

**FOUNDATION INVESTIGATION & DESIGN REPORTS
HIGHWAY 112 ONR OVERHEAD BRIDGE REPLACEMENT
4.0 KM SOUTH OF HIGHWAY 66
KIRKLAND LAKE, ONTARIO
G.W.P. 140-88-00
SITE: 47-015
GEOCRES NO. 42A-66**

Prepared For:

LEA CONSULTING LIMITED

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1104B
September 18, 2007**



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1. INTRODUCTION

Highway 112 is a two-lane rural highway which provides a southerly link from Swastika (Kirkland Lake) to Highway 11 north of Englehart. The proposed improvements to Highway 112 include the reconstruction of the existing ONR overhead bridge structure.

The site is located about 4 km south of Highway 66, near Swastika (Kirkland Lake), Ontario.

Shaheen & Peaker Limited (S&P) was retained by LEA Consulting Limited to carry out a foundation investigation for the replacement of the existing ONR overhead bridge structure.

The findings of the investigation are presented in this report.

2. SITE DESCRIPTION AND GEOLOGY

The site of this investigation for Highway 112/ONR structure is located approximately 4 km south of Highway 66 junction, which is located west of the Town of Kirkland Lake, Ontario. At this location, Highway 112 has a distinct 'S' shaped curve over a single Ontario Northland Railway (ONR) track. The existing single span ONR overhead bridge structure is supported on exposed bedrock.

The terrain in the general area of the site is bedrock controlled and exhibits significant local relief, especially in the vicinity of the Murdock Creek valley, located some 150 m north of the ONR structure. The vertical alignment of Highway 112 is classified as moderate to significantly rolling. The existing top of rail elevation under the existing Highway 112 is approximately 309.7 m (which is cut into the bedrock by about 2 to 3 m – see photographs in Appendix E).

The grade at the ONR site falls from east to west, from about Elevation 312 m at the existing bridge to about Elevation 307 m some 170 m to the west. In addition, the grade at the new ONR location also falls from south to north towards the Murdock Creek, from Elevation 312 m to about Elevation 297 m.

North of the Murdock Creek valley is a rock outcrop where the elevation of the bedrock surface is about 35 m above the elevation of Murdock Creek.

Quaternary geology map of Kirkland Lake area indicates the bedrock cover in the general area of the site is an extensive but discontinuous thin drift cover consisting of glaciofluvial (sand, gravel cobbles, boulders), glaciolacustrine (clay and silt) and glacial till (silty sand till) deposits. The overburden generally varies in depth from zero to about 6 m but can be expected to be deeper to the west of the ONR structure and further north towards the Murdock Creek.

Published geological information indicates that the bedrock in the area is Precambrian in age. In particular, the bedrock at the site belongs to the Keewatin period. The Keewatin series is largely composed of lavas ranging in composition from acid andesite to basaltic rocks; these are commonly called "greenstone" in the field, as well as, gneiss, meta-volcanic rocks. They are typically fine to medium-grained, dark green rocks and dark grey, but the texture varies with the thickness of the lava flows. Throughout the Keewatin series, there are bodies of gabbro and diorite. Quartz veins are also common.

The surface drainage within the project limits is generally towards the Murdock River.

3. METHOD OF INVESTIGATION

The fieldwork for this investigation was carried out during the period of May 1-12, 2007 and consisted of drilling and sampling a total of eleven boreholes (Boreholes SP1 through SP10 and SP12). The plan locations of the boreholes are given on Drawing No. 1.

The boreholes were advanced using solid-stem, continuous flight augers with a track-mounted drill owned and operated by Landcore Drilling of Chelmsford, Ontario, under the full time supervision of a Geotechnical Professional Engineer from S&P.

The depths of the boreholes ranged from 3.8 to 9.5 m. In the boreholes, sampling in the overburden was effected 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.

All boreholes were extended to refusal on the augers. In Boreholes SP1, SP4, SP7 and SP10, coring by diamond drilling was conducted in order to investigate the nature of the material on which refusal was encountered, to verify the presence of bedrock and to investigate the nature of the bedrock. The length of coring in each of the four boreholes ranged from 2.6 to 3.4 m.

Groundwater conditions in the boreholes were observed during drilling and at the completion of each borehole. The boreholes were grouted as per MTO requirements upon their completion.

The subsurface stratigraphy encountered in the boreholes, type of samples and sampling depths, N-values, together with coring data are presented on the Record of Borehole Sheets, in Appendix A of this report.

A laboratory testing programme, consisting of natural moisture content measurements, Atterberg Limits, grain-size analyses was performed on selected soil samples. Rock samples from Borehole SP10 were subjected to point load tests to obtain a measure of the hardness condition of the bedrock. The results of the laboratory are presented on the appropriate Record of Borehole Sheets and also in Appendix B.

In addition to the boreholes, four test pits (TP1 through TP4) were dug using a backhoe. The results of the test pits are given in Appendix C.

The borehole locations were established in the field by LEA's surveyors, who also provided to us the ground surface elevations at the boreholes and the test pits, along with coordinates.

4. SUBSURFACE CONDITIONS

The boreholes were drilled from the surface of the existing road from about El. 317.7 – 318.0 m and contacted about 3 to 5 m of pavement and embankment fill underlain by a 0.7 to 1.5 m thick sandy silt/silt/clayey silt deposit typically to the surface of auger refusal on the augers (i.e. boulders or bedrock) or proven bedrock (Boreholes SP1, SP4, SP7 and SP10) at depths of about 4 to 6 m below the road surface or at El. 314.2 (Borehole SP12) to 311.6 m (Boreholes SP7 and SP10). In some of the boreholes, a 0.1 to 0.7 m thick mantle of coarse grained sand till was found immediately above the refusal or bedrock surface.

Details of the subsurface conditions encountered in the boreholes drilled are presented on the Record of Borehole Sheets in Appendix A. Four supplementary test pits were also put down beyond the toe of the road embankment and the results of the test pits are presented in Appendix C. The individual strata encountered in the boreholes are briefly described in the following paragraphs.

4.1 FILL

4.1.1 ASPHALTIC CONCRETE

Boreholes SP1 through SP10, which were drilled from the top of the paved road surface, contacted a 150 to 300 mm thick asphaltic concrete layer (typically 225 mm).

4.1.2 PAVEMENT FILL

Underlying the asphalt (Boreholes SP1 through SP10) or the embankment surface (SP12) granular pavement fill was encountered, which extended to 0.3 to 0.9 m below the ground surface.

4.1.3 GRANULAR EMBANKMENT FILL

Underlying the granular pavement fill, all eleven boreholes encountered a granular embankment fill to depths ranging between 2.7 m (El. 315.0 m) at Borehole SP4 to 5.0 m (El. 312.8 m) at Borehole SP7. The granular embankment fill ranges from sand (primarily fine sand) with traces of gravel (see grain-size distribution curves in Figure B-1 in Appendix B) somewhat coarser granular fill (i.e. sand with some gravel, see Figure B-2) to ever coarser granular fill (i.e. gravelly sand to sand & gravel, see Figure B-3). The presence of occasional cobbles and boulders in the fill was inferred during the drilling. In Borehole SP4, the fill was found to be silty.

In Boreholes SP1 through SP4, which were put down adjacent to the east abutment of the existing railway bridge, the recorded N-values ranged from 2 to 11 blows/0.3 m (typically 2 to 6 blows/0.3 m) indicating a very loose to compact but typically very loose to loose condition. From these results, it appears that the fill placed in this area had not received a systematic compaction when it was first placed. In addition, its grain-size distribution is relatively finer (i.e. primarily fine sand with traces of medium to coarse sand and gravel, see Figure B-1, in Appendix B), in comparison with other areas.

Boreholes SP5 and SP6, which were drilled further east, the recorded N-values range from 4 to 24 blows/0.3 m, indicating a very loose to compact condition. These results indicate that fill may have received some compaction at various depths. In this area, the grain-size distribution of the fill is somewhat coarser (i.e. sand with some gravel, see Figure B-2). The recorded values are generally between and 18 blows/0.3 m and

In Boreholes SP7, SP8, SP9 and SP10, which were put down adjacent to the west abutment of the existing bridge, the fill is relatively coarser (see Figures B-2 and B-3) than the fill encountered at the east abutment location. In these boreholes, the recorded N-values range from 5 to 24 blows/0.3 m (typically 9 to 19 blows/0.3 m) indicating a loose to compact condition. In this general area, the recorded N-values are relatively higher than the

east abutment location and it appears that the fill has received some minimal compaction when it was first placed.

Further east in Borehole SP12, the recorded N-values range from 3 to 5 blows/0.3 m indicating a very loose condition and it appears that no systematic compaction has been applied.

4.1.4 OTHER EMBANKMENT FILL

Underlying the granular embankment fill in Boreholes SP2 and SP4, a 0.5 to 0.8 m thick fine-grained granular (i. e. non-cohesive) silt and sandy silt fill layer or pocket was contacted. Based on N-values of 5 and 26 blows/0.3 m, which were recorded, the relative density of the fill is described as very loose to compact.

4.2 BURIED TOPSOIL

A 0.15 to 0.3 m thick buried topsoil layer was contacted underlying the embankment fill in Boreholes SP2, SP4, SP6, SP8, SP9, SP10 and SP12, at depths ranging from 3.4 to 4.6 m below the ground surface or between El. 314.4 and 313.2 m.

4.3 SILT/SANDY SILT/CLAYEY SILT

Underlying the embankment fill and buried topsoil, in all boreholes, except for Boreholes SP3 and SP12, a native silt deposit was contacted. The silt deposit ranges from a basically fine-grained granular material consisting of sandy silt to silt with silty sand layers to cohesive silt with clayey silt and silty clay layers.

This silt deposit was contacted at depths ranging from 3.4 to 5.0 m or at El. 314.3 to 312.8 m and was found to extend to depths of 4.3 to 5.8 m or El. 313.5 to 312.0 m. The thickness of the deposit was found to range from 0.5 m (in Borehole SP4) to 2.1 m (Borehole SP8).

The grain-size distribution of a silty clay layer in the cohesive silt deposit is given in Figure B-4 in Appendix B.

Atterberg limits tests performed on two samples from the clayey zones of the deposit yielded the following index values (see Figure B-5).

Liquid Limit:	27 – 31%
Plastic Limit:	16 – 21%
Plasticity Index:	10 - 11%
Natural Moisture content:	18-27%

Standard Penetration tests performed in this deposit yielded N-values which range from 7 to 35 blows/0.3 m. These results indicate a compact to dense relative density within the non-cohesive (granular zones – i.e. silt, sandy silt, silty sand) to firm to very stiff consistency in the cohesive (i.e. cohesive silt/clayey silt/silty clay) zones/layers.

It should be pointed out that in two of the boreholes (i.e. Boreholes SP5 and SP8) the deposit contains traces of gravel. This is considered to a transition zone from the silt to the underlying glacial till.

4.4 SAND TILL

At Boreholes SP4, SP5, SP7, SP8 and SP10, a 0.1 to 0.7 m thick mantle of coarse-grained granular glacial till was contacted immediately over the surface of the bedrock or cobbles/boulders/broken rock. The deposit consists of a heterogenous mixture of sand with some gravel, silt and clay size particles. The grain-size distribution curve determined from a sample from Borehole SP4 is as follows (see Figure B-6 in Appendix B).

Gravel	14%
Sand	67%
Silt & Clay	19%

Visual examination of the soil samples recovered showed that in some of the boreholes, a greater percentage of gravel prevails in comparison with Borehole SP4. In addition, the presence of cobbles and boulders and rock fragments can be expected in this basal deposit owing to its mode of deposition.

From the recorded N-values which range from 24 to in excess of 50 blows/0.3 m, the deposit is considered to be compact to very dense, but typically compact.

4.5 COBBLES AND BOULDERS

Boreholes SP1, SP4, SP7 and SP10 were extended by coring below the refusal depths after encountering refusal to further augering. In one of these boreholes (Borehole SP10) refusal was found to be due to cobbles and boulders, while in the other three the refusal was found to be on the surface of bedrock. In Borehole 10, the thickness of the layer of cobbles and boulders was found to be 0.3 m below which bedrock was contacted.

From these findings, it can be surmised that cobbles and boulders may be found immediately above bedrock at some of the other borehole locations which were terminated upon encountering refusal on the augers.

4.6 BEDROCK

Refusal to augering in the boreholes was encountered at Elevations ranging from 314.2 m in Borehole SP12 to 311.6 m in Borehole SP7. Four of the boreholes (BH SP1, SP4, SP7 and SP10) were cored. At Boreholes SP1, SP4 and SP7, the refusal was found to be on the surface of the bedrock at El. 312.8, 312.7 and 311.6 m, respectively, while at Borehole SP10, a 0.3 m thick cobble/boulder zone was found, before encountering the surface of the bedrock at El. 311.6 m.

In Boreholes SP1 SP4, SP7 and SP10, the bedrock was cored for a vertical distance of 2.6 to 3.3 m. In this zone, the bedrock was identified as a meta-volcanic gneiss formation with metamorphosed extrusions. The colour is generally dark grey with grey to light grey and occasional greyish green zones. The presence of occasional milky white veins was also noted. The formation belongs to the Precambrian Era.

