GEOTECHNICAL INVESTIGATION
DUKE RIVER BRIDGE REPLACEMENT
KM 1768, ALASKA HIGHWAY NO. 1
YUKON, CANADA

Golder Associates

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REPORT ON

GEOTECHNICAL INVESTIGATION
DUKE RIVER BRIDGE REPLACEMENT
KM 1768, ALASKA HIGHWAY NO. 1
YUKON, CANADA

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

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 Duke River at km 1768 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 Duke River Bridge. A two-span integral abutment bridge located on the same highway alignment has been selected as the new replacement bridge structure. The new bridge is about 100 m in length with two 50 m spans. The centre pier and 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 Duke River Bridge was built in 1955 by the Canadian Military. The existing bridge is a two-lane two span steel through truss structure. Each span measures about 61 m in length. A photograph of the existing bridge taken on June 2006 is shown in Figure 2.

Based on information provided by Yukon and EarthTech, the abutments and the centre pier are supported on spread footings. The base of the footings for the abutment and the centre pier are founded at some 7 m below the existing road grade.

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. S-01 dated June 6, 2007, the proposed new bridge will be a two span bridge with integral abutments. The abutments will be supported on 610 mm diameter driven steel pipe piles and the centre pier will be supported on 914 mm diameter driven steel pipe piles. The new bridge will be about 100 m long, 11 m wide, and will be located at the existing bridge alignment. The abutments will include two 6 m long approach slabs at either end. A temporary bridge south 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 858 m. The underside of the abutment walls will be at approximately elevation 853.6 m resulting in an approximately 4.4 m high abutment wall. 610 mm diameter steel pipe are being considered for foundation support of the abutments. Following the standard design practice established for integral abutment bridges, the 610 mm diameter piles will be installed through a 3 m long 914 mm diameter sleeve and then driven into the competent dense to very dense sand and gravel soil unit. The annulus surrounding the first 3 m of the piles within the 914 mm sleeve will be left as a void to provide the lateral flexibility necessary for the integral bridge abutments.

Reinforced concrete wing walls will be installed at the back of both south and north abutments. The wing walls will be approximately 6 m in length, and will be installed parallel to the bridge alignment on either side of the abutments to contain the fills to be placed behind the abutments.

3.0 GEOTECHNICAL FIELD INVESTIGATION

3.1 1954 Field Investigation

Two boreholes were drilled at the site by others in October and November 1954 as part of geotechnical investigation for the design and construction of the existing bridge over Duke River. A borehole at the centre pier was drilled to 24.4 m (80 ft.) below ground surface and another borehole at the south abutment was drilled to 17.1 m (56 ft.) below ground surface. The boreholes indicate that the soils encountered at the site as alluvial deposits of gravel, sand, and silt. Occasional boulders were also encountered.

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 6 and May 11, 2007, during which three (3) boreholes (BH07-1D to BH07-3D) were advanced to depths of up to 42 m below existing ground surface. Boreholes BH07-1D and BH07-2D were drilled from the guide bank on the south and north side, respectively, and borehole BH07-3D was put down within the floodplain near the centre pier of the existing bridge. 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. Frozen and/or moist soils were encountered in the upper portion and the dense to very dense soils were encountered at depth. Therefore, borehole instability was not a concern and drilling was carried out without mud over the full depth of up to 42 m below ground surface.

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 put down from the guide bank at the two abutments were surveyed by others for locations and elevations. Borehole BH07-3D was not surveyed for location and elevation. The approximate locations of the boreholes 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 purposes of soil classification. A total of 5 gradation analyses and 1 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 inferred subsurface soil and groundwater conditions at the bridge site are summarized below. The following sections provide a summary of the inferred subsurface soil conditions 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.

4.1 Fill

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

In BH07-1D drilled on the south side, the fill materials extended to a depth of about 2.7 m below ground surface. In BH07-2D drilled on the north side, the fill materials extended to a depth of about 4.6 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 in excess of 100 blows/0.3 m. These larger N values reflect the presence of coarser material sizes rather than the in-situ relative density. In borehole BH07-1D, full penetration of the split spoon sampler into the fills could not be achieved in some instances.

4.2 Sand and Gravel

Underlying the fill materials at boreholes BH07-1D and BH07-2D and the ice and snow at borehole BH07-3D, a deposit of dense to very dense, grey grading to brown, moist to wet, sand and gravel with a trace of silt was encountered.

The SPT N values measured within this deposit were larger than 100 blows/0.3 m. These larger N values reflect the presence of coarser material sizes rather than the in-situ relative density. At most locations, penetration of the split spoon sampler was limited to about 100 to 150 mm. Further penetration of the split spoon sampler was not attempted upon reaching 100 blows in order to prevent damage to the sampler.

The gradation results of representative samples taken from this deposit are shown on Figure II-1 to II-4 in Appendix II. The results indicate that the material is generally well-graded with 31 to 51% gravel sizes, 37 to 77% sand sizes, and 5 to 39% finer sizes.

At borehole BH07-3D, a more siltier layer was encountered at a depth of about 8.7 m below the river bed. Thickness of this siltier layer was found to be about 1 m at this borehole location. 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. The gradation results of representative sample taken from this deposit are shown on Figure II-5 in Appendix II. The samples tested contained 30% gravel sizes, 27% sand sizes, and 43% finer sizes.

Atterberg limit test was carried out on a sample of the fine-grained layer obtained from borehole BH07-3D. The test results show that this layer is low plastic. The test results show that this layer may undergo cyclic mobility under design ground motions.

4.3 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 6 and May 11, 2007, it is difficult to estimate the groundwater table due to frozen ground conditions encountered during drilling.

Based on the information provided in EarthTech Drawing No. S-01, we understand that the 100-year flow in Duke River is at elevation 854.3 m or some 3.5 m below the current road grade elevation.

5.0 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.

5.1 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 1.

TABLE 1: 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 over a 6 m depth measured from the road grade.

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.

5.2 Piled Foundations

Consideration may be given to use of large diameter (0.6 m or larger) steel pipe piles for support of the foundations. Use of steel pipe piles has also the following benefits:

- The stresses induced in the piles during driving could be high due to presence of coarse grained soils including boulders at the site. Steel pipe piles will be able to withstand relatively high level of stresses during installation.

- The steel pipe piles will be able to handle any required length of extension relatively easily.

Steel pipe piles can be designed to provide the required axial and lateral load carrying capacity. Considering the presence of cobbles and boulders and the in-situ density of the granular strata at the site, it is recommended that the steel pipe piles be installed open-ended to achieve the required pile embedment to resist lateral loads. Larger diameter piled driven open-ended will provide the opportunity to remove boulder and other obstructions should the piles encounter refusal prior to reaching the required pile embedment and/or pile capacity. However, it is difficult to estimate the axial capacity of open-ended large diameter piles since the axial pile capacity depends on the effectiveness of the soil plug at the tip level. For this reason, it is strongly recommended that the piling contract be written such that PDA testing of a production pile at the very early stage of pile installation is conducted so that pile length can be optimized without any delay on the construction schedule. Provision for deletion and addition of pile quantities should be included. Provision should also be included in the contract for possible clean out and removal of obstructions should the piles met refusal prior to reaching the required pile embedment or pile capacity.

The table below provides our best estimate of the geotechnical ultimate axial compression capacity of open ended driven piles of various diameters. The settlements under static loading conditions are expected to be less than 25 mm.

TABLE 2: 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
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 is 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.

5.3 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 steel pipe piles to develop the required capacity, and estimated the potential stresses induced in the pile during installation. The analyses were carried out for 610 mm and 914 mm diameter steel pipe piles driven open-ended to a depth of 20 m below the underside of the pile cap. The wall thickness of the 610 mm and 914 mm piles were assumed to be 12.7 mm and 19 mm, respectively based on the information provided by Earth Tech (Drawing No 01-1768-08 dated September 5, 2007).

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 D62-02) with a maximum rated energy of 206 kJ (153 kip-ft) per blow for the drivability analyses. The results of our analyses with a D 62-02 hammer are summarized in Table 3.

Table 3: Results of WEAP Analyses with D 62-02 Hammer

Pile Diameter (mm)	Ultimate Capacity (kN/pile)	Estimated Final Set (Blows/25mm)	Compressive Stress (MPa)	ENTHU (kJ)
610	3000	3	~250	~90
914	6000	6	~170	~80

Note: ENTHU - Energy transferred at pile top

The analyses indicate that a final set of 3 blows/25 mm and 6 blows/25 mm is required for the 610 mm and 914 mm diameter piles, respectively, to achieve the ultimate capacity indicated in Table 3. The Delmag D62-02 diesel hammer is expected to operate at a stroke height of 2.6 m and a strike rate of about 40 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.

5.4 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. The "p-y" curves for the 610 mm diameter piles and the 914 mm diameter piles were developed using API guidelines. These are provided in Tables 4 and 5, 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.

5.5 Seismic Design Considerations

5.5.1 Site Seismicity

The site-specific peak firm-ground horizontal accelerations established by the Geological Survey of Canada are summarized below:

TABLE 6: Site-Specific Ground Motion Parameters – NBCC 2005

Return Period	100 yr.	475 yr.	1000 yr.
Probability of Exceedance in 50 years	40%	10%	5%
Peak firm ground horizontal Acceleration	0.106g	0.194g	0.25g

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 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.

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 1.2 that corresponds to Soil Profile Type II.

5.5.2 Liquefaction Potential of Foundation Soils

The liquefaction potential of the overburden soils has been assessed based on Seed's simplified method of analysis. A peak ground surface accelerations were also estimated based on Seed's simplified attenuation relationship and the estimated peak ground surface accelerations of 0.25 g and 0.3 g were used in the liquefaction assessment for 475-year and 1,000 year ground motions. The water table was taken at a depth of about 4 m below the current road grade, which approximately corresponds to the 100-year flow level in the river.

