain-size curves, the deposit has some clay content. This clay content imparts some plasticity to the deposit (i.e. a plasticity index of 3 – 6%). The measured natural moisture contents range from 6 to 10% indicating a damp to moist condition.

Standard Penetration tests performed in this deposit gave N-values which range from 15 to in excess of 50 blows for 0.3 m penetration, indicating a compact to very dense relative density.

4.3.5 CLAYEY SILT TILL

In Borehole JB6 (which are extended to a lower elevation in comparison with the other two boreholes), a clayey silt till deposit was contacted at a depth/elevation of 7.5 m/El. 95.9 m, underlying the silty sand to sandy silt till. Borehole JB6 was terminated in this deposit at a depth of 9.6 m below the ground surface or at El. 93.8 m.

Due to their mode of deposition, the presence of cobbles and boulders can always be expected in the glacial till deposits.

The measured natural moisture contents of samples from this cohesive deposit range from 17 to 22%. Based on the measured 'N' values between 12 and 22 blows/0.3 m, this deposit is considered to possess a stiff to very stiff consistency.

4.3.6 GROUNDWATER CONDITIONS

Groundwater levels in the boreholes were observed in the open boreholes during the drilling and at the completion of each borehole. In addition, a piezometer was installed in Boreholes JB6 to enable us to monitor the groundwater levels without interference from surface water. The recorded values are detailed on the individual Record of Borehole Sheets presented in Appendix A.

Water level in the piezometer in Borehole JB6 was measured at Elevation 98.4 m or 5.0 m below the ground surface 2 days after the installation. Based on these observations and the change of the soil colour from brown to grey, the permanent groundwater table at the site can be expected to be at about El. 102.0 \pm 1.0 m. In addition, a perched water condition is likely to occur within the embankment fill/backfill, due to the accumulation of surface water in the relatively pervious fill materials overlying the relatively impervious natural clayey silt deposit. It should be pointed out that the groundwater table is subject to seasonal fluctuations and in response to major weather events.

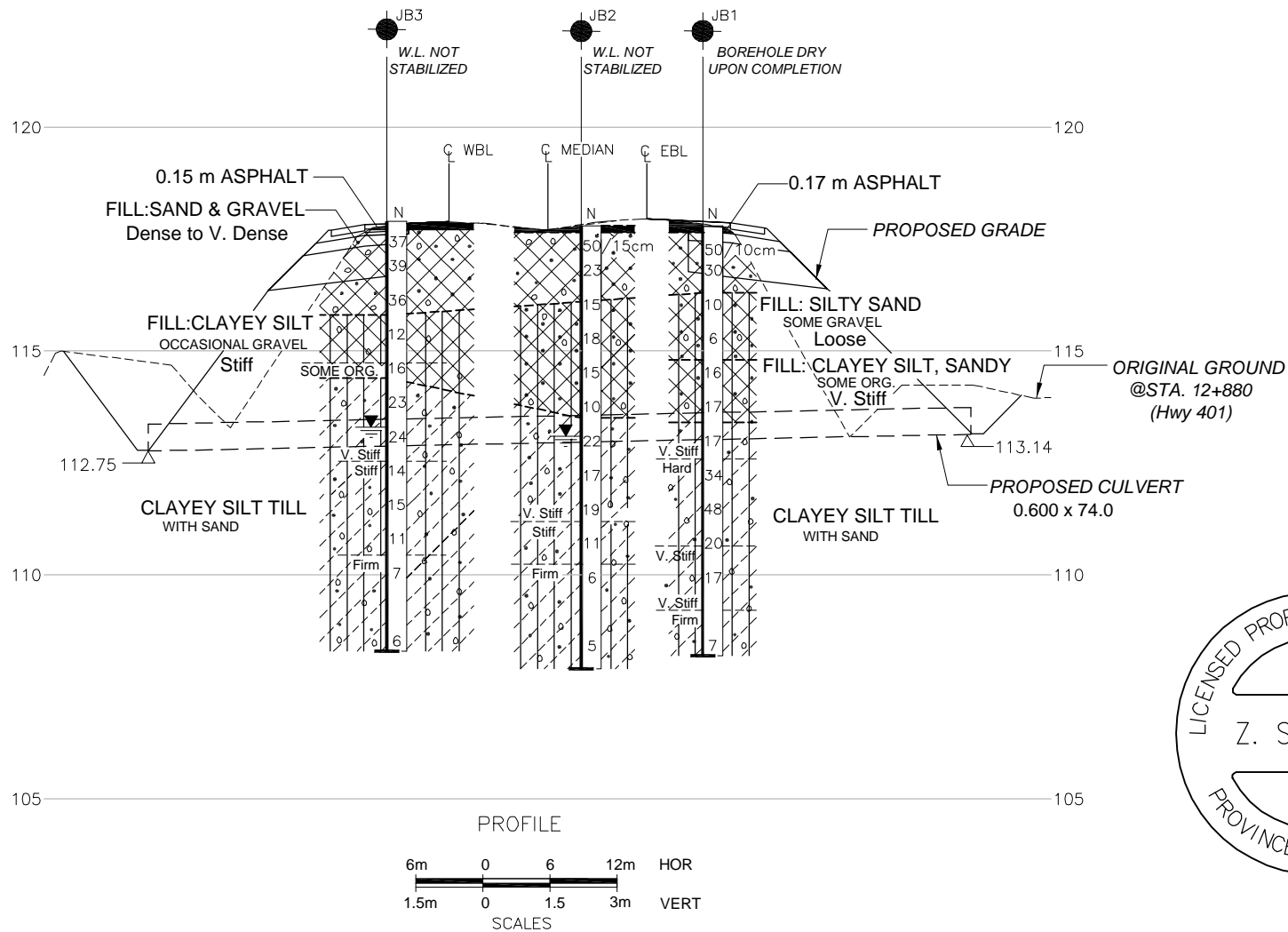
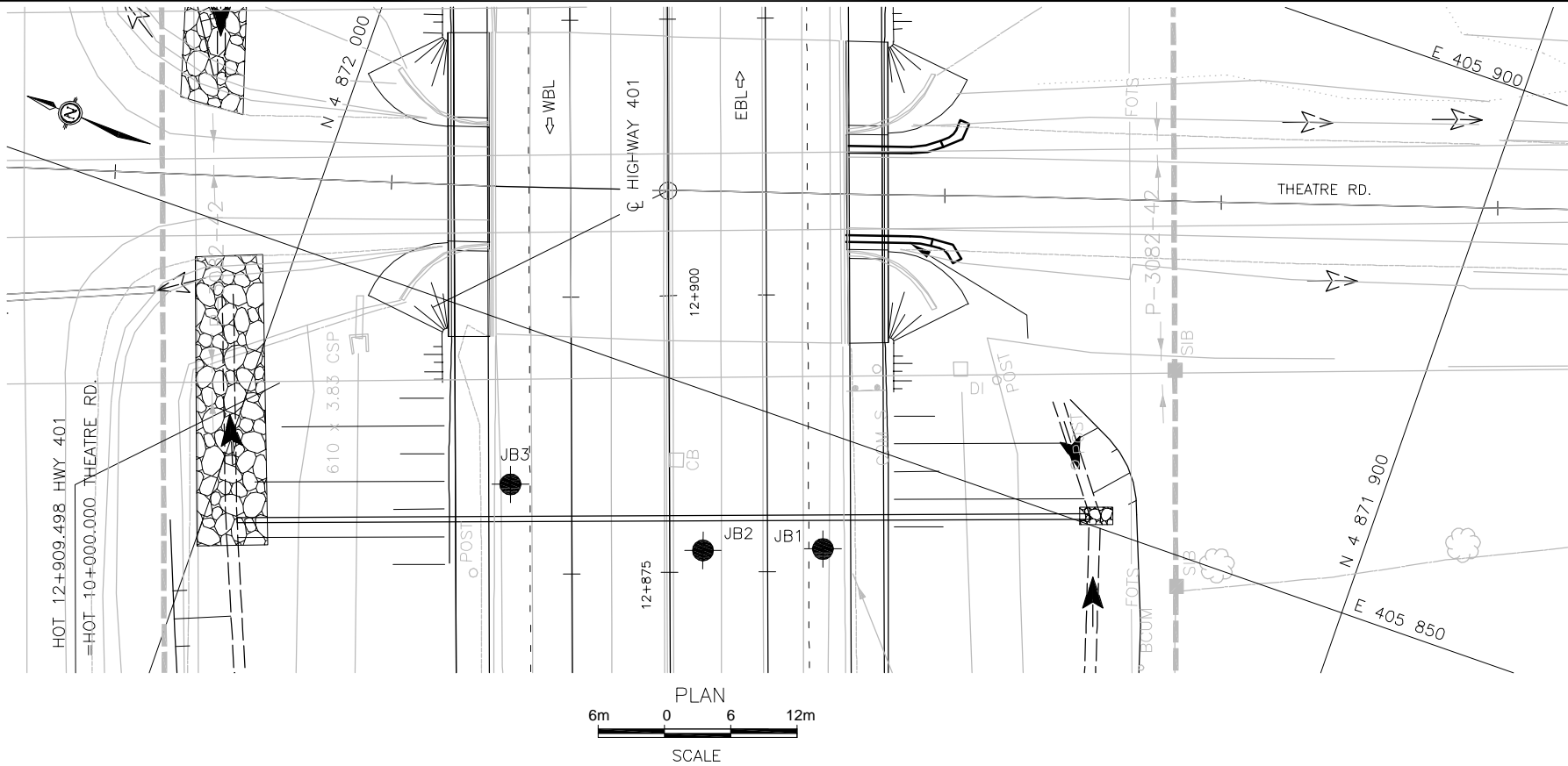
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Drawings



METRIC

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

NOTE:
• FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.
GWP: 4073-01-00

HIGHWAY 401
WEST OF THEATRE RD.
BORE HOLE LOCATIONS & SOIL STRATA



SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Bore Hole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation
Mar., 2004

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
JB 1	117.8	4 871 946.2	405 840.0
JB 2	117.8	4 871 956.4	405 836.3
JB 3	117.9	4 871 974.8	405 836.2

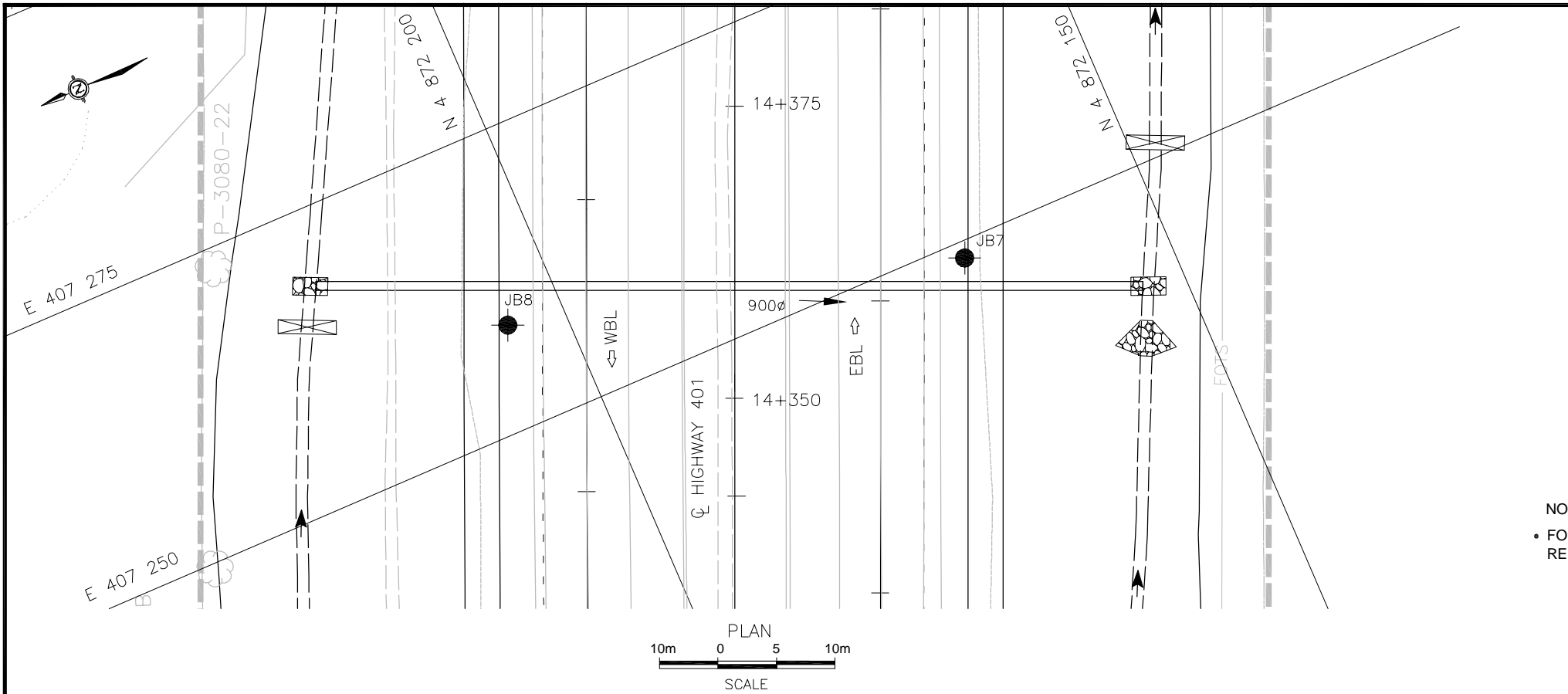
NOTE:
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION
------	------	----	-------------

Geocres No.			
HWY No. 401			DIST
SUBM'D ZO	CHECKED RM	DATE Jan, 2005	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 1

