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**GEOTECHNICAL INVESTIGATION
SLIMS RIVER BRIDGE REPLACEMENT
KM 1702, ALASKA HIGHWAY NO. 1
YUKON, CANADA**



GR-01-046

REPORT ON

**GEOTECHNICAL INVESTIGATION
SLIMS RIVER BRIDGE REPLACEMENT
KM 1702, ALASKA HIGHWAY NO. 1
YUKON, CANADA**

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Government of Yukon (Yukon) to provide geotechnical engineering services for the design and construction of the proposed new replacement bridge over Slims River at km 1702 along the Alaska Highway No. 1 in Yukon. The replacement of the bridge is required due to highway and structure upgrading and is being performed under the American-funded Shakwak project. The location of the bridge site is shown in Figure 1.

EarthTech (Canada) Inc. has completed a conceptual design for the new Slims River Bridge. A single span bridge located on the same highway alignment has been selected as the new replacement bridge structure. The new bridge is about 80 m in length. The two abutments will be supported on driven steel pipe piles. The new structure is classified as a *Lifeline Bridge* in accordance with the criteria given in the Canadian Highway Bridge Design Code (CHBDC) CSA-S6-06.

This report summarizes the site-specific geotechnical data collected during the field investigation carried out in May 2007 and recommendations for the design of the foundations of the proposed new bridge. All elevations reported herein are with respect to the Geodetic datum.

This report should be read in conjunction with “**Important Information and Limitations of This Report**” which is appended following the text. The reader’s attention is specifically drawn to this information for the proper use and interpretation of this report.

2.0 SITE CONDITIONS AND PROPOSED NEW BRIDGE

2.1 Site Conditions

The existing Slims River Bridge was built in 1955 by the Canadian Military. The existing bridge is a two-lane two span steel through truss structure which measures about 122 m in total length. Based on the recent information provided by Yukon and EarthTech, we understand that the abutments are supported on driven timber piles and the centre pier is supported on driven steel H piles. It is reported that the timber piles were about 15 m (50 ft.) long and the steel H piles were about 19.8 m (65 ft.) long. All piles were driven using a number 7B steam hammer. The base of the pile cap for the abutment and the centre pier are founded at some 7 m below the existing road grade. A photograph of the existing bridge taken on June 2006 is shown in Figure 2.

The existing bridge has river training works, which includes guide banks extending upstream on both sides.

2.2 Proposed New Bridge

As shown in EarthTech Drawing No. 1702-01a dated September 7, 2007, the proposed new bridge will be a single span bridge. The abutments will be supported on both 610 mm diameter and 914 mm diameter driven steel pipe piles. The new bridge will be about 80 m long, 13.4 m wide, and will be located at the existing bridge alignment. The abutments will include 8.5 m long approach slabs at either end. A temporary bridge upstream of the existing bridge will be constructed on a parallel alignment to divert traffic during construction of the new bridge.

The new bridge deck will be at approximately elevation 788.2 m. The underside of the abutment walls will be at approximately elevation 780 m resulting in an approximately 8 m high abutment wall. 914 mm diameter steel pipe piles are being considered for foundation support of the abutments No. 1 and 610 mm diameter steel pipe piles for abutment No. 2. Based on information provided by EarthTech, the superstructure is longitudinally fixed at abutment No.1 and free to move at abutment No. 2.

3.0 GEOTECHNICAL FIELD INVESTIGATION

3.1 1954 Field Investigation

Three boreholes were drilled at the site in 1954 by Hardy & Associates as part of the investigation for the existing bridge. The boreholes were drilled up to a depth of about 46 m below the ground surface. The 1954 boreholes indicated that the site soils are alluvial deposits, which consist of fine silty sand, silty clay and clayey silt with layers of loose fine sand, clay and gravelly clay. In general, the upper soils consist mainly of loose silty sand of low plasticity and the underlying clayey silt encountered at deeper depth are firm to stiff in consistency.

3.2 2007 Field Investigation

The 1954 investigation has limited subsurface information for the design and construction of the new bridge. Some of the test methods used in the 1954 investigation to obtain geotechnical data differs from those used in the current state of practice for the assessment of liquefaction potential. For this reason, a drilling program has been carried out primarily to obtain supplementary geotechnical field and laboratory data for a proper assessment of the liquefaction potential and estimation of geotechnical engineering parameters for the foundation design. The following section describes the 2007 field investigation program.

The 2007 geotechnical field investigation was carried out during the period between May 13 and May 20, 2007, during which two cone penetration tests (SCPT07-1S and CPT07-2S) and two air rotary boreholes (BH07-1S and BH07-2S) were completed. A pair of one borehole and one CPT was put down from the existing guide bank near each of the existing abutment. The upper portion of the guide bank was drilled out to avoid premature refusal of the cone within denser materials. Downhole seismic shear wave velocity measurements were also obtained during cone penetration testing at the test hole SCPT07-1S. The test holes SCPT07-1S and CPT07-2S were advanced to a depth of about 58 m and 41 m, respectively, below the existing ground surface. Boreholes BH07-1S and BH07-2S were drilled to depth of about 33 m and 20 m, respectively, below the existing ground surface. Standard Penetration Tests (SPT's) were also carried out within the boreholes at 1.5 m interval to assess the relative density of the soils encountered at the site.

The boreholes were drilled using a truck-mounted drill rig supplied and operated by Geotech Drilling Services Limited of Prince George, BC. The boreholes were advanced using an Odex (air rotary) drill bit with 100 mm diameter steel casings advancing together with the bit. The cone penetration tests were carried out by ConeTec Investigations Ltd. of Richmond, BC.

The fieldwork was carried out under the full-time inspection of a member of Golder Associates' geotechnical staff who logged the soil conditions encountered in the boreholes, and brought representative recovered soil samples to our Burnaby laboratory for detailed examination and testing.

Following completion of the field investigation, the boreholes were backfilled with sand and drill cuttings. The boreholes were surveyed by others for locations and elevations. The locations of the boreholes established based on survey measurements are shown on Figure 3.

3.2.1 Laboratory Testing Program

Following a detailed examination of the samples collected from the field investigation program, routine laboratory testing was carried out on selected soil samples recovered from the drilling program for the purpose of soil classification. A total of 9 gradation analyses and 3 Atterberg Limits test were carried out on disturbed soil samples collected from the boreholes.

4.0 SUBSURFACE CONDITIONS

Detailed descriptions of the subsurface soil and groundwater conditions encountered in the boreholes put down during our field investigation are presented in the Record of Borehole Log sheets presented in Appendix I. The results of grain size analysis testing carried out on selected samples of soil are presented in Appendix II.

The following sections provide a summary of the inferred subsurface soil and groundwater conditions at the bridge site based on the results of our field investigation.

It should be noted that sampling procedures using the SPT sampler precludes sampling of sizes larger than 35 mm in diameter, although larger sizes may be present within the different soil units. These larger particles are not reflected in the gradation data provided in this report. It should also be noted that the boreholes were drilled below the groundwater table with the air-rotary drill method. Sloughing and heave were observed during SPT due to unbalanced pressure at the base of the borehole and the penetration resistance obtained during SPT may not reflect the actual relative density of the soils.

4.1 Fill

Fill consisting of loose to very dense, moist, brown-grey sand and gravel to silty sand (with some cobbles and boulders) was encountered in the two boreholes (BH07-1S and BH07-2S) drilled near the two abutments of the existing bridge.

In BH07-1S drilled on the northeast side of the existing Slims River Bridge, the fill materials extended to a depth of about 3.3 m below ground surface. In BH07-2S drilled on the southwest side of the existing Slims River Bridge, the fill materials extended to a depth of about 5.8 m below ground surface. Based on visual examination of the fill materials obtained from drill cuttings and split spoon sampling, it is inferred that the fill may consist of the approach embankment fills and/or the guide bank fills placed during construction of the existing bridge.

The SPT N values measured within this deposit were found to be between 12 blows/0.3m to 30 blows/0.3 m. These larger N values reflect the presence of coarser material sizes rather than the in-situ relative density.

4.2 Sand

Underlying the fill materials at boreholes BH07-1S and BH07-2S, a deposit of loose to compact, grey grading to brown, moist to wet, sand with a trace to some silt and trace gravel was encountered. Some sandy silt layers were also encountered within this predominantly sand deposit.

The cone tip resistance values up to about 100 bars were obtained within the sand deposit. The SPT N values measured within this deposit were found to be generally between 3 blows/0.3 m to 12 blows/0.3 m except in one case a SPT value of 22 blows/0.3 m was obtained. Based on these, the sand deposit is generally loose to compact in relative density.

The gradation results of representative samples taken from this deposit are shown on Figure II-1 to II-5 in Appendix II. The results indicate that samples contained 0 to 5% gravel sizes, 26 to 98% sand sizes, and 2 to 74% finer sizes. At borehole BH07-2S, a clayey silt layer was encountered at a depth of about 7.9 m below the ground surface. Thickness of this clayey silt layer was found to be about 0.3 m at this borehole location. The gradation results of representative sample taken from this deposit are shown on Figure II-6 in Appendix II. The samples tested contained 7% sand sizes and 93% finer sizes. The SPT N value obtained within this layer was found to be 22 blows/0.3m indicating that the layer is very stiff in consistency.

4.3 Silty Sand

Underlying the native sand deposit at borehole BH07-1S, a deposit of loose to compact, grey grading to brown, moist to wet, silty sand inter-layered with sandy silt was encountered. The thickness of this deposit is found to be about 12 m at the borehole BH07-1S location. Based on the cone tip resistances values recorded at the two CPT locations, the tip resistance ranged between 14 and 150 bars. The SPT N values of 2 blows/0.3 m and 3 blows/0.3 m were obtained within this deposit.

The gradation results of the sandy silt layer encountered within the silty sand deposits are shown on Figures II-7 to II-9 in Appendix II. The samples tested contained 0 to 10% sand sizes and 90 to 100% finer sizes. Atterberg Limit tests conducted on two samples of this sandy silt layer yielded the following results:

- Liquid Limit = 27.7 % and 27.6 %;
- Plastic Limit = 20.5 % and 23.5 %; and
- Natural water content = 29 % and 25.3 %.

Based on the index test results, the interlayer of sandy silt is considered to be low plastic.

4.4 Sandy Silt to Silt

Underlying silty sand layer, a deposit of sandy silt to silt was encountered at CPT locations. Based on the cone tip resistances values recorded at the two CPT locations, the tip resistance ranged between 16 and 33 bars.

Based on the index test results of samples within this layer from the 1954 investigation and the evaluation of the results of the cone penetration testing from the current 2007 investigation, the sandy silt to silt deposit is considered to be low plastic. Some clayey silt layers were also encountered within this deposit.

4.5 Groundwater Conditions

The natural groundwater level at the site is expected to vary with the water level in the river, season, and precipitation. Based on the information collected from the drilling program between May 13 and May 20, 2007, the groundwater table is encountered at a depth of about 3 m below the existing ground surface.

Based on the information provided in EarthTech Drawing No. 1702-01a, we understand that the 100-year flow in Slims River is approximately at elevation 783 m or some 5 m below the new road grade elevation.

5.0 SEISMIC DESIGN CONSIDERATIONS

5.1 Site Seismicity

Several large earthquakes have occurred in the recent past some 180 to 380 km west of the site; i.e. 1912 M7.2 Earthquake, 1958 M6.2 Earthquake, 2002 M6.7 Earthquake, and 2002 M7.9 Earthquake. These earthquakes occurred due to rupturing of the Denali/Totschunda Fault System that runs approximately in a southeast alignment. It is reported that the 2002 M7.9 earthquake resulted from predominantly a right lateral offset along portions of the fault system that ruptured over estimated length of 300 km between towns of Northway (Alaska) to the east and Cantwell (Alaska) to the west.

A site-specific seismic hazard analysis completed by the Geological Survey of Canada (GSC) has established the following ground motion parameters for the Slims River Bridge site:

TABLE 1: Site-Specific Ground Motion Parameters

Return Period	100 yr.	475 yr.	1,000 yr.
Probability of Exceedance in 50 years	40%	10%	5%
Peak ground horizontal Acceleration – Class C Soils	0.108g	0.196g	0.252g

The GSC has established the peak ground acceleration for Class C ground conditions or for soil profiles where the average shear wave velocity of soils within the upper 30 m varies between 360 m/s and 760 m/s.

The corresponding response spectra (5% damped) that are applicable for Class C ground conditions are summarized in Table 2.

TABLE 2: Site-Specific Spectral Accelerations (5% Damped)

Period (Seconds)	0.0 (PGA)	0.10	0.15	0.20	0.30	0.40	0.50	1.0	2.0
475-Year Spectral Accelerations (g)	0.196	0.275	0.385	0.410	0.366	0.307	0.258	0.143	0.079
1,000-Year Spectral Accelerations (g)	0.252	0.366	0.507	0.540	0.488	0.411	0.346	0.199	0.110

5.2 Seismic Design Criteria

The bridge is classified as a lifeline bridge. Accordingly, the design guidelines given in Section 4.4.2 of the Canadian Highway Bridge Design Code (CHBDC) S-6-06 indicates that the bridge must remain open for all traffic following a 475 year return period ground motions (10% probability of exceedance in 50 years) and also usable by emergency vehicles and for security/defence purposes immediately following a 1000 year return period ground motions.

According to section 4.4.6.1 of the CHBDC and based on soil conditions encountered at the site, the effects of site conditions on bridge response can be taken into consideration by considering a Site Coefficient (S) of 1.5 that corresponds to Soil Profile Type III.

5.3 Site Specific Ground Response Analyses

Following our discussion with EarthTech, ground response analyses were undertaken to assess the liquefaction potential of site soils under design earthquake ground motions. As the proposed bridge is designed to be a life line bridge, analyses were only carried out for the 1,000-year ground motions to assess the liquefaction potential of the site soils due to budget considerations.

Based on the available subsurface data, the subsurface conditions across the site are consistent. Therefore, the ground response analyses were carried out for a single soil column which is representative of both north and south abutments of the proposed bridge. We have carried out the ground response analyses for an average shear wave velocity (V_s) profile, which was derived, based on measured and computed shear wave velocities, shown on Figure 4.

It should be noted that the firm ground, which is defined as Class C ground conditions, is not known at the site. We have carried out the ground response analyses assuming that the firm ground is just below the maximum depth of investigation (i.e., 60 m depth) conservatively. Ground response analyses were also carried out for a firm ground depth of 100 m to assess the sensitivity of the firm ground to liquefaction potential of the site soils.

5.3.1 Firm-Ground Acceleration Time-Histories

For the site specific ground response analyses, an acceleration time history from one set of ground motions recorded during 2002 Delani Earthquake was selected as a representative time-history for the 1,000 year motions event based on distance, magnitude, and peak horizontal ground acceleration (PHGA). Records from the 1992 Landers, 1989 Loma Prieta, and 1999 Chi Chi earthquakes were also selected as representative time-histories for the 1,000 year motions event.

The above selected time histories were spectrally-matched to the GSC target spectrum for the 1,000-year motions event. Spectral matching was carried out in the time-domain using the computer program EZ-FRISK (Risk Engineering, Version 7.23).

5.4 Liquefaction Susceptibility of Site Soils

Liquefaction susceptibility of the site soils were assessed based on the results of ground response analyses carried out using the computer program ProShake. Ground response analyses were carried out for a total of eight (8) time-histories obtained from the earthquakes noted in the previous section spectrally matched to the target spectrum.

Design earthquakes of magnitude M7 to M7.3 that is representative of 10 to 15 cycles of effective loading, was used in the assessment of liquefaction susceptibility of soils under the 1,000-year ground motions.

5.4.1 Liquefaction Susceptibility of Granular Soils (Sand and Silty Sand)

The liquefaction susceptibility of granular soils was evaluated by comparing the penetration resistance required to prevent liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required. The penetration resistance required to trigger liquefaction was computed using the cyclic resistance ratio (CRR) induced by the design earthquake and NCEER (1997) liquefaction resistance charts. The variations in cyclic stress ratio (CSR) induced by the design earthquake with depth were computed using computer program ProShake.

The near-continuous profiles of equivalent SPT $(N_1)_{60}$ values obtained from SCPT 07-1S and CPT07-2S were used to assess the liquefaction potential of the sand and silty sand deposits. The results of the ground response analyses were carried out for a firm ground depth of 60 m are shown in Figure 5. As shown on Figure 5, the sand and the silty sand deposits are susceptible to liquefaction under the design 1,000-year ground motions. It is important to note that the influence of fines content within the silty sand deposits were also considered in the liquefaction assessment of the deposit and, based on our assessment, only the upper half of the silty sand layer is considered to be susceptible to liquefaction. Our analyses indicate that the potentially liquefiable zone is estimated to be about 29 m below the existing ground surface at the abutments.

Similar potential liquefiable zone was also estimated from the ground response analyses carried out for the firm ground depth of 100 m.

It should be noted that ground improvement measures are required to prevent liquefaction and associated ground failure for the proposed bridge to meet the seismic design requirements.

5.4.2 Liquefaction Susceptibility of Fine-Grained Soils (Sandy Silt to Silt)

Based on the recent Task Force Report on “Geotechnical Guidelines For Buildings On Liquefiable Sites in Greater Vancouver” dated May 8, 2007, liquefaction susceptibility of the fine-grained soils was evaluated using the criterion developed by Boulanger and Idriss (2006). Based on the criterion, soils with $PI < 7\%$ are generally susceptible to liquefaction.

Based on the results of index tests, the deeper sandy silt to silt layer underlying the site is considered to be low plastic with $PI < 7\%$, and potentially liquefiable. However, the low plastic silt deposit requires large number of effective loading cycles to undergo liquefaction. Based on the results of laboratory tests carried out by Wijewickreme et. al. (2004) on a low plastic silt deposit, for effective loading cycles of 10 to 15, the CRR required for the deposit to liquefy is about 0.17.