Based on the historical seismicity along the Denali/Totschunda Fault System extending some several hundred meters distance from the bridge site, an M7.5 earthquake that is considered to be representative of 10 to 15 cycles of effective loading was considered in the analysis of soil liquefaction.

The results of our analyses indicate that site soils comprising of dense to very dense sand and gravel has a low risk liquefaction potential under the 475-year and 1,000-year ground motions. Therefore, ground improvement measures are not considered necessary at this site for seismic performance of the bridge.

5.6 Lateral Earth Pressure Coefficients

The lateral earth pressure coefficients on the walls of the integral abutments under static loading conditions 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 4. The curve labeled as 'medium dense' in Figure 4 is appropriate for backfill compacted to 95 percent of Standard Proctor density.

It should be noted that the K_p values shown on Figure 4 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.

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 then 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 0.57 and 0.73 be used in design for the 475-year and 1,000-year ground motions, respectively, under restrained wall condition.

Alternatively, abutment walls may be designed with a seismic lateral earth pressure coefficient (K_{ae}) of 0.34 and 0.37, corresponding to a yielding wall condition, for the 475-year and 1,000-year ground motions, respectively, 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
- The peak horizontal ground surface acceleration for the 475-year and 1,000-year ground motions are 0.25g and 0.30 g, respectively

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

$$Pae = ½ (K_{ae}) \gamma H^2$$

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/m^3 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/m^3 should be used when computing the lateral earth pressures.

5.7 Overall 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. S-01 and S-02. 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 854.3 m.

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 1.7 or greater which is considered to be adequate. The results of the slope stability analyses are presented in Figure 5.

Stability analyses were also conducted for the 1,000 year ground motions. A peak firm ground acceleration of 0.25 g and a peak ground surface acceleration of 0.3 g were assumed for the site. The pseudo-static factor of safety against an overall failure of the abutment slope under the 1,000 year return period seismic event is approximately 1.1 (see Figure 6) indicating that only minor displacements of the abutment slope should 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 1000 year seismic events, provided that the site preparation measures identified in section 5.1 are adhered to during construction.

5.8 Lateral Stiffness of Abutment-Backfill Unit

The lateral subgrade modulus of the abutment-backfill unit has been calculated based on recommendations by Caltrans (Version 1.4, June 2006). The abutment embankment fill stiffness (**Kabut**) is non-linear and is dependent on material properties of the backfill. Based on the recommendations provided in Caltrans document (Section 7.8 of Seismic Design Criteria), the following equation can be used to compute the stiffness of a typical embankment backfill.

$$\mathbf{Kabut}\ (\text{kN/mm}) = \mathbf{11.5}\ (\text{kN/mm/m})*\mathbf{w}*(\mathbf{h}/1.7)$$

where w is the width of the abutment wall (m) and h is the wall height (m)

5.9 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 2.5 m if there is no snow cover.

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.

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.

6.0 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 pile installation and 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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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the 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.

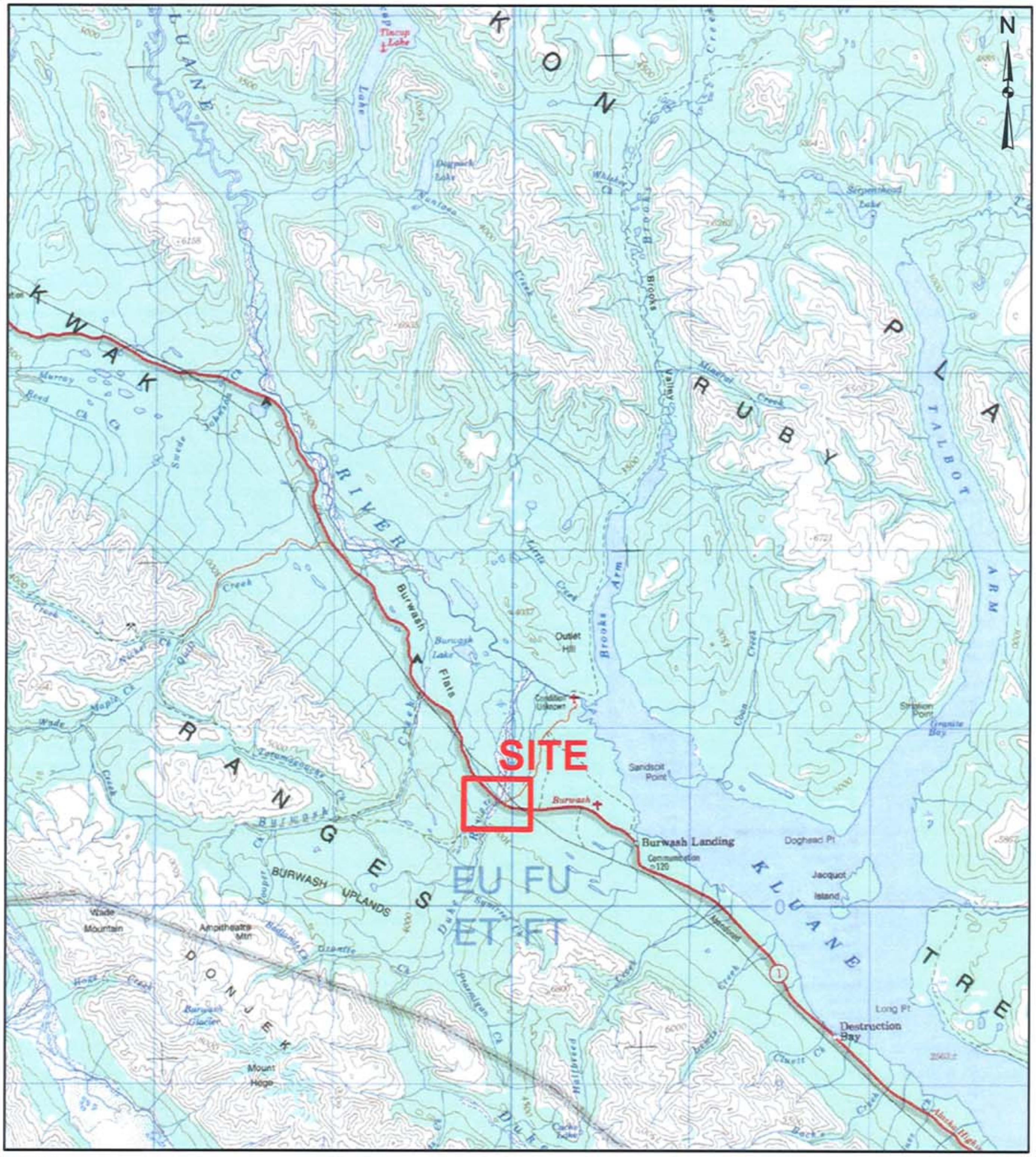
Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed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

TABLE 4: p-y Curves for 610 mm Pipe Pile

Depth below Pile Cap (m)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
y (m)	**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
0.0008	24.0	47.6	72.6	97.4	122.1	146.9	171.6	196.2	220.9	245.6	270.2	294.8	319.4	343.9	368.5	393.1	417.6	442.2	466.8	491.4
0.0017	44.9	87.1	138.1	188.7	239.1	289.2	339.1	388.9	438.5	488.1	537.7	586.8	635.7	684.6	733.5	782.4	831.3	880.2	929.0	977.9
0.0025	61.0	115.0	192.0	269.5	346.5	422.9	498.9	574.5	649.7	724.8	799.7	873.1	945.9	1018.6	1091.4	1164.2	1236.9	1309.7	1382.4	1455.2
0.0034	72.2	132.6	233.1	337.1	441.4	545.1	648.0	750.2	851.8	953.0	1053.8	1151.3	1247.3	1343.2	1439.2	1535.1	1631.1	1727.0	1823.0	1918.9
0.0042	79.5	142.9	262.6	391.4	522.5	653.7	784.2	913.8	1042.5	1170.5	1297.8	1419.2	1537.4	1655.7	1774.0	1892.2	2010.5	2128.8	2247.0	2365.3
0.0051	84.0	148.7	282.9	433.3	589.9	748.2	906.4	1063.7	1220.1	1375.5	1530.1	1674.8	1814.3	1953.9	2093.5	2233.0	2372.6	2512.1	2651.7	2791.3
0.0059	86.8	151.8	296.5	464.8	644.4	828.6	1014.0	1199.2	1383.6	1566.9	1749.1	1916.6	2076.3	2236.0	2395.7	2555.5	2715.2	2874.9	3034.6	3194.3
0.0068	88.5	153.5	305.4	488.0	687.7	895.9	1107.5	1320.1	1532.3	1743.7	1954.0	2143.6	2322.2	2500.9	2679.5	2858.1	3036.8	3215.4	3394.0	3572.7
0.0076	89.4	154.4	311.1	504.8	721.6	951.4	1187.6	1426.6	1666.3	1905.7	2144.1	2355.1	2551.3	2747.6	2943.9	3140.1	3336.4	3532.6	3728.9	3925.1
0.0085	90.0	154.9	314.8	516.8	747.8	996.6	1255.5	1519.6	1786.0	2052.8	2319.2	2550.7	2763.3	2975.9	3188.4	3401.0	3613.6	3826.1	4038.7	4251.2
0.0093	90.3	155.2	317.1	525.4	767.8	1033.1	1312.5	1600.1	1891.9	2185.5	2479.5	2730.6	2958.1	3185.7	3413.2	3640.8	3868.3	4095.9	4323.4	4551.0
0.0102	90.5	155.3	318.6	531.4	783.0	1062.3	1360.0	1669.1	1985.0	2304.3	2625.2	2894.9	3136.2	3377.4	3618.7	3859.9	4101.1	4342.4	4583.6	4824.9
0.0229	90.8	155.5	321.2	545.5	828.1	1167.8	1562.4	2008.4	2501.5	3036.5	3608.3	4036.4	4372.7	4709.1	5045.5	5381.8	5718.2	6054.6	6390.9	6727.3
0.6329	90.8	155.5	321.2	545.6	828.7	1170.4	1570.8	2029.9	2547.6	3124.1	3759.1	4226.3	4578.5	4930.7	5282.8	5635.0	5987.2	6339.4	6691.6	7043.8
1.2429	90.8	155.5	321.2	545.6	828.7	1170.4	1570.8	2029.9	2547.6	3124.1	3759.1	4226.3	4578.5	4930.7	5282.8	5635.0	5987.2	6339.4	6691.6	7043.8
1.8529	90.8	155.5	321.2	545.6	828.7	1170.4	1570.8	2029.9	2547.6	3124.1	3759.1	4226.3	4578.5	4930.7	5282.8	5635.0	5987.2	6339.4	6691.6	7043.8