The total rock core recovery (TCR) was 100% (except for a 1.5 m long core in Borehole SP 1 where the percentage of core recovery was 90% and a 0.76 m long run in Borehole SP 7, where the percentage of core recovery was recorded as 80%), indicating a generally sound rock. Rock quality designation (R.Q.D.) values ranged from 0 to 92%. The zero RQD was found in the upper 1.9 m of the bedrock cored in Borehole SP7. A value of 20% was recorded in the upper 1.5 m of the core from SP1, while the remaining values ranged from 62 to 92%. These RQD values indicate a very poor rock quality (i.e. 0% in the upper zones of Borehole SP7 and 20% in the upper 1.5 m in Borehole SP1) in the upper zones of Boreholes SP7 and SP1. In the remaining cores, values of 62 to 92% indicate a fair to excellent but typically fair quality.

Point Load Tests were performed on the core samples recovered from Borehole SP 10. Unconfined compressive strengths inferred from these Point Load Tests are all in excess of 200 MPa. Unconfined compressive strength inferred from Point Load Tests, performed by Thurber Engineering Limited, on cores obtained from similar bedrock at a site some 175 m to the east of the existing bridge site are reported to range from 114 to 232 MPa, with an average value of 185 MPa, while at values in excess of 200 MPa, the correlation of Point Load Test results to unconfined compressive strength is not reliable, nevertheless, these values indicate that the intact rock can be classified as very strong.

4.7 GROUNDWATER CONDITIONS

Water levels in the open boreholes were observed during the drilling and at the completion of each borehole. In addition, water was used in four of the eleven boreholes to facilitate rock coring and, as well, the boreholes were grouted upon completion. For these reasons, the water levels shown on the individual Record of Borehole sheets may not necessarily represent stabilized groundwater levels.

Based on the data collected and the moisture contents of the soil samples recovered from the boreholes, it is our opinion that at the time of our investigation, the groundwater level in the boreholes ranged from El. 314.1 to 312.8 m but was typically at El. 313 m or about 4.5 to 5.0 m below the road surface grade.

It should, however, be pointed out that the groundwater table at the site can be expected to be subject to seasonal fluctuations and fluctuations in response to major weather events.


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ZO:tr/idrive

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APPENDICES

APPENDIX G: BEDROCK ELEVATIONS AT BOREHOLE AND TEST PIT LOCATIONS

APPENDIX H: LIMITATIONS OF REPORT

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5. DISCUSSION & RECOMMENDATIONS

The existing bridge over a single Ontario Northland Railway track was constructed in 1938. It is a single span (approximately 14.1 m clear span), two-lane bridge with a slab on steel girder superstructure. It crosses the railway at a 15° skew angle (see photographs in Appendix E). As shown on the photographs, the bridge abutments are founded on the bedrock.

The existing structure will be replaced with a wider (12.6 m wide) single span bridge. The new bridge will be somewhat longer than the existing bridge (i.e. 21.5 m long). It will be a reinforced concrete structure. The location of the west abutment will be very close to the existing bridge west abutment while the east abutment will be located several meters further east.

The top elevation of the existing bridge is at about El. 318.0± m, while the top of the tracks underneath the bridge is at about El. 309.7 m. The new bridge will be about 0.3 m higher.

As the single span bridge will be re-built at the existing location, to maintain traffic on Highway 112, a two-stage construction is being planned. First, during construction, half of the existing bridge will be demolished, followed by the construction of half of the new bridge, while the traffic is maintained through remaining half of the existing bridge. The traffic will subsequently be diverted to the newly built (half) structure, while the other half of the existing bridge is demolished and the remaining half of the new bridge is built. This staging will involve significant roadway protection (i.e. shoring, etc.) during the construction.

This investigation has shown the presence of 3 to 5 m of embankment fill followed by typically 1.5 m of natural overburden to the surface of bedrock/inferred bedrock at generally between El. 313.0 and 311.6 m). Proven or inferred bedrock surface elevations at the borehole and test pit locations are given on Drawing G-1 in Appendix G. The groundwater table at the time of our investigation was inferred to be at about El. 313 m (at or about 1 m above the bedrock surface) but would be subject to fluctuations.

5.1 FOUNDATIONS

The boreholes show that at the proposed abutment locations, beneath the embankment fill and buried topsoil, there is little overburden of variable compactness condition or in the case of cohesive soils a consistency of generally firm to stiff. Supporting the new structure on the overburden is not recommended since the overburden is considered unsuitable for this purpose as well since the existing bridge is supported on bedrock, at most locations the overburden would have been stripped to construct the existing bridge. As well existing footings will interfere with this approach. It is, therefore, recommended that the new bridge be supported on bedrock, similar to the existing bridge based on reliability and economics. A summary of foundation alternatives is given in Table 5.1.1 below.

Table 5.1.1
Summary of Foundation Alternatives

Foundation Type	Comments	Recommendations
o Normal spread footings on bedrock	Well suited for the prevailing site conditions	Recommended based on cost and reliability.
o Spread Footings on compacted Granular 'A' pad	The site will need to be stripped to or immediately above bedrock to build the granular pad.	Possible but unlikely to be economical, as well as increasing the span of the bridge.
o Drilled caissons	Will require socketing into the bedrock.	Considered to be reliable but considerably less cost-effective than normal spread footings on bedrock
o Driven piles	Not suitable for the prevailing subsurface conditions	Not recommended.

5.1.1 SPREAD FOOTING FOUNDATIONS ON BEDROCK

Both abutments can be founded on sound bedrock. For this purpose, all loose or weathered rock under the footprint of the footing should be removed and replaced with concrete. Mass concrete may be placed to raise the grade to the founding level, where necessary.

5.1.2 EAST ABUTMENT

The east abutment can be founded on sound bedrock. For this purpose, all loose or weathered rock under the footprint of the footing should be removed and replaced with concrete. Mass concrete may be placed to raise the grade to the founding level, where necessary.

Table 5.1.2.1
East Abutment Spread Footing on Sound Bedrock

Foundation Location	Reference Borehole/Existing Ground Surface Elevation (m)	Recommended Highest Founding Depth Below Existing Grade/Elevation (m)	Remarks
East Abutment South Side	SP1 317.7	5.0/312.7	Cored borehole*
East Abutment South Side	SP2 317.8	5.1/312.7	Borehole advanced to refusal on augers only (i.e. no coring)
East Abutment North side	SP3 317.7	5.0/312.7	Borehole advanced to refusal on augers only (i.e. no coring)
East Abutment North side	SP4 317.7	5.0/312.7	Cored borehole*

*Cored borehole provides more reliable data.

Based on the borehole results, for design purposes, the following Canadian Highway Bridge Design Code (C.H.B.D.C.) geotechnical resistances may be used.

Factored Bearing Resistance at U.L.S. = 5,000 kPa
Bearing Resistance at S.L.S. will not govern

It should be noted that in between and beyond borehole locations, the bedrock surface and the depth to the suitable bedrock surface may vary considerably. Additionally, auger refusal depths may be due to refusal on cobbles or boulders within the overburden and thus the actual bedrock surface may be lower than given by the refusal depths in Boreholes SP2 and SP3.

All footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer who is familiar with the findings of this investigation. As mentioned before, all footings should be founded on sound bedrock. For this purpose, all loose or weathered rock under the footprint of the footing should be removed to the surface of the sufficiently sound bedrock and replaced with concrete.

Bedrock would be prone to deterioration due to the opening of existing joints or fractures in the bedrock as a result of frost action. Provided that surface water is diverted away from the footings, full frost protection need not be provided for footings placed on massive, sound bedrock. If however the bedrock is not massive and water can accumulate in the joints or fractures of the rock (thus causing deterioration of the founding medium by expansion due to freezing) then there may be a requirement to provide up to full frost protection (i.e. 2.4 m). For this purpose, the proposed bearing surface should be inspected by qualified engineering personnel. If the rock is not massive, then the excavation can be extended deeper until acceptable rock is found or to the full frost protection depth of 2.4 m, whichever comes first. The final decision regarding this should rest on the engineer in the field inspecting the

exposed rock. Sealing (e.g. grouting) of the fractures in the rock to a depth of about 0.6 m may also be considered to prevent water from entering the rock underneath the foundation.

Sliding resistance can be provided by penetrating into the bedrock (i.e. keying-in and utilizing passive rock resistance), utilizing the sliding resistance between concrete and bedrock, shear in grouted dowels and/or rock anchors. For the evaluation of the sliding resistance of the foundation (C.H.B.D.C. 6.7.5) the ultimate angle of friction between the underside of the foundations and the clean, intact bedrock surface (or between concrete surfaces) can be taken as 30 degrees. If additional horizontal resistance is required or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered. Such measures would be required if the rock surface is smooth and/or inclined. In addition, the surface of the bedrock can be chiseled (i.e. roughened), increasing the ultimate angle of friction to 35 degrees.

If there are net uplift forces which are to be resisted by rock anchors, or for increasing sliding resistance, the factored rock/grout bond capacity at U.L.S. can be taken as 500 kPa and S.L.S. will not govern. The upper 0.3 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 1.5 m into the sound rock. The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

Under the inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6.7.4 of C.H.B.D.C.

In order to ensure the stability of the abutment, the footing should be located far enough from the face of the rock along the railway to outside a plane defined by 1/2H:1V plus a horizontal distance of 0.5 m from the toe of the rock cut. This will, however, unlikely be a problem since with the present configuration the east abutment is to be moved further away (i.e. easterly) from the existing.

5.1.3 WEST ABUTMENT

Based on the results of Boreholes SP7 through SP10, the following table details the recommended bearing surface elevations on sufficiently sound bedrock.

Table 5.1.2.2
West Abutment Spread Footing on Sound Bedrock

Foundation Location	Reference Borehole/Existing Ground Surface Elevation (m)	Recommended Highest Founding Depth Below Existing Grade/Elevation (m)	Remarks
West Abutment South Side	SP7 317.8	8.1/309.7	Cored borehole*
West Abutment South Side	SP8 317.8	6.2/311.6-8.1/309.7	Borehole advanced to refusal on augers only (i.e. no coring)
West Abutment North side	SP9 317.8	6.0/311.8	Borehole advanced to refusal on augers only (i.e. no coring)
West Abutment North side	SP10 317.8	6.2/311.6	Cored borehole*

*Cored borehole provides more reliable data.

While the rock cores obtained from Borehole SP10 showed a competent rock, in Borehole SP7, the upper 1.9 m of the cored rock (i.e. to El. 309.7 m) is highly disintegrated, despite the fact that recovery was high (i.e. 100% between El. 311.6 m and 310.5 m and 80% between El. 310.5 and 309.7 m). This may be a local condition. For this reason, the following can be considered. The bottom of the footing can be designed to be at El. 311.6 m but depending on the conditions encountered, the contractor should be prepared to remove all the weathered and highly fractured rock and replace with mass concrete. This would be decided by means of careful field inspection. An NSSP should be prepared for this purpose, warning the contractor of this likelihood. In this case, the following geotechnical resistances may be used for the approved rock surface.

Factored Bearing Resistance at U.L.S. = 4,000 kPa
Bearing Resistance at S.L.S. will not govern

Alternatively, the footing can be designed to extend to El. 309.7 m in which case the U.L.S. value would be increased to 5,000 kPa and S.L.S. will not govern.

It should be noted that in between and beyond borehole locations, the bedrock surface and the depth to the suitable bedrock surface may vary considerably. Additionally, auger refusal depths may be due to refusal on cobbles or boulders within the overburden and thus the actual bedrock surface may be lower than given by the refusal depths in Boreholes SP8 and SP9.

In any case, all footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer who is familiar with the findings of this investigation. As mentioned before, all footings should be founded on sound bedrock. For this purpose, all loose or weathered rock under the footprint of the footing should be removed to the surface of the sufficiently sound bedrock and replaced with mass concrete.

Bedrock would be prone to deterioration due to the opening of existing joints or fractures in the bedrock as a result of frost action. Provided that surface water is diverted away from the footings, full frost protection need not be provided for footings placed on massive, sound bedrock. If however the bedrock is not massive and water can accumulate in the joints or fractures of the rock (thus causing deterioration of the founding medium by expansion due to freezing) then there may be a requirement to provide up to full frost protection (i.e. 2.4 m). For this purpose, the proposed bearing surface should be inspected by qualified engineering personnel. If the rock is not massive, then the excavation can be extended deeper until acceptable rock is found or to the full frost protection depth of 2.4 m, whichever comes first. The final decision regarding this should rest by the engineer in the field inspecting the exposed rock. Alternatively, sealing of the fractured rock immediately underneath the foundation may be considered, as was mentioned for the east abutment.

Sliding resistance can be provided by penetrating into the bedrock (i.e. keying-in and utilizing passive rock resistance), utilizing the sliding resistance between concrete and bedrock, shear in grouted dowels and/or rock anchors. For the evaluation of the sliding resistance of the foundation (C.H.B.D.C. 6.7.5) the ultimate angle of friction between the underside of the foundations and the clean, intact bedrock surface (or between concrete surfaces) can be taken as 30 degrees. If additional horizontal resistance is required or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered. Such measures would be required if the rock surface is smooth and/or inclined. In addition, the surface of the bedrock can be chiseled (i.e. roughened), increasing the ultimate angle of friction to 35 degrees.

If there are net uplift forces which are to be resisted by rock anchors, or for increasing sliding resistance, the factored rock/grout bond capacity at U.L.S. can be taken as 500 kPa and S.L.S. will not govern. The upper 0.3 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 1.5 m into the sound rock. The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

Under the inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6.7.4 of C.H.B.D.C.