Figure 6 shows the CSR generated by the design earthquake motions and, as shown on Figure 6, the maximum CSR within the low plastic sandy silt to silt deposit encountered at the site is computed to be about 0.12 and, therefore, the liquefaction potential for the deposits is considered to be low. However, some strain softening or cyclic mobility will occur in this deposit as a result of seismic shaking which could lead to permanent deformations.

Based on the laboratory test results published by Wijewickreme et. al. (2004), the permanent lateral deformations within this deposit is expected to be in order of 0.3 m under the 1,000-year ground motions.

We have also estimated the vertical settlements following cessation of earthquake ground motions and the post-earthquake vertical ground surface settlements are estimated to be in the order of 150 mm for the 1,000-year ground motions in areas where ground improvement of the upper liquefiable soils are carried out as outlined below.

5.5 Ground Improvement Measures to Mitigate Liquefaction

As mentioned in the previous sections, liquefaction mitigation measures are required to minimize the risk of abutment and approach embankment slope failures and to reduce the liquefaction-induced lateral ground movements at the bridge foundations.

Based on soil conditions encountered at the site, ground improvement measures involving vibro-replacement (i.e., stone columns) can be used. Provision should be made to carry out local excavation to remove or relocate large size cobbles or boulders and/or rip rap materials that may obstruct vibro probe installation.

5.5.1 Soil Improvement Requirements for Foundations and Abutments

As discussed in section 5.4.1 and shown on Figure 5, the sand and silty sand deposits underlying the site is susceptible to liquefaction. It is recommended that the proposed ground improvement zone should be directly beneath the entire foundation footprint plus a minimum horizontal distance outside the foundation footprint depending on the pile cap geometry. The depth of ground improvement should be at least 25 m below the pile cap.

It is important to note that the approach fills will also undergo significant deformation due to liquefaction of the site soils and the ground improvement zone should be extended adequately to protect abutments, settlement slab areas, etc. For preliminary design purposes, a minimum ground improvement zone of 17 m x 17 m in plan is assumed based on seismic performance requirements.

6.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS

Geotechnical engineering analyses were carried out to provide geotechnical design input on selected aspects of the design and construction of the proposed new bridge foundations. We have also assessed the requirements for site preparation and engineered fill placement from a geotechnical perspective.

The following sections present the results of our analyses together with our comments and recommendations.

6.1 Subgrade Preparation

Given the soils at the bridge site is granular in nature, it is anticipated that the proposed site grade increase is achievable with properly designed side slopes and associated rip rap protection. Free slopes of the filled embankment should be developed not steeper than 2 horizontal to 1 vertical.

Engineered fill should consist of 75 mm minus pit run sand and gravel, or equivalent, with a grain size distribution that falls within the envelope shown in Table 3.

TABLE 3: Recommended Gradation For Engineered Backfill

Sieve Size (mm)	75	37.5	19	4.75	1.18	0.3	0.075
Percent Passing (%)	100	30 - 100	20 - 100	10 - 60	6 - 32	4 - 15	0 - 5

Consideration may be given to the use of clean, well-graded on-site sand and gravel subject to adequate testing of representative samples and review by geotechnical engineer. These fills should be placed in lifts not exceeding 300 mm in loose thickness and compacted to a minimum of 95 percent of Standard Proctor maximum dry density immediately behind (within ~1 to 2 m) the integral abutments.

Temporary cut slopes may be excavated using slopes that are no steeper than 1.3H:1V. If work is carried out during or following wet weather conditions, or if the water content of the subgrade soils is relatively high, it is recommended that proof rolling be carried out using a static roller to reduce the risk of "pumping" or "weaving" of subgrade soils.

If work is carried out during winter months with subzero temperatures, it is recommended that the stockpiled fill materials and any unfinished work surfaces be covered at the end of each working day to reduce the risk of snow/ice contamination of the fill materials. Compaction of wet or snow/ice contaminated materials under subzero temperature is not recommended.

Any soft or disturbed zones or pockets of soils, or soils containing organic materials, encountered during excavation and proof-rolling should be sub-excavated and replaced with well-compacted engineered fill or select sand and gravel excavation materials, or equivalent.

6.2 Piled Foundations

As mentioned before, the bridge site is located in a zone with high seismic risk, and the lateral stiffness required for the foundation piles to resist seismic loading could be quite high. Therefore, consideration should be given to use of large diameter (0.6 m or larger) steel pipe piles for foundations support. In addition, use of steel pipe piles has the following benefits:

- The compressive stresses induced in the piles during driving could be high due to presence of coarse grained soils including boulders at the site and the steel pipe piles would be able to withstand such relatively high level of stresses; and
- The steel pipe piles would be able to handle any required length of extension quite easily.

We have estimated geotechnical ultimate axial compression capacities for 610 mm, 762 mm, and 914 mm diameter steel pipe piles driven closed-end with improved ground conditions and the table below summarizes the estimated axial compression capacities for the piles. Settlements of the piles under static loading conditions are expected to be less than 25 mm.

TABLE 4a: Estimated Geotechnical Ultimate Axial Compression Capacity

Diameter (mm)	Length of Piles (m)	Ultimate Capacity (kN)	Resistance* Factor	Factored Resistance (kN)
610	20	3000	0.5	1500
762	20	4500	0.5	2250
914	20	6000	0.5	3000

* Based on conducting appropriate number of PDA tests

A geotechnical resistance factor of 0.5 is recommended for the computation of the factored resistance for ultimate limit state (ULS) design, provided that field verification of pile capacity would be carried out using Pile Driving Analyzer (PDA) testing during construction. A resistance factor of 0.4 should be used if verification testing is not considered.

The estimated geotechnical factored resistance under tension/uplift conditions are given in Table 4b below. These are for piles driven 20 m into the improved/densified ground below the pile cap level and are based on a geotechnical resistance factor of 0.3 which is the recommended value as per Table 6.1 of Section 6.6.2.4 of the Canadian Highway Bridge Design Code (CHBDC) S-6-06.

TABLE 4b: Estimated Geotechnical Factored Resistance Under Tension/Uplift

Dia. (mm)	Length of Piles (m)	Factored Resistance Under Tension/Uplift (kN)
610	20	690
762	20	860
914	20	1035

6.3 Pile Driveability Analyses

We have carried out driveability analyses using the computer program GRL-WEAP (Version 2005, by Goble Rausche Likins and Associates, Inc.) to assess the final set requirements for the 914 mm diameter steel pipe piles to develop the required capacity, and to estimate the potential stresses induced in the pile during installation. The analyses were carried out for the 914 mm diameter steel pipe piles driven open-ended to a depth of 20 m below the underside of the pile cap. The piles were assumed to be driven following ground improvement work at the abutment areas. The wall thickness of the 914 mm piles were assumed to be 19 mm based on the information provided by Earth Tech (Drawing No 01-1702-06 dated April 2008).

Since the actual hammer that will be used for the installation of the piles is unknown at the time of the preparation of this report, we have assumed a diesel hammer (Delmag D62-22) with a maximum rated energy of 223 kJ (165 kip-ft) per blow for the drivability analyses. The results of our analyses with a D 62-22 hammer are summarized in Table 4c.

TABLE 4c: Results of WEAP Analyses With D 62-22 Hammer

Pile Diameter (mm)	Ultimate Capacity (kN/pile)	Estimated Final Set (Blows/25mm)	Compressive Stress (MPa)	ENTHU (kJ)
914	6000	7	~175	~90

Note: ENTHU - Energy transferred at pile top

The analyses indicate that a final set of 7 blows/25 mm is required for the 914 mm diameter pile to achieve the ultimate capacity indicated in Table 4a. The Delmag D62-22 diesel hammer is expected to operate at a stroke height of 2.8 m and a strike rate of about 39 blows per minute.

The pile driveability analyses should be revisited and the final set requirements should be revised once the information of actual pile driving hammer is available prior to the commencing of pile installation.

6.4 Soil Springs

Soil response for laterally loaded piles can be modeled using non-linear “p-y” curves. The “p-y” curves are dependent on various parameters, including pile diameter and depth. For the preliminary design, the “p-y” curves for the 610 mm and 914 mm diameter piles were developed using API guidelines, and are provided in Tables 5 and 6, respectively.

It should be noted that these “p-y” curves are dependent on the diameter of the piles and depth. Modifications would be required if the diameter is different than assumed. The “p-y” curves between depths given can be linearly interpolated

6.5 Lateral Earth Pressure Coefficients – Static Loading Conditions

The lateral earth pressure coefficients on the walls of the integral abutments should be estimated using the chart shown in Figure C6.16 of the Canadian Highway Bridge Design Code (Commentary on CAN/CSA-S6-06, page 243) as a function of the wall movement or rotation. A copy of this chart is included in this report as Figure 7. The curve labeled as ‘medium dense’ in Figure 7 is appropriate for backfill compacted to 95 percent of Standard Proctor density.

It should be noted that the K_p values shown on Figure 7 represent the passive resistance of the soil in an *unfrozen* state. The passive resistance of soil in a *frozen* state will be much higher than the resistance of soil in an *unfrozen* state.

An at-rest lateral earth pressure coefficient of 0.40 is recommended for the design of non-yielding abutment walls (or walls that are not allowed to translate or rotate) under static loading conditions. A minimum soil unit weight of 20 kN/m^3 should be used when computing the lateral earth pressures.

It should be noted that the lateral earth pressure values are dependent on the interface friction between the wall and backfill and the amount of permissible wall movement.

6.6 Lateral Earth Pressure Coefficients – Seismic Loading Conditions

The lateral earth pressure on the abutments under seismic loading conditions can be determined using the Mononobe-Okabe formulation. The conventional Mononobe-Okabe formulation estimates the combined static and lateral pressure coefficient (K_{ae}) that is applicable to a wall that can move outwards and develop active pressure coefficient in the soils retained. However, if the wall is held or “restrained” against movement, the lateral earth pressures developed against the wall are larger than the pressure computed from the conventional Mononobe-Okabe formulation.

In practice, the increased lateral earth pressure behind restrained walls is computed using the Mononobe-Okabe formulation following the guidelines provided in ATC-6 (1982) and CSA-S6 (2006) for restrained wall conditions. Based on our calculations, it is recommended that a dynamic lateral earth pressure coefficient (K_{ae}) of 0.50 be used in design for the 1,000 year ground motions under restrained wall condition.

Alternatively, abutment walls may be designed with a seismic lateral earth pressure coefficient (K_{ae}) of 0.31, corresponding to a yielding wall condition, provided lateral movement of up to 25 mm can be accommodated.

The following additional assumptions have been made in the derivation of seismic lateral earth pressure coefficients:

- Backfill retained by the wall is dry or moist;
- Backfill retained by the wall is horizontal;
- No backfill surcharge is included;
- Backfill friction angle of 36 degrees;
- Interface friction angle of 18 degrees between the wall and backfill; and,
- The peak horizontal ground surface acceleration for the 1,000 year ground motions is 0.21g.

The combined static and seismic lateral earth pressure (P_{ae}) against the abutment wall should be calculated using the following equation;

$$P_{ae} = \frac{1}{2} (K_{ae}) \gamma H^2$$

where, H is the embedded height of the wall and γ is the saturated unit weight of backfill. A saturated unit weight of 20 kN/m³ should be used in computing the lateral earth pressures. The computed triangular earth pressure distribution should be distributed as an inverted triangle over the depth of the embedded portion of the wall with the apex of the triangle at the base of the wall. The effects of water pressure, where applicable, should be added to the lateral earth pressures computed above. For the portion of soil below the water table, a submerged unit weight of 10.2 kN/m³ should be used when computing the lateral earth pressures.

7.0 STABILITY OF ABUTMENT SLOPES

Engineering analyses were carried out to assess the stability of the post-construction abutment slopes as shown in EarthTech Drawings Nos. 1702-01a. The global stability of the bridge abutments was evaluated using the computer program SLOPE/W (Version 6.16). The water level in the river was taken as 783 m. A minimum ground improvement zone of 17 m x 17 m in plan was assumed in the analyses. The depth of ground improvement was assumed to be about 25 m below the pile cap level.

Under static loading conditions, our analyses indicate that the static factor of safety against an overall slip circle failure of the new abutment slope is approximately 1.53 or greater which is considered to be adequate. The results of the slope stability analyses are presented in Figure 8.

Under the 1,000 year ground motions, the pseudo-static factor of safety against an overall failure of the abutment slope is estimated to be about 0.93. The results of the slope stability analyses are presented in Figure 9.

Stability analyses were also conducted for the 1,000 year ground motions for the post liquefaction case. The pseudo-static factor of safety against an overall failure of the abutment slope following liquefaction under the 1,000 year return period seismic event is approximately 1.15 (see Figure 10). This indicates that flow slide following the seismic event involving the abutment slope should not be expected.

The above analyses indicate that the seismic loading-induced soil deformations at the abutments are expected to be small following the 475 and 1,000 year seismic events, provided that the ground improvement and site preparation measures identified in sections 5 and 6 are adhered to during construction.

8.0 FROST ACTION

The bridge site is located at high latitude with long cold winters. Available climatic data for a nearby location of the bridge site has a *mean* freezing index of about 3500+ degree C days (Canadian Foundation Engineering Manual). If we extrapolate from a US Corps of Engineers empirical relationship, it is estimated that the mean frost penetration in a well-drained non-frost susceptible granular material is of the order of 3.0 m if there is no snow cover.

Due to the presence of highly frost susceptible soils at the project site, and the large frost heave induced deformations occurred on the original bridge during the period between 1943 and 1955, suitable engineering measures should be implemented to provide sufficient protection to the bridge foundations against the potential frost hazard. The engineered fill to be used for construction of the new bridge abutments should consist of non-frost susceptible, clean, well-graded, granular materials.

Based on the preliminary design drawing (General Arrangement, drawing #1702-01 dated April 11, 2008) provided to us by Earth Tech, the granular fill extends to about 2.0 m below the underside of the pile cap at the north (towards Alaska) abutment and the south (towards Whitehorse) abutment. This is considered adequate to reduce frost heave hazard.

Based on our discussions with Earth Tech during a meeting on August 3, 2007, we understand that installation of drains pipes near the base of the granular fill will be considered. Installation of drains is considered one of the practical engineering measures that can help reducing the future risk of frost heave hazard, and has been implemented in bridges constructed in the cold region. It is recommended that the drains be incorporated.

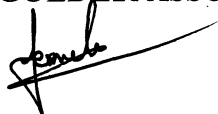
9.0 CLOSURE

It is recommended that the geotechnical aspects of the final design and specifications be reviewed prior to tendering. Provision should be made for periodic geotechnical field review, geotechnical inspection during ground improvement, pile installation, verification testing and subgrade preparations to permit confirmation that the actual subgrade conditions and construction operations are as anticipated and in overall conformance with our recommendations.


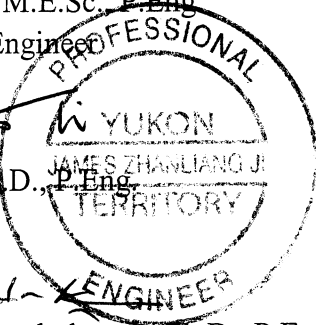
We trust that this report provides the information required. Should you have any questions or require any further information regarding the above, please do not hesitate to contact us.