TABLE 5: p-y Curves for 914 mm Pipe Pile

Depth below Pile Cap (m)	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
y (m)	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	36.0	71.5	107.1	144.4	181.7	218.9	256.0	293.1	330.1	367.1	404.1	441.1	478.0	515.0	551.9	588.9	625.8	662.6	699.4	736.2
0.0025	67.3	131.6	195.8	272.2	348.4	424.2	499.7	575.0	650.0	724.8	799.5	874.0	948.4	1022.8	1097.1	1171.3	1245.5	1318.8	1392.0	1465.3
0.0038	91.5	175.1	258.5	373.5	489.7	605.8	721.3	836.3	950.6	1064.6	1178.1	1291.2	1404.1	1516.7	1629.0	1741.3	1853.3	1962.4	2071.4	2180.4
0.0051	108.4	203.2	298.1	447.1	601.6	757.9	914.3	1070.2	1225.3	1379.6	1533.3	1686.2	1838.6	1990.5	2142.0	2293.1	2443.8	2587.7	2731.4	2875.2
0.0063	119.4	220.2	321.3	497.3	685.7	879.9	1076.3	1273.2	1469.6	1665.3	1860.1	2054.1	2247.1	2439.4	2631.0	2821.9	3012.3	3189.7	3366.9	3544.1
0.0076	126.4	230.0	334.3	529.9	746.3	974.2	1208.1	1444.7	1682.1	1919.2	2155.7	2391.3	2625.8	2859.4	3092.0	3323.7	3554.6	3764.1	3973.2	4182.3
0.0089	130.6	235.5	341.5	550.6	788.7	1045.2	1312.7	1586.3	1863.0	2140.8	2418.7	2696.0	2972.5	3247.9	3522.3	3795.5	4067.8	4307.6	4546.9	4786.3
0.0102	133.1	238.5	345.3	563.5	817.7	1097.4	1393.9	1701.1	2014.4	2331.1	2649.3	2967.9	3286.1	3603.5	3919.9	4235.3	4549.4	4817.8	5085.5	5353.1
0.0114	134.6	240.2	347.4	571.4	837.4	1135.3	1456.1	1792.6	2139.3	2492.4	2849.1	3207.6	3566.8	3925.8	4284.2	4641.7	4998.1	5293.1	5587.2	5881.3
0.0127	135.4	241.1	348.4	576.2	850.5	1162.4	1503.1	1864.8	2241.2	2627.5	3020.2	3416.9	3815.7	4215.4	4615.3	5014.6	5413.2	5732.9	6051.4	6369.9
0.014	136.0	241.6	349.0	579.1	859.2	1181.7	1538.3	1921.1	2323.3	2739.6	3165.5	3598.0	4034.6	4473.6	4913.8	5354.4	5794.6	6137.1	6478.1	6819.0
0.0152	136.3	241.9	349.3	580.9	865.0	1195.4	1564.5	1964.7	2389.2	2831.8	3287.9	3753.5	4225.6	4702.1	5181.3	5661.9	6143.1	6506.5	6867.9	7229.4
0.0343	136.7	242.2	349.7	583.6	876.2	1227.3	1636.7	2103.7	2627.3	3205.8	3837.1	4518.3	5246.6	6018.6	6830.8	7679.7	8561.7	9071.9	9575.9	10079.9
0.9483	136.7	242.2	349.7	583.6	876.2	1227.4	1637.3	2105.9	2633.1	3219.1	3863.7	4566.9	5328.8	6149.4	7028.7	7966.6	8963.2	9498.7	10026.4	10554.1
1.8623	136.7	242.2	349.7	583.6	876.2	1227.4	1637.3	2105.9	2633.1	3219.1	3863.7	4566.9	5328.8	6149.4	7028.7	7966.6	8963.2	9498.7	10026.4	10554.1
2.7763	136.7	242.2	349.7	583.6	876.2	1227.4	1637.3	2105.9	2633.1	3219.1	3863.7	4566.9	5328.8	6149.4	7028.7	7966.6	8963.2	9498.7	10026.4	10554.1



LEGEND

🟥 Project Site

6 3 0 6

SCALE 1:300,000 KILOMETRES

PROJECT

GOVERNMENT OF YUKON
DUKE RIVER BRIDGE REPLACEMENT KM 1768
ALASKA HWY, YUKON

TITLE

KEY PLAN

REFERENCE

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

Golder Associates
Burnaby, B.C.

PROJECT No. 07-1411-0005			SCALE AS SHOWN	REV. 0
DESIGN	YY	07 Jun. 2007	**FIGURE 1**	
GIS	AL	25 Jun. 2007		
CHECK	MY	17 Oct. 2007		
REVIEW	MY	17 Oct. 2007		

N:\Bur-Graphics\Projects\Key-plans\2007\aprs\07-1411-0005.mxd

Photograph taken in June 2006.

PROJECT

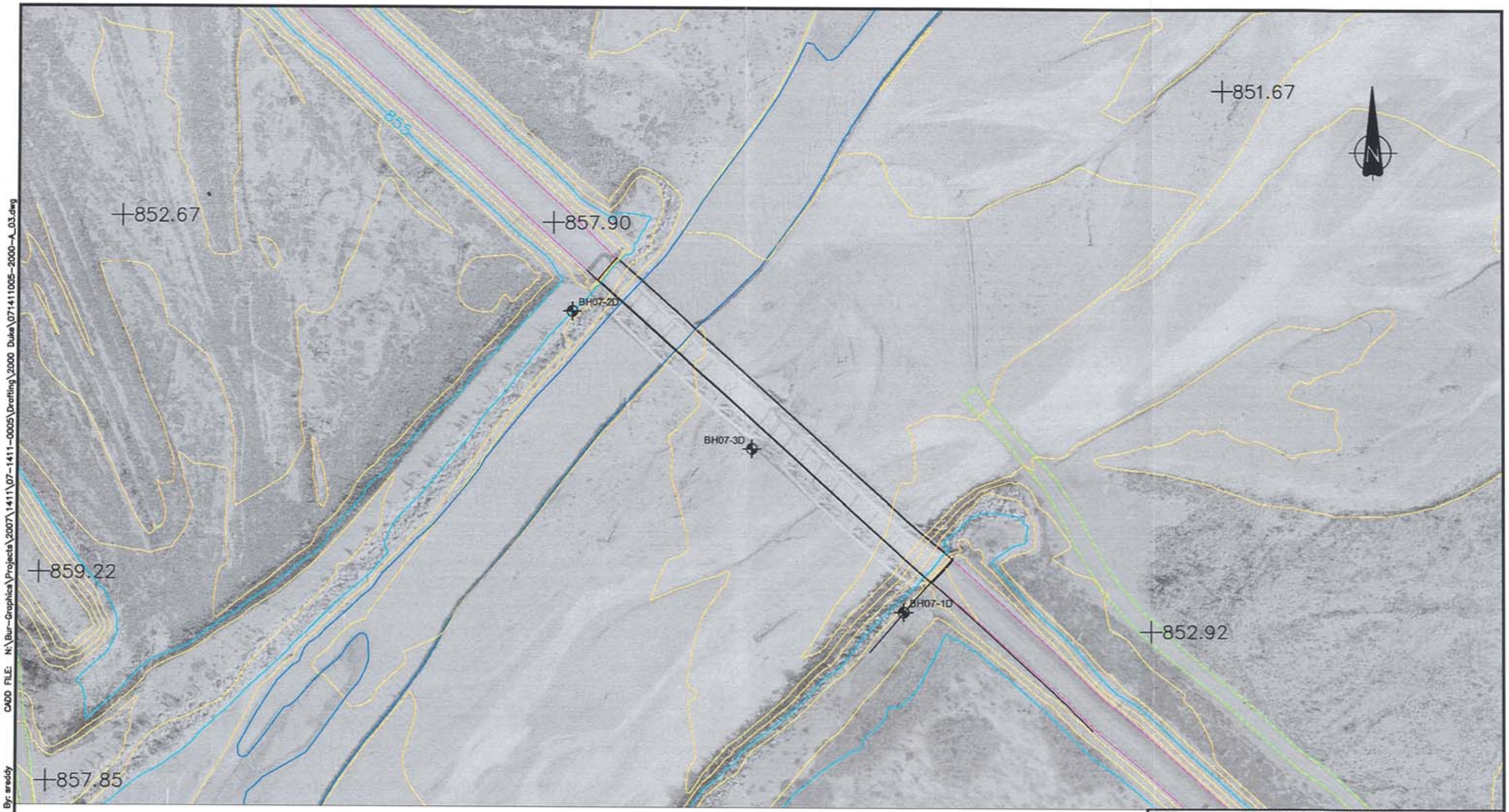
GOVERNMENT OF YUKON
DUKE RIVER BRIDGE REPLACEMENT
KM 1768, ALASKA HWY, YUKON

TITLE

PHOTOGRAPH OF EXISTING DUKE RIVER BRIDGE

Golder Associates

PROJECT No.	07-1411-0005		PHASE / TASK No.	2000
DESIGN	JNT	25JUN07	SCALE NTS	REV.
CADD	JNT	25JUN07	**FIGURE 2**	
CHECK				
REVIEW				



REVISION DATE: 07/10/10 01:17PM By: sreddy CADD FILE: N:\Bur-Graphics\Projects\2007\1411\07-1411-0005\Drafting\2000 Duke\071411005-2000-A_03.dwg

REFERENCES

1) Challenger Geomatics Ltd. drawing 36144-for TYG, dated June 11, 2007.