As was mentioned before, in order to ensure the stability of the abutment, the footing would need to be located far enough from the rock cut face along the railway to outside a plane defined by 1/2H:1V plus a horizontal distance of 0.5 m from the toe of the rock cut. For this reason, it may be advantageous to lower the footing elevation to El. 309.7 m, especially at the northwest corner area of the abutment in order to meet this criterion. The rock portion of the slope can be maintained at the existing slopes but depending on the orientation of joints and fractures, rock bolting of the exposed rock face may possibly be required.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation Standards.

Granular backfill to be placed behind the abutment walls and wingwalls should conform to the minimum requirements illustrated in OPSD 3501.00. The granular backfill should conform to OPSS 1010 for either Granular 'A' or 'B' Type I and Type II. To maintain free draining characteristics in these granular fill materials, the maximum percentage passing the No. 200 sieve (75 μ m) should be limited to 5%.

The backfill should be placed in accordance with OPSS 501. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3501.00 to maintain the granular fill in a drained condition. The subdrain should be directed to a positive outlet or highway drainage system.

Computation of earth pressures acting against rigid abutment walls and any wingwalls should be in accordance with the Canadian Highway Bridge Design Code (C.H.B.D.C.), Current Edition. For design purposes, the following properties can be assumed for backfill.

Compacted Granular 'A' and Granular 'B' Type II

Angle of Internal Friction $\phi=35^\circ$ (unfactored)

Unit weight = 22 kN/m³

Coefficient of Lateral Earth Pressure

$K_a = 0.27$

$K_b = 0.35$

$K_o = 0.43$

$K^* = 0.45$

Compacted Granular 'B' Type I

Angle of Internal Friction $\phi = 32^\circ$ (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure

$K_a = 0.31$

$K_b = 0.41$

$K_o = 0.47$

$K^* = 0.57$

Rock Fill

Angle of Internal Friction $\phi = 40^\circ$ (unfactored)

Unit Weight = 19 kN/m³

Coefficient of Lateral Earth Pressure

$K_a = 0.22$

$K_b = 0.30$

$K_o = 0.36$

$K^* = 0.40$

NOTE:

K_a is the coefficient of active earth pressure

K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction efforts

K_o is the coefficient of earth pressure at rest

K^* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

These values are based on the assumption that the backfill behind the retaining structure is free-draining granular material and adequate drainage is provided. As well, it is also assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or some movements can occur such that the active state of earth pressure can develop. In this case since all foundations will be supported on bedrock (i.e. unyielding), at rest pressures should be used, as per Clause 6.9.2 of CAN/CSA-S6-00 C.H.B.D.C., Current Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6.9.2 of CAN/CSA-S6-00 C.H.B.D.C., Current Edition. The use of vibratory compaction equipment behind the retaining walls should be restricted in size as per current MTO practice.

If rock fill is used for backfill, special care is required to prevent damage to the retaining structures. In such a case, a cushion of Granular 'A' or Granular 'B' Type II, material or finely graded rock fill (e.g. less than 250 mm nominal diameter) should be placed between the structure and the rock fill. This cushion should be at least 0.45 m wide and if Granular 'A' or Granular 'B' Type II is used, proper filtering should be provided to prevent the loss of finer particles from the Granular 'A' or Granular 'B' Type II cushion into the coarse rock fill. In accordance with OPSS 902.07.07, rock fill should be placed in such a manner that the structure is not damaged. End dumping of rock backfill against a structure should not be permitted.

5.3 EMBANKMENT STABILITY

Judging from the original grade (bottom of fill) encountered at the borehole locations, original grades at the site were at about El. 313-314 m. Since the new bridge deck elevation will be about El. 318.4 m, the height of the approach fills will be about 4 to 5 m.

Provided that all organic soils, unsuitable fill, weak and otherwise unsuitable materials are removed as per MTO standards before placing the fill, no instability problems are anticipated due to foundation conditions for the proposed height of embankments (i.e. approximately 4 to 5 m). Conventional embankment slopes of 2 horizontal in 1 vertical would be stable, provided that the subgrade is properly prepared.

All organic and otherwise unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1:1 from the toe of the proposed embankment. After stripping, the exposed subgrade should be inspected and approved. It should then be compacted from the surface using a suitable compactor. For preliminary estimating purposes, the average thickness of unsuitable materials can be assumed to be 0.3 m, based on the results of TP1 through TP4.

Proper benching of the existing embankment slope should be implemented when widening the embankment, as per MTO procedures and in accordance with OPSD 208.01.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. select subgrade materials or Granular 'B' – OPSS 1010). In as much as possible, the fill used should match the existing embankment fill, especially within the frost zone. The embankment fill should be placed on the approved and properly rolled subgrade in lifts not exceeding 300 mm when loosely placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density.

Embankment loadings would likely result in a settlement of the order of 20 mm due to the settlement of foundation soils. About one-third of this settlement should take place within one month, with the majority of the remaining within the next 10 months.

One problem that may arise is the state of compaction of the existing embankments. The materials used for the construction of the embankments, in most cases, appear to be satisfactory, consisting of sand, with occasional silt zones. The sand ranges from silty fine sand to sand & gravel, but in most part, sand with some gravel.

At the location of Boreholes SP1 through SP4 (i.e. proposed east abutment location), the embankment material was found to consist of typically sand (mostly in fine sand range of gradation) with traces of gravel. It appears that relatively finer granular fills were used in this area in comparison with other borehole areas. The recorded N-values are typically 2 to 6 blows/0.3 m (see Section 4.1.3 of this report). These results lead us to believe that when

the bridge was built, the fill was placed without any systematic compaction. If uncompacted soils are left in place some future settlements may occur. While most of the soil encountered at these borehole locations will be removed for the construction of the abutment, these conditions may continue to exist further east.

Further east, in Boreholes SP5 and SP6 the embankment fill is somewhat better graded (i.e. higher gravel content), as well, the N-values are relatively higher. Here, the soil may have received some degree of compaction (e.g. heavy construction equipment movement during construction) and this is possibly why the recorded asphaltic concrete thickness at these two borehole locations (i.e. Boreholes SP5 and SP6) are less in comparison with those recorded at Borehole SP1-SP4 locations). Based on these findings, you may consider recompacting the existing fill to a distance of about 10 m beyond the east abutment.

As was discussed before, embankment fill soils encountered in the boreholes drilled on the east side appear to be basically granular materials and would be suitable for re-use and to widen the embankments. Depending on the conditions at the time of the construction, however, some moisture conditioning may be required to achieve a high degree of compaction. However, the silt fill, encountered below 2.9 m in Borehole SP2 and below 2.7 m in Borehole SP4, should not be re-used but rather treated as waste materials, as well as the topsoil and the natural overburden soils, with the exception of the sand till. The sand till can be re-used after adjusting its moisture or mixing it with other dryer embankment fill materials to be placed.

On the west side, at Boreholes SP7 through SP10 locations, the embankment fill appears to be relatively well-graded and the N-values are also relatively higher. Some of this can be attributed to the influence of higher gravel content. As well, N-values recorded in Borehole SP12 are quite low. Based on this, recompaction of the existing embankment fills to a safe distance beyond the west abutment may be considered.

As mentioned before, the granular embankment fills encountered in all five boreholes (i.e. Boreholes SP7-SP10 and SP12) are suitably graded and it is our opinion they would be suitable for re-use. Again, some minor moisture conditioning, (e.g. wetting) may be required. The buried topsoil, organic rich soils and natural silt deposits are considered unsuitable for proper compaction, as well as being frost susceptible. The sand till can be re-used but it will need to be mixed with dryer materials as its moisture may need to be adjusted.

In addition to foundation settlements, the embankments will experience settlements under their own weight. For the proposed 4 to 5 m high embankments, these settlements should not exceed 25 mm, provided that the construction is carried out with current MTO procedures (e.g. stripping, compaction, etc). The combined settlements due to foundation and self-weight stresses should not exceed 50 mm (i.e. 20 mm + 25 mm) and do not, in our opinion, warrant surcharging.

As was mentioned before, the existing embankments do not appear to have received proper compaction when they were first placed. For this reason, it is likely that some settlements will occur where new embankments (i.e. widened portion) will abut on to the existing embankment (i.e. along the present shoulders and side slopes). If the existing embankments are re-compacted, this would not present a problem. However, if and/or where recompaction is not applied, some additional settlements can be expected. It is expected that most of such additional settlements would manifest themselves within six weeks of the construction of the embankments and if the paving is delayed, apparent problems can be rectified before paving, if required.

Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

5.4 CONSTRUCTION COMMENTS

Staging will involve demolishing one-half of the existing bridge, as well as removing half of the existing approach fill embankment near the bridge. The traffic will then be diverted to the newly built half of the bridge and the approach fill, while the other portion is being demolished, removed and the new bridge constructed. This will necessitate shoring to support up to about 7 m high embankments (fill and natural soils) during Stage 1. During Stage 2, shoring (roadway protection) would be required to support the newly built section, while the second half is being built. Ideally, the same shoring system would be used in both cases with some modifications.

The temporary shoring (roadway protection) system could consist of conventional soldier piles and timber lagging. In this instance, soldier piles will need to be socketed into the bedrock. Since steel sheet piles will not be able to penetrate the bedrock, roadway protection system using steel sheet piling is not considered to be a feasible alternative.

The roadway protection system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for structure performance level. In this case, the Performance Level should be 2. The roadway protection system should be designed by a Professional Engineer who is experienced in this type of work.

If a soldier pile and timber lagging system is selected, the soldier piles will have to be socketed a sufficient distance into the relatively sound bedrock. The socket length into the bedrock will be determined by the designer of the shoring system but the minimum embedment should not be less than 0.5 m. The socket hole diameter will probably be 0.6 m. As the intact rock encountered at this site is considered very hard, socketing into the intact bedrock will be difficult but it is believed to be achievable. Due to the anticipated height (i.e. up to 7 m) cantilever type design will not be feasible. Since the use of raker footings will

probably be impractical (i.e. will take up space and restrict construction activities), one or even two levels of rock anchoring will likely be required. As mentioned before, rock anchors can be designed on the basis of the factored rock/grout bond capacity at U.L.S. as being equal to 500 kPa and S.L.S. will not govern. The design should be based on extending into the sound rock (i.e. the resistance given above is for sound rock). The upper 0.3 m of the rock should not be included in calculating the resistance and the minimum embedment into the sound rock should be 1.5 m. The anchors should also be checked for rock wedge pull-out, assuming a 60 degree apex cone/wedge and the anchor group resistance should be checked. Proper field monitoring including proof loading of all anchors will be required including load testing of a representative number of anchors.

The use of contiguous caisson type rigid road protection system with or without rock anchors can be considered but it is believed that such a temporary shoring system will not be cost-effective due to anticipated rather difficult rock socketing.

For design, the following unfactored soil and bedrock parameters can be used.

Table 5.4.1
Recommended Unfactored Parameters for Temporary Shoring Design

Material Type	K_a	K_o	K_p	Bulk Unit Weight kN/m ³
Granular Embankment Fill (Sand)	0.34	0.48	2.9	20.0
Other Embankment Fill	0.35	0.47	2.7	19.0
Silt/Sandy Silt/Clayey Silt	0.34	0.48	2.9	18.5
Sand till	0.29	0.44	3.5	21.5
Weathered bedrock	0.2	0.4	4.5	24.0
Sound bedrock	0.10	3.0	6.0	25.0

At the time of our investigation, the groundwater at the site was contacted at about El. 313.5 m, about 0.2 to 1.5 m above the bedrock surface. This indicates that, depending on the conditions at the time of construction, dewatering will be required when excavations extend to about El. 313.5 m. It is believed that the dewatering can be accomplished by gravity drainage and pumping from open sumps on the surface (or some distance below, in case highly weathered rock) of the bedrock.

All excavations, shoring and backfilling should be carried out in conformance with the safety regulations of the Province, as well as the following specifications:

SP539S01 – Protection Schemes

SP902S01 – Excavation and Backfilling to Structures

The embankment fills and the underlying natural soils can be classified as Type 3 soil (Type 4 where very loose or below water level).

The sloping of the sides of temporary excavations in weathered rock will need to be determined in the field, based on visual observations and the extent of weathering. Typically $\frac{1}{2}$ H in 1V side slopes with protection for the workers from falling rock will be suitable for excavations less than 2 m depth into the bedrock. For intact rock vertical or nearly vertical side slopes of temporary excavations are typically acceptable provided that provision is made for the protection of workers from any loose rock pieces from any overlying fractured rock or from overlying overburden. Typically this is provided by means of wire-mesh.

The intact rock is considered to be extremely hard and will create problems when extending the excavations (including socketing of shoring holes) into the intact rock, especially in the close vicinity of the railway where the use of high vibration generating equipment and/or blasting will be objectionable.

The intact rock will likely be broken up into small pieces using powerful equipment such as powerful jack hammers, shot drills and large diameter diamond drills. We recommend that potential difficulties with the excavation of the intact rock for the footings, shoring (caisson) holes and for anchoring be made known to potential contractors, including letting them know that the exposed rock can be examined at the site and/or rock cores obtained for this investigation can be made available for examination in our laboratory in Etobicoke.