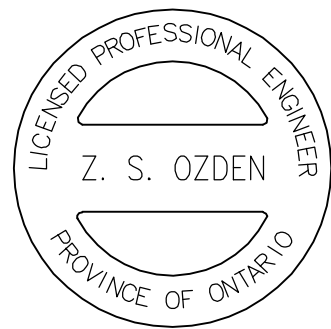
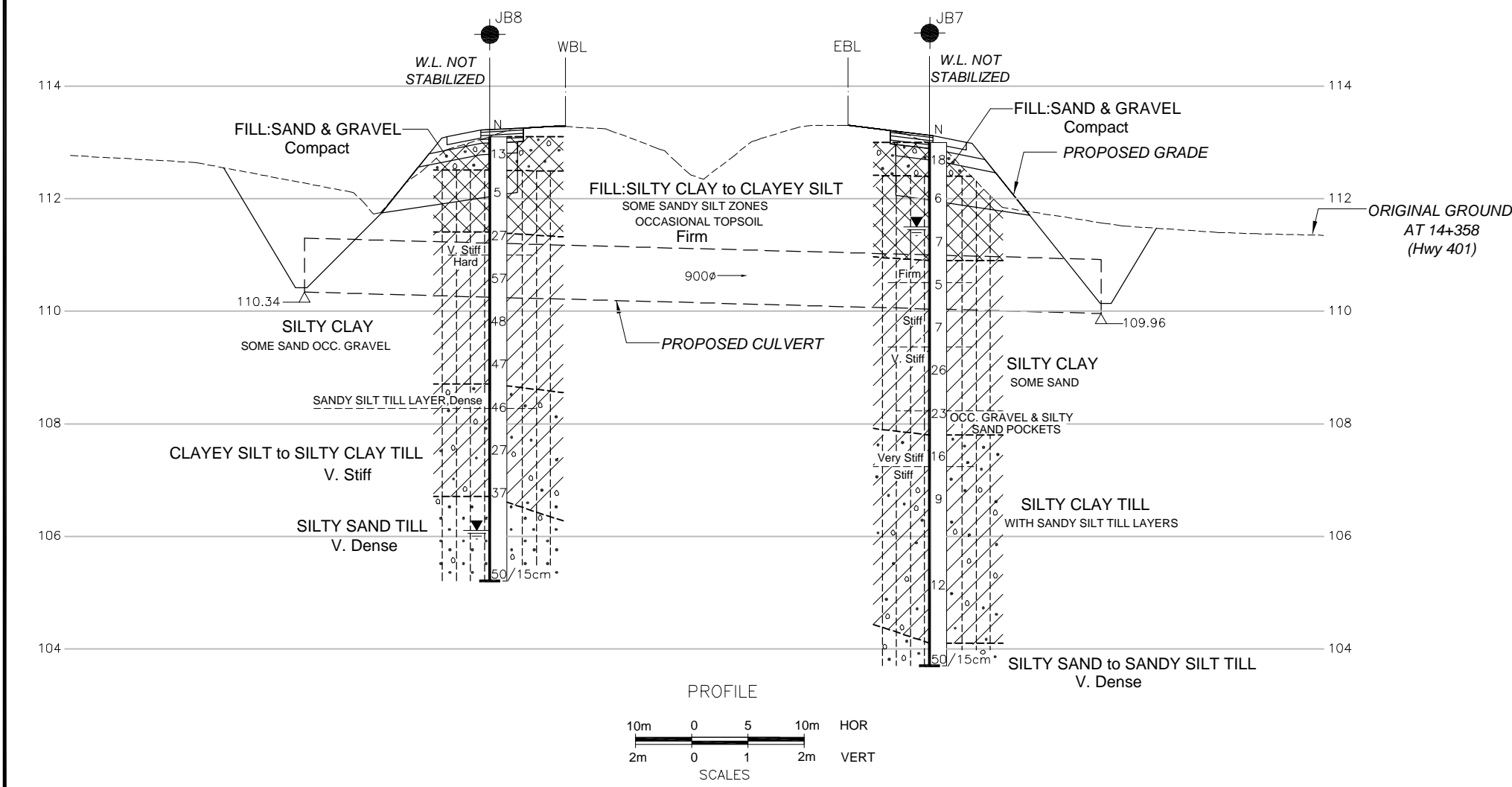




METRIC

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

NOTE:
• FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.



CONT No.
GWP: 4073-01-00

HIGHWAY 401
WEST OF BURNHAM STREET
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED

KEY PLAN
N.T.S

LEGEND

Bore Hole

N

Blows/0.3m (Std. Pen. Test, 475 J/blow)

Water Level at Time of Investigation
Dec., 2004

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
JB 7	113.0	4 872 248.6	407 248.6
JB 8	113.1	4 872 204.9	407 258.8

NOTE

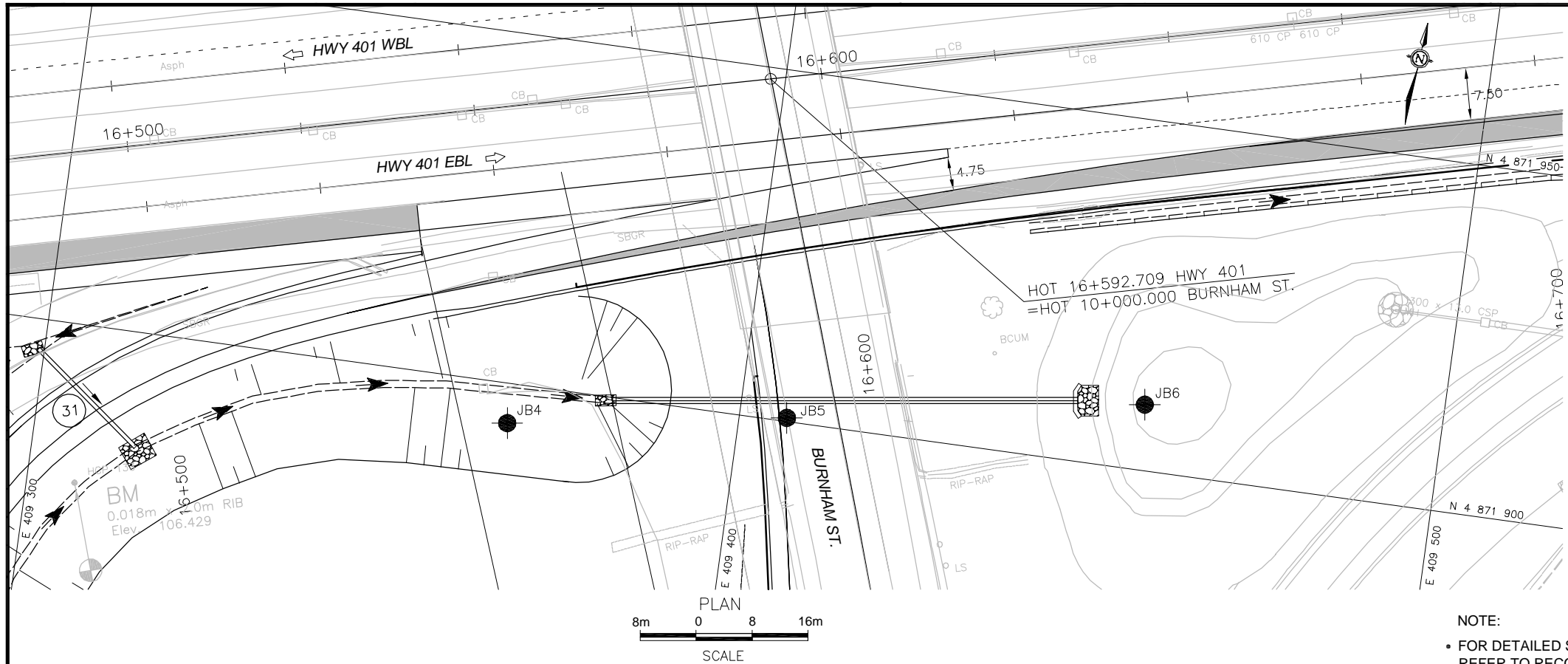
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No.

HWY No. 401	DIST		
SUBM'D ZO	CHECKED RM	DATE Jan, 2005	SITE
DRAWN JZ	CHECKED	APPROVED	DWG 2

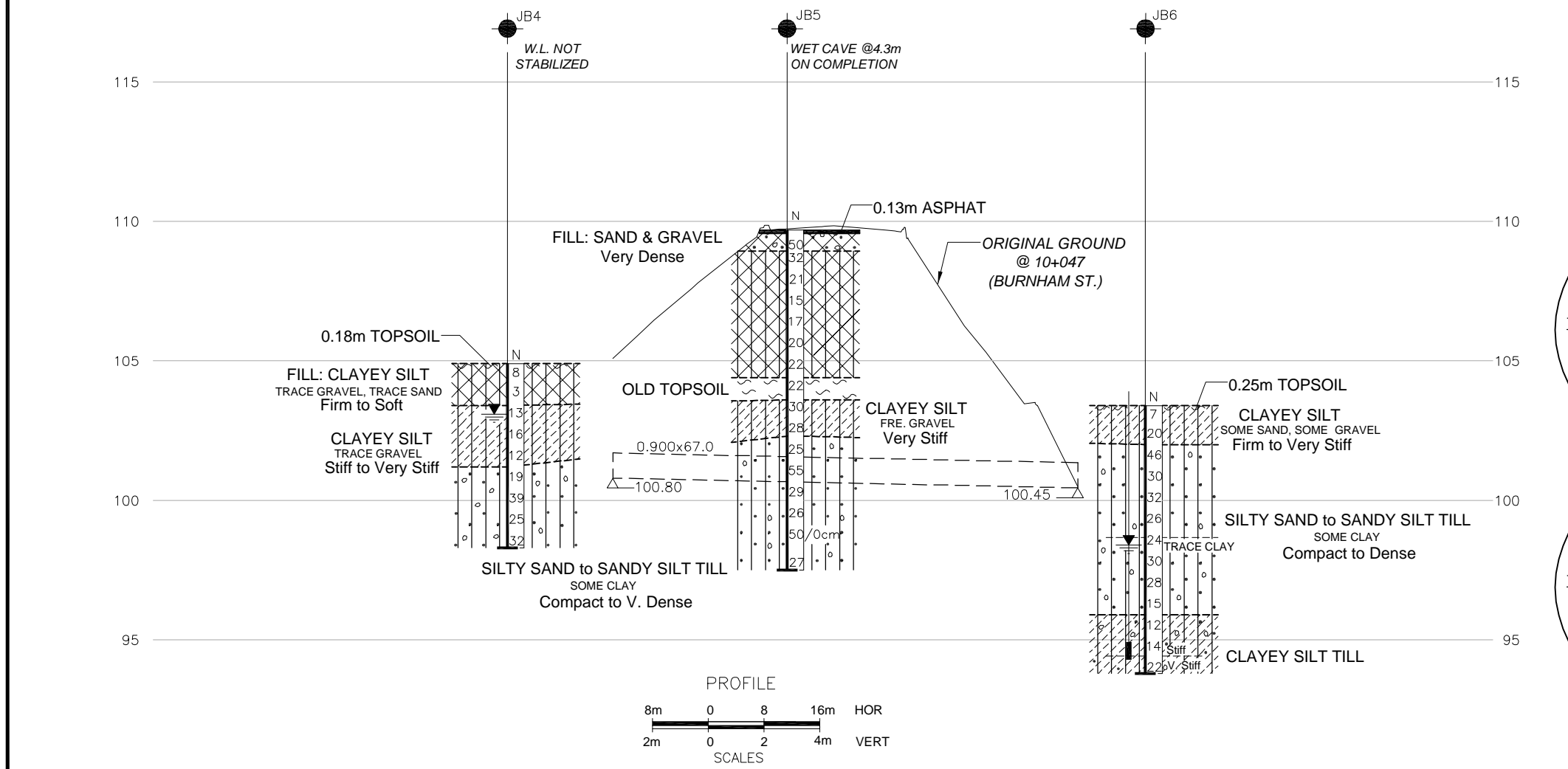


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

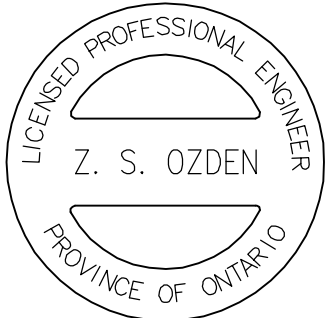
CONT No.
GWP: 4073-01-00

BURNHAM STREET
SOUTH OF HWY 401
BORE HOLE LOCATIONS & SOIL STRATA

KEY PLAN
N.T.S



NOTE:
• FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.



LEGEND

Bore Hole

Blows/0.3m (Std. Pen. Test, 475 J/blow)

Water Level at Time of Investigation
Mar., 2004

Water Level in Piezometer

Piezometer

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
JB 4	104.9	4 871 894.3	409 367.2
JB 5	109.7	4 871 900.5	409 406.8
JB 6	103.4	4 871 909.4	409 457.4

NOTE:
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

REV.	DATE	BY	DESCRIPTION

Geocres No.

HWY No. 401

SUBM'D ZO

CHECKED RM

DATE Jan, 2005

DRAWN JZ

DIST

SITE

DWG 3

Appendix A

Records of Boreholes

SPT 1098

RECORD OF BOREHOLE No JB1

1 OF 1

METRIC

GWP 4073-01-00 LOCATION Hwy 401 at Theatre Road; Sta: 12+877; 13.8 m Rt - Coords: N 4 871 946.2; E 405 840.0 ORIGINATED BY Y.L.
 DIST HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
 DATUM Geodetic DATE 3/24/2004 CHECKED BY R.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
117.8	Ground Surface													
0.0	0.17 m ASPHALT					*								
	FILL: SAND & GRAVEL brown, moist, very dense to dense		1	SS	50/10		117							
			2	SS	30									
116.3							116							
1.5	FILL: SILTY SAND some gravel brown, moist, loose		3	SS	10									
			4	SS	6									
114.8							115							
3.0	FILL: CLAYEY SILT, SANDY some organics brownish grey, moist, very stiff		5	SS	16								19.7	0 45 32 23
			6	SS	17		114						19.6	
113.4							113						19.6	0 33 48 19
4.4			7	SS	17									
	CLAYEY SILT TILL with sand, moist		8	SS	34		112							
			9	SS	48								20.9	
			10	SS	20		111						20.8	
			11	SS	17		110							
							109							
108.2			12	SS	7									sampler wet
9.6	End of Borehole. *Borehole dry (not stabilized) and open to the full depth on completion.													

