April 2011

REPORT ON

Geotechnical Investigation The Chancery of the Canadian High Commission Bridgetown, St. Michael Parish, Barbados

Submitted to:
J.L. Richards & Associates Ltd.
864 Lady Ellen Place
Ottawa, Ontario
K1Z 5M2

Report Number: 10-1121-0089-1

Distribution:

2 Copies - J.L. Richards & Associates Ltd.

2 Copies - Golder Associates Ltd.

Golder Associates

Golder Associates

Project No. 10-1121-0089

April 20, 2011

Jennifer Stephenson
J.L. Richards & Associates Ltd.
864 Lady Ellen Place
Ottawa, Ontario
K1Z 5M2

GEOTECHNICAL INVESTIGATION
THE CHANCERY OF THE CANADIAN HIGH COMMISSION
BRIDGETOWN, ST. MICHAEL PARISH, BARBADOS

Dear Madam;

Please find attached our report on the geotechnical investigation for the proposed seismic evaluation and potential upgrades and developments to the Chancery of the Canadian High Commission in Bridgetown, St. Michael Parish, Barbados.

We trust that this report is sufficient for your present requirements. If you have any questions concerning this report, or if we can be of further service to you on this project, please call us.

Yours truly,

GOLDER ASSOCIATES LTD.

Bruce D. Goddard, P.Eng. Senior Geotechnical Engineer	Michael S. Snow, P.Eng. Principal

BDG/MSS/tm

n:\active\2010\1121 - geotechnical\10-1121-0089 jlr bridgetown chanery & or\10-1121-0089-1 chancery ger april 2011.docx

Table of Contents

Figures

APPENDICES

1.0 INTRODUCTION

1.1 Mandate

This report presents the results of a geotechnical investigation carried out on the grounds of Chancery of the Canadian High Commission in Bridgetown, St. Michael Parish, Barbados. The work was carried out in general conformance with our proposal dated July 14, 2010 and authorized by J.L. Richards and Associates Ltd. (JLR) on October 20, 2010.

The purpose of the geotechnical investigation was to assess the subsurface conditions at the site by means of a limited number of boreholes, geophysical surveys, and laboratory tests.

Based on an interpretation of the factual information available for this site, a general description of the subsurface conditions across the site is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this document.

1.2 Project Description

It is our understanding that Department of Foreign Affairs and International Trade Canada (DFAIT) has employed JLR to evaluate the existing Chancery building for its structural integrity under the current seismic building code. As part of this seismic evaluation, JLR has retained Golder Associates (Golder) to undertake geotechnical and geophysical investigations, provide foundation design guidance, seismic site classification in accordance with the National Building Code of Canada, and a karst delineation beneath select portions of the Chancery property in Bridgetown, Barbados. At this time it is understood that DFAIT is investigating an option to retrofit the existing Chancery building and to bring the existing structure up to current code and also upgrade the structure to meet security requirements. It is understood that if this retrofit is cost prohibitive, a study into constructing a new building somewhere else on the existing property will be undertaken. At this time, the potential location for a new building has not been determined, but the southeast corner of the property has been ruled out, where the existing tennis court, swimming pool and other related structures are located.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 Site Description

The Chancery is located near the eastern limits of Bridgetown, as shown in Figure 1. The property is about 100 metres wide by 130 metres depth and approximately 3.2 acres. The majority of the existing property is fully developed and surrounded by either perimeter fencing or stone walls. The northwest corner and the west side of the property are, however, undeveloped and consist of mature trees and thick undergrowth. Several buildings are currently located on the grounds of the Chancery as well as a tennis court and swimming pool. The two major buildings are the Chancery and Ambleside. There is an asphalt paved access road that splits the property into northern and southern halves. This access road leads to the employee parking lot west of the Chancery and to Ambleside and the garage on the west side of the property. To the north and east of Ambleside there are large open lawn areas. A smaller lawn is located east of the Chancery building. Along the west side of the property there is a tall stone retaining wall (about 2 metres high) separating the developed portion of the property and the forested area.

The topography of the property generally drains from east to west. The ground surface elevation of the developed areas ranges from about elevation 37 metres near the main entrance on the east side of the property to about elevation 32.5 metres, Geodetic, near the garage on the west side of the property. In the undeveloped portion of the property along the west side the ground slopes from about elevation 32.5 metres to about elevation 26.0 metres at about a 4 horizontal to 1 vertical slope. The ground surface continues to fall towards the perimeter access road, reaching about elevation 21.0 metres at the southwest property corner.

The existing Chancery building is a two story steel frame structure, with outside dimensions of about 26 metres by 27 metres and a ground floor area of about 590 square metres. The existing foundations are a combination of cast in place concrete spread footings and concrete blocks. Due to the shallow depth of the coral limestone, it is assumed that these foundations are bearing on this rock.

2.2 Geological Setting

Published geologic maps indicate that the geological conditions in the area of the Chancery consist of coral limestone of the Middle Coral Reef Terraces, as shown in Figure 2. According to available geological and topographic maps, the coral rock formations are approximately 70 metres thick and are underlain by the Tertiary rock of the Upper Scotland Formation.

It is understood that the island of Barbados was formed not by volcanic activity, like most Caribbean islands, but by the Atlantic tectonic plate being folded under the Caribbean tectonic plate. During this subsidence, a trough was formed, which allowed deep marine sediments to collect over time and was later covered by a dome of oceanic clay. As the plates continued to fold, the Atlantic plate was uplifted and eventually rose above the ocean level. This area in Barbados is called the Scotland District and is located in the northeast portion of the island. As conditions allowed, a coral reef began to form in the shallow waters west and south of the Scotland District. After a tectonic uplift during the Pleistocene Period, the coral reef was pushed out of the ocean, forming what is known today as the Upper Coral Rock Terraces. The coral reefs continued to grow on the south and west sides of the island protected from the rough Atlantic conditions and with continued periodic tectonic plate uplifts and changes in the ocean level during Pleistocene Period, the Middle and Lower Coral Rock Terraces were formed. Today, coral reefs continue to grow on the south and west coasts.

As the coral reefs harden, limestone rock is formed. Due to the composition of this limestone, the coral rock is susceptible to karst activities. Karstification is a geologic process where the limestone rock is dissolved by the acidic rainfall infiltrating through the porous and fractured limestone rock, which forms cavities within the rock and eventually under higher groundwater flows caves are formed. The karst activities are more prevalent in the older and higher coral rock terraces. The Middle Coral Rock Terraces, where the Chancery is located, are susceptible to karst activities which have been observed in outcrops at the school and neighbouring properties to the north and east of the Chancery. The most recent karst activity was discovered in 2007 with the collapse of an apartment building in Britton Hill, approximately 500 metres from the Chancery.

2.3 Seismotectonic Setting and Seismic Hazard

Barbados is located in the eastern Caribbean above a west-dipping, seismically- and volcanically-active subduction zone. The subduction zone is where the North American tectonic plate to the east sinks under the Caribbean plate to the west. Most earthquakes felt within the eastern Caribbean occur at the contact between the two plates or within the North American plate that dips at about 45 degrees beneath the eastern Caribbean.

Historical records of felt and instrumentally-recorded earthquakes that extend back about 500 years indicate that most of the large and damaging earthquakes have occurred within the Windward and Leeward Islands to the west of Barbados. A search of the US Geological Survey Preliminary Determination of Earthquake Epicentres (PDE) catalogue indicates that 14 earthquake epicentres with magnitudes (M) ≥ 5 have been located within about 200 kilometres of the Chancery site between 1973 and end of March 2011. All but one of these epicentres are located more than 100 kilometres from Bridgetown. The closest recorded event of these was a M 5.0 earthquake in April 1986 about 90 kilometres south of Bridgetown, and at a moderate depth of about 50 kilometres. The largest events were two M 5.7 earthquakes in August 1987 and July 1990 at distances of about 120 kilometres and 200 kilometres from Bridgetown, respectively. The historic earthquake record indicates that Barbados is located in a region of moderate earthquake activity.

Several estimates of seismic hazard have been developed for Barbados based principally on the 50-year instrumental record of earthquakes. In 2010, The University of West Indies Seismological Research Center and the European Center for Training and Research in Engineering (EUcenter) presented maps from a comprehensive, regional probabilistic seismic hazard analysis (PSHA) for the eastern Caribbean. The study results were developed using state-of-practice analytical methods, earthquake ground motions attenuation relations and hazard computing software. Key seismic hazard parameters from this study that are suitable for engineering analyses are shown below.

Selected Earthquake Accelerations from Probabilistic Seismic Hazard Analysis for Barbados[1]

Return Period (yrs)	Peak Horizontal Ground Acceleration (PGA) (g)	0.2-second Spectral Acceleration (g)	1-second Spectral Acceleration (g)
475	0.21 to 0.23	0.50 to 0.70	0.15 to 0.20
2,475	0.40 to 0.45	1.00 to 1.09	0.32 to 0.35

Note: (1) All values taken from 2010 SRC/EUcenter probabilistic seismic hazard analysis maps and tables, accessed April 3, 2011).

These 2010 PSHA results indicate higher levels of seismic hazard than previous studies, perhaps because of the longer record of earthquakes, inclusion of known faults and application of up-to-date acceleration attenuation functions in the 2010 study. Peak horizontal ground acceleration (PGA) and spectral acceleration values shown in the table above indicate a moderate level of hazard for the Chancery site in Bridgetown.

3.0 PROCEDURE

Our investigation consisted of a desktop study to gather existing information relevant to the project and a two phased field investigation to further define and confirm the findings developed during the desktop study. The first phase of the field investigation consisted of a geophysical survey using three different techniques. The second phase of the field investigation consisted of a conventional geotechnical borehole and laboratory investigation.

3.1 Desktop Study

A desktop study was carried out prior to the field investigations. During this study several reports were made available. A building condition survey report for the Chancery (Dessau, January 2008) was available, which provided basic structural and limited foundation information. A previous geotechnical investigation report (Dessau, April 2008) was also available, which provided subsurface information in the immediate vicinity of the Chancery building. A search using our in-house report database was also carried out and resulted in several past projects that Golder had carried out in Barbados which provided general geological information and coral rock properties.

3.2 Subsurface Investigations

3.2.1 Geophysical Surveys

As part of the current investigations, surface geophysical surveys were conducted over a large portion of the Chancery property, with the exception of the southeast portion of the property. A Multichannel Analysis of Surface Waves (MASW) survey line was conducted in the lawn east of the Ambleside Building to aid in selecting a seismic site classification for the property. Furthermore, electrical resistivity imaging (ERI) and ground penetrating radar (GPR) surveys were used to identify potential karstic areas. The following sections provide a summary of the geophysical surveys. More detailed results are presented in our technical memorandum titled “Chancery Property – Geophysical Survey Results”, dated April 11, 2011 in Appendix A.

3.2.1.1 MASW Survey

The MASW line was oriented southwest to northeast in the grassy area between the swimming pool and Ambleside building. For the MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 1.5 metre intervals. A sledgehammer was used as the seismic source for this investigation. Seismic records were collected with seismic sources located 20, 15, 10 and 5 metres from the end and collinear with the geophone array.

The MASW test results were used to produce a vertical shear wave velocity profile, shown below. The shear wave velocity profile presented below indicates that within the upper 7 metres velocities range between 785 and 1,098 metres per second while below a depth of 7 metres there is a gradual increase in velocity from 785 metres per second to approximately 2,400 metres per second at 11 metres. These results may indicate a transition within the bedrock.

Model Layer (mbgs)		Layer Thickness (metres)	Shear Wave Velocity (metres/second)	Shear Wave Travel Time Through Layer (seconds)
Top	Bottom			
0.00	1.07	1.07	996	0.001075
1.07	2.31	1.24	892	0.001387
2.31	3.71	1.40	1,098	0.001276
3.71	5.27	1.57	886	0.001767
5.27	7.01	1.73	785	0.002204
7.01	8.90	1.90	1,208	0.001570
8.90	10.96	2.06	1,838	0.001121
10.96	30.00	19.04	2,380	0.008000
V_S Average to 30 mbgs (m/s)			**1,630**	

Note: (1) Metres below existing ground surface (mbgs)

3.2.1.2 Karst Investigation

As part of the karst investigation, ERI and GPR geophysical surveys were completed around the Chancery property. Five ERI survey lines and thirty GPR survey lines were completed. These survey techniques can indicate changes in the subsurface conditions based on variations in either electrical resistance or radar reflections. These changes are then noted as anomalies, which were investigated further by obtaining physical samples and recording subsurface conditions through geotechnical boreholes placed along select survey lines.

Due to numerous site features, the depth of these surveys was limited. These survey techniques require long straight lines for deep penetration into the ground. Typically, the most accurate survey data is obtained within depths equal to approximately one sixth of the survey line. Thus, the ERI survey depths ranged from 7 to 15 metres and the GPR survey depths ranged from about 14 to 19 metres.

Several anomalies were observed in these surveys. Most of the anomalies were observed within 10 metres of the surface and generally located sporadically in the north half of the property. A deeper and more significant anomaly was observed along ERI Line C1 and possibly along ERI Line 2 in the western portion of the property as indicated in Figures 2, 3 and 10 of our technical memorandum in Appendix A. This deep anomaly was observed at a depth of 9 metres to the survey depth limit (15 metres) from the existing ground surface along ERI Line C1. Another possibly significant anomaly was observed in the staff parking lot. This shallower anomaly was also observed along GPR Lines C28 and C30. These areas were later explored by geotechnical boreholes to better define the composition of the subsurface conditions in these areas. These findings are presented in the Section 4.0, Subsurface Conditions.

More detailed results are presented in our technical memorandum titled "Chancery Property – Geophysical Survey Results" located in Appendix A.

3.2.2 Current Geotechnical Investigation

The field work for the geotechnical investigation was carried out between January 26, 2011 and February 8, 2011. During this period, a total of six boreholes (numbered C11-1 to C11-6, inclusive) were put down at the locations shown on Figure 3. The boreholes were advanced using a trailer-mounted drill rig supplied and operated by S.B. Testing and Engineering Ltd. of Bridgetown, Barbados. The boreholes were advanced to depths which vary from 4.6 to 25.9 metres below existing ground surface.

Within the boreholes, sampling and in situ testing was carried out in the overburden soils consisting of standard penetration tests (ASTM D1586), and samples of the soils encountered were recovered using drive-open sampling equipment.

In all six boreholes, coral limestone was proven to depth(s) of between 4.6 to 25.9 metres below the existing ground surface by rotary core drilling in NQ size. The bedrock core obtained was sequentially packed into core boxes.

The boreholes were covered, but left open until the investigation was completed which allowed for subsequent measurement of the groundwater levels at the site. The boreholes were eventually backfilled with granular/crushed coral and capped with concrete in paved areas and capped with topsoil in grassed areas.

The field work was supervised by an experienced technician from our staff who directed the drilling operations, logged the boreholes and took custody of the samples.

The borehole locations were selected by Golder based on the geophysical survey results and the potential developments on the site, and were staked in the field by Golder Associates personnel in relation to existing site features. The borehole elevations were referenced by the existing topography as shown on the base plan provided by JLR shown on Figure 3.

Upon completion of the drilling operations, the bedrock core obtained from the boreholes was transported to our laboratory for further examination by the project engineer and for laboratory testing. Photographs of the obtained rock core are contained in Appendix B.

The laboratory testing included uniaxial compressive strength (UCS) on eight (8) bedrock core specimens and Point Load index strength on twelve (12) rock specimens. The density of the tested samples was also determined. The test results are summarized in Section 4.3 and detailed results are included in Appendix D.

3.2.3 Previous Geotechnical Investigation

A previous geotechnical investigation was carried out around the existing Chancery building (Dessau, April 2008). This investigation consisted of four vertical boreholes and 25 GPR survey lines. The vertical boreholes extended to depths ranging from 10.7 to 13.7 metres below the existing ground surface. The location of these boreholes is shown on Figure 3. The GPR survey lines are shown on Drawing 1 in Appendix C, and generally surrounded the building and were also located inside the existing Chancery. The uniaxial compressive strength was determined on three bedrock core samples. The borehole logs and relevant GPR results from this previous investigation are given in Appendix C.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the current boreholes are shown on the Record of Borehole sheets in Appendix B. In general, the subsurface conditions encountered in these boreholes at this site consist of relatively thin overburden soils consisting of topsoil and import fill overlying vuggy coralline limestone.

The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes.

4.2 Overburden

In the lawns and less developed areas, the overburden soils generally consisted of topsoil directly overlying the coral limestone. The thickness of the topsoil ranged from 60 to 460 millimetres across the entire site. In the paved areas and more developed areas, granular crushed coral fill was encountered below the asphaltic pavement overlying the coral limestone. The surrounding areas had topsoil overlying the imported fill overlying the coral limestone. The fill generally consisted of fine to medium sand with varying amounts of crushed coral fragments. The fill thickness varied from 0.15 to 1.83 metres.

4.3 Coral Limestone

The overburden soils are generally underlain by coral limestone "rock" or cemented limestone deposits constructed from the broken debris of corals and the shells of the other organism that lived on the coral reefs (Harrison and Jukes-Brown, 1890). The coral limestone was encountered beneath the thin layer of topsoil and/or surficial fill and extended to the termination depths of all the boreholes in both investigations. The termination depths ranged from 4.6 to 25.9 metres below existing ground surface (Elevation 29.93 to 6.59 metres, Geodetic).

The coral limestone rock consists of discontinuous layers of highly fractured rock interbedded with layers of massive intact rock with numerous small cavities. In the rock core samples retrieved, the cavities within the coral limestone ranged from 5 to 30 millimetres in diameter. Larger voids were also encountered and are summarized below.

Borehole Number	Void Depth Range (m)	Void Elevation Range (m)	Approximate Size (m)
C11-2	3.66 - 4.27	29.04 – 28.43	0.61
C11-2	4.88 – 5.18	27.82 – 27.52	0.30
C11-3 (possible void)	3.05 – 4.57	31.45 – 29.93	1.52
C11-4	1.22 – 1.52	31.28 – 30.98	0.30
C11-4 (possible void)	4.57 – 6.10	27.93 – 26.40	1.53
C11-5 (possible void)	4.57 – 6.10	31.53 – 30.00	1.53
C11-6	4.27 – 4.42	31.93 – 31.78	0.15
BH-03-08	7.32 – 7.62	29.18 – 28.88	0.30
BH-04-08	5.03 – 6.40	31.62 – 30.25	1.37

Note: (1) Elevations are Geodetic.

In areas where no rock coring samples were retrieved, these areas were identified as possible voids. In these areas the limestone could also be very brittle and extremely porous such that during the coring process the rock was washed away.

The Total Core Recovery (TCR) percentage ranged from zero to 97 percent, however the majority of TCR values were below 45 percent. The Solid Core Recovery (SCR) percentage ranged from zero to 77 percent, however the majority of SCR values were below 35 percent. The Rock Quality Designation (RQD) values of the limestone ranged widely from zero to 44 percent, however the majority of RQD values were below 20 percent indicating a very poor to poor quality rock, with the majority of the rock being of very poor rock quality. The areas noted as having very low SCR and RQD values may reflect the rock coring process rather than the actual rock conditions since the rock is weak and very brittle and thus susceptible to breakage during the coring process.

The laboratory test results on samples of the coral limestone indicate a bedrock compressive strength which ranges widely from 5 to 52 megapascals. It should be noted that much of the rock core recovered during the current investigation was too small for proper testing and the compressive strength indicated below are from the limited samples that fit the proper dimensions for testing and could be providing the upper limits of compressive strength for this formation and may not be entirely representative of the entire formation. The density varies widely within the tested samples, as well. From the data points collected there is a very loose correlation between density and compressive strength, with lower densities having lower compressive strengths as shown in Figure D1 in Appendix D. However, at the higher densities there is a wider range of compressive strengths.

Borehole Number	**Sample Depth (m)**	**Sample Elevation (m)**	**Unconfined Compressive Strength (MPa)**	**Density (kg/m^3)**
C11-1	1.25 - 1.35	33.65 - 33.55	10.5	1814
C11-1	9.91 - 10.05	24.99 - 24.85	4.7	1268
C11-2	1.37 - 1.52	31.33 - 31.18	39.3	2287
C11-2	17.37 - 17.48	15.33 - 15.22	51.6	2188
C11-3	1.29 - 1.42	33.21 - 33.08	9.4	1801
C11-3	2.24 - 2.36	32.26 - 32.14	9.2	1794
C11-4	11.58 - 11.73	20.92 - 20.77	19.5	1709
C11-5	3.66 - 3.81	32.44 - 32.29	25.0	2054
BH-01-08-CR-8	8.70	27.65	-	1292
BH-03-08-CR-1	0.76	35.74	18.2	2250
BH-03-08-CR-6	8.43	28.07	18.5	1508
BH-04-08-CR-2	2.25	34.40	10.1	2256

Point Load Index testing was also carried out on selected core samples. It should be noted that much of the rock core recovered during the current investigation was too small for proper testing similar to the unconfined compressive strength testing, therefore the results could be providing the upper limits of compressive strength for this formation and may not be entirely representative of the entire formation.

Borehole Number	Sample Depth (m)	Geodetic Sample Elevation (m)	Point Load Index, $I_{s(50)}$, (MPa)	Correlated Uniaxial Compressive Strength[1] (MPa)	Volumetric Density, (kg/m^3)
C11-1	1.37 – 1.52	33.53 – 33.38	1.8	12	1694
C11-1	9.75 – 9.91	25.15 – 24.99	0.8	6	1290
C11-2	1.07 – 1.22	31.63 – 31.48	2.6	18	2056
C11-2	2.44 – 2.47	30.26 – 30.23	3.9	27	1718
C11-3	0.91 – 1.22	33.59 – 33.28	3.8	27	1967
C11-4	11.28 – 11.38	21.22 – 21.12	1.9	13	1603
C11-4	12.80 – 12.92	19.70 – 19.58	1.2	9	1306
C11-4	17.37 – 17.50	15.13 – 15.00	0.7	5	1177
C11-5	7.01 – 7.13	29.09 – 28.97	1.4	9	1566
C11-5	14.02 – 14.14	22.08 – 21.96	3.3	23	1726
C11-6	2.90 – 3.05	33.30 – 33.15	4.6	32	1486
C11-6	15.09 – 15.24	21.11 – 20.96	2.7	19	1696

[1] A conversion factor (K=6.9) was used to convert the point load strength index to an unconfined compressive strength. This value was based on unconfined compressive strengths of tested coral limestone samples.

Based on the borehole records, field observations, and the laboratory test results of the recovered core, it appears the coral limestone is banded in layers at the site. There are bands of coral limestone that have higher density with smaller voids and higher compressive strength and then there are also bands of highly fractured coral limestone that have larger voids and lower densities and strengths. Figure 4 illustrates this banding and the varying rock properties throughout the site. The following table provides a summary of this stratigraphy across the site. It should be noted that at certain locations at the site each stratum level may differ from the following given elevation range and there is high variability across the site.

Stratum Description	Approximate Elevation Range (m)	RQD Range (%)	UCS Range (MPa)	Density Range (kg/m^3)
Upper Dense Cap	31 – 36	0 – 68	9 – 39	1490 – 2290
Upper Fragmented Zone	25 – 31	0 – 65	6 – 27	1290 – 1720
Mid Dense Layer	19 – 25	0 – 37	5 – 23	1270 – 1730
Mid Fragmented Zone	16 – 19	0	-	-
Lower Dense Layer	11 – 16	7 – 15	5 - 52	1180 – 2190
Lower Fragmented Zone	7 – 11	0	-	-

4.3.1 Karst Formations

From the desktop study, three significant karst formations were found near the Chancery property. The school and neighbouring property just north of the Chancery (approximately 90 metres to the north), the apartment building collapse at Britton Hill (approximately 500 metres to the east) and Harrison's Cave (approximately 10 kilometres to the north).

Numerous small (<100 mm) and several moderate sized (<300 mm) voids were observed in all the core samples retrieved from the boreholes put down during the current investigation. These voids and cavities have been created by the nature process of surface water (slightly acid) infiltrating through the coral limestone and over time the water dissolves the limestone and creates these voids.

The geophysical investigation previously carried out by Dessau around the immediate vicinity of the Chancery building, indicated several small karst formations outside the south and east portions of the Chancery building. These karst formations were observed in the confirmatory boreholes and consisted of voids/cavities with dimensions of 300 to 1,400 millimetres in size within the coral limestone. These voids/cavities were observed at depths ranging from 5.0 to 7.6 metres below the existing ground surface.

The recent geophysical investigation carried out by Golder throughout much of the property indicates several anomalies or significant changes within the coral limestone, which could be related to karst activities. These anomalies were observed at a variety of depths and were grouped into shallow anomalies (less than 10 metres depth) and deep anomalies (greater than 10 metres) as shown in Figures 2 through 10 of our technical memorandum in Appendix A. Based on the subsurface conditions encountered within the boreholes put down along or near these survey lines, these anomalies generally consisted of less dense coral limestone with an increased in karst activity/formations. No significant large voids (> 3 metres) or caves were encountered in the boreholes, but several smaller voids (up to 1.5 metres in size) were encountered within the anomalies that were noted during the geophysical survey.

4.4 Groundwater

Groundwater was not encountered in any of the boreholes drilled during the current or past investigations. From the desktop study, groundwater levels are anticipated to be between 30 and 35 metres below the existing ground surface. Due to the high porosity of the coral limestone, it should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as raining or wet seasons and after significant rainfall events.

5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities.

The foundation engineering guidelines presented in this section have been developed in a manner consistent with the procedures outlined in the National Building Code of Canada (NBCC) for Limit States Design.

Since no significant karst formations were observed in our investigations, the site could be considered for future development. However, localized void/sink hole repairs or limited subgrade improvements may be needed during construction, depending on the layout of the proposed developments.

5.2 Seismic Site Response Classification

The NBCC 2005 contains an updated seismic analysis and design methodology which uses a seismic site response site classification system defined by the shear stiffness of the upper 30 metres of ground of interest. Seismic response is now defined by uniform hazard spectra (UHS) corresponding to a design earthquake with a probability of exceedance of 2% in 50 years. There are six site classes (from A to F), decreasing in soil stiffness from A (hard rock) to E (soft soil); Site Class F denotes problematic soils for which a site-specific evaluation is required. The site class is used to obtain soil factors (Fa and Fv) used to modify the UHS to account for the effects of site-specific soil conditions on the seismic response of the site to the design earthquake.

During the MASW analysis, the limited low frequency content of the MASW dispersion curve did not permit to sufficiently resolve shear-wave velocities at depths below 11 metres. Thus, the average shear wave velocity in the upper 30 metres was calculated assuming that the velocity from the maximum resolved depth (approximately 11 metres) to a depth of 30 metres was constant and equal to 2,380 metres per second. The average shear-wave velocity (V_{S30}) was found to be 1,630 metre per second. Interpreting the shear wave velocity data available indicates that a Site Class A designation would be appropriate for this site.

5.3 Foundations

Considering the shallow depth to rock across the property, it is considered that any proposed structures could be founded on spread footings founded directly on or within the rock; all footings should be supported by the intact competent rock.

Due to the karstic and porous nature of this rock formation and the banded layers of coral limestone located at the Chancery, two bearing failure modes need to be considered. For foundations bearing on the upper competent layer (i.e. Upper Dense Cap), a punching failure through this thin upper layer into the lower less competent layer (i.e. Upper Fragmented Zone) needs to be considered. The second failure mode is subsidence of the bearing surface as a result of the bearing surface being undercut by subsurface voids caused by the natural dissolving process (karst) of the coral limestone.

Footings placed directly on the rock surface of the “Upper Dense Cap” may be sized using a preliminary Ultimate Limit States (ULS) factored bearing resistance of 300 kilopascals. Rock probes should be carried out every 5 metres along wall foundations and at least one per column foundation to further assess the presence of localized voids under the foundations. Rock probes should be extended to at least 3 metres below the foundation bearing elevation. If higher ULS values are required, then the upper fragmented zone will need to be treated.

The rock surface needs to be properly cleaned of loose rock, soil and other debris at the time of construction, the settlement of footings sized using this factored bearing resistance should be negligible, and therefore Serviceability Limit States (SLS) need not be considered.

A stress analysis using the Boussinesq theory was carried out using the software program Settle 3D program developed by RocScience. To reduce the stress influence of the proposed foundations on the “Upper Fragmented Zone”, the foundation width should be limited to 1 metre in size or width. Larger foundations will have greater impacted on the “Upper Fragmented Zone” and the bearing resistances will need to be reviewed, if larger foundations we required.

If the rock needs to be treated to increase its bearing resistance, then compaction grouting would be a suitable method of treating the less competent rock formations and the larger karst voids. Due to the possibly of these karst formations being connected large volumes of grout should be anticipated during the planning of this project.

If a karst formation/void is located at the bearing surface then this void will need to be filled. Section 5.6.2 provides guidance on treating these voids.

These guidelines will need to be reviewed once the proposed construction is furthered defined.

5.4 Slab on Grade

For predictable performance of the floor slab, the existing topsoil and fill material should be removed from within the proposed construction. Provision should be made for at least 150 millimetres of crushed stone having a maximum aggregate size of 19 millimetres (local terminology ¾-inch stone mix) to form the base for the floor slab. Any bulk fill required to raise the grade to the underside of this granular pad should consist of crushed stone with a maximum aggregate size of 150 millimetres (local terminology: 6-inch minus stone). The underslab fill should be placed in maximum 300-millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

If the floor slabs are to be surface covered with non breathable floor coverings, a vapour barrier should be provided above the granular pad. The concrete slab should then be poured on a 50 millimetre thick layer of concrete sand to promote uniform curing, control the frequency of shrinkage cracks, and control the curling of the formed and saw cut edges of the concrete slab.

5.5 Rock Anchor Capacity

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

i) Failure of the steel tendon or top anchorage;

ii) Failure of the grout/tendon bond;

iii) Failure of the rock/grout bond; and,

iv) Failure within the rock mass, or rock cone pull-out.

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 200 kilopascals for ULS design purposes. The fragmented sections of rock should be ignored in determining bond lengths. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilized by the anchor. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where:

Q_r = Factored uplift resistance of the anchor, kilonewtons;

ϕ = Resistance factor, use 0.4;

γ' = Effective unit weight of rock, use 12 kilonewtons per cubic metre;

D = Anchor length in metres; and,

θ = ½ of the apex angle of the rock failure cone, use 30 degrees.

Where the anchor load is applied at an angle to the vertical, the anchor capacity should be reduced as follows:

$$Q_r' = Q_r \cos(\alpha)$$

Where:

Q_r' = Factored uplift resistance of the anchor subject to inclined load in kilonewtons;

Q_r = Factored uplift resistance of the anchor, kilonewtons; and,

α = Angle between the load direction and the vertical.

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors.

In the case of group effects for a series of rock anchors in a rectangle with width “a” and length “b” installed to a depth “D” with the bottom of the pit at depth “H” the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$\mathrm{V} = \frac{4}{3}\,(D+H)^3 sin^2\varphi + \; a(D+H)^2 \sin\varphi + b(D+H)^2 \sin\varphi + abD$$

Where:

V = Volume of the truncated trapezoid failure zone;
D = Depth of anchor group in metres;
H = Depth to the bottom of the pit, metres;
a = Width of anchor group in metres;
b = Length of the anchor group in metres; and,
φ = ½ of the apex angle of the rock failure cone, use 30 degrees.

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \phi\gamma \mathrm{V}$$

Where:

Q_r = Factored uplift resistance of the anchor, kilonewtons;
ϕ = Resistance factor, use 0.4;
γ = Effective unit weight of rock, use 12 kilonewtons per cubic metre; and,
V = Volume of truncated trapezoid.

Due to the highly variable conditions of the underlying coral limestone, it is suggested that both verification and proof-load tests be carried out on the anchors. At the beginning of the anchor installation three anchors should be verification tested to twice the design load with a creep test (to PTI standards). The proof load tests should be carried out to 1.3 times the anchor service loads, and at least 50 percent of the anchors should be tested in this manner.

It is suggested that the installation and testing of the anchors be supervised by the geotechnical engineer. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grout area with a minimum of voids. It is also suggested that the anchor holes be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to ensure an adequate bond between the grout and the rock.

Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

These rock anchor design guidelines will need to be reviewed once the additional geotechnical investigation is complete and the rock strengths have been verified.

5.6 Construction Considerations

5.6.1 Site Preparation

The initial step in the development of this site should be to remove topsoil, root matter, and other deleterious materials from the areas to be developed. Any existing foundations and abandon services should be to remove prior to earthwork activities within the area of proposed development.

After stripping, areas to be filled or where pavements or structures will be placed should be proofrolled with a heavily-loaded (15-20 ton) dump truck or another pneumatic-tired vehicle of similar size and weight. The purpose of the proofrolling is to provide surficial densification and to locate any isolated areas of soft soils or week rock. Unsuitable areas should be undercut and replaced with controlled compacted fill as described in Section 5.6.2. A professional geotechnical engineer or an engineering technician under the supervision of such an engineer should witness the stripping and proofrolling operations. All stripping and earthwork activities should be performed in a manner consistent with good erosion and sediment control practices.

5.6.2 Karst and Bedrock Void Treatment

It should be anticipated that during the excavation for foundations and other below grade constructions that karst formations and or voids may be encountered within the underlying coral limestone. These voids will need to be treated to provide a suitable bearing surface for the proposed foundation construction. Depending on the size and nature of the karst formation as well as the required bearing resistance, the most feasible and/or economical treatment can be determined. Figure 5 provides further detail of these treatment options.

For smaller karst formations (such as voids under 1 cubic metre) and under both load and non load bearing conditions, it is likely more feasible and economical to fill these voids with lean concrete ($\geq$15 MPa). All loose rock and any overhangs should be removed prior to filling these small voids.

For larger karst formations (such as voids greater than 1 cubic metre) under non load bearing conditions, it is most likely more economical to fill these voids with granular stone (crushed coral) capped with a layer of lean concrete ($\geq$15 MPa). This treatment option is detailed below and further illustrated in Figure 5:

- Excavate loose material from the karst formation/void. Expose intact coral rock on all sides of the excavation. Break off any rock overhangs and remove from the excavation. Excavate a minimum of 0.3 metres laterally beyond the limits of the karst formation/void.
- Place a geotextile (woven or non-woven) on all sides of the excavation extending past top of bedrock, to the extent practical. The geotextile should have a minimum tensile strength of 2 kilonewtons and a puncture strength of 0.6 kilonewtons.
- Backfill with crushed coral or other suitable granular stone to fill the void up to within 300 millimetres of the top of the void. The maximum particle size should not be larger than 200 millimetres (local terminology: 8-inch minus stone).
- Place one 150 millimetres compacted lift of crushed coral or other suitable granular stone to choke the larger stone backfill. The maximum particle size should not be larger than 25 millimetres (local terminology: 1-inch stone mix).
- Pour a minimum 150 millimetre thick layer of concrete ($\geq$15 MPa) over the crushed stone backfill. Allow 48 hours before continuing with construction to allow the concrete to gain 50-percent of it target compressive strength.

For larger karst formations (such as voids greater than 1 cubic metre) and under load bearing conditions, the void will need to filled with lean concrete ($\geq$15 MPa). All loose rock and any overhangs should be removed prior to filling the void.

5.6.3 Excavation and Backfill

Excavation for foundations and the installation of site services will be through a thin layer of overburden and into the underlying coral limestone throughout the site.

No unusual problems are anticipated in trenching in the overburden using conventional hydraulic excavating equipment. All excavations through the overburden should be sloped no steeper than 1 horizontal to 1 vertical. Side slopes should be stable in the short term at 1 horizontal to 1 vertical to depths of approximately 1 metre.

It is expected that rock removal for this project will be carried out using mechanical methods, such as hoe ramming or ripping, however, this work would likely be slow and tedious. Due to the friable nature of the coral limestone blasting is not recommended at this site.

Near vertical trench walls in the coral limestone should stand unsupported for the construction period.

Some groundwater inflow into the trenches and excavations may be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps established in the floor of the excavations.

For backfilling excavations, engineered fill should be placed in maximum 300-millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

For pipe bedding for services, at least 150 millimetres of crushed granular stone having a maximum aggregate size of 19 millimetres (local terminology: 3/4-inch stone mix) should be used. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density.

Cover material, from spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of a crushed granular stone having a maximum aggregate size of 26.5 millimetres (local terminology: 1-inch stone mix) with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the material's standard Proctor maximum dry density.

Well fractured or well broken bedrock will be acceptable as backfill for the lower portion of the service trenches in areas where the excavation is in rock. The rock fill, however, should only be placed from at least 300 millimetres above the pipes to minimize damage due to impact or point load. The rock fill should be limited to a maximum of 300 millimetres in size.

6.0 ADDITIONAL CONSIDERATIONS

Due to the karstic nature of the underlying coral limestone and the nearby building collapse, future and periodic subsurface investigations are warranted at this location to determine rate of degradation within the limestone formation. The subsurface conditions should be investigated every 10 years as a benchmark. The results of the future investigations may warrant more frequent investigations.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that limestone rock has adequate bearing capacity and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction view point.

At the time of the writing of this report, only conceptual details for the proposed development were available. Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

7.0 LIMITATIONS

This report has been prepared for the exclusive use by J.L. Richards & Associates, Department of Foreign Affairs and International Trade of Canada and their agents for specific application to the proposed developments on the grounds of the Chancery of the Canadian High Commission in Bridgetown, Barbados. The findings and guidelines presented in this report were prepared in accordance with generally accepted geotechnical engineering practice at the time of this study. It is stressed that the information in this portion of the report is provided for the guidance of the designers and is intended for this project only.

The client has the responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. This report contains information useful in the preparation of tender documents. However, the report is not intended as a construction specification and would require modification for use as such.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. 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.

GOLDER ASSOCIATES LTD.

Bruce D. Goddard, P.Eng. Senior Geotechnical Engineer	Michael S. Snow, P.Eng. Principal

BDG/MSS/tm
n:\active\2010\1121 - geotechnical\10-1121-0089 jlr bridgetown chanery & or\10-1121-0089-1 chancery ger april 2011.docx

REFERENCES

Boyles, J.E., (1988), “Foundation Analysis and Design”, McGraw-Hill, 4th Edition, 1004 p.

Canadian Geotechnical Society. 2006. Canadian Foundation Engineering Manual, 4th Edition. BiTech Publishers Ltd., Richmond, British Columbia.

Dessau, (2008), Building Condition Report for Chancery, Bridgetown, Bishop Court Hill, Barbados, (Report No. 034-P015952-0100-000-ME-0001-00).

Dessau, (2008), Building Condition Report: Geotechnical Investigation on Chancery, Bridgetown, Bishop Court Hill, Barbados (Report No. 034-P015952-0160-SC-0001-00).

Harrison, J.B. and Jukes-Brown, A.J. 1890. The Geology of Barbados. Published by authority of the Barbadian Legislature.

Giardini, D., Grünthal, G., Shedlock, K., Zhang P., and Global Seismic Hazards Program, 1999., Global Seismic Hazards Map, accessed January 2007.

Tanner, J.G., Shepherd, J.B., (1997), Seismic hazard in Latin America and the Caribbean. Final Report to the International Development Research Centre, Ottawa, Canada, Seismic Hazard in Latin America and the Caribbean, Vol. 1. Instituto Panamericano de Geografia y Historia, Mexico, D.F. 142 pp.

Veneziano, D., Cornell, C.A., O'Hara, T., (1984), Historic method of seismic hazard analysis, Elec. Power Res Inst., Report NPn3428, Palo Alto, CA.

Wyllie, Duncan C., “Foundations on Rock”, Chapman & Hall, 1st Edition, 1992.

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, J.L. Richards & Associates Ltd. . 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.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

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.

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

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.

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.