LEGEND

BH07-1D ⊕ Approximate Borehole Location

0 10 20 30 40 50
Scale In Meters

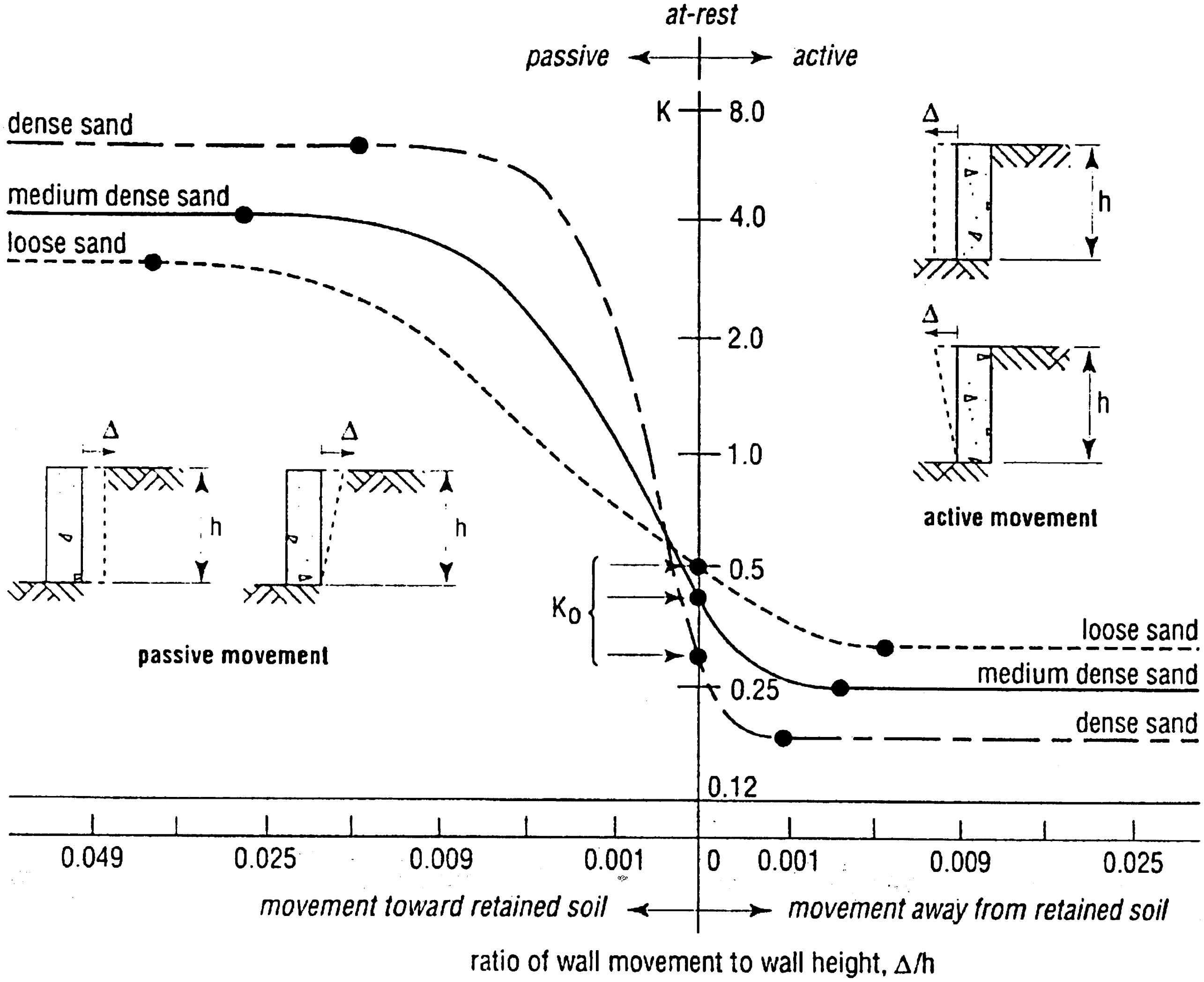
PROJECT
GOVERNMENT OF YUKON
DUKE RIVER BRIDGE REPLACEMENT
KM 1768, ALASKA HWY, YUKON

TITLE
BOREHOLE LOCATION PLAN

Golder Associates

PROJECT No. 07-1411-0005			FILE No.	
DESIGN	MY	21JUN07	SCALE AS SHOWN	REV. -
CADD	GG	21JUN07	FIGURE 3	
CHECK	MY	25JUN07		
REVIEW	MY	25JUN07		

REVISION DATE: 07/09/18 09:52AM By: MHintoy CADD FILE: N:\Bur-Graphics\Projects\2007\1411\07-1411-0005\Drafting\2000 Duke\07141105-2000-A_04.dwg



PROJECT
GOVERNMENT OF YUKON
DUKE RIVER BRIDGE REPLACEMENT
KM 1768, ALASKA HWY, YUKON

TITLE

VARIATION OF EARTH PRESSURE WITH LATERAL WALL MOVEMENT

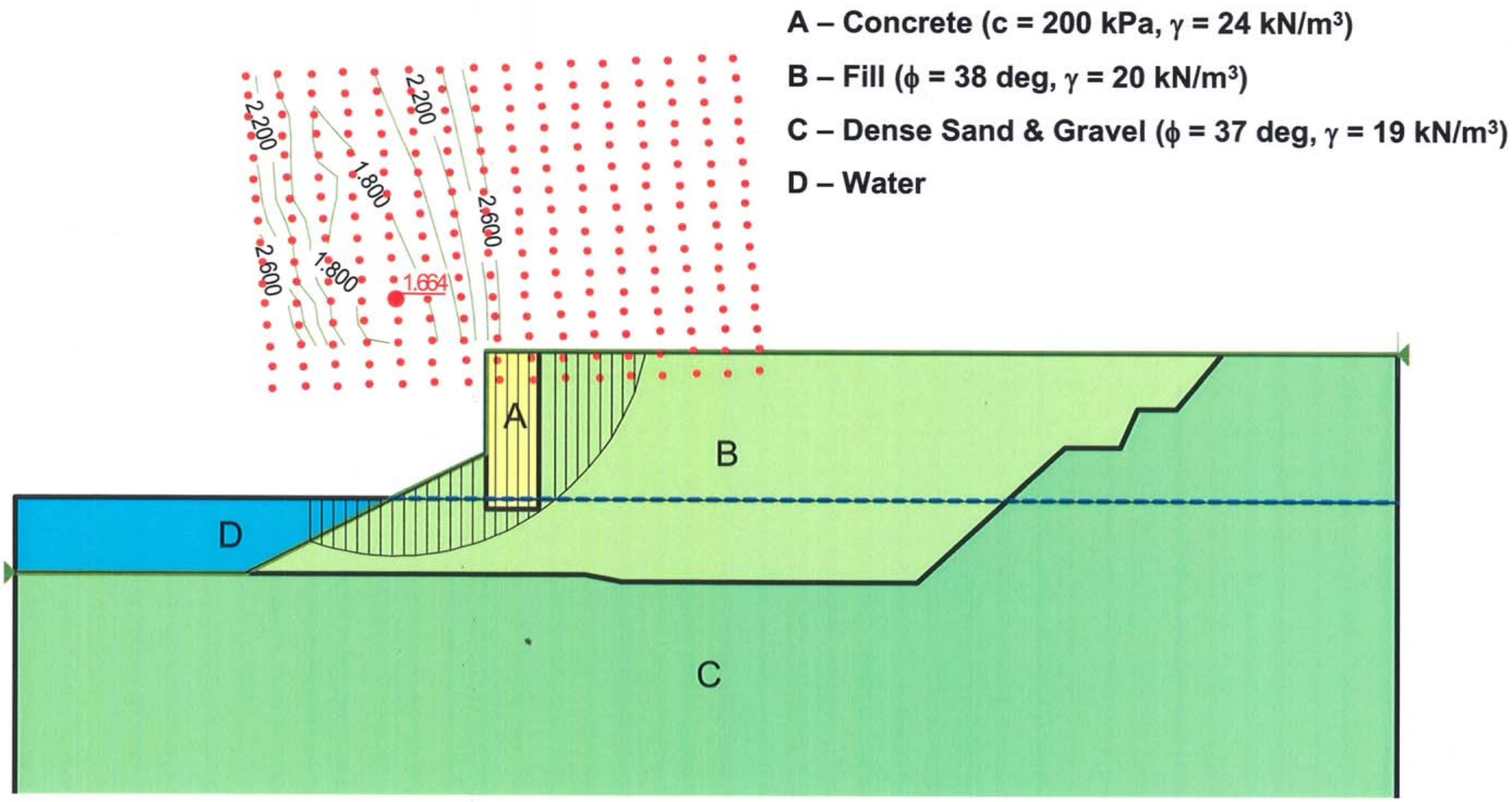
Golder Associates

PROJECT No. 07-1411-0005			FILE No.
DESIGN	MY	25JUN07	SCALE NTS REV. -
CADD	BAD	25JUN07	
CHECK	MY	25JUN07	FIGURE 4
REVIEW	MY	25JUN07	

REFERENCE

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

DATE: 22/06/07 BY: Y PATHAK FILE:O:\Active_2007\1411\07-1411-0005 Slims & Duke River Bridges\Duke River\Slope Stability Analysis\Static load_slope stability



A – Concrete (c = 200 kPa, γ = 24 kN/m³)