SPT 1098

RECORD OF BOREHOLE No JB2

1 OF 1

METRIC

GWP 4073-01-00 LOCATION Hwy 401 at Theatre Road; Sta: 12+877; 3 m Rt - Coords: N 4 871 956.4; E 405 836.3 ORIGINATED BY Y.L.
 DIST HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
 DATUM Geodetic DATE 3/24/2004 CHECKED BY R.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
117.8	Ground Surface													
0.0	0.15 m ASPHALT													
	FILL: SAND & GRAVEL brown, moist, very dense		1	SS	50/15									
	sandy silt pockets		2	SS	23									
116.1														
1.7			3	SS	15									
	clayey silt layer		4	SS	18									
	FILL: SANDY SILT some clay brown, moist, compact		5	SS	15									
			6	SS	10									
113.5														
4.3	CLAYEY SILT TILL with sand, moist		7	SS	22									
			8	SS	17									
			9	SS	19									
	brown, very stiff		10	SS	11									
	grey, stiff		11	SS	6									
	firm													
			12	SS	5									
107.9														
9.9	End of Borehole.													
	* Water level at 4.7 m (not stabilized) and hole open to full depth on completion.													

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1098

METRIC

+³, ×³: Numbers refer to Sensitivity

SPT 1098

RECORD OF BOREHOLE No JB4

1 OF 1

METRIC

GWP 4073-01-00 LOCATION Burnham Street; Sta: 10+041; 46.7 m Rt - Coords: N 4 871 894.3; E 409 367.2 ORIGINATED BY Y.L.
DIST HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 3/18/2004 CHECKED BY R.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
104.9	Ground Surface													
0.0	0.18 m TOPSOIL FILL: CLAYEY SILT trace gravel, trace sand brown, moist, firm to soft		1	SS	8		104							frozen to 0.6 m
103.4			2	SS	3									
1.5	trace rootlets		3	SS	13		103						19.2	
	CLAYEY SILT trace gravel brown, moist, stiff to very stiff		4	SS	16		102						19.9	
101.2			5	SS	12									sampler wet
3.7	SILTY SAND to SANDY SILT TILL some clay brown, moist compact to dense		6	SS	19		101							2 46 35 17
			7	SS	39		100							5 42 33 20
	greyish		8	SS	25		99							
98.3			9	SS	32									
6.6	End of Borehole. * Water level at 1.8 m (not stabilized) and hole open to full depth on completion.													

SPT 1098

RECORD OF BOREHOLE No JB5

1 OF 1

METRIC

GWP 4073-01-00 LOCATION Burnham Street; Sta: 10+048; 7.2 m Rt - Coords: N 4 871 900.5; E 409 406.8 ORIGINATED BY Y.L.
 DIST HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
 DATUM Geodetic DATE 3/24/2004 CHECKED BY R.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
109.7	Ground Surface													
0.0	0.13 m ASPHALT													
108.9	FILL: SAND & GRAVEL brown, moist, very dense		1	SS	50		109							
0.8			2	SS	32									
			3	SS	21		108							
	FILL: CLAYEY SILT occasional gravel brown, moist, very stiff		4	SS	15		107						20.4	
			5	SS	17									
	occasional topsoil pockets		6	SS	20	*	106						19.0	
			7	SS	22		105							
104.4														
5.3	OLD TOPSOIL trace rootlets brown, moist		8	SS	22		104							
103.6														
6.1	CLAYEY SILT frequent gravel grey, moist, very stiff		9	SS	30		103						19.6	
			10	SS	28									
102.3							102							
7.4			11	SS	25									
			12	SS	55		101							sampler wet
	SILTY SAND to SANDY SILT TILL some clay moist, compact to very dense		13	SS	29									
			14	SS	26		100						22.2	10 37 33 20
			15	SS	50/0		99							
			16	SS	27		98							rod bouncing auger sample taken
97.5														
12.2	End of Borehole. * Borehole wet cave and open to 4.3 m on completion.													

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1098

RECORD OF BOREHOLE No JB6

1 OF 1

METRIC

GWP 4073-01-00 LOCATION Burnham Street; Sta: 10+056; 43.6 m Lt - Coords: N 4 871 909.4; E 409 457.4 ORIGINATED BY Y.L.
DIST HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
DATUM Geodetic DATE 3/18/2004 CHECKED BY R.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
103.4	Ground Surface																
0.0	0.25 m TOPSOIL		1	SS	7		103							o			
	CLAYEY SILT some sand, some gravel brown, moist firm to very stiff		2	SS	20		102							o			
102.0																	
1.4			3	SS	46		101							o			
	SILTY SAND to SANDY SILT TILL some clay brownish grey to grey moist, compact to dense		4	SS	30		100							o			
			5	SS	32		99							o			
			6	SS	26		98							o			
	trace clay		7	SS	24		97							o			
			8	SS	30		96							o			
			9	SS	28		95							o			
			10	SS	15		94							o			
95.9																	
7.5			11	SS	12		93							o			
	CLAYEY SILT TILL grey, moist		12	SS	14		92							o			
							91										
			13	SS	22		90							o			
93.8																	
9.6	End of Borehole. Piezometer installed to 9.1 m. *Water level on: Mar.18, 2004 - 8.1 m (El. 95.3 m) Mar.19, 2004 - 5.0 m (El. 98.4 m)																

SPT 1098

RECORD OF BOREHOLE No JB7

1 OF 1

METRIC

GWP 4073-01-00 LOCATION Hwy 401 at Burnham Street; Sta: 14+362; 19.7 m Rt - Coords: N 4 872 248.6; E 407 248.6 ORIGINATED BY Y.L.
 DIST HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
 DATUM Geodetic DATE 12/9/2004 CHECKED BY R.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE					PLASTIC LIMIT W _P NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L
113.0	Ground Surface					* ▽	113						
0.0	FILL: SAND & GRAVEL brown, moist, compact		1	SS	18								
112.4							112						
0.6	FILL: CLAYEY SILT to SILTY CLAY some sandy silt zones mixed with some topsoil, trace rootlets brown to dark brown, moist, firm		2	SS	6								
			3	SS	7		111						
110.9													
2.1		firm	4	SS	5								
		stiff	5	SS	7		110						
		very stiff	6	SS	26		109						
	SILTY CLAY some sand brownish grey, moist		7	SS	23		108						
	occasional gravel & silty sand pockets												
107.8													
5.2		moist, very stiff	8	SS	16	107							
		moist to wet, stiff	9	SS	9	106							
	SILTY CLAY TILL with sandy silt till layers brownish grey		10	SS	12	105							
104.1													
8.9	SILTY SAND to SANDY SILT TILL grey, moist, very dense		11	SS	50/15	104						hard drilling from 8.9 m	
103.7													
9.3	End of Borehole.												
	* Water level at 1.5 m (not stabilized) and hole open to full depth on completion.												

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT 1098

RECORD OF BOREHOLE No JB8

1 OF 1

METRIC

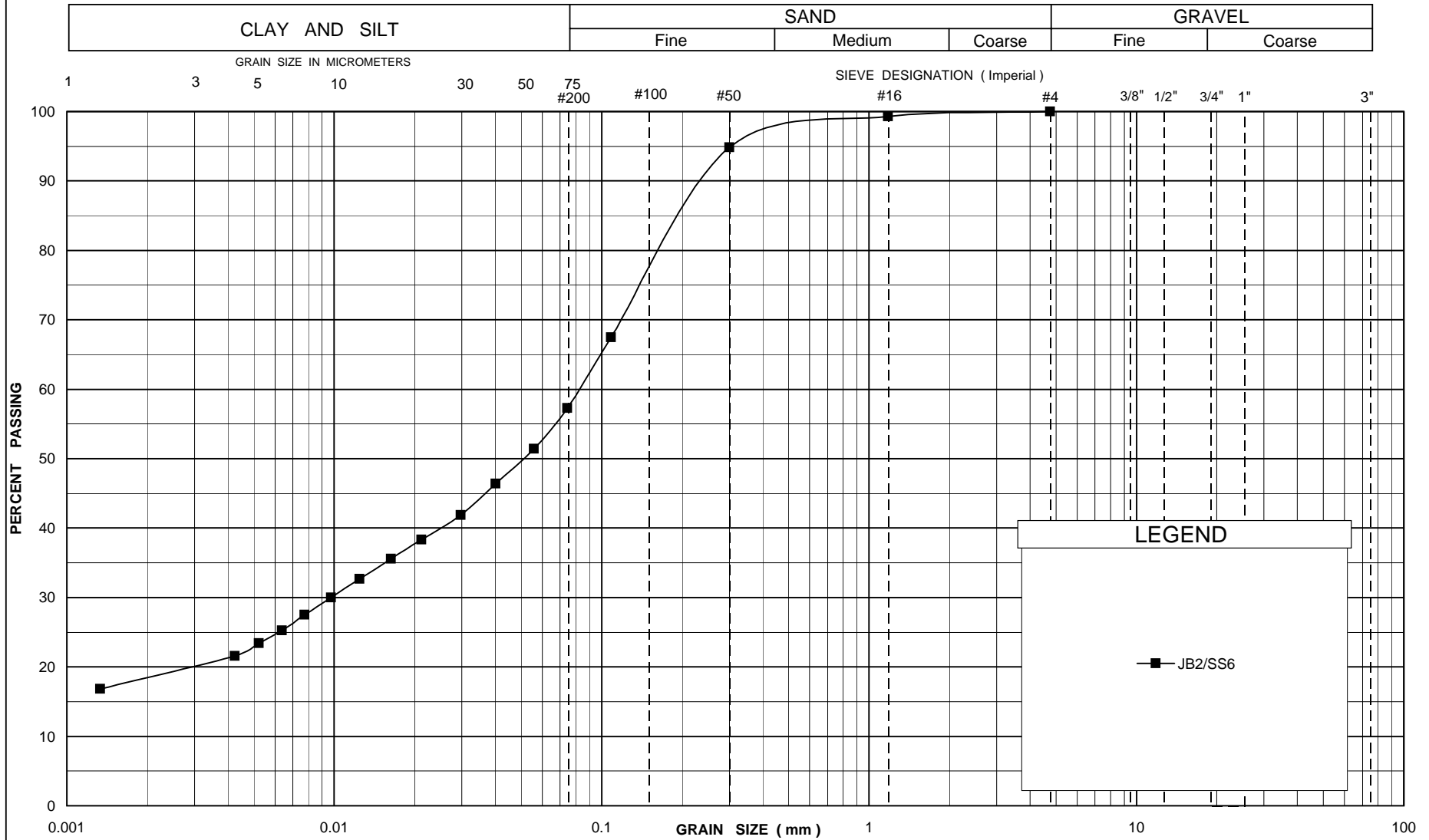
GWP 4073-01-00 LOCATION Hwy 401 at Burnham Street; Sta: 14+356; 19.4 m Lt - Coords: N 4 872 204.9; E 407 258.8 ORIGINATED BY Y.L.
 DIST HWY 401 BOREHOLE TYPE Solid Stem Augers COMPILED BY J.Z.
 DATUM Geodetic DATE 12/9/2004 CHECKED BY R.M.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
113.1	Ground Surface						113							
0.0	FILL: SAND & GRAVEL brown, moist, compact		1	SS	13		113							
112.5														
0.6	FILL: SILTY CLAY to CLAYEY SILT occasional topsoil brown, moist, firm		2	SS	5		112							
111.4														
1.7			3	SS	27		111							
	very stiff ----- hard													
	SILTY CLAY occasional gravel & sand pockets some sand brown, moist		4	SS	57		111							
			5	SS	48		110							
			6	SS	47		109							
108.7														
4.4	sandy silt till layer, dense -----		7	SS	46		108							
	CLAYEY SILT to SILTY CLAY TILL brownish grey, moist, very stiff		8	SS	27		108							
			9	SS	37		107							
106.7														
6.4	SILTY SAND TILL grey, moist, very dense						106							
			10	SS	50/15									
105.2														
7.9	End of Borehole. * Water level at 7.0 m (not stabilized) and hole open to full depth on completion.													

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER LIMITED

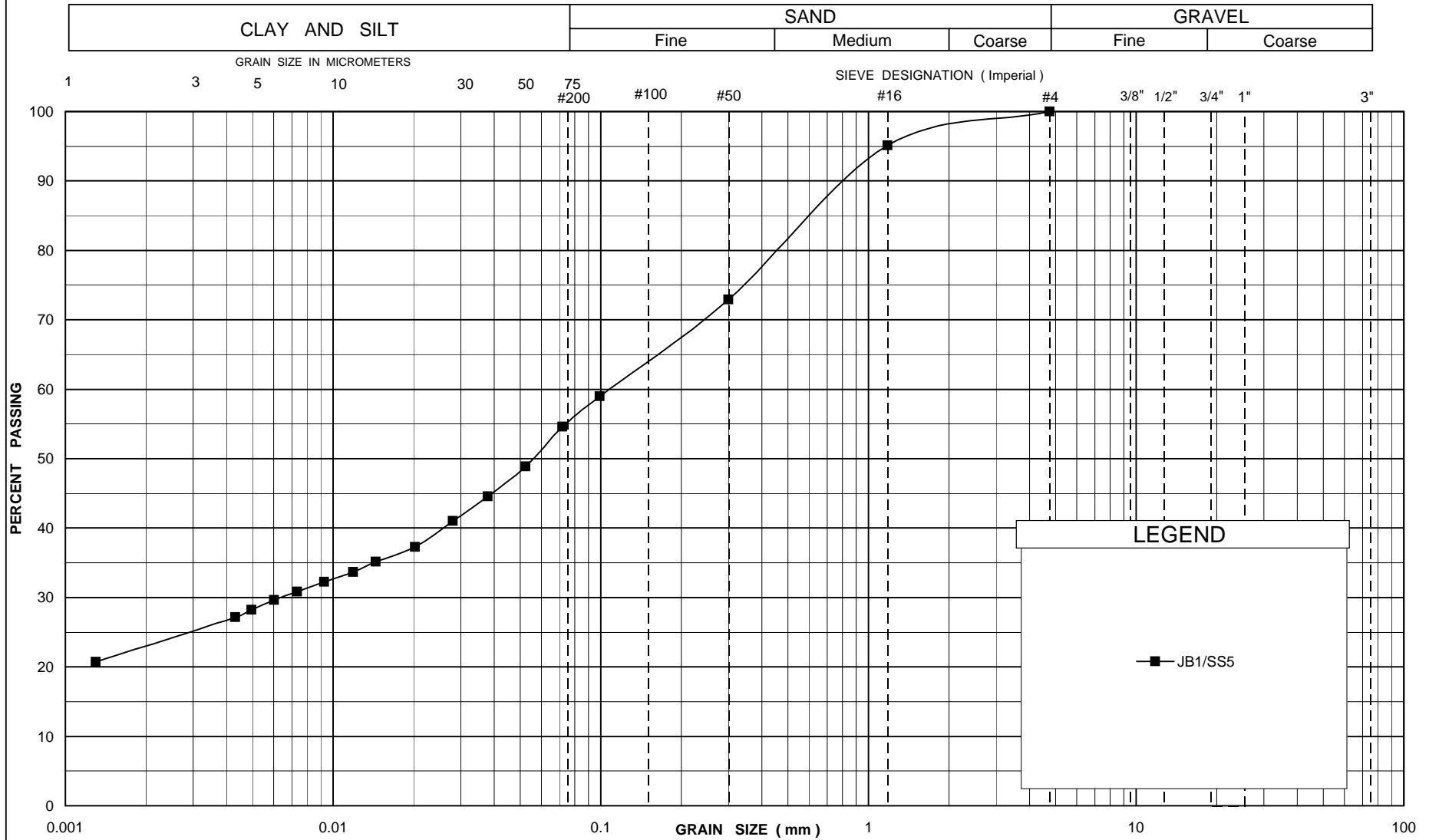
GRAIN SIZE DISTRIBUTION
FILL: SANDY SILT, some clay