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 10-1121-0089

REFERENCE

DIGITAL BASE MAP DATA COPYRIGHT:© 2009 ESRI, AND, TANA, ESRI JAPAN, UNEP-WCMC

0 2 4 8

SCALE 1:185,000 KILOMETRES

Golder Associates
Ottawa, Ontario

PROJECT No. 10-1121-0089
SCALE AS SHOWN
REV. 0

DATE 25 MAR. 2011
DESIGN BDL
GIS BT
CHECK BDL
REVIEW ISS

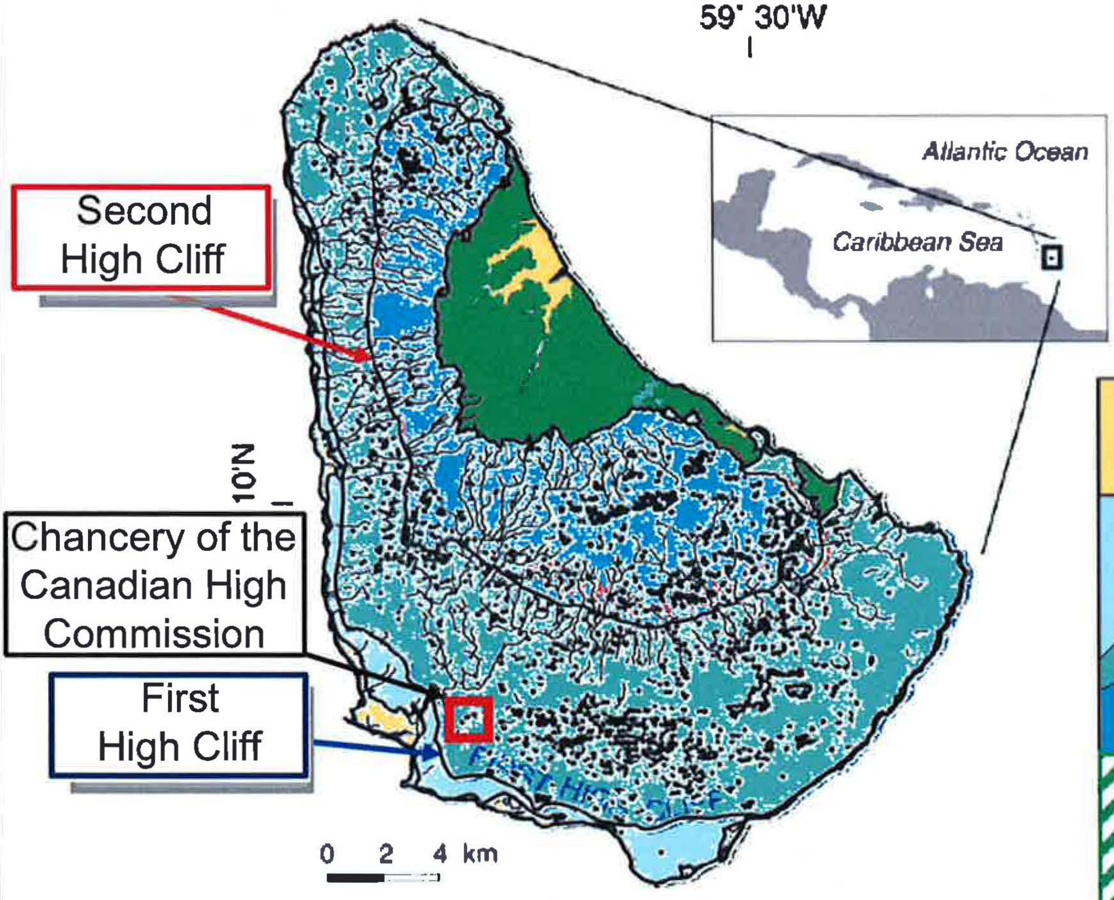
TITLE

KEY PLAN

PROJECT GEOTECHNICAL INVESTIGATION FOR CANADIAN HIGH COMMISSION

FIGURE 1

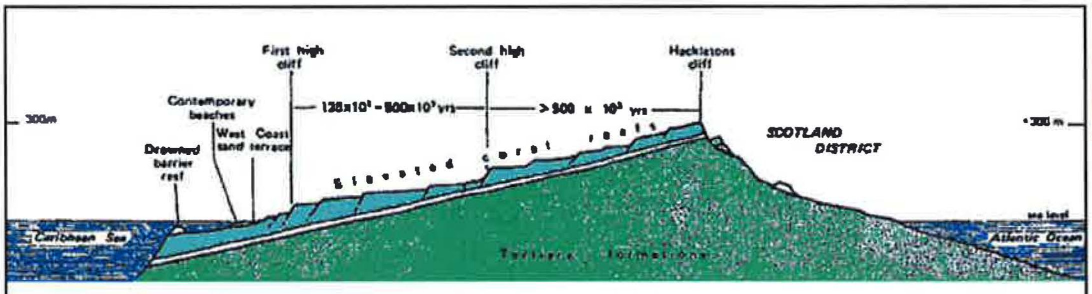
Path: N:\Active\2010\1121 - Geotechnical\10-1121-0089 JLR Bridgetown Chanery & OR\GIS\1011210089-1000-01.mxd



LEGEND

- 🟨 Quaternary sediment
- Pleistocene limestone
- 🟩 Tertiary rock
- Dry valleys
- Sinkholes

Location of First & Second High Cliffs and Karst Zones (Modified from Jones, I.C., & Banner J.L (2003), after Directorate of Overseas Surveys 1:50,000 geologic map (1983))



Geomorphological cross section of Barbados: figures in parenthesis indicate approximate dates of emergence from the sea.

Golder Associates
Ottawa, Ontario, Canada

SCALE	AS SHOWN	TITLE	SUMMARY OF BARBADOS GEOLOGY
DATE	25 MAR. 2011		
DESIGN	BDG		
CAD	J.F.C./J.M.		
CHECK	B.D.G.		
REVIEW	M.S.S.		

FILE No. 1011210089-1000-02.dwg
PROJECT No. 10-1121-0089 REV. 0

GEOTECHNICAL INVESTIGATION FOR CANADIAN HIGH COMMISSION

FIGURE 2

PLOT DATE: April 08, 2011
FILENAME: N:\Active\2010\1121 - Geotechnical\10-1121-0089 JLR Bridgetown Chanery & OR\ACAD\Phase 1000\1011210089-1000-02.dwg

LEGEND

⊕	APPROXIMATE BOREHOLE LOCATION IN PLAN
●	APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATION BY DESSAU INC. (MARCH 2008)
A	APPROXIMATE CROSS-SECTION LOCATION

GEOPHYSICS LEGEND

🟥	ERI AND GPR LINES
🟦	MASW LINE
🟩	GEORADAR LINES, PREVIOUS INVESTIGATION BY DESSAU, MARCH 2008

REFERENCE

BASE PLAN SUPPLIED IN ELECTRONIC FORMAT BY J.L. RICHARDS & ASSOCIATES LIMITED

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 10-1121-0089-1

REV	DATE	DES	REVISION DESCRIPTION	CAD	CHK	RVW

PROJECT

GEOTECHNICAL INVESTIGATION
FOR CANADIAN HIGH COMMISSION

TITLE

SITE PLAN

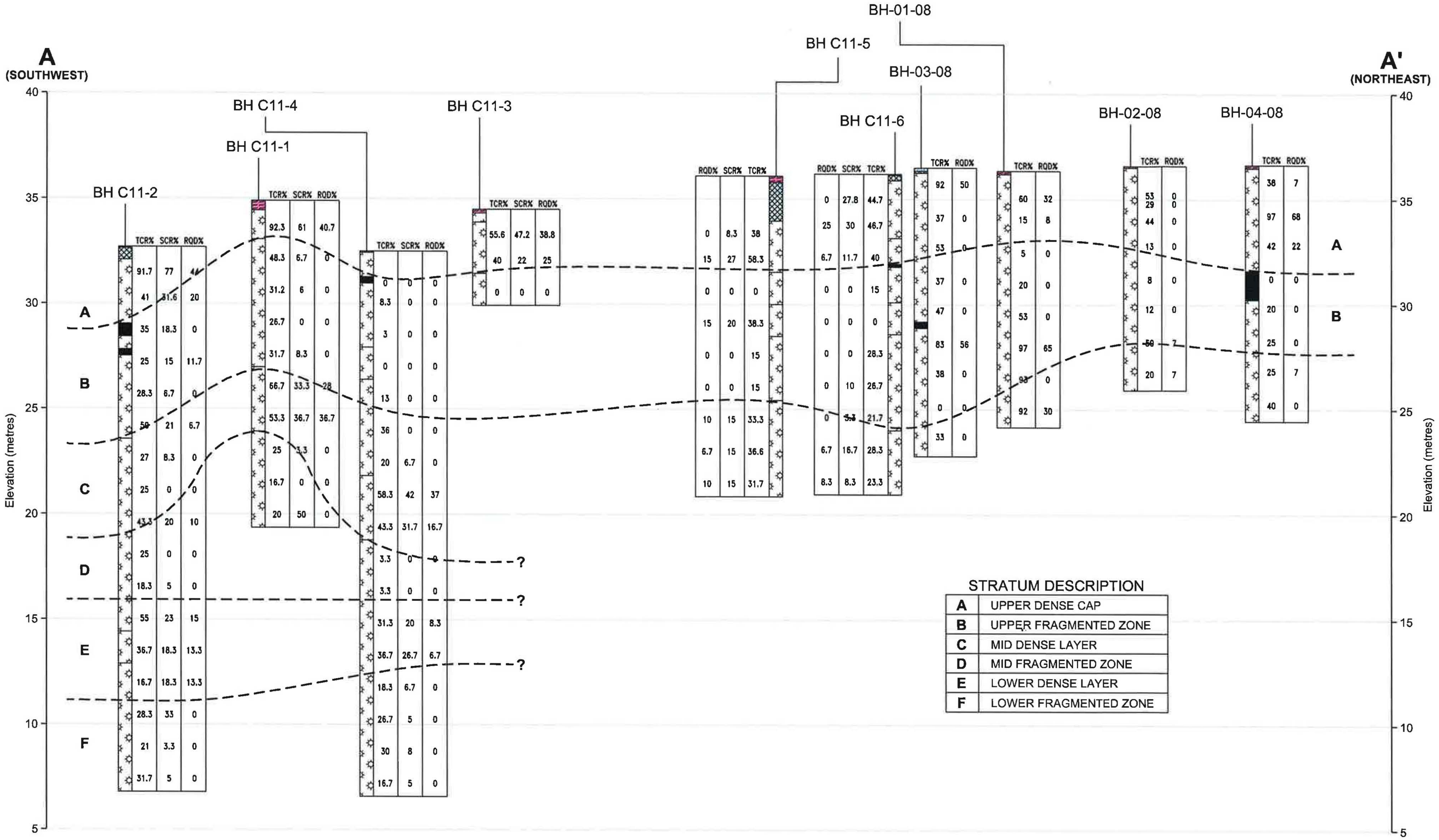
Golder Associates
Ottawa, Ontario, Canada

PROJECT No.		10-1121-0089
DESIGN	B.D.G.	15 APR. 2011
CAD	J.M.	18 APR. 2011
CHECK	B.D.G.	21 APR. 2011
REVIEW	M.S.S.	21 APR. 2011

FILE No. 1011210089-1000-03.dwg
SCALE 1:250 REV.
DRAWING No.
3

PLOT DATE: April 20, 2011
FILENAME: N:\Active\2010\1121 – Geotechnical\10-1121-0089 JLR Bridgetown Chanery & OR\ACAD\Phase 1000\1011210089-1000-03.dwg

PLOT DATE: April 20, 2011
FILENAME: N:\Active\2010\1121 - Geotechnical\10-1121-0089 JLR Bridgetown Chancery & OR\ACAD\Phase 1000\1011210089-1000-04.dwg



STRATUM DESCRIPTION

A	UPPER DENSE CAP
B	UPPER FRAGMENTED ZONE
C	MID DENSE LAYER
D	MID FRAGMENTED ZONE
E	LOWER DENSE LAYER
F	LOWER FRAGMENTED ZONE

LEGEND

- - - - - APPROXIMATE STRATUM LINE / BANDING

SUBSURFACE STRATIGRAPHY

▤	TOPSOIL	▨	CONCRETE
▭	ASPHALTIC CONCRETE	✲	CORAL LIMESTONE
▩	FILL	■	VOID

HORIZONTAL
8 0 8 16
SCALE 1:400 METRES

VERTICAL
4 0 4 8
SCALE 1:200 METRES

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 10-1121-0089-1

Golder Associates
Ottawa, Ontario, Canada

FILE No. 1011210089-1000-04.dwg
PROJECT No. 10-1121-0089 REV.

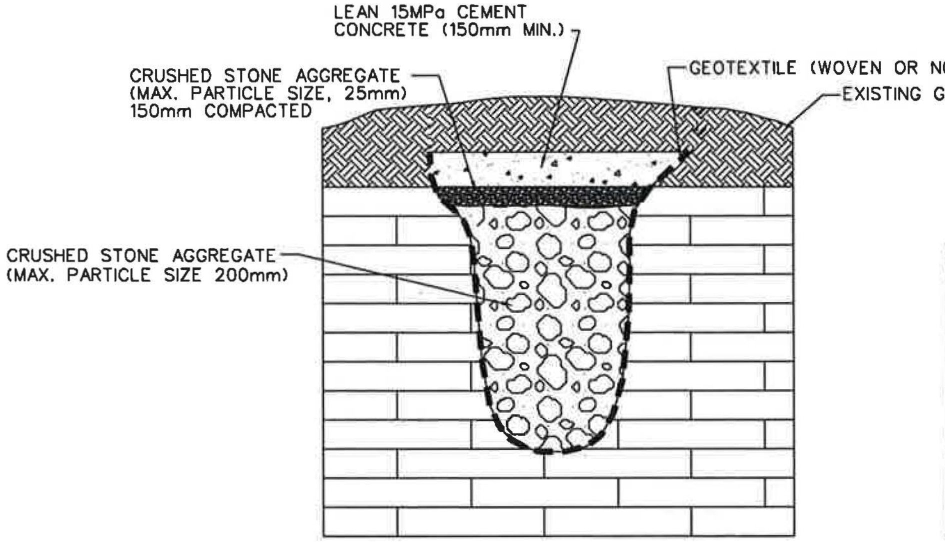
SCALE AS SHOWN
DATE 7 APR. 2011
DESIGN B.D.G.
CAD J.M.
CHECK B.D.G.
REVIEW 155

TITLE

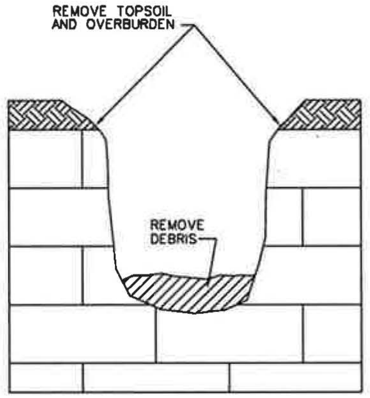
CROSS-SECTION A-A' SUBSURFACE PROFILE

GEOTECHNICAL INVESTIGATION FOR CANADIAN HIGH COMMISSION

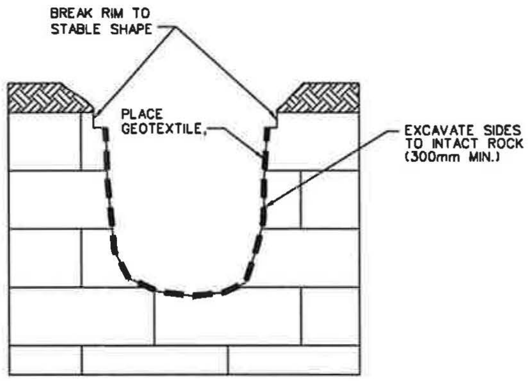
FIGURE 4



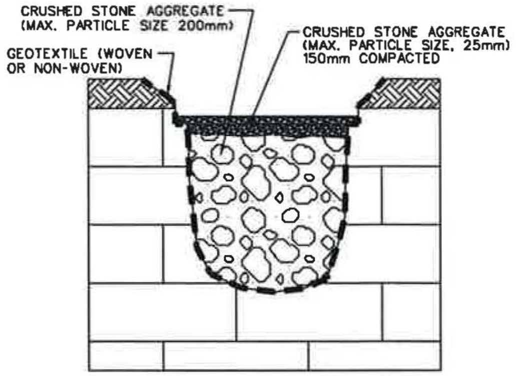
LARGE VOID IN ROCK - NON LOAD BEARING



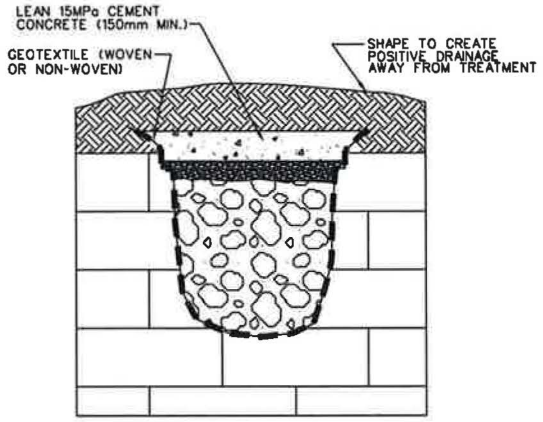
STEP 1:
REMOVE DEBRIS



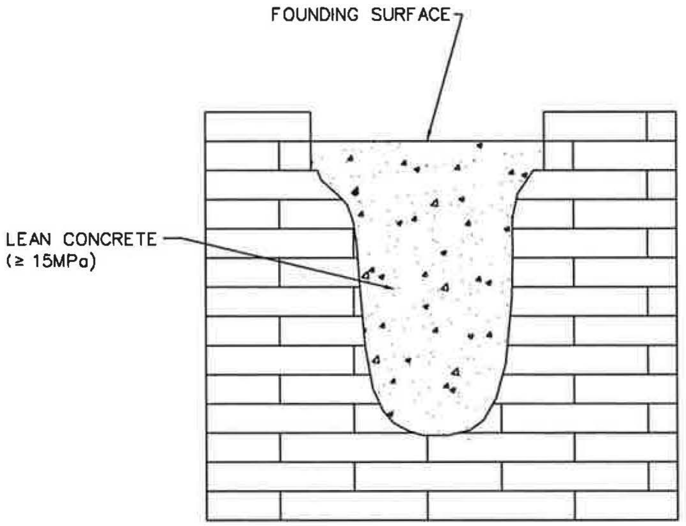
STEP 2:
SHAPE EXCAVATION AND PLACE GEOTEXTILE



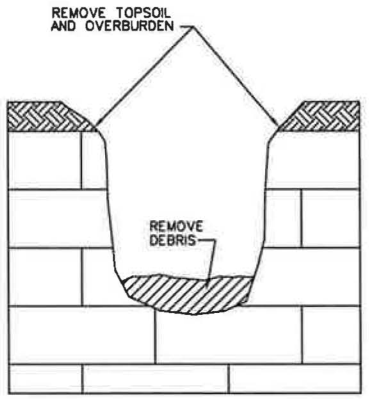
STEP 3:
FILL VOID WITH AGGREGATE



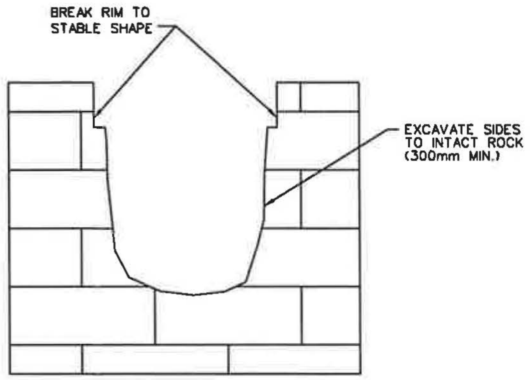
STEP 4:
CAP TREATED AREA WITH CONCRETE



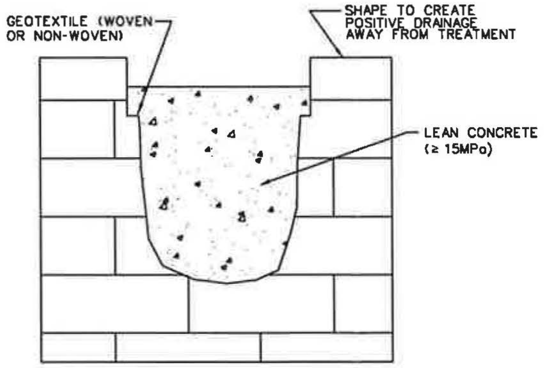
SMALL/LARGE VOID IN ROCK - LOAD BEARING



STEP 1:
REMOVE DEBRIS



STEP 2:
SHAPE EXCAVATION



STEP 3:
FILL VOID WITH CONCRETE

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 10-1121-0089-1

Golder Associates
Ottawa, Ontario, Canada

FILE No. 1011210089-1000-05.dwg
PROJECT No. 10-1121-0089
REV.

SCALE NTS
DATE 19 APR. 2011
DESIGN B.D.G.
CAD J.M.
CHECK B.D.G.
REVIEW JSS

TITLE
TREATMENT PROCESS FOR KARST FORMATION AND VOIDS

GEOTECHNICAL INVESTIGATION FOR CANADIAN HIGH COMMISSION

FIGURE
5

PLOT DATE: April 20, 2011
FILENAME: N:\Active\2010\1121 - Geotechnical\10-1121-0089 JLR Bridgetown Chanery & OR\ACAD\Phase 1000\1011210089-1000-05.dwg

APPENDIX A

Technical Memorandum (April 11, 2011)
Chancery Property - Geophysical Survey Results

Golder Associates

TECHNICAL MEMORANDUM

DATE April 11, 2011 **PROJECT No.** 10-1121-0089

TO John Elliot
J.L. Richards & Associates Ltd.

CC Bruce Goddard

FROM Stephane Sol, Christopher Phillips **EMAIL** ssol@golder.com; cphillips@golder.com

CHANCERY PROPERTY – GEOPHYSICAL SURVEY RESULTS

This technical memorandum presents the geophysical field work and data analysis completed for the geotechnical investigation at the Canadian Government Chancery in Bridgetown, Barbados. The purpose of the investigation was to use geophysical methods to aid in delineating the location of potential voids and karstic cavities beneath portions of the property site to provide borehole targets for the geotechnical drilling program, and to determine the National Building Code of Canada (NBCC) seismic site classification of the property.

Three geophysical methods were used as part of the investigation: Electrical Resistivity Imaging (ERI) and Ground-Penetrating Radar (GPR) for detection of areas of potential voids and to direct the location of the boreholes and Multichannel Analysis of Surface Waves (MASW) for site classification determination.

Five (5) ERI lines and thirty (30) GPR lines were acquired across the property in areas, and were laid out as access and space permitted. The MASW survey line was collected in the grassy area just west of the pool and tennis court.

Topography information provided by J.L. Richards and Associates Ltd. (J.L. Richards) was used to support the analysis and interpretation of the electrical resistivity.

This technical memorandum presents the results of the geophysical investigation.

Methodology

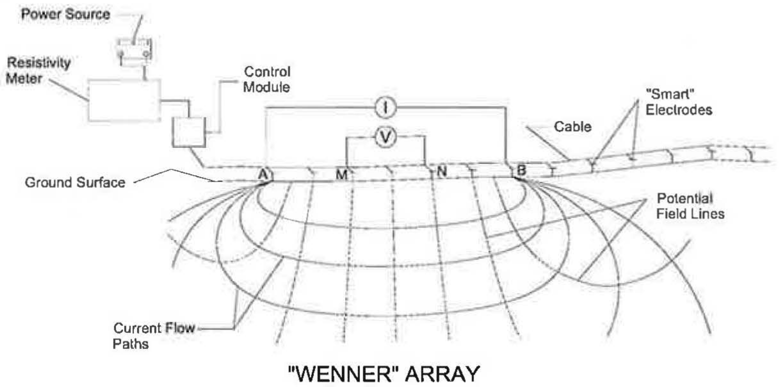
Electrical Resistivity Imaging (ERI)

The electrical resistivity imaging (ERI) technique measures the electrical resistivity (reciprocal of conductivity) of the subsurface to infer rock/soil types, stratigraphy and soil conditions. The physical principles for this technique are the same as that established for direct-current (DC) resistivity, in which the apparent resistivity of the subsurface is calculated for increasing electrode separations by applying a current to the ground using two electrodes and measuring the potential difference (voltage) between two different electrodes.

Apparent resistivity of the subsurface is calculated from the potential to current ratio multiplied by a constant. This constant is a function of the electrode spacing and geometry. The depth of investigation possible is also a function of the electrode separation. Thus, with larger electrode separations, information from greater depths can be acquired, but at the cost of decreased resolution.

ERI differs from the traditional DC sounding techniques in that a "spread" of electrodes (typically 56, 72 or more) are staked along a survey line and connected to a resistivity meter by a cable fitted with multiple takeouts. The resistivity meter is a computer-controlled device consisting of a current supply capable of producing switched +/- constant current and a high impedance voltmeter.

A software routine is loaded on to the resistivity meter and the electrodes are switched on and off as required throughout the measurement process. This equipment and procedure allows for automated collection of high-density data along the entire spread. As the line of resistivity coverage is continued, cables from the start of the electrode array are moved (rolled) to the end and measurements are continued. By "leap-frogging" the array system along the survey line, a semi continuous pseudo-section of apparent resistivity values versus apparent depth beneath the profile line can be generated. These data are then inverted to calculate a two-dimensional resistivity model for the profile with modelled true depths and resistivity. RES2DINV is the computer program that is used to invert the survey data to determine two-dimensional resistivity models for the subsurface.

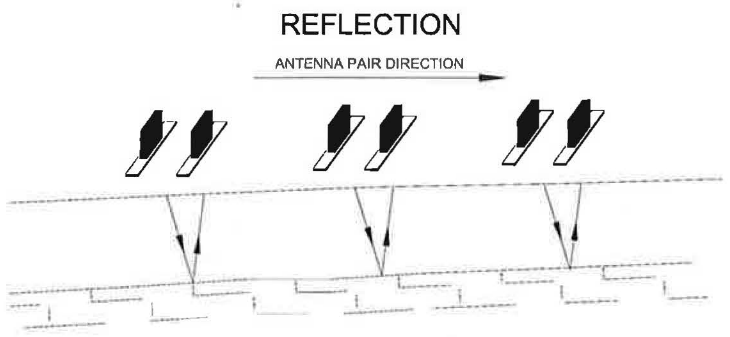


Example 1: Principle of the Wenner- layout for resistivity survey.

Ground Penetrating Radar (GPR)

The GPR system consists of two antennae (transmitter and receiver), a control console and a computer for real-time, graphic display and data recording. In reflection profiling mode, the antennae, separated a fixed distance, are moved stepwise along a traverse and readings are taken at discrete intervals. At each step, pulses of radar frequency electromagnetic energy (megahertz range) are transmitted and reflections received from subsurface horizons. The reflecting horizons occur where there is an abrupt change in the subsurface material dielectric permittivity such as at the interface between host rock and an underground void. The amplitude of received radar energy is recorded as a function of time, processed in real-time for display purposes, and the raw data recorded digitally for later processing and presentation.

GPR sections are presented as time-sections, with the position (in metres) of each trace recorded as the horizontal axis across the top of the section and the GPR travel time (in nanoseconds, increasing downward) as the principal vertical axis. A second vertical axis is included to provide an estimate of depth or elevation and is calculated assuming a constant GPR velocity for the subsurface, which is obtained through common-midpoint tests at several locations along a survey line.



Example 2: Typical GPR Surveying Methods.

Electromagnetic pulses, like those used in a GPR system, are strongly attenuated when travelling through conductive materials. The depth of investigation of a GPR system is therefore strongly influenced by the conductivity of the subsurface, where the greater the conductivity the shallower the depth of investigation. Conductive materials (e.g., clay) will attenuate the GPR signal at the subsurface.

Multichannel Analysis of Surface Waves (MASW)

The Multichannel Analysis of Surface Waves (MASW) method measures variations in surface wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

A typical MASW survey requires a seismic source, to generate surface-waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium.

The depth of penetration of a surface-wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium surface-waves are dispersive, i.e., each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that particular wavelength of surface-wave propagates through. The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, sledge hammer and vibrating pads.

Examples of passive sources are road traffic, micro-tremors and water-wave action (in near-shore environments).

The geophone receivers measure the wave-train associated with the surface wave travelling from a seismic source at different distances from the source.

The participation of surface-waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to wavelength (called the 'dispersion curve') is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear modulus of the medium as a function of depth.

Field Work and Processing

The geophysical field work was carried out by Golder personnel from the Mississauga office between November 7 and 16, 2010. Locations of the geophysical lines, surveyed using a GPS, are presented in Figure 1. Layout and location of the geophysical lines were determined on site based on access and space available.

Electrical Resistivity Imaging

The ERI geophysical survey consisted of three steps: survey design, line layout and ERI surveying. The survey design and lines were laid out using a hand held GPS for positioning in the field. The ERI survey was carried out using a SYSCAL R1 Plus Switch 72 channel resistivity system (manufactured by IRIS Instruments). The resistivity data were collected using a Wenner type of electrode array. Based on available survey line length, electrode spacings of either 1 or 1.5 m were used, yielding depths of investigation of approximately 10 or 15 m below ground surface (mbgs), respectively.

For each setup of the ERI system, a continuity check and contact resistance check was made for all electrodes prior to initiating a reading cycle. Contact resistances at the electrodes during the survey were typically within the optimal range (100 ohms or less).

The resistivity system was set up to pass enough current at the current electrodes to generate a measurable voltage at the potential electrodes in the range of 300 mV, in order to yield data with high signal to noise ratio. Data was analyzed in the field at the time of data collection for quality control and to decide if a GPR survey was required in areas where the resistivity sections indicates the presence of anomalous zones. Upon completion of the survey the ERI data were first processed to remove spurious data points. Spurious data points in a data set can be caused by several factors, including presence of localized buried metal objects, poor coupling of electrodes to the ground, and the undue influence of infrastructure. Generally, less than 1% of the readings along each survey line were removed from the raw data set.

The elevations along the ERI lines were extracted from the topography data provided by J.L. Richards, using the GPS positions collected along each resistivity line at the time of the survey. The topographic data were combined with the ERI data to include topography along the line in the model results. The ERI survey results were modelled using the inversion program RES2DINV, an industry standard software package developed by Dr. M.H. Loke.

The ERI models were contoured using the Surfer Surface Mapping System (Golden Software) using a Kriging algorithm and a cell size of 0.5 m for the 1 m electrode spacing and 0.75 m for the 1.5 m electrode spacing. The contoured models were then imported to AutoCAD (Autodesk Inc.) for interpretation and presentation.

Ground Penetrating Radar

The GPR data were collected using the PulseEkko 100 ground penetrating radar system manufactured by Sensors and Software Inc. The survey parameters for each system are summarized in the table below.

Table 1: GPR Collection Parameters

Parameter	**50 MHz Antennas**	**100 MHz Antennas**
System Centre Frequency	50 MHz	100 MHz
Antenna Separation	2 metres	1 metres
Step Size along Line	0.2 metres	0.2 metres
Number of Stacks	8	8

Processing of the GPR data was accomplished using the ReflexW software package (Sandmeier, 2005). The radar profiles were processed to improve the presentation quality of the data to aid with the interpretation. Processing included, dewowing (removal of early time data bias), energy decay and low pass filter. A GPR velocity of 0.11 m/ns, typical for soils/rock, was used to estimate the depth. The velocity was selected based on common midpoint surveys conducted at the site and on diffraction patterns from point reflectors within the collected datasets.

The resolution and penetration of a GPR system is dependent on the centre frequency of its operation. Lower frequency antennas penetrate deeper into the subsurface, but have less vertical resolution than do higher frequency antennas. At the Chancery property, the lower frequency antennas did not penetrate deeper than the higher frequency antennas. The 50 MHz antennas were only used the first day of field work and were replaced by 100 MHz antennas for increased resolution in the datasets.

GPR sections are presented as time-sections, with the position (in metres) of each trace recorded as the horizontal axis across the top of the section and the GPR travel time (in nanoseconds, increasing downward) as the principal vertical axis. A second vertical axis is included to provide an estimate of depth or elevation and is calculated assuming a constant GPR velocity for the subsurface of 0.11 m/ns.

A key aspect to interpretation of the GPR profiles is to have control at one or preferably more locations along the survey line. Although it is generally reasonable to provide preliminary interpretations of GPR data, it is necessary to confirm interpretation of the GPR data with results from intrusive investigations such as boreholes.

GPR antennae, whether shielded or unshielded, tend to pick up air wave reflections from objects at surface proximal to the survey line such as columns or buildings. These air wave events are, in general, distinct in their shape and frequency content as observed on reflection profiles and can usually be identified with confidence on the GPR sections during interpretation.

Multichannel Analysis of Surface Waves

The MASW line was oriented southwest to northeast in the grassy area west of the pool and tennis court (Figure 1). For the MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 1.5 m intervals. A sledgehammer was used as the seismic source for this investigation. Seismic records were collected with seismic sources located 20, 15, 10 and 5 m from the end and collinear with the geophone array. An example of an active seismic record collected is shown in Plate 1 (below).

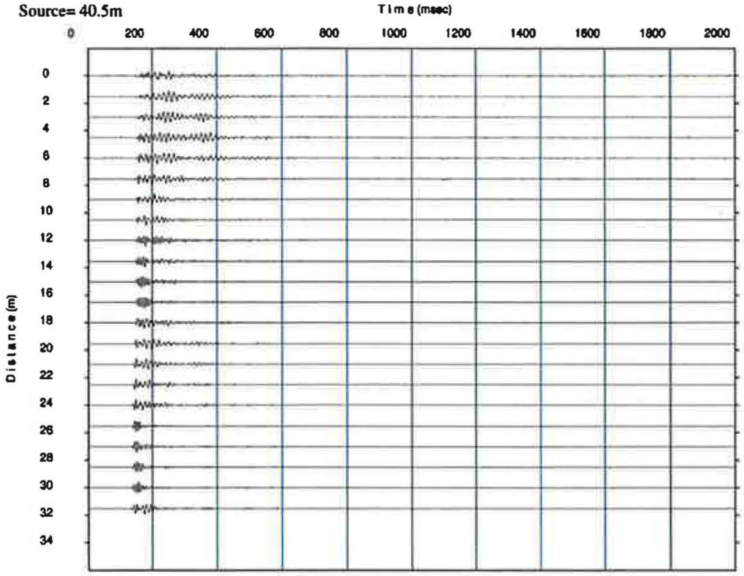


Plate1: Typical seismic record collected at the site.

Processing of the MASW test results consisted of the following main steps:

1) Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;
2) Calculation of the phase for each frequency component;
3) Linear regression to calculate phase velocity for each frequency component;
4) Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r^2) between the data and the linear regression best fit line used to calculate phase velocity;
5) Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and
6) Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown on Plate 2. Shear wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves.

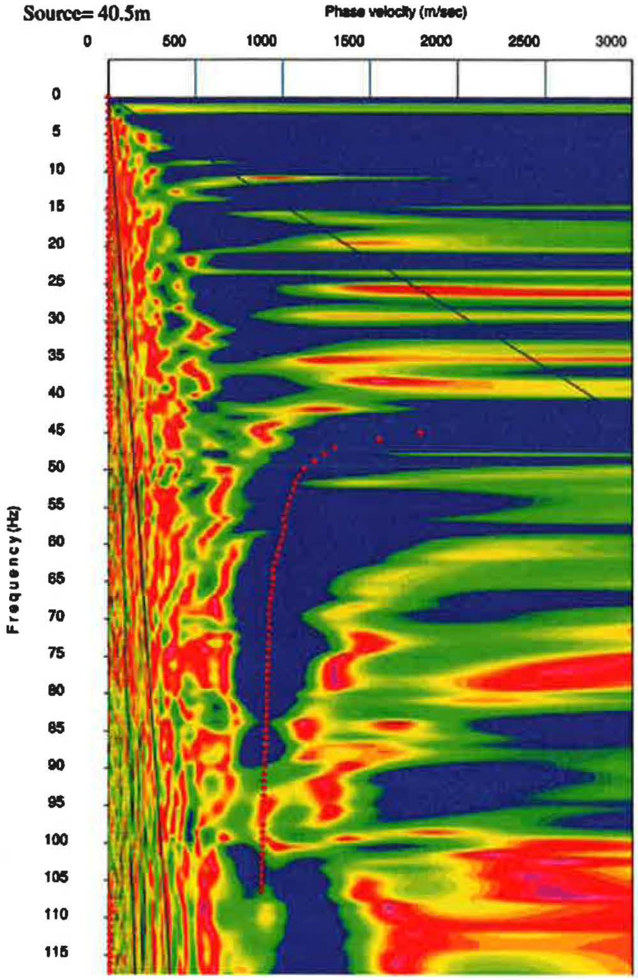


Plate 2: MASW Dispersion Curve Picks (red dots)

The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 45 Hz.

Survey Results

ERI and GPR Results

Interpreted results for the resistivity lines are shown on Figures 2 through 6. Three boreholes were available in proximity of a few ERI lines. Borehole BH C11-1 was located approximately 7 m west of ERI line C4, borehole BH C11-4 was located approximately 10 m from the south edge of ERI line C2, and borehole BH C11-5 located at the northern edge of ERI line C5. The geological interpretation is largely based on changes in resistivity contrast because correlation with borehole information provided at only a few locations was difficult to establish. None of the boreholes were located in zones where large resistive anomalies were observed. The presence of fractured coral bedrock within both the low and high resistive layers suggests that the major change in resistivity contrast reflects a change in moisture content within the bedrock.

The interpreted massive and/or fractured with low moisture content coral bedrock profile is presented as a high resistivity layer at depth, with resistivity typically greater than 600 ohm-metres. The interpreted topsoil and/or fractured bedrock with high moisture content is presented in the ERI data as a low resistivity layer, with resistivities typically less than 200 ohm metres. For each ERI line, the root mean squared error associated with the final resistivity inversions were on average 5 to 10% after five (5) iterations. Areas of low resistivity within the bedrock are interpreted as karstic features (voids, fracture zones, etc.). A few anomalies are observed within the interpreted bedrock along ERI lines C1 and C2, the two lines located in a forest area along the western portion of the property. The depth and the lateral extent of these anomalies are variable. The interpreted karstic zones extend down to at least 15 mbgs beneath the center part of ERI line C1. The two anomalies observed along ERI line C2 appear to not extend as deep as the large anomaly observed on line C1. ERI line C5 indicates a potential dip in the bedrock at the northern end of the line. No significant anomalies were detected along ERI lines C3 and C4.

Three representative examples of GPR sections are presented on Figures 7 through 9. Figures 7 and 8 showed two types of anomalies that have been detected along GPR lines GC28 and GC12. These anomalies are interpreted as karstic zones representing zones of highly fractured coral bedrock and/or void as seen in borehole BH C11-4. Figure 9 shows the presence of several uniform large amplitude reflectors that represent air waves. No significant anomalies, indicative of karstic zones, are evident in the majority of the other GPR sections (Appendix A).

An anomaly map summarizing anomalies observed on both the ERI and GPR lines is presented on Figure 10.

MASW Results

The MASW test results are presented in Plate 3 which presents the calculated shear wave velocity profiles measured from the field testing. These results have been inferred using a sledgehammer located at 5 m from the first geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Plate 4. There is a good correlation between the field collected and model calculated dispersion curves, with a root mean squared error of 1%. The shear wave velocity depth profile indicates a gradual increase in velocity at approximately 7 m from 785 m/s to approximately 2,400 m/s at 11 m.

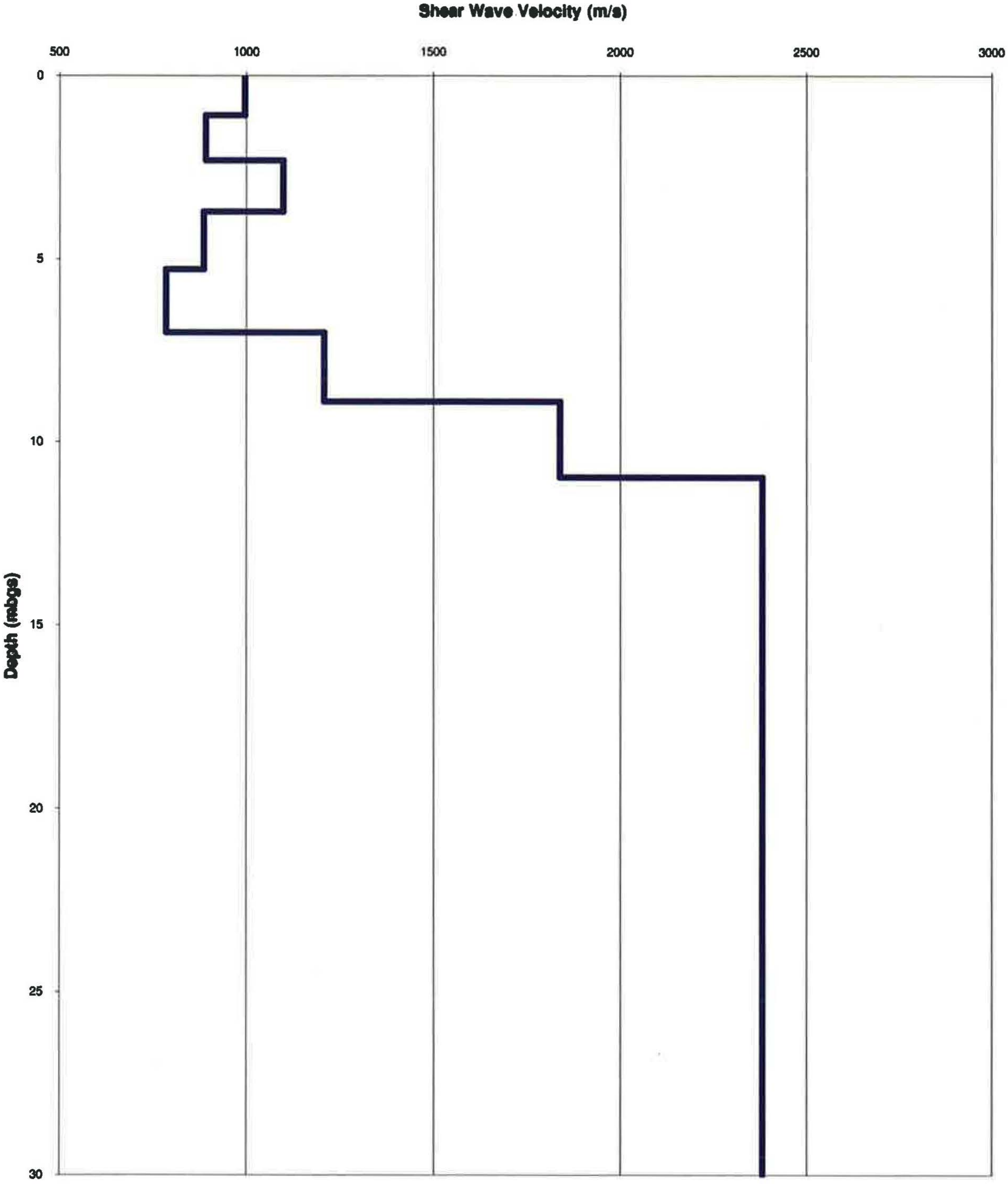


Plate 3: MASW Modelled Shear Wave Velocity Depth profile

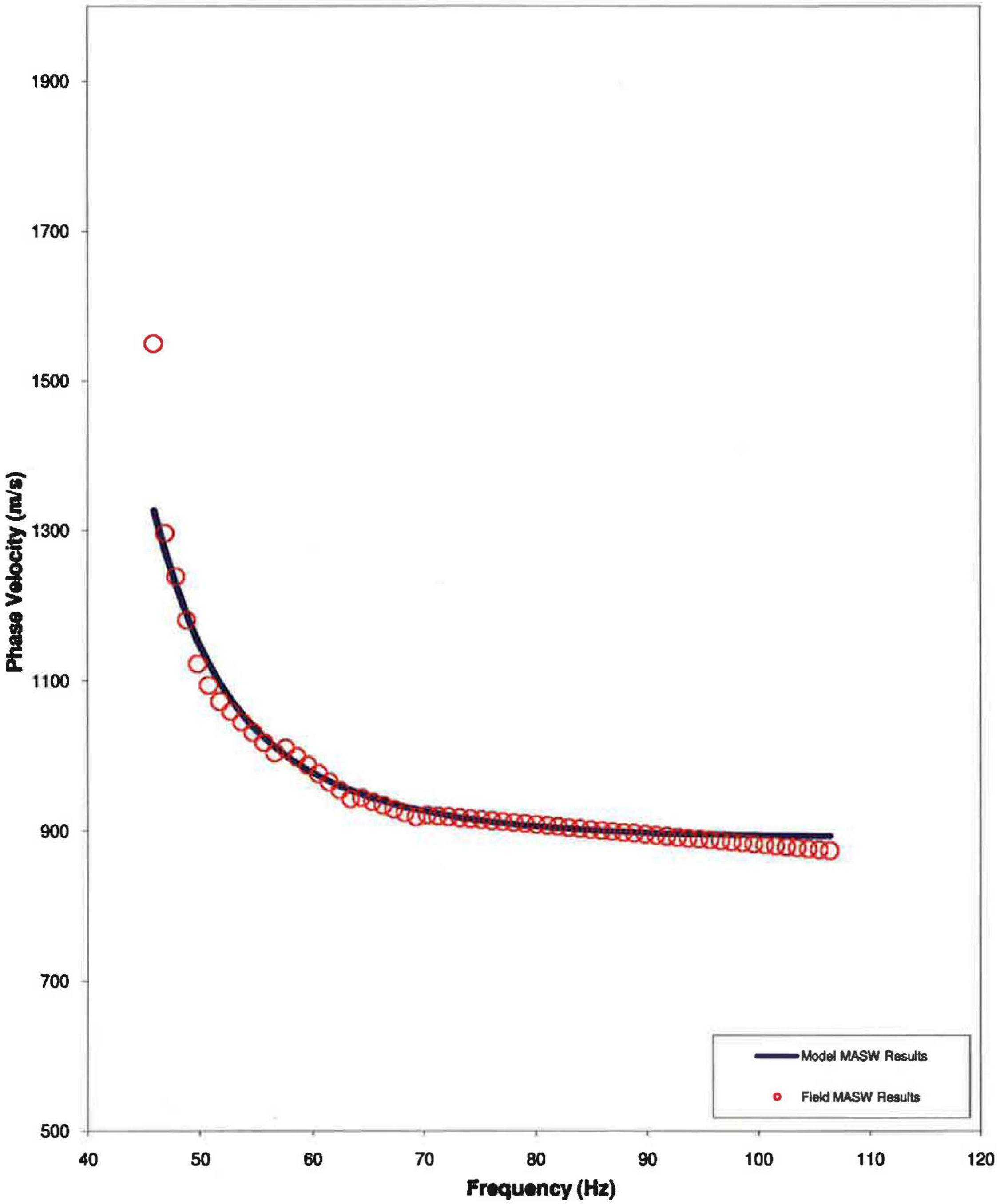


Plate 4: Comparison of Field vs. Modelled Data for the MASW Line

To calculate the average shear wave velocity as required by the National Building Code of Canada, 2005 (NBCC2005), the results were modelled to 30 mbgs. The limited low frequency content of the dispersion curve did not allow us to sufficiently resolve shear-wave velocities at depth below 11 m. The average shear wave velocity was calculated assuming that the velocity from the maximum resolved depth (approximately 11 m) to a depth of 30 m was constant and equal to the velocity of the bedrock. The average shear-wave velocity was found to be 1630 m/s (Table 2).