B – Fill (ϕ = 38 deg, γ = 20 kN/m³)

C – Dense Sand & Gravel (ϕ = 37 deg, γ = 19 kN/m³)

D – Water

PROJECT

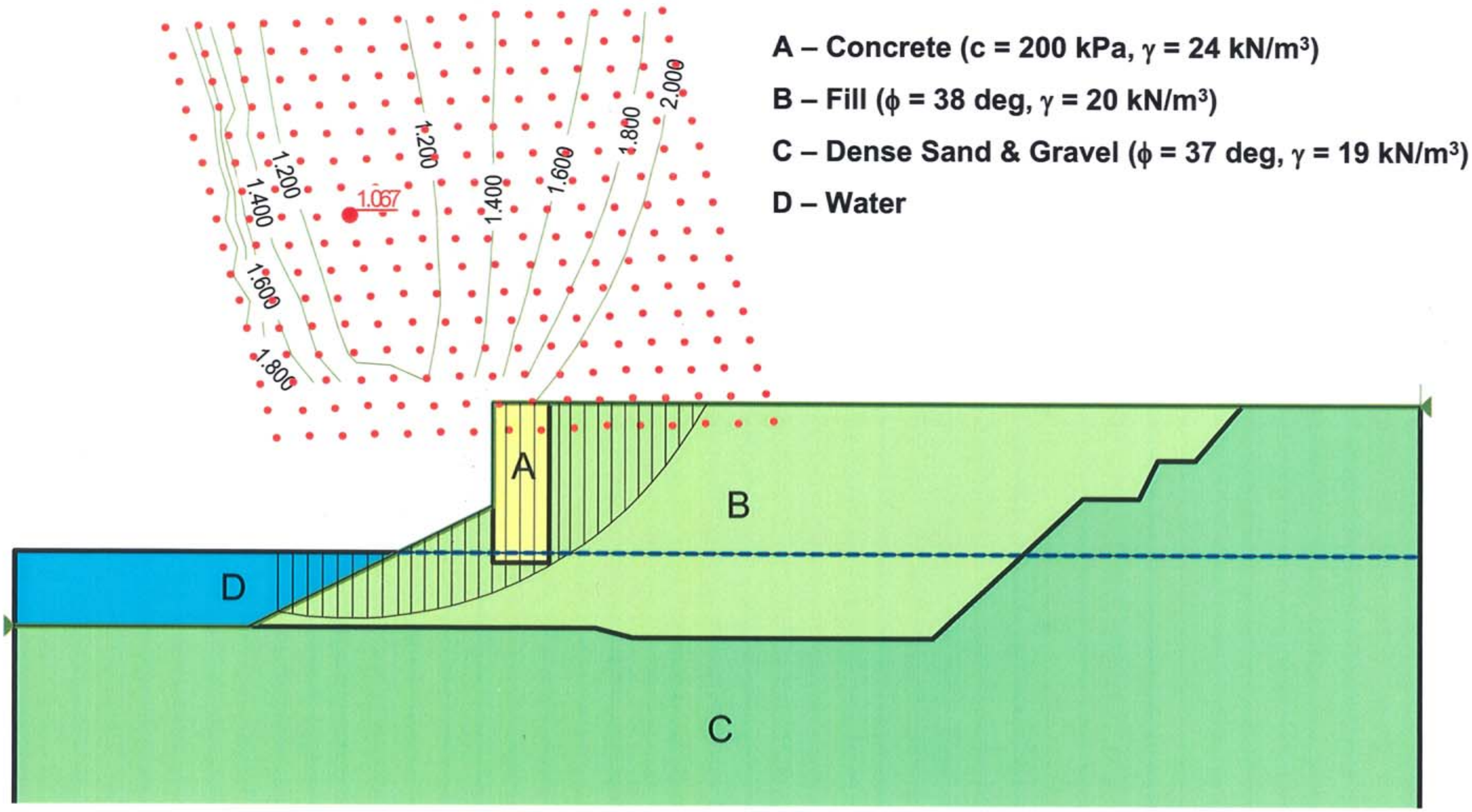
GOVERNMENT OF YUKON
DUKE RIVER BRIDGE REPLACEMENT
KM 1768, ALASKA HWY, YUKON

TITLE

Global Stability Analysis
Static Condition

Golder Associates

PROJECT No.	07-1411-0005		PHASE / TASK No. 2000
DESIGN	YP	22JUN07	SCALE AS SHOWN | REV.
CADD	--		**FIGURE 5**
CHECK	MY	22JUN07	
REVIEW	MY		



A – Concrete (c = 200 kPa, γ = 24 kN/m^3)

B – Fill (ϕ = 38 deg, γ = 20 kN/m^3)

C – Dense Sand & Gravel (ϕ = 37 deg, γ = 19 kN/m^3)

D – Water

PROJECT

GOVERNMENT OF YUKON
DUKE RIVER BRIDGE REPLACEMENT
KM 1768, ALASKA HWY, YUKON

TITLE

Global Stability Analysis
Seismic Condition (1000 year Ground Motion)

Golder Associates

PROJECT No.	07-1411-0005		PHASE / TASK No. 2000
DESIGN	YP	22JUN07	SCALE AS SHOWN REV.
CADD	--		FIGURE 6
CHECK	MY	22JUN07	
REVIEW	MY		

APPENDIX I

BOREHOLE LOGS

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-1D

SHEET 1 OF 5

LOCATION: Southwest Side of Duke Bridge
N: 6806874.084 E: 600129.301

BORING DATE: May 6-9, 2007

DATUM: Geodetic

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 2, 4, 6, 8
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○; 10, 20, 30, 40

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}
WATER CONTENT PERCENT: Wp ├──○W──┤ Wl; 5, 10, 15, 20

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	ADDITIONAL LAB. TESTING
0	Geotech Drilling Track Mounted Mud Rotary	Ground Surface	855.95 0.00				
1		Loose to compact, moist, brown SAND and GRAVEL, some brown silt, contains cobbles and boulders. (FILL) - frozen soils ~0.3m - 2.74m depth.		2	50 DO	17/ 127mm	
2.74		Dense, moist, brown SAND, some gravel and silt, contains cobbles and boulders.	853.21 2.74	4	50 DO	>100	
3.96		Dense, moist, brown-grey SAND and GRAVEL, trace to some silt, contains cobbles and boulders. - cobbles at 4.27m and 5.03m depth. - boulder from 7.92m - 8.53m depth (0.6m dia.).	851.99 3.96				
				6	50 DO	43	M
				7	50 DO	66	
				8	50 DO	>100	
10							

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-1D

SHEET 2 OF 5

LOCATION: Southwest Side of Duke Bridge
N: 6806874.084 E: 600129.301

BORING DATE: May 6-9, 2007

DATUM: Geodetic

BORING METHOD: Geotech Drilling, Track Mounted Mud Rotary

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 2 4 6 8

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ ; 10 20 30 40

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl ; 5 10 15 20

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Dense, moist, brown-grey SAND and GRAVEL, trace to some silt, contains cobbles and boulders. - cobbles at 4.27m and 5.03m depth. - boulder from 7.92m - 8.53m depth (0.6m dia.). *(continued)*		9	50 DO	84		
12		843.76 12.19					
13	Dense to very dense, moist to wet, brown-grey SAND and GRAVEL, some silt, contains cobbles and boulders.		10	50 DO	75	M	
16		839.80 16.15					
16	Very dense, moist, grey-brown, sandy SILT, some gravel, trace clay, contains cobbles.		11	50 DO	>100		
17		838.27 17.68					
19	Very dense, moist to wet, grey-brown SAND and GRAVEL, some silt, contains cobbles and boulders. - boulder from 20.7m - 21.2m depth.		12	50 DO	107		
20							

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-1D

SHEET 3 OF 5

LOCATION: Southwest Side of Duke Bridge
N: 6806874.084 E: 600129.301

BORING DATE: May 6-9, 2007

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (2 4 6 8); SHEAR STRENGTH Cu, kPa (10 20 30 40) nat V. + Q - ● rem V. ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (5 10 15 20)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	Geotech Drilling Track Mounted Mud Rotary	Very dense, moist to wet, grey-brown SAND and GRAVEL, some silt, contains cobbles and boulders. - boulder from 20.7m - 21.2m depth. *(continued)*									
22					13	50 DO	144				
23				832.79 23.16							
25		Very dense, moist to wet, grey-brown SAND and GRAVEL, some silt, contains cobbles and boulders. - cobbles at 25.5m and 37.6m depths. - boulder from 25.6m - 25.9m depth.			14	50 DO	>100				
28					15	50 DO	114				
30											

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

LOCATION: Southwest Side of Duke Bridge
N: 6806874.084 E: 600129.301

RECORD OF BOREHOLE: BH07-1D

BORING DATE: May 6-9, 2007

SHEET 4 OF 5

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (2, 4, 6, 8); SHEAR STRENGTH Cu, kPa (10, 20, 30, 40) nat V. + Q - ● rem V. ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
30	Geotech Drilling Track Mounted Mud Rotary	Very dense, moist to wet, grey-brown SAND and GRAVEL, some silt, contains cobbles and boulders. - cobbles at 25.5m and 37.6m depths. - boulder from 25.6m - 25.9m depth. *(continued)*									
31					16	50 DO	166				
34		Very dense, moist to wet, brown-grey GRAVEL, some sand and silt, contains cobbles and boulders. - cobbles at 38.1m depth.		821.51 34.44	17	50 DO	>120				
37					18	50 DO	>120				
40											

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-1D

SHEET 5 OF 5

LOCATION: Southwest Side of Duke Bridge
N: 6806874.084 E: 600129.301

BORING DATE: May 6-9, 2007

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 2 4 6 8 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 10 20 30 40	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 5 10 15 20	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
40 41 42	Geotech Drilling Track Mounted Mud Rotary	Very dense, moist to wet, brown-grey GRAVEL, some sand and silt, contains cobbles and boulders. - cobbles at 38.1m depth. *(continued)*		813.89 42.06	19	50 DO	120/152mm				
43 44 45 46 47 48 49 50		End of BOREHOLE.									