FIGURE No. 1-1

REF. No. SPT 1098

DATE DECEMBER, 2005

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER LIMITED

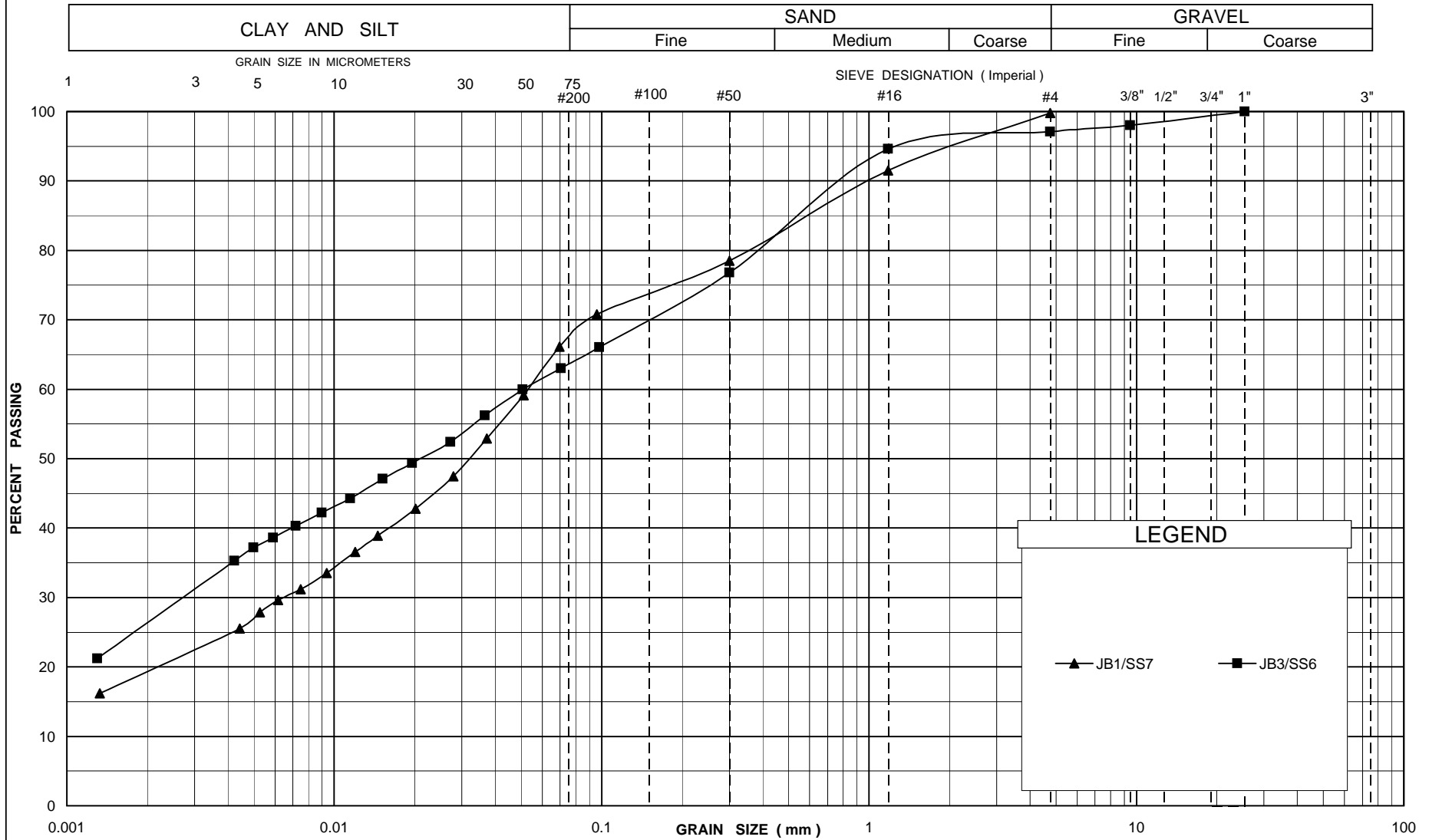
GRAIN SIZE DISTRIBUTION
FILL: CLAYEY SILT, SANDY

FIGURE No. 1-2

REF. No. SPT 1098

DATE DECEMBER, 2005

UNIFIED SOIL CLASSIFICATION SYSTEM



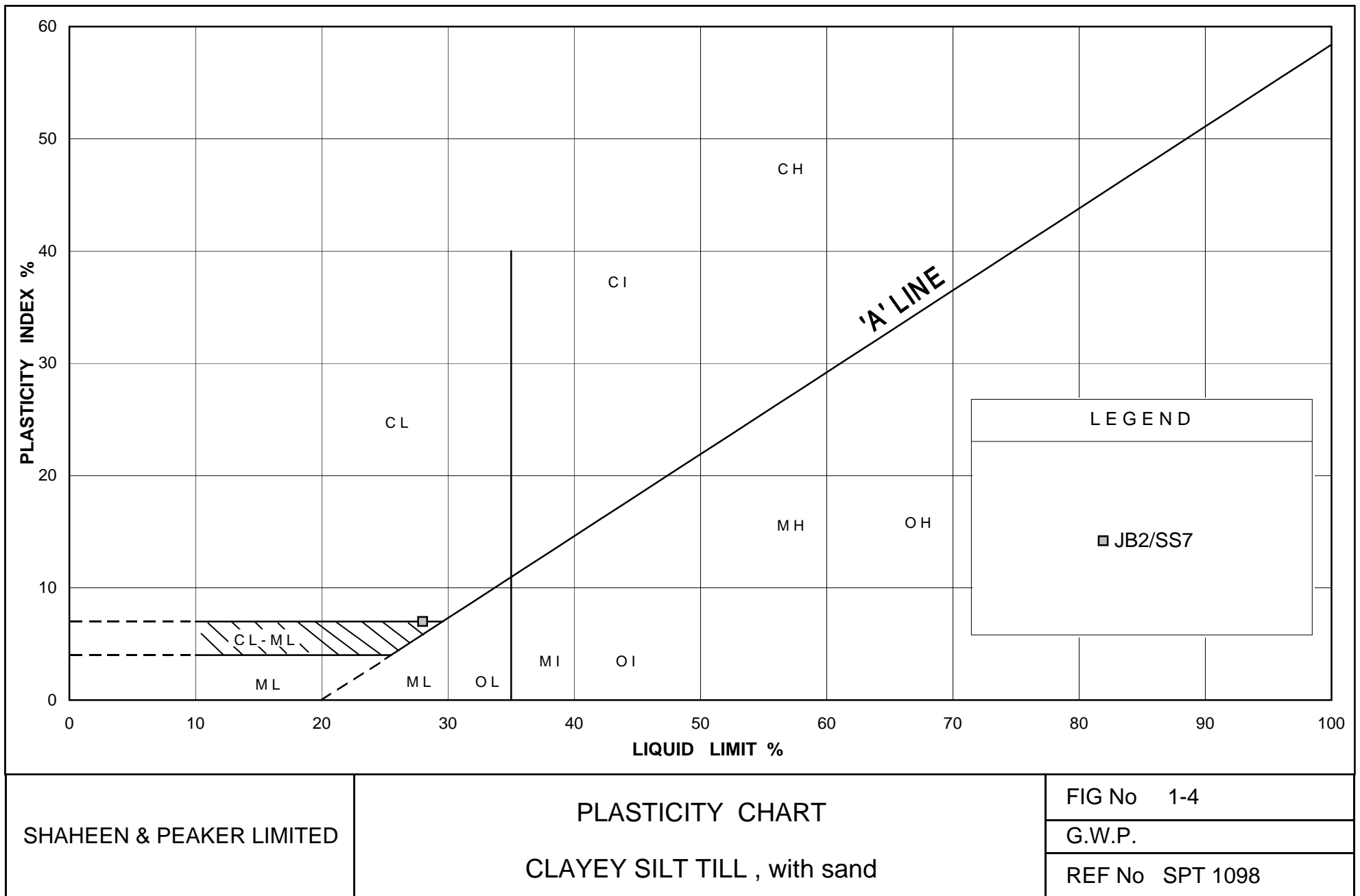
SHAHEEN & PEAKER LIMITED

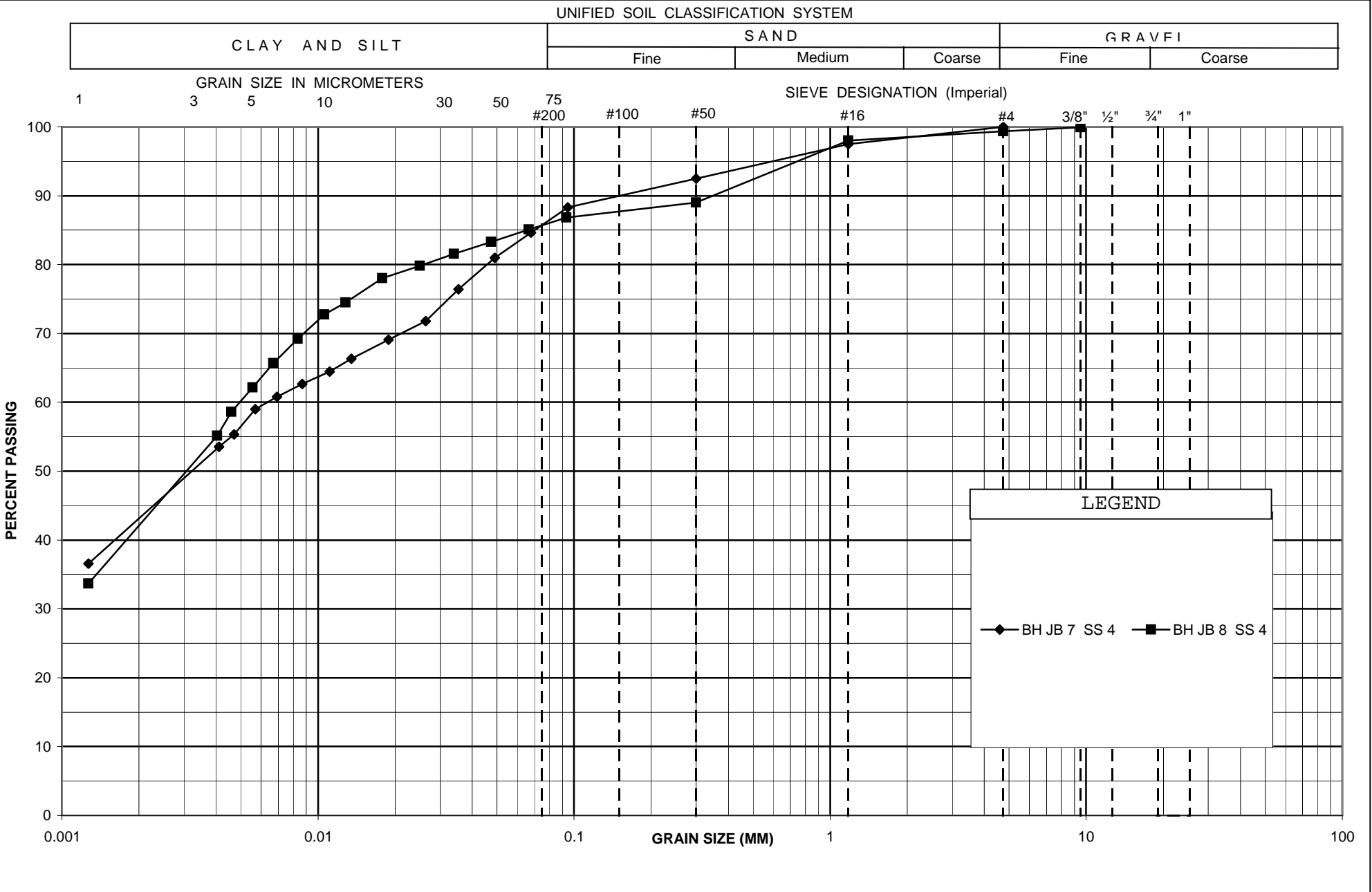
GRAIN SIZE DISTRIBUTION
CLAYEY SILT TILL , with sand