Table 2: Shear Wave Velocity Profile

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.07	1.07	996	0.001075
1.07	2.31	1.24	892	0.001387
2.31	3.71	1.40	1098	0.001276
3.71	5.27	1.57	886	0.001767
5.27	7.01	1.73	785	0.002204
7.01	8.90	1.90	1208	0.001570
8.90	10.96	2.06	1838	0.001121
10.96	30.00	19.04	2380	0.008000
Vs Average to 30 mbgs (m/s)				**1630**[1]

[1] This value should be revised if foundations are located below the ground surface.

Limitations and Use of This Report

The geophysical interpretation presented in this technical memorandum is based on the interpretation of geophysical data and accompanying geotechnical findings. As with any geophysical method, interpretation presented in this report should be confirmed by intrusive methods (boreholes, test pits, etc.). Assumptions made in the geophysical interpretation have been stated, where applicable, throughout the technical memorandum.

This geophysical survey was carried out in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practising under similar conditions, subject to the time limits and financial and physical constraints applicable to the services provided. This technical memorandum provides a professional opinion and therefore no warranty is either expressed, implied, or made as to the conclusions, advice and recommendations offered in this report.

Any use which a third party makes of this technical memorandum, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this report.

Closure

We trust that this report meets your current needs. If you have any questions or require clarification, please contact the undersigned.

Yours Very Truly

GOLDER ASSOCIATES LTD.

PROFESSIONAL GEOSCIENTISTS ONTARIO
CHRISTOPHER R. PHILLIPS
PRACTISING MEMBER
1503

Stephane Sol, Ph.D.
Geophysics Group

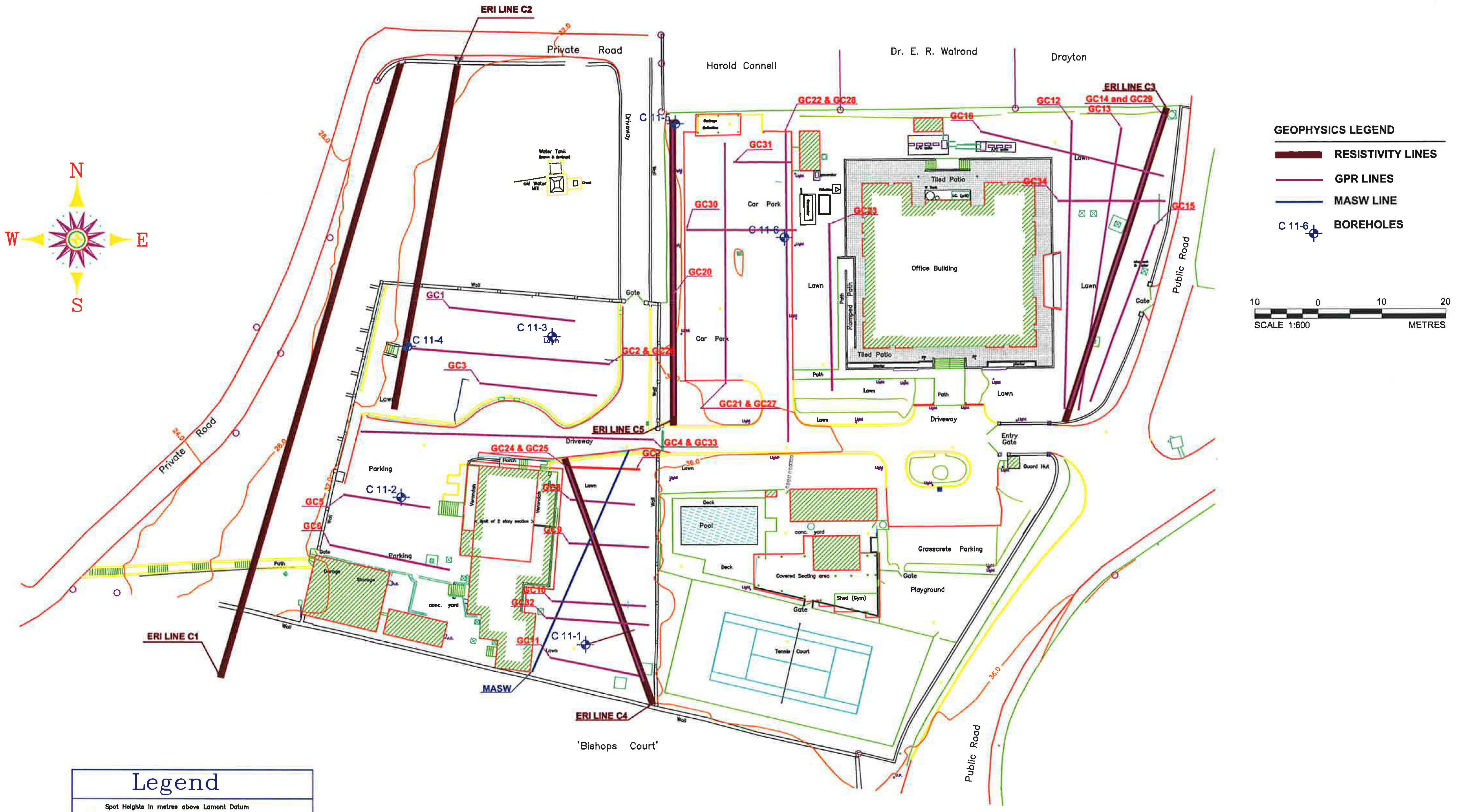
Christopher Phillips, M.Sc., P.Geo. (Ontario)
Senior Geophysicist, Associate

Attachments: Figures 1 to 10
Appendix A – Figures A1 to A17

SS/CRP/cg/wlm

n:\active\2010\other offices\10-1121-0089 barbados-embassy\reporting\chancery\final\april 11 2011\10-1121-0089 tm chancery geophysical results final 11apr11.docx

FIGURES



Legend

Spot Heights in metres above Lamont Datum

▲	Control Station	▨	Building (conc.)
	Tree	═	Wall
	Golden Palm (cluster)	━	Kerb
	Other Palm	—	Road edge
		—	Path edge
—	Edge of Garden Bed	—	Fence/shed
⊠ ◯	Inspection Cover/Well	‖‖‖‖‖	Steps (conc.)
═ ■	Conc. Gutter / Grill	—	Roof overhang
—	Electrical Utilities	—	Conc. overhang
—	Water Pipe	○	Boundary corner from plan
U.P. ○	Utility Pole	◯	Boundary Mark found

Boundary information taken from a plan by D.G. Godson Certified 1983–11–14

NOTES

1. This Figure is to be analyzed in conjunction with the accompanying report.
2. Base plan used in the figure provided by J.L. Richards and Associates Ltd.

Golder Associates
Mississauga, Ontario, Canada

SCALE	1:600
DATE	MARCH 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	15

FILE No.
PROJECT No. 10-1121-0089
REV.

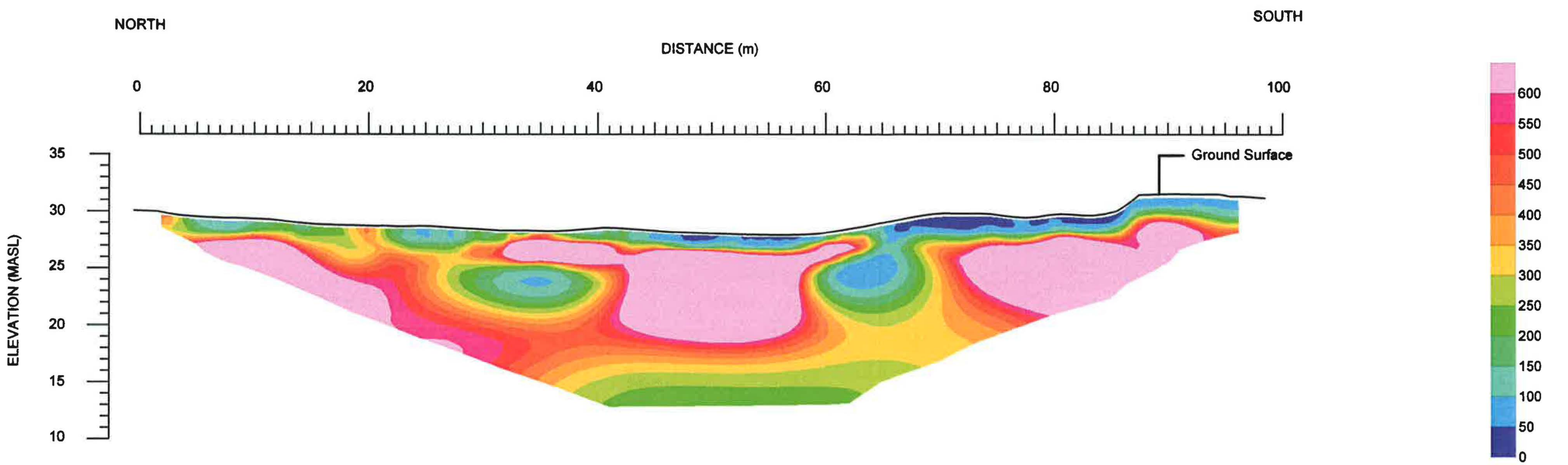
TITLE

GEOPHYSICAL SITE MAP
CANADIAN HIGH COMMISSION

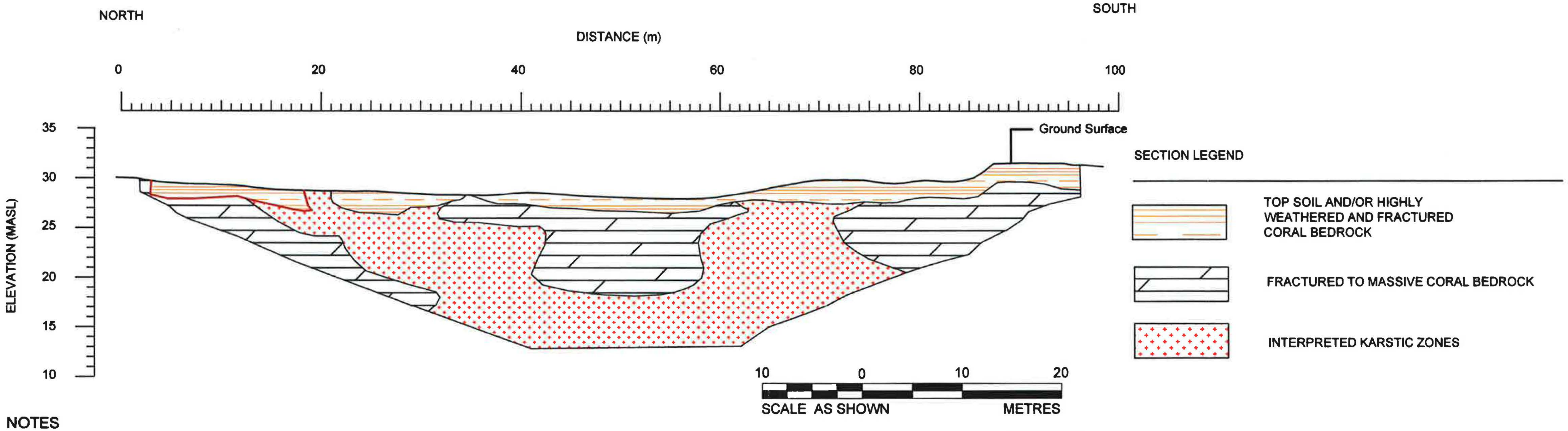
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE
1

ERI LINE C1 - SURVEY RESULTS



ERI LINE C1 - INTERPRETATION RESULTS



NOTES

1. This Figure is to be analyzed in conjunction with the accompanying report.
2. ERI section generated using the Res2D Software Package.
3. Elevations of the ERI lines presented based on topographic data provided by J.L. Richards and Associates Ltd.
4. Boreholes logs provided by Golder Associates Ltd.

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089
REV.

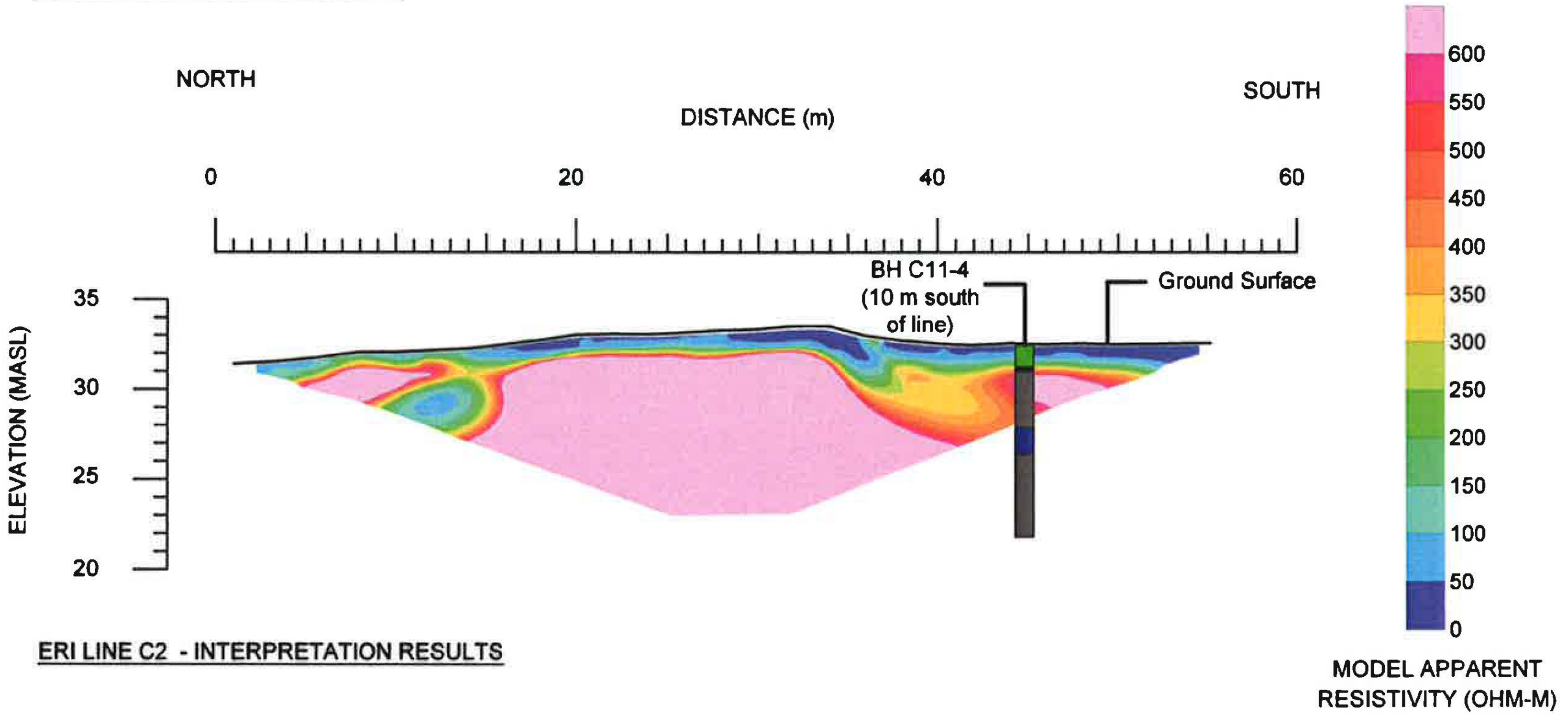
SCALE AS SHOWN
DATE MARCH 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASP

TITLE
ERI LINE C1 INTERPRETED SURVEY RESULTS

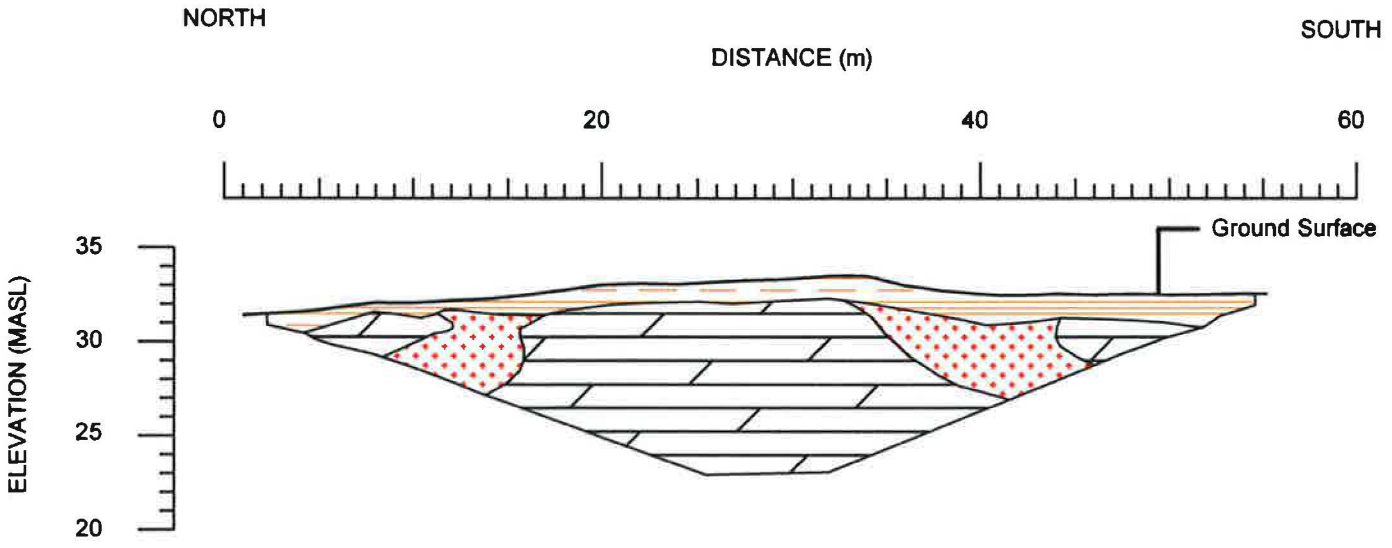
J.L. RICHARDS AND ASSOCIATES LTD.

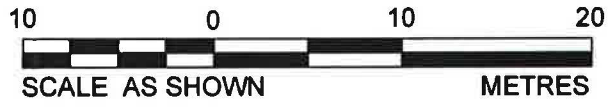
FIGURE 2

ERI LINE C2 - SURVEY RESULTS



ERI LINE C2 - INTERPRETATION RESULTS





SECTION LEGEND

▤	TOP SOIL AND/OR HIGHLY WEATHERED AND FRACTURED CORAL BEDROCK
▱	FRACTURED TO MASSIVE CORAL BEDROCK
▨	INTERPRETED KARSTIC ZONES

SIMPLIFIED BOREHOLE LEGEND

🟨	TOP SOIL
🟩	HIGHLY WEATHERED TO HIGHLY FRACTURED CORAL BEDROCK
⬛	VOID
▩	FRACTURED TO MASSIVE CORAL BEDROCK WITH VOIDS
🟦	NO CORE RECOVERY
🟥	FILL: BROWN SAND WITH GRAVEL CORAL

NOTES

1. This Figure is to be analyzed in conjunction with the accompanying report.
2. ERI section generated using the Res2D Software Package.
3. Elevations of the ERI lines presented based on topographic data provided by J.L. Richards and Associates Ltd.
4. Boreholes logs provided by Golder Associates Ltd.

Golder Associates
Mississauga, Ontario, Canada

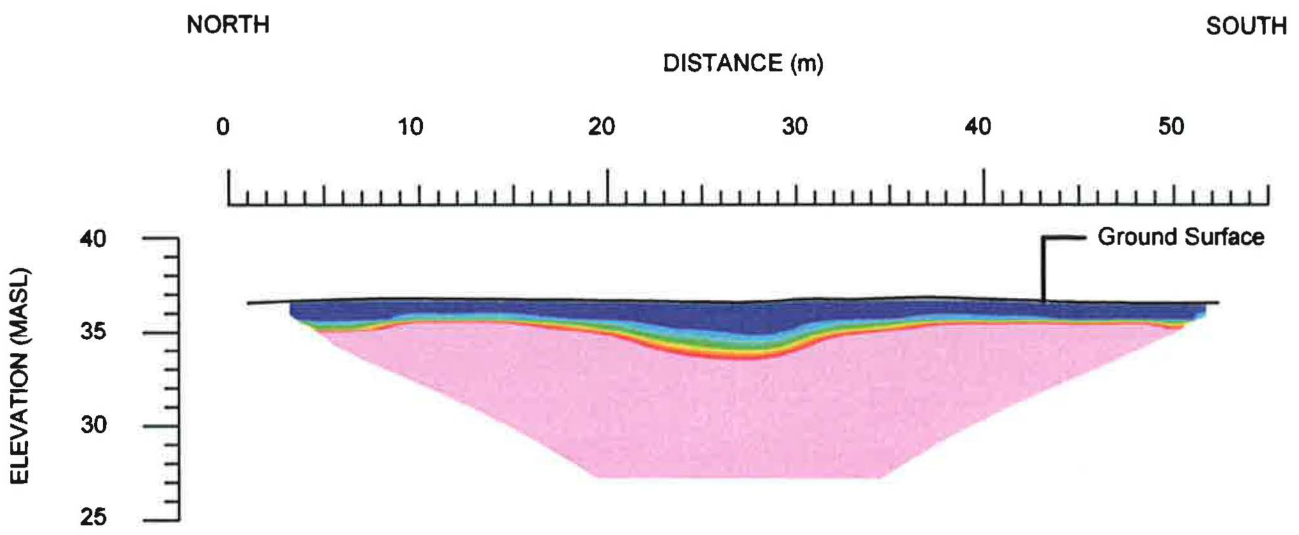
FILE No.	
PROJECT No.	10-1121-0089
REV.	
SCALE	AS SHOWN
DATE	MARCH 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	

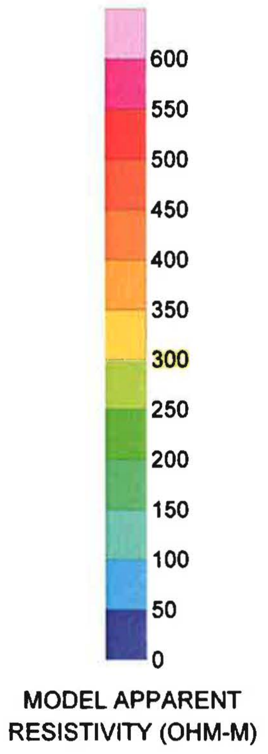
TITLE
ERI LINE C2
INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE
3

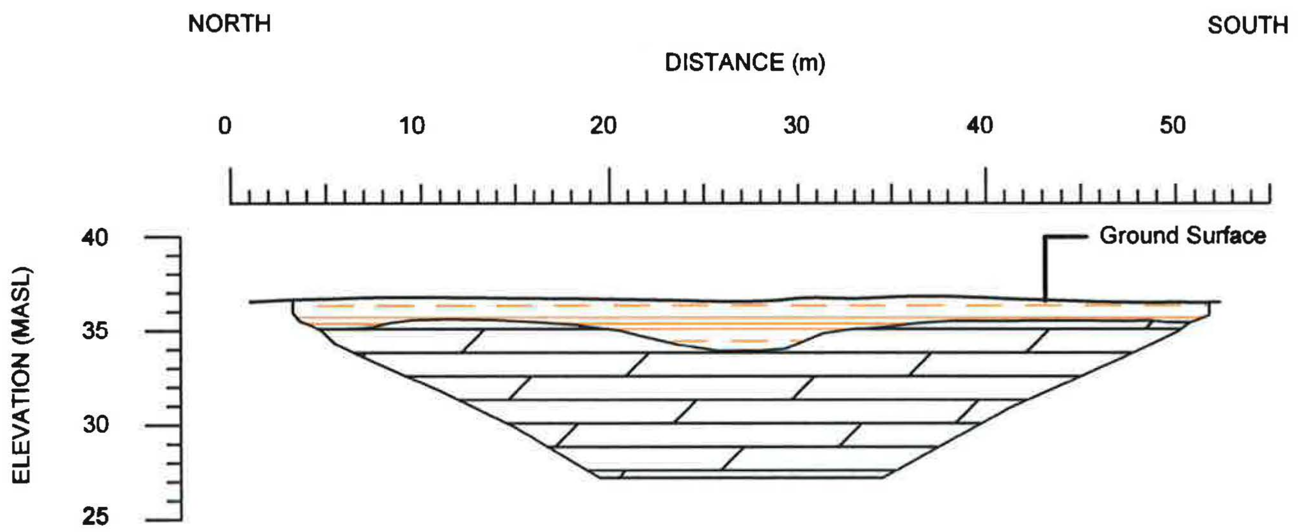
ERI LINE C3 - SURVEY RESULTS



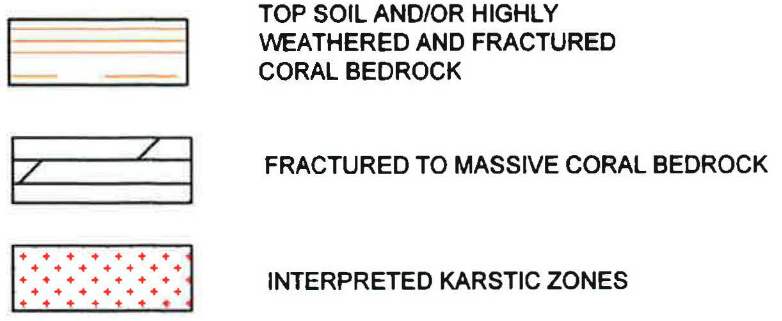


MODEL APPARENT RESISTIVITY (OHM-M)

ERI LINE C3 - INTERPRETATION RESULTS



SECTION LEGEND



NOTES

1. This Figure is to be analyzed in conjunction with the accompanying report.
2. ERI section generated using the Res2D Software Package.
3. Elevations of the ERI lines presented based on topographic data provided by J.L. Richards and Associates Ltd.
4. Boreholes logs provided by Golder Associates Ltd.

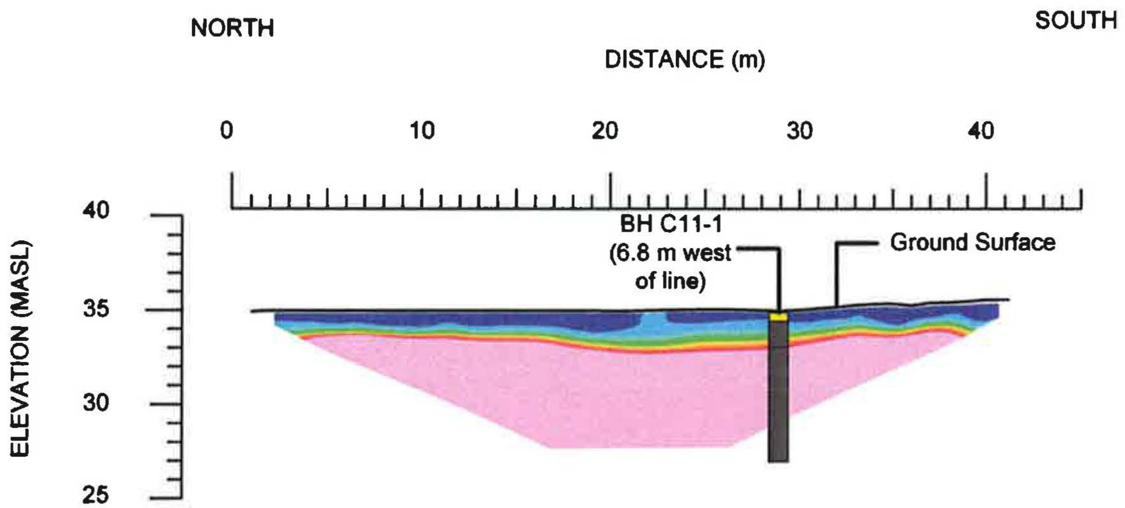
Golder Associates
Mississauga, Ontario, Canada

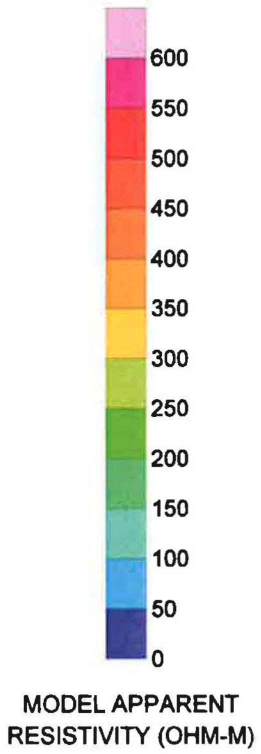
SCALE	AS SHOWN	TITLE	ERI LINE C3 INTERPRETED SURVEY RESULTS
DATE	DEC 2010		
DESIGN	SS		
CAD	SS		
FILE No.		CHECK	CRP
PROJECT No.	10-1121-0089	REV.	
REVIEW	AG		

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE 4

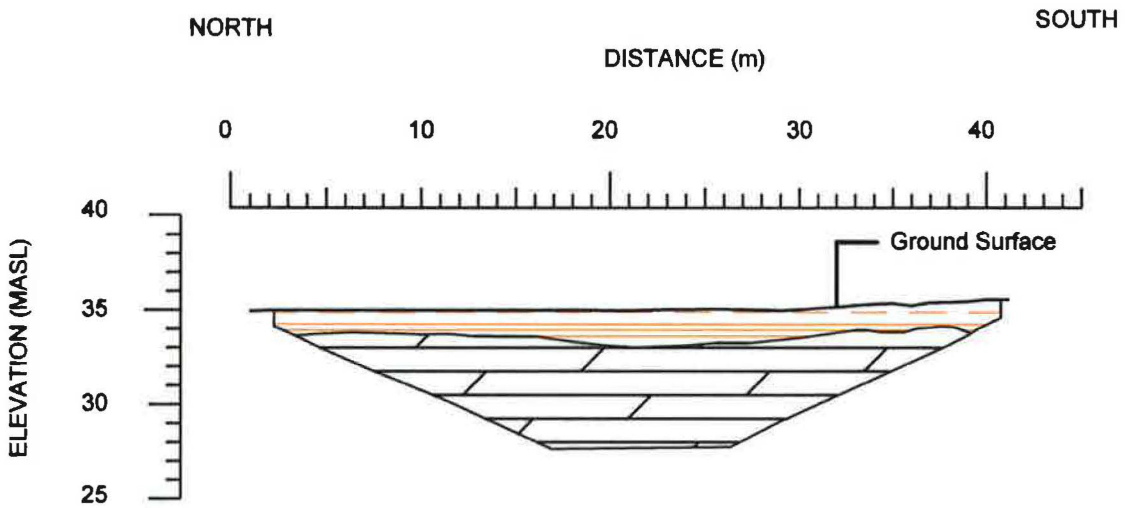
ERI LINE C4 - SURVEY RESULTS





MODEL APPARENT RESISTIVITY (OHM-M)

ERI LINE C4 - INTERPRETATION RESULTS



SECTION LEGEND

▤	TOP SOIL AND/OR HIGHLY WEATHERED AND FRACTURED CORAL BEDROCK
▱	FRACTURED TO MASSIVE CORAL BEDROCK
▨	INTERPRETED KARSTIC ZONES

SIMPLIFIED BOREHOLE LEGEND

🟨	TOP SOIL
🟩	HIGHLY WEATHERED TO HIGHLY FRACTURED CORAL BEDROCK
⬛	VOID
■	FRACTURED TO MASSIVE CORAL BEDROCK WITH VOIDS
🟦	NO CORE RECOVERY
🟥	FILL: BROWN SAND WITH GRAVEL CORAL

SCALE AS SHOWN METRES

NOTES

1. This Figure is to be analyzed in conjunction with the accompanying report.
2. ERI section generated using the Res2D Software Package.
3. Elevations of the ERI lines presented based on topographic data provided by J.L. Richards and Associates Ltd.
4. Boreholes logs provided by Golder Associates Ltd.

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089 REV.

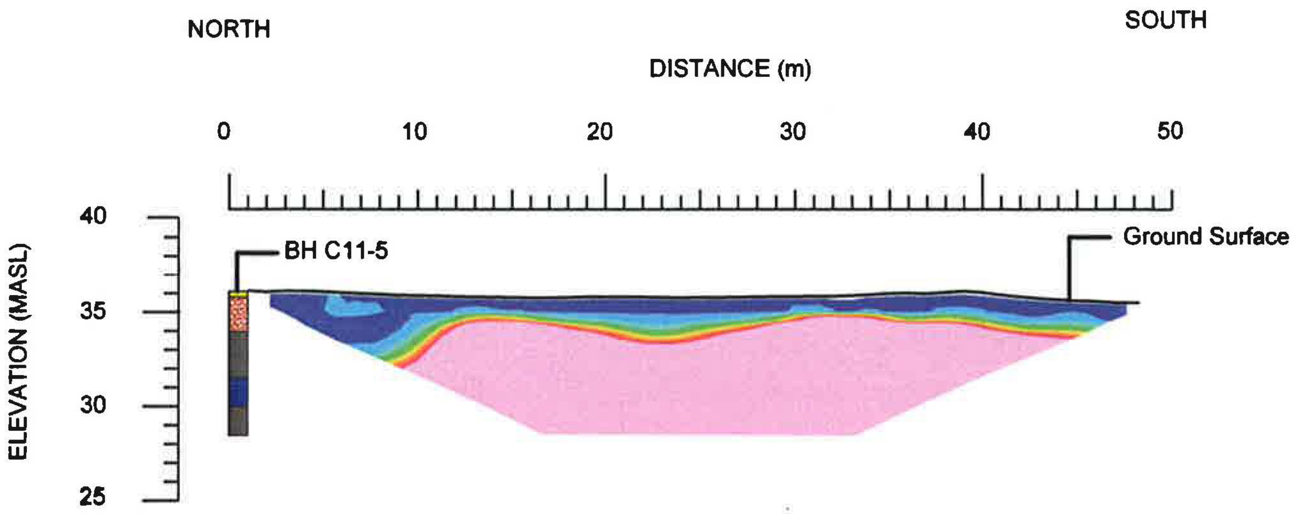
SCALE AS SHOWN
DATE MARCH 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASS

TITLE
ERI LINE C4 INTERPRETED SURVEY RESULTS

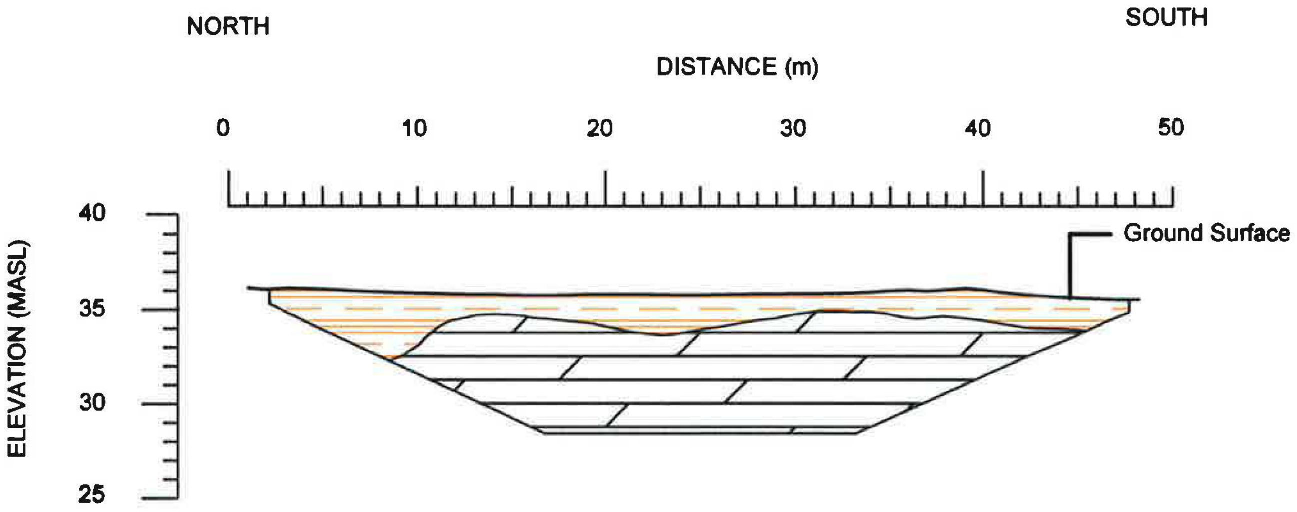
J.L. RICHARDS AND ASSOCIATES LTD.

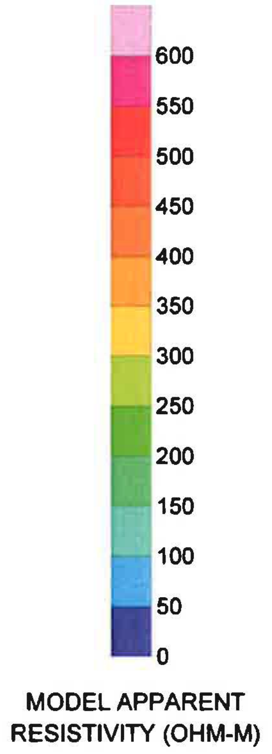
FIGURE 5

ERI LINE C5 - SURVEY RESULTS



ERI LINE C5 - INTERPRETATION RESULTS



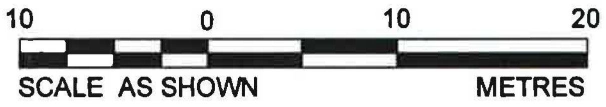


SECTION LEGEND

▤	TOP SOIL AND/OR HIGHLY WEATHERED AND FRACTURED CORAL BEDROCK
▱	FRACTURED TO MASSIVE CORAL BEDROCK
▨	INTERPRETED KARSTIC ZONES

SIMPLIFIED BOREHOLE LEGEND

🟨	TOP SOIL
🟩	HIGHLY WEATHERED TO HIGHLY FRACTURED CORAL BEDROCK
⬛	VOID
▩	FRACTURED TO MASSIVE CORAL BEDROCK WITH VOIDS
🟦	NO CORE RECOVERY
🟥	FILL: BROWN SAND WITH GRAVEL CORAL



NOTES

1. This Figure is to be analyzed in conjunction with the accompanying report.
2. ERI section generated using the Res2D Software Package.
3. Elevations of the ERI lines presented based on topographic data provided by J.L. Richards and Associates Ltd.
4. Boreholes logs provided by Golder Associates Ltd.