Yours very truly,

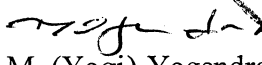
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O:\Final\2007\1411\07-1411-0005\8000\0609_08\Frpt-0609_08 Yukon-Slims River Bridge Replacement.Doc

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

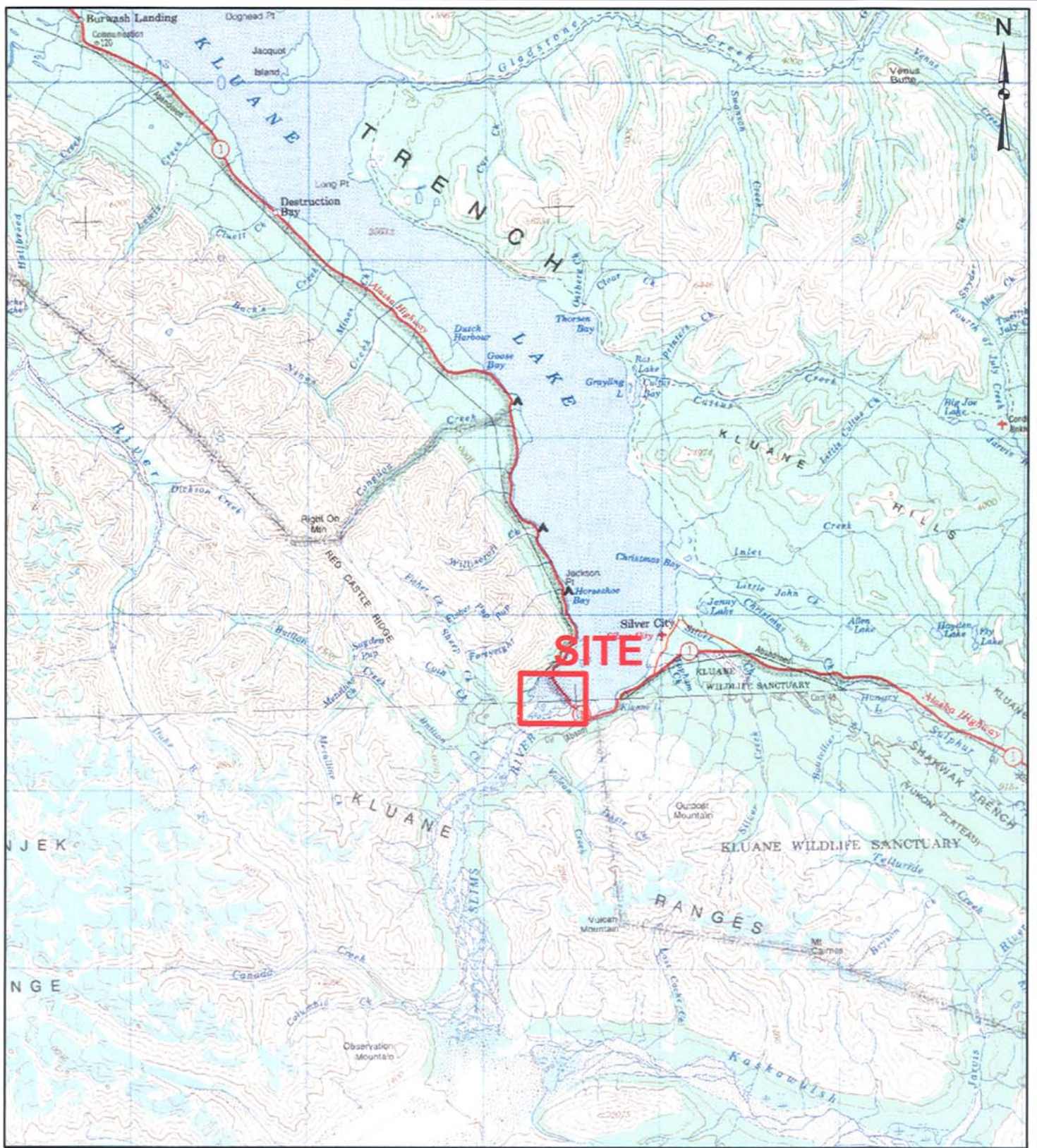
Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system

Table 5: p-y Curves for 610 mm Pipe Pile

Depth below Pile Cap (m)	P (kN/m)																			
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0008	21.7	43.1	65.4	87.6	109.8	131.9	154.1	176.2	198.3	220.4	242.5	264.6	286.6	308.7	330.7	352.8	374.8	396.9	418.9	441.0
0.0017	41.1	80.3	125.7	170.9	215.9	260.7	305.3	349.9	394.3	438.8	483.1	527.0	570.9	614.8	658.7	702.6	746.5	790.5	834.4	878.3
0.0025	57.0	108.5	177.5	246.6	315.2	383.3	451.1	518.6	585.8	652.9	719.7	785.1	850.5	915.9	981.3	1046.8	1112.2	1177.6	1243.0	1308.5
0.0034	69.0	128.0	219.3	312.4	405.3	497.6	589.2	680.2	770.8	861.1	950.5	1036.9	1123.3	1209.7	1296.2	1382.6	1469.0	1555.4	1641.8	1728.2
0.0042	77.4	140.6	251.2	367.6	484.9	601.7	717.8	833.0	947.6	1061.6	1174.0	1280.7	1387.5	1494.2	1600.9	1707.6	1814.4	1921.1	2027.8	2134.5
0.0051	83.2	148.3	274.8	412.4	553.4	694.9	835.7	975.7	1114.8	1253.0	1388.7	1515.0	1641.2	1767.4	1893.7	2019.9	2146.2	2272.4	2398.7	2524.9
0.0059	87.0	152.9	291.6	447.8	611.2	776.8	942.4	1107.3	1271.3	1434.2	1593.5	1738.3	1883.2	2028.1	2172.9	2317.8	2462.6	2607.5	2752.4	2897.2
0.0068	89.4	155.6	303.3	475.3	659.0	847.6	1037.6	1227.5	1416.5	1604.5	1787.4	1949.9	2112.4	2274.9	2437.4	2599.8	2762.3	2924.8	3087.3	3249.8
0.0076	91.0	157.2	311.4	496.3	698.1	908.2	1121.6	1336.0	1550.1	1763.2	1969.8	2148.9	2328.0	2507.1	2686.1	2865.2	3044.3	3223.4	3402.4	3581.5
0.0085	91.9	158.1	316.9	512.1	729.6	959.3	1195.0	1433.2	1671.9	1910.2	2140.4	2335.0	2529.6	2724.2	2918.7	3113.3	3307.9	3502.5	3697.1	3891.7
0.0093	92.6	158.6	320.7	523.9	754.7	1002.1	1258.6	1519.6	1782.4	2045.4	2299.0	2508.0	2717.0	2926.0	3135.0	3344.0	3552.9	3761.9	3970.9	4179.9
0.0102	93.0	158.9	323.2	532.7	774.6	1037.6	1313.2	1595.8	1881.8	2169.0	2445.5	2667.9	2890.2	3112.5	3334.8	3557.2	3779.5	4001.8	4224.1	4446.5
0.0229	93.6	159.3	328.3	556.7	843.6	1186.8	1582.6	2026.2	2511.6	3032.9	3544.2	3866.4	4188.6	4510.9	4833.1	5155.3	5477.5	5799.7	6121.9	6444.1
0.6329	93.6	159.3	328.3	556.9	845.0	1192.7	1599.9	2066.7	2593.0	3178.9	3769.8	4112.5	4455.2	4798.0	5140.7	5483.4	5826.1	6168.8	6511.5	6854.2
1.2429	93.6	159.3	328.3	556.9	845.0	1192.7	1599.9	2066.7	2593.0	3178.9	3769.8	4112.5	4455.2	4798.0	5140.7	5483.4	5826.1	6168.8	6511.5	6854.2
1.8529	93.6	159.3	328.3	556.9	845.0	1192.7	1599.9	2066.7	2593.0	3178.9	3769.8	4112.5	4455.2	4798.0	5140.7	5483.4	5826.1	6168.8	6511.5	6854.2

Table 6: p-y Curves for 914 mm Pipe Pile

Depth below Pile Cap (m) y (m)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	p (kN/m)																			
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0013	32.5	64.7	96.8	130.2	163.6	196.8	230.1	263.3	296.5	329.7	362.9	396.0	429.2	462.3	495.4	528.6	561.6	594.6	627.7	660.7
0.0025	61.7	121.1	180.4	248.6	316.5	384.2	451.6	518.8	585.9	652.8	719.6	786.3	853.0	919.6	986.1	1052.6	1118.6	1184.4	1250.2	1316.0
0.0038	85.6	165.0	244.0	347.2	451.0	554.3	657.2	759.6	861.6	963.2	1064.5	1165.6	1266.4	1367.0	1467.5	1567.9	1666.5	1764.5	1862.5	1960.6
0.0051	103.7	195.9	287.9	423.7	562.7	702.4	841.7	980.4	1118.4	1255.8	1392.6	1528.9	1664.8	1800.3	1935.5	2070.4	2201.1	2330.5	2460.0	2589.5
0.0063	116.6	216.4	316.1	479.7	651.4	826.4	1002.2	1177.8	1352.7	1526.7	1700.0	1872.5	2044.3	2215.5	2386.2	2556.3	2718.6	2878.5	3038.4	3198.3
0.0076	125.3	229.3	333.5	518.9	719.3	927.2	1136.2	1350.2	1562.2	1773.4	1983.8	2193.3	2401.9	2609.6	2816.5	3022.7	3215.8	3404.9	3594.1	3783.2
0.0089	131.1	237.3	343.9	545.7	769.9	1006.9	1250.9	1498.0	1746.3	1994.7	2242.4	2489.3	2735.2	2980.1	3224.0	3467.0	3689.9	3907.0	4124.0	4341.1
0.0102	134.9	242.0	350.0	563.6	806.7	1068.8	1342.4	1622.5	1905.8	2190.4	2475.2	2759.5	3043.0	3325.4	3606.8	3887.1	4139.0	4382.4	4625.9	4869.4
0.0114	137.3	244.9	353.5	575.3	833.0	1116.1	1415.7	1725.8	2042.0	2361.5	2682.4	3003.7	3324.6	3644.7	3963.8	4281.7	4561.4	4829.8	5098.1	5366.4
0.0127	136.8	246.6	355.6	583.0	851.8	1151.7	1473.7	1810.6	2157.1	2509.5	2865.2	3222.4	3580.1	3937.6	4294.3	4650.1	4956.4	5248.0	5539.6	5831.1
0.014	139.8	247.6	356.8	588.0	864.9	1178.4	1519.1	1879.5	2253.5	2636.3	3024.9	3416.8	3810.3	4204.5	4598.5	4991.9	5323.6	5636.8	5949.9	6263.1
0.0152	140.4	248.2	357.5	591.2	874.2	1198.1	1554.4	1935.1	2333.4	2744.2	3163.5	3588.2	4016.3	4446.1	4876.7	5307.3	5663.0	5996.2	6329.3	6662.4
0.0343	141.5	249.0	358.4	597.0	895.1	1252.4	1668.2	2141.4	2669.9	3251.1	3851.7	4558.1	5276.3	6032.4	6822.3	7642.2	8207.2	8690.0	9172.8	9655.5
1.8623	141.5	249.0	358.4	597.0	895.1	1252.8	1670.1	2146.9	2683.3	3279.2	3934.7	4649.7	5424.3	6258.4	7152.1	8105.4	8729.6	9243.1	9756.6	10270.1
2.7763	141.5	249.0	358.4	597.0	895.1	1252.8	1670.1	2146.9	2683.3	3279.2	3934.7	4649.7	5424.3	6258.4	7152.1	8105.4	8729.6	9243.1	9756.6	10270.1



LEGEND
 Project Site

6 3 0 6
 SCALE 1:300,000 KILOMETRES

PROJECT GOVERNMENT OF YUKON
 SLIMS RIVER BRIDGE REPLACEMENT KM 1702
 ALASKA HWY, YUKON

TITLE
KEY PLAN

REFERENCE
 NTS 250k provided by Geogratis - Canmatrix
 Datum: NAD 83 Projection: UTM Zone 7



PROJECT No. 07-1411-0005	SCALE AS SHOWN	REV. 0
DESIGN VF 17 Sept. 2007		
GIS AL 17 Sept. 2007		
CHECK		
REVIEW		


FIGURE 1

Photograph of Slims River Bridge – Looking North

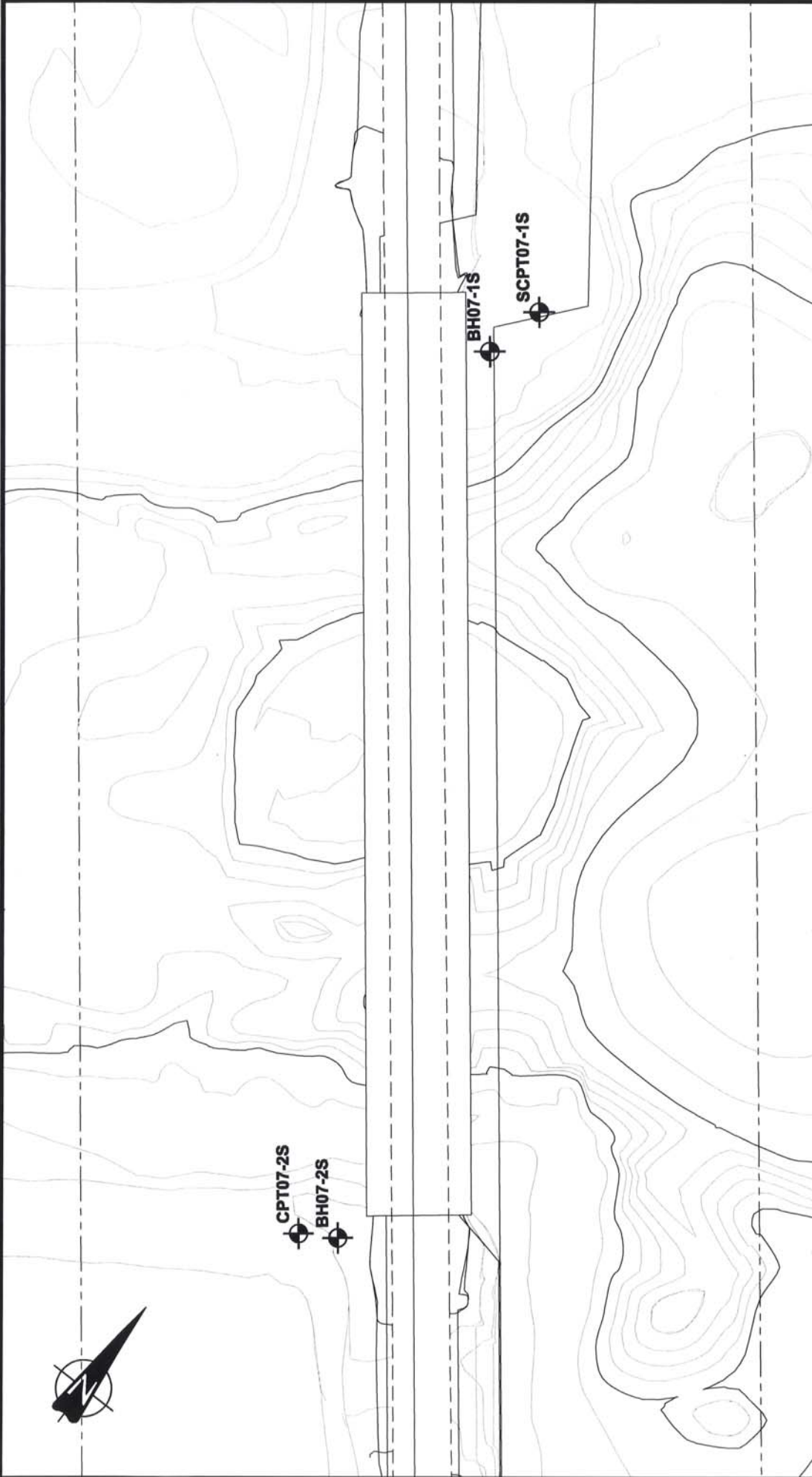


Floodplain of Slims River adjacent to the existing Bridge – Looking West



PROJECT		GOVERNMENT OF YUKON SLIMS RIVER BRIDGE REPLACEMENT KM 1702, ALASKA HWY, YUKON	
TITLE		Photographs of Existing Slims River Bridge	
	PROJECT No.	07-1411-0005	PHASE / TASK No. 6000
	DESIGN	VF 14SEP07	SCALE NTS REV.
	CADD	--	
	CHECK	MY 14SEP07	
REVIEW			FIGURE 2

REVISION DATE: BY: FILE:



PROJECT
 GOVERNMENT OF YUKON
 SLIMS RIVER BRIDGE REPLACEMENT
 KM 1702, ALASKA HWY, YUKON

TITLE
 BOREHOLE LOCATION PLAN

PROJECT No.	07-1411-0005	FILE No.	0714110005_1000_A_01
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CADD	M/JH	17SEP07	REV. 0
CHECK			
REVIEW			

FIGURE 3

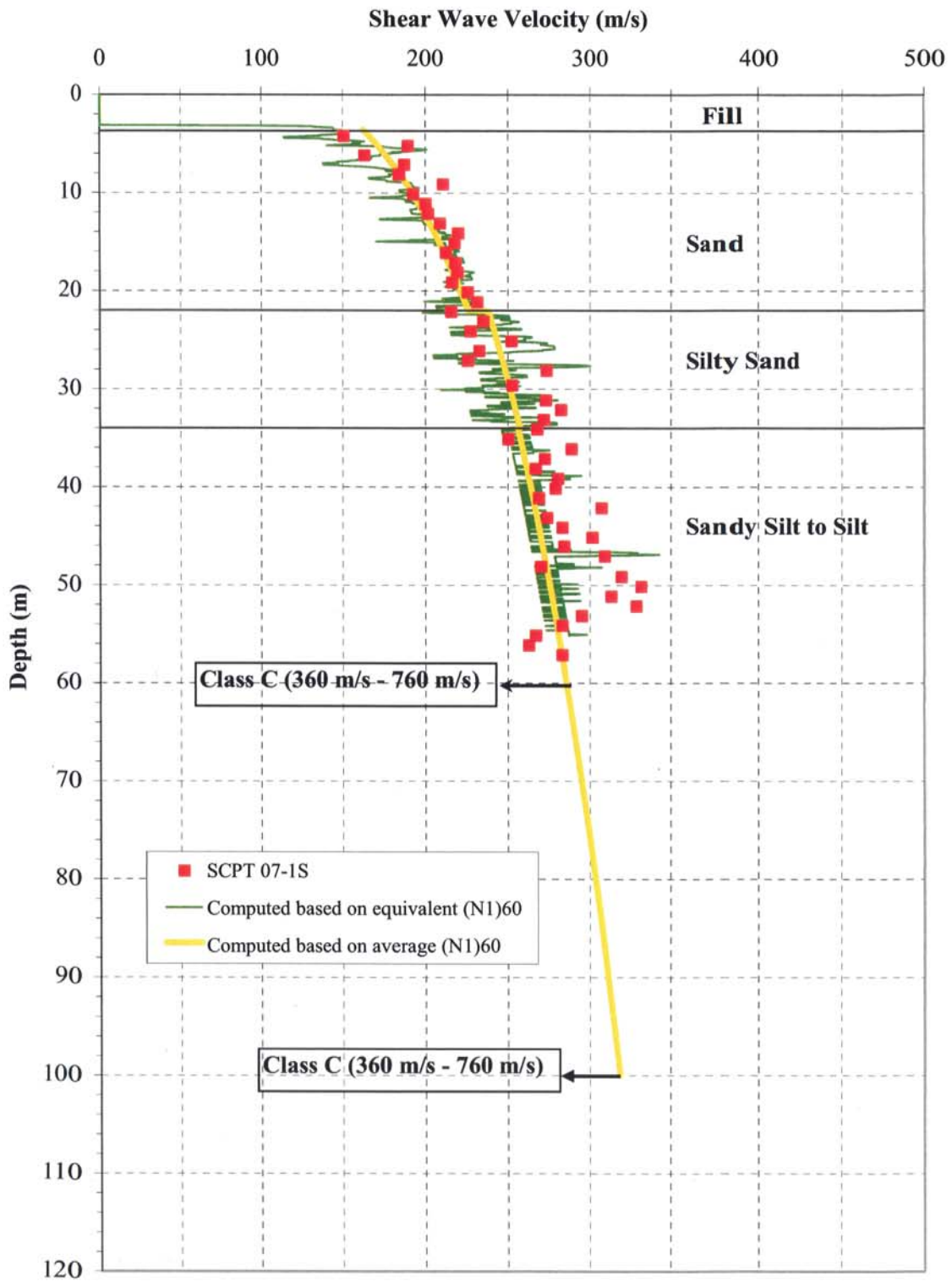
LEGEND

Test Hole Location

REFERENCES

Base plan provided by EarthTech, drawing no. SK-01, dated July, 2005.