5.5 FROST PROTECTION

Design frost penetration for the general area is 2.4 m. Therefore, a permanent soil cover of 2.4 m or its thermal equivalent is required for frost protection of foundations placed on any overburden and weathered bedrock. For massive rock, this requirement is less stringent as discussed in Sections 5.1.1 and 5.1.2 of this report. Frost protection is not required for footings constructed on sound bedrock. In case of riprap or rock fill, only one half of the riprap or rock fill thickness should be taken as being effective in providing frost protection.

6. CLOSURE

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

SHAHEEN & PEAKER LIMITED



Ramon Miranda, P.Eng.



Z.S. Ozden, P.Eng.



K. R. Peaker, Ph.D., P.Eng.

ZO:tr/idrive



Drawings

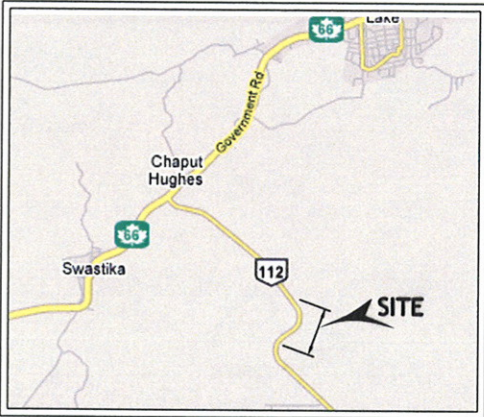
NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.5005-A-000308
GWP: 140-88-00

Hwy 112 / ONR
BOREHOLE LOCATIONS

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Boreholes
- Test Pits

No.	ELEV.	STATION NO.	OFFSET
SP1	317.7	25+970	3.5 m Rt C/L
SP2	317.8	25+968.9	0.9 m Rt C/L
SP3	317.7	25+963.2	3.1 m Lt C/L
SP4	317.7	25+969.4	3.0 m Lt C/L
SP5	317.8	25+979.5	1.8 m Lt C/L
SP6	317.7	25+986.2	3.9 m Rt C/L
SP7	317.8	25+944.8	3.4 m Rt C/L
SP8	317.8	25+942.3	3.9 m Rt C/L
SP9	317.8	25+944.5	1.8 m Lt C/L
SP10	317.8	25+941	2.6 m Lt C/L
SP12	318.0	25+924	4.2 m Lt C/L

NOTE

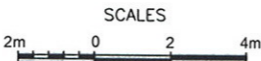
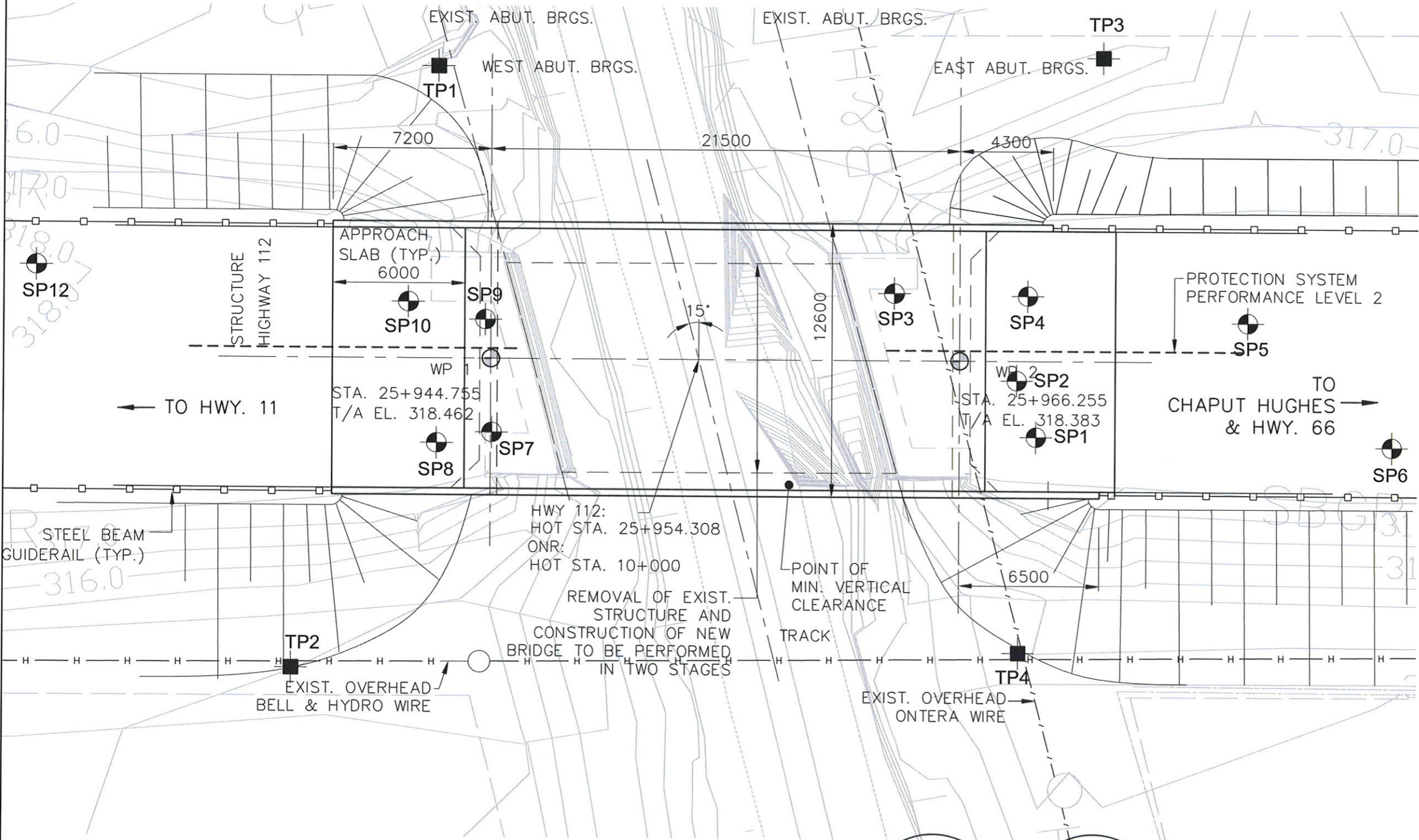
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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. 42A-66

SPT 1104B			DIST 53
SUBM'D	CHECKED	DATE Sept. 2007	SITE 47-015
DRAWN XS	CHECKED RM	APPROVED ZO	DWG 1



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

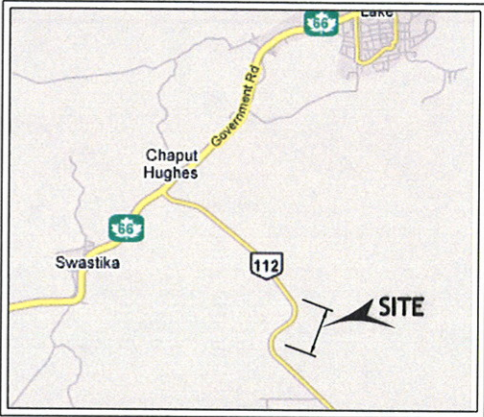
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DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.5005-A-000308
GWP: 140-88-00



Hwy 112 / ONR
BOREHOLE LOCATIONS

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Borehole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION NO.	OFFSET
SP1	317.7	25+970	3.5 m Rt C/L
SP3	317.7	25+963.2	3.1 m Lt C/L
SP5	317.8	25+979.5	1.8 m Lt C/L
SP6	317.7	25+986.2	3.9 m Rt C/L
SP7	317.8	25+944.8	3.4 m Rt C/L
SP10	317.8	25+941	2.6 m Lt C/L
SP12	318.0	25+924	4.2 m Lt C/L

NOTE

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REV.	DATE	BY	DESCRIPTION

Geocres No. 42A-66

SPT 1104B			DIST 53
SUBM'D	CHECKED	DATE Sept. 2007	SITE 47-015
DRAWN XS	CHECKED RM	APPROVED ZO	DWG 2

SP12

SP10

SP7

SP3

SP1

SP5

SP6

WEST

WEST ABUT. BRGS.

EXIST. OVERHEAD
ONTERA WIRE
EL. 323.86

EAST ABUT. BRGS.

EAST

7200

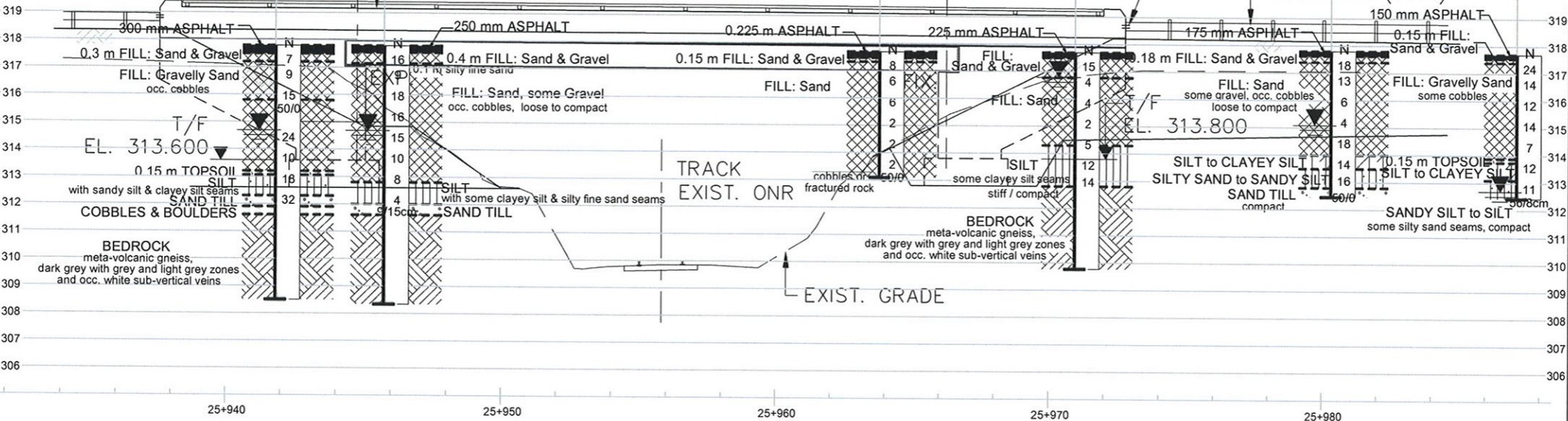
21500

6500

CONC. PARAPET WALL
WITH RAILING

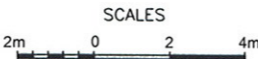
GUIDERAIL AND CHANNEL
ANCHORAGE (TYP.)

STEEL BEAM
GUIDERAIL (TYP.)



TRACK
EXIST. ONR

EXIST. GRADE



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

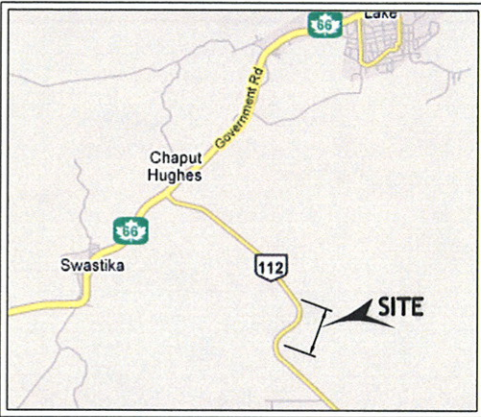
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DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.5005-A-000308
GWP: 140-88-00



Hwy 112 / ONR
BOREHOLE LOCATION PLAN & SOIL STRATA
ALONG EAST BOUND ABUTMENT

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Borehole
- Test Pits
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