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089
REV.

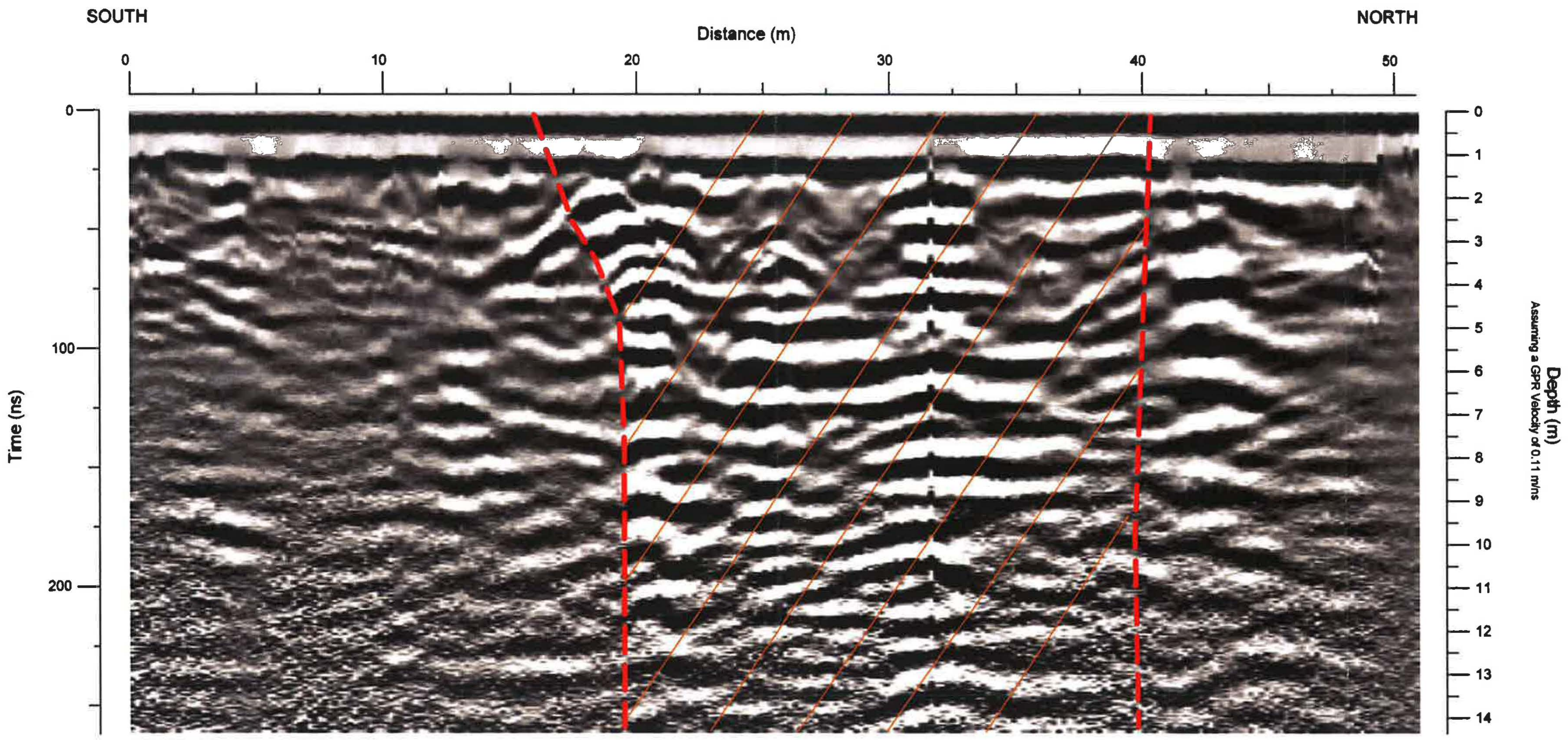
SCALE AS SHOWN
DATE MARCH 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW

TITLE
ERI LINE C5
INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE 6

GPR LINE GC28 - 100 MHz



Legend

🟥 / 🟧	Interpreted Karstic Zone
▬	Interpreted Air Wave

Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN
DATE	MARCH 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	18

FILE No.
PROJECT No. 10-1121-0089 REV.

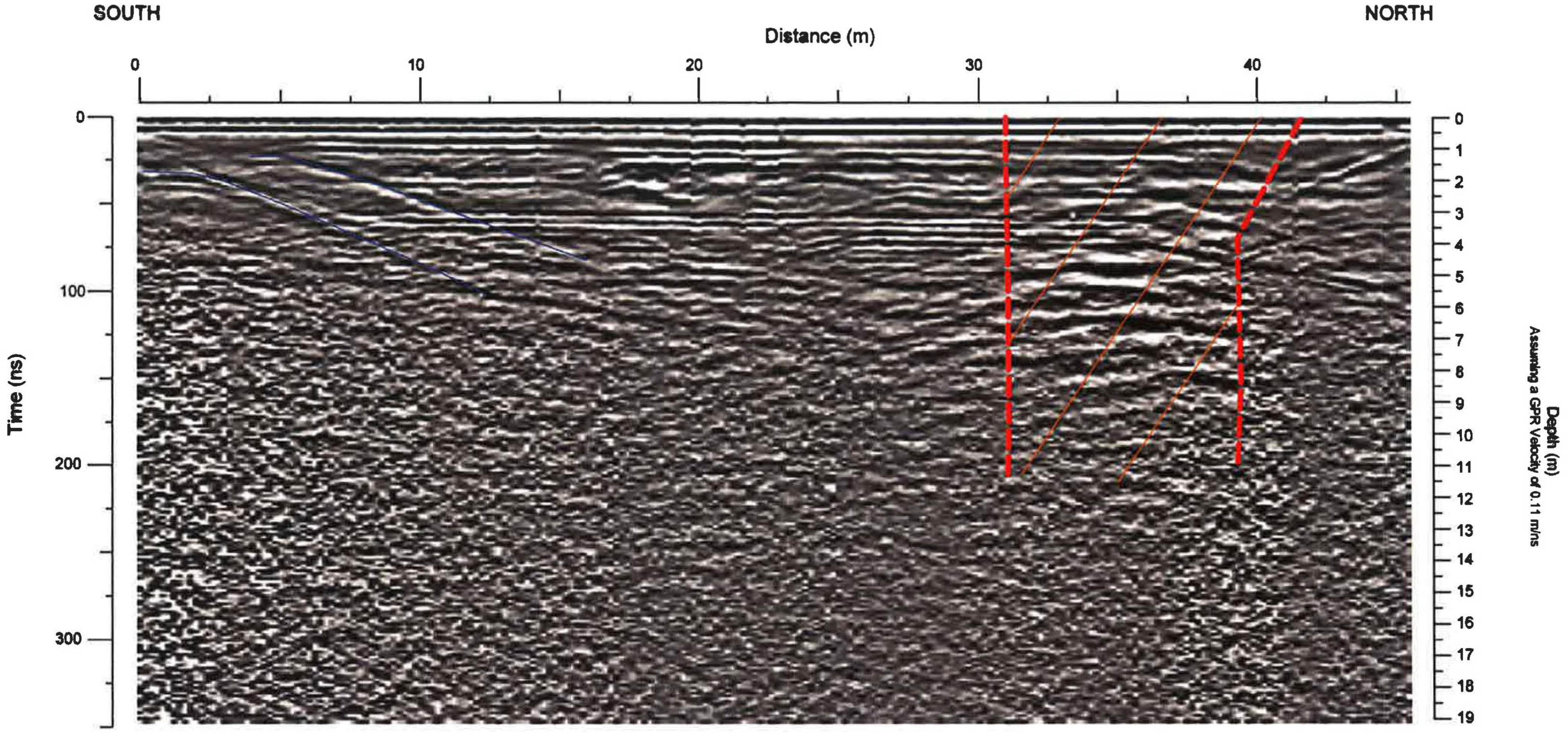
TITLE

GPR LINE GC28 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE 7

GPR LINE GC12 - 50 MHz



Legend

Interpreted Karstic Zone

Interpreted Air Wave

Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE AS SHOWN
DATE MARCH 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW JS

FILE No.
PROJECT No. 10-1121-0089
REV.

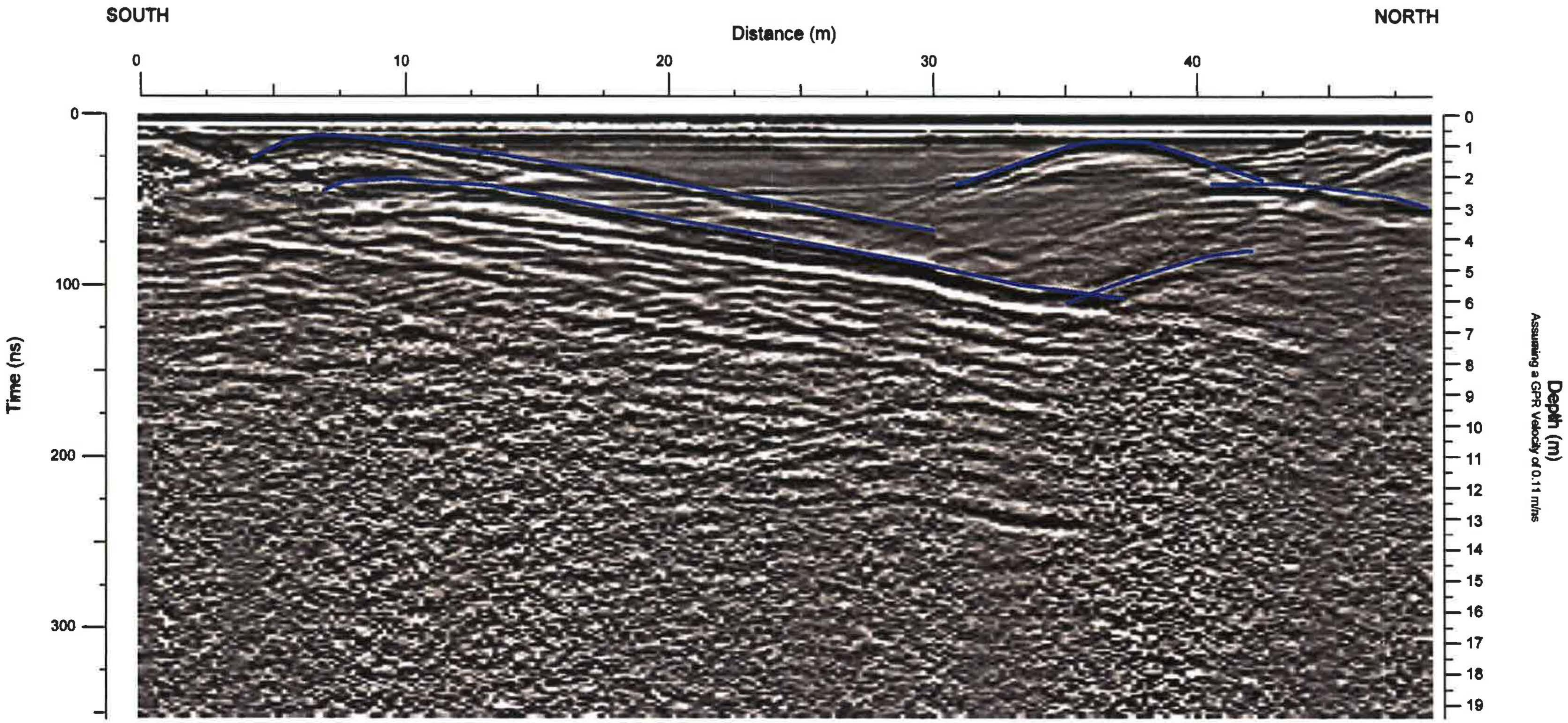
TITLE

GPR LINE GC12 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE 8

GPR LINE GC14 - 50 MHz



Legend

╎/ /╎	Interpreted Karstic Zone
―	Interpreted Air Wave

Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

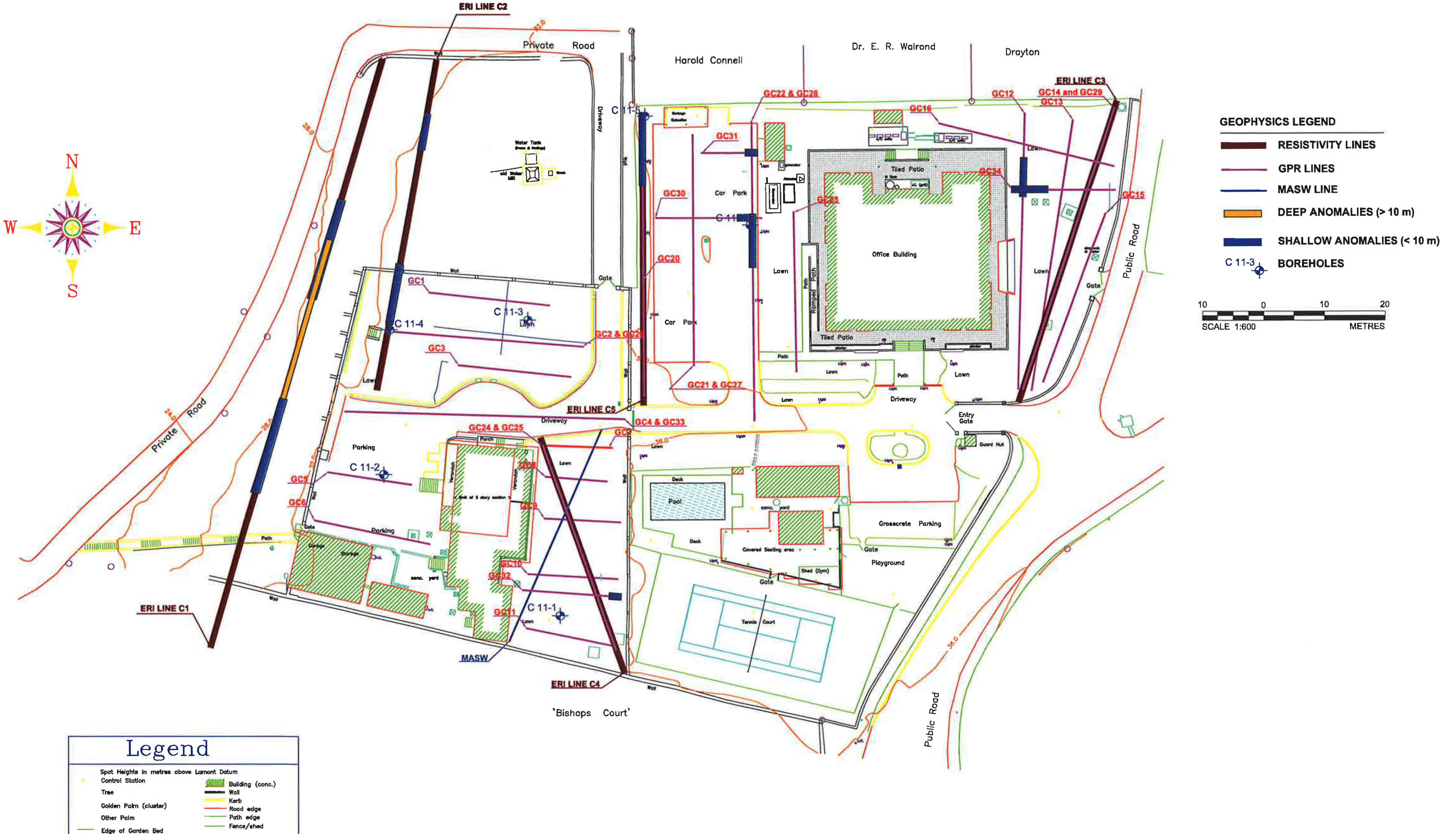
SCALE	AS SHOWN
DATE	DEC 2010
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	
FILE No.	
PROJECT No.	10-1121-0089
REV.	

TITLE

GPR LINE GC14 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE 9



Legend

Spot Heights in metres above Lamont Datum

Control Station	▨ Building (conc.)
Tree	Wall
Golden Palm (cluster)	Kerb
Other Palm	Road edge
Edge of Garden Bed	Path edge
⊠ Inspection Cover/Well	Fence/shed
Conc. Gutter / Grill	Steps (conc.)
Electrical Utilities	Roof overhang
Water Pipe	Conc. overhang
U.P. ○ Utility Pole	○ Boundary corner from plan
	○ Boundary Mark found

Boundary information taken from a plan by D.G. Godson Certified 1963–11–14

NOTES

1. This Figure is to be analyzed in conjunction with the accompanying report.
2. Base plan used in the figure provided by J.L. Richards and Associates Ltd.

Golder Associates
Mississauga, Ontario, Canada

FILE No.	
PROJECT No.	10-1121-0089
REV.	
SCALE	1:600
DATE	MARCH 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	155

TITLE

GEOPHYSICAL ANOMALY MAP - CANADIAN HIGH COMMISSION

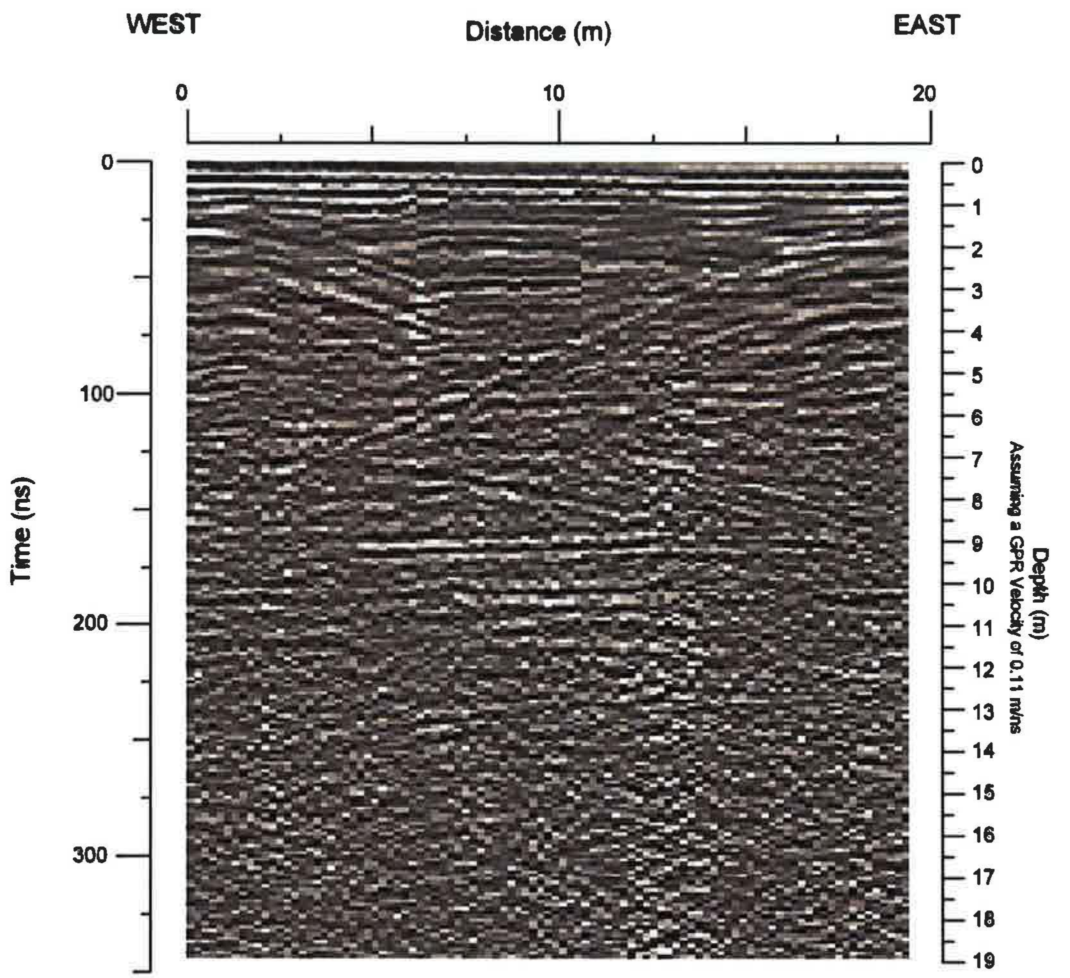
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE
10

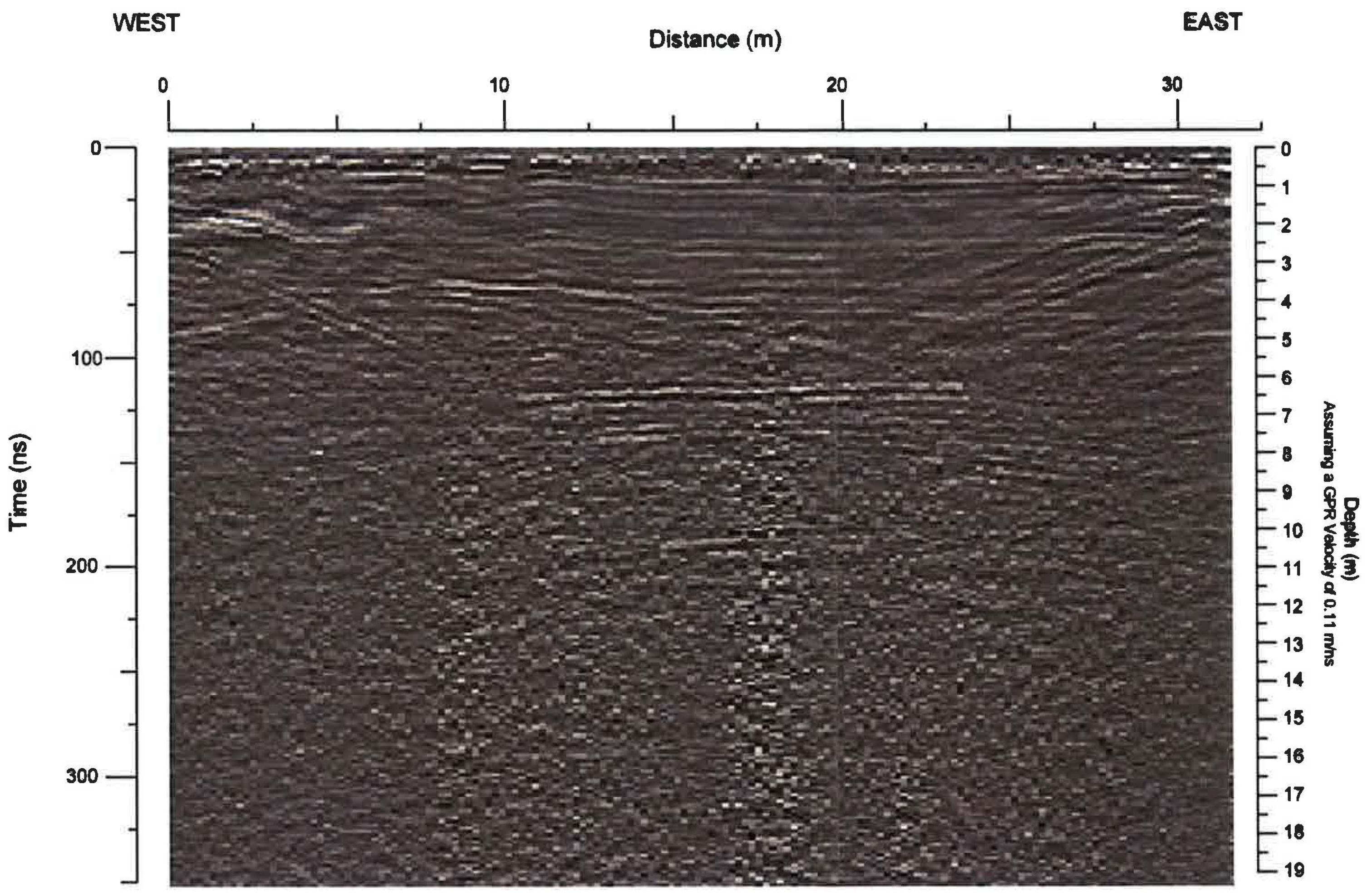
APPENDIX A

Figures A1 to A17

GPR LINE GC1 - 50 MHz



GPR LINE GC2 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN
DATE	APRIL 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	

FILE No.
PROJECT No. 10-1121-0089 REV.

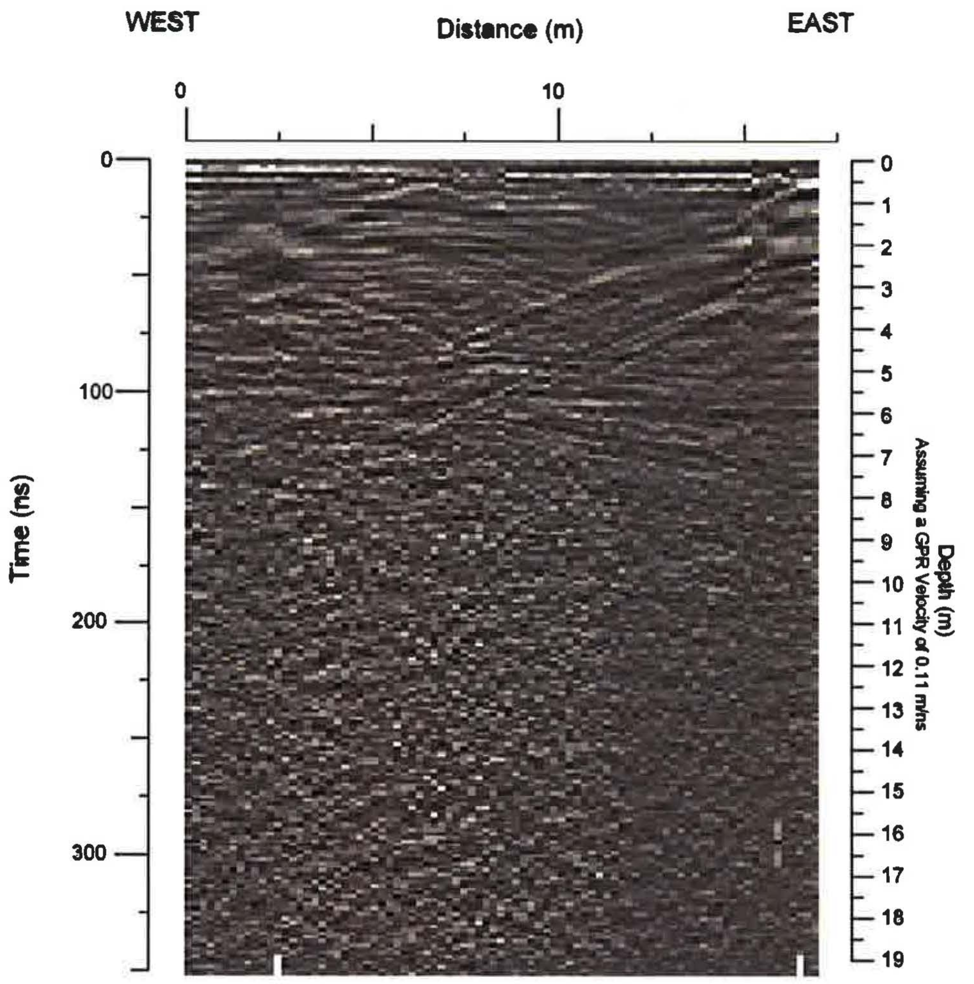
TITLE

GPR LINES GC1 and GC2 INTERPRETED SURVEY RESULTS

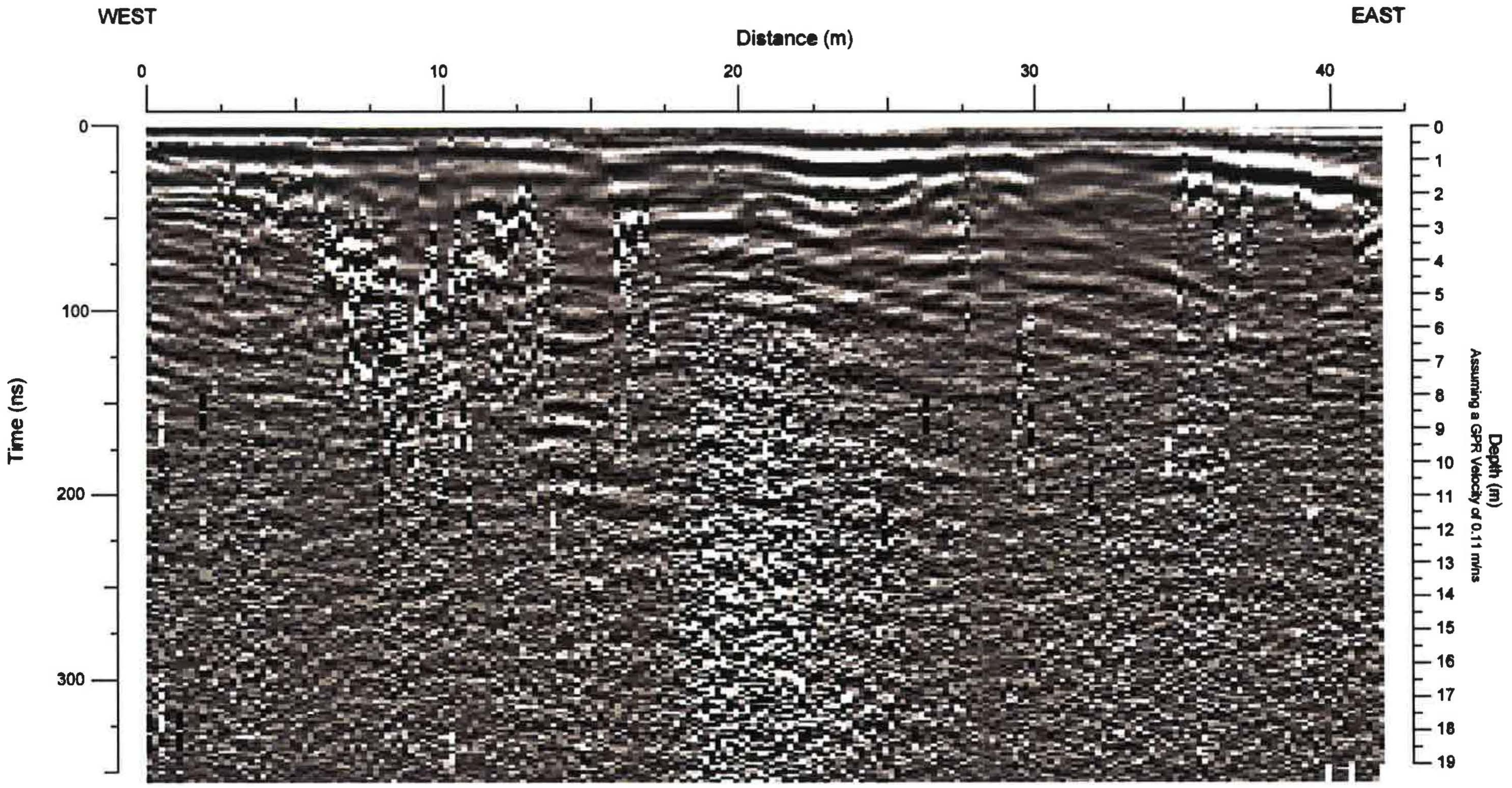
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE A1

GPR LINE GC3 - 50 MHz



GPR LINE GC4 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN
DATE	APRIL 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	155

FILE No.
PROJECT No. 10-1121-0089 REV.

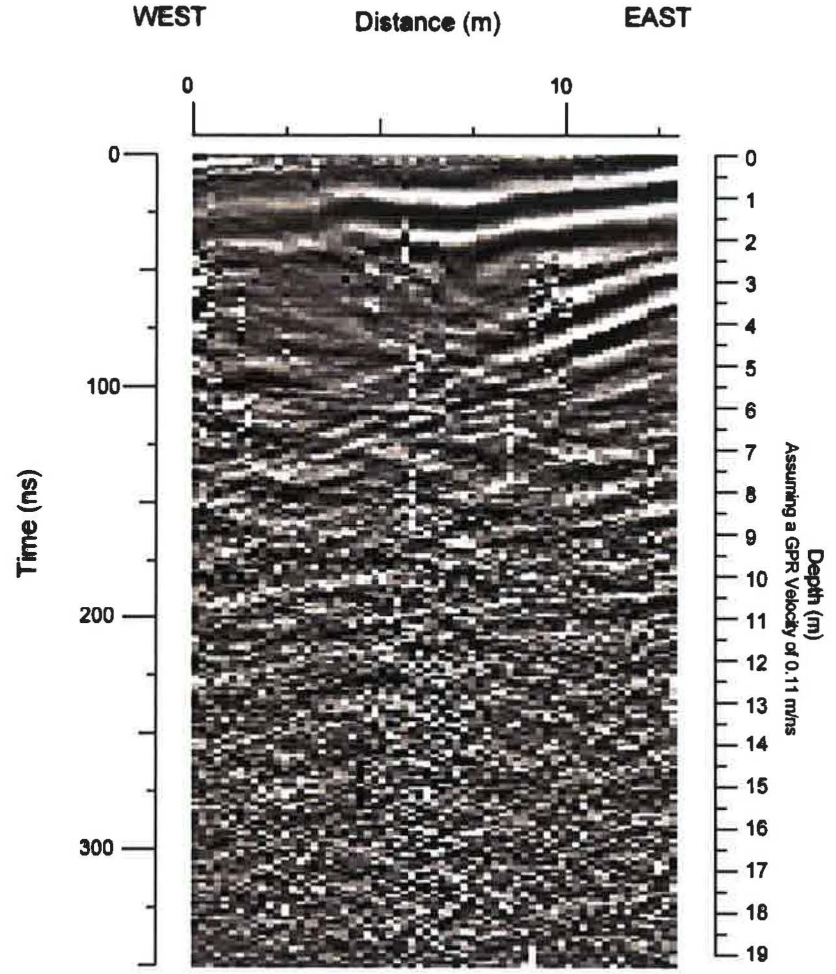
TITLE

GPR LINES GC3 and GC4 INTERPRETED SURVEY RESULTS

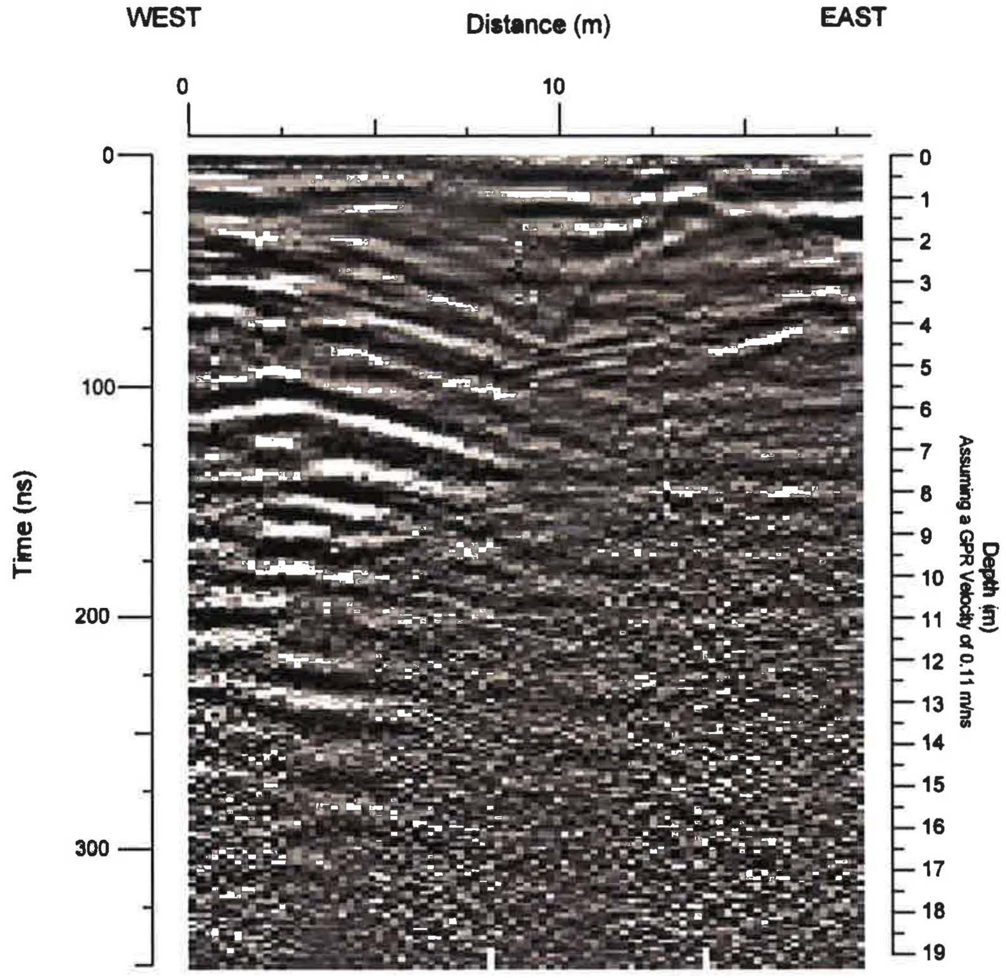
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE A2

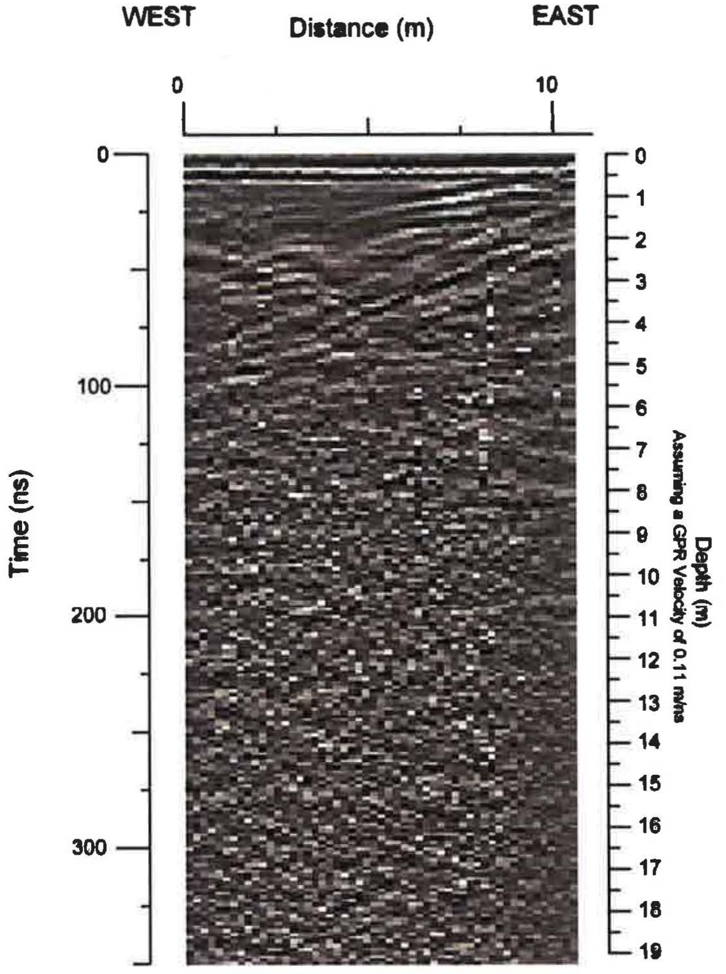
GPR LINE GC5 - 50 MHz



GPR LINE GC6 - 50 MHz



GPR LINE GC7 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089
REV.

SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASS

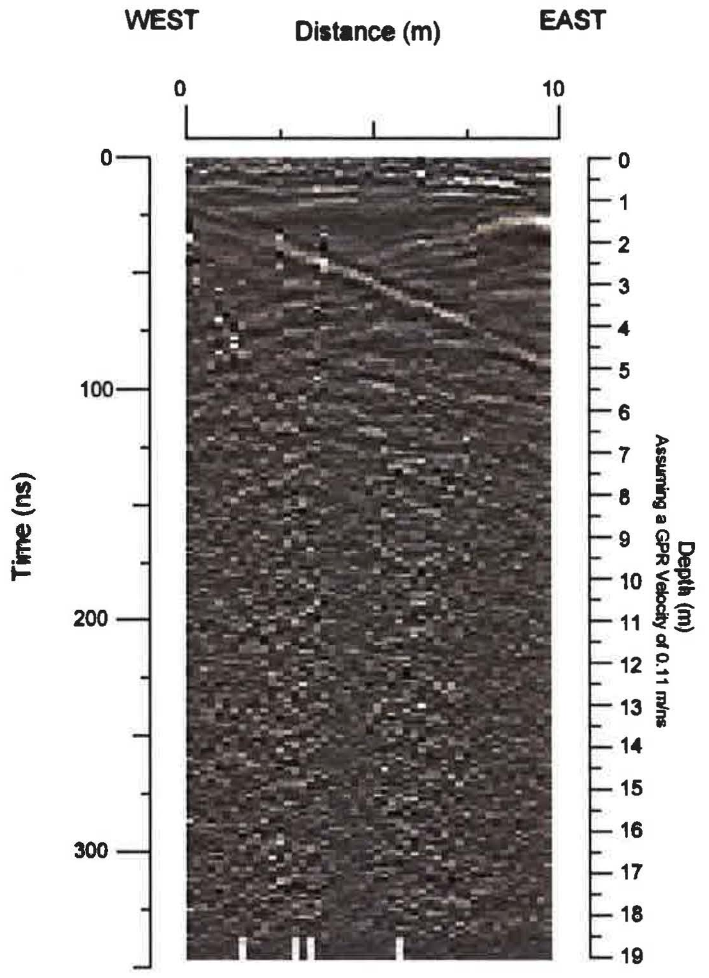
TITLE

GPR LINES GC5, GC6 and GC7 INTERPRETED SURVEY RESULTS

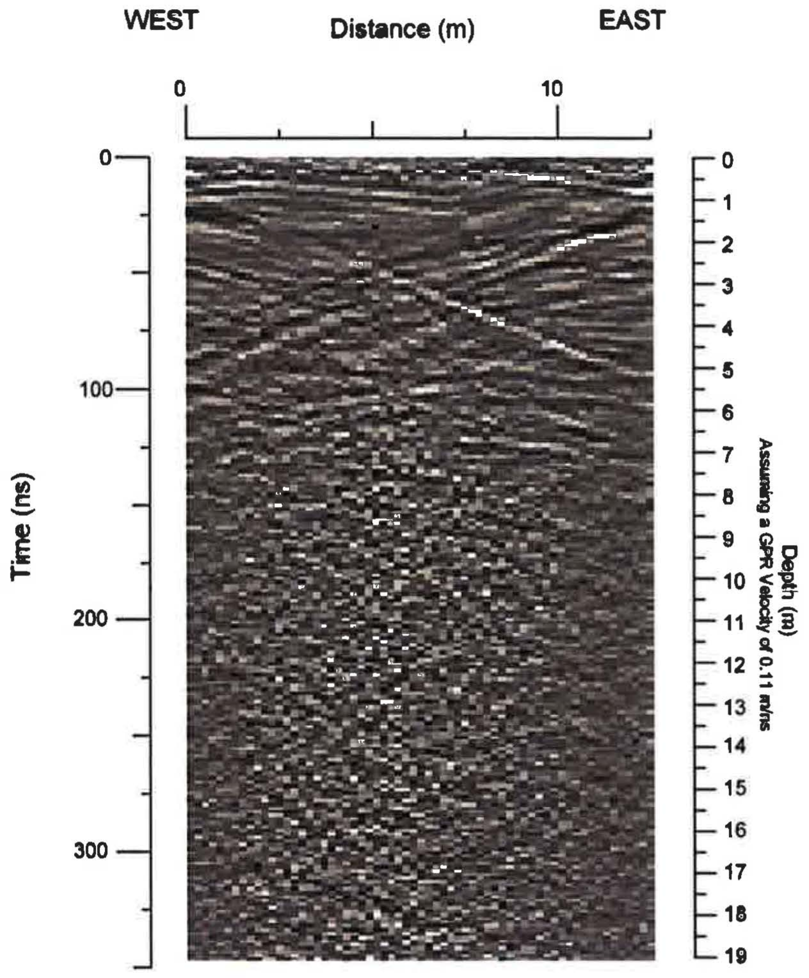
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE A3

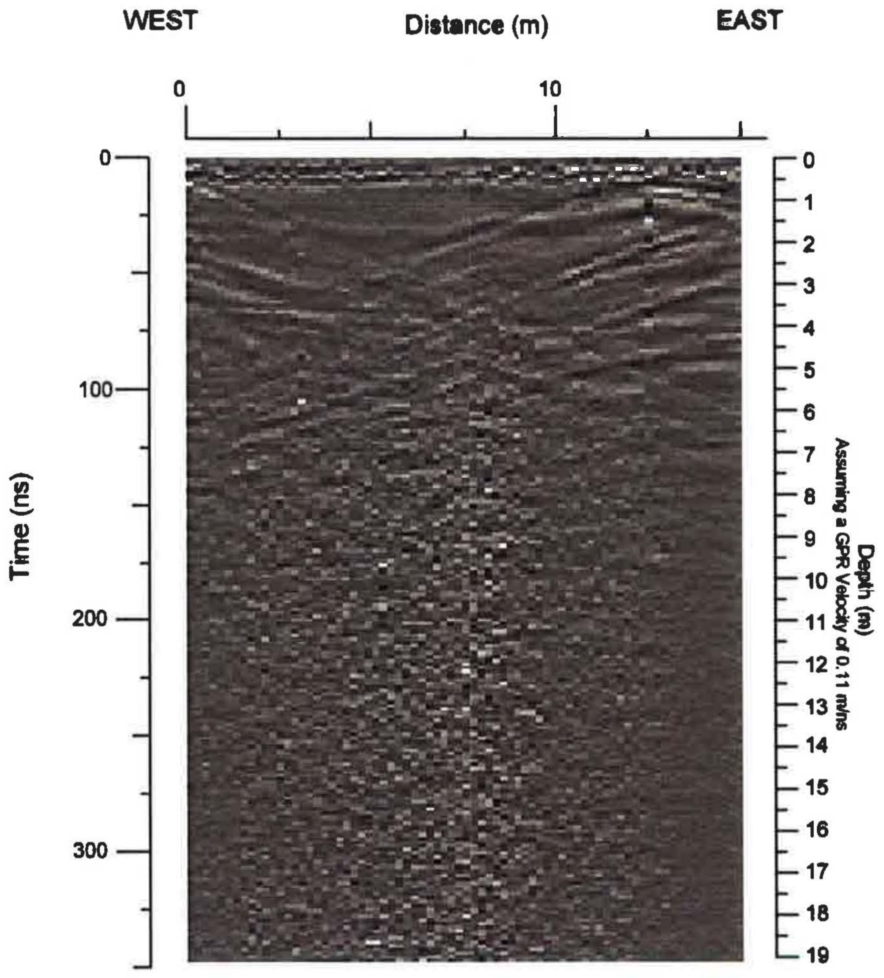
GPR LINE GC8 - 50 MHz



GPR LINE GC9 - 50 MHz



GPR LINE GC10 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW
FILE No.
PROJECT No. 10-1121-0089
REV.

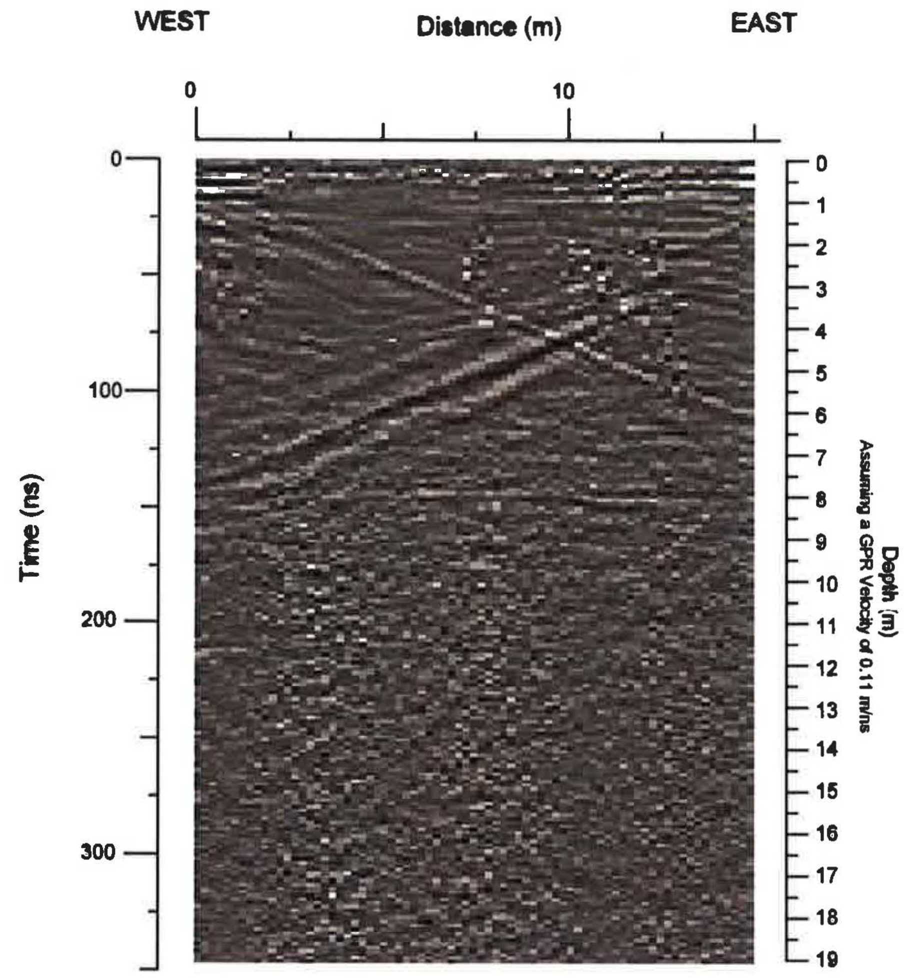
TITLE

GPR LINES GC8, GC9 and GC10 INTERPRETED SURVEY RESULTS

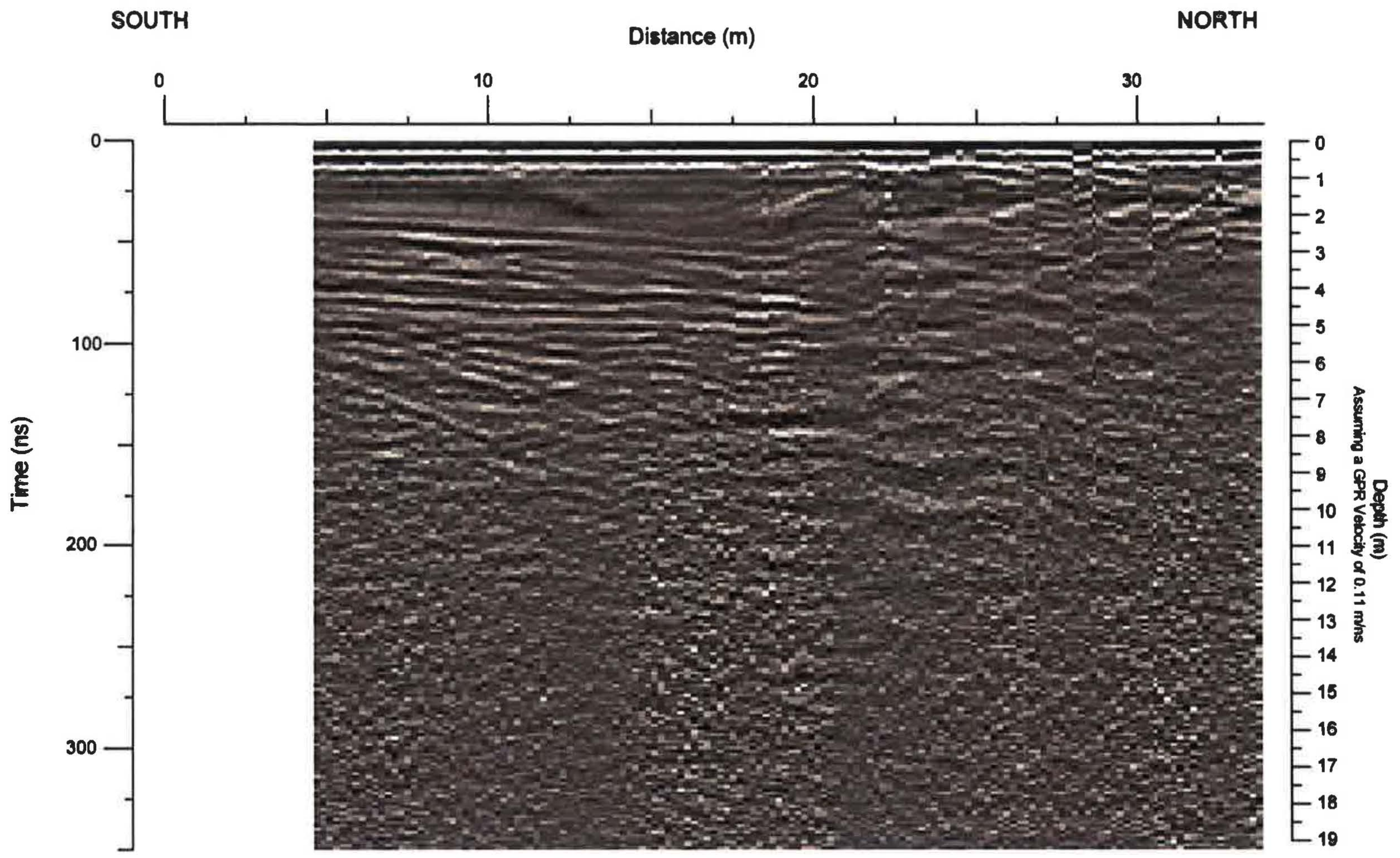
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE A4

GPR LINE GC11 - 50 MHz



GPR LINE GC13 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089
REV.

SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASS

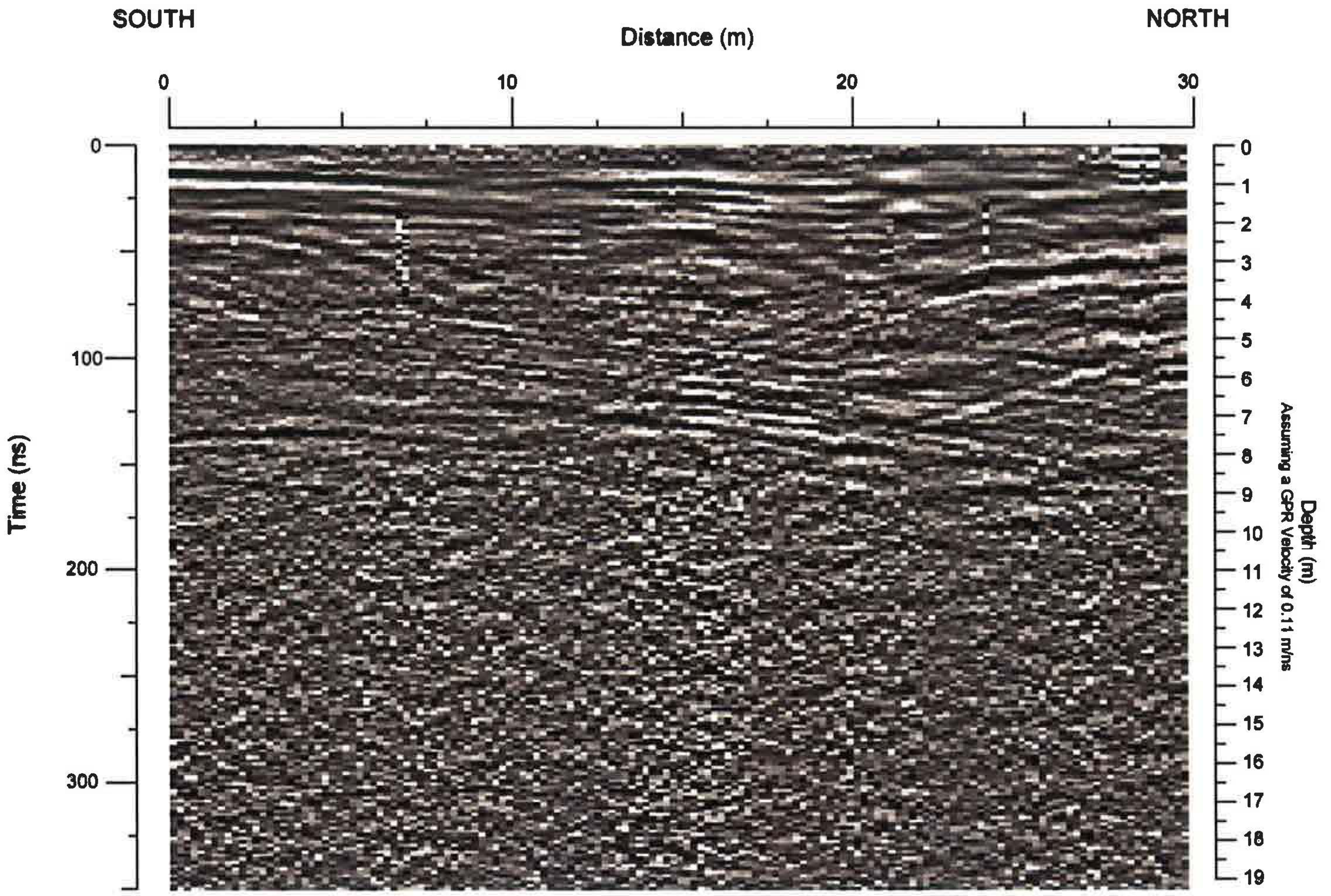
TITLE

GPR LINES GC11 and GC13 INTERPRETED SURVEY RESULTS

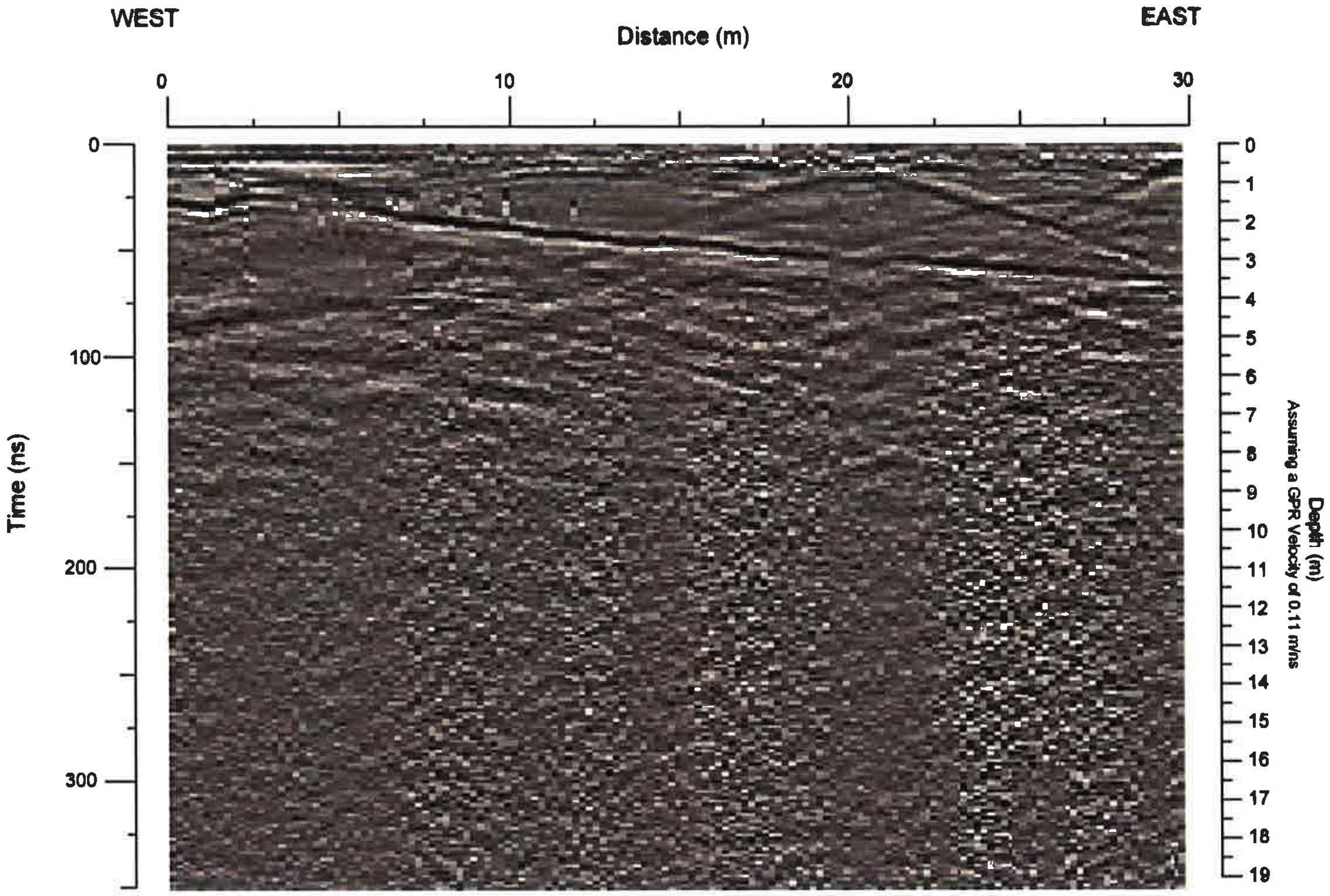
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE A5

GPR LINE GC15 - 50 MHz



GPR LINE GC16 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089
REV.

SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW 1SS

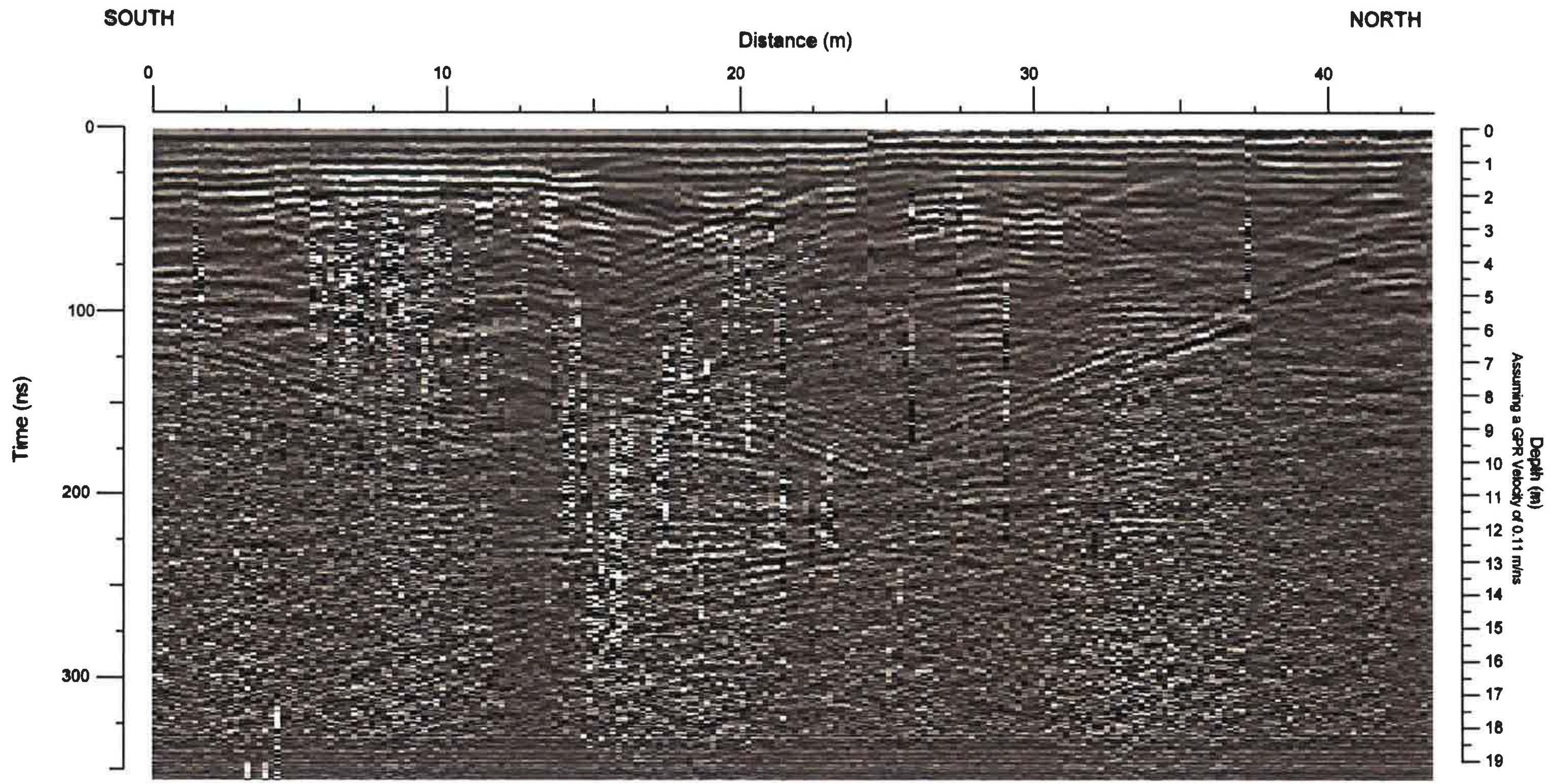
TITLE

GPR LINES GC15 and GC16 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE **A6**

GPR LINE GC20 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089
REV.

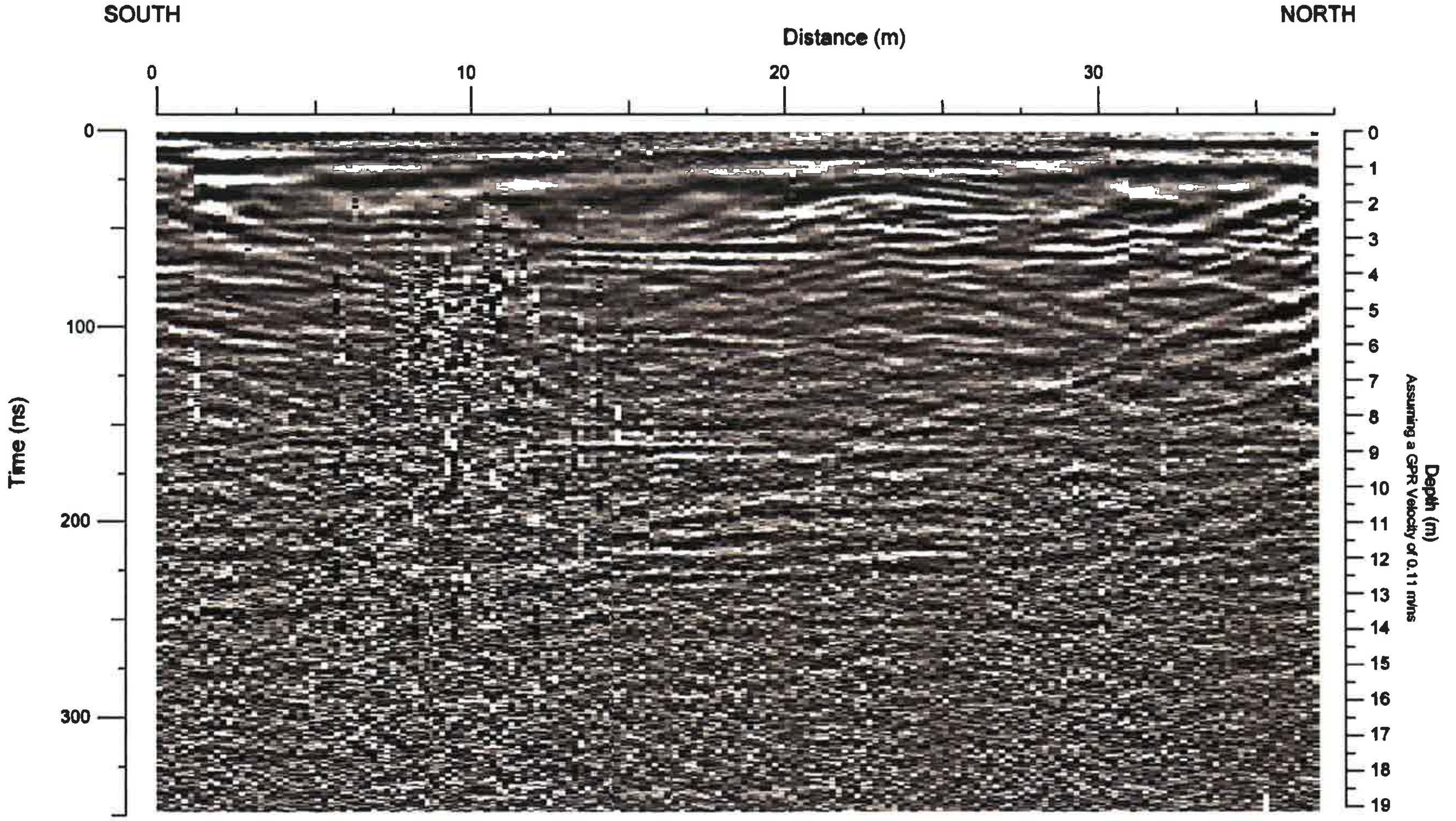
SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASS

TITLE
GPR LINE GC20 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE A7

GPR LINE GC21 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN
DATE	APRIL 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	ASS

FILE No.

PROJECT No. 10-1121-0089 REV.

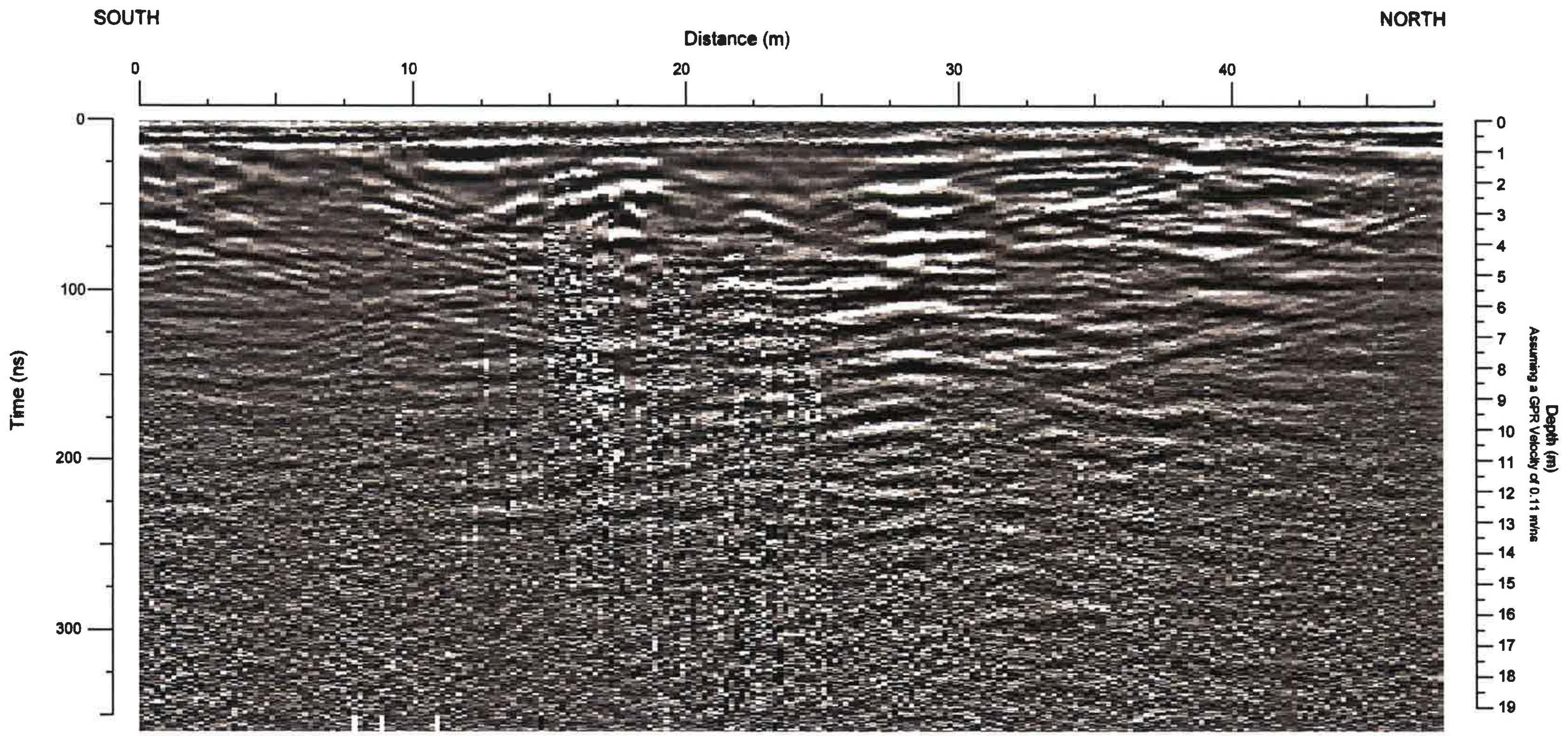
TITLE

GPR LINE GC21 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE A8

GPR LINE GC22 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

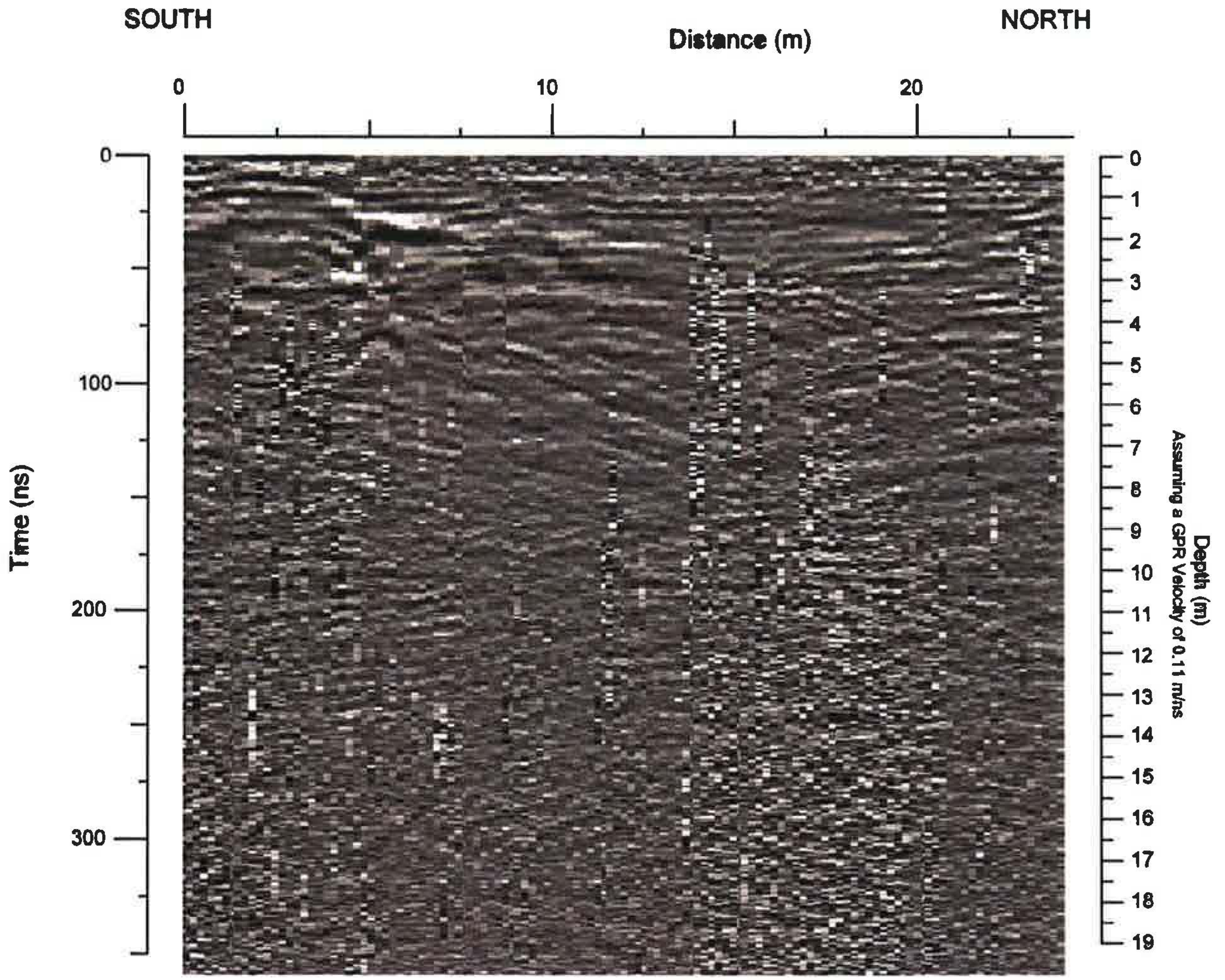
Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN	TITLE	GPR LINE GC22 INTERPRETED SURVEY RESULTS
DATE	APRIL 2011		
DESIGN	SS		
CAD	SS		
CHECK	CRP		J.L. RICHARDS AND ASSOCIATES LTD.
REVIEW	155	FIGURE	A9

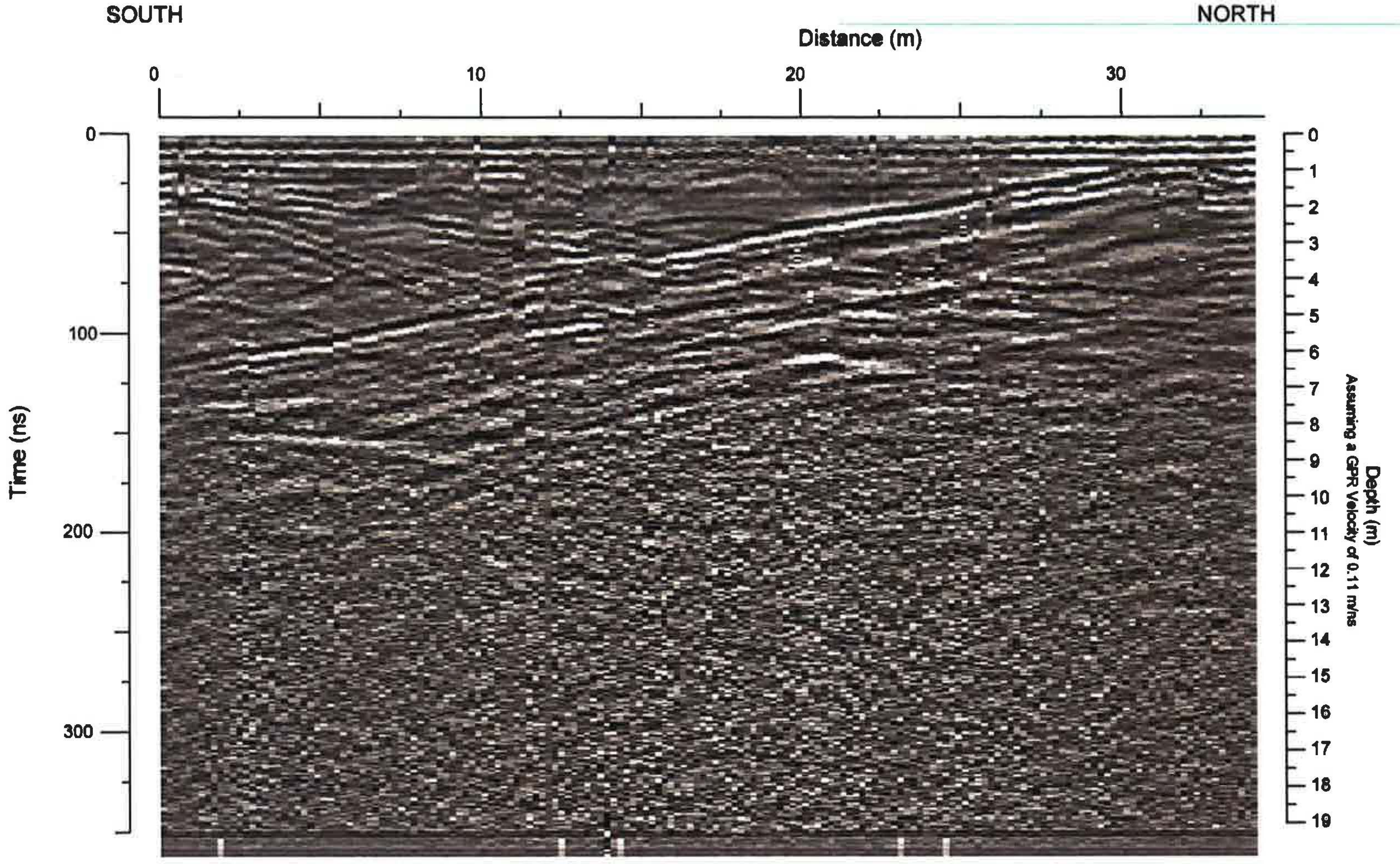
FILE No.

PROJECT No. 10-1121-0089 REV.

GPR LINE GC23 - 50 MHz



GPR LINE GC24 - 50 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN
DATE	APRIL 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	SS
FILE No.	
PROJECT No.	10-1121-0089
REV.	

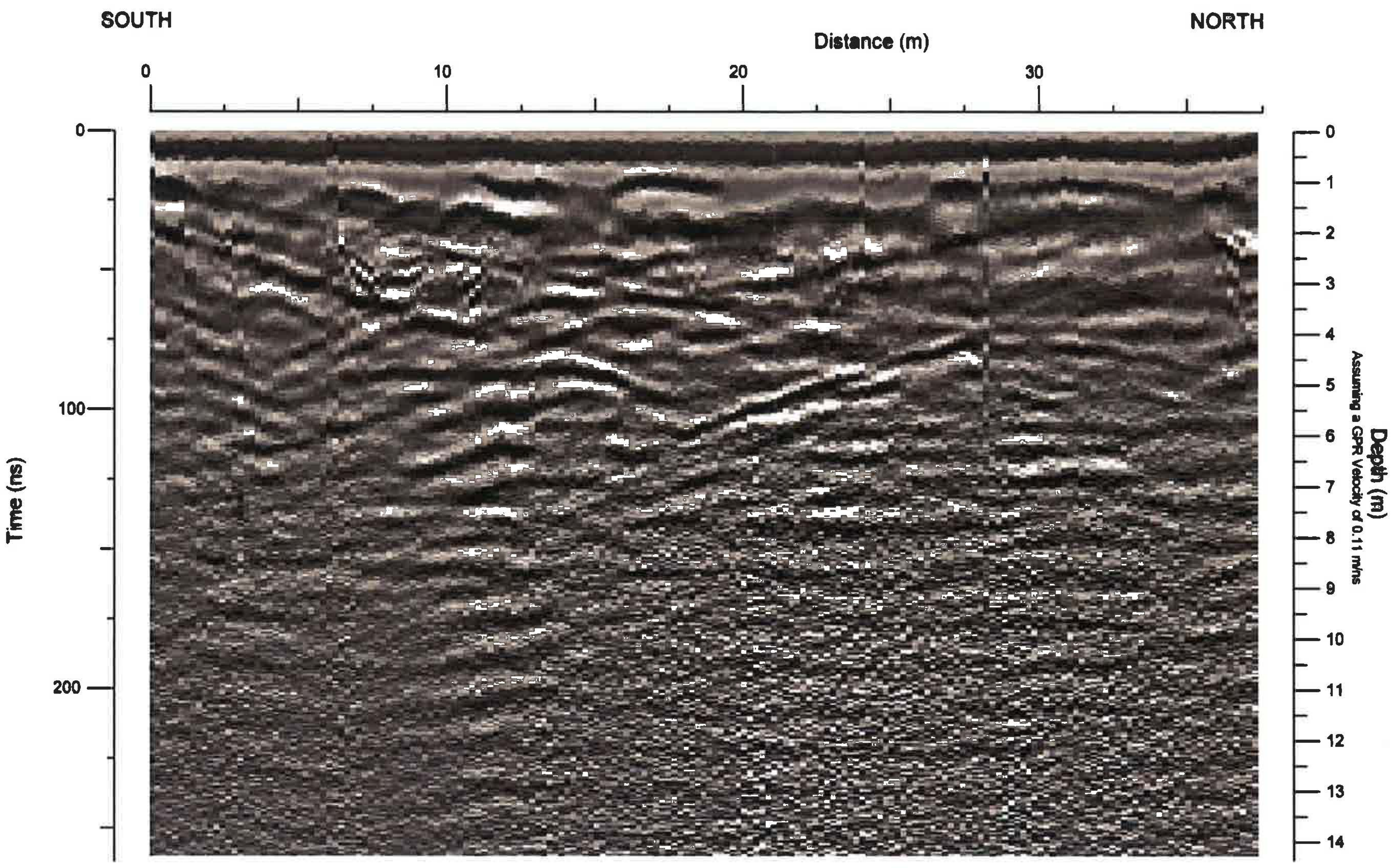
TITLE

GPR LINES GC23 and GC24 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE **A10**

GPR LINE GC25 - 100 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASS

FILE No.
PROJECT No. 10-1121-0089
REV.