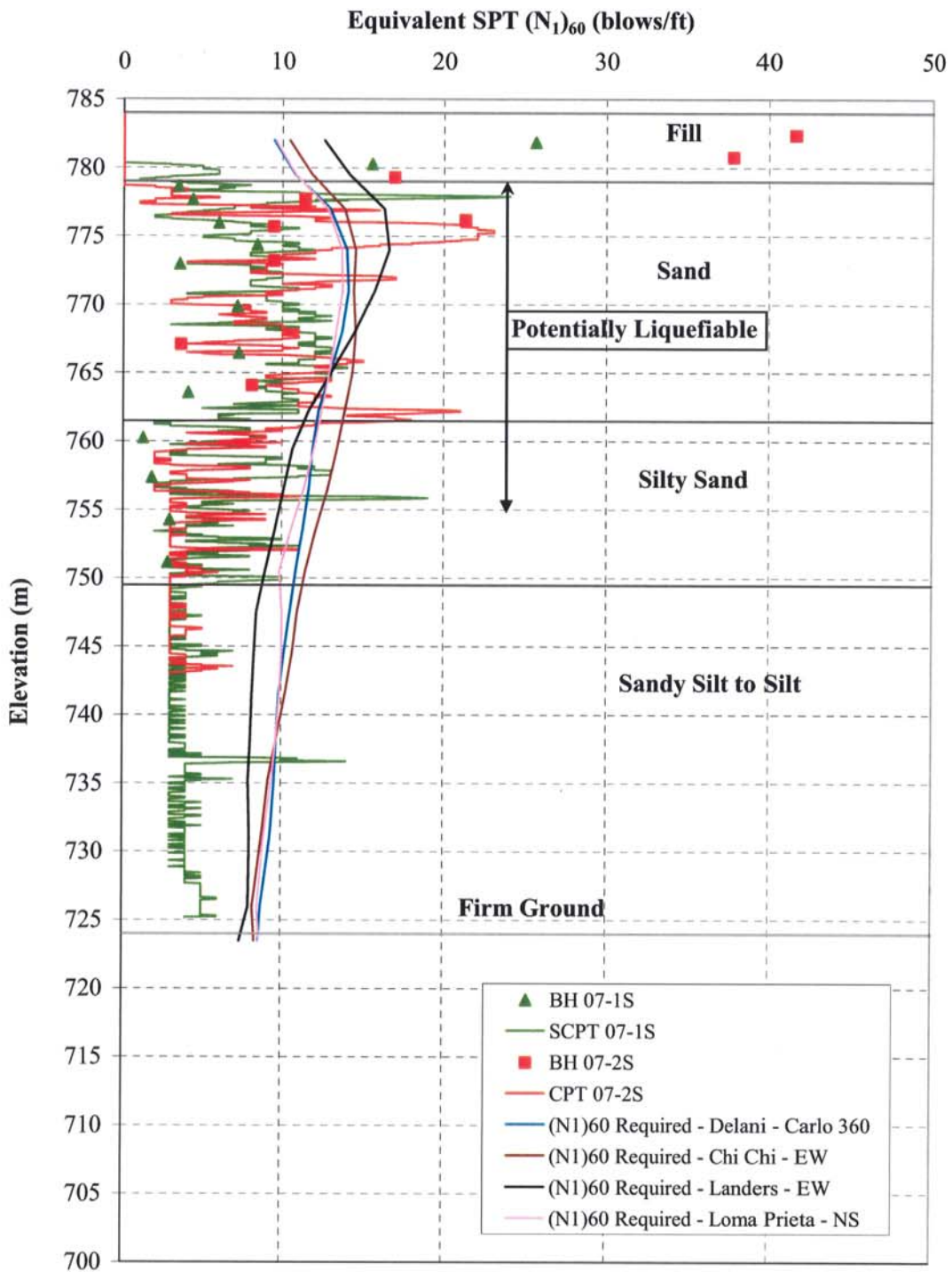




REVISION DATE: BY: FILE:

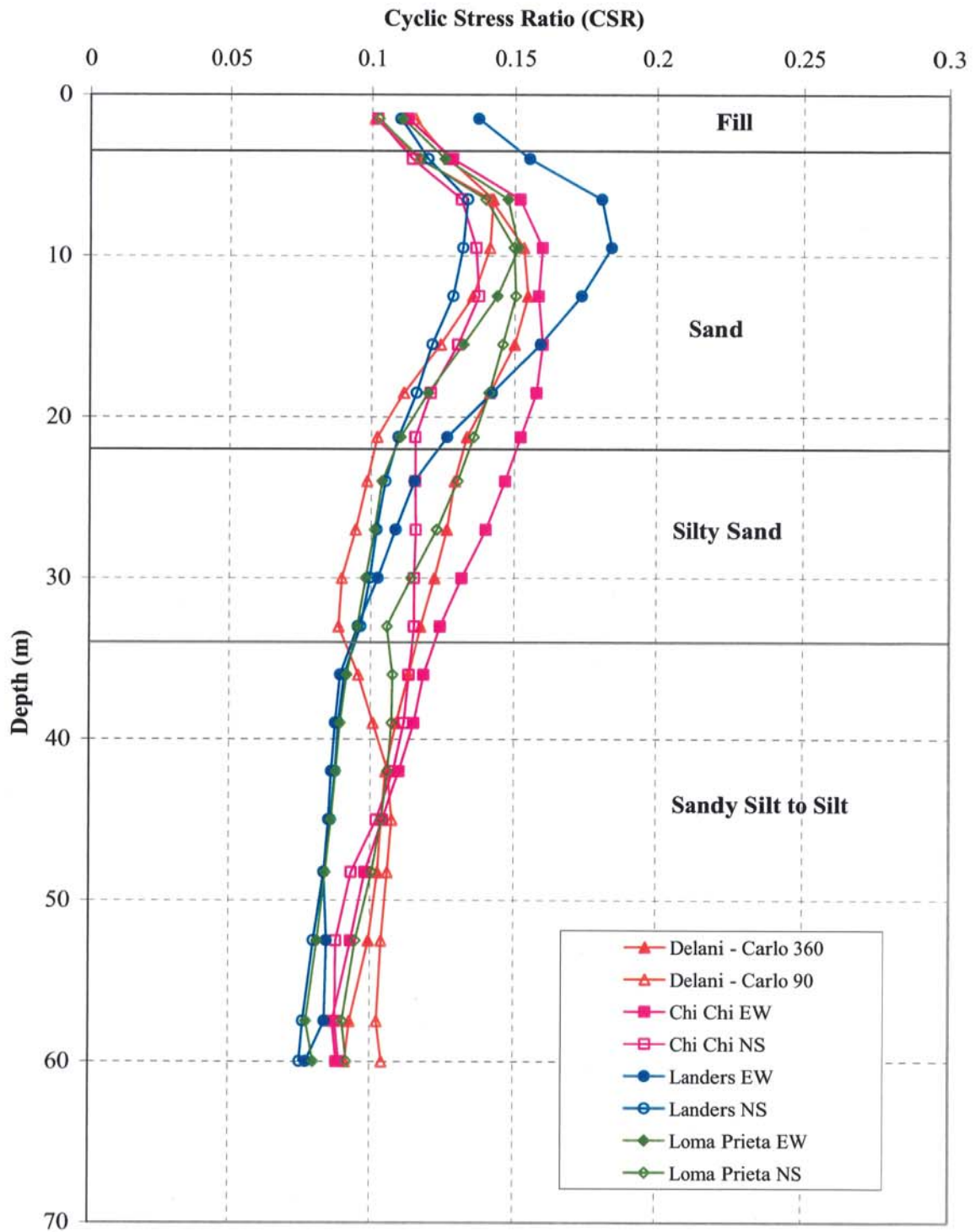
PROJECT		GOVERNMENT OF YUKON SLIMS RIVER BRIDGE REPLACEMENT KM 1702, ALASKA HWY, YUKON	
TITLE		Measured and Computed Shear Wave Velocity Profiles	
PROJECT No. 07-1411-0005		PHASE / TASK No. 6000	
DESIGN	VF	14SEP07	SCALE NTS REV.
CADD	--		
CHECK	MY	14SEP07	FIGURE 4
REVIEW			





PROJECT				GOVERNMENT OF YUKON SLIMS RIVER BRIDGE REPLACEMENT KM 1702, ALASKA HWY, YUKON			
TITLE				Predicted Zone of Liquefaction (1000-year Ground Motions)			
		PROJECT No. 07-1411-0005		PHASE / TASK No. 6000			
		DESIGN	VF	14SEP07	SCALE	NTS	REV.
		CADD	--				
		CHECK	MY	14SEP07	FIGURE 5		
		REVIEW					

REVISION DATE: BY: FILE:



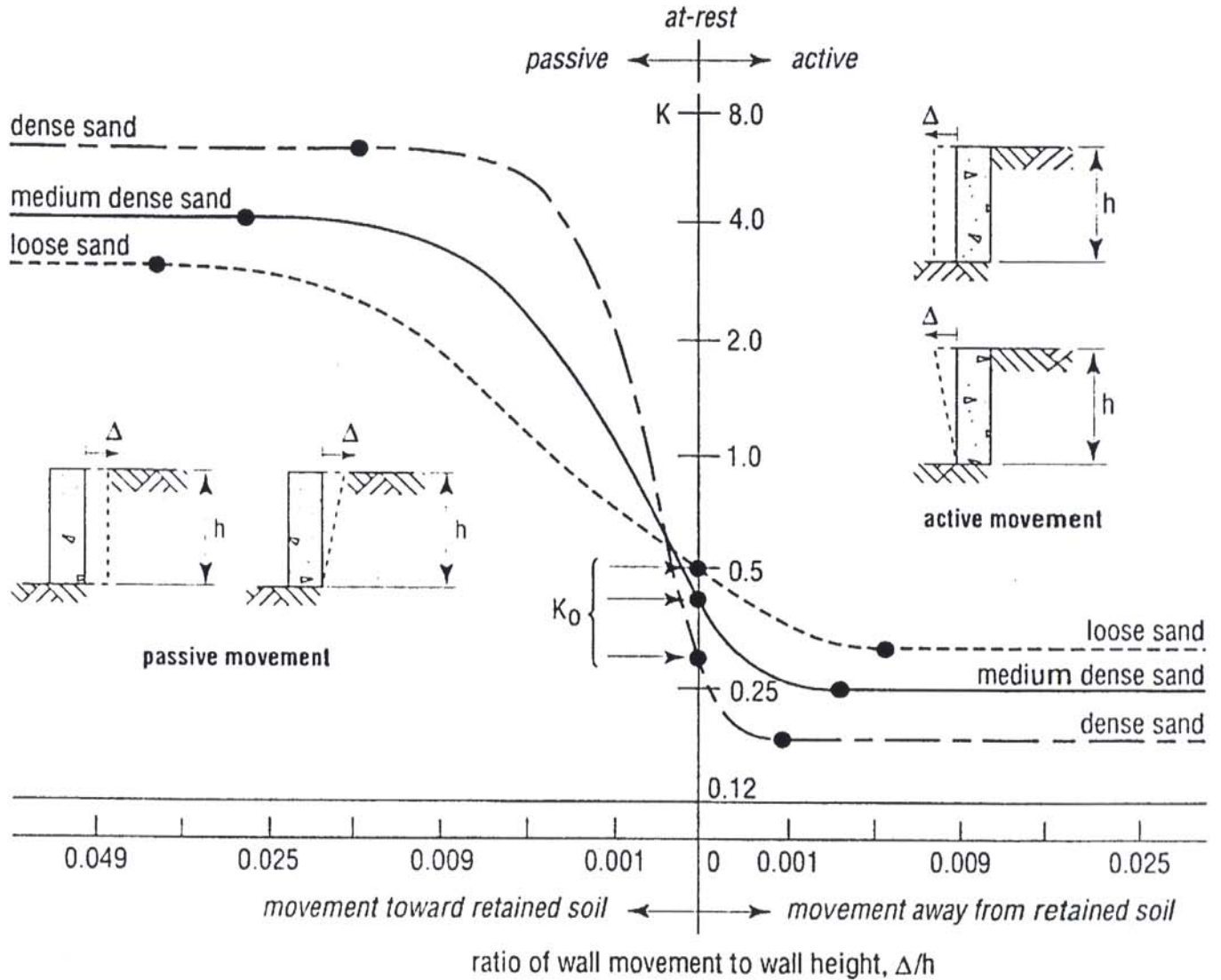
PROJECT		GOVERNMENT OF YUKON	
		SLIMS RIVER BRIDGE REPLACEMENT	
		KM 1702, ALASKA HWY, YUKON	
TITLE			
Profiles of Cyclic Stress Ratio			
(1000-year Ground Motions)			
PROJECT No. 07-1411-0005		PHASE / TASK No. 6000	
DESIGN	VF	14SEP07	SCALE NTS REV.
CADD	--		
CHECK	MY	14SEP07	FIGURE 6
REVIEW			



REVISION DATE: BY: FILE:

CADD FILE: N:\Bur-Craphics\Projects\2007\1411\07-1411-0005\Drafting\2000 Duke\071411005-2000-A_04.dwg

REVISION DATE: 07/09/18 09:52AM By: MHintoy



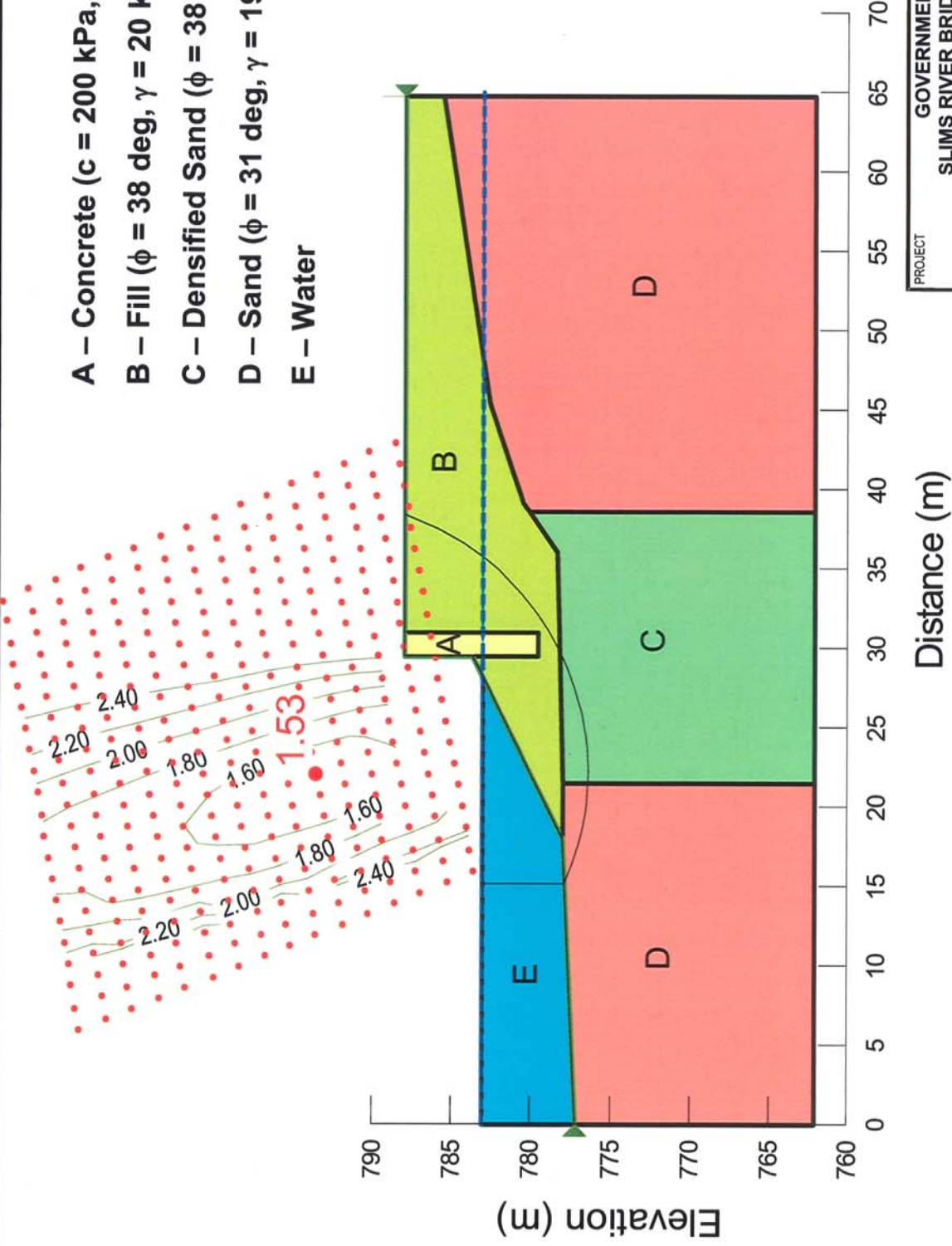
REFERENCE

1) Canadian Highway Bridge Design Code (commentary on CAN/CSA-S6-06, Page 243).