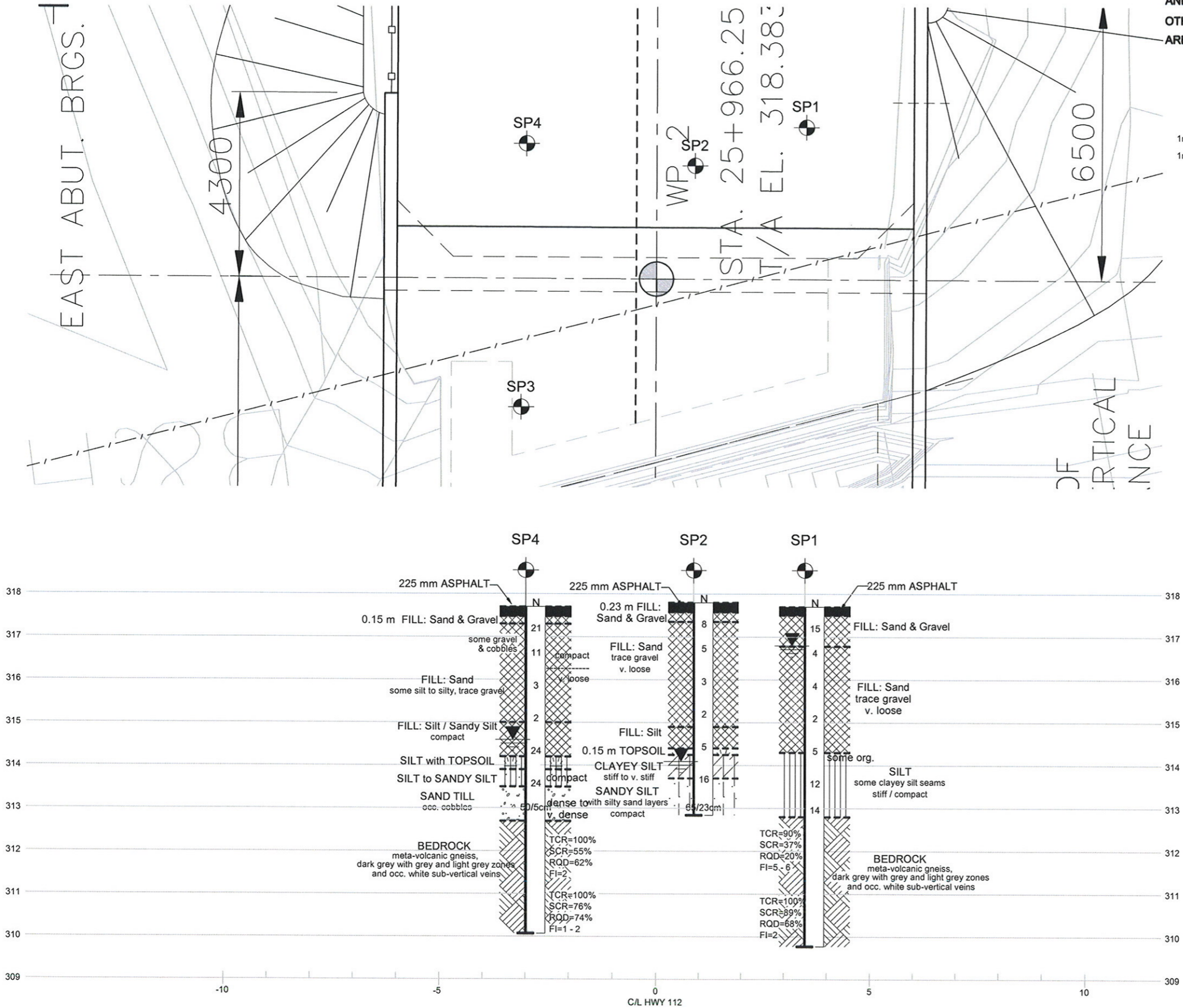
No.	ELEV.	STATION NO.	OFFSET
SP1	317.7	25+970	3.5 m Rt C/L
SP2	317.8	25+968.9	0.9 m Rt C/L
SP4	317.7	25+969.4	3.0 m Lt C/L

NOTE

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REV.	DATE	BY	DESCRIPTION
Geocres No. 42A-66			
SPT 1104B			DIST 53
SUBM'D	CHECKED	DATE Sept. 2007	SITE 47-015
DRAWN XS	CHECKED RM	APPROVED ZO	DWG 3



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

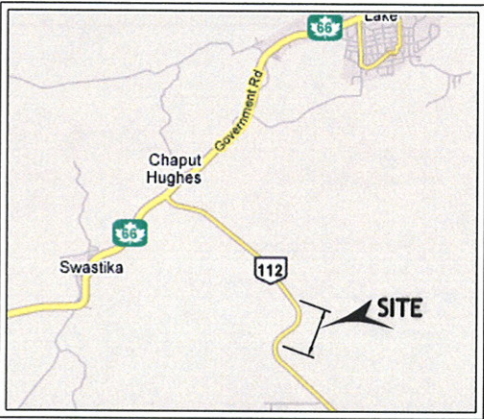
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DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.5005-A-000308
GWP: 140-88-00



Hwy 112 / ONR
STRATIGRAPHIC SECTION
ALONG WEST BOUND ABUTMENT

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

LEGEND

- Borehole
- Test Pits
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION NO.	OFFSET
SP7	317.8	25+944.8	3.4 m Rt C/L
SP10	317.8	25+941	2.6 m Lt C/L

= NOTE =

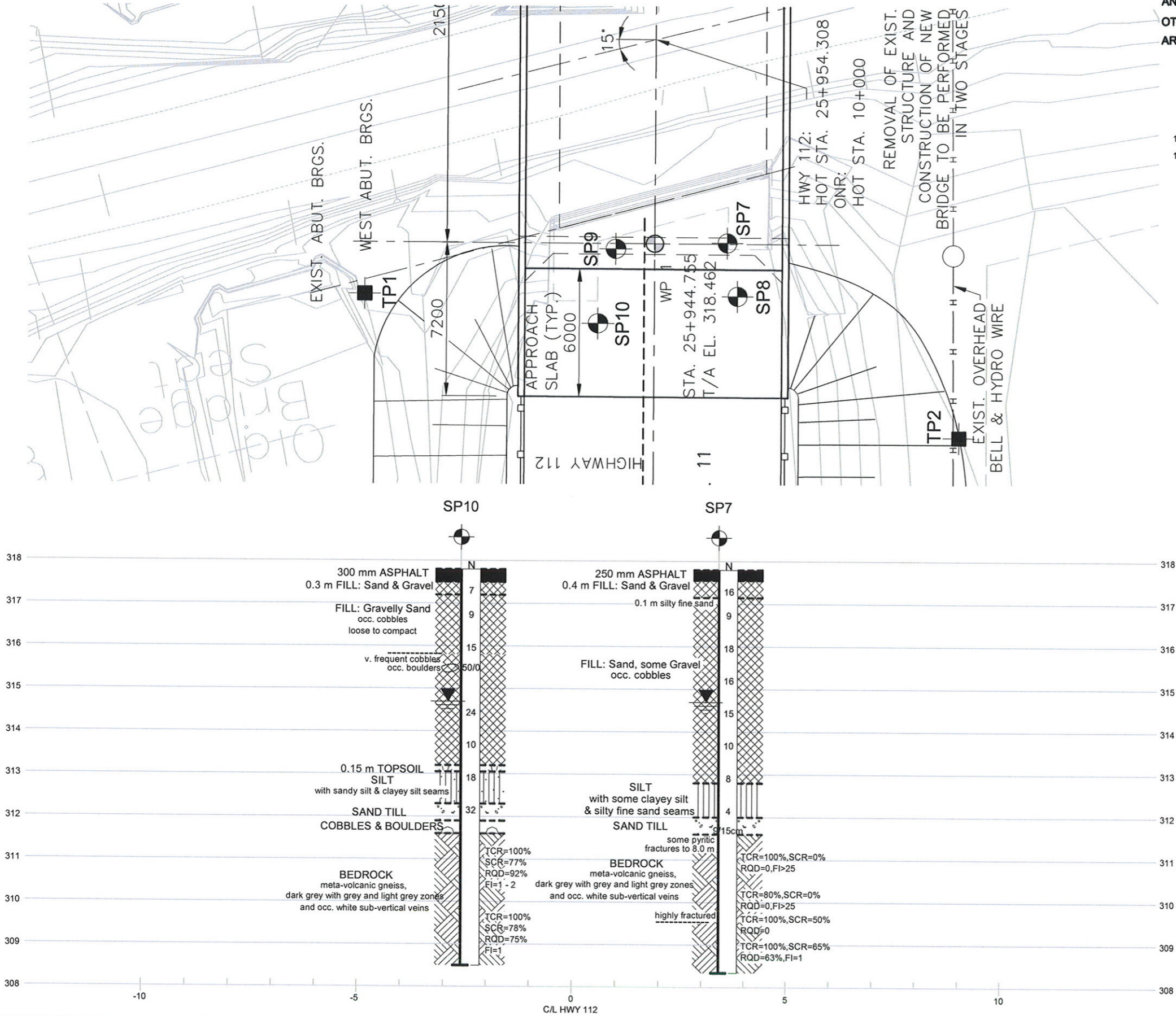
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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. 42A-66

SPT 11048			DIST 53
SUBM'D	CHECKED	DATE	SITE
DRAWN XS	CHECKED RM	APPROVED ZO	DWG 4



Appendix A

Records of Boreholes



RECORD OF BOREHOLE No SP1

1 OF 1

METRIC

GWP 140-88-00 LOCATION Hwy 112 / ONR - Sta. 25+970, 3.5 m Rt C/L
DIST 53 HWY 112 BOREHOLE TYPE Solid Stem Auger & NQ Rock Coring
DATUM Geodetic DATE 2007/05/02
ORIGINATED BY GI
COMPILED BY XS
CHECKED BY RM

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								
							WATER CONTENT (%)									
							PLASTIC LIMIT w _p			NATURAL MOISTURE CONTENT w			LIQUID LIMIT w _L			
							20 40 60 80 100			10 20 30						
317.7	GROUND SURFACE															
317.5	225 mm ASPHALT															
0.2																
	FILL: Sand & Gravel		1	SS	15		317									
316.8			2	SS	4											
0.9			3	SS	4		316									
	FILL: Sand trace gravel brown, damp to moist very loose		4	SS	2		315									
314.3			5	SS	5											
3.4	some organics						314									
314.2	SILT some clayey silt seams greyish, moist, stiff / compact		6	SS	12											
3.6			7	SS	14		313									
312.8			8	RC			312								TCR=90% SCR=37% RQD=20% FI=5 - 6	
4.9	BEDROCK meta-volcanic gneiss, dark grey with grey and light grey zones and occasional white sub-vertical veins		9	RC			311								TCR=100% SCR=89% RQD=68% FI=2	
309.8							310									
7.9	End of borehole.															
	* Water level in open borehole at 0.9 m (El. 316.8 m) upon completion of coring. Water used for coring (not stabilized).															
	Borehole caved-in at 3.1 m.															

RECORD OF BOREHOLE No SP2

1 OF 1

METRIC

GWP 140-88-00

LOCATION Hwy 112 / ONR - Sta. 25+968.9, 0.9 m Rt C/L

ORIGINATED BY G1

DIST 53 HWY 112

BOREHOLE TYPE Solid Stem Auger

COMPILED BY XS

DATUM Geodetic

DATE 2007/05/09

CHECKED BY RM

[illegible]

• 3_{+} , 3_{\times} : Numbers refer to Sensitivity

(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No SP3

1 OF 1

METRIC

GWP 140-88-00 LOCATION Hwy 112 / ONR - Sta. 25+963, 3.1 m L/C/L
DIST 53 HWY 112 BOREHOLE TYPE Solid Stem Auger
DATUM Geodetic DATE 2007/05/09
ORIGINATED BY GI
COMPILED BY XS
CHECKED BY RM

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								
							20	40	60	80	100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	WATER CONTENT (%)	
							20	40	60	80	100	10	20	30		
317.7	GROUND SURFACE															
317.5	225 mm ASPHALT															
0.2																
317.3	0.15 m FILL: Sand & Gravel		1	SS	8		317									
0.4			2	SS	6											
			3	SS	6		316									
	FILL: Sand trace gravel brown, damp, loose to very loose		4	SS	2		315									
			5	SS	2											
			6	SS	2		314									
313.1	cobbles or fractured rock		7	SS	50/0											
4.6	End of borehole. Auger refusal @ 4.6 m on possible bedrock. Borehole dry upon completion. Borehole caved-in at 3.1 m.															



RECORD OF BOREHOLE No SP4

1 OF 1

METRIC

GWP 140-88-00 LOCATION Hwy 112 / ONR - Sta. 25+969.4, 3 m Lt C/L
DIST 53 HWY 112 BOREHOLE TYPE Solid Stem Auger & NQ Rock Coring
DATUM Geodetic DATE 2007/05/01
ORIGINATED BY GI
COMPILED BY XS
CHECKED BY RM

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					
							20 40 60 80 100	○ UNCONFINED	+ FIELD VANE	● POCKET PENETR.	× LAB VANE	WATER CONTENT (%)	
							20 40 60 80 100					10 20 30	
317.7	GROUND SURFACE												
317.5	225 mm ASPHALT												
0.2													
317.3	0.15 m FILL: Sand & Gravel		1	SS	21								
0.4													
	some gravel & cobbles		2	SS	11								
	FILL: Sand compact												
	some silt to silty trace gravel brown, damp		3	SS	3								
	very loose												
315.0			4	SS	2								
2.7	FILL: Silt / Sandy Silt greyish, moist, compact												
314.2			5	SS	24								
3.5	SILT with TOPSOIL												
313.9													
3.8	SILT to SANDY SILT grey, wet, compact		6	SS	24								
313.5													
4.3	SAND TILL occasional cobbles brown, wet, dense to very dense		7	SS	50/5cm								
312.7													
5.0													
	BEDROCK meta-volcanic gneiss, dark grey with grey and light grey zones and occasional white sub-vertical veins		8	RC									
			9	RC									
310.1													
7.6	End of borehole.												
	* Water level in open borehole at 3.1 m (El. 314.6 m) upon completion of coring. Water used for coring (not stabilized).												
	Borehole caved-in at 3.7 m.												



RECORD OF BOREHOLE No SP5

1 OF 1

METRIC

GWP 140-88-00 LOCATION Hwy 112 / ONR - Sta. 25+979.5, 1.8 m Lt C/L
DIST 53 HWY 112 BOREHOLE TYPE Solid Stem Auger
DATUM Geodetic DATE 2007/05/01
ORIGINATED BY GI
COMPILED BY XS
CHECKED BY RM

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 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. x LAB VANE							
							20	40	60	80	100				
317.8	GROUND SURFACE														
317.6	175 mm ASPHALT														
0.2															
317.4	0.18 m FILL: Sand & Gravel		1	SS	18										
0.4			2	SS	13										
			3	SS	6										
	FILL: Sand some gravel occasional cobbles brown, moist, loose to compact		4	SS	4										
			5	SS	18										
314.0															
3.8	SILT to CLAYEY SILT grey, moist, stiff		6	SS	14										
313.5															
4.3	SILTY SAND to SANDY SILT trace gravel brown, moist, compact		7	SS	16										
312.8															
5.0	SAND TILL brown, moist, compact		8	SS	50/0										
312.5															
5.3	End of borehole. Auger refusal @ 5.3 m on possible bedrock. * Water level in open borehole at 2.7 m (El. 315.1 m) upon completion (not stabilized). Borehole caved-in at 2.8 m.														