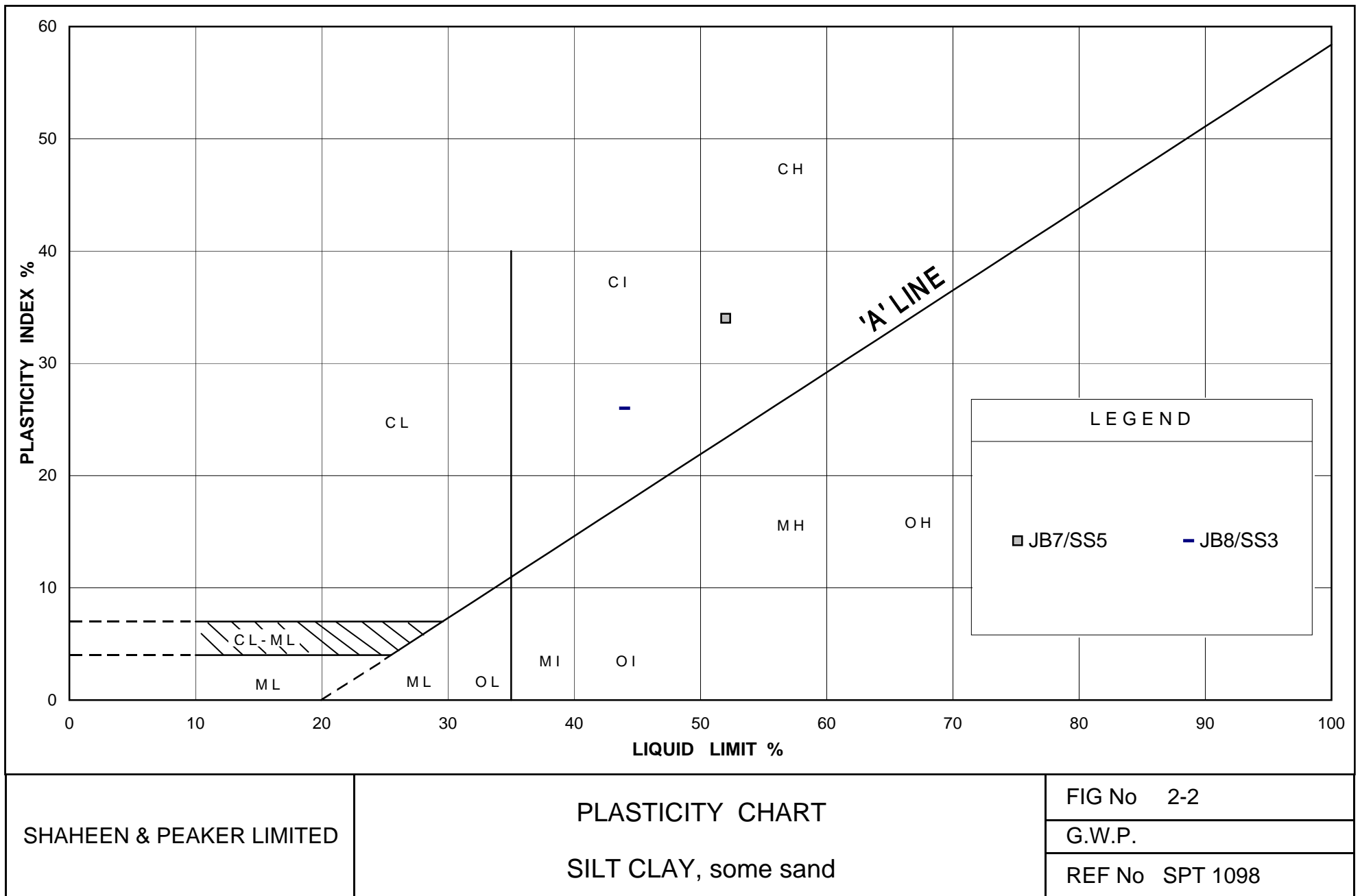
FIGURE No. 1-3

REF. No. SPT 1098

DATE DECEMBER, 2005







SHAHEEN & PEAKER LIMITED

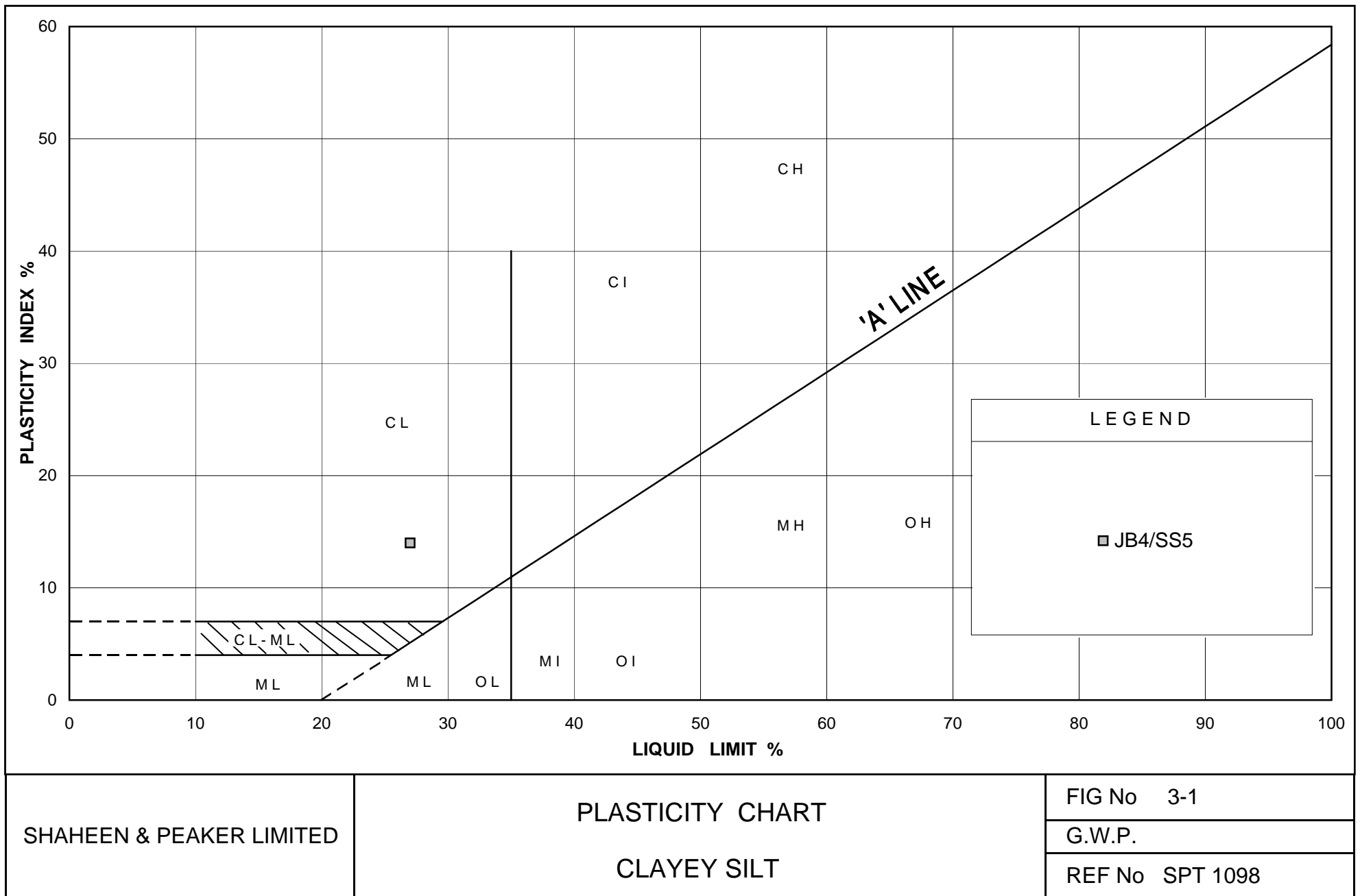
PLASTICITY CHART

SILT CLAY, some sand

FIG No 2-2

G.W.P.

REF No SPT 1098



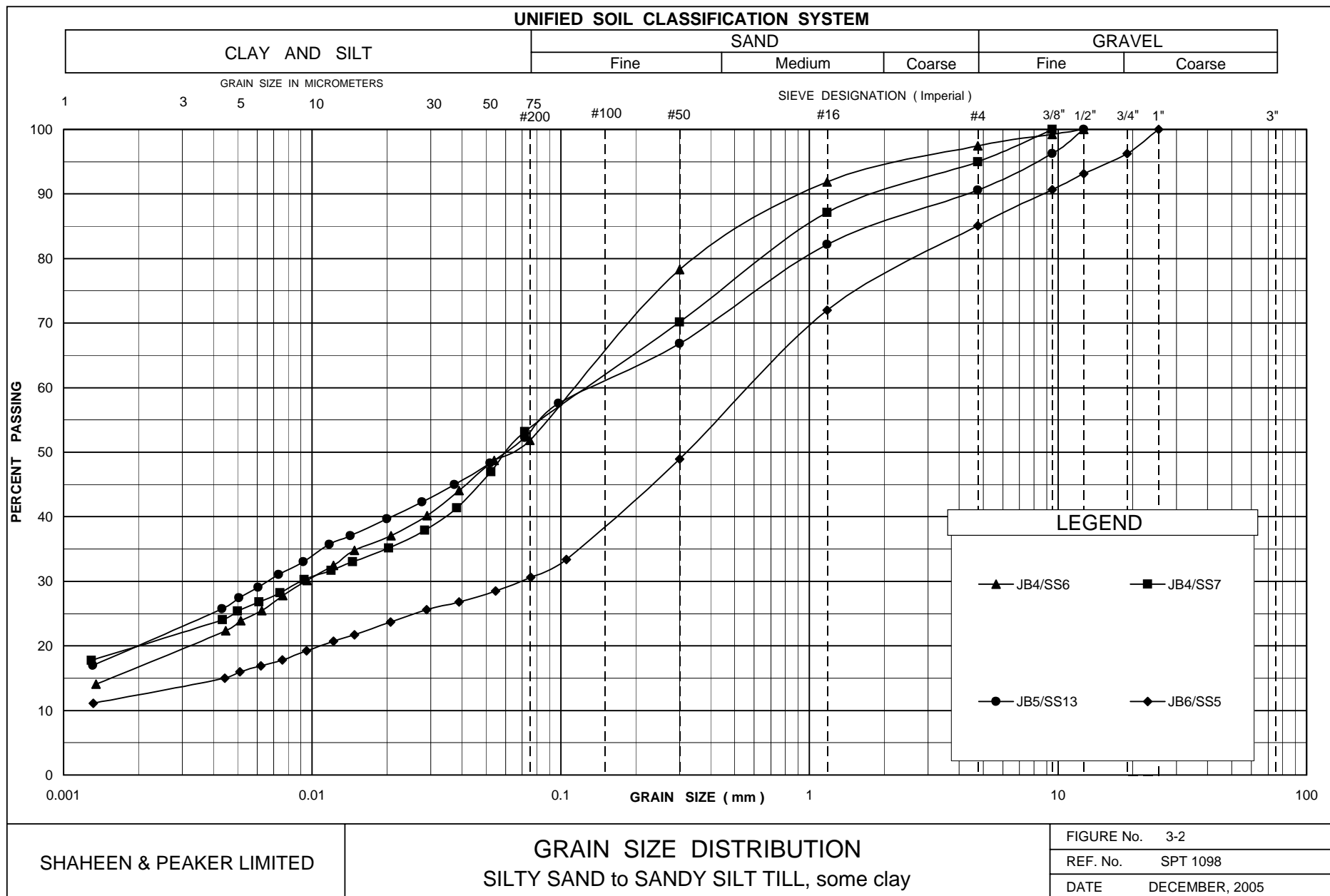
SHAHEEN & PEAKER LIMITED

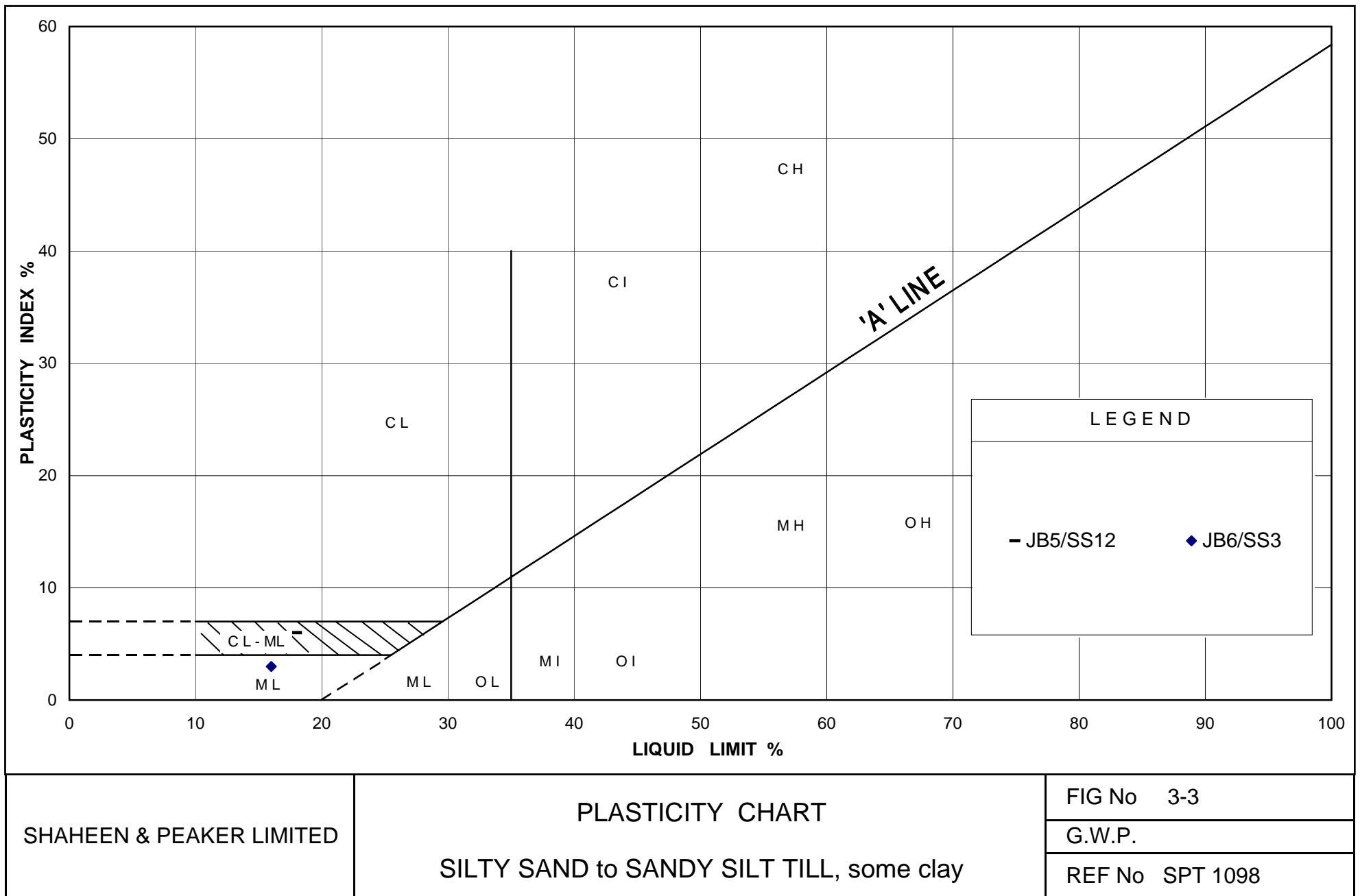
PLASTICITY CHART
CLAYEY SILT

FIG No 3-1

G.W.P.