DEPTH SCALE
1 : 50

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-2D

SHEET 1 OF 4

LOCATION: Northwest Side of Duke Bridge
N: 6806964.384 E: 600033.801

BORING DATE: May 10-11, 2007

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 2 4 6 8 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O 10 20 30 40	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ⊖W ⊣ Wl 5 10 15 20	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		855.33							
0	Geotech Drilling Track Mounted Mud Rotary	Compact, moist, brown SAND and GRAVEL, some silt, contains cobbles and boulders. (Berm FILL)		0.00							
1		Dense to very dense, damp to moist, brown-grey SAND and GRAVEL, some silt, contains cobbles and boulders. (Berm FILL) -		854.11 1.22	2	50 DO	109				
2					3	50 DO	50/ 100mm				
4					4	50 DO	110				
4.57		Dense to very dense, moist, brown and grey SAND and GRAVEL, some silt, contains cobbles and boulders. - cobbles at 7.47m depth.		850.76 4.57							
6					5	50 DO	81				
7					6	50 DO	72				
8		Very dense, moist to wet, brown-grey GRAVEL, some sand, trace to some silt, contains cobbles and boulders. - boulders from 11.0m - 11.6m and 16.8m - 17.1m depths. - cobble at 16.56m depth.		847.10 8.23							
9					7	50 DO	63				
10											

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-2D

SHEET 2 OF 4

LOCATION: Northwest Side of Duke Bridge
N: 6806964.384 E: 600033.801

BORING DATE: May 10-11, 2007

DATUM: Geodetic

SOIL PROFILE

SAMPLES

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 2 4 6 8

SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 10 20 30 40

HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 5 10 15 20

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
10	Geotech Drilling Track Mounted Mud Rotary	Very dense, moist to wet, brown-grey GRAVEL, some sand, trace to some silt, contains cobbles and boulders. - boulders from 11.0m - 11.6m and 16.8m - 17.1m depths. - cobble at 16.56m depth. *(continued)*					
				8	50 DO	46	
11							
12							
13							
				9	50 DO	85	
14							
15							
16							
				10	50 DO	85/ 152mm	M
17							
18							
19							
				11	50 DO	148	
20							

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-2D

SHEET 3 OF 4

LOCATION: Northwest Side of Duke Bridge
N: 6806964.384 E: 600033.801

BORING DATE: May 10-11, 2007

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (2, 4, 6, 8); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O (10, 20, 30, 40)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20	Geotech Drilling Track Mounted Mud Rotary	Very dense, moist to wet, brown-grey GRAVEL, some sand, trace to some silt, contains cobbles and boulders. - boulders from 11.0m - 11.6m and 16.8m - 17.1m depths. - cobble at 16.56m depth. *(continued)*								
22				12	50 DO	60/ 76mm				
23			832.16 23.16							
25		Very dense, moist to wet, brown-grey GRAVEL, some sand and silt, contains cobbles and boulders. - gravel layer at 28.65m depth. - cobbles at 28.72m depth.		13	50 DO	120				
28				14	50 DO	50/ 76mm				
30		*CONTINUED NEXT PAGE*								

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-2D

SHEET 4 OF 4

LOCATION: Northwest Side of Duke Bridge
N: 6806964.384 E: 600033.801

BORING DATE: May 10-11, 2007

DATUM: Geodetic

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (2, 4, 6, 8); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (10, 20, 30, 40)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
30	Geotech Drilling Track Mounted Mud Rotary	Very dense, moist to wet, brown-grey GRAVEL, some sand and silt, contains cobbles and boulders. - gravel layer at 28.65m depth. - cobbles at 28.72m depth. *(continued)*									
31					15	50 DO	108				
32											
33											
34					16	50 DO	125				
35											
36				818.88 36.45	17	50 DO	>100				
37		End of BOREHOLE.									
38											
39											
40											

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: LW
CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-3D

SHEET 1 OF 2

LOCATION: 6m East of Centre Pier

BORING DATE: May 12, 2007

DATUM: Local

BORING METHOD: Geotech Drilling, Track Mounted Mud Rotary

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 2 4 6 8

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ 10 20 30 40

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl 5 10 15 20

PIEZOMETER OR STANDPIPE INSTALLATION

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	0.00				
Ice and snow (beside centre pier).					
Loose to compact, moist to wet, grey GRAVEL, some sand and silt, contains cobbles and boulders. - cobbles at 1.32m depth.	0.61	1	50 DO	70/ 152mm	
Compact to dense, wet to moist, brown and grey GRAVEL, some sand, trace to some silt, contains cobbles and boulders. - cobbles at 2.82m and 5.79m depths.	2.74	2	50 DO	80/ 152mm	
		3	50 DO	128	
		4	50 DO	20/ 50mm	
		5	50 DO	87	
Very stiff, moist, brown, sandy SILT, some gravel to gravelly.	8.69	6	50 DO	22	M
	9.75				

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: LW

CHECKED: MY

PROJECT No.: 07-1411-0005

RECORD OF BOREHOLE: BH07-3D

SHEET 2 OF 2

LOCATION: 6m East of Centre Pier

BORING DATE: May 12, 2007

DATUM: Local

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (2 4 6 8); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (10 20 30 40)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (5 10 15 20)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Geotech Drilling Track Mounted Mud Rotary	Dense to very dense, moist, brown-grey SAND, GRAVEL and SILT, contains cobbles and boulders. *(continued)*			7	50 DO	136				
12					8	50 DO	106				
13		Dense, moist to wet, brown, silty SAND, some gravel, contains cobbles.		13.41	9	50 DO	40			M	
14		Very dense, moist to wet, brown-grey GRAVEL, some sand and silt, contains cobbles and boulders.		14.63	10	50 DO	106				
15		End of BOREHOLE.		15.39							