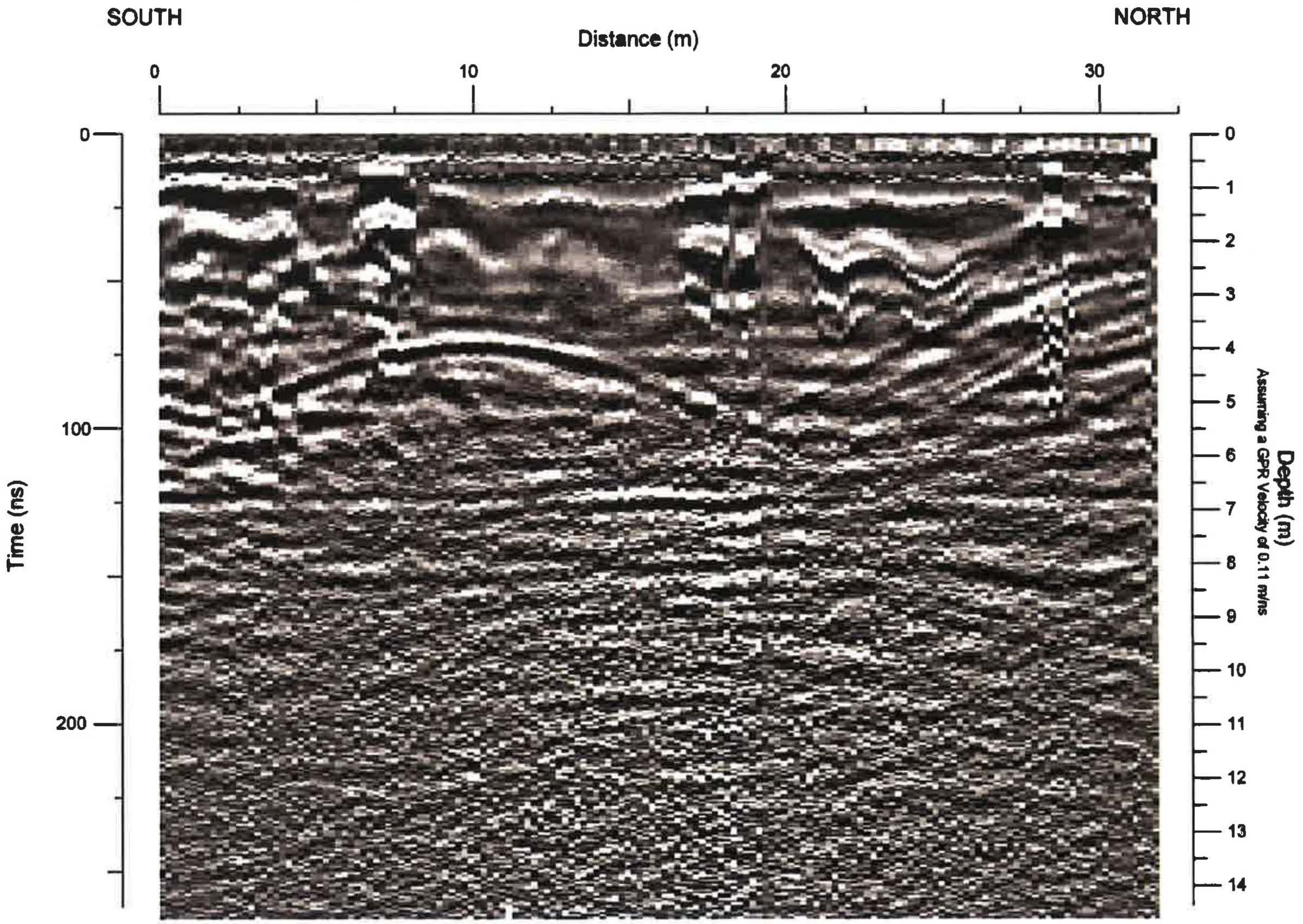
TITLE

GPR LINE GC25 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE A11

GPR LINE GC26 - 100 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN	DATE	APRIL 2011
DESIGN	SS	CAD	SS
CHECK	CRP	REVIEW	ASS
FILE No.		PROJECT No.	10-1121-0089
REV.			

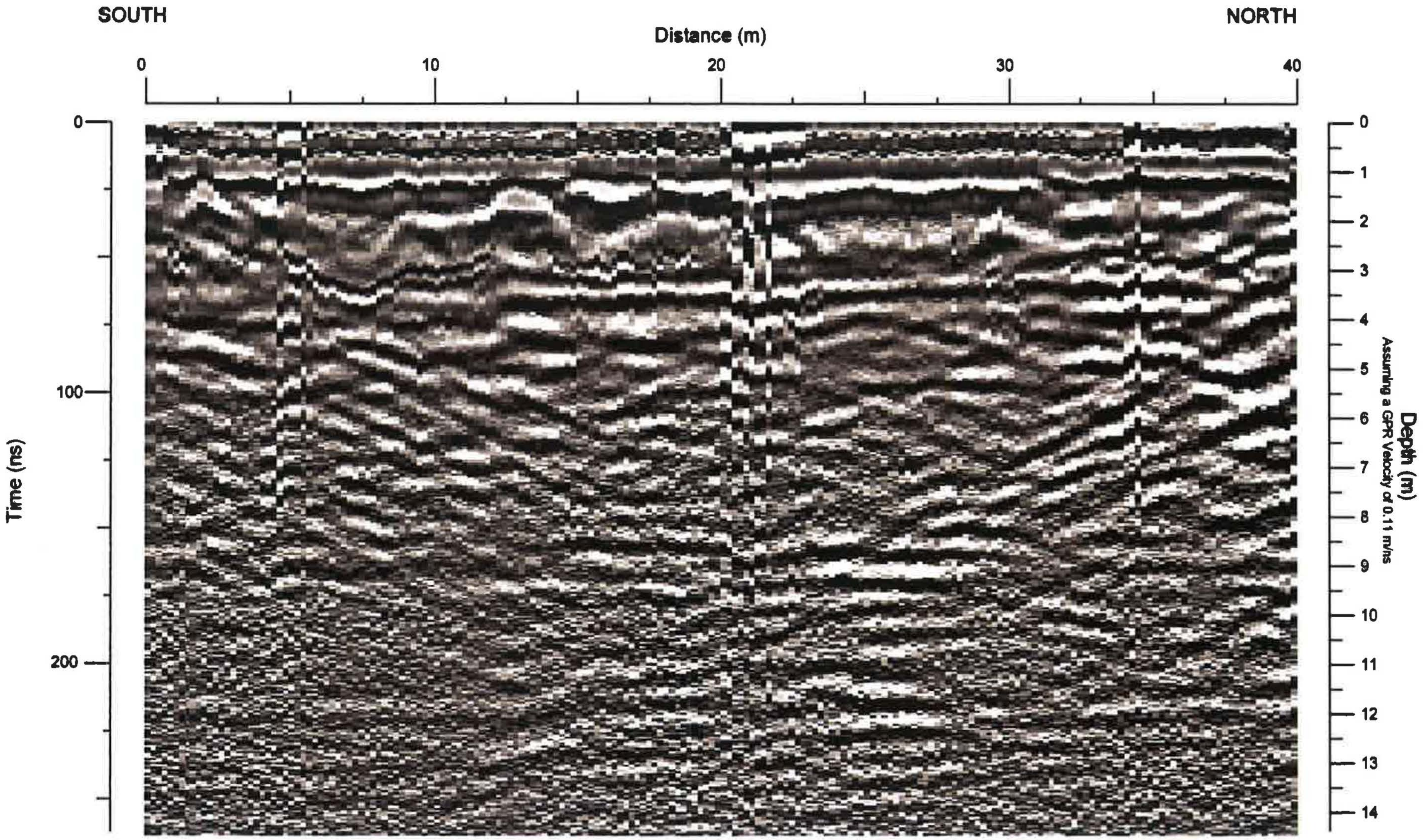
TITLE

GPR LINE GC26 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE **A12**

GPR LINE GC27 - 100 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

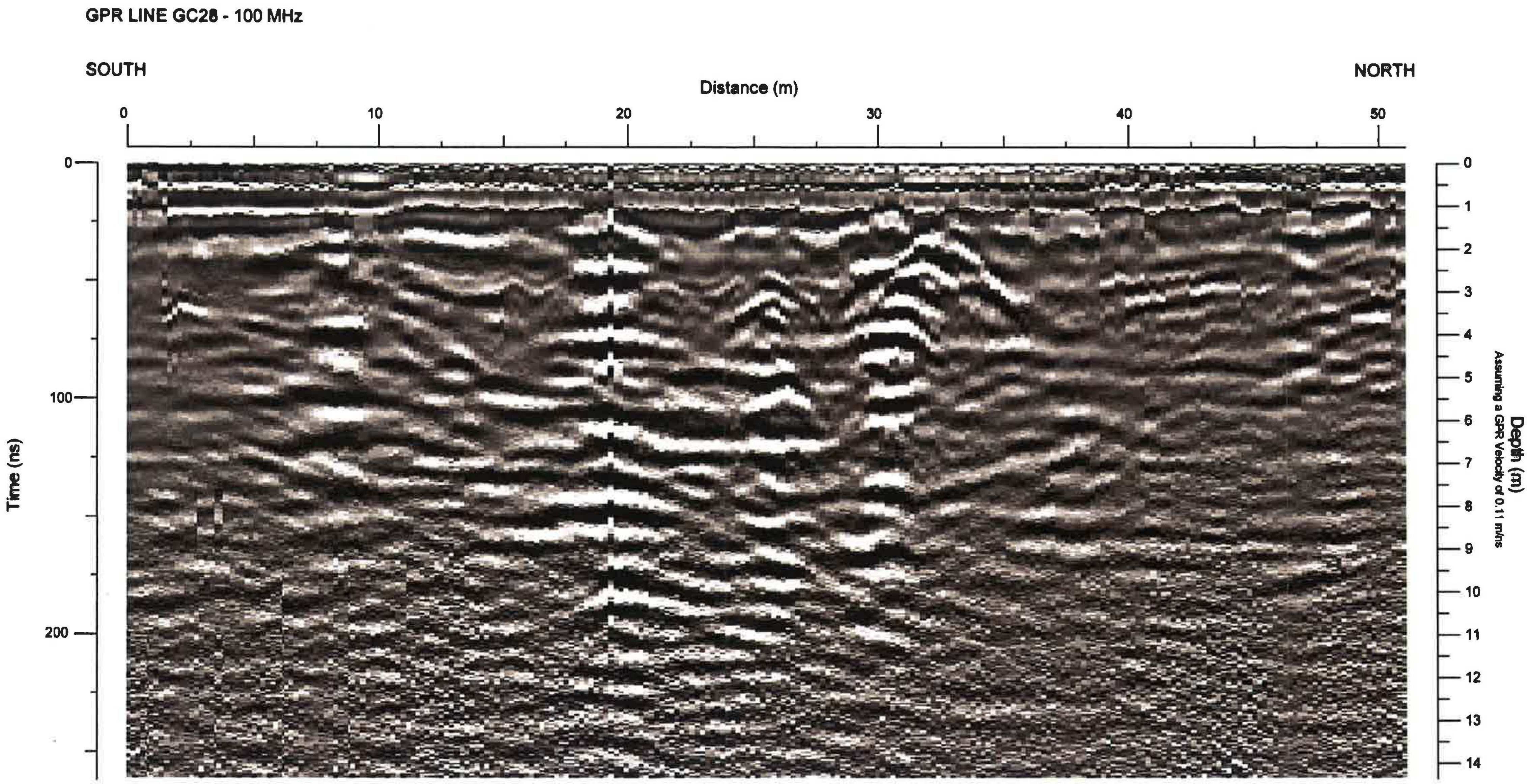
Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN
DATE	APRIL 2011
DESIGN	SS
CAD	SS
CHECK	CRP
REVIEW	ISS
FILE No.	
PROJECT No.	10-1121-0089
REV.	

TITLE
GPR LINE GC27
INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE
A13



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

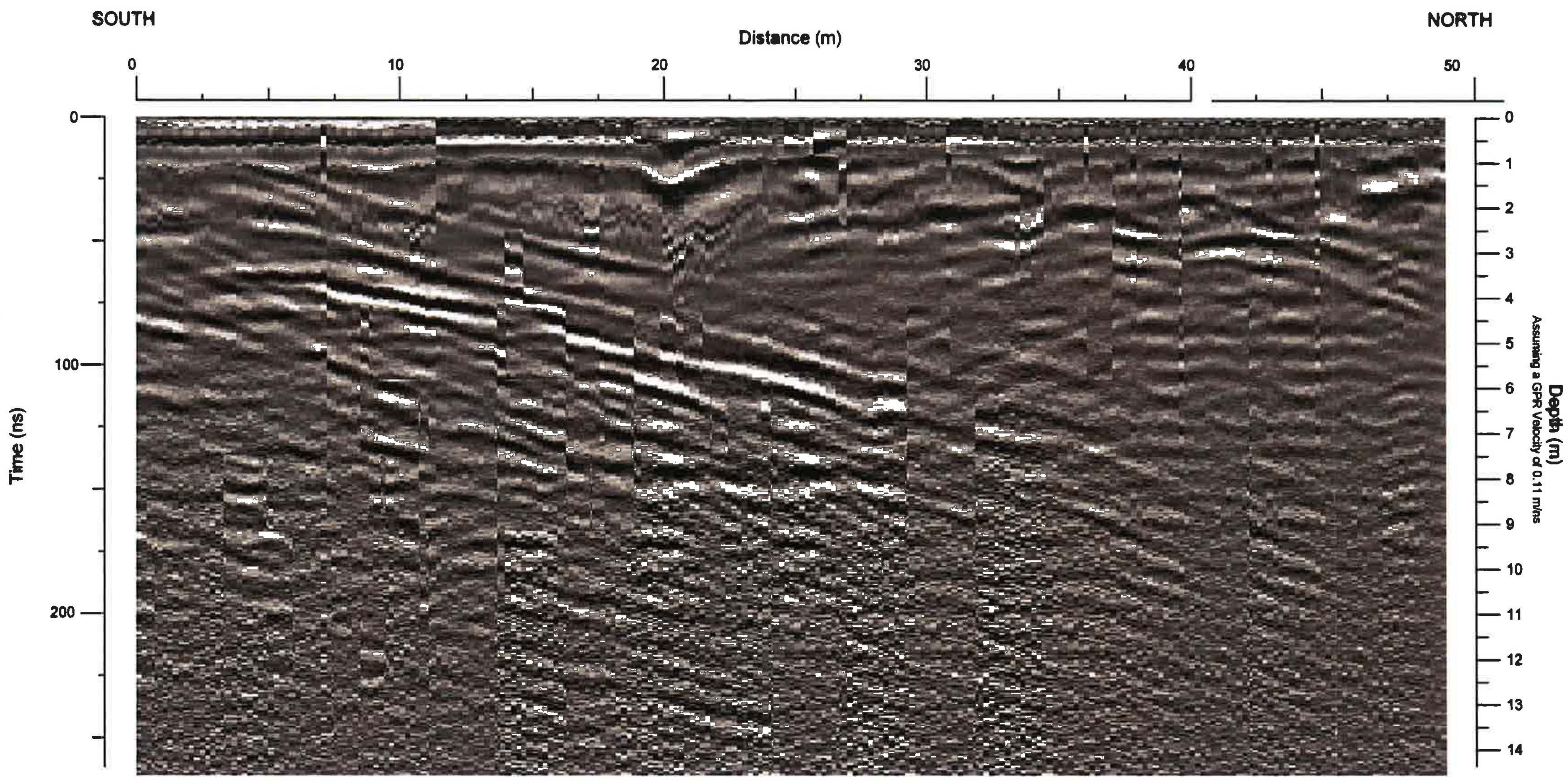
Golder Associates
Mississauga, Ontario, Canada

SCALE	AS SHOWN	TITLE	GPR LINE GC28 INTERPRETED SURVEY RESULTS
DATE	APRIL 2011		
DESIGN	SS		
CAD	SS		
CHECK	CRP		J.L. RICHARDS AND ASSOCIATES LTD.
REVIEW	1SS	FIGURE	A14

FILE No.

PROJECT No. 10-1121-0089 REV.

GPR LINE GC29 - 100 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASS

FILE No.
PROJECT No. 10-1121-0089 REV.

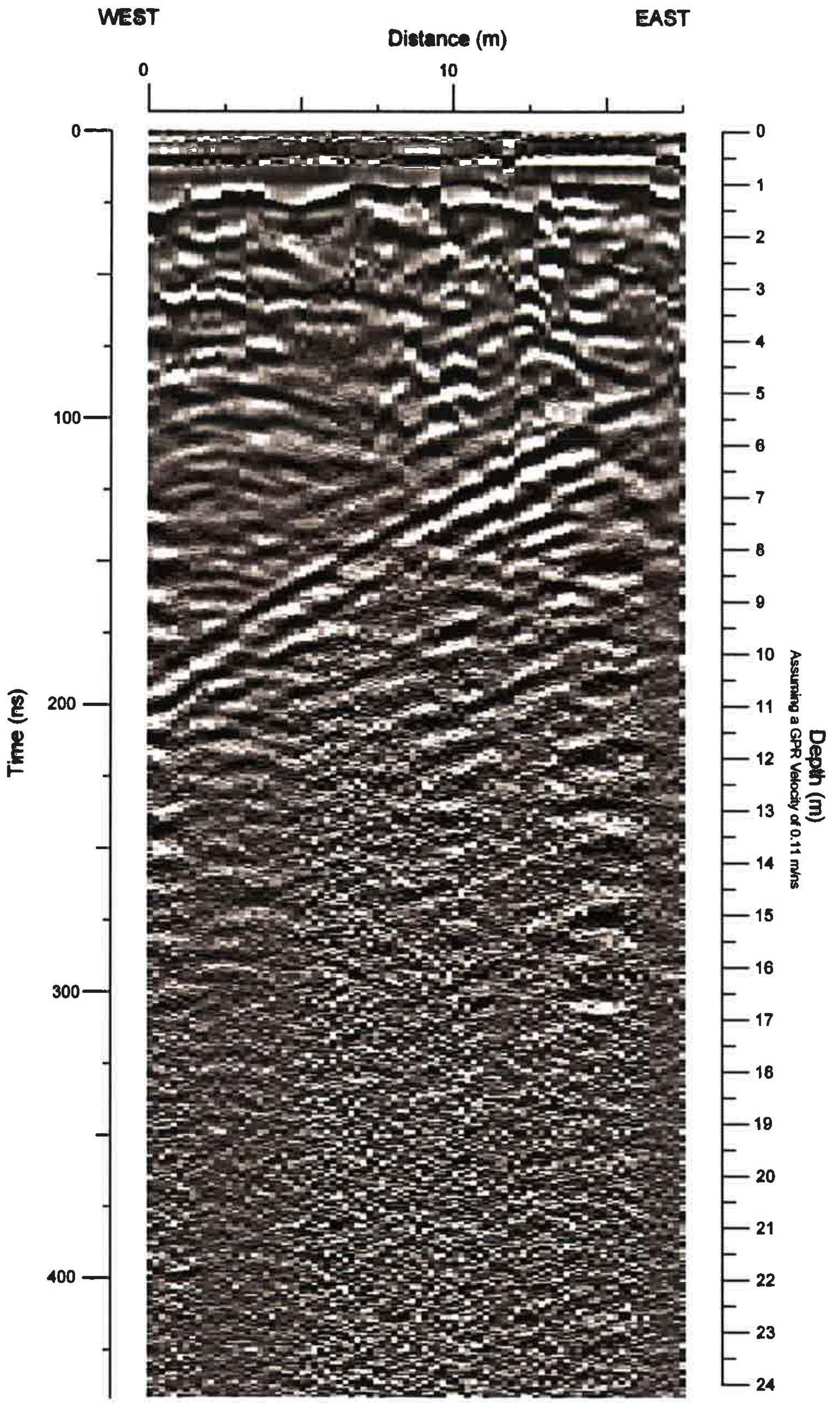
TITLE

GPR LINE GC29 INTERPRETED SURVEY RESULTS

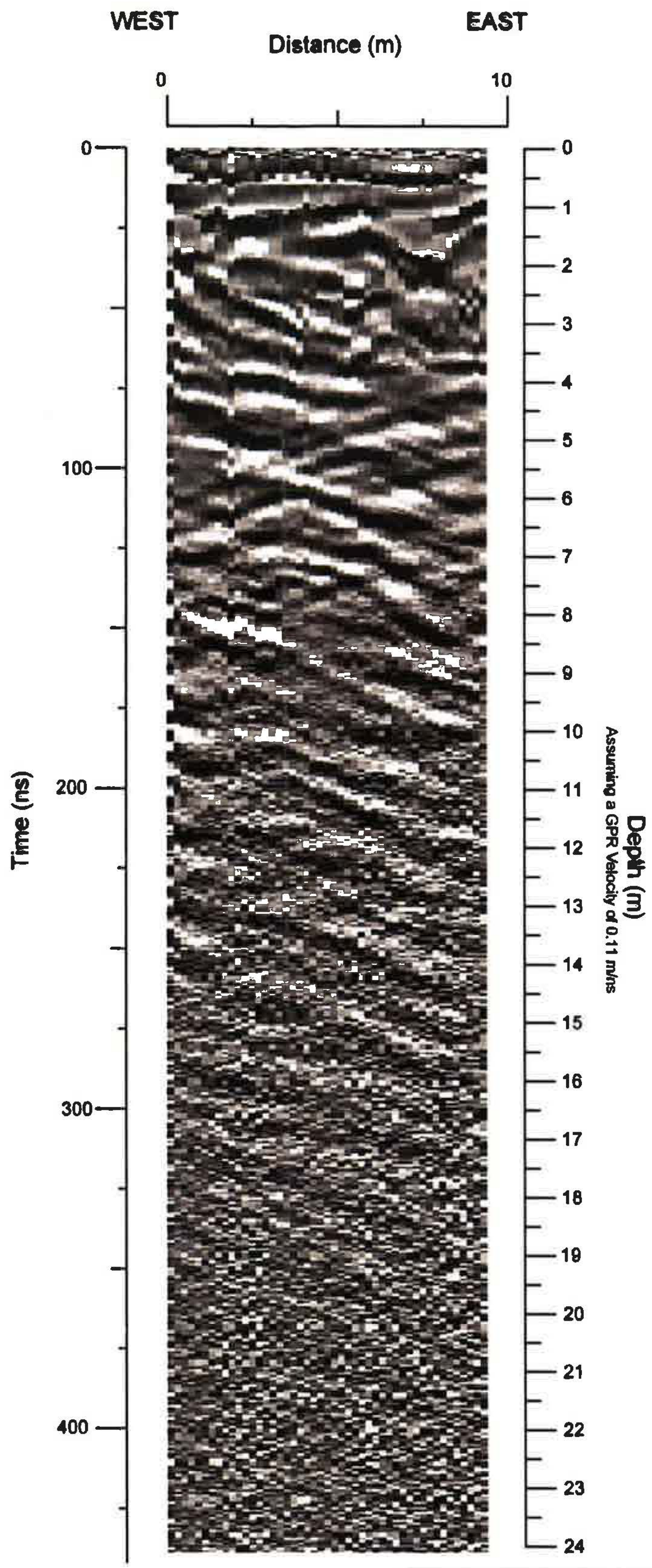
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE **A15**

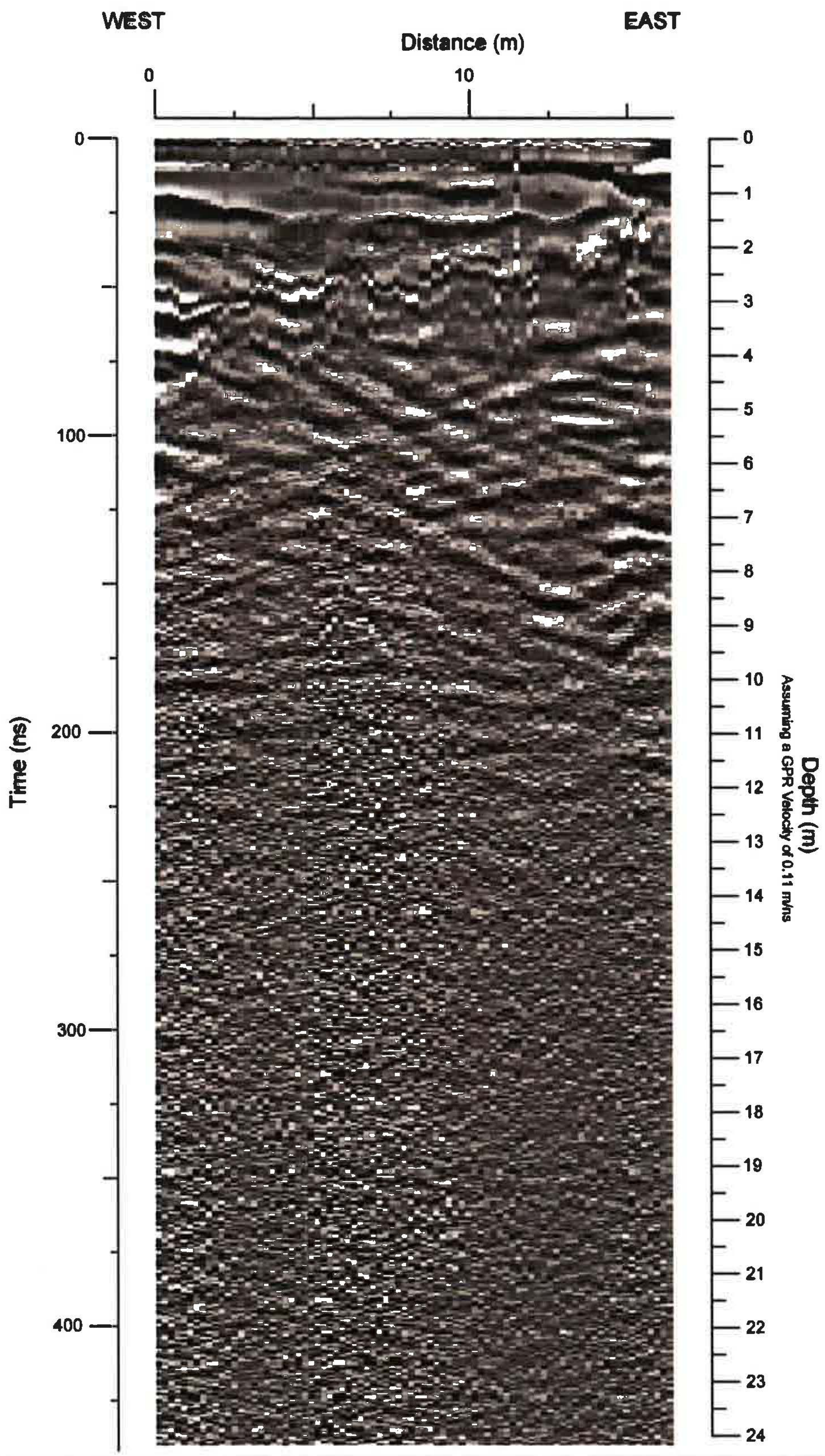
GPR LINE GC30 - 100 MHz



GPR LINE GC31 - 100 MHz



GPR LINE GC32 - 100 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089 REV.

SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASS

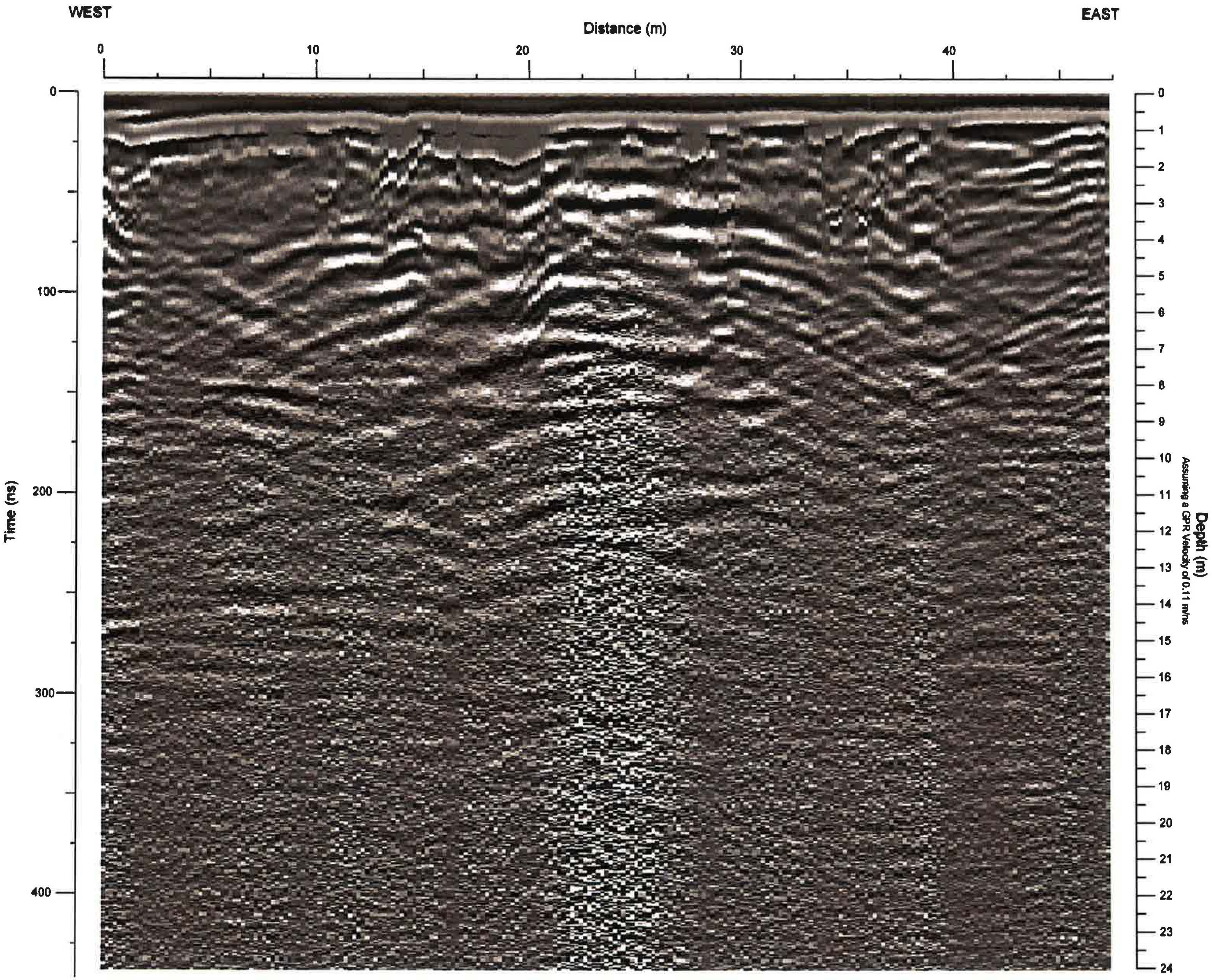
TITLE

GPR LINES GC30, GC31 and GC32 INTERPRETED SURVEY RESULTS

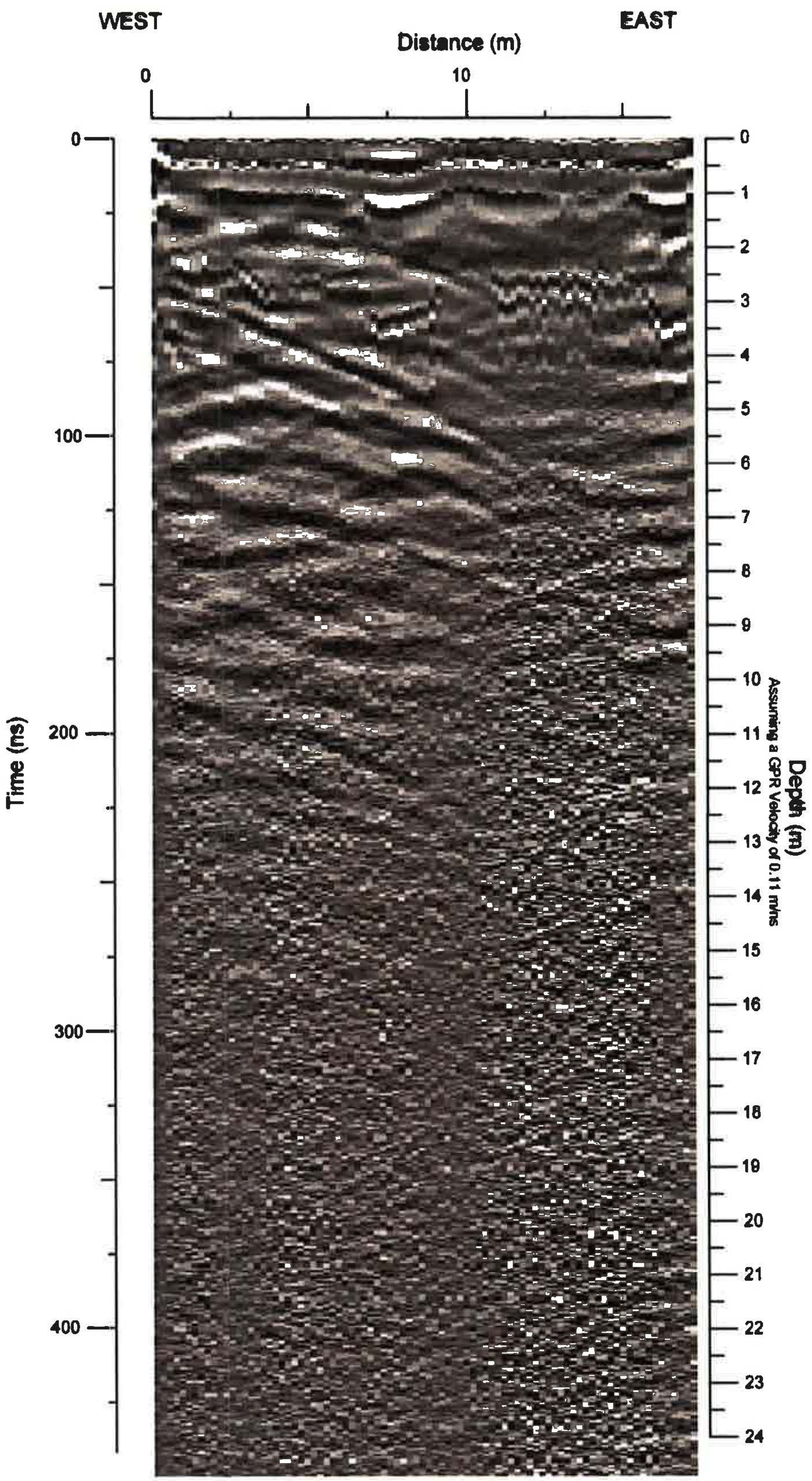
J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE
A16

GPR LINE GC33 - 100 MHz



GPR LINE GC34 - 100 MHz



Notes

1. This Figure is to be analyzed in conjunction with the accompanying report

Golder Associates
Mississauga, Ontario, Canada

FILE No.
PROJECT No. 10-1121-0089
REV.

SCALE AS SHOWN
DATE APRIL 2011
DESIGN SS
CAD SS
CHECK CRP
REVIEW ASS

TITLE

GPR LINES GC33 and GC34 INTERPRETED SURVEY RESULTS

J.L. RICHARDS AND ASSOCIATES LTD.

FIGURE
A17

APPENDIX B

List of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Record of Borehole Logs, Current Investigation
Rock Core Photographs

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual Tube sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open Sampler for a distance of 300 mm (12 in.)

DD- Diamond Drilling

Dynamic Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive Uncased a 50 mm (2 in.) diameter, 60^0 cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Peizo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

C_u or S_u

Consistency	Kpa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_l	liquid limit
C	consolidaiton (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	modified Proctor compaction test
SPC	standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:

1. Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	$= 3.1416$
ln x, natural logarithm of x	
$\log_{10}$ x or log x, logarithm of x to base 10	
g	Acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma''-u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1\sigma_2\sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma'=\gamma-\gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R= p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity Index=(w_l-w_p)
w_s	shrinkage limit
I_L	liquidity index=$(w-w_p)/I_p$
I_c	consistency index=$(w_l-w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index-$(e_{max}-e)/(e_{max}-e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction=tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ=0 analysis)
p	mean total stress $(\sigma_1+\sigma_3)/2$
p'	mean effective stress $(\sigma'_1+\sigma'_3)/2$
q	$(\sigma_1-\sigma_3)/2$ or $(\sigma'_1-\sigma_3)/2$
q_u	compressive strength $(\sigma_1-\sigma_3)$
S_t	sensitivity

Notes: 1. $\tau=c'\sigma'$ tan |'

2. Shear strength=(Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly Weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	>2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	<6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	>3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	<50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	>60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns - 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	<2 microns

Note: *Grains >60 microns diameter are visible to the naked eye.