PROJECT		GOVERNMENT OF YUKON SLIMS RIVER BRIDGE REPLACEMENT KM 1702, ALASKA HWY, YUKON		
TITLE				
VARIATION OF EARTH PRESSURE WITH LATERAL WALL MOVEMENT				
PROJECT No. 07-1411-0005		FILE No.		
DESIGN	MY	25JUN07	SCALE	NTS REV. -
CADD	BAD	25JUN07	FIGURE 7	
CHECK	MY	25JUN07		
REVIEW	MY	25JUN07		



- A – Concrete ($c = 200 \text{ kPa}$, $\gamma = 24 \text{ kN/m}^3$)
- B – Fill ($\phi = 38 \text{ deg}$, $\gamma = 20 \text{ kN/m}^3$)
- C – Densified Sand ($\phi = 38 \text{ deg}$, $\gamma = 20 \text{ kN/m}^3$)
- D – Sand ($\phi = 31 \text{ deg}$, $\gamma = 19 \text{ kN/m}^3$)
- E – Water



PROJECT
 GOVERNMENT OF YUKON
 SLIMS RIVER BRIDGE REPLACEMENT
 KM 1702, ALASKA HWY, YUKON

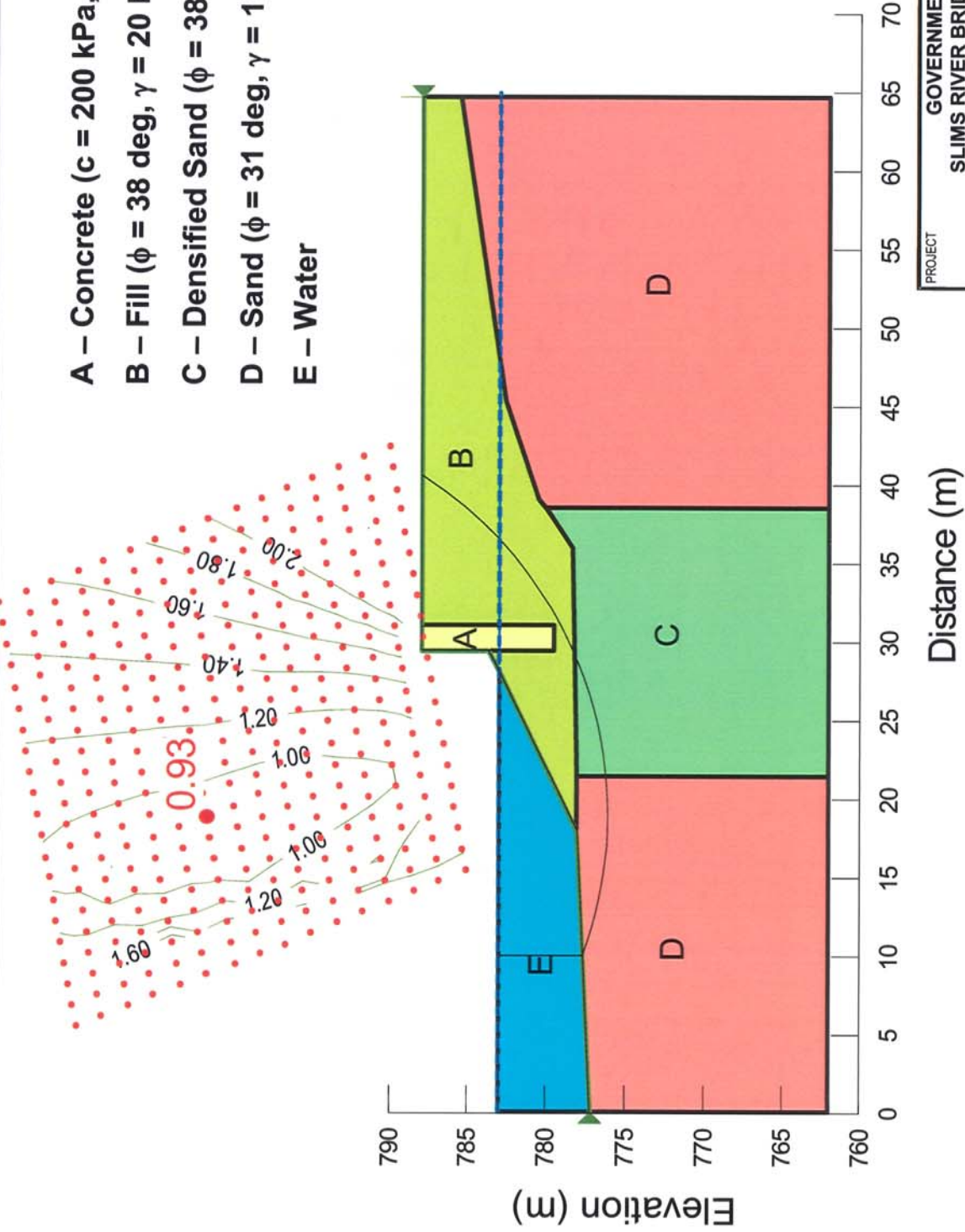
TITLE
 Global Stability Analysis
 Static Condition



PROJECT No.	07-1411-0005	PHASE / TASK No.	7000
DESIGN	YP	09JAN08	SCALE AS SHOWN / REV.
CADD	--		
CHECK	MY	10JAN08	
REVIEW	MY	10JAN08	

FIGURE 8

- A – Concrete ($c = 200 \text{ kPa}$, $\gamma = 24 \text{ kN/m}^3$)
- B – Fill ($\phi = 38 \text{ deg}$, $\gamma = 20 \text{ kN/m}^3$)
- C – Densified Sand ($\phi = 38 \text{ deg}$, $\gamma = 20 \text{ kN/m}^3$)
- D – Sand ($\phi = 31 \text{ deg}$, $\gamma = 19 \text{ kN/m}^3$)
- E – Water

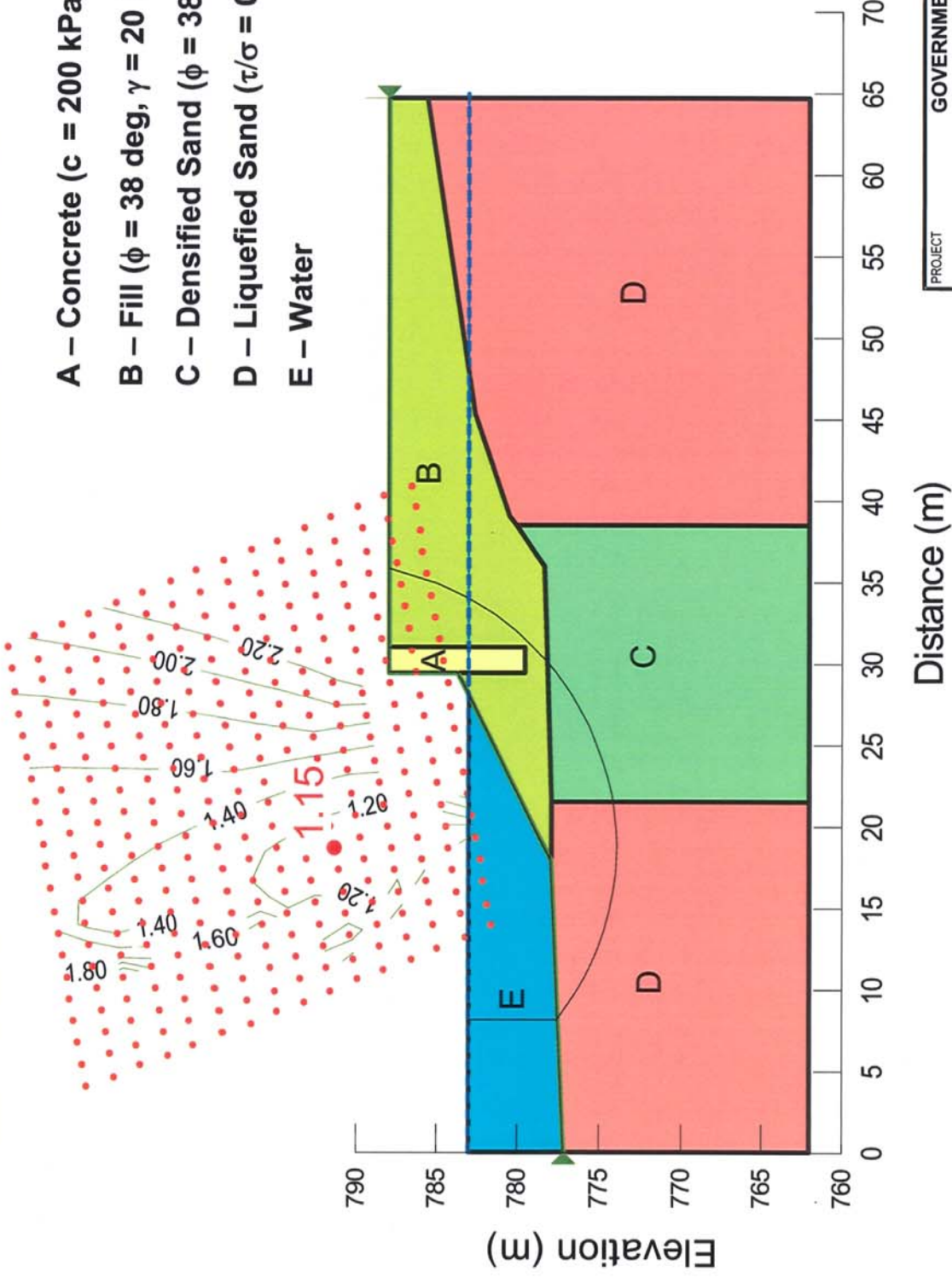


PROJECT		GOVERNMENT OF YUKON	
SLIMS RIVER BRIDGE REPLACEMENT		SLIMS RIVER BRIDGE REPLACEMENT	
KM 1702, ALASKA HWY, YUKON		KM 1702, ALASKA HWY, YUKON	
TITLE			
Global Stability Analysis			
Seismic Condition (1000 year Ground Motion)			
PROJECT No. 07-1411-0005		PHASE / TASK No. 7000	
DESIGN	YP	10JAN08	SCALE AS SHOWN / REV.
CADD	--	10JAN08	
CHECK	MY	10JAN08	
REVIEW	MY	10JAN08	



FIGURE 9

- A – Concrete ($c = 200 \text{ kPa}$, $\gamma = 24 \text{ kN/m}^3$)
- B – Fill ($\phi = 38 \text{ deg}$, $\gamma = 20 \text{ kN/m}^3$)
- C – Densified Sand ($\phi = 38 \text{ deg}$, $\gamma = 20 \text{ kN/m}^3$)
- D – Liquefied Sand ($\tau/\sigma = 0.1$, $\gamma = 19 \text{ kN/m}^3$)
- E – Water



PROJECT		GOVERNMENT OF YUKON	
		SLIMS RIVER BRIDGE REPLACEMENT KM 1702, ALASKA HWY, YUKON	
TITLE		Global Stability Analysis Post Seismic Condition (Liquefied Sand)	
PROJECT No. 07-1411-0005		PHASE / TASK No. 7000	
DESIGN	YP	10/JAN/08	SCALE AS SHOWN / REV.
CADD	--		
CHECK	MY	10/JAN/08	
REVIEW	MY	10/JAN/08	
			FIGURE 10



APPENDIX I
BOREHOLE LOGS

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PILOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH				WATER CONTENT PERCENT					
								Cu, kPa		nat V. rem V.		Q - U		Wp			W
0		Ground Surface		783.00 0.00													
1		Loose to compact, moist, silty SAND, some gravel, contains cobbles and boulders. (Berm FILL) - boulder from 0.61m - 1.07m depth. - cobbles at 1.83m depth.															
2		Compact to dense, moist SAND and GRAVEL, some silt, contains cobbles and boulders. (Berm FILL) - cobbles at 3.05m depth.		781.48 1.52	1	50 DO	16										
3					2	50 DO	12										
4				779.34 3.66													
5	Geotech Drilling Track Mounted Mud Rotary	Loose, wet, grey, fine to medium SAND, some silt, trace gravel. - gravelly layer from 4.88m - 5.18m depth.			3	50 DO	3										
6				777.21 5.79	4	50 DO	4									M	
7		Loose, wet, grey, fine to medium SAND, trace to some silt, some sandy silt layers.			5	50 DO	6										
8					6	50 DO	9										
9		Loose, wet, grey, fine to medium SAND, trace to some silt, trace sandy silt layers.		774.77 8.23													
10																	

CONTINUED NEXT PAGE

BOREHOLE 07-1411-0005.GPJ GLDR, CAN.GDT 6/9/08

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-1S

SHEET 2 OF 4

LOCATION: NE Side of Slims Bridge
N: 195584.36 E: 253550.701

BORING DATE: May 13-14, 2007

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION			
		DESCRIPTION	STRATA PILOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	2	4	6	8	10 ⁶	10 ⁵	10 ⁴			10 ³		
							SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT								
							nat V. + Q - ● rem V. ⊕ U - ○				Wp W WI								
							10 20 30 40				5 10 15 20								
10	Geotech Drilling Track Mounted Mud Rotary	Loose, wet, grey, fine to medium SAND, trace to some silt, trace sandy silt layers. <i>(continued)</i>	●	7	50 DO	4													
11																			
12																			
13																			
14							8	50 DO	9									M	
15																			
16																			
17							9	50 DO	10										
18																			
19																			
20				10	50 DO	6													
		CONTINUED NEXT PAGE																	

BOREHOLE 07-1411-0005.GPJ_GLDR_CAN.GDT 6/9/08

DEPTH SCALE

1 : 50



LOGGED: LW

CHECKED: MY

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES			DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	2	4	6	8	10 ⁵	10 ⁵	10 ⁴			10 ³	
							SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT							
							nat V. + Q. ● rem V. ⊕ U. ○				Wp ○ W WI							
							10 20 30 40				5 10 15 20							
20	Geotech Drilling Track Mounted Mud Rotary	Loose, wet, grey, fine to medium SAND, trace to some silt, trace sandy silt layers. (continued)		761.97	10	50 DO	6											
21				21.03														
22																		
23		Loose, wet, grey, silty SAND, some silt and sand layers (interlayering).			11	50 DO	2								M			
24																		
25																		
26		Loose, wet, grey, silty SAND, some silt and sand layers (interlayering).			12	50 DO	3											
27																		
28																		
29		Firm, grey, wet, sandy SILT, some clayey silt layers (interlayering).		754.04	13	50 DO	5								M			
30				28.96														
		CONTINUED NEXT PAGE																

BOREHOLE 07-1411-0005.GPJ GLDR_CAN.GDT 6/9/08

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER TYPE	BLOWS/0.3m	2	4	6	8	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴		
						SHEAR STRENGTH				WATER CONTENT PERCENT					
						Cu, kPa		nat V. + rem V. ⊕ U - ○		Wp		WI			
						10	20	30	40	5	10	15	20		
0		Ground Surface		784.08											
0.00															
1															
2						1	50 DO	26							
3		Loose to compact, moist, brown and grey, silty SAND, some gravel, contains cobbles and boulders. (Berm FILL) - boulder from 0.61m - 0.91m depth. - rock at 1.9m depth.				2	50 DO	30							
4															
5	Geotech Drilling Track Mounted Mud Rotary	- wet rock fill layer from 4.57m - 5.18m depth.				3	50 DO	15							
6				778.29											
5.79															
7		Loose to compact, wet, grey, fine to medium SAND, some sandy silt - clayey silt layer from 7.62m - 7.92m depth.				4	50 DO	11							M
8															
8.23															
8						5	50 DO	22						26.6	M
775.85															
8.23															
9		Loose to compact, wet, grey, fine to medium SAND, some sandy silt layers.				6	50 DO	10							
10															

CONTINUED NEXT PAGE

BOREHOLE 07-1411-0005.GPJ GLDR_CAN.GDT 6/9/08

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION			
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa		nat V. + rem V. ⊕ ⊙ ⊖ ⊗		WATER CONTENT PERCENT Wp — W — Wi							
10	Geotech Drilling Track Mounted Mud Rotary	- trace gravel at 13.7m, 15.2m and 16.8m depths. Loose to compact, wet, grey, fine to medium SAND, some sandy silt layers. <i>(continued)</i>																
11				7	50 DO	11												
12																		
13																		
14																		M
15																		
16																		
17																		M
18																		
19																		
20																		

CONTINUED NEXT PAGE

BOREHOLE 07-1411-0005.GPJ GLDR_CAN.GDT 6/9/08



PROJECT No.: 07-1411-0005


RECORD OF BOREHOLE: BH07-2S

SHEET 3 OF 3

LOCATION: SW Side of Slims Bridge
N: 195475.184 E: 253602.451

BORING DATE: May 16, 2007

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE				SAMPLES				DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)		NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT						
				763.66	20.42				2	4	6	8	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³			
20		Loose to compact, wet, grey, fine to medium SAND, some sandy silt layers. (continued)		10	50 DO	12	10	20	30	40	5	10	15	20					
21							End of BOREHOLE.												
22																			
23																			
24																			
25																			
26																			
27																			
28																			
29																			
30																			