REF No SPT 1098





SHAHEEN & PEAKER LIMITED

PLASTICITY CHART

SILTY SAND to SANDY SILT TILL, some clay

FIG No 3-3

G.W.P.

REF No SPT 1098

Appendix C

Explanation of Terms Used in Report

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 63.5 kg, 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 N.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J 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/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCUTRAL 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 (RQD), FOR MODIFIED RECOVERY IS:

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

JOINT 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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

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

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_e	1	SWELLING INDEX
c_a	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 = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
i_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
i_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
i	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
i_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_U)$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
i_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
i'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
PROPOSED CULVERT INSTALLATIONS UNDER
HIGHWAY 401 AND BURNHAM STREET
PORT HOPE TO COBOURG, ONTARIO
G.W.P. 4073-01-00**

Prepared For:

UMA ENGINEERING LIMITED

Prepared by:

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**Project: SPT1098
February 1, 2006**



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**FOUNDATION DESIGN REPORT
PROPOSED CULVERT INSTALLATIONS
UNDER HIGHWAY 401 AND BURNHAM STREET
PORT HOPE TO COBOURG, ONTARIO
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5. DISCUSSION & RECOMMENDATIONS

5.1 GENERAL

As part of the proposed six-laning of Highway 401 from County Road 28 in Port Hope to 700 m east of Burnham Street in Cobourg, three new culverts are proposed to be installed under the embankments of Highway 401 and Burnham Street, using jack and bore method. The proposed culvert locations, sizes, invert elevations and the approximate earth cover thicknesses are summarized in Table 5.1.1.

Table 5.1.1 New Culverts

No.	Station	Diameter (mm)	Length (m)	Proposed Invert Elevations (m)		Approximate Earth Cover Thickness (m) under Embankment
				Lt	Rt	
Culvert 1	12+880 (Hwy 401)	600	74.0	112.8	113.1	4.3
Culvert 2	14+360 (Hwy 401)	900	71.5	110.3	110.0	2.2
Culvert 3	10+047 (Burnham St.)	900	67.0	110.8	100.4	8.2

A total of 8 boreholes was drilled at these three culvert locations. These boreholes indicate, in general, below some pavement/embankment fills and topsoil which extend to depths ranging from 0.25 to 6.1 m below the existing road grades, the presence of firm to hard clayey silt or silty clay deposits which are further underlain by some cohesive deposits at Culvert 1 location, and by some basically non-cohesive glacial till deposits (silty sand to sandy silt till) at Culverts 2 and 3 locations.

Permanent groundwater levels at the proposed culvert locations are believed vary from about El. 101 m to 113 m. In addition, perched water conditions are likely occurring due to the accumulation of surface water in the relatively pervious fill materials overlying the relatively impervious clayey deposits. It should also be pointed out that the groundwater is subject to seasonal fluctuations and fluctuations in response to major weather events.

5.2 CULVERT CONSTRUCTION OPTIONS

Based on the results of the investigation, a number of design options in constructing the proposed culvert(s) was considered for this project. The following six options, which may possibly be feasible for this project, are listed below:

- Option 1: Jack and Bore
- Option 2: Pipe Ramming
- Option 3: Pipe Jacking with TBM
- Option 4: Micro-Tunnelling
- Option 5: Hand Mining
- Option 6: Open Excavation with Shoring

The above proposed design options are feasible in various degrees for the construction of the culverts in terms of geotechnical engineering. The selection of a preferred option will also depend on the construction cost, risk of ground subsidence, schedule factors and so on.

The following section presents a discussion of the suitability of six construction methods and is intended only to assist in the evaluation of tender documents estimated by contractors. Summary of the construction methods are presented in Table 5.1.2. Details of each proposed design option are discussed below.

Option 1: Jack and Bore

This technique forms a bore hole from a drive shaft to a reception shaft by means of rotating cutting head. Spoil is transported back to the drive shaft by helical auger flights rotating inside a steel casing. The casing is jacked in place simultaneously with the augering operation. After the installation of the steel casing, a reinforced concrete pipe will be installed inside the casing and the gap between the casing and the pipe will be grouted.

The maximum casing diameter used in this operation is limited to about 1.3 m for most contractors in Ontario. Based on the results of the investigation, the proposed culverts will be constructed primarily through the clayey silt till/clayey silt deposits and clayey silt fill at Culverts 1 and 2 locations, while at Culvert 3 through primarily compact to very dense silty sand to sandy silt till. These subsurface conditions are considered feasible for a jack and bore operation.

Option 2: Pipe Ramming

In a pipe ramming operation, a pneumatic ramming tool attached to the rear of a steel casing drives the casing into the ground with repeated percussive blows. The installed pipe usually has an open end that allows the soil to enter the casing during the installation. The spoils inside the casing can be removed either during or after the installation, by auger,

TABLE 5.1.2: SUMMARY OF CONSTRUCTION METHODS

Option	Construction Method	Application Range		Control of Open Face Stability	Capability of Boulder Excavation	Temporary Support	Permanent Lining	Estimated Settlement (mm)	Alignment Control	Advantage	Limitation	Cost Comparison
		Length (m)	Diameter (m)									
1	Jack and Bore	Up to 150 m	0.2 to 1.5 m	mild wet/flowing sand condition may possibly be controlled by pulling the augers back in the casing and compressing a plug of soil in front of the casing to prevent soil from flowing in.	Possible to handle small boulders of size up to 25% of the casing diameter.	Provided by the steel casing during the jack and bore operations.	Pipe to be installed inside the steel casing	15 to 50	<ul style="list-style-type: none">❖ By hydraulic jacks in shafts pushing steel casing❖ Not very good control in mixed face	<ul style="list-style-type: none">❖ Technique commonly used locally❖ Skill labour, equipment and contractor available locally❖ Relatively lower cost❖ Technique suitable for large variety of soils conditions❖ Minimum ground subsidence if operated properly	<ul style="list-style-type: none">❖ Not suitable in wet flowing and/or dry runny (above water table) sand conditions❖ Not suitable for soil with large boulders❖ Maximum pipe diameter limited to 1.3 m for most contractors in Ontario	Low
2	Pipe Ramming	Up to 45 m	0.2 to 1.4 m	The excess soils entering inside the casing during the construction may act as a plug to provide face stability.	Possible to handle boulders of the size smaller than the casing diameter.	Not required	Pipe to be installed inside the steel casing	<5	<ul style="list-style-type: none">❖ Controlled by guide rails set to the alignment in the insertion pit❖ Not very good control depending on workmanship and ground condition	<ul style="list-style-type: none">❖ Can be used in a wide variety of soils, even under 'running' condition❖ Superior for installation up to 20 m pipe length range❖ Fast installation❖ Minimum ground subsidence❖ Required width and depth of the pits are smaller	<ul style="list-style-type: none">❖ Difficult for long installation (normally less than 45 m)❖ Difficult for very dense soils❖ High noise level❖ May generate significant soil disturbance	Low
3	Pipe Jacking with TBM	Up to 400 m	1 to 3 m	Earth Pressure Balance Machine or slurry shield TBM is required to maintain face stability.	Possible to handle small boulders of size up to 25% of the casing diameter.	Not required	Pipe installed during the tunneling operations	10 to 35	<ul style="list-style-type: none">❖ Very tight alignment and grade tolerance❖ Good control	<ul style="list-style-type: none">❖ Method can be executed with virtually any ground condition (except large boulders) with adequate precautions❖ Pipe installed during the operations, avoid double handling❖ Minimum ground subsidence if operated properly❖ Mobilization cost for smaller diameter TBM (<1.2 m) is less in comparison with larger (>2 m) diameter TBM	<ul style="list-style-type: none">❖ High capital cost and set-up cost❖ Specialized operation requiring good operator skill and experience❖ Very tight alignment and grade tolerance and expensive corrective actions if mis-aligned.❖ TBM over 2.5 m diameter may not be available locally	High
4	Micro-Tunnelling	Up to 250 m	0.2 to 3 m	Micro-Tunnelling Machine is designed to provide ability to maintain face stability	Possible to handle small boulders of size up to 25% of the casing diameter.	Not required.	Pipe installed during the tunneling operations	<15	<ul style="list-style-type: none">❖ Remotely controlled pipe jacking process❖ Good control	<ul style="list-style-type: none">❖ Method can be executed with virtually any ground condition (except large boulders) with adequate precautions❖ Pipe installed during the operations, avoid double handling❖ Minimum ground subsidence if operated properly	<ul style="list-style-type: none">❖ Very high capital cost and set-up cost❖ Specialized operation requiring good operator skill and experience.❖ Sophisticated equipment❖ Availability of equipment locally	Very high
5	Hand mining	No theoretical limit	1 to 3 m	It can be achieved by dewatering or grouting in advance of excavation	Good	Segmental lining or ribs & lagging with geo-synthetics	Permanent lining required to provide long-term stability and satisfy hydraulic requirements	20 to 50	<ul style="list-style-type: none">❖ Hand-mining and jacking in direction of advance❖ Good control depending on workmanship	<ul style="list-style-type: none">❖ Skill labour, equipment and contractor available locally❖ Relatively lower cost❖ Capable to handle large boulder	<ul style="list-style-type: none">❖ Require temporary support and permanent lining❖ Higher risk of ground subsidence❖ Require groundwater control❖ Difficult to control grouting quality	Medium
6	Open Excavation with Shoring	No theoretical limit	N/A	Open face supported by temporary shoring such as sheet piles	Boulders may affect the sheet pile installation.	Temporary shoring by sheet piling	Not applicable	25 to 50	<ul style="list-style-type: none">❖ Good control by surveying	<ul style="list-style-type: none">❖ Skill labour, equipment and contractor available locally❖ Lower capital and setup cost	<ul style="list-style-type: none">❖ Require traffic control and lane closure❖ Require groundwater control❖ Large space requirement❖ Boulders in the till may affect sheet pile installation	Very High

compressed air or water jetting. After completing the installation, concrete pipe will be installed inside the casing and the gap between the casing and the pipe will be grouted.

This method of installation is mostly used on pipes between 0.1 and 1.4 m in diameter and for pipe installation over relatively short distances (i.e. less than 45 m). Although longer distance (up to 100 m) has been achieved in favorable ground condition (i.e. soft clayey soil), the length of the tunnel installed by pipe ramming is limited by the ground conditions. Based on the investigation results, the tunnels will go through generally stiff to very stiff cohesive soils (i.e. clayey silt fill, clayey silt till and silty clay deposits) at Culvert 1 and 2 locations and through compact to dense silty sand to sandy silt till at Culvert 3 location. Considering the proposed culvert lengths of about 70 m through the above mentioned site conditions, as well as nuisance factors, pipe ramming method is not considered suitable for the construction of the culverts.

Option 3: Pipe Jacking with TBM

Pipe jacking with TBM installs a prefabricated pipe through the ground from a drive shaft to a reception shaft. The pipe is pushed by jacks located in the drive shaft and the jacking force is transmitted through the pipe to the face of the excavation. The excavation with this method is accomplished by a TBM (Tunnel Boring Machine) and the spoil is transported out of the jacking pipe and shaft manually or mechanically. Typically, pipe jacking with TBM is applicable to tunnels with relatively larger diameter (e.g. 1 to 3 m).

This tunneling method is so versatile that it can be executed with virtually any ground conditions (except large boulders) with adequate precautions. In unstable soil conditions such as the embankment fill and the underlying wet sand and silt layers, Earth Pressure Balance Machine (EPBM) or slurry shield TBM is required to counterbalance the ground and hydrostatic pressures and to minimize ground subsidence. With EPBM, dewatering of the wet sand and silt is not required, which will not impact on fisheries and the environment.

The main disadvantage of this method is its high capital and set-up costs. This technique also requires good operator skill and experience. In addition, this method has a very tight alignment and grade tolerance since the permanent lining (the pipe segments) is being installed during the tunnel operation. If large boulders are encountered, hand-mining may have to be employed which could lead to project delay and extra costs.

With pipe jacking and the use of EPBM, if operated properly, the maximum ground settlement is expected to be minimal in the order of 10 to 35 mm, which is considered acceptable under the road.

Option 4: Micro-Tunnelling

This technique is the improvement of the pipe jacking technique with TBM. It is a remotely controlled, guided pipe-jacking process that provides continuous support to the excavation face. The guidance system usually consists of a laser mounted in the drive shaft communicating a reference line to a target mounted inside the tunnelling machine. This technique provides ability to control excavation face stability by applying mechanical or fluid pressure to counterbalance the earth and hydrostatic pressures.

The main advantage of this technique is that it is sophisticated and will most likely complete the project in shorter time. It will also complete the tunneling operation with even less ground subsidence, if operated properly, which is estimated to be about less than 15mm.

The main disadvantage of this method is its very high cost and that it may not be available locally.

Option 5: Hand Mining

In a hand mining operation, the excavation of the tunnel is accomplished manually and a temporary ground support system is required during the operation. The temporary ground support system can be steel or concrete segmental liner or steel ribs with wood lagging. Groundwater control may be required to minimize water leakage into the tunnel. Workers are required inside the tunnel to perform the excavation and/or spoil removal. The excavation will be accomplished by hand mining with the assistance of small excavation tools.