O:\ Templates\Rock Description Terminology

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including naturally occurring fractures but not including mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^0 angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B -	Bedding	Ca -	Calcite
FO -	Foliation/Schistosity	P -	Polished
CL -	Cleavage	S -	Slickensided
SH -	Shear Plane/Zone	SM -	Smooth
VN -	Vein	R -	Ridged/Rough
F -	Fault	ST -	Stepped
CO -	Contact	PL -	Planar
J -	Joint	FL -	Flexured
FR-	Fracture	UE -	Uneven
MF -	Mechanical	W -	Wavy
A-	Angular	C -	Curved
BP-	Bedding Plane	H -	Hackly
BL-	Blast Induced	SL -	Sludge Coated
∥ -	Parallel To	TCA -	To Core Axis
⊥ -	Perpendicular To	STR -	Stress Induced

PROJECT: 10-1121-0089

RECORD OF DRILLHOLE: BH C11-1

SHEET 1 OF 1

LOCATION: See Site Plan

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Feb 1 & 2, 2011

DRILL RIG: CME

DRILLING CONTRACTOR: S & B TESTING

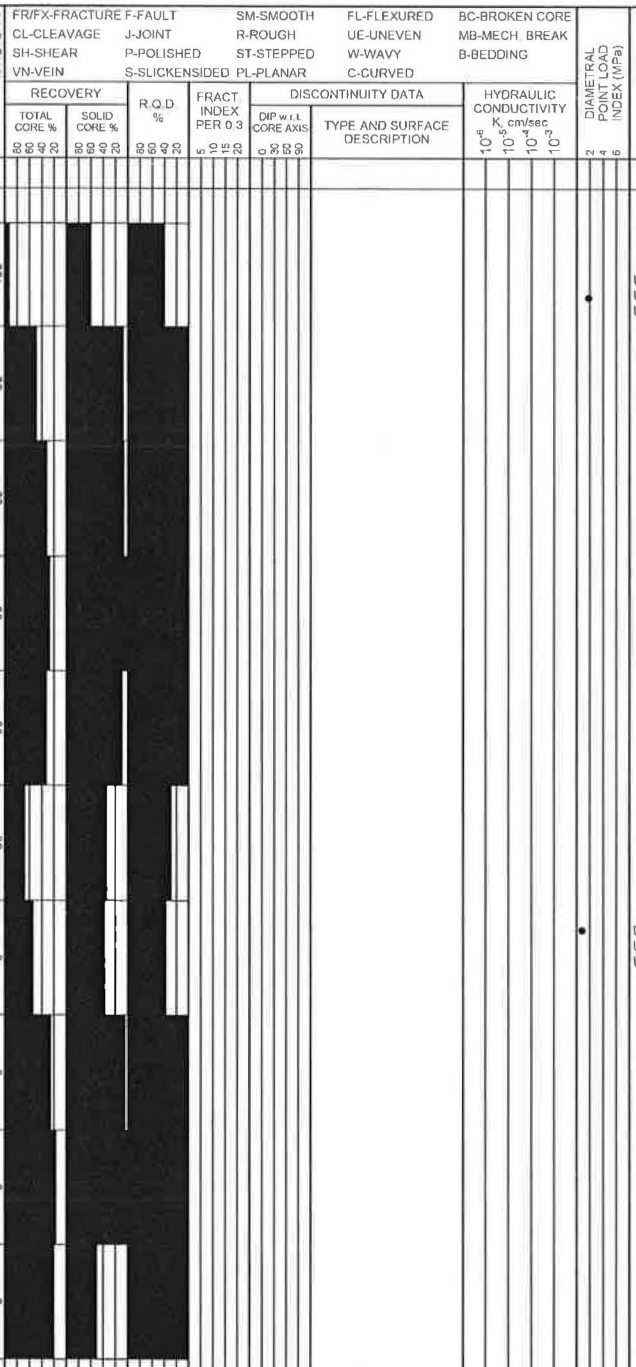
DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DRILLING RECORD: Rotary Drill, NQ Core

DESCRIPTION	ELEV. DEPTH (m)	NOTES WATER LEVELS INSTRUMENTATION
GROUND SURFACE	34.90	
TOPSOIL, with coral fragments	0.00	
White to light tan, moderately fractured to massive, porous to dense cellular structured CORAL LIMESTONE, with many small voids from 5mm to 25mm from dissolution process	34.44 0.46	UCC: 10.5MPa UW: 1814kg/m³ UW: 1694kg/m³
White to light tan, moderately to hightly fractured (fragments up to 50mm), porous CORAL LIMESTONE, with many voids up to 5mm to 30mm from dissolution process	33.07 1.83	
White to light tan, moderately fractured to massive, dense cellular structure CORAL LIMESTONE, with some to many voids up to 20mm to 30mm from dissolution process	26.98 7.92	UW: 1290kg/m³ UCC: 4.7MPa UW: 1268kg/m³
White to light tan, moderately to hightly fractured (fragments up to 50mm), porous CORAL LIMESTONE, with many voids up to 5mm to 30mm from dissolution process	23.93 10.97	
END OF DRILLHOLE	19.36 15.54	

RUN No.	FLUSH % RETURN
1	100
2	50
3	50
4	50
5	50
6	50
7	0
8	0
9	0
10	0



DEPTH SCALE METRES: 0 2 4 6 8 10 12 14 16 18 20

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: C.A.
CHECKED: BDR

PROJECT: 10-1121-0089

RECORD OF DRILLHOLE: BH C11-2

SHEET 1 OF 2

LOCATION: N ;E

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Jan. 31, 2011

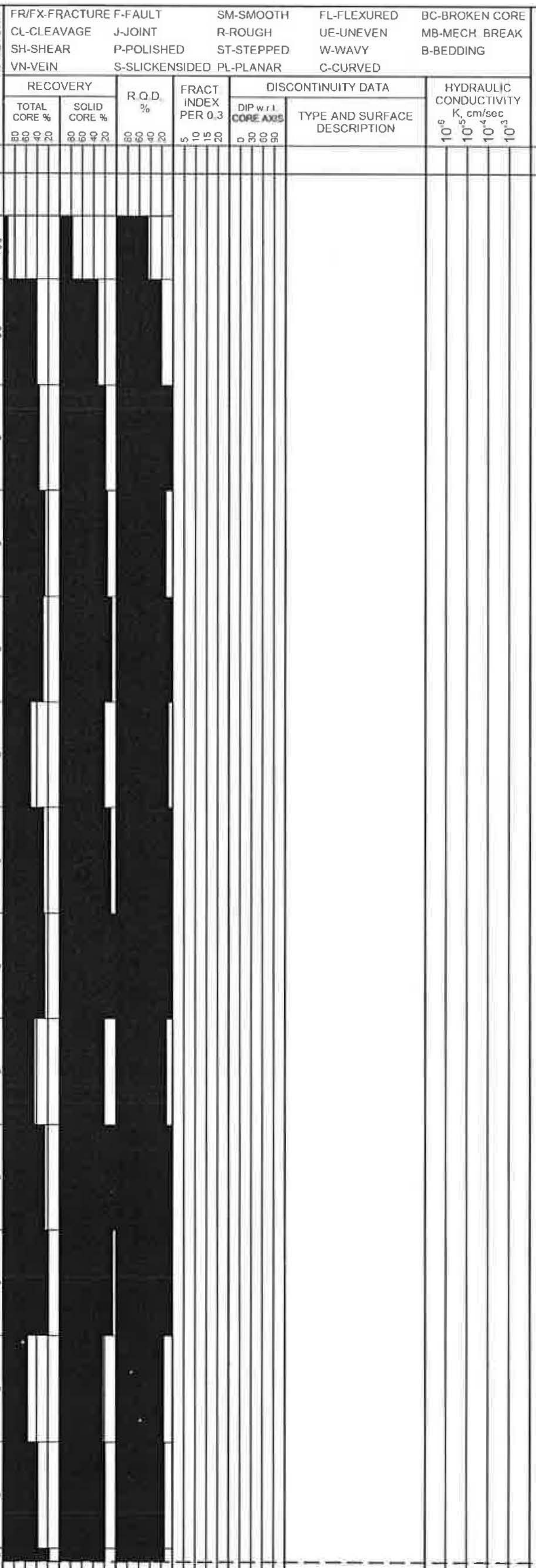
DRILL RIG: CME

DRILLING CONTRACTOR: S & B TESTING

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH % RETURN	NOTES WATER LEVELS INSTRUMENTATION
0		GROUND SURFACE	32.70			
		ASPHALTIC CONCRETE	0.05			
		Crushed coral (0.75mm diam. and smaller) (FILL)	32.09 0.61			
		White to light tan, massive, dense cellular structure CORAL LIMESTONE, with voids up to 25mm		1	100	UW: 2056kg/m³ UCC: 39.3MPa UW: 2287kg/m³
2				2	50	UW: 1718kg/m³
4		Void from 3.66m to 4.27m	29.04 3.66	3	0	
		White to light tan, moderately fractured to massive (fragments up to 80mm), dense cellular structure CORAL LIMESTONE, some to many voids up to 30mm from dissolution process	28.43 4.27			
		Void from 4.88m to 5.18m	27.82 4.88			
6		White to light tan, moderately fractured to massive (fragments up to 80mm), dense cellular structure CORAL LIMESTONE, some to many voids up to 30mm from dissolution process	5.18	4	0	
				5	0	
8				6	0	
10	Rotary Drill NQ Core	White to light tan, highly fractured (powdery from 13.7m to 14.02m), trace massive fragments (up to 180mm), highly porous CORAL LIMESTONE, with numerous small voids up to 10mm from dissolution process	23.56 9.14	7	0	
				8	0	
12				9	0	
14				10	0	
16				11	0	
				12	0	UCC: 51.6MPa UW: 2188kg/m³
18		White to light tan, moderately fractured to massive (fragments up to 250mm), highly porous (with numerous small voids up to 10mm) to dense, cellular structured CORAL LIMESTONE, with some small voids up to 15mm from dissolution process	14.41 18.29	13	0	
20			12.89 19.81	14	0	



CONTINUED NEXT PAGE

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: C.A.
CHECKED: BMh

PROJECT: 10-1121-0089

RECORD OF DRILLHOLE: BH C11-2

SHEET 2 OF 2

LOCATION: N ;E

DRILLING DATE: Jan. 31, 2011

DATUM: Geodetic

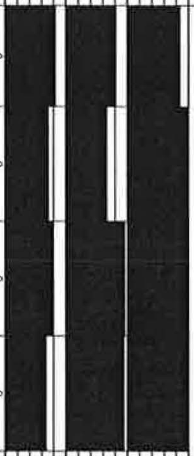
INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME

DRILLING CONTRACTOR: S & B TESTING

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 (5 10 15 20)	DIP w.r.t. CORE AXIS (0 30 60 90)	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)	NOTES WATER LEVELS INSTRUMENTATION
		--- CONTINUED FROM PREVIOUS PAGE ---														
20	Rotary Drill NQ Core	White to light tan, highly fractured (fragments up to 50mm, some massive pieces up to 170mm), highly porous CORAL LIMESTONE, with numerous small voids up to 20mm from dissolution process	☼		14		0									
22			☼		15		0									
24			☼		16		0									
			☼	6.79	17		0									
26		END OF DRILLHOLE		25.91												
28																
30																
32																
34																
36																
38																
40																



MIS-RCK 001 1011210089.GPJ GAL-MISS.GDT 4/8/11 JM

DEPTH SCALE

1 : 100

Golder Associates

LOGGED: C.A.

CHECKED: JBD 92

PROJECT: 10-1121-0089

RECORD OF DRILLHOLE: BH C11-3

SHEET 1 OF 1

LOCATION: N ;E

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Jan. 26, 2011

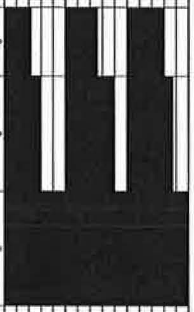
DRILL RIG: CME

DRILLING CONTRACTOR: S & B TESTING

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY: TOTAL CORE % (80 60 40 20) / SOLID CORE % (80 60 40 20); R.Q.D. % (80 60 40 20); FRACT INDEX PER 0.3 (5 10 15 20)	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90); TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)	NOTES WATER LEVELS INSTRUMENTATION
		GROUND SURFACE		34.50								
0	Rotary Drill NQ Core	TOPSOIL		0.00 0.18								
		White, highly fragmented CORAL LIMESTONE (fragments up to 50mm)		33.89 0.61	1							
2		White to light tan, moderately fractured, very porous (with numerous small voids up to 10mm) dense cellular structure CORAL LIMESTONE, some voids up to 25mm from dissolution process		31.45 3.05	2 3		0 0				●	UCC: 9.4MPa UW: 1801kg/m³ UW: 1967kg/m³ UCC: 9.2MPa UW: 1794kg/m³
4		Probably Coral (No core recovery)		29.93 4.57	4		0					
		END OF DRILLHOLE										
6												
8												
10												
12												
14												
16												
18												
20												



MIS-RCK 001 1011210089.GPJ GAL-MISS.GDT 4/8/11 JM

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: C.A.
CHECKED:

PROJECT: 10-1121-0089

RECORD OF DRILLHOLE: BH C11-4

SHEET 1 OF 2

LOCATION: N ;E

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: Jan. 28, 2011

DRILL RIG: CME

DRILLING CONTRACTOR: S & B TESTING

DATUM: Geodelic

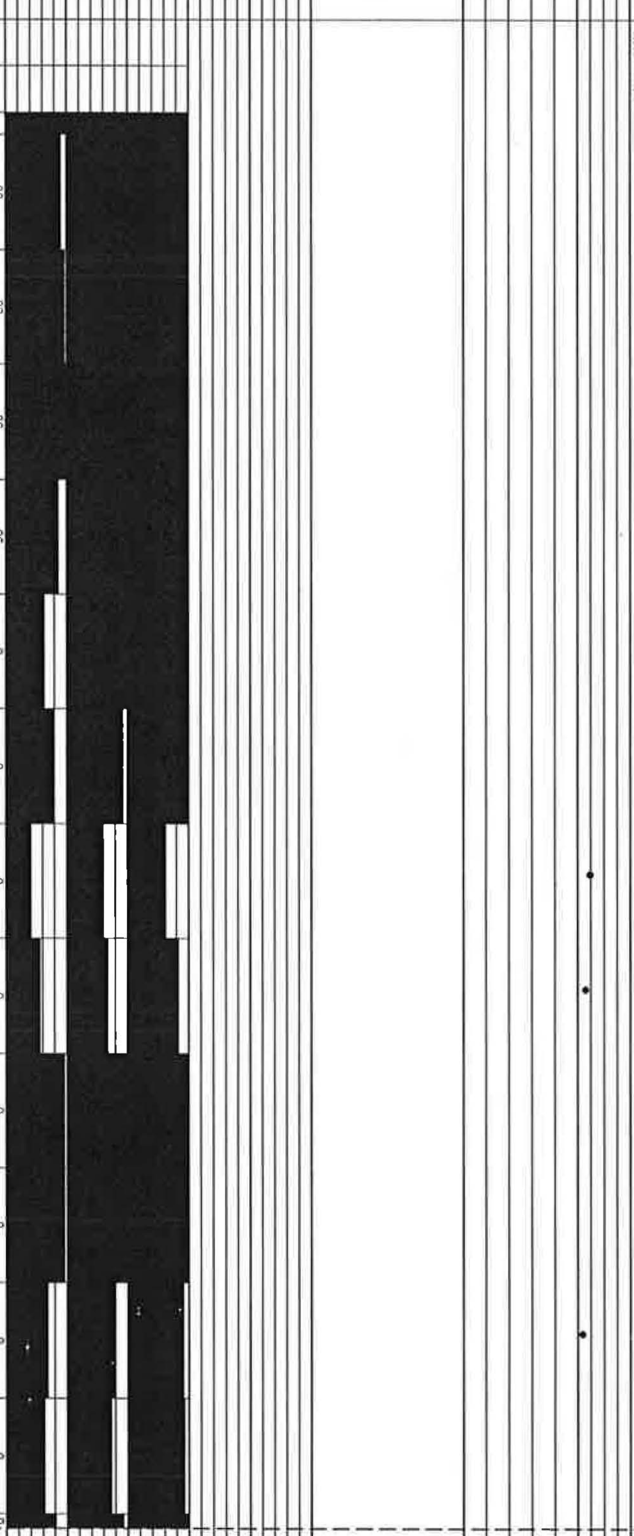
FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DRILLING RECORD: Rotary Drill, NQ Core

DESCRIPTION	ELEV.	DEPTH (m)
GROUND SURFACE	32.50	
TOPSOIL		0.06
White to light tan, highly weathered (powdery) to highly fractured CORAL LIMESTONE		
Void from 1.22m to 1.52m	31.28	1.22
White to light tan, highly fractured (fragments up to 50mm) CORAL LIMESTONE, with some small voids up to 10mm from dissolution process		1.52
Possible void (No core recovery)	27.93	4.57
White to light tan, highly fractured (fragments up to 50mm) to massive (fragments up to 120mm) CORAL LIMESTONE, with many voids from 10mm to 30mm in size	26.40	6.10
White to light tan, massive, well defined cellular structure CORAL LIMESTONE, trace small voids up to 10mm to porous with many small voids and some voids up to 30mm	21.83	10.67
White to light tan, highly to moderately fractured (fragments from 40mm to 120mm) CORAL LIMESTONE, with some to many voids up to 25mm from dissolution process	18.78	13.72

CONTINUED NEXT PAGE

RUN No.	FLUSH
1	
2	
3	
4	50
5	50
6	50
7	50
8	0
9	0
10	0
11	0
12	0
13	0
14	0
15	0
16	0



NOTES WATER LEVELS INSTRUMENTATION

S#1
SPT-N: 14bpf

S#2
SPT-N: 80bpf

UW: 1603kg/m³
UCC: 19.5MPa
UW: 1709kg/m³

UW: 1306kg/m³

UW: 1177kg/m³

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: C.A.
CHECKED: BDK

PROJECT: 10-1121-0089

RECORD OF DRILLHOLE: BH C11-4

SHEET 2 OF 2

LOCATION: N ;E

DRILLING DATE: Jan. 28, 2011

DATUM: Geodetic

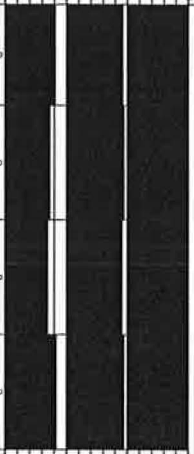
DRILL RIG: CME

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: S & B TESTING

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT INDEX PER 0.3 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}	DIAMETRAL POINT LOAD INDEX (MPa) 2 4 6	NOTES WATER LEVELS INSTRUMENTATION
		— CONTINUED FROM PREVIOUS PAGE —														
20	Rotary Drill NQ Core	White to light tan, highly to moderately fractured (fragments from 40mm to 120mm) CORAL LIMESTONE, with some to many voids up to 25mm from dissolution process	☼		16		0									
22					17		0									
24					18		0									
				6.59 25.91	19		0									
26		END OF DRILLHOLE														
28																
30																
32																
34																
36																
38																
40																



DEPTH SCALE

1 : 100

Golder Associates

LOGGED: C.A.

CHECKED: BDZ

PROJECT: 10-1121-0089

RECORD OF DRILLHOLE: BH C11-5

SHEET 1 OF 1

LOCATION: N ;E

INCLINATION: -90° AZIMUTH: ---

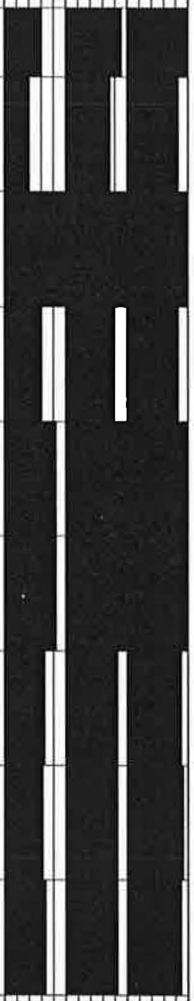
DRILLING DATE: Feb. 8, 2011
DRILL RIG: CME
DRILLING CONTRACTOR: S & B TESTING

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DRILLING RECORD: Rotary Drill, NQ Core

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN FLUSH	NOTES WATER LEVELS INSTRUMENTATION
	GROUND SURFACE	36.10				
0	TOPSOIL	0.00				
	Dark brown sand, mixed with coral gravel (FILL)	0.30				
2	Tan to white, moderately fractured to massive (fragments up to 180mm) dense, porous CORAL LIMESTONE, some voids up to 15mm from dissolution process	33.97 2.13	1		100	
4			2		50	UCC: 25.0MPa UW: 2054kg/m³
	Probably Coral (No core recovery)	31.53 4.57	3		0	
6	Tan to white, moderately fractured to massive (fragments up to 200mm) dense, porous CORAL LIMESTONE, some voids up to 20mm to 30mm from dissolution process	30.00 6.10	4		0	UW: 1566kg/m³
8	White to light tan, highly fractured (fragments up to 30mm), porous CORAL LIMESTONE, with some small voids up to 10mm to 15mm	28.48 7.62	5		0	
10			6		0	
	White to light tan, moderately fractured to massive (fragments up to 150mm), porous CORAL LIMESTONE, with some to many voids from 10mm to 30mm	25.43 10.67	7		0	
12			8		0	
14			9		0	UW: 1726kg/m³
	END OF DRILLHOLE	20.86 15.24				
16						
18						
20						



DEPTH SCALE
1 : 100

Golder Associates

LOGGED: C.A.
CHECKED: RDM

PROJECT: 10-1121-0089

RECORD OF DRILLHOLE: BH C11-6

SHEET 1 OF 1

LOCATION: N ;E

DRILLING DATE: Jan. 29, 2011

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: --

DRILL RIG: CME

DRILLING CONTRACTOR: S & B TESTING

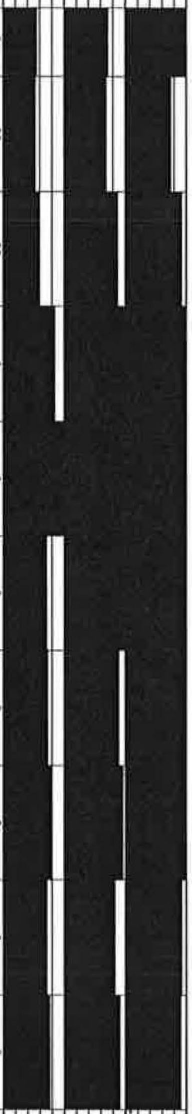
FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | PENETRATION RATE (m/min) | COLOUR % RETURN | FLUSH | RECOVERY: TOTAL CORE %, SOLID CORE % | R.Q.D. % | FRACT. INDEX PER 0.3 | DISCONTINUITY DATA: DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION | HYDRAULIC CONDUCTIVITY K, cm/sec | DIAMETRAL POINT LOAD INDEX (MPa) | NOTES WATER LEVELS INSTRUMENTATION

Drilling record: Rotary Drill, NQ Core

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	36.20
ASPHALTIC CONCRETE	0.05
Crushed coral (FILL)	0.30
White to light tan, highly fractured to massive (fragments up to 250mm), porous CORAL LIMESTONE, with many voids from 10mm to 40mm from dissolution process	33.15 3.05
White to light tan, highly to moderately fractured (fragments up to 5mm to 80mm), porous CORAL LIMESTONE, with many small voids up to 10mm to 15mm from dissolution process	31.93
Void from 4.27m to 4.42m	4.42
White to light tan, highly to moderately fractured (fragments up to 5mm to 80mm), porous CORAL LIMESTONE, with many small voids up to 10mm to 15mm from dissolution process	30.10 6.10
Possible void (No core recovery)	28.58 7.62
White to light tan, highly to moderately fractured (fragments up to 5mm to 40mm), porous CORAL LIMESTONE, with many small voids up to 10mm to 15mm from dissolution process	24.01 12.19
White to light tan, moderately fractured to massive (fragments up to 100mm) dense cellular structure CORAL LIMESTONE, with some small voids up to 15mm	20.96 15.24
END OF DRILLHOLE	

RUN No.	FLUSH
1	
2	75
3	50
4	50
5	0
6	0
7	0
8	0
9	0
10	0
11	0



NOTES:

S #1
SPT-N: >100bpf

UW: 1486kg/m³

UW: 1696kg/m³

DEPTH SCALE
1 : 100

Golder Associates

LOGGED: C.A.

CHECKED:

Bedrock Core from Borehole C11-1

Depth 0.46 m to 6.40 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-1

Bedrock Core from Borehole C11-1

Depth 6.40 m to 15.54 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-2

Bedrock Core from Borehole C11-2

Depth 0.61 m to 7.47 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-3

Bedrock Core from Borehole C11-2

Depth 7.47 m to 15.24 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-4

Bedrock Core from Borehole C11-2

Depth 15.24 m to 25.91 m

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-5

Bedrock Core from Borehole C11-3

Depth 0.18 m to 3.05 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-6

Bedrock Core from Borehole C11-4

Depth 0.06 m to 12.80 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-7

Bedrock Core from Borehole C11-4

Depth 12.80 m to 25.91 m

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-8

Bedrock Core from Borehole C11-5

Depth 2.13 m to 12.95 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-9

Bedrock Core from Borehole C11-5

Depth 12.95 m to 15.24 m

Golder Associates	Geotechnical Investigation The Chancery for the Canadian High Commission Bridgetown, St. Michael Parship, Barbados	Project No. 10-1121-0089 Drawn: BDG Date: 11/04/2011 Checked: MSS Review: MSS	Figure A-10

Bedrock Core from Borehole C11-6

Depth 0.30 m to 10.67 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-11

Bedrock Core from Borehole C11-6

Depth 10.67 m to 15.24 m

Golder Associates

Geotechnical Investigation

The Chancery for the Canadian High Commission

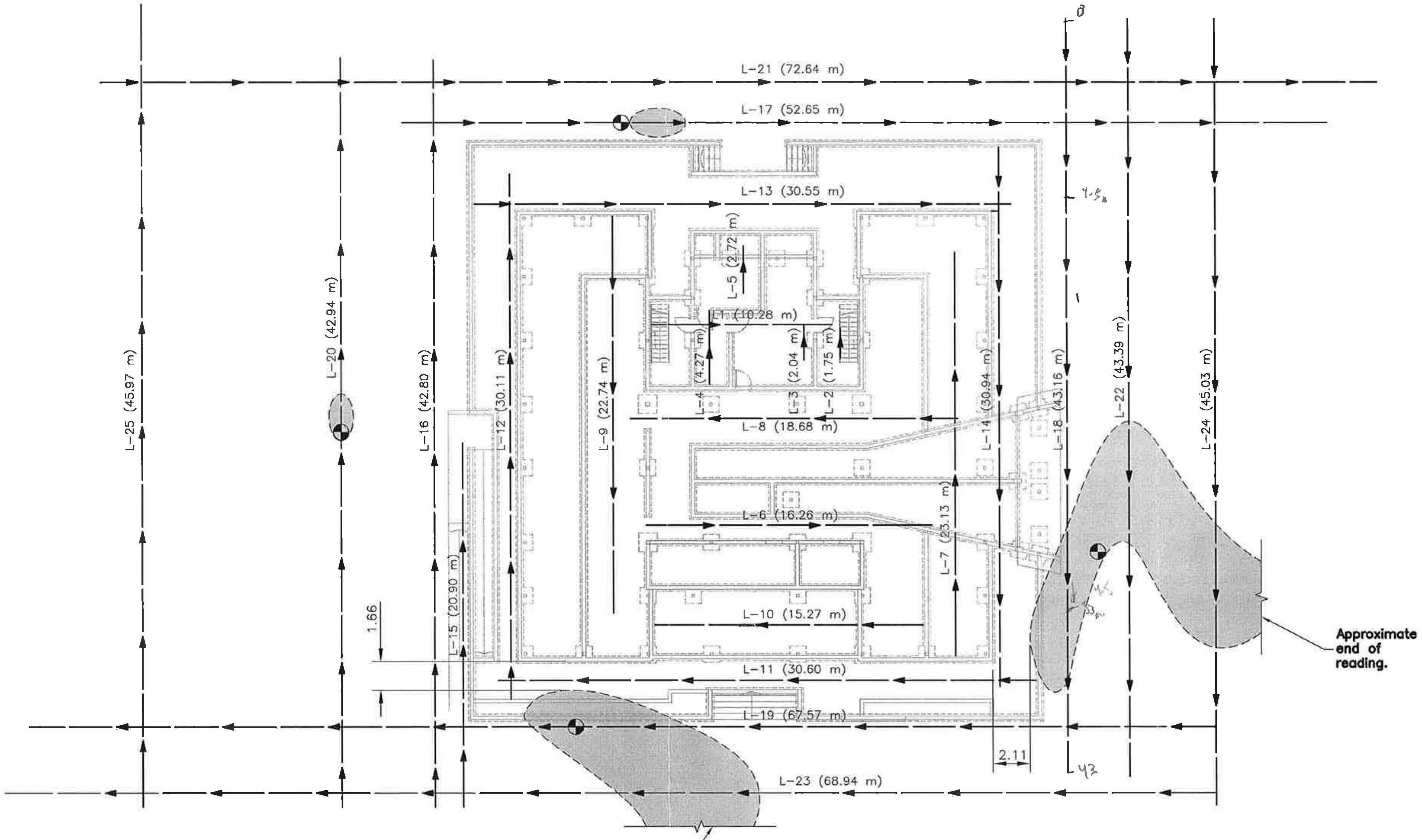
Bridgetown, St. Michael Parship, Barbados

Project No.	10-1121-0089
Drawn:	BDG
Date:	11/04/2011
Checked:	MSS
Review:	MSS

Figure A-12

APPENDIX C

Georadar Line Location Plan
Record of Borehole Logs (4)
GPR Survey Plots (Lines L-16 through L-25)
Previous Investigation



Approximate end of reading.

Approximate end of reading.

LEGEND

⟶	GEORADAR LINE
▨	POSSIBLE KARSTIC FORMATION
◉	POTENTIAL TARGET OF DRILLING

THIS ENGINEERING DOCUMENT IS THE WORK OF DESSAU AND, AS SUCH, IS PROTECTED BY LAW. IT IS SOLELY INTENDED FOR THE USE MENTIONED HEREIN. IT IS STRICTLY FORBIDDEN TO DUPLICATE OR ADAPT IT EITHER IN PART OR IN ITS ENTIRETY WITHOUT HAVING FIRST OBTAINED DESSAU'S WRITTEN AUTHORIZATION TO DO SO.

Project

CANADIAN HIGH COMMISSION CHANCERY

BRIDGETON BISHOPS COURT HILL - BARBADOS

Title

GEORADAR LINES LOCATION

HYDROGÉO-SOL

245, rue Riverin, bureau 101
Chicoutimi (Québec) G7H 4R6
Téléphone : 418.545.8256
Télécopieur : 418.545.8254

Prepared A.G.
Drawn M.C.
Checked A.G.

Discipline GEOLOGICAL SCIENCES
Scale 1:250
Date 2008-03-26

Project manager
Andy Guyaz
Extract from:
Rev.: 00

M. dept.	Project	Work pckg.	Sub-w.p.	Disc.	Drawing no.	Rev.
034	P015952	0170	000	SC	0001	00

Rapport de forage du : 2008-04-16 12h

DESSAU

BOREHOLE REPORT

Client : D.F.A. I.T. Canada

Project: **Geotechnical investigation - High Commission of Canada**
Location: **Bishop's Court Hill, St-Michael, Barbados**
File n°: **P015952-160**
Borehole n° : **BH-01-08**

Coordinates (m): **1449108.00 N** **218059.00 E**
Dates : **2008-02-29** **Drilling equipement :** **Devis, USA**

Reference Datum: **Arbitrary**
Elevation: **100.41 m**
Bedrock depth m
End of borehole depth 12.19 m

Sample condition ▨ Intact ☒ Remouled ■ Lost ◨ Core

SAMPLE TYPE

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Core Sample, gauge
TO	Open Tube
LA	By Washing
TA	Auger
MA	Bulk sample
TF	Split Tube
PW	LVM-Fondatec Mega-Sampler

TESTS

L	Consistancy Limits
W_L	Liquid Limit (%)
W_P	Plastic Limit (%)
I_P	Plasticity Index (%)
I_L	Liquidity Index
W	Natural Water Content (%)
AG	Grain Size Analysis
S	Hydrometer analysis
R	Refusal
P_{80}	Grain Size Analysis by washing at the 80 μm sieve
M.O.	Organic Matter (%)
K	Permeability (cm/s)
KL	Lefranc Permeability (cm/s)
UW	Unit Weight (kN/m³)
A	Absorption (l/min. m)
U	Uniaxial Compresses strength (MPa)
RQD	Rock Quality Designation (%)
AC	Chemical Analysis
P_L	Limit Pressure (kPa)
E_M	Pressuremeter Modulus (kPa)
E_r	Modulus of subgrade reaction (kPa)
▼	Water Level
N	Standard Penetration test (blows/150mm)
Nc	Dynamic Penetration test (blows/300mm) ●
σ'p	Preconsolidation Pressure (kPa)
σ'vo	Effective Pressure (kPa)

Shear Strength

		Field	Laboratory
Cu	Undisturbed (kPa)	▲	■
Cur	Remoulded (kPa)	△	□

LITHOLOGY

ELEVATION - m DEPTH - m	SOILS OR ROCK DESCRIPTION
100.41	Ground level
0.00	Grass and topsoil
100.26 0.15	
99.55 0.86	Coral rock fragments. Fragment sizes smaller than 2.5 cm
99.37 1.04	Massive coral rock with very close to closely spaced discontinuities. (spaced from 6 cm to 7 cm)
97.36 3.05	Coral rock fragments. Fragment sizes smaller than 2.5 cm
92.66 7.75	Massive coral rock with voids of approximately 2.5 cm in size created from dissolution process with very close to closely spaced discontinuities (spaced from 6 cm to 10 cm). Powdery material.
89.74 10.67	Massive coral rock with voids of approximately 2.5 cm in size created from dissolution process with very close to closely spaced discontinuities (spacing of approximately 8 cm). Powdery material. In the last 20 cm coral rock with well developed corals. Discontinuity spacing of approximately 5 cm.
88.22 12.19	END OF BOREHOLE

SAMPLES / FIELD AND LABORATORY TESTS

TYPE AND NUMBER	CALIBER	CONDITION	RECOVERY %	BLOWS / 150mm	N or RQD	RESULTS
SS-1		☒	67	6-16 28-16	44	
SS-2		☒	82	11-50 /100	R	
RC-3		☒	60		32	
RC-4		☒	15		8	
RC-5		☒	5		0	
RC-6		☒	20		0	
RC-7		☒	53		0	
RC-8		☒	97		65	UW = 12.7 kN/m³
RC-9		☒	93		0	
RC-10		☒	92		30	

WATER CONTENT AND LIMITS (%) Wp W WL — 20 40 60 80 100
UNDRAINED SHEAR STRENGTH OR DYNAMIC PENETRATION — 20 40 60 80 100

Remarks: - Coral rock of middle reef terraces Formation (MRT)
- Elevation with respect to center point of catch bassin in parking lot (see plan 033-P015952-0160-GE-0001-00) BM = 100.00 m)

Échelle verticale = 1 : 125

L:\33\Geotec\StyleLog_Forage_dessau_2006_Anglais3.sty

Prepared by: David Noël sr. tech. | Approved by: Nancy Verreault, Eng., M.A.Sc. | 2008-04-16 | Page: 1 of 1

Rapport de forage du : 2008-04-16 12h

DESSAU

BOREHOLE REPORT

Client : D.F.A. I.T. Canada

Project: **Geotechnical investigation - High Commission of Canada**
File n°: **P015952-160**
Location: **Bishop's Court Hill, St-Michael, Barbados**
Borehole n° : **BH-02-08**

Coordinates (m): 1449118.00 N 218070.00 E
Dates : 2008-02-29
Drilling equipement : Devis, USA

Reference Datum: **Arbitrary**
Elevation: **100.60 m**
Bedrock depth m
End of borehole depth 10.67 m

Sample condition ▨ Intact ☒ Remoulded ■ Lost ◧ Core

SAMPLE TYPE

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Core Sample, gauge
TO	Open Tube
LA	By Washing
TA	Auger
MA	Bulk sample
TF	Split Tube
PW	LVM-Fondatec Mega-Sampler

TESTS

L	Consistancy Limits	**M.O.**	Organic Matter (%)	▼	Water Level
W_L	Liquid Limit (%)	**K**	Permeability (cm/s)	**N**	Standard Penetration test (blows/150mm)
W_P	Plastic Limit (%)	**KL**	Lefranc Permeability (cm/s)	**Nc**	Dynamic Penetration test (blows/300mm) ●
I_P	Plasticity Index (%)	**UW**	Unit Weight (kN/m³)	σ'p	Preconsolidation Pressure (kPa)
I_L	Liquidity Index	**A**	Absorption (l/min. m)	σ'vo	Effective Pressure (kPa)
W	Natural Water Content (%)	**U**	Uniaxial Compresses strength (MPa)		**Shear Strength** Field Laboratory
AG	Grain Size Analysis	**RQD**	Rock Quality Designation (%)		
S	Hydrometer analysis	**AC**	Chemical Analysis	**Cu**	Undisturbed (kPa) ▲ ■
R	Refusal	P_L	Limit Pressure (kPa)	**Cur**	Remoulded (kPa) △ □
P_{80}	Grain Size Analysis by washing at the 80 µm sieve	E_M	Pressuremeter Modulus (kPa)		
		E_r	Modulus of subgrade reaction (kPa)		

ELEVATION - m DEPTH - m	SOILS OR ROCK DESCRIPTION	WATER LEVEL (m) / DATE	TYPE AND NUMBER	CALIBER	CONDITION	RECOVERY %	BLOWS / 150mm	N or RQD	RESULTS
100.60	Ground level								
0.00	Grass and topsoil.								
100.52 0.08	Fine to medium sand backfilled								
99.56 1.04	Coral rock fragments. Fragments sizes smaller than 8 cm		RC-1		☒	53		0	
			RC-2		☒	29		0	
			RC-3		☒	44		0	
			RC-4		☒	13		0	
			RC-5		☒	8		0	
			RC-6		☒	12		0	
92.98 7.62	Coral rock fragments. Fragments sizes of 8 cm on average.		RC-7		☒	50		7	
91.46 9.14	Coral rock fragments. Fragments sizes smaller than 5 cm. Interbedding with more massive coral rock with well developped coral layers of approximately 10 cm in length. Powdery material.		RC-8		☒	20		7	
89.93 10.67	END OF BOREHOLE								

FIELD AND LABORATORY TESTS: WATER CONTENT AND LIMITS (%) Wp W WL 20 40 60 80 100; UNDRAINED SHEAR STRENGTH OR DYNAMIC PENETRATION 20 40 60 80 100

Échelle verticale = 1 : 125

Remarks: - Coral rock of middle reef terraces Formation (MRT)
- Elevation with respect to center point of catch bassin in parking lot (see plan 033-P015952-0160-GE-0001-00) BM = 100.00 m)

Prepared by: David Noël sr. tech.
Approved by: Nancy Verreault, Eng., M.A.Sc. 2008-04-16
Page: 1 of 1

L:\33\Geotec\StyleLog_Forage_dessau_2006_Anglais3.sty

Rapport de forage du : 2008-04-16 12h

DESSAU

BOREHOLE REPORT

Client : D.F.A. I.T. Canada

Project:	**Geotechnical investigation - High Commission of Canada**	File n°:	**P015952-160**
Location:	**Bishop's Court Hill, St-Michael, Barbados**	Borehole n° :	**BH-03-08**
Coordinates (m):	**1149095.00 N** **218061.00 E**	**Dates :**	**2008-03-03**
		Drilling equipement :	**Devis, USA**

Reference Datum:	**Arbitrary**	Bedrock depth		m
Elevation:	**100.45 m**	End of borehole depth		13.72 m

Sample condition: ▨ Intact ☒ Remoulded ■ Lost ◫ Core

SAMPLE TYPE

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Core Sample, gauge
TO	Open Tube
LA	By Washing
TA	Auger
MA	Bulk sample
TF	Split Tube
PW	LVM-Fondatec Mega-Sampler

TESTS

L	Consistancy Limits	M.O.	Organic Matter (%)
W_L	Liquid Limit (%)	K	Permeability (cm/s)
W_P	Plastic Limit (%)	KL	Lefranc Permeability (cm/s)
I_P	Plasticity Index (%)	UW	Unit Weight (kN/m³)
I_L	Liquidity Index	A	Absorption (l/min. m)
W	Natural Water Content (%)	U	Uniaxial Compresses strength (MPa)
AG	Grain Size Analysis	RQD	Rock Quality Designation (%)
S	Hydrometer analysis	AC	Chemical Analysis
R	Refusal	P_L	Limit Pressure (kPa)
P_{80}	Grain Size Analysis by washing at the 80 µm sieve	E_M	Pressuremeter Modulus (kPa)
		E_r	Modulus of subgrade reaction (kPa)

▼	Water Level		
N	Standard Penetration test (blows/150mm)		
Nc	Dynamic Penetration test (blows/300mm) ●		
σ'p	Preconsolidation Pressure (kPa)		
σ'vo	Effective Pressure (kPa)		
Shear Strength		Field	Laboratory
Cu	Undisturbed (kPa)	▲	■
Cur	Remoulded (kPa)	△	□

LITHOLOGY

ELEVATION - m / DEPTH - m	SOILS OR ROCK DESCRIPTION
100.45	Ground level
0.00	Concrete slab
100.30 / 0.15	Coral crush stone of 10 mm diameter backfilled
100.20 / 0.25	
98.93 / 1.52	Coral rock fragments. Fragment size smaller than 5 cm. Interbedding with more massive coral rock.
97.40 / 3.05	Coral rock fragments with voids of approximately 1 cm size created from dissolution process. Fragment sizes of approximately 7-8 cm.
95.88 / 4.57	Coral rock fragments. Average sizes between 5 cm to 8 cm. Powdery material.
94.35 / 6.10	Coral rock fragments with voids of approximately 1 cm in size created by discolution process. Fragment size smaller than 8 cm
93.14 / 7.32	Karst (void)
92.83 / 7.62	Massive coral rock. Corals are very well developped. Discontinuities are very close to closely spaced (average spaces of 8 cm)
91.38 / 9.07	Coral rock fragments. Fragment sizes smaller than 2.5 cm.
88.26 / 12.19	Coral rock fragments. Fragment sizes smaller than 8 cm. Powdery material.
86.73 / 13.72	END OF BOREHOLE

SAMPLES / FIELD AND LABORATORY TESTS

TYPE AND NUMBER	CALIBER	CONDITION	RECOVERY %	BLOWS / 150mm	N or RQD	RESULTS
RC-1		☒	92		50	U = 18 MPa UW = 22.0 kN/m³
RC-2		☒	37		0	
RC-3		☒	53		0	
RC-4		☒	37		0	
RC-5		☒	47		0	
RC-6		☒	83		56	U = 18.5 MPa UW = 14.8 kN/m³
RC-7		☒	38		0	
RC-8		■	0		0	
RC-9		☒	33		0	

WATER LEVEL (m) / DATE

WATER CONTENT AND LIMITS (%) Wp W WL: 20 40 60 80 100

UNDRAINED SHEAR STRENGTH OR DYNAMIC PENETRATION: 20 40 60 80 100

Échelle verticale = 1 : 125

Remarks: - Coral rock of middle reef terraces Formation (MRT)
- Elevation with respect to center point of catch bassin in parking lot (see plan 033-P015952-0160-GE-0001-00) BM = 100.00 m)

Prepared by: David Noël sr. tech.	Approved by: Nancy Verreault, Eng., M.A.Sc.	2008-04-16	Page: 1 of 1

L:\33\Geotec\StyleLog_Forage_dessau_2006_Anglais3.sty

Rapport de forage du : 2008-04-16 12h

DESSAU

BOREHOLE REPORT

Client : D.F.A. I.T. Canada

Project: **Geotechnical investigation - High Commission of Canada**

Location: **Bishop's Court Hill, St-Michael, Barbados**

File n°: **P015952-160**

Borehole n° : **BH-04-08**

Coordinates (m): **1449125.00 N** **218101.00 E**

Dates : **2008-03-03**

Drilling equipement : **Devis, USA**

Reference Datum: **Arbitrary**

Elevation: **100.72 m**

Bedrock depth m

End of borehole depth 12.19 m

Sample condition ▨ Intact ☒ Remoulded ■ Lost ◧ Core

SAMPLE TYPE

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Core Sample, gauge
TO	Open Tube
LA	By Washing
TA	Auger
MA	Bulk sample
TF	Split Tube
PW	LVM-Fondatec Mega-Sampler

TESTS

L	Consistancy Limits
W_L	Liquid Limit (%)
W_P	Plastic Limit (%)
I_P	Plasticity Index (%)
I_L	Liquidity Index
W	Natural Water Content (%)
AG	Grain Size Analysis
S	Hydrometer analysis
R	Refusal
P_{80}	Grain Size Analysis by washing at the 80 µm sieve
M.O.	Organic Matter (%)
K	Permeability (cm/s)
KL	Lefranc Permeability (cm/s)
UW	Unit Weight (kN/m³)
A	Absorption (l/min. m)
U	Uniaxial Compresses strength (MPa)
RQD	Rock Quality Designation (%)
AC	Chemical Analysis
P_L	Limit Pressure (kPa)
E_M	Pressuremeter Modulus (kPa)
E_r	Modulus of subgrade reaction (kPa)

▼	Water Level		
N	Standard Penetration test (blows/150mm)		
Nc	Dynamic Penetration test (blows/300mm) ●		
σ'p	Preconsolidation Pressure (kPa)		
σ'vo	Effective Pressure (kPa)		
Shear Strength		Field	Laboratory
Cu	Undisturbed (kPa)	▲	■
Cur	Remoulded (kPa)	△	☐

LITHOLOGY

ELEVATION - m / DEPTH - m	SOILS OR ROCK DESCRIPTION
100.72	Ground level
0.00	Grass and topsoil
100.57 0.15	Massive coral rock with very close to closely spaced discontinuities (average spacing of approximately 6 cm)
97.04 3.68	Coral rock fragments. Size of fragments smaller than 5 cm.
95.69 5.03	Karst (void)
94.32 6.40	Coral rock fragments. Size of fragments smaller than 2.5 cm.
93.10 7.62	Coral rock fragments with voids created from discolution process. Powdery material.
91.58 9.14	Coral rock fragments with voids created from dissolution process. Interbedding with massive coral rock with well developped coral layers of approximately 10 cm in length.
88.53 12.19	END OF BOREHOLE

SAMPLES / FIELD AND LABORATORY TESTS

TYPE AND NUMBER	CALIBER	CONDITION	RECOVERY %	BLOWS / 150mm	N or RQD	RESULTS
RC-1		☒	38		7	
RC-2		☒	97		68	U = 10 MPa UW = 22.1 kN/m³
RC-3		☒	42		22	
RC-4		☒	0		0	
RC-5		☒	20		0	
RC-6		☒	25		0	
RC-7		☒	25		7	
RC-8		☒	40		0	

WATER CONTENT AND LIMITS (%) Wp W WL — 20 40 60 80 100

UNDRAINED SHEAR STRENGTH OR DYNAMIC PENETRATION — 20 40 60 80 100

Échelle verticale = 1 : 125

Remarks: - Coral rock of middle reef terraces Formation (MRT)
- Elevation with respect to center point of catch bassin in parking lot (see plan 033-P015952-0160-GE-0001-00) BM = 100.00 m)

Prepared by: David Noël sr. tech.