DEPTH SCALE

1 : 50

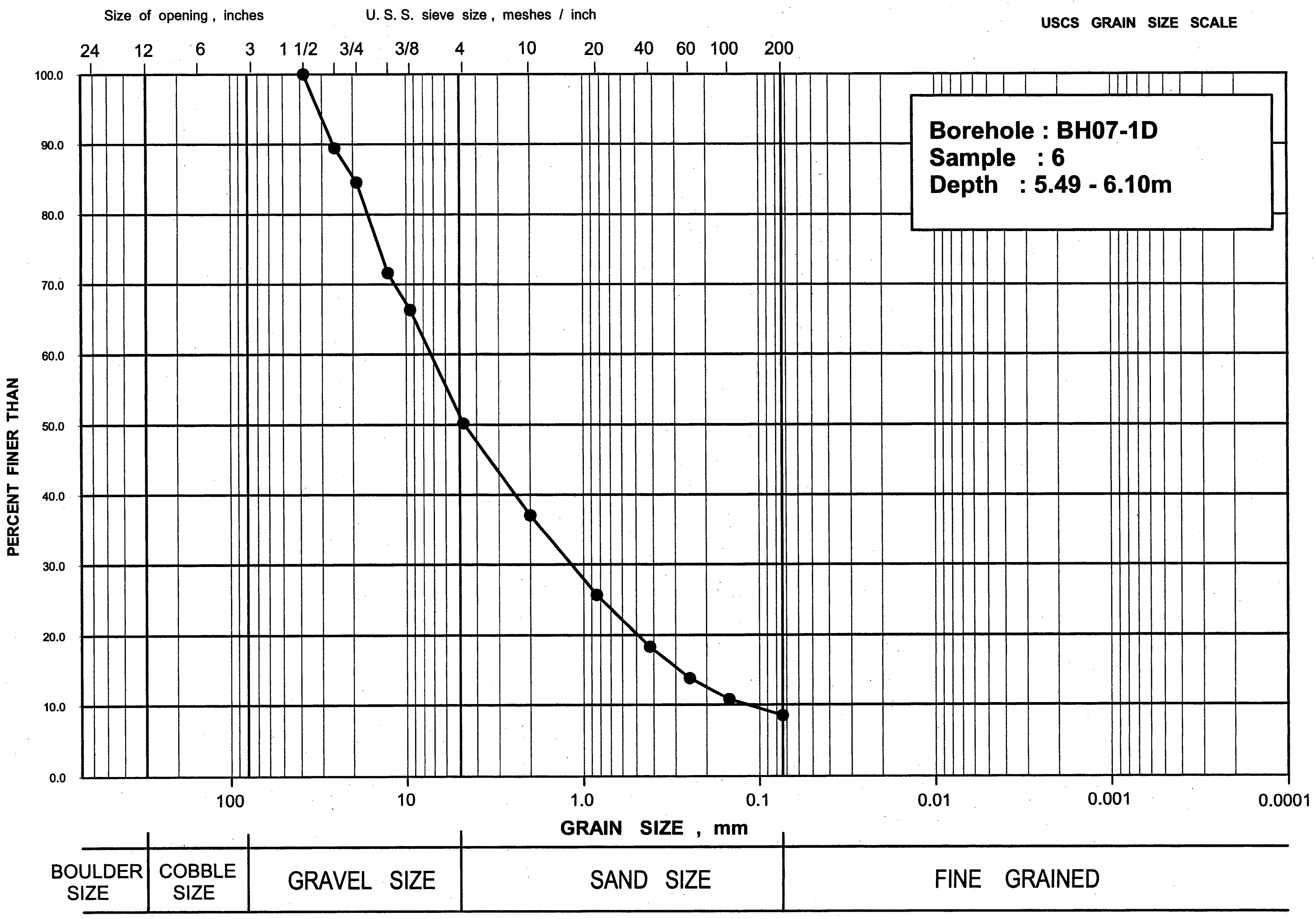
Golder Associates

LOGGED: LW

CHECKED: MY

APPENDIX II

LABORATORY TEST RESULTS

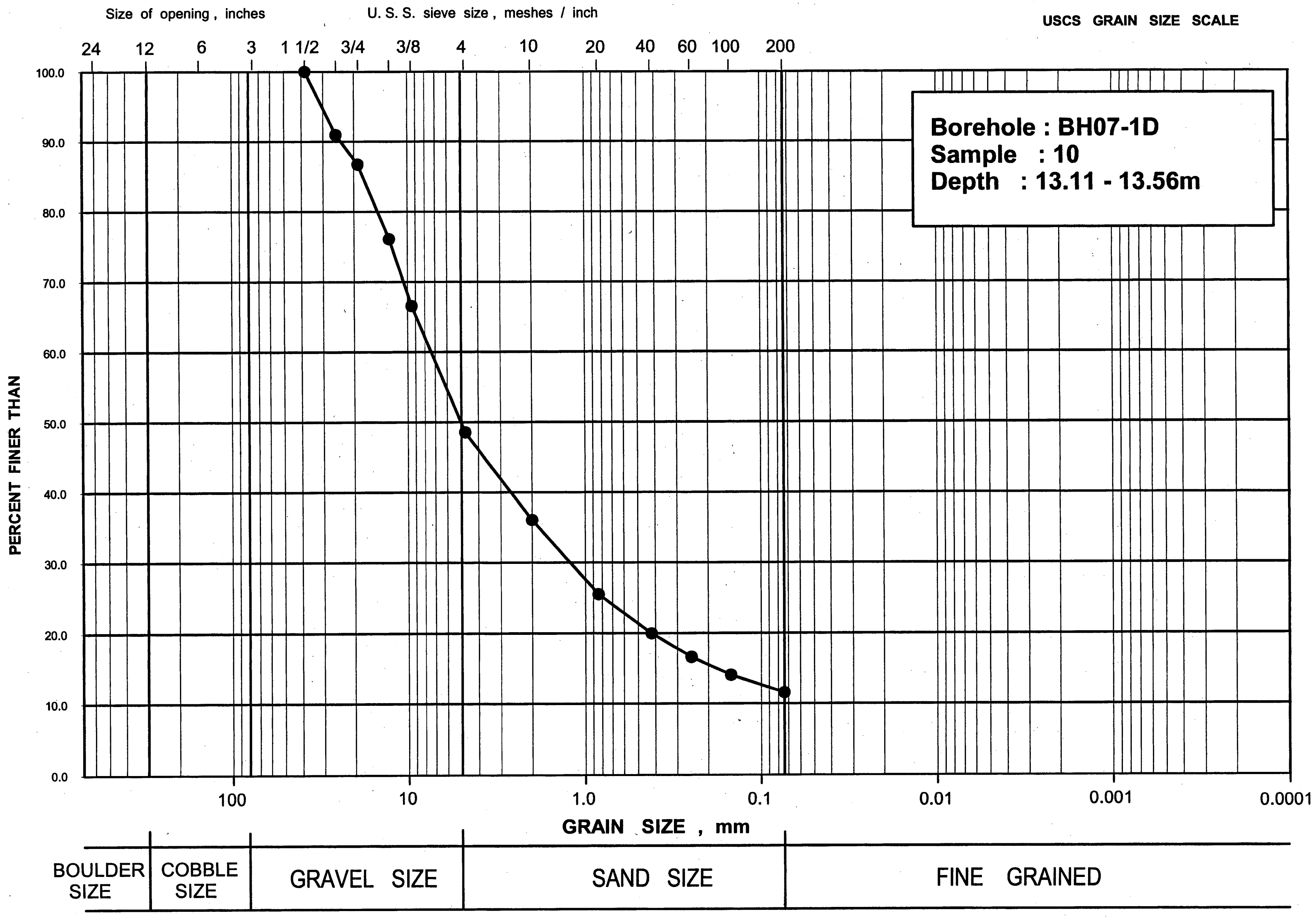


Project No. ..07-1411-0005
Drawn ....TM.......
Reviewed ....LL.......
Date ....06/05/07.......