With this method, control of alignment and grade is accomplished by overmining in the direction of the change and the pipe will move into the overmined area as it is pushed forward.

The main advantage of this technique is that it is relatively economical, and that large boulders can be removed.

This technique is limited by the difficulty of controlling the grouting quality and its impact to the environment. Higher risk of ground subsidence may also be encountered with anticipated settlements could be in the order of 20 to 50 mm. In addition, with this method, the tunnel project will take a longer time to complete.

In order to accommodate the working space for workers, the minimum diameter of the tunnel using hand mining is about 1 m.

Option 6: Open Excavation with Shoring

This consists of conventional open cut excavation technique with probably vertical excavation and the ground supported by temporary shoring due to limited space on the road. Shoring could be in the form of sheet piles or soldier piles and lagging.

This method will require lane closures on Highway 401 and Burnham Street which are considered undesirable. In this case, the existing embankment fill, which contains primarily of sand and gravel and clayey silt, will be excavated and shored.

The shoring system could be designed to resist a constant lateral earth pressure distribution with depth and hydrostatic water pressure, where applicable. A coefficient of lateral earth pressure of 0.3 can be used in the design of temporary shoring and support system.

Special Provision 902S01 – Excavation and Backfilling- Structures should be included in the Contract Documents. Lower sections of the shoring (e.g., sheet piling) may have to be left in place, if withdrawal of the shoring could cause excessive settlement of the newly constructed culvert, prior to backfilling. Backfilling requirements will be provided if this option is selected as one of the preferred options.

5.3 RECOMMENDED CONSTRUCTION OPTION

We understand that due to the high traffic volume, traffic disruption of Highway 401 and Burnham Street during the culverts construction should be kept to a minimum, and therefore, trenchless installation methods should be utilized in this project (i.e. open cut method should not be used). The design diameters of the proposed pipe culverts are about 600 and 900 mm for this project, which are not considered adequate to be able to provide sufficient space for the hand mining method, unless a larger size primary liner is used, which would increase costs. Similarly, the use of Tunnel Boring Machine (i.e TBM) is considered to be costly for these relatively small diameter culverts. On the other hand, micro-tunnelling technique can be used in smaller culvert installation and can be executed with any ground condition (except boulders) with adequate precautions, however, it is considered not a cost effective method in comparison with other tunnelling methods. Although pipe ramming is a cost effective option and is ideal for smaller and shorter tunnel constructions, it is not considered suitable for about 70 m long tunnels under the existing site condition. As well this method will cause nuisance to travelling public during construction, as well as possible damage to the paved road surface. The remaining economically feasible option is the jack and bore alternative.

Based on the results of the investigation, the proposed culverts will be tunnelled through the clayey silt till/clayey silt deposits and clayey silt fill at Culverts 1 and 2 locations, while at Culvert 3 through the compact to very dense silty sand to sandy silt till. These underground conditions are considered sufficiently suitable for jack and bore operation.

Based on the above discussion, we recommend jack and bore (auger boring) option as the design method of constructing culverts for this project.

5.4 CULVERT INSTALLATIONS USING JACK AND BORE

The jack and bore method of horizontal earth boring is a process of simultaneously jacking casing through the earth while removing the spoil inside the encasement by means of a rotating auger. The casing also serves to support the soil around it as the spoil is removed.

This technique is limited by the presence of ravelling, flowing or running ground conditions, in which case, dewatering and/or grouting should be performed prior to proceeding with boring.

5.4.1 TUNNELLING GROUND CLASSIFICATION

Tunnelling procedures depend upon a number of factors, the most important of which are the groundwater conditions and the soil type through which the tunnel must pass. Table I in Appendix D presents the classification of the ground according to terminology used by tunnel laborers (commonly known as the Tunnelman's Ground Classification System), the soil types and the probable tunnel working condition for each classification.

According to the Tunnelman's Ground Classification System, the compacted sand and gravel fill layer encountered under the pavement is classified as 'slow ravelling' if damp but could be 'fast ravelling' when wet. Any sand and silt layers above the water table is also considered 'slow ravelling'. Below the water table these layers are considered 'flowing' to 'fast ravelling'.

In ravelling ground, the material above the tunnel or in the upper portion of the tunnel working face may sooner or later tend to flake off and fall into the heading. In fast ravelling ground, the process starts within a few minutes; otherwise, it is described as slow ravelling. The action is progressive and may lead to open cavities above the tunnel or even lead to sinkholes at the surface. The ravelling can be prevented if at least moderate support (or temporary support) is provided at an early stage before the loosening becomes extensive. Alternatively, ravelling ground can be modified by drainage, grouting, or freezing.

Running ground condition may occur in perfectly cohesionless materials such as dry sand or clean gravels. Tunnel construction in this type of soil is difficult since the face runs off to form a stable slope during excavation; and without rapid and continuous support of the tunnel crown, advance of the tunnel will be impossible. Pre-grouting of the open face prior to excavation may be considered to avoid running ground condition during excavation.

Flowing ground condition is often caused by seepage pressures at the heading. For sands below the water table, available data and experience suggests the material will flow into the

tunnel if the uniformity coefficient D_{60}/D_{10} , is less than 5 and if the fines (particle sizes smaller than 0.075 mm) content is less than 20%. These types of soils are considered dangerous in tunnelling even when closable face shields or tunnelling machines are used. Most flowing ground can be transformed into ravelling or even firm ground by drainage, air pressure or by grouting.

In firm ground, the materials generally consist of stiff to hard clays or cemented or cohesive granular materials. The generally stiff to hard clayey deposits can be classified as 'firm' ground. In this case, a heading may be advanced by about 0.5 m or more without immediate support. Tunnelling in firm ground can be carried out with lower risk of collapse because there is enough time for the installation of temporary support.

5.4.2 DESIGN AND PROCEDURES

Jack and bore method of tunnelling is generally suitable in majority of conditions presented above, if promptly operated. This operation is not suitable for wet flowing sand condition and fast ravelling condition. To utilize this technique, the water in these pervious layers will first have to be removed by pumping from filtered deep wells and/or well points (if less than 5 m deep). It should be noted, however, that extensive dewatering may cause settlements of adjacent ground or structures such as pavements, underground utilities or foundations of nearby structures. Flowing condition can also be controlled in smaller diameter tunnel by pulling back in the casing and compressing a plug of soil in front of the casing to prevent soil from flowing in.

It is a normal practice to install the tunnel on a small grade such that any water seepage into the opening is directed away from the tunnel face. Control of alignment and grade of the steel casing is accomplished by the hydraulic jacks in the shaft. The jacking equipment generally has steel guide rails installed to provide proper alignment and grade to the pipe as it is pushed forward into the tunnel excavation. Problem with alignment could occur when boulders are encountered in upper portion of the tunnel face. In this case, the boulder may have to be hand-mined, but this should be carried out with care to avoid fast ravelling condition and ensure that all voids are filled in with grout. Pre-grouting from the tunnel face may be necessary to ensure stability of the sand fill above the tunnel.

The estimated coefficient of friction between the steel casing and the surrounding soils and the estimated coefficient of lateral earth pressure are discussed for each site in the following sections.

5.4.3 CULVERT 1 – STATION 12+880 (HIGHWAY 401)

Based on the profile drawing provided by UMA, we understand that the invert elevations of this 600 mm diameter culvert under Highway 401 EBL and WBL embankments are about

El. 112.8 and 113.1 m on the north and south sides of the highway, respectively. The existing grade (top of the pavement) of the highway is at approximately El. 118 m.

From the borehole information, the proposed tunnelling is expected to be carried through a mixed face condition. Based on the borehole data, at Borehole JB1 location (drilled from the south shoulder of the highway), the upper (i.e. near crown) soil can be expected to be embankment fill, while the main lower portion would be in the very stiff clayey silt till. The clayey silt till is a stable cohesive material which can be classified as 'firm' in accordance with Table 1 in Appendix D. The clayey silt fill which is described as sandy and with some organics can be classified as 'slow ravelling' and as such will require some support. If a perched water condition is encountered due to the water accumulation in the fill overlying the clayey silt till, the water should be removed by dewatering. A similar condition occurs further north at Borehole JB2 location where the upper portion of the tunnel may be excavated through embankment fill while the lower portion through more stable clayey silt till. The embankment fill encountered in this borehole is described as sandy silt with some clay. While the clay content may impart some apparent cohesion, the soil can be described as 'slow ravelling' similar to Borehole JB1, and again it may be prudent to effect some dewatering. Borehole JB3 drilled from the north shoulder, indicates that the tunnelling will likely be effected through clayey silt till, which is a stable material, as mentioned before. It should however be pointed out that in fill conditions the elevation and particularly the composition of the fill can vary within short distance in between and beyond borehole locations.

Considering the material above the roof of the tunnel contains some cohesionless fill (sandy silt fill), it is recommended that the advance of the augering beyond the front end of the liner be limited to no more than 0.5 m at a time prior to jacking the liner. An experienced tunnel contractor should be retained for this project.

To avoid the mixed face condition and/or dewatering of the perched water in the fill materials, consideration may be given to construct the culvert entirely in the native clayey silt till by lowering the invert to about El. 111.5 m if possible. This will probably involve lowering of the ditches.

Depending on the site condition, groundwater seepage during the construction of jack and bore is expected to be slow to moderate. In this case, as mentioned before, the liner should be installed on a sufficient grade upward such that the seepage water can flow by gravity away from the face towards the entrance shaft where the accumulated water can be pumped out.

The coefficient of friction between the steel casing and the well compacted clayey silt fill and native clayey silt till can be taken as 0.25; between the steel casing and the well compacted sandy silt fill as 0.3. The bulk unit weight of the materials above the tunnel crown could be assigned as follows:

Granular pavement fill:	21.5 kN/m ³
Embankment fill:	19.5 kN/m ³
Clayey silt till:	20.5 kN/m ³

For the soils surrounding the tunnel, the approximate coefficient of lateral earth pressure at rest, K_o , could be taken as 0.5.

5.4.4 CULVERT 2 – STATION 14+360 (HIGHWAY 401)

From the profile drawing, we understand that the invert elevations of this 900 mm diameter culvert are about El. 110.3 and 110.0 m on the north and south sides of the highway, respectively. The existing grade (top of the pavement) of the highway is at approximately El. 113 m.

From the borehole information (Boreholes JB7 and JB8), the proposed tunnelling will likely be carried through a mixed face condition (i.e. clayey embankment fill near the top and natural silty clay throughout the main and bottom sections). Boreholes JB7 and JB8 show that similar to the previously discussed Culvert 1 location at Station 12+880, at this Culvert 2 location, the immediate cover and upper portion of the crown of the pipe can be expected to be tunnelled through a clayey silt to silty clay embankment fill. The presence of sandy silt zones and some topsoil was also noted within the fill. N-values recorded in this primarily cohesive fill range from 5 to 7 blows/0.3 m, indicating a firm consistency (i.e. the fill does not appear to be systematically compacted when it was first placed). The fill can thus be classified as 'firm' to 'slow ravelling' ground. It should however be pointed out that the composition of the fill can change in between and beyond borehole locations. As well, at this site, cover above the tunnel is relatively shallow (i.e. about 2.0 m or somewhat less). It is always possible that the thickness of the clayey fill encountered at the shoulder areas may decrease or even may be replaced by granular pavement fill at other locations. In such a case, granular type soils may be encountered in the upper portions of the tunnel, which could present a 'running' condition. If a perched water condition is encountered due to the accumulation of surface water above the more impervious silty clay, then a 'flowing' condition could occur. For this reason, the tunnel should proceed with caution and plans should be put in place to deal with this condition should it be encountered. As well, any perched water should be removed by dewatering.

To avoid the mixed face condition and/or dewatering of the perched water in the fill materials, consideration may be given to construct the culvert entirely in the native silty clay deposit by lowering the invert to about El. 109.0 m if possible. This will probably involve lowering of the ditches.

The native soil underlying the embankment fill consists of silty clay (with some sand and occasional gravel) which is a basically cohesive material. Its consistency at Borehole JB7 location is firm only with N-values of 5 and 7 at the culvert elevation, while N-values of

between 27 and 57 were encountered at the north shoulder area in Borehole JB8, indicating a generally hard consistency. In accordance with Table 1 in Appendix D, the silty clay can be described as 'firm' ground, but this difference in consistency from one end of the tunnel to the other should be taken into consideration when pushing the pipe as well as for the selection of optimum shaft location, if required.

Depending on the site condition, groundwater seepage during the construction of jack and bore is expected to be slow to moderate. In this case, as mentioned before, the liner should be installed on a sufficient grade upward such that the seepage water can flow away from the face towards the entrance shaft where the accumulated water can be pumped out. In addition, dewatering may be required as discussed before.

The estimated coefficient of friction between the steel casing and the the clayey embankment fill can be taken as 0.25; between the steel casing and the firm to hard clayey silt deposit as 0.25 to 0.35. The bulk unit weight of the materials above the tunnel crown could be taken as 21.5 kN/m^3 for the granular pavement fill, 19 kN/m^3 for the clayey silt to silty clay embankment fill, 18 kN/m^3 for firm silty clay and 20 kN/m^3 for the hard silty clay. For the soils surrounding the tunnel, the estimated coefficient of lateral earth pressure at rest, K_0 , could be taken as 0.5.

5.4.5 CULVERT 3 – STATION 10+047 (BURNHAM STREET)

From the profile drawing, we understand that the inverts of the proposed culvert will be at about El. 100.6 m or about 9 m below existing road surface (at El. 109.7 m). The culvert will be 900 mm in diameter and approximately 67 m long.

From the borehole information, the proposed tunnelling will likely be carried primarily through the compact to very dense silty sand to sandy silt till. Clayey silt can possibly also be encountered near the top portion. The clayey silt is considered 'firm' ground in accordance with Table 1 in Appendix D, while the silty sand to sandy silt till can be considered as a 'slow ravelling' to 'firm' material, as it has some clay content that imparts it some cohesion. It should be noted that, due to the nature of its formation, cobbles and boulders can always be expected in the glacial till deposits. Based on these, the proposed jack and bore operation is considered feasible, but certain construction difficulties could be encountered, such as, hard drilling through very dense till, ravelling or possible boulders and misalignment due to mix face condition.