BOREHOLE 07-1411-0005.GPJ GLDR CAN.GDT 6/9/08

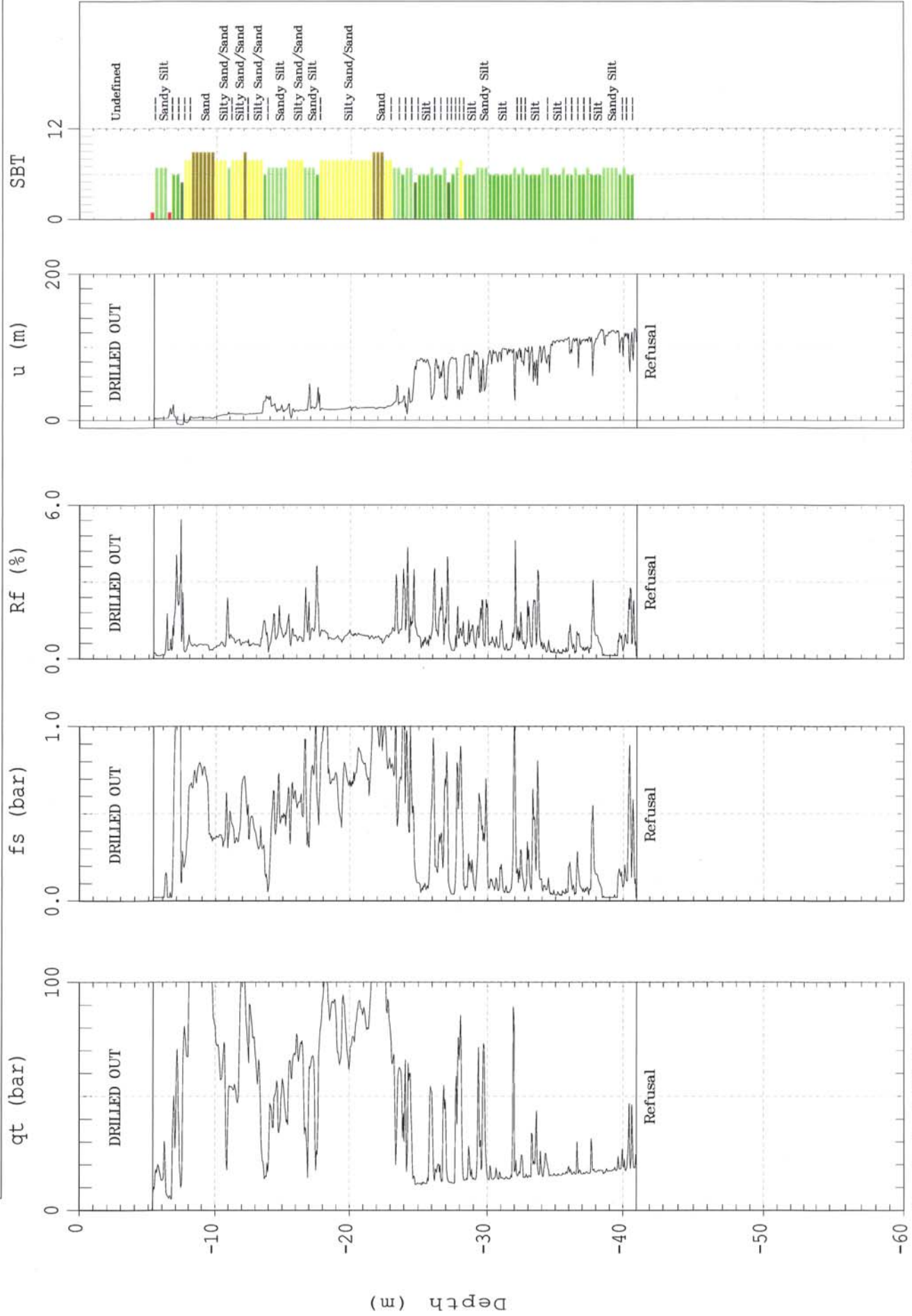
DEPTH SCALE

1 : 50



LOGGED: LW

CHECKED: MY



SBT: Soil Behavior Type (Robertson 1990)

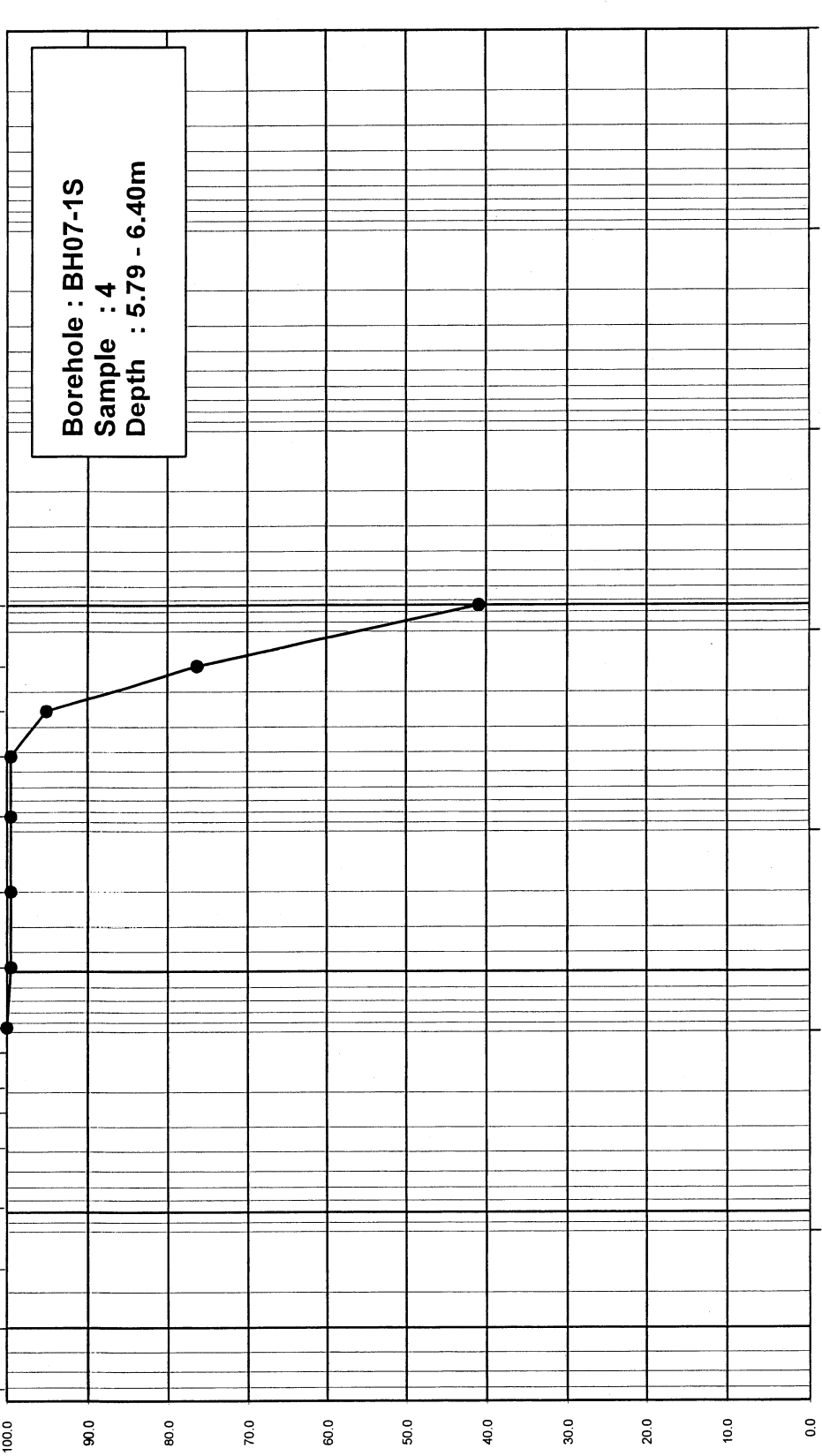
Max. Depth: 41.00 (m)
Depth Inc.: 0.050 (m)

APPENDIX II
LABORATORY TEST RESULT

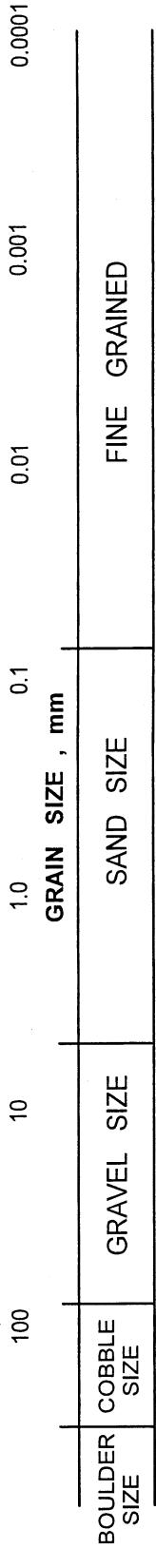
USCS GRAIN SIZE SCALE

U. S. S. sieve size, meshes / inch

Size of opening, inches



Borehole : BH07-1S
 Sample : 4
 Depth : 5.79 - 6.40m



Project No. 07-14.11-0005
 Drawn
 Reviewed
 Date09/17/07.....