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5 10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SP6

1 OF 1

METRIC

GWP	140-88-00	LOCATION	Hwy. 112 / ONR - Sta. 25+986.2, 3.9 m RI C/L	ORIGINATED BY	GI
DIST	53	HWY	112	BOREHOLE TYPE	Solid Stem Auger
DATUM	Geodetic	DATE	2007/05/01	COMPILED BY	XS
				CHECKED BY	RM

[illegible]

\cdot^3, \times^3 : Numbers refer to Sensitivity

(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No SP7

1 OF 1

METRIC

GWP 140-88-00 LOCATION Hwy 112 / ONR - Sta. 25+944.8, 3.4 m Rt C/L ORIGINATED BY Gi
DIST 53 HWY 112 BOREHOLE TYPE Solid Stem Auger & NQ Rock Coring COMPILED BY XS
DATUM Geodetic DATE 2007/05/02 CHECKED BY RM

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. x LAB VANE							
317.8	GROUND SURFACE							20 40 60 80 100							
317.6	250 mm ASPHALT														
0.3	0.4 m FILL: Sand & Gravel		1	SS	16										
317.2	0.1 m silty fine sand		2	SS	9		317								22 72 (6)
0.7	FILL: Sand, some Gravel occasional cobbles brown, damp to moist loose to compact		3	SS	18		316								
			4	SS	16		315								
			5	SS	15		314								
			6	SS	10		313								
			7	SS	8		312								
			8	SS	4		311								
312.8	SILT with some clayey silt & silty fine sand seams grey, wet, soft / very loose		9	SS	9/15cm		310								TCR=100%,SCR=0% RQD=0,FI>25
5.0	SAND TILL brown, wet, compact		10	RC			309								TCR=80%,SCR=0% RQD=0,FI>25
312.0			11	RC											TCR=100%,SCR=50% RQD=0
5.8			12	RC											TCR=100%,SCR=65% RQD=63%,FI=1
311.6			13	RC											
6.2															
	BEDROCK meta-volcanic gneiss, dark grey with grey and light grey zones and occasional white sub-vertical veins	some pyritic fractures to 8.0 m highly fractured													
308.3	End of borehole.														
9.5	* Water level in open borehole at 3.1 m (El. 314.7 m) upon completion of coring. Water used for coring (not stabilized). Borehole caved-in at 7.0 m.														



RECORD OF BOREHOLE No SP8

1 OF 1

METRIC

GWP 140-88-00 LOCATION Hwy 112 / ONR - Sta. 25+942.3, 3.9 m Rt C/L
DIST 53 HWY 112 BOREHOLE TYPE Solid Stem Auger
DATUM Geodetic DATE 2007/05/01
ORIGINATED BY GI
COMPILED BY XS
CHECKED BY RM

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					
								20 40 60 80 100					
								○ UNCONFINED + FIELD VANE					
								● POCKET PENETR. × LAB VANE					
								WATER CONTENT (%)					
								20 40 60 80 100					
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT					
								W _p W W _L					
317.8 0.0	GROUND SURFACE												
	225 mm ASPHALT												
317.3 0.5	0.3 m FILL: Sand & Gravel		1	SS	8		317						
			2	SS	9								
	FILL: Gravelly Sand some gravel brown, damp loose to compact		3	SS	19		316						30 63 (7)
			4	SS	5		315						
			5	SS	10								
314.1 3.7	0.2 m TOPSOIL		6	SS	7		314						
313.9 3.9	SILT with clayey silt and silty clay seams grey, wet, firm to stiff		7	SS	16		313						0 4 51 45
312.8 5.0	SANDY SILT to SILTY FINE SAND trace gravel brown, wet, compact to dense		8	SS	35		312						
312.0 5.8	SAND TILL												
312.0 5.9	End of borehole. Auger refusal @ 5.9 m on possible bedrock. * Water level in open borehole at 5.5 m (El. 312.3 m) upon completion (not stabilized). Borehole caved-in at 5.5 m.												

+ 3, × 3: Numbers refer to
Sensitivity

20
15
10
5
0
5
10
15
20
(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No SP9

1 OF 1

METRIC

GWP 140-88-00 LOCATION Hwy 112 / ONR - Sta. 25+944.5, 1.8 m Lt C/L
DIST 53 HWY 112 BOREHOLE TYPE Solid Stem Auger
DATUM Geodetic DATE 2007/05/01
ORIGINATED BY GI
COMPILED BY XS
CHECKED BY RM

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						
317.8	GROUND SURFACE						20	40	60	80	100			
317.6	250 mm ASPHALT													
0.3	0.4 m FILL: Sand & Gravel		1	SS	27									
317.2														
0.7														
								</						

1 OF 1

METRIC

GWP	140-88-00	LOCATION	Hwy 112 / ONR - Sta. 25+941, 2.6 m LI CL	ORIGINATED BY	GI
DIST	53	HWY	112	BOREHOLE TYPE	Solid Stem Auger & NQ Rock Coring
DATUM	Geodetic	DATE	2007/05/08	COMPILED BY	XS
				CHECKED BY	RM

3, x 3; Numbers refer to Sensitivity



RECORD OF BOREHOLE No SP12

1 OF 1

METRIC

GWP 140-88-00 LOCATION Hwy 112 / ONR - Sta. 25+924, 4.2 m Lt C/L
DIST 53 HWY 112 BOREHOLE TYPE Solid Stem Auger
DATUM Geodetic DATE 2007/05/04
ORIGINATED BY GI
COMPILED BY XS
CHECKED BY RM

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 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									
318.0 0.0	GROUND SURFACE						20	40	60	80	100						
317.3 0.7	FILL: Gravelly Sand brown, damp, compact		1	SS	17												
			2	SS	5												
	FILL: Sand, Some Gravel occasional cobbles brown, moist, very loose		3	SS	3												
			4	SS	5												
			5	SS	5												
314.4 3.6	0.2 m TOPSOIL		6	SS	50/0												
314.2 3.8	End of borehole. Auger refusal @ 3.8 m on possible bedrock. Borehole dry upon completion. Borehole caved-in at 2.4 m.																

+ 3, × 3: Numbers refer to
Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

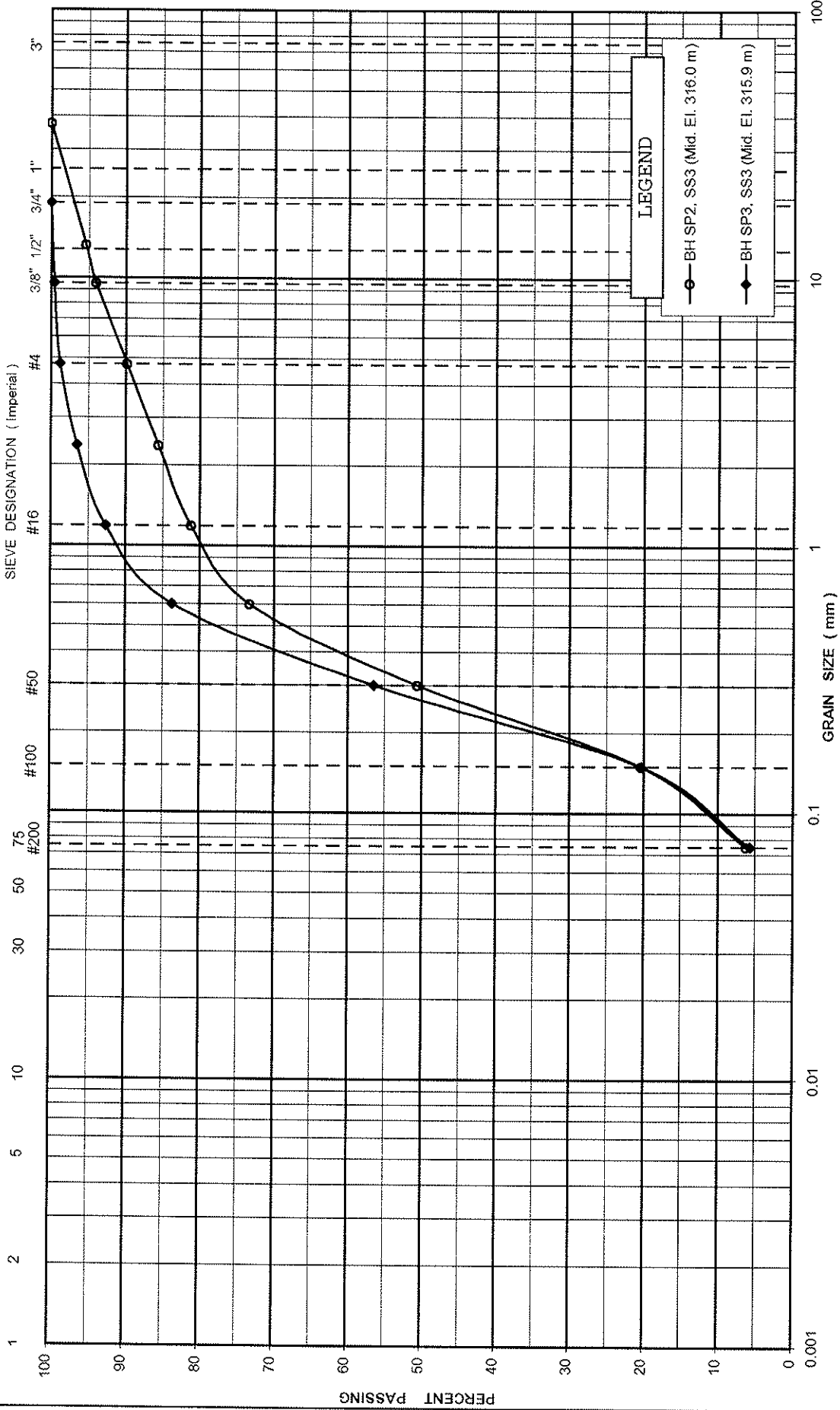
Appendix B

Laboratory Test Results

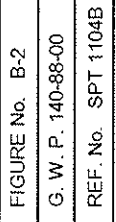
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS



CLAY AND SILT	SAND		GRAVEL
	Fine	Medium	
	Coarse		
	Fine	Coarse	

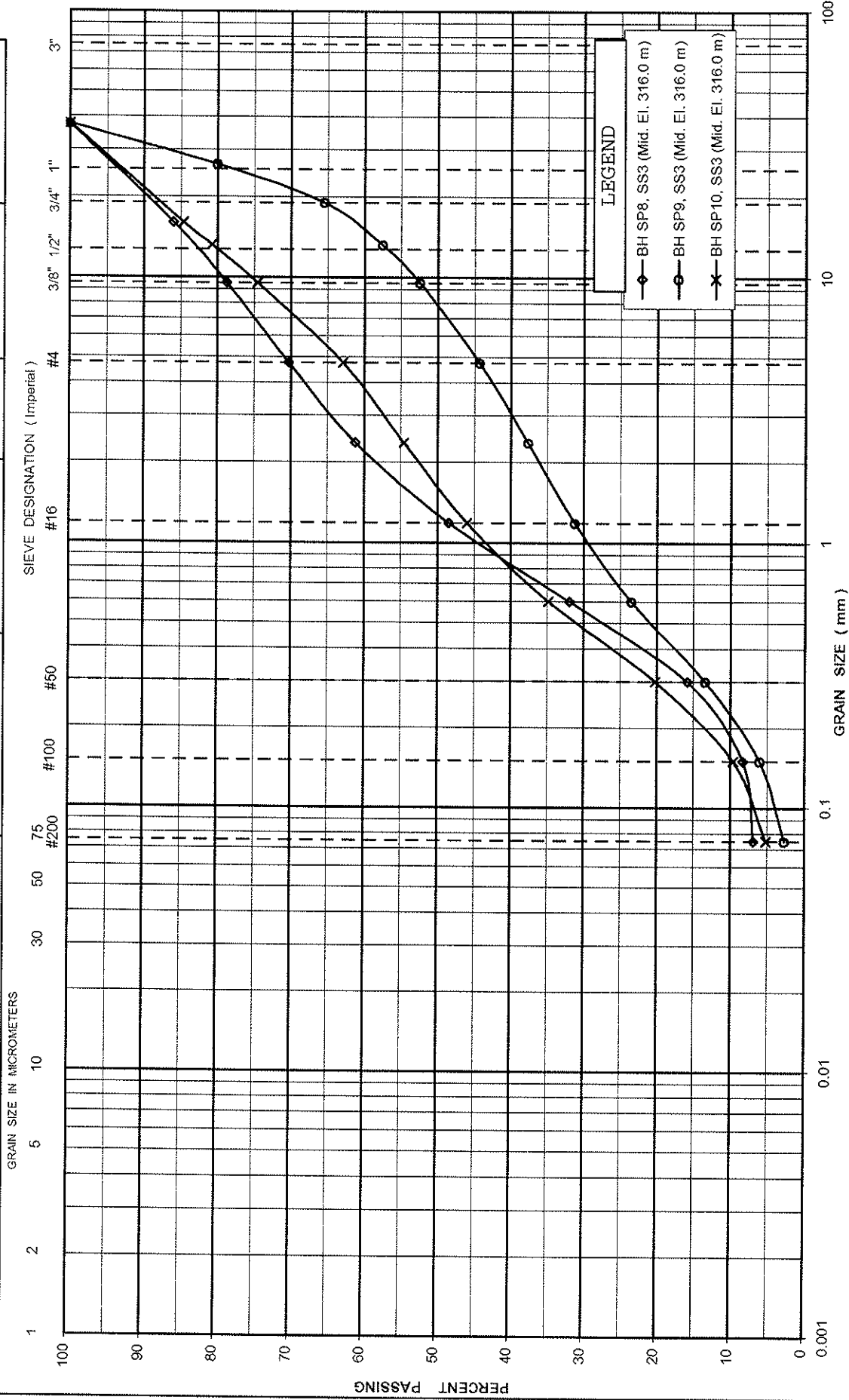


GRAIN SIZE DISTRIBUTION

SHAHEEN & PEAKER LIMITED

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



GRAIN SIZE DISTRIBUTION
EMBANKMENT FILL: Gravelly Sand to Sand & Gravel

FIGURE No. B-3

G. W. P. 140-88-00

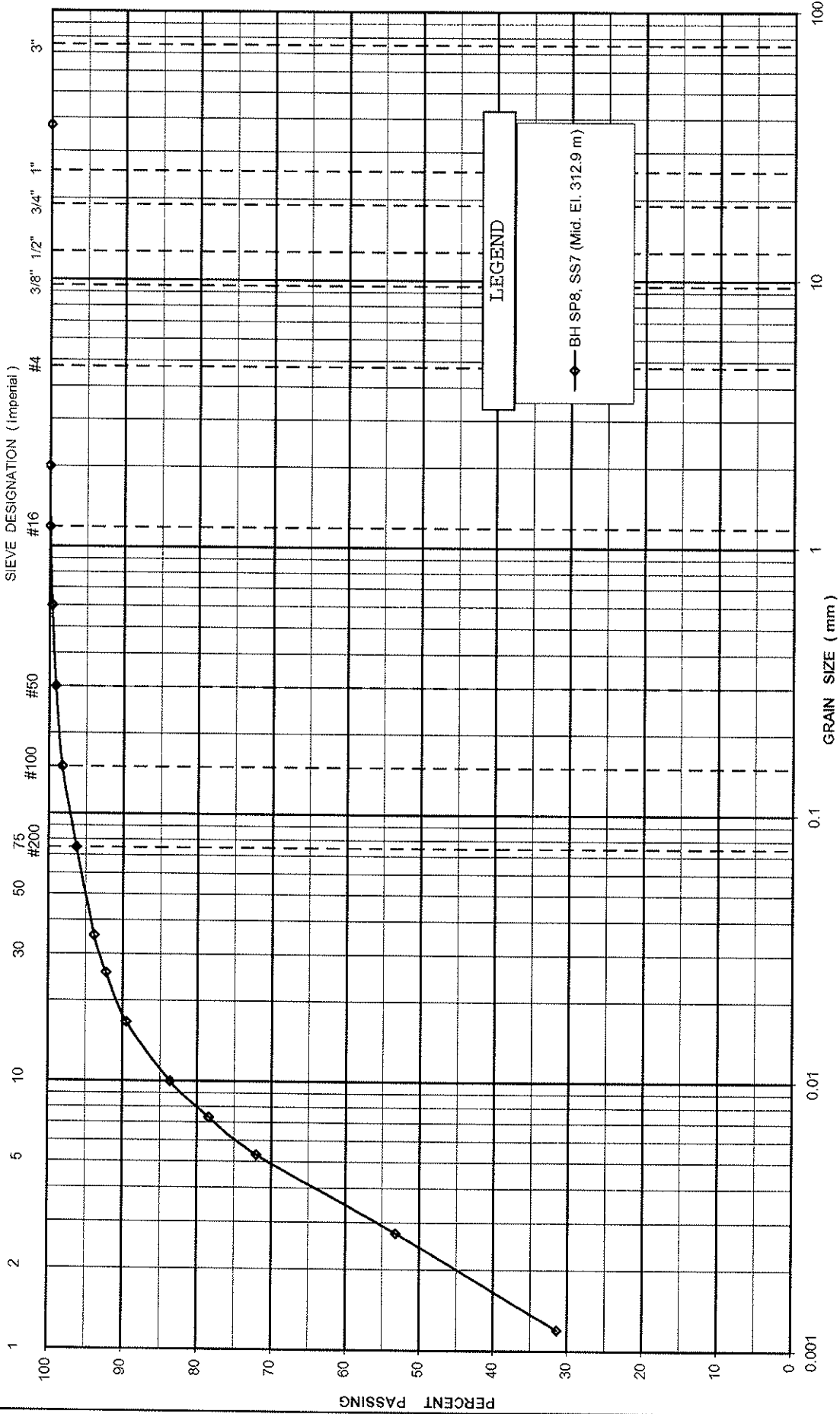
REF. No. SPT 1104B

SHAHEEN & PEAKER LIMITED

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	Coarse

GRAIN SIZE IN MICROMETERS



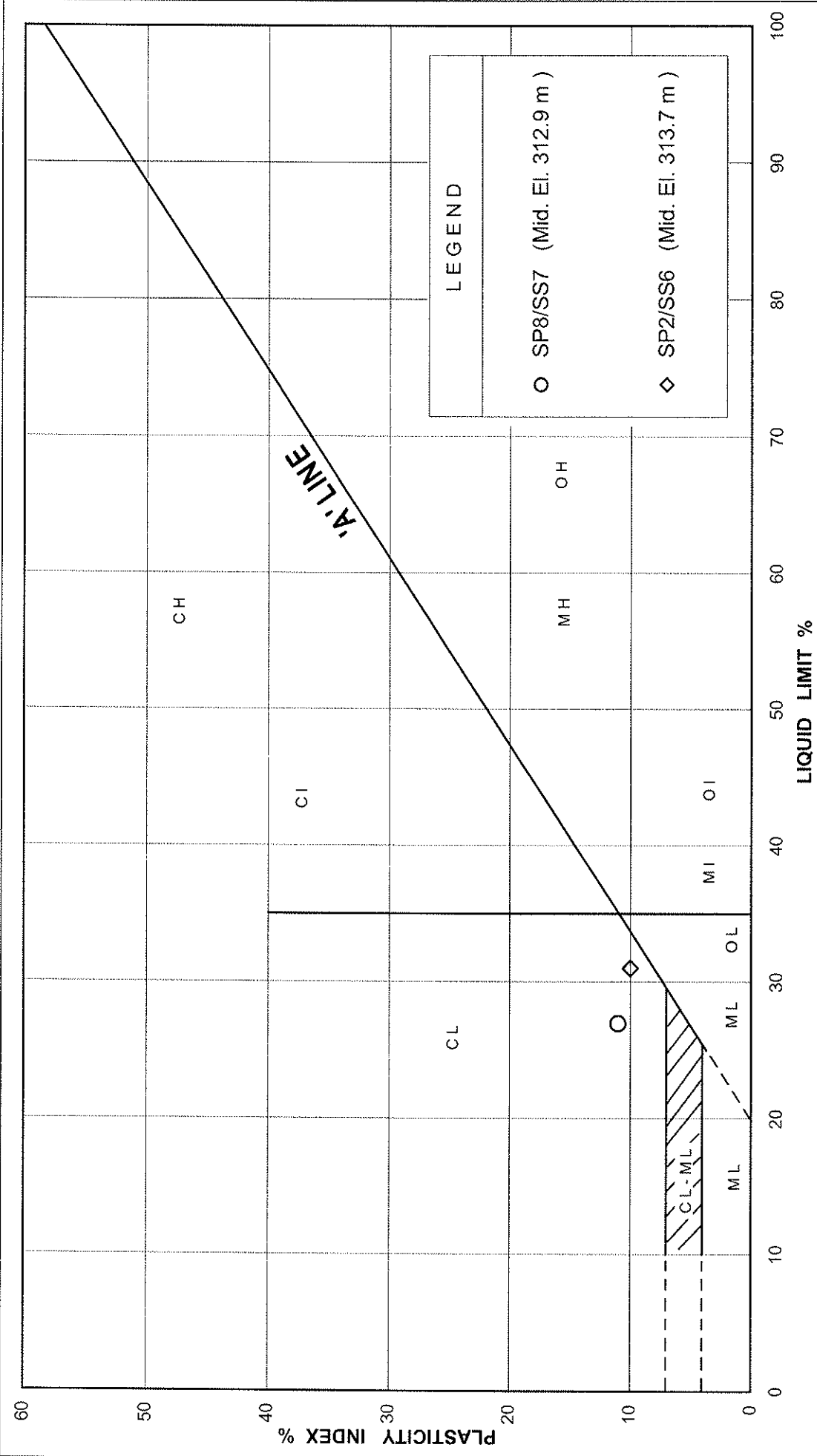
GRAIN SIZE DISTRIBUTION
SILTY CLAY layer in SILT deposit