Depending on the site condition, groundwater seepage during the construction of jack and bore is expected to be slow to moderate. In this case, as mentioned before, the liner should be installed on a sufficient grade upward such that the seepage water can flow away from the face towards the entrance shaft where the accumulated water can be pumped out.

The estimated coefficient of friction between the steel casing and the silty sand to sandy silt till can be taken as 0.35. The following bulk unit weight for materials above the tunnel crown can be assigned.

Granular pavement fill:	21.5 kN/m ³
Clayey silt embankment fill:	19.5 kN/m ³
Old topsoil:	17 kN/m ³
Clayey silt:	20 kN/m ³
Silty sand to Sandy silt till:	21.5 kN/m ³

For the soils surrounding the tunnel, the estimated coefficient of lateral earth pressure at rest, K_0 , could be taken as 0.5.

5.5 SETTLEMENTS

Settlement caused by tunnelling in soft ground is the aggregate of two basic types of settlement which consist of ground loss or 'immediate' settlement, and consolidation settlement.

The 'immediate' settlement is the direct result of the movement of ground into the tunnel heading. The factors which influence the magnitude of immediate settlement due to tunnelling include soil strength and stiffness, the method of tunnelling and the quality of tunnel operations (including the method of handling localized factors such as encountering boulders). Even when tunnelling is carried out apparently through homogenous soils with the same equipment and crews, ground settlements typically vary by a factor of 2 or 3. This variation can be ascribed to items such as ploughing of the shield or use of overcutters, quality and speed of ring grouting and localized variations in soil type, strength or stiffness.

Despite the uncertainty, the large quantity of information on case studies available in the literature allows approximate estimates of peak settlements to be made based on typical parameters for various conditions. Generally, estimates of maximum immediate settlements over the tunnel centerline are less than 25 mm. These settlements are likely to be experienced within a horizontal distance on either side of the tunnel equal to about one-half the depth to the tunnel axis. Smaller settlements will likely occur within a horizontal distance equal to about one tunnel axis depth.

Good workmanship and site control are the most effective way to reduce immediate settlements to a practical minimum. Factors to consider in the specification and review of tenders include grouting behind the temporary support system as quickly as possible, minimizing the use of overcutters and minimizing overbreak due to excavation of boulders. Full-face machines can roll boulders around the head causing overbreak. Where overbreak occurs due to boulders or overcutting, the voids should be filled as soon as possible.

The term 'ground loss' as discussed above does not include loss of ground resulting from face instability. Much greater settlements than the estimates given above are likely to occur where there is instability at the face.

Consolidation settlement is the settlement caused by pore-pressure changes in compressible deposits as related to dissipation of excess pore pressures induced by tunnelling and also from long term seepage effects into the tunnel. For this project, the consolidation settlements are estimated to be less than 6 mm.

As mentioned before, settlements due to tunnelling are difficult to estimate. Generally, estimates of maximum immediate settlements over the tunnel centerline are less than 25 mm.

5.6 TUNNEL SHAFT

Where insufficient space exists to permit open excavation, appropriately braced shafts may be required for methods of construction using horizontal boring technique. The depth of overburden excavation at the shaft locations will range from about 2 to 6 m below existing ground surface. The excavation will be carried out through variety of materials such as topsoil, some surficial fill and embankment fill as well as clayey silt and silty sand to sandy silt till. Since groundwater and perched water could be encountered, appropriate dewatering control will be required at the shaft excavations. Groundwater seepage into the excavation can be handled by continuous pumping from properly filtered sumps located within the excavation, by well points or deep wells.

The design of the braced excavation/shafts in the overburden should take into account horizontal soil loads, hydrostatic water pressure, traffic load and any surcharge due to construction loadings. The shoring system could be designed to resist a constant lateral earth pressure distribution with depth and hydrostatic water pressure, where applicable. A coefficient of lateral earth pressure of 0.3 can be used in the design of temporary shoring and support system.

The protection system for the construction of the tunnel shafts should be designed as per SP539S01 – Protection Schemes. Due to the proximity of the tunnel shafts to Highway 401 and Burnham Street, the performance level of the protection system is recommended as Level 2, i.e., lateral movement of any portion of the protection system shall not exceed 25 mm.

5.7 CONSTRUCTION CONSIDERATIONS

All excavations, shoring and backfilling should be carried out in accordance with the Safety Regulations of the Province (i.e., Occupational Health and Safety Act O.Reg. 213/91), as well as the following specifications:

SP 539S01	-	Protection Schemes
SP 902S01	-	Excavation and Backfilling to Structures

The boreholes show that the excavations for the tunnel shaft can be expected to extend through embankment fill, topsoil, firm to very stiff clayey soils and compact to dense silty sand to sandy silt till.

Depending on the construction method that is selected for proposed culverts, Non-Standard Special Provision (NSSP) should be provided in the Contract Documents to specify the requirement of the selected construction method.

A typical NSSP used for jack and bore construction on a previous MTO project is presented in Appendix E.

At some locations, the proposed jack and bore culverts are expected to go through clayey silt till and silty sand to sandy silt till. Being of glacial origin, the glacial till deposits can be expected to contain random cobbles and boulders. Cobbles may also be present in the other natural soil types and in the fill deposits. Therefore, an NSSP should be included in the Contract Document to make the contractor aware that the presence of cobbles and boulders can always be expected which can cause problems during the jack and bore operation, such as increasing the time required for excavating, the employment of special equipment, etc. In addition, the sandy silt fill material and the clayey silt fill which is described as sandy and with some organics at Culvert 1 location and the possible granular type soils at Culvert 2 location may become ravelling or even running under the influence of the perched water. In this case, an NSSP should be included in the Contract Document to warn the contractor that the tunneling procedure may require dewatering and/or other measures to prevent the ground from "raveling," or 'running' or 'flowing' which can lead to excessive immediate settlements.

Water can be removed by pumping from filtered sumps and the collected water from dewatering operations should be filtered or passed through sediment traps to prevent turbidity. If however the dewatering scheme does not produce adequate dewatering of the culvert excavations, then other dewatering/depressurizing methods may need to be resorted to.

We recommend that any surface water be diverted away from the culvert excavation, in addition to the chosen groundwater control scheme, to enable the culvert construction and fill placement to be carried out in the dry. Major problems due to groundwater seepage are not anticipated, provided groundwater control is carried out properly.

5.8 INSTRUMENTATION AND MONITORING

It is recommended that instrumentation in the form of settlement monitoring be carried out for the jack and bore culverts under Highway 401 and Burnham Street. In this case, surface settlement markers (e.g. surveyors nails) could be installed on the shoulders of the highway, while settlement rods can be used on the embankment slopes. The settlement monitoring points will be placed along the tunnel axis and a distance from this axis, on top and slopes of the embankment. In addition to the settlement monitoring, the excavated soil types and quantity should also be determined during the tunnelling operation by an experienced geotechnical personnel. The quantity of excavated materials will be compared with the theoretical volume of excavation in order to assess the risk of over-excavation. Such monitoring is necessary to confirm that any settlement/movement associated with the proposed construction would be within tolerable limits.

5.9 SOIL DISPOSAL

The excavated materials from the tunnel and shaft construction should be stockpiled and checked for contamination prior to removal/disposal off-site, in order to determine which disposal option is best suited for the excavated materials. If found "clean", these materials can be used to flatten slopes.

It is recommended that a programme of geotechnical/material inspection and testing be carried out during the construction phase of the project to confirm that the conditions exposed in the excavations are consistent with those encountered in the boreholes and the design assumptions, and to confirm that the various project specifications and materials requirements are being met.

6. CLOSURE

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

The Limitations of Report, as quoted in Appendix F are an integral part of this report.

SHAHEEN & PEAKER LIMITED

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Appendix D

Tunnelman's Ground Classification and Probable Working Conditions

Table 1
Tunnelman's Ground Classification
and Probable Working Conditions

Soil Classification	Representative Soil Types	Tunnel Working Conditions
Hard	Very hard calcareous clay; cemented sand and gravel	Tunnel heading may be advanced without roof support
Firm	Loess above GWT; various calcareous clay with low plasticity	Tunnel heading may be advanced without roof support. Permanent support can be constructed before the ground will start to move.
Slow ravelling and Fast ravelling	Fast raveling occurs in residual soils or in sand with clay binder below the GWT. Above the GWT, the same soils may be <u>Slow Ravelling</u> or even <u>Firm</u> .	Chunks of material may drop out of the crown or the sides some time after the ground has been exposed. In <u>Fast Ravelling</u> ground, the process starts within a few minutes; otherwise, it is classed as <u>Slow Ravelling</u> .
Squeezing	Soft or medium soft clay	Ground slowly advances into tunnel without fracturing and without perceptible increase of water content in ground surrounding the tunnel.
Swelling	Heavily pre-compressed clays with a plasticity index greater than 30. Sedimentary formations containing layers of anhydrite.	Like squeezing ground, moves slowly into tunnel, but the movement is associated with a very considerable volume increase in the ground surrounding the tunnel.
Cohesive Running and Running	Occurs in clean, fine moist sand Occurs in clean, coarse or medium sand above the GWT	Removal of the lateral support of any surface rising at an angle of more than about 34° to the horizontal is followed by a "run", whereby the material flows like granulated sugar until the slope angle is approx. 34°. If the "run" is preceded by a brief period of raveling, the ground is called <u>Cohesive Running</u> .
Very Soft Squeezing	Clays and silts with high plasticity indices	Ground advances rapidly into the tunnel in a plastic flow
Flowing	Any ground below the GWT that has an effective grain size in excess of about 0.005 mm	Flowing ground moves like a viscous liquid. It can invade the tunnel not only through the roof and the sides, but also through the invert. If the flow is not stopped, it will eventually completely fill the tunnel
Bouldery	Boulder glacial till; riprap fill; some land slide deposits, some residual soils. The matrix between boulders may be gravel, sand, silt, clay and in any combination.	Problems incurred in advancing shield or in fore poling; blasting or hand mining ahead of machine may become necessary.

Appendix E

Non-Standard Special Provision (NSSP) for Jack and Bore

JACKING AND BORING FOR 375MM, 450MM, 525MM, 600 MM, 825MM, 1200MM CONCRETE STORM PIPE

Special Provision

March 2003

Construction Specification for Jacking and Boring

416.02 REFERENCES

Section 416.02 of OPSS 416, February, 1990, is amended by the addition of the following:

OPSS 510

416.03 Added - "DEFINITIONS"

Section 416.03 of OPSS 416, February, 1990, is amended by the addition of the following:

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to Jacking and Boring of sewers, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s) of conformance.

416.04 SUBMISSION AND DESIGN REQUIREMENTS

Section 416.04 of OPSS 416, February, 1990, is amended by the addition of the following:

416.04.01 Site Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator a geodetic surveys of the centre line and offset 5 metres in each direction at 10 metre intervals of the proposed jack and bore horizontal alignment. The survey shall include station, offset and elevation recorded. This information shall be submitted at the following intervals,

- At least one week prior to commencement of the work.
- Weekly during jack and bore operations
- Bi-weekly after completion of the work for one month

Each report shall include all survey data collected to date and identify any settlement that may have occurred in comparison to survey data collected prior to the commencement of the work.

416.04 Submission and Design Requirements

Section 416.04 of OPSS 416, February, 1990, is amended by the addition of the following:

416.04.01 Submission of Certificate of Conformance

The Contractor shall submit details of the sequence and method of construction to the Quality Verification

Engineer for review and stamping. The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the construction procedures are in conformance with the requirements and specifications of the contract documents.

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations and prior to commencement of each subsequent operation:

- Site Surveying (at intervals noted in Section 416.04.01 of this Special Provision)
- Excavation for pits including dewatering of excavation
- Jacking of Casing
- Installation of Sewer
- Grouting Operations

Each Certificate of Conformance shall state that the work has been carried out in general conformance with the contract documents, specifications and/or stamped working drawings.

In addition, upon completion of the installation of the concrete storm pipe, the Contractor shall submit to the Contract Administrator a **final** Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate shall state that the concrete storm pipe has been installed in general conformance with the stamped working drawings and contract documents

416.07 CONSTRUCTION

416.07.03 Dewatering

Subsection 416.07.03 of OPSS 416, February, 1990, is amended by addition of the following paragraph:

The Contractor shall assume that overburden is comprised of cohesionless soils submerged below the groundwater table. The Contractor is advised that cohesionless soil submerged below the groundwater table is susceptible to sloughing and cave-in.

After the dewatering operation, the excavation shall be inspected and certified by the Quality Verification Engineer. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

416.07.04 Pipe Installation

Subsection 416.07.04 of OPSS 416, February, 1990, is amended by the addition of the following paragraph:

The pipe installation shall be inspected and certified by the Quality Verification Engineer. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

416.07.05 Grouting

Subsection 416.07.05 of OPSS 416, February, 1990, is amended by the addition of the following paragraph:

The grouting operations and installation shall be inspected and certified by the Quality Verification Engineer. Immediately after the inspection and prior to commencement of subsequent activity, a certificate of conformance shall be submitted to the Contract Administrator.

Section 416.07 is modified by the addition of the following new subsections:

416.07.08 Removal of Boulders

The Contractor is alerted that boulders and cobbles should be anticipated at the site. Accordingly, the Contractor shall address the removal of boulders and cobbles in the proposed method of construction. The Contractor shall immediately inform the Contract Administrator of any obstruction encountered.

416.07.09 Criteria for Assessment of Roadway Subsidence

Based on the monitoring of settlement as specified in subsection 416.04.01, the following represent trigger levels that define levels of settlement and corresponding action:

Review Level: If a maximum value of 6 mm relative to the baseline readings is reached, the method, rate or sequence of construction, or ground stabilization measures should be reviewed or modified to mitigate further ground displacements.

Alert Level: If a maximum value of 10 mm relative to the baseline readings is reached, the Contractor may be required to cease construction operation or to execute pre-planned measures to secure the site, to mitigate further unacceptable settlements and to assure safety of public.

Appendix F

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Shaheen & Peaker Limited at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. 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. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

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.

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 conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

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. Shaheen & Peaker Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.