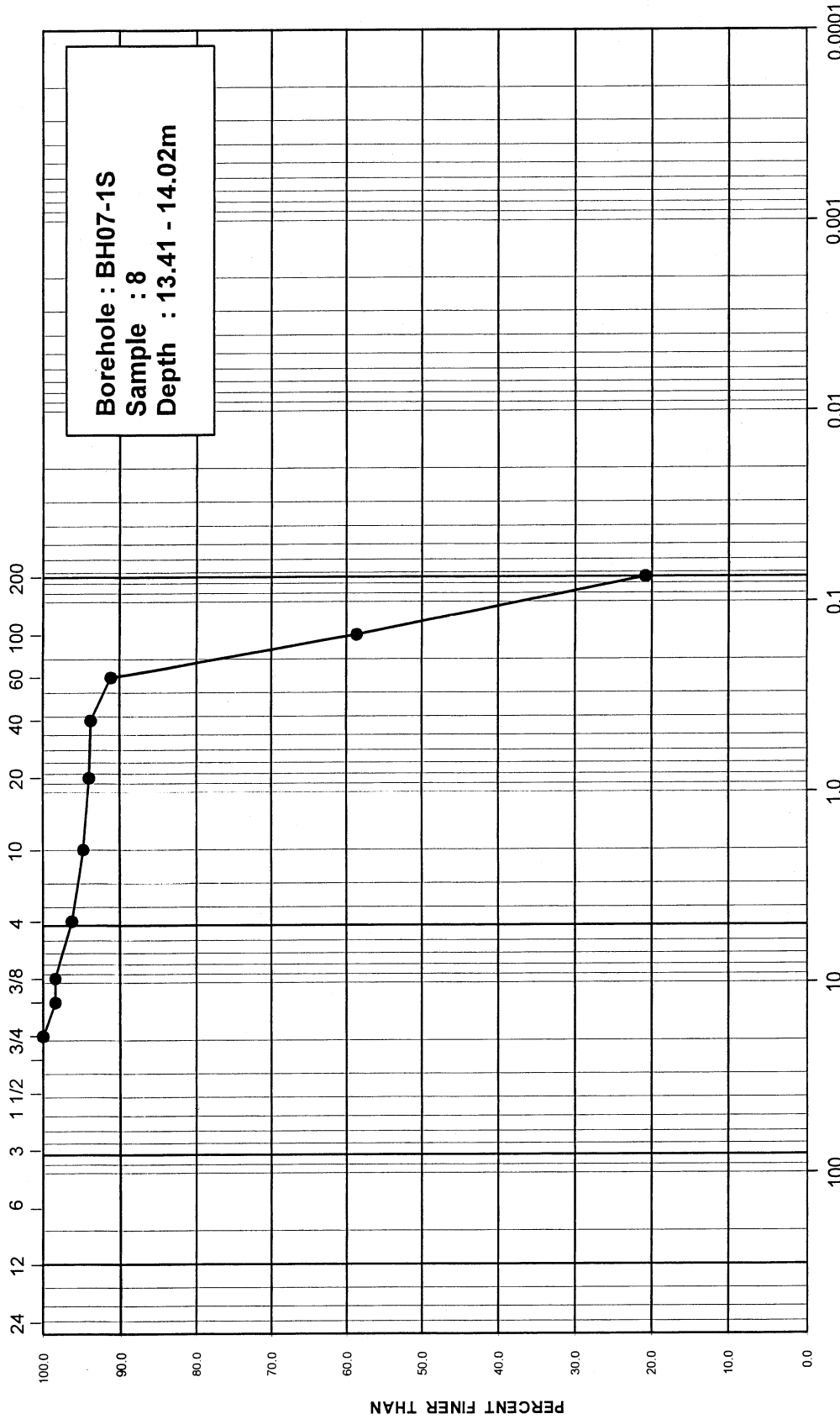
GRAIN SIZE DISTRIBUTION

Figure II-1

USCS GRAIN SIZE SCALE

U. S. S. sieve size, meshes / inch

Size of opening, inches



Borehole : BH07-1S
 Sample : 8
 Depth : 13.41 - 14.02m

BOULDER SIZE	COBBLE SIZE	GRAVEL SIZE	SAND SIZE	FINE GRAINED

Project No. 07-1411-0005
 Drawn TM
 Reviewed LL
 Date 09/17/07



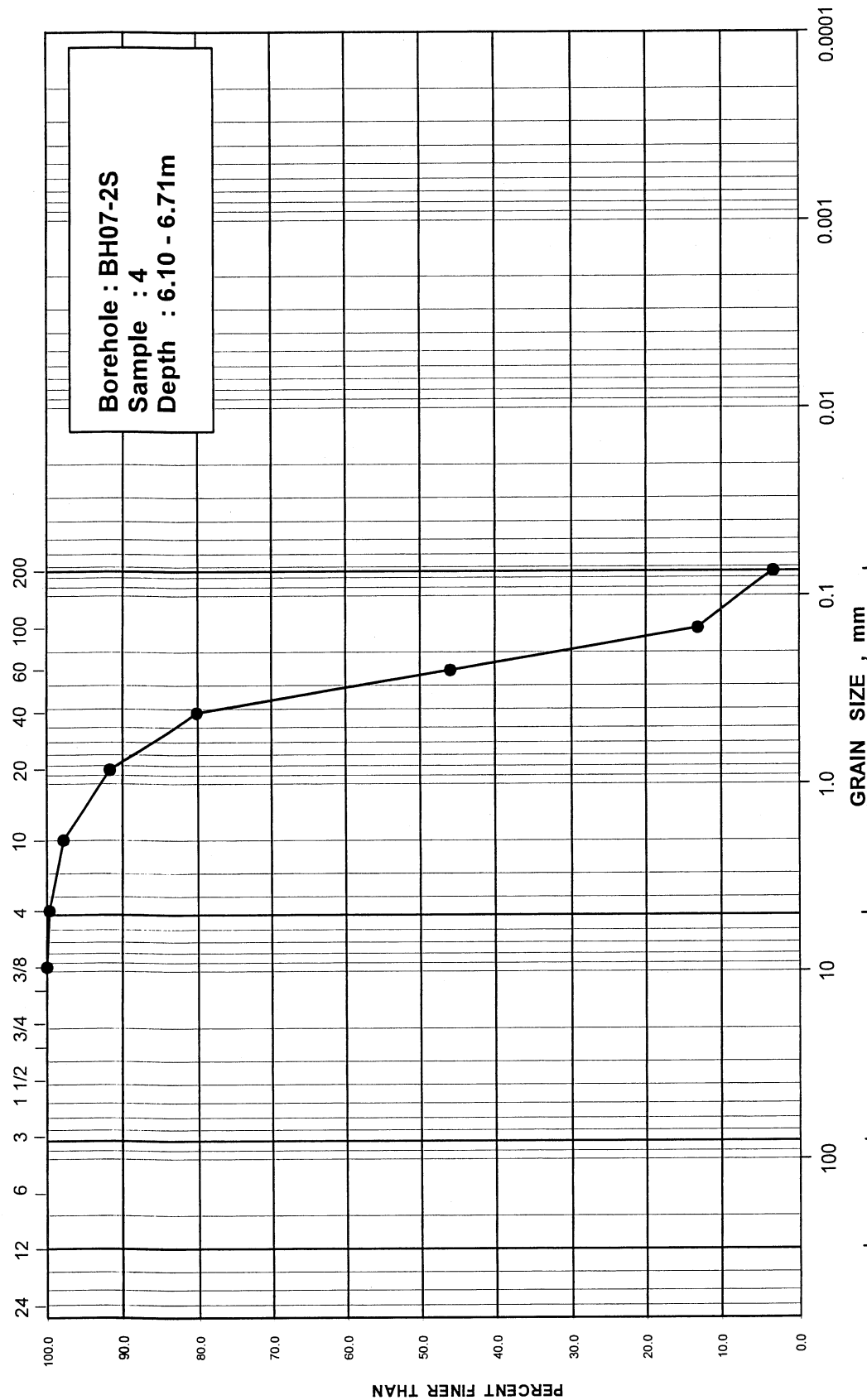
GRAIN SIZE DISTRIBUTION

Figure II-2

USCS GRAIN SIZE SCALE

U. S. S. sieve size, meshes / inch

Size of opening, inches



Borehole : BH07-2S
 Sample : 4
 Depth : 6.10 - 6.71m

BOULDER SIZE COBBLE SIZE GRAVEL SIZE SAND SIZE FINE GRAINED

Project No. 07-1411-0005
 Drawn
 Reviewed
 Date 09/17/07



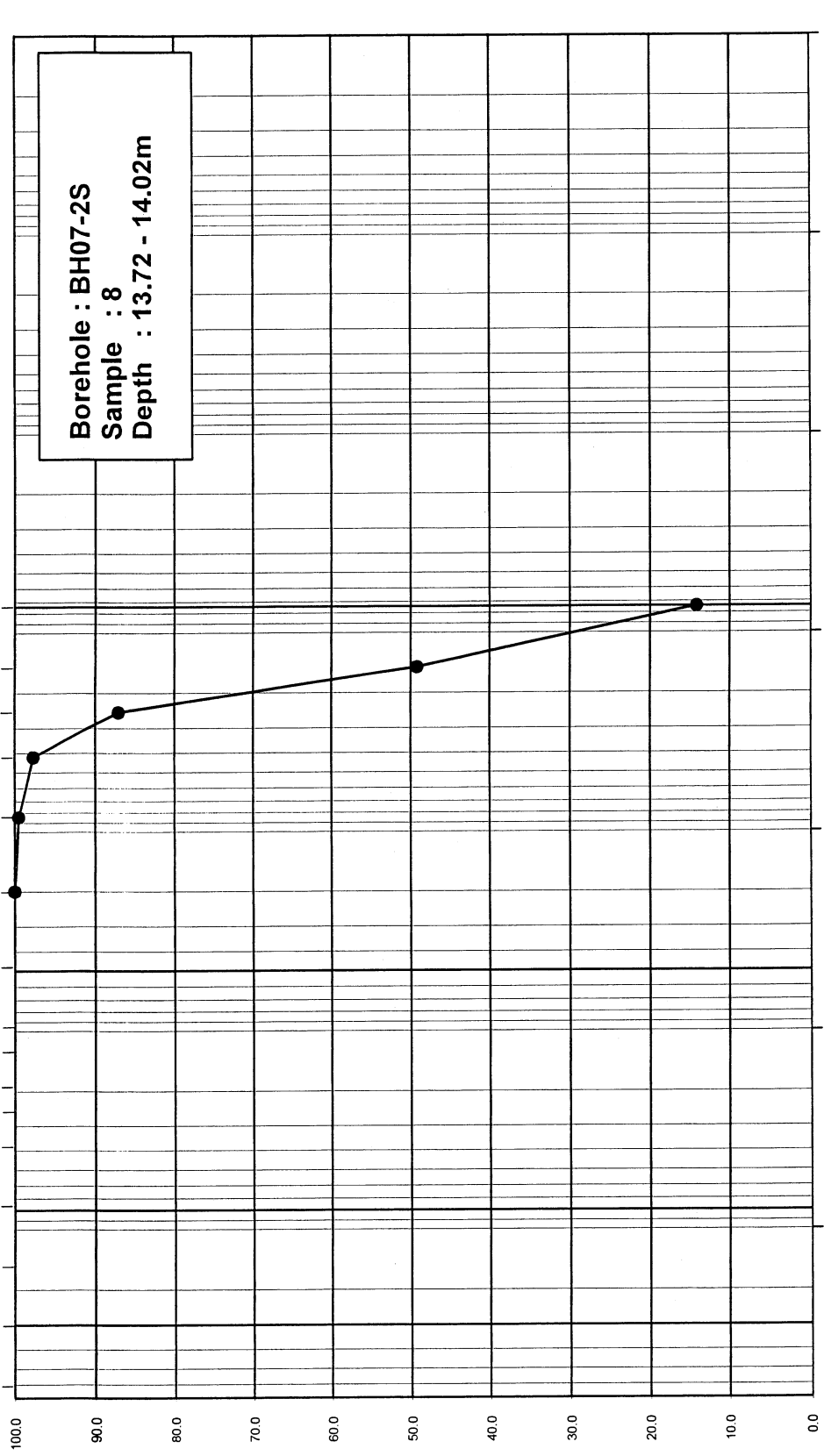
GRAIN SIZE DISTRIBUTION

Figure II-3

USCS GRAIN SIZE SCALE

U. S. S. sieve size, meshes / inch

Size of opening, inches



Borehole : BH07-2S
 Sample : 8
 Depth : 13.72 - 14.02m

BOULDER SIZE	COBBLE SIZE	GRAVEL SIZE	SAND SIZE	FINE GRAINED
--------------	-------------	-------------	-----------	--------------

Project No. 07-14.11-0005
 Drawn
 Reviewed
 Date09/17/07.....



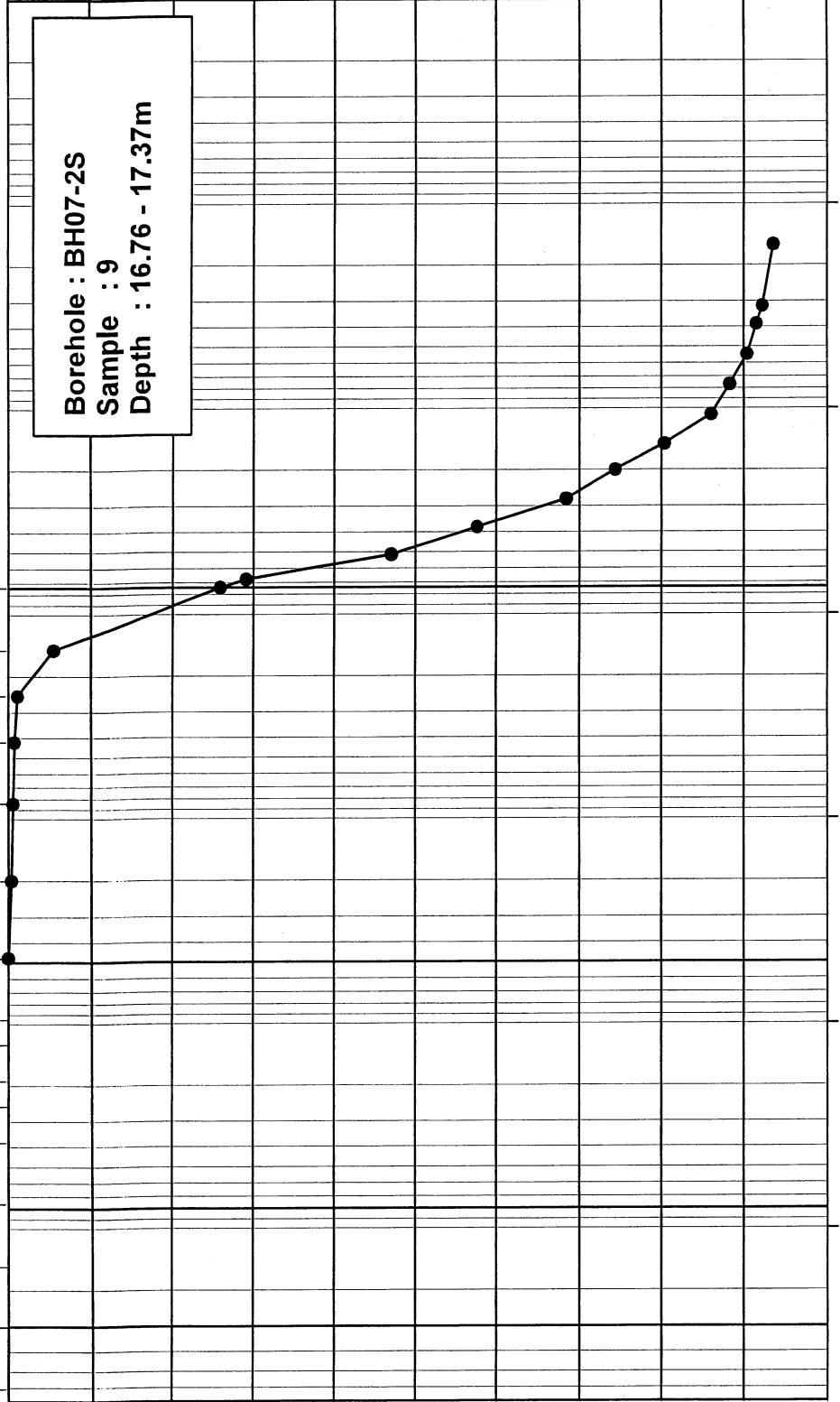
GRAIN SIZE DISTRIBUTION

Figure II-4

USCS GRAIN SIZE SCALE

U. S. S. sieve size, meshes / inch

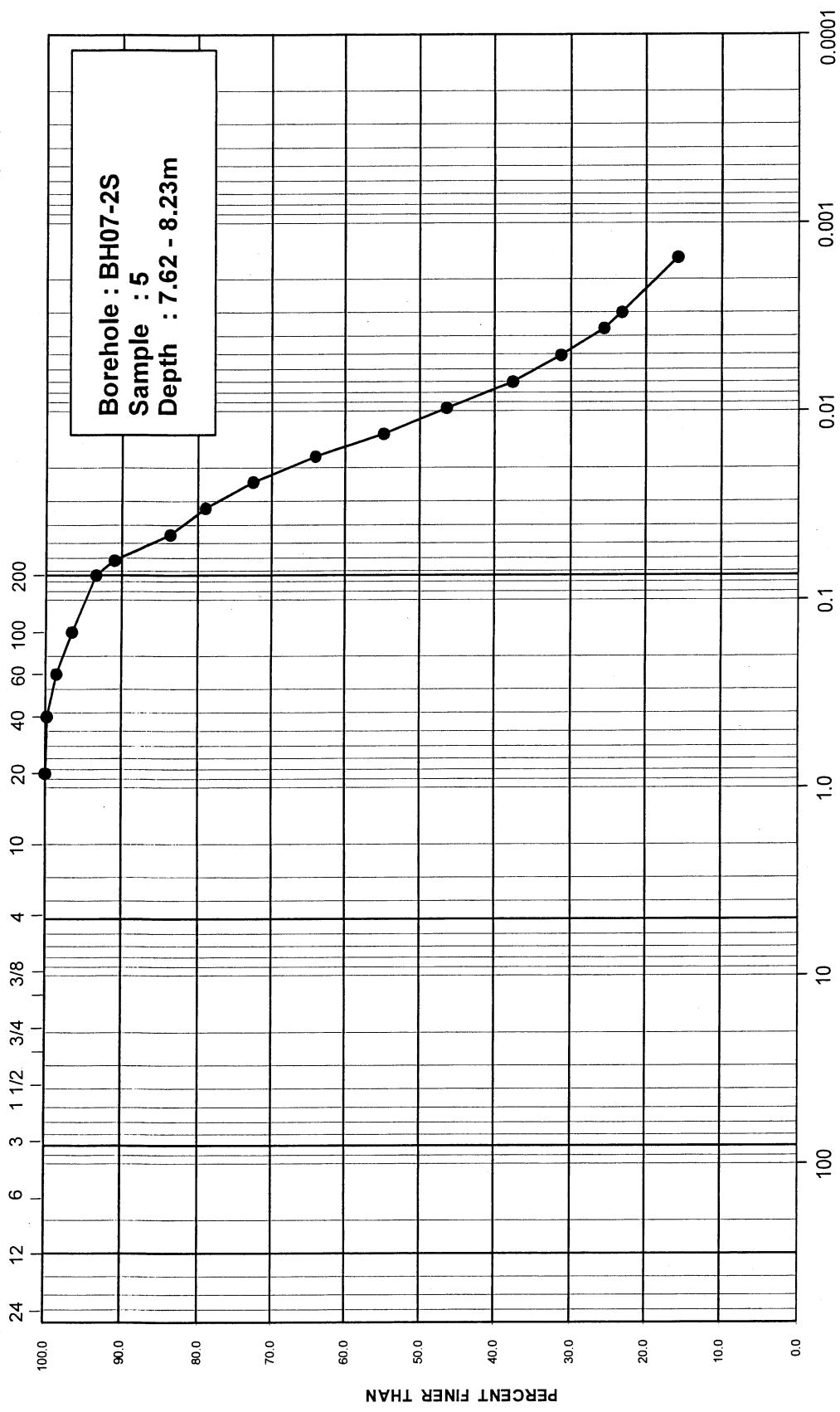
Size of opening, inches



USCS GRAIN SIZE SCALE

U. S. S. sieve size, meshes / inch

Size of opening, inches



Borehole : BH07-2S
 Sample : 5
 Depth : 7.62 - 8.23m

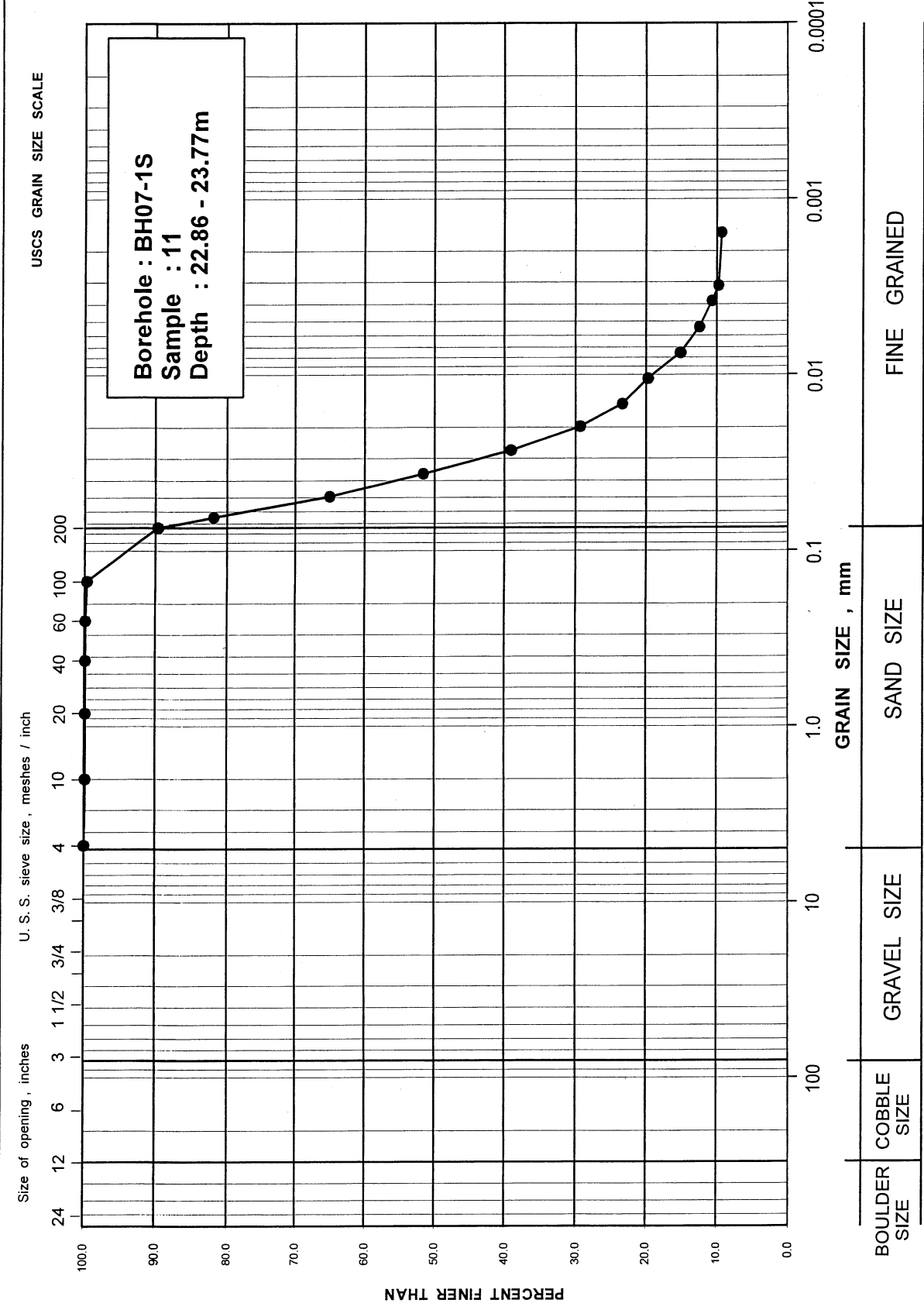
BOULDER SIZE	COBBLE SIZE	GRAVEL SIZE	SAND SIZE	FINE GRAINED
--------------	-------------	-------------	-----------	--------------