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure II-1

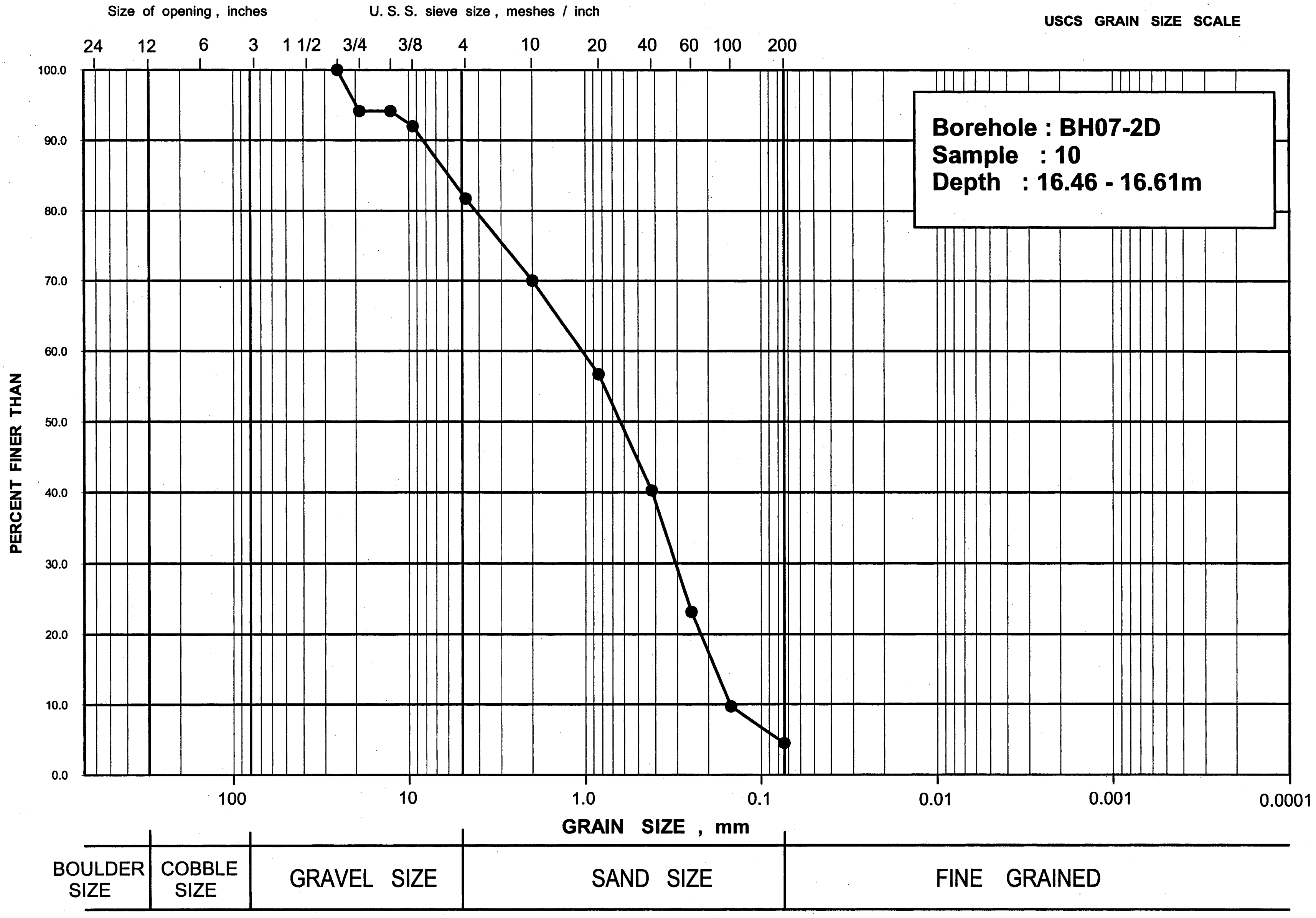


Project No. 07-1411-0005
Drawn TM
Reviewed LL
Date 06/05/07

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure II-2



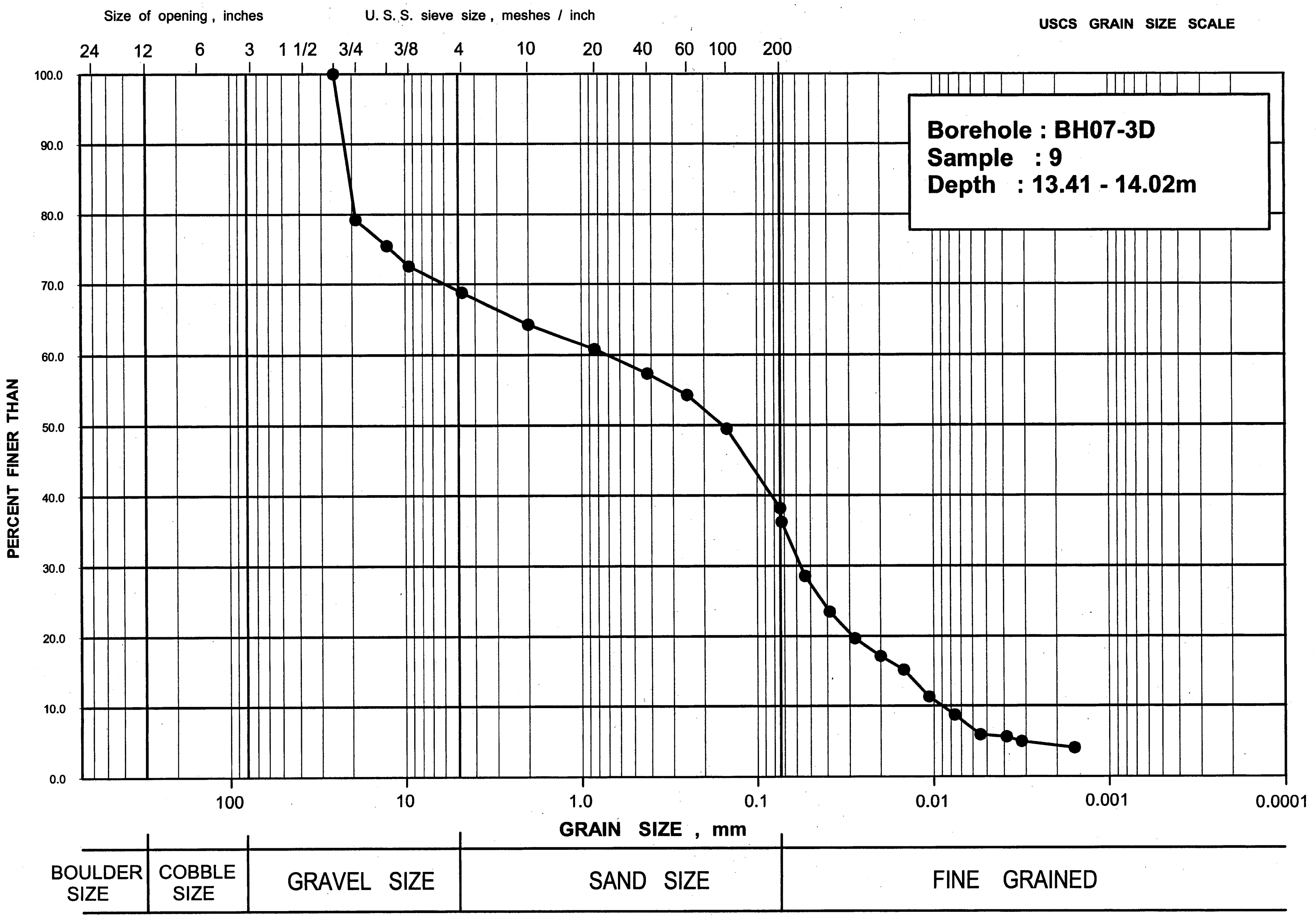
Borehole : BH07-2D
Sample : 10
Depth : 16.46 - 16.61m

Project No. 07-1411-0005
Drawn TM
Reviewed LL
Date 06/05/07

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure II-3



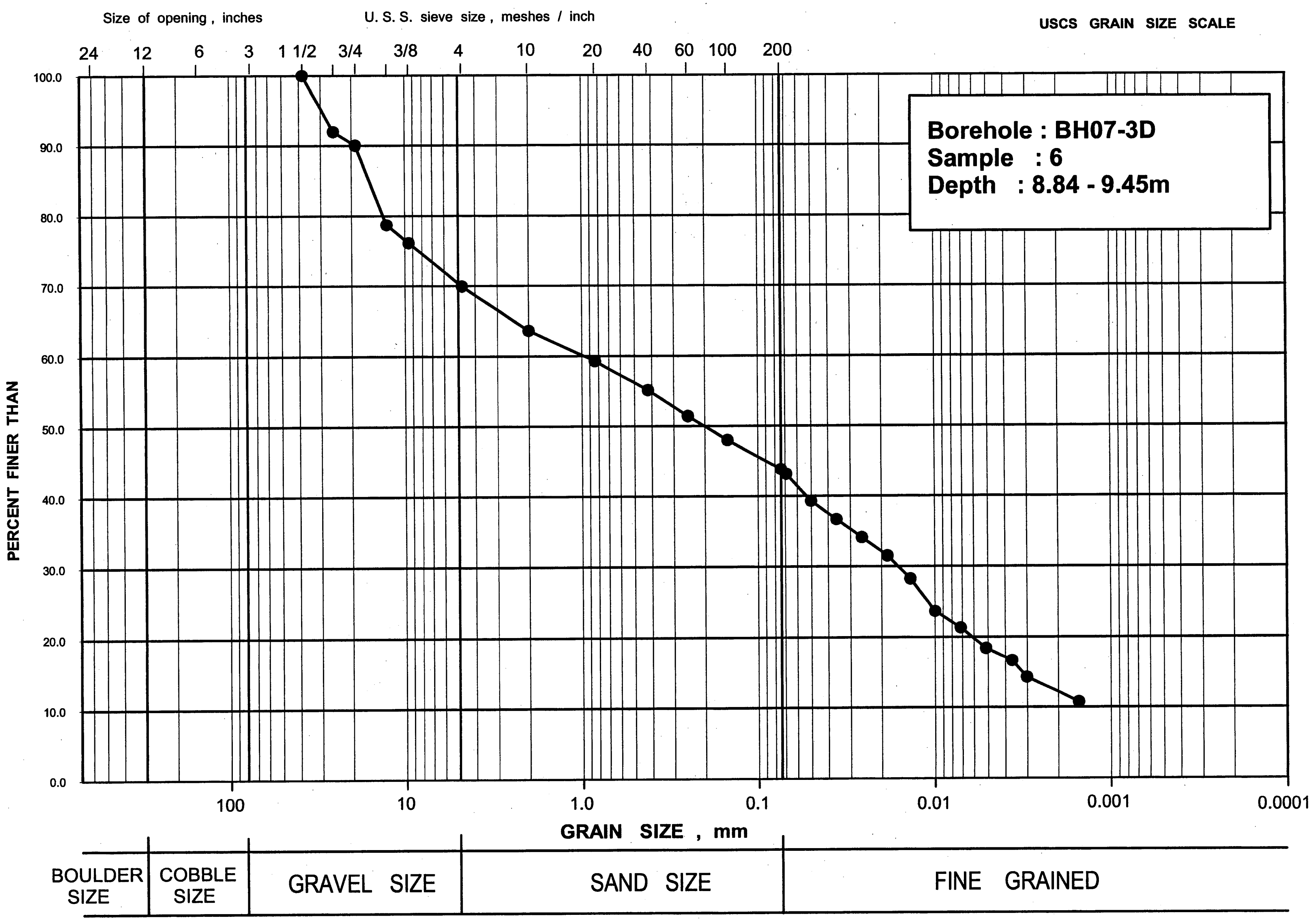
Borehole : BH07-3D
Sample : 9
Depth : 13.41 - 14.02m

Project No. 07-1411-0005
Drawn TM
Reviewed LL
Date 06/05/07

Golder Associates

GRAIN SIZE DISTRIBUTION

Figure II-4



Project No. ..07-1411-0005
Drawn ....TM..........
Reviewed ....LL..........
Date ....06/05/07.......

Golder Associates

GRAIN SIZE DISTRIBUTION

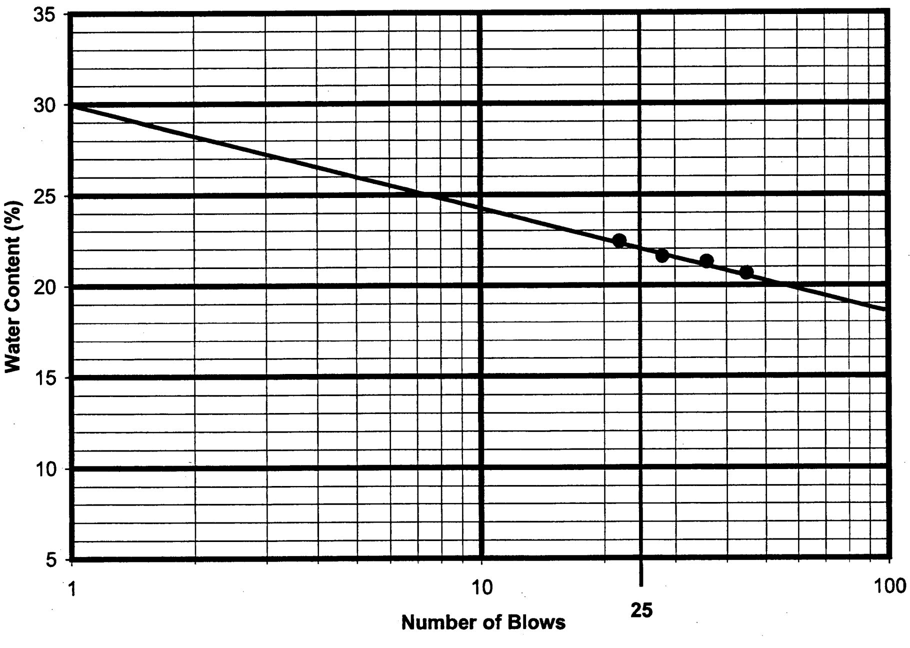
Figure II-5

Project # 07-1411-0005 **Tech : TM**

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	45	36	28	22		
MASS WET SOIL + TARE	18.00	21.38	15.49	19.01		884.70
MASS DRY SOIL + TARE	15.19	17.91	13.03	15.81		799.40
MASS OF WATER	2.81	3.47	2.46	3.20		85.30
MASS OF CONTAINER	1.57	1.60	1.63	1.54		15.20
MASS OF DRY SOIL	13.62	16.31	11.40	14.27		784.2
WATER CONTENT W (%)	**20.6**	**21.3**	**21.6**	**22.4**		**10.9**

TYPE OF TEST	PL	PL	BOREHOLE NO.	BH07-3D
CONTAINER NUMBER			SAMPLE	6
MASS WET SOIL + TARE	13.60	12.31	DEPTH	8.84-9.45m
MASS DRY SOIL + TARE	11.93	10.83	LIQUID LIMIT (%)	22.1
MASS OF WATER	1.67	1.48	PLASTIC LIMIT (%)	15.9
MASS OF CONTAINER	1.56	1.45	PLASTICITY INDEX (%)	6.2
MASS OF DRY SOIL	10.37	9.38	W% Natural (%)	10.9
WATER CONTENT W (%)	**16.1**	**15.8**	LIQUIDITY INDEX	-0.82



SAMPLE DESCRIPTION : CL-ML