SHAHEEN & PEAKER LIMITED

FIGURE No. B-4

G. W. P. 140-88-00

REF. No. SPT 1104B

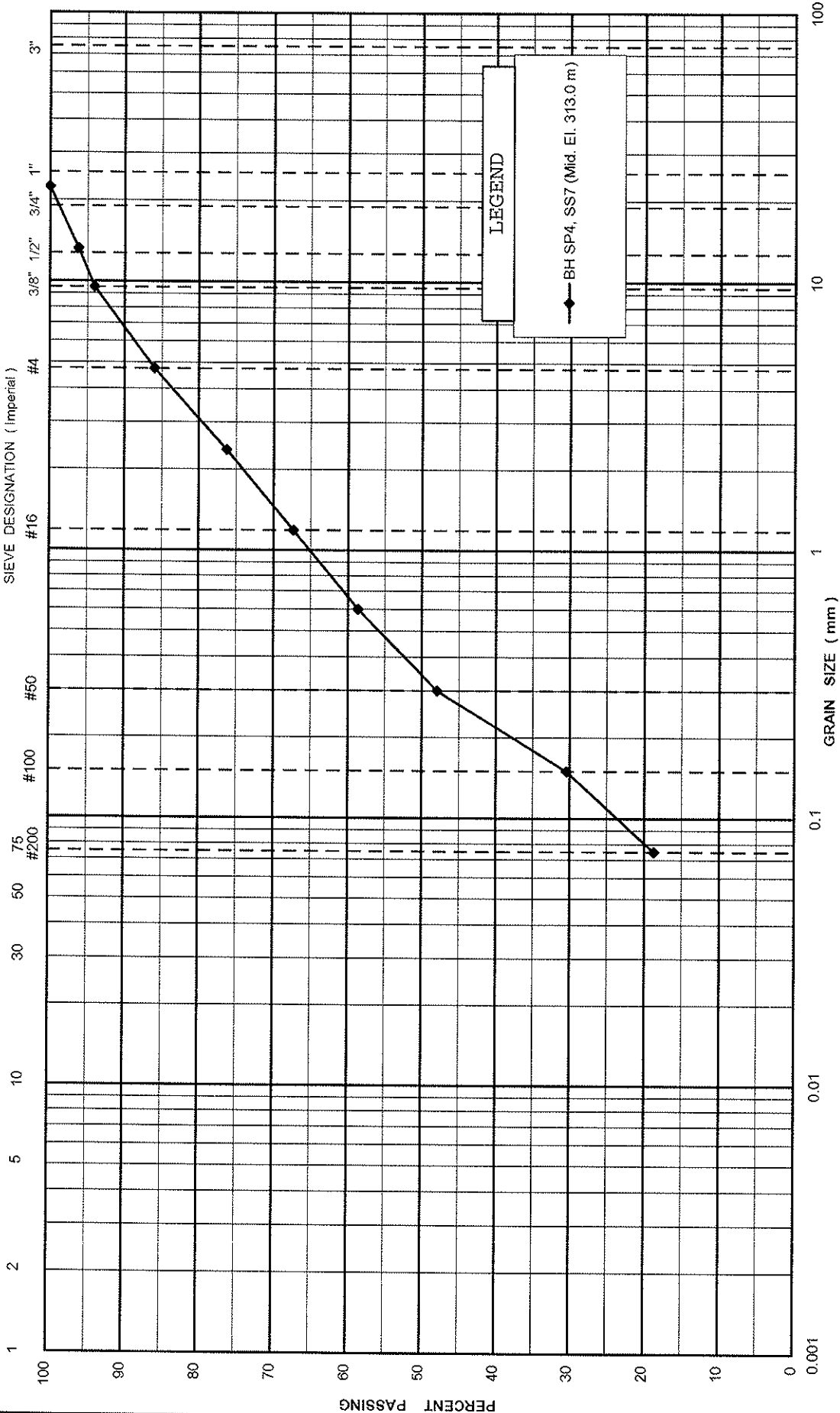


SHAHEEN & PEAKER LIMITED	PLASTICITY CHART	
	CLAYEY SILT	
	FIGURE No. B-5	
	G. W. P. 140-88-00	
	REF. No. SPT 1104B	

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	

GRAIN SIZE IN MICROMETERS



GRAIN SIZE DISTRIBUTION SAND TILL

SHAHEEN & PEAKER LIMITED

FIGURE No. B-6

G. W. P. 140-88-00

REF. No. SPT 1104B

Appendix C

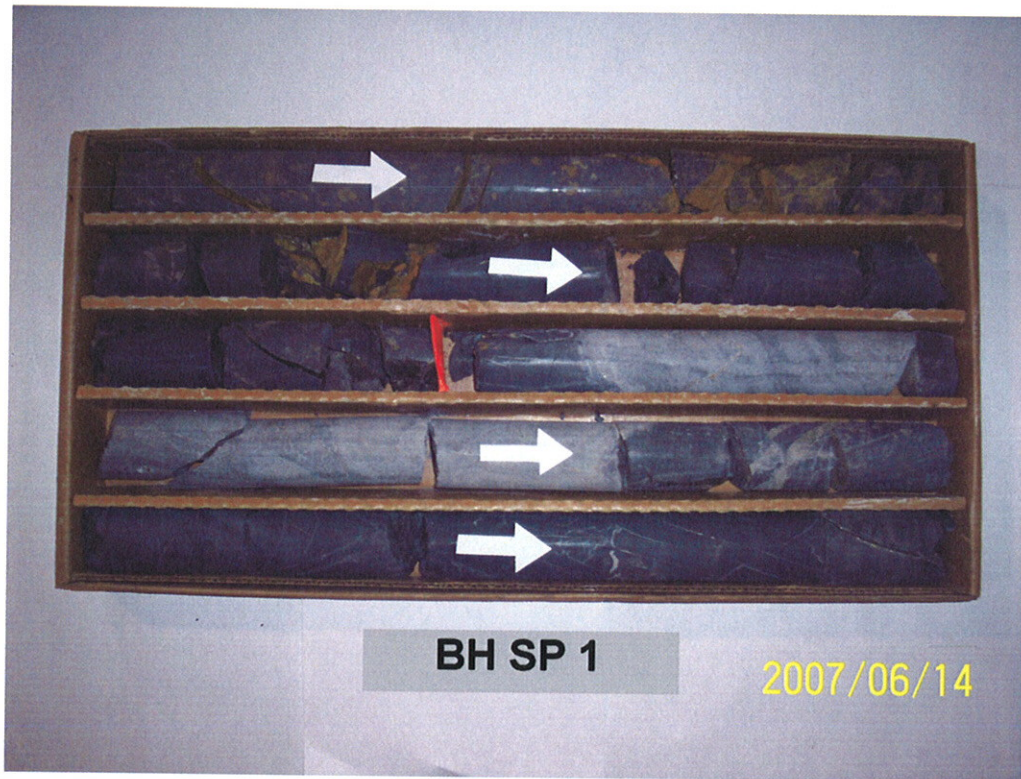
Test Pits

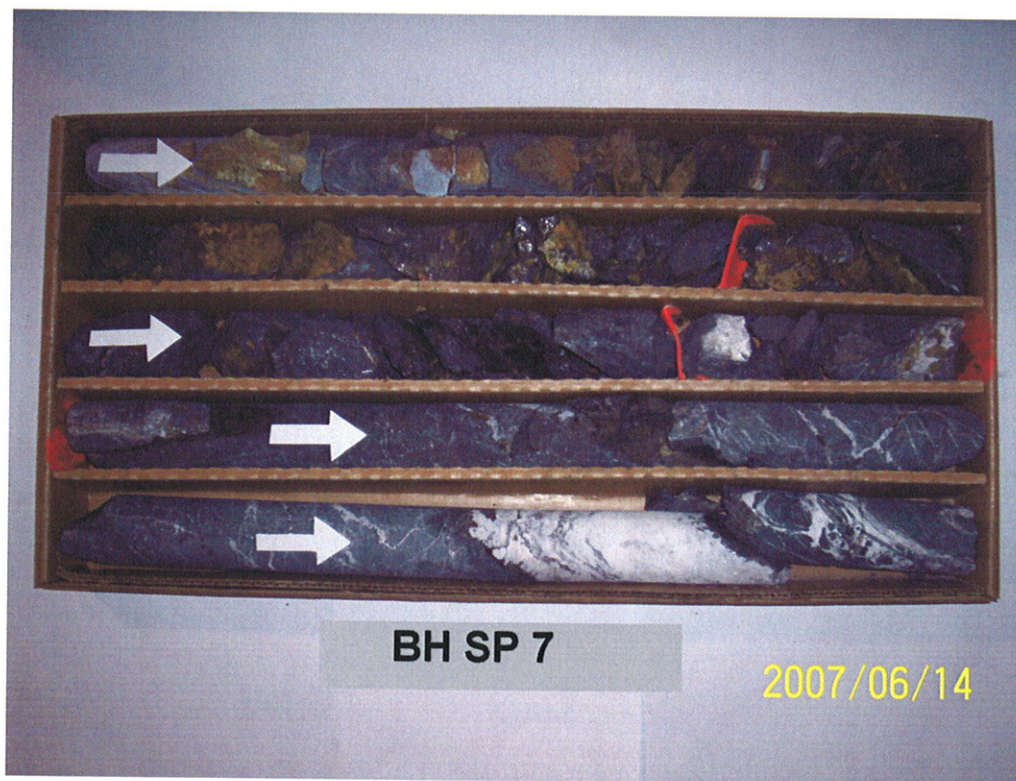
Test Pit Logs

Test Pit Number/Ground Elevation	Depth	Description of Soil Encountered	Groundwater Observations
TP1/ 315.9	0 – 0.10	TOPSOIL, sandy	No free water observed in the test pit upon completion
	0.10 – 0.25	SAND, brown	
	0.25 – 1.0	SILTY FINE SAND, brown, Occ. Cobbles/boulders below 0.6 m	
	1.0 – 1.8	Large boulders/broken rock probable bedrock below 1.8 m	
TP2 314.2	0 – 0.15	TOPSOIL, sandy	Perched groundwater seeping into the test pit @ 0.8 m
	0.15 – 0.45	FILL, sandy silt to silty fine sand, traces of gravel, brown	
	0.45 – 0.65	TOPSOIL, with cobbles	
	0.65 – 0.8	SILT, traces of rootlets, brown, moist	
	0.8 – 1.5	CLAYEY SILT, grey, v. stiff	
	1.5 – 3.0	SILT TO SANDY SILT, greyish	
	3.0	Large boulders & broken rock possible bedrock	
TP3 315.3	0 – 0.15	TOPSOIL	
	0.15	BEDROCK	
TP4 314.0	0 – 0.15	TOPSOIL, sandy	Soil wet @ below 1.5 m
	0.15 – 0.3	FILL, sand, brown	
	0.3 – 0.4	TOPSOIL	
	0.4 – 1.3	CLAYEY SILT, grey, v. stiff	
	1.3 – 2.3	SANDY SILT TO SILT, grey wet some cobbles & boulders below 1.8 m	
	2.3	BEDROCK	

Appendix D

Photographs of Rock Cores





Appendix E

Site Photographs



Appendix F

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 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 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 STRUCTURAL FEATURES AND/OR STRENGTH.

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

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
ζ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
ζ'_p	kPa	PRECONSOLIDATION PRESSURE
τ	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
i'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
i_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ

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
γ_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
γ_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
γ'	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_c	%	SHRINKAGE LIMIT	q	m ² /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	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	HYDRAULIC GRADIENT
γ_{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 SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

Appendix G

Bedrock Elevations at Borehole and Test Pit Locations

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

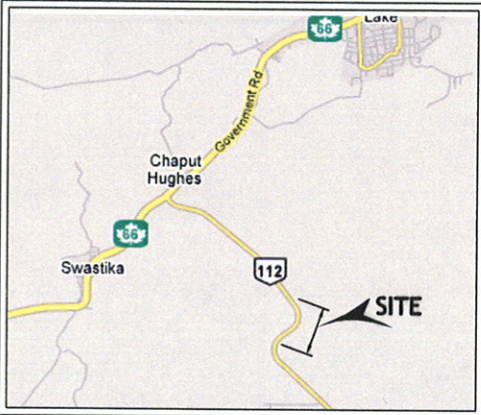
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.

CONT No.5005-A-000308
GWP: 140-88-00



Hwy 112 / ONR
BOREHOLE LOCATIONS

SHAHEEN & PEAKER LIMITED



KEY PLAN
N.T.S

TYPICAL BOREHOLE & TEST PIT LOG

- SP5 ← Borehole No.
317.8 ← Elev.
(312.5) ← Elev. of Inferred Bedrock Surface, Unless Otherwise Cored (see *)
- TP1 ← Test Pit No.
315.9 ← Elev.
(314.0)* ← Elev. of Bedrock Surface

* Refers to Elevations of Bedrock Surface Confirmed by Coring / Test Pit.

No.	ELEV.	STATION NO.	OFFSET
SP1	317.7	25+970	3.5 m Rt C/L
SP2	317.8	25+968.9	0.9 m Rt C/L
SP3	317.7	25+963.2	3.1 m Lt C/L
SP4	317.7	25+969.4	3.0 m Lt C/L
SP5	317.8	25+979.5	1.8 m Lt C/L
SP6	317.7	25+986.2	3.9 m Rt C/L
SP7	317.8	25+944.8	3.4 m Rt C/L
SP8	317.8	25+942.3	3.9 m Rt C/L
SP9	317.8	25+944.5	1.8 m Lt C/L
SP10	317.8	25+941	2.6 m Lt C/L
SP12	318.0	25+924	4.2 m Lt C/L

NOTE

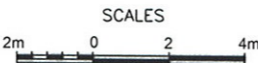
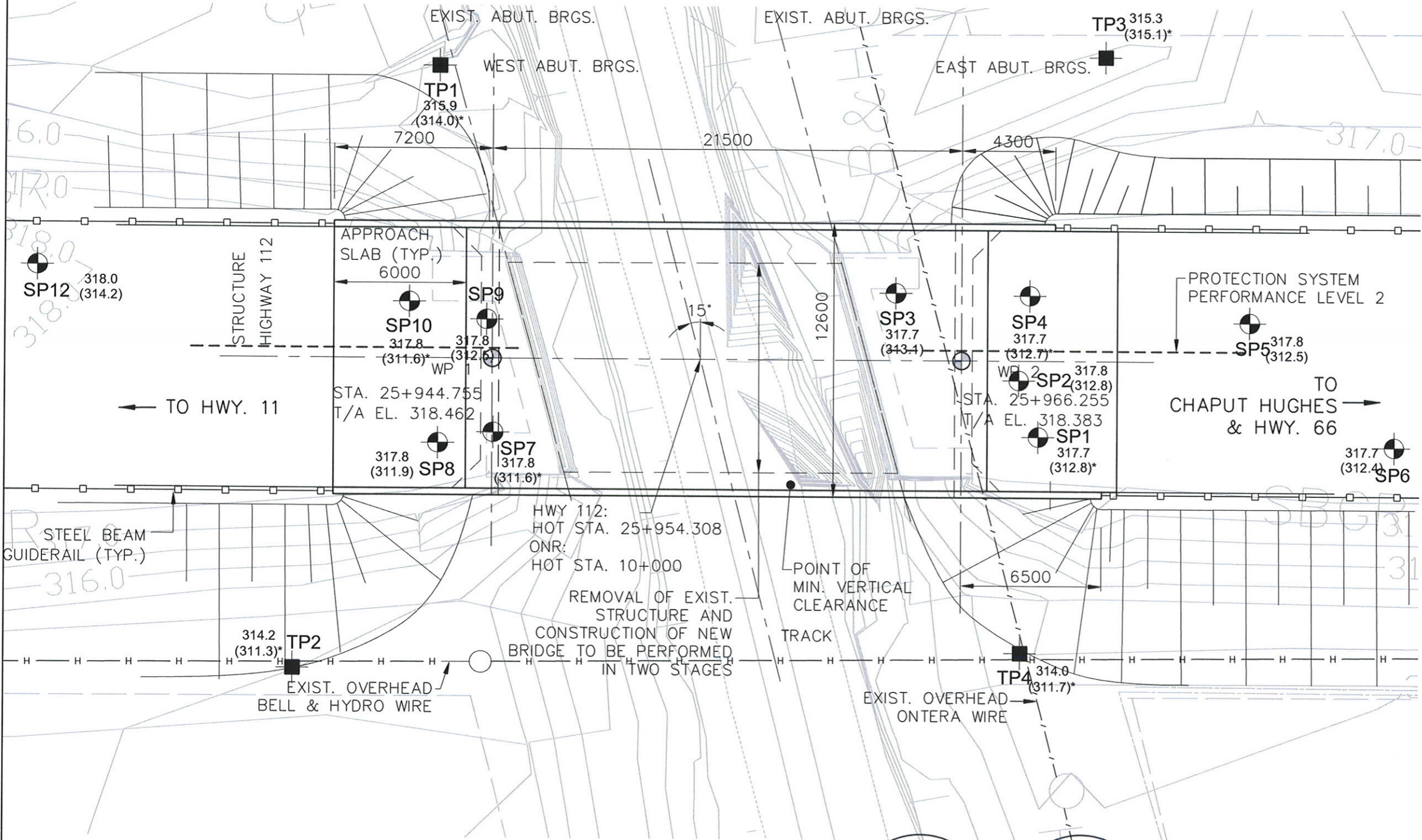
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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. 42A-66

SPT 1104B			DIST 53
SUBM'D	CHECKED	DATE Sept. 2007	SITE 47-015
DRAWN XS	CHECKED RM	APPROVED ZO	DWG G-1



Appendix H

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.