Project No. 07-1411-0005
 Drawn
 Reviewed
 Date 09/17/07



GRAIN SIZE DISTRIBUTION

Figure II-6



U.S. S. sieve size, meshes / inch

24 12 6 3 1 1/2 3/4 3/8 4 10 20 40 60 100 200

U.S. S. sieve size, meshes / inch

4 10 20 40 60 100 200

GRAIN SIZE, mm

10 1.0 0.1 0.01 0.0001

BOULDER SIZE

COBBLE SIZE

GRAVEL SIZE

SAND SIZE

FINE GRAINED

Project No. 07-1411-0005
 Drawn TM
 Reviewed
 Date 09/17/07



GRAIN SIZE DISTRIBUTION

Figure II-7

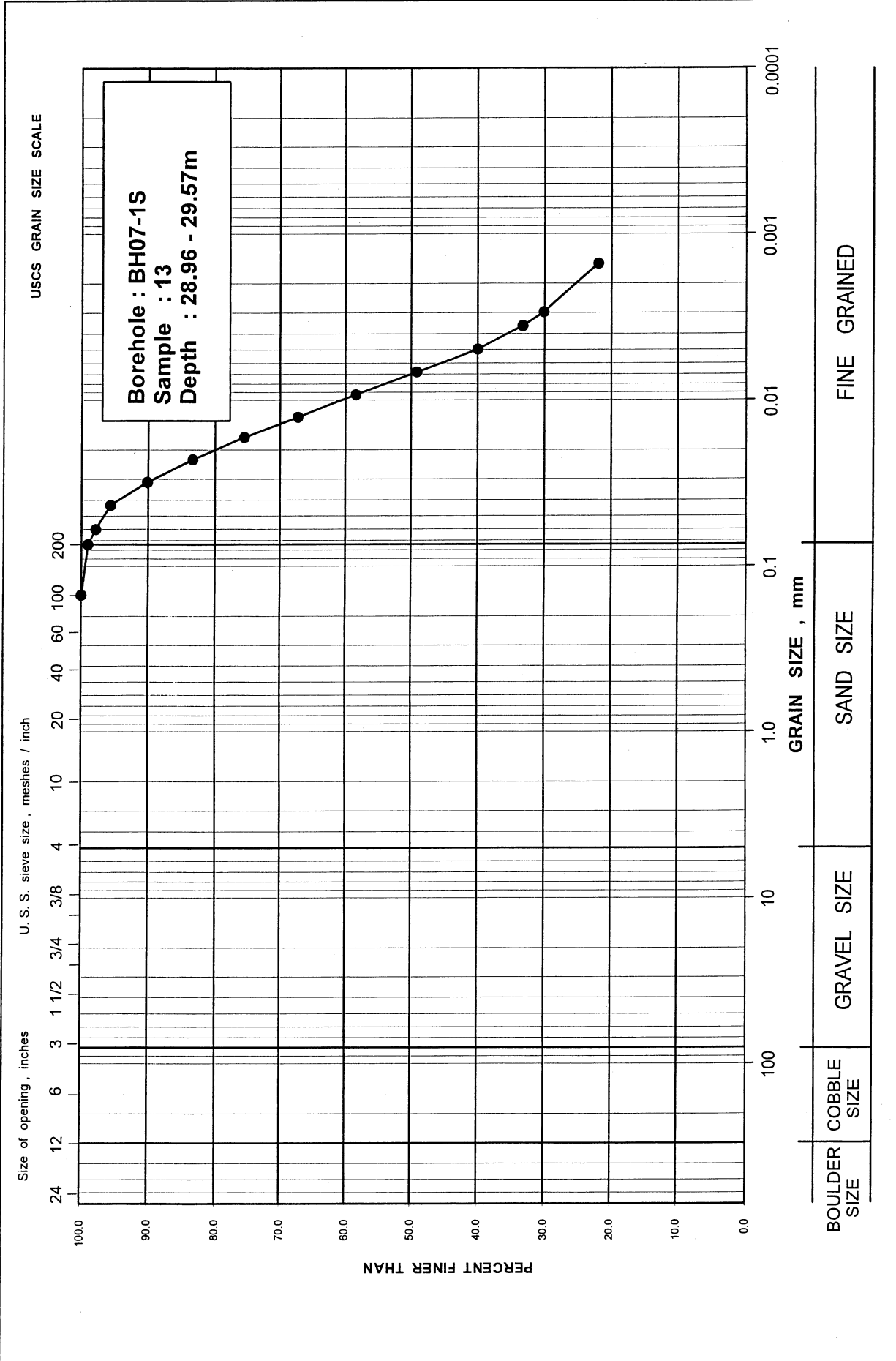
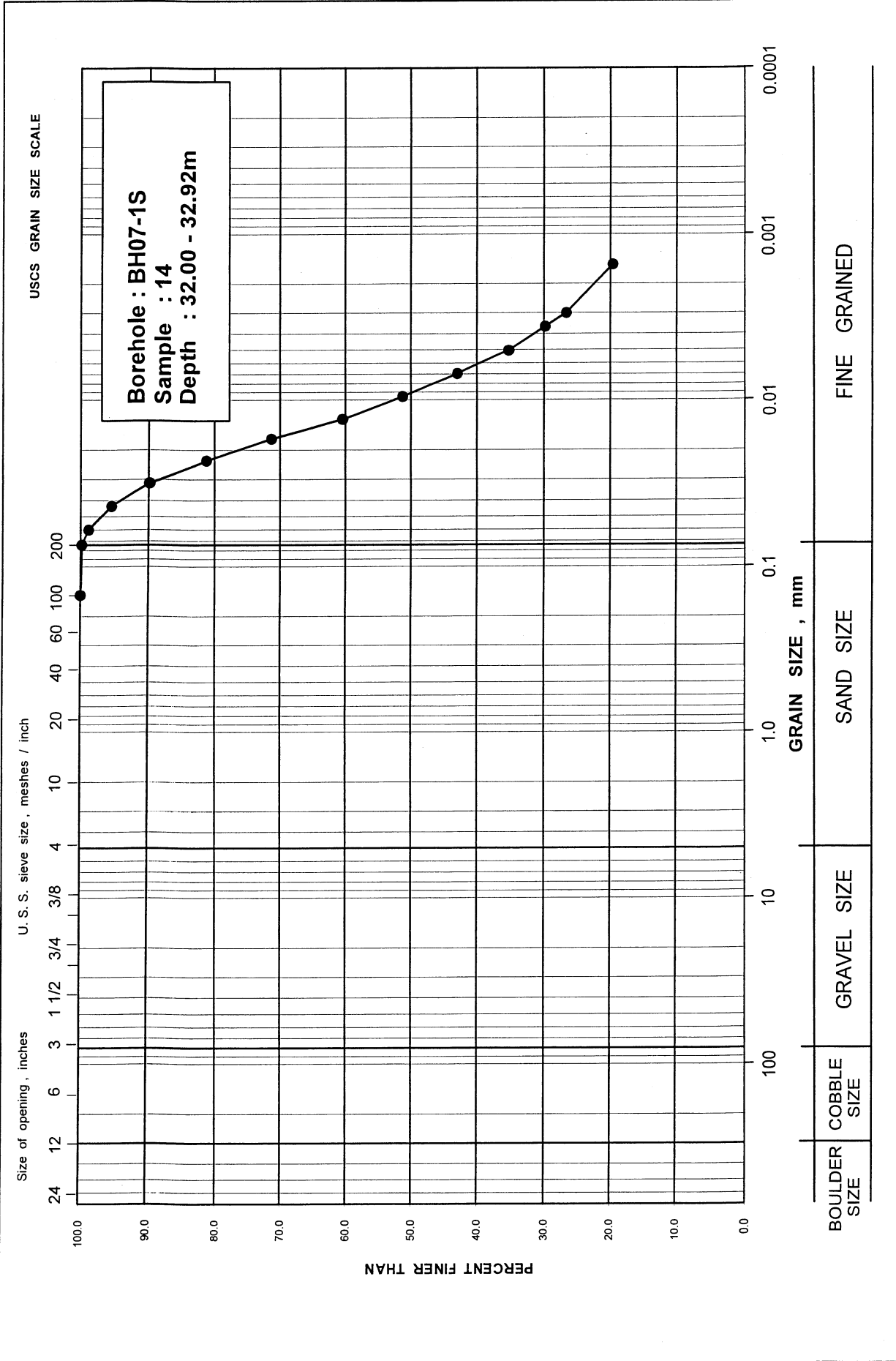



Figure II-8

GRAIN SIZE DISTRIBUTION



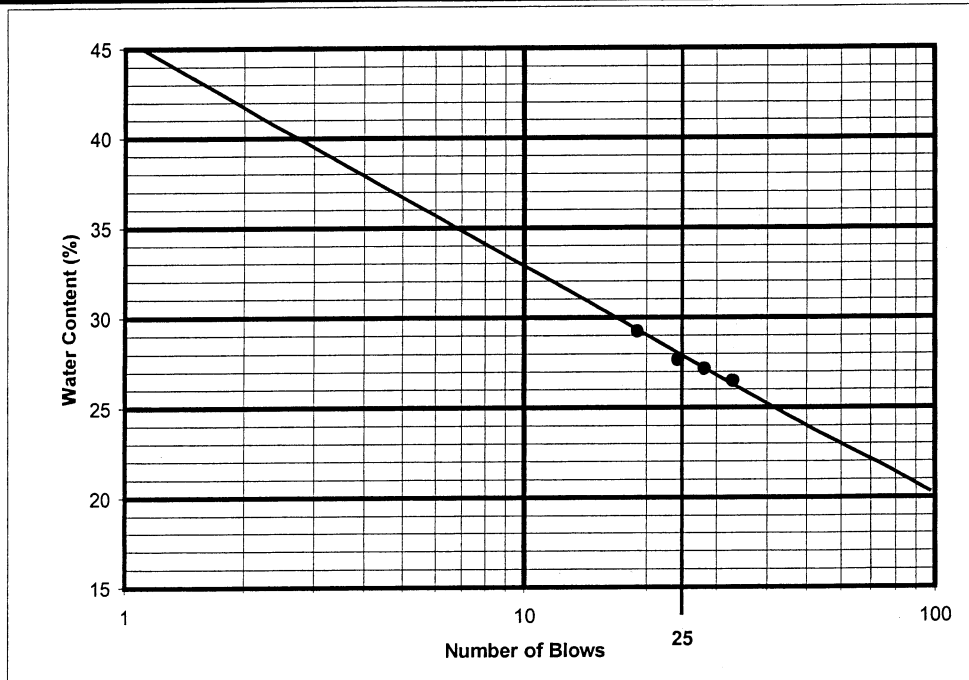
Project No. 07-14.1.1.0005
 Drawn TM
 Reviewed
 Date 09/17/07



Project No. 07-14.1.0005 Drawn Reviewed Date 09/17/07	 Golder Associates	<h2 style="margin: 0;">GRAIN SIZE DISTRIBUTION</h2>	Figure II-9
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**Liquid Limit, Plastic Limit and Plasticity Index of Soils
ASTM D 4318-93**

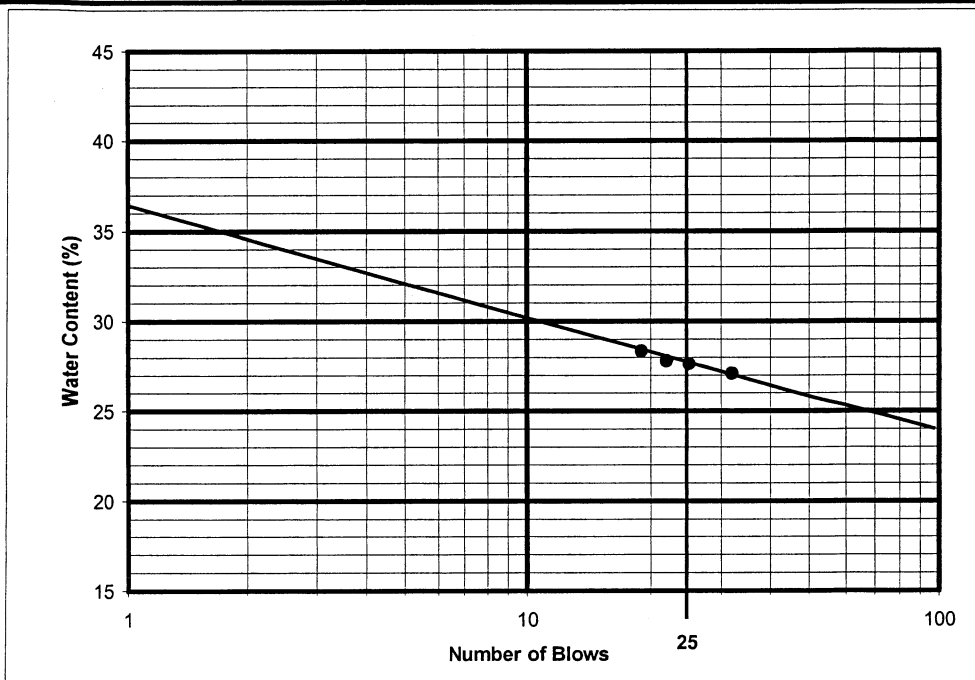
TYPE OF TEST	LL	LL	LL	LL	W% Nat.
CONTAINER NUMBER					
NUMBER OF BLOWS	33	28	24	19	
MASS WET SOIL + TARE	43.50	43.62	48.41	47.64	267.90
MASS DRY SOIL + TARE	40.14	40.45	44.29	43.19	211.50
MASS OF WATER	3.36	3.17	4.12	4.45	56.40
MASS OF CONTAINER	27.46	28.78	29.40	27.96	17.10
MASS OF DRY SOIL	12.68	11.67	14.89	15.23	194.4
WATER CONTENT W (%)	26.5	27.2	27.7	29.2	29.0
TYPE OF TEST	PL	PL	BOREHOLE NO.		BH07-1S
CONTAINER NUMBER			SAMPLE		13
MASS WET SOIL + TARE	39.82	41.58	DEPTH		28.96-29.57m
MASS DRY SOIL + TARE	38.27	39.49	LIQUID LIMIT (%)		27.7
MASS OF WATER	1.55	2.09	PLASTIC LIMIT (%)		20.5
MASS OF CONTAINER	30.75	29.25	PLASTICITY INDEX (%)		7.2
MASS OF DRY SOIL	7.52	10.24	W% Natural (%)		29.0
WATER CONTENT W (%)	20.6	20.4	LIQUIDITY INDEX		1.18



SAMPLE DESCRIPTION : CL

**Liquid Limit, Plastic Limit and Plasticity Index of Soils
ASTM D 4318-93**

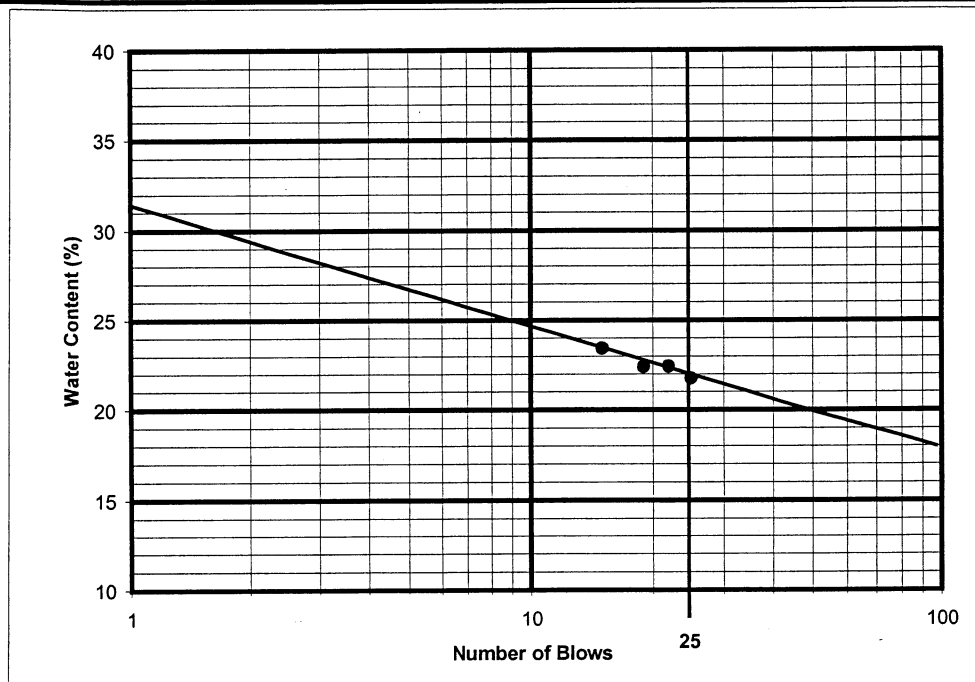
TYPE OF TEST	LL	LL	LL	LL		W% Nat.
CONTAINER NUMBER						
NUMBER OF BLOWS	32	25	22	19		
MASS WET SOIL + TARE	45.91	46.01	46.11	44.95		335.60
MASS DRY SOIL + TARE	42.48	42.11	42.16	41.21		271.40
MASS OF WATER	3.43	3.90	3.95	3.74		64.20
MASS OF CONTAINER	29.82	27.99	27.94	28.00		17.40
MASS OF DRY SOIL	12.66	14.12	14.22	13.21		254.0
WATER CONTENT W (%)	27.1	27.6	27.8	28.3		25.3
TYPE OF TEST	PL	PL	BOREHOLE NO.		BH07-1S	
CONTAINER NUMBER			SAMPLE		14	
MASS WET SOIL + TARE	37.07	38.80	DEPTH		32.00-32.92m	
MASS DRY SOIL + TARE	35.34	36.64	LIQUID LIMIT (%)		27.6	
MASS OF WATER	1.73	2.16	PLASTIC LIMIT (%)		23.5	
MASS OF CONTAINER	27.99	27.47	PLASTICITY INDEX (%)		4.1	
MASS OF DRY SOIL	7.35	9.17	W% Natural (%)		25.3	
WATER CONTENT W (%)	23.5	23.6	LIQUIDITY INDEX		0.43	



SAMPLE DESCRIPTION : ML

**Liquid Limit, Plastic Limit and Plasticity Index of Soils
ASTM D 4318-93**

TYPE OF TEST	LL	LL	LL	LL		W% Nat.
CONTAINER NUMBER						
NUMBER OF BLOWS	25	22	19	15		
MASS WET SOIL + TARE	15.27	16.87	19.08	20.45		412.80
MASS DRY SOIL + TARE	12.82	14.08	15.88	16.86		329.10
MASS OF WATER	2.45	2.79	3.20	3.59		83.70
MASS OF CONTAINER	1.56	1.63	1.59	1.54		14.50
MASS OF DRY SOIL	11.26	12.45	14.29	15.32		314.6
WATER CONTENT W (%)	21.8	22.4	22.4	23.4		26.6
TYPE OF TEST	PL	PL	BOREHOLE NO.		BH07-2S	
CONTAINER NUMBER			SAMPLE		5	
MASS WET SOIL + TARE	12.13	15.18	DEPTH		7.62-8.23m	
MASS DRY SOIL + TARE	10.35	12.89	LIQUID LIMIT (%)		21.8	
MASS OF WATER	1.78	2.29	PLASTIC LIMIT (%)		20.2	
MASS OF CONTAINER	1.60	1.52	PLASTICITY INDEX (%)		1.6	
MASS OF DRY SOIL	8.75	11.37	W% Natural (%)		26.6	
WATER CONTENT W (%)	20.3	20.1	LIQUIDITY INDEX		4.08	



SAMPLE DESCRIPTION : ML