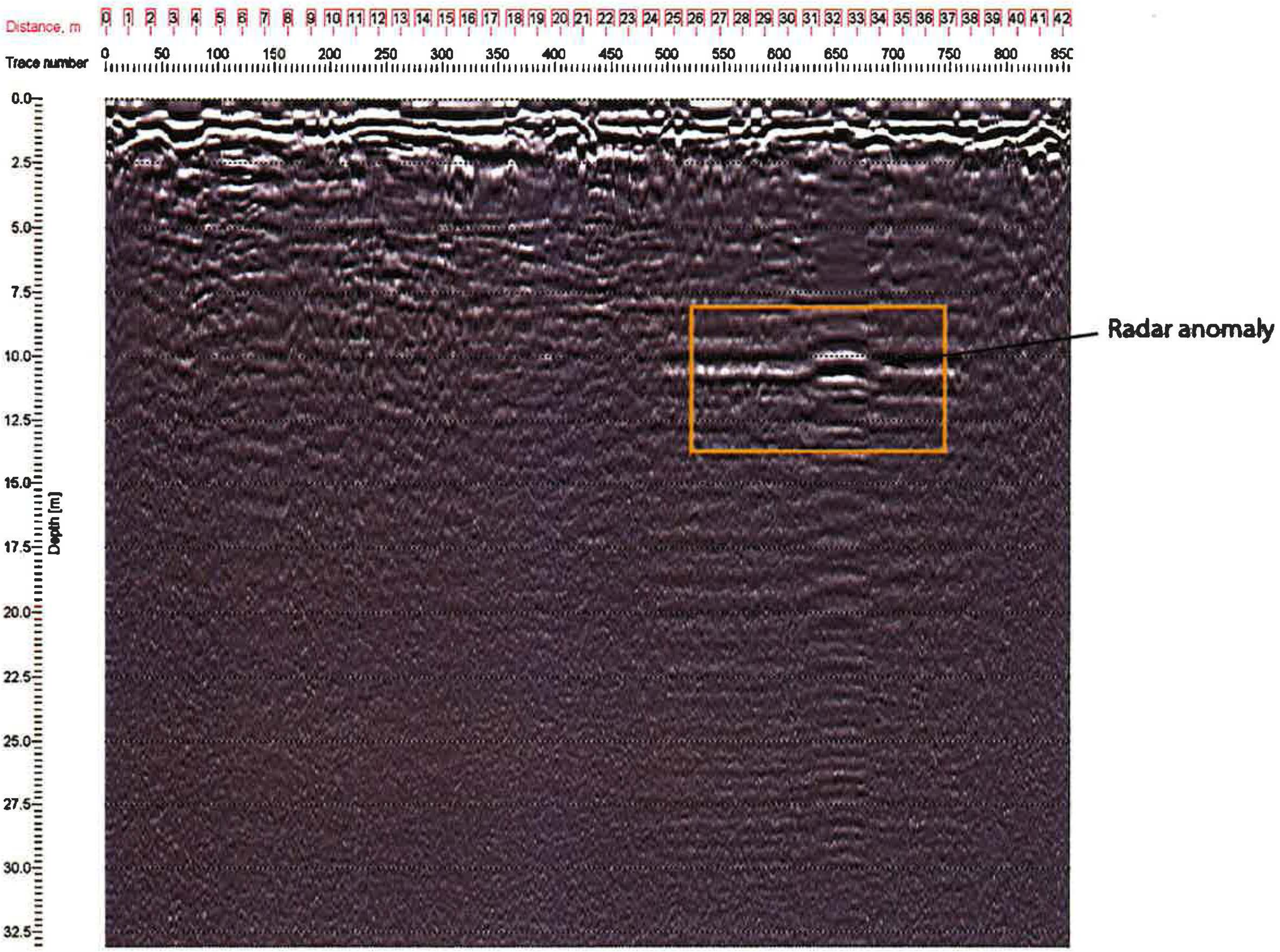
Approved by: Nancy Verreault, Eng., M.A.Sc. 2008-04-16

Page: 1 of 1

L:\33\Geotec\StyleLog_Forage_dessau_2006_Anglais3.sty

HYDROGÉO-SOL

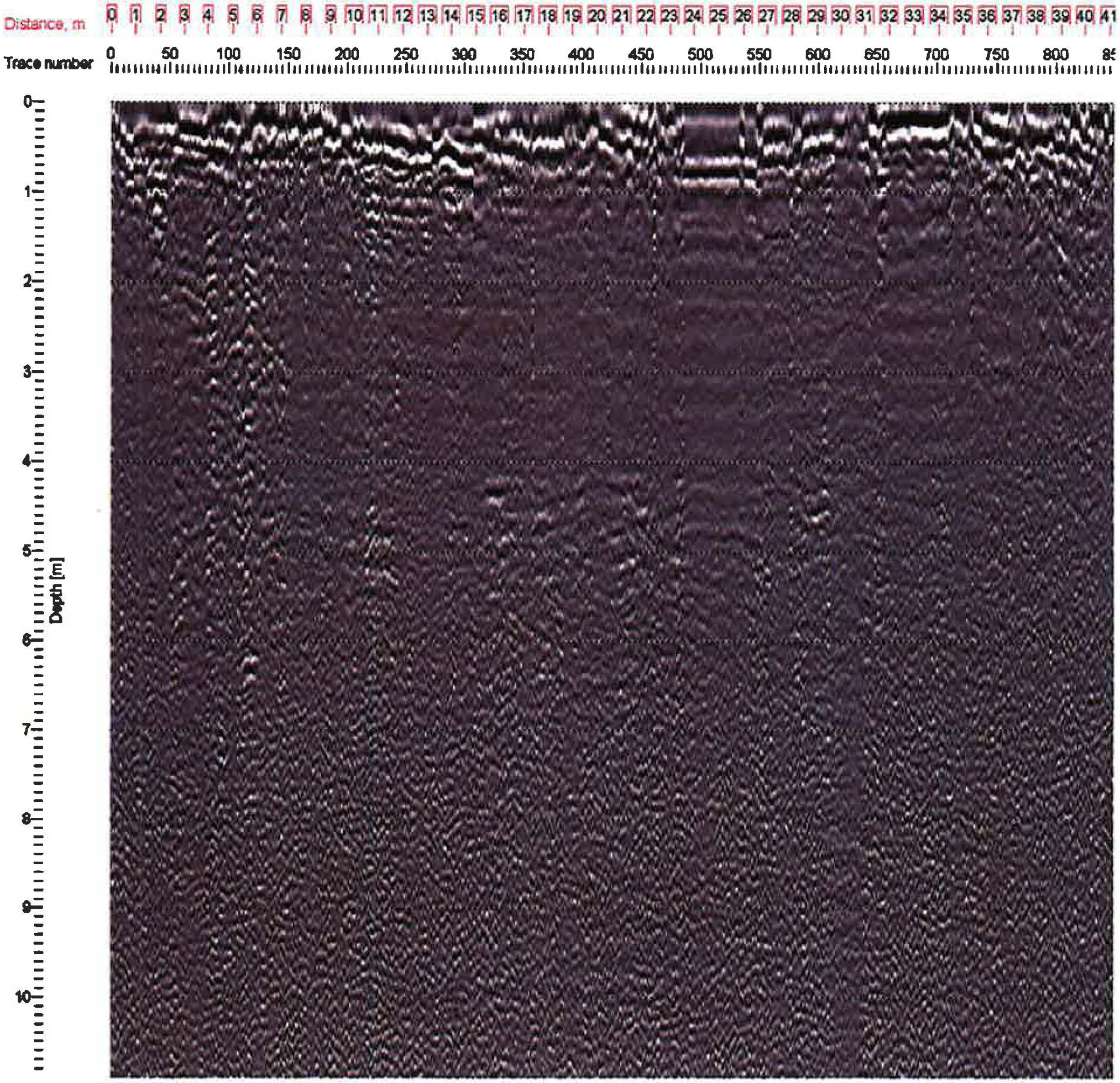
Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-16
Antenna type:	100 MHz
Distance between antennas:	0,50 m
Date:	20 february 2008



Radar anomaly

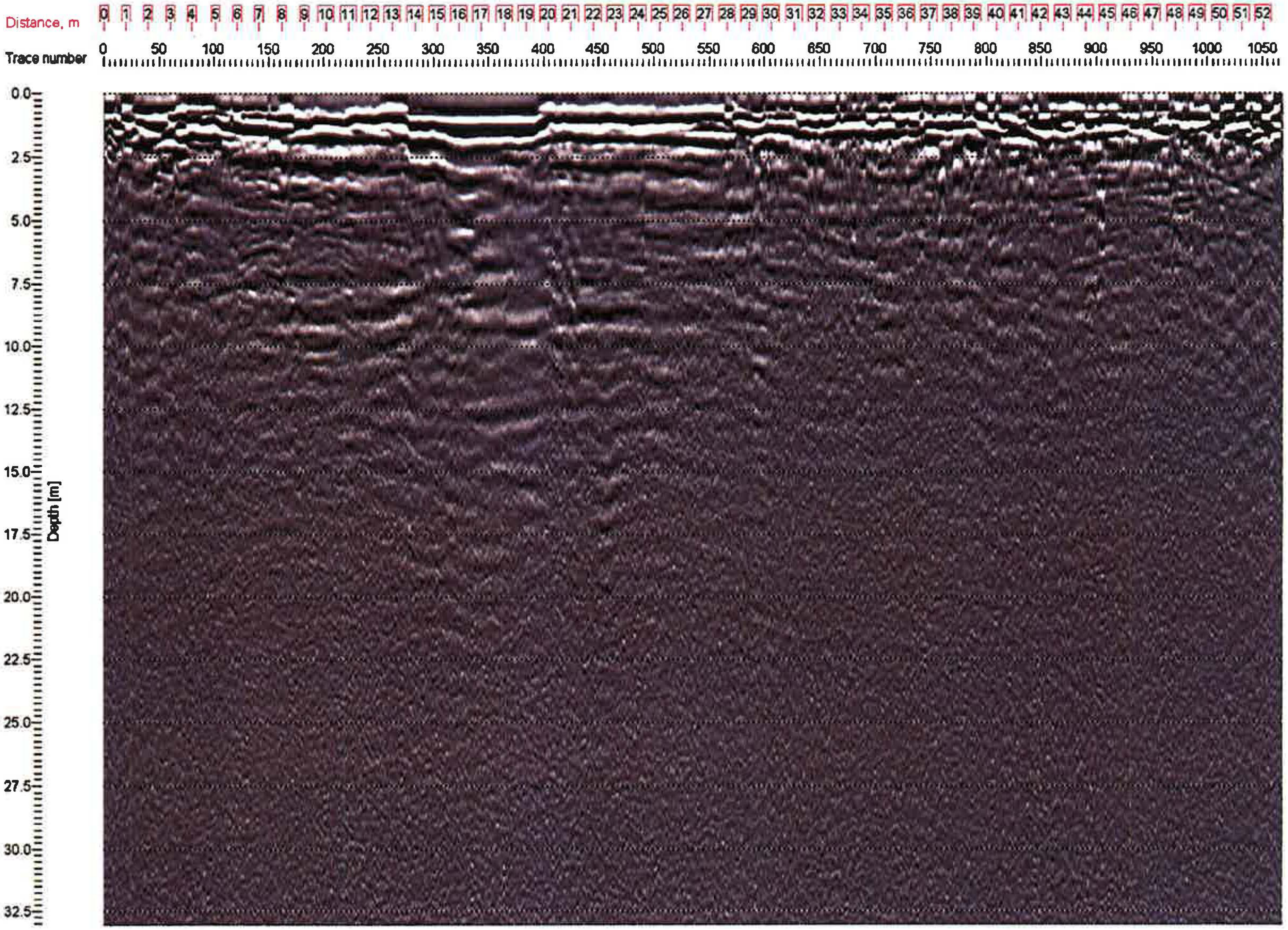
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-16
Antenna type:	250 MHz
Distance between antennas:	0,31 m
Date:	20 february 2008



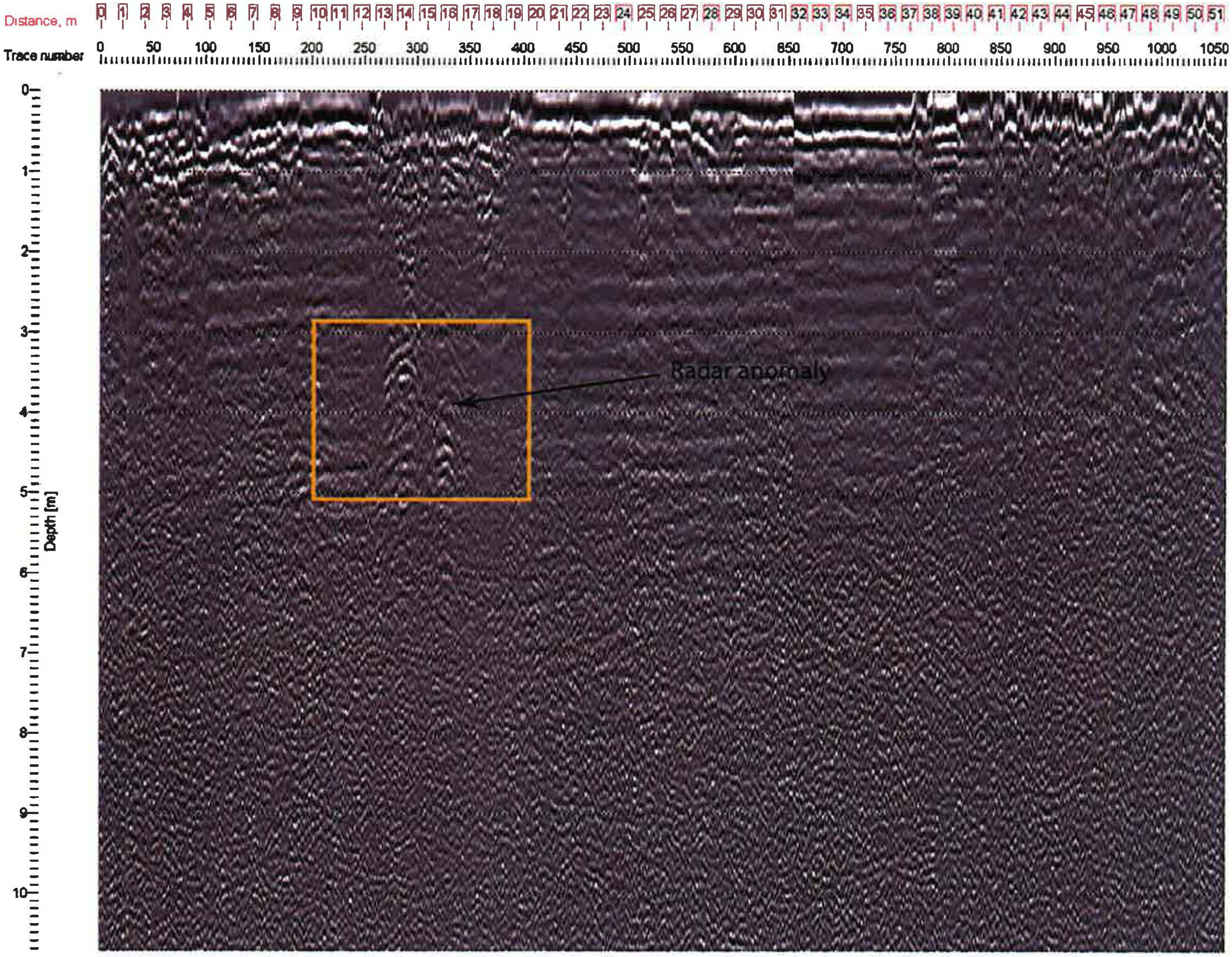
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-17
Antenna type:	100 MHz
Distance between antennas:	0,50 m
Date:	20 february 2008



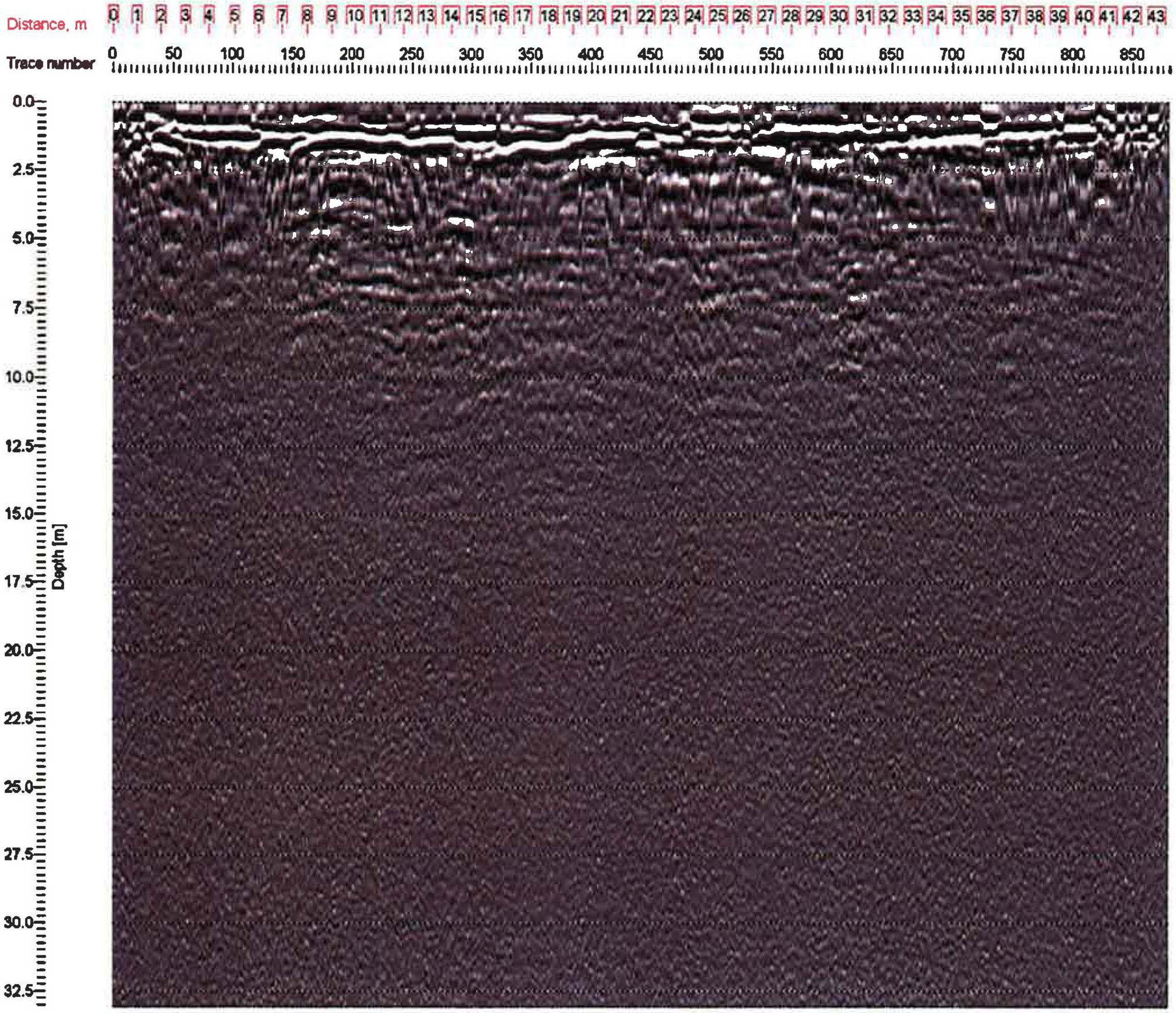
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-17
Antenna type:	250 MHz
Distance between antennas:	0,31 m
Date:	20 february 2008



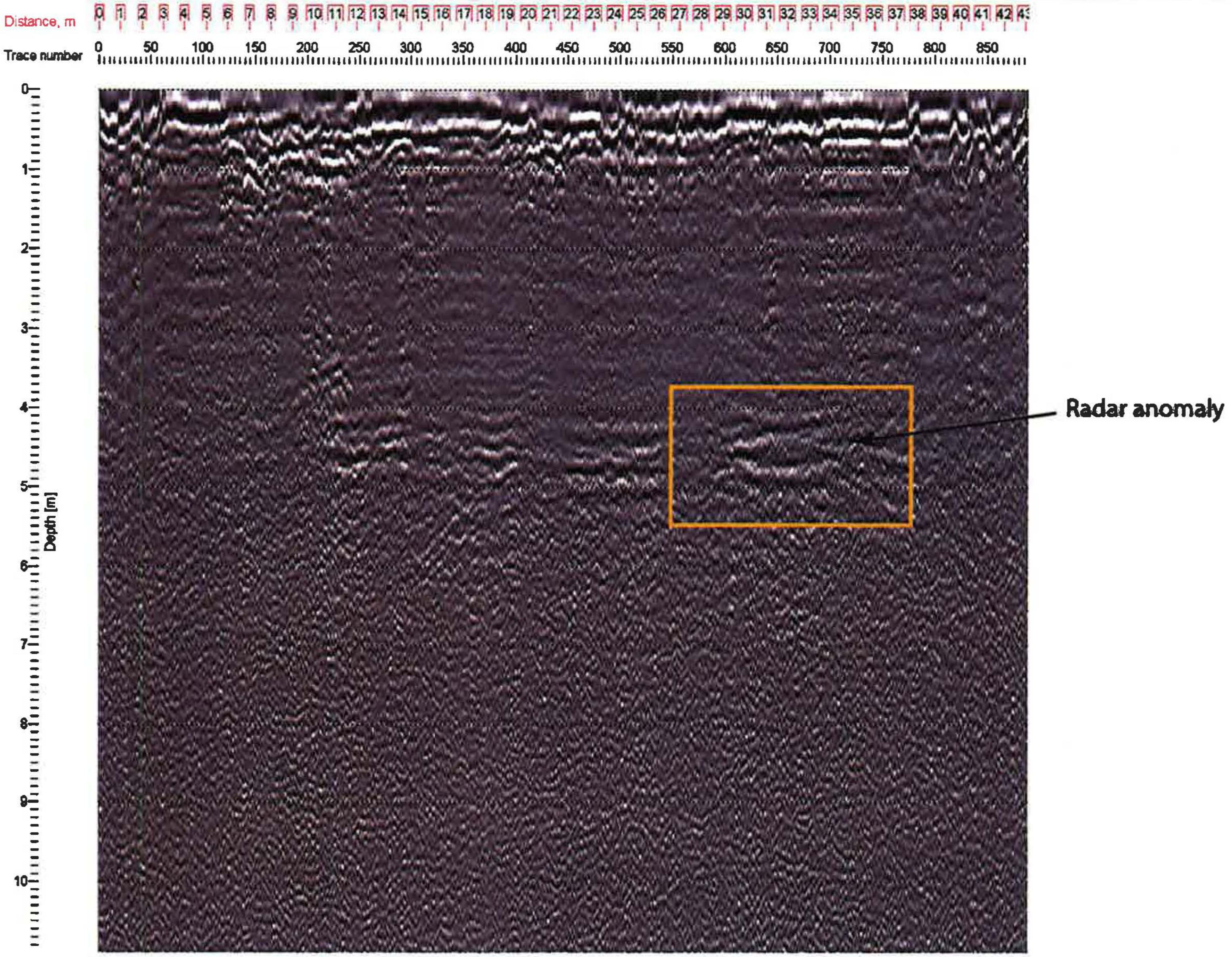
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-18
Antenna type:	100 MHz
Distance between antennas:	0,50 m
Date:	20 february 2008



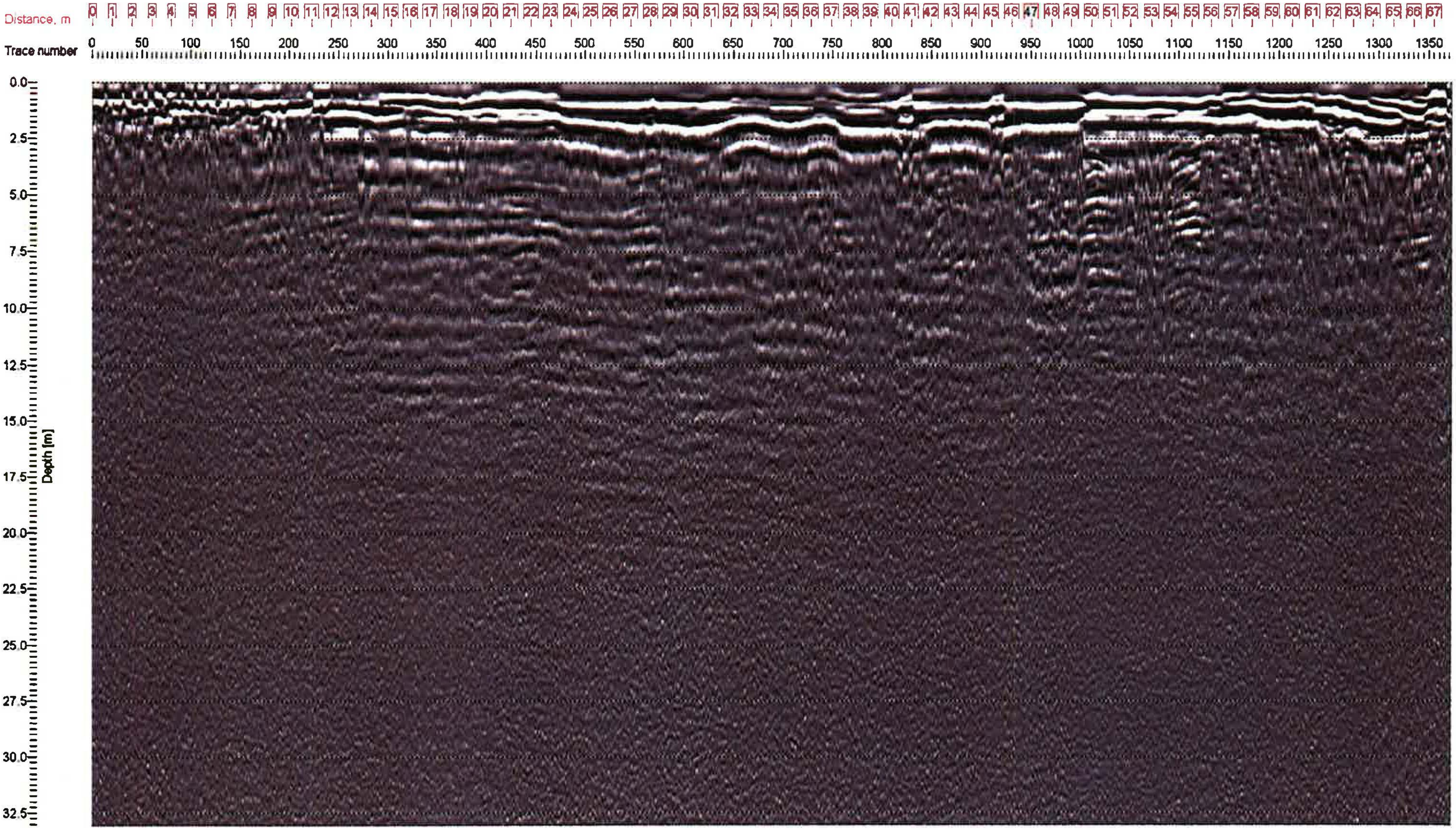
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-18
Antenna type:	250 MHz
Distance between antennas:	0,31 m
Date:	20 february 2008



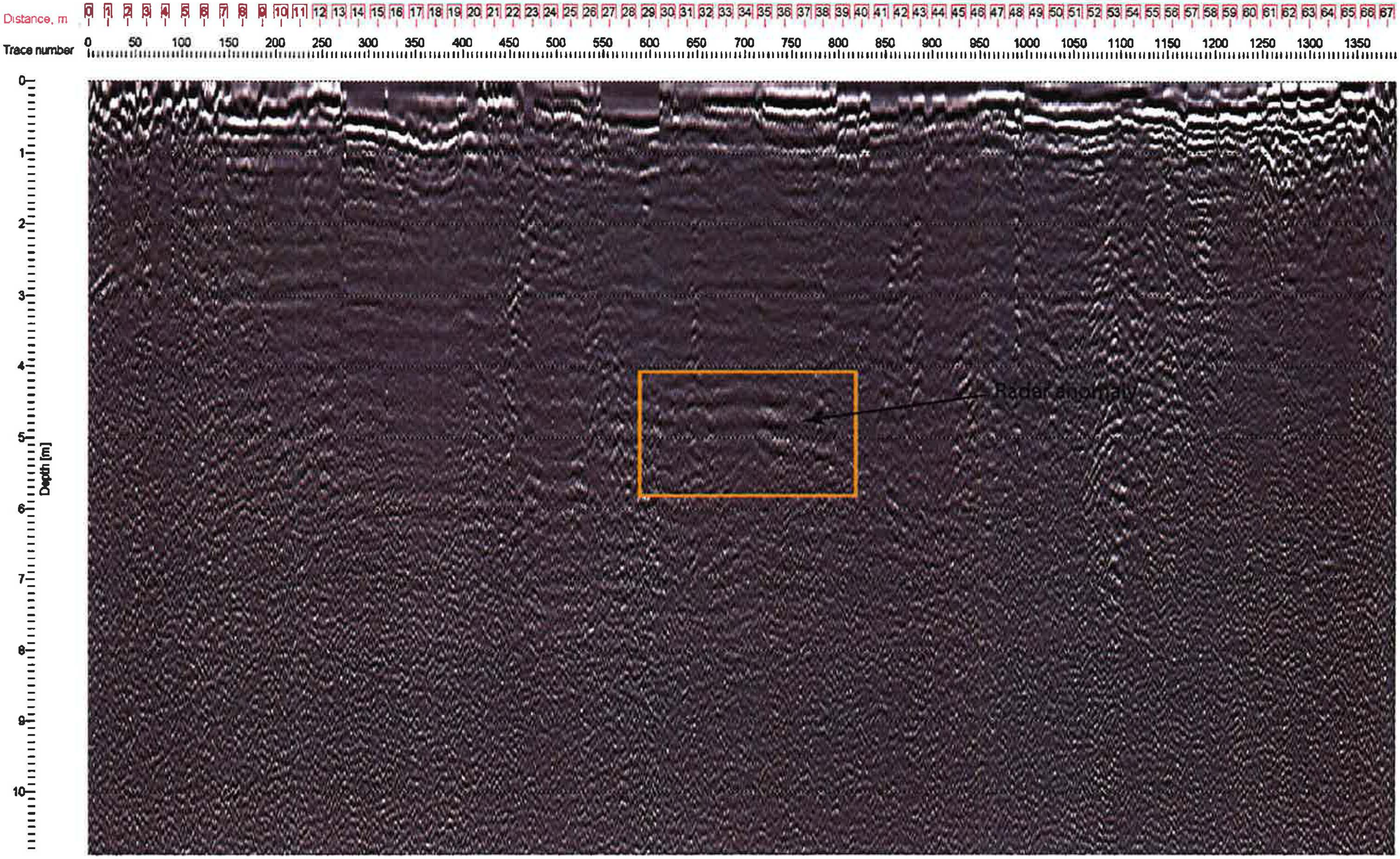
HYDROGÉO-SOL

Place: High Commission of Canada, Barbados
Project: Ground penetrating radar data
053-P015952-0170-SC-0001-00
Line: L-19
Antenna type: 100 MHz
Distance between antennas: 0,50 m
Date: 20 february 2008



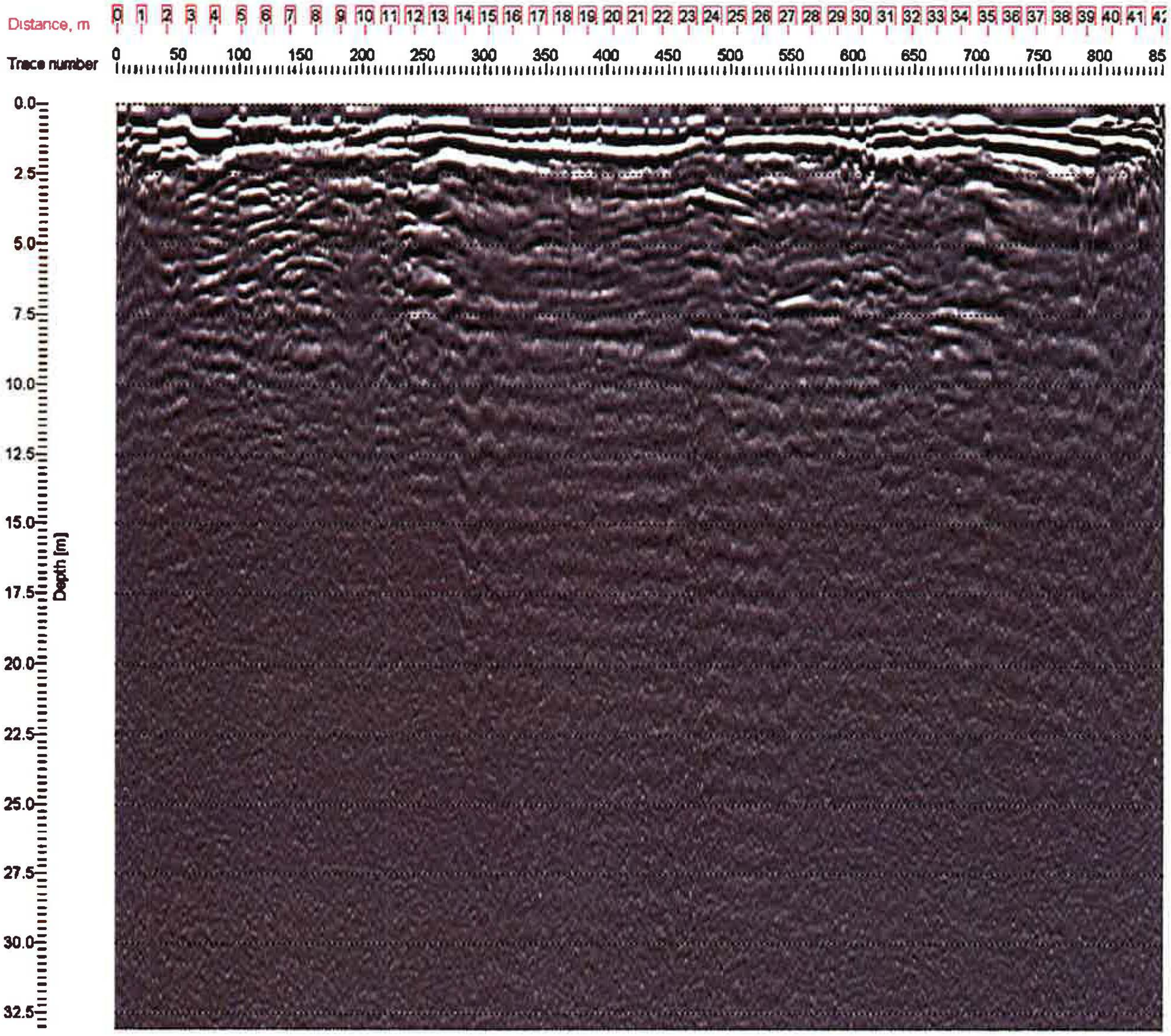
HYDROGÉO-SOL

Place: High Commission of Canada, Barbados
Project: Ground penetrating radar data
053-P015952-0170-SC-0001-00
Line: L-19
Antenna type: 250 MHz
Distance between antennas: 0,31 m
Date: 20 february 2008



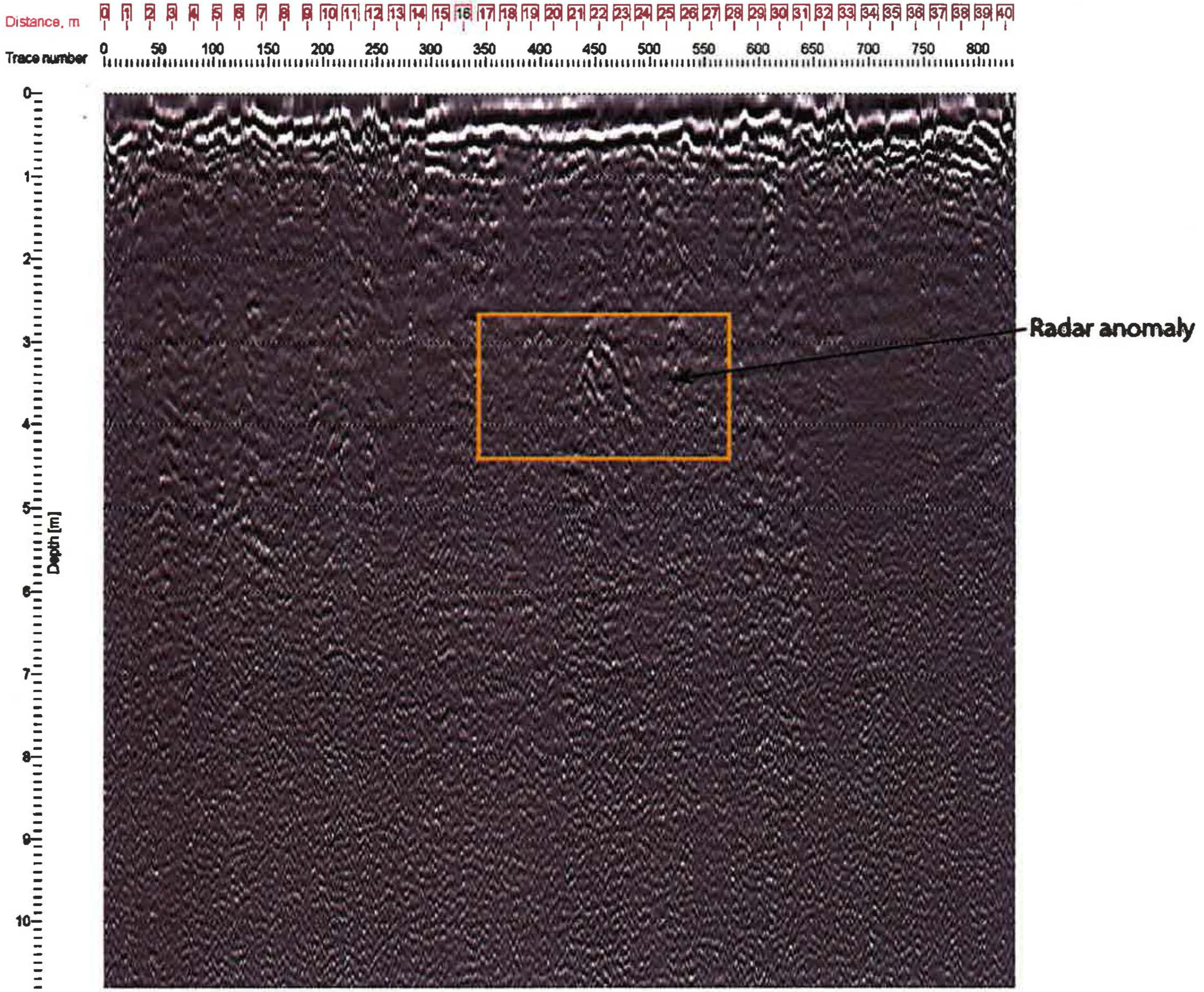
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-20
Antenna type:	100 MHz
Distance between antennas:	0,50 m
Date:	20 february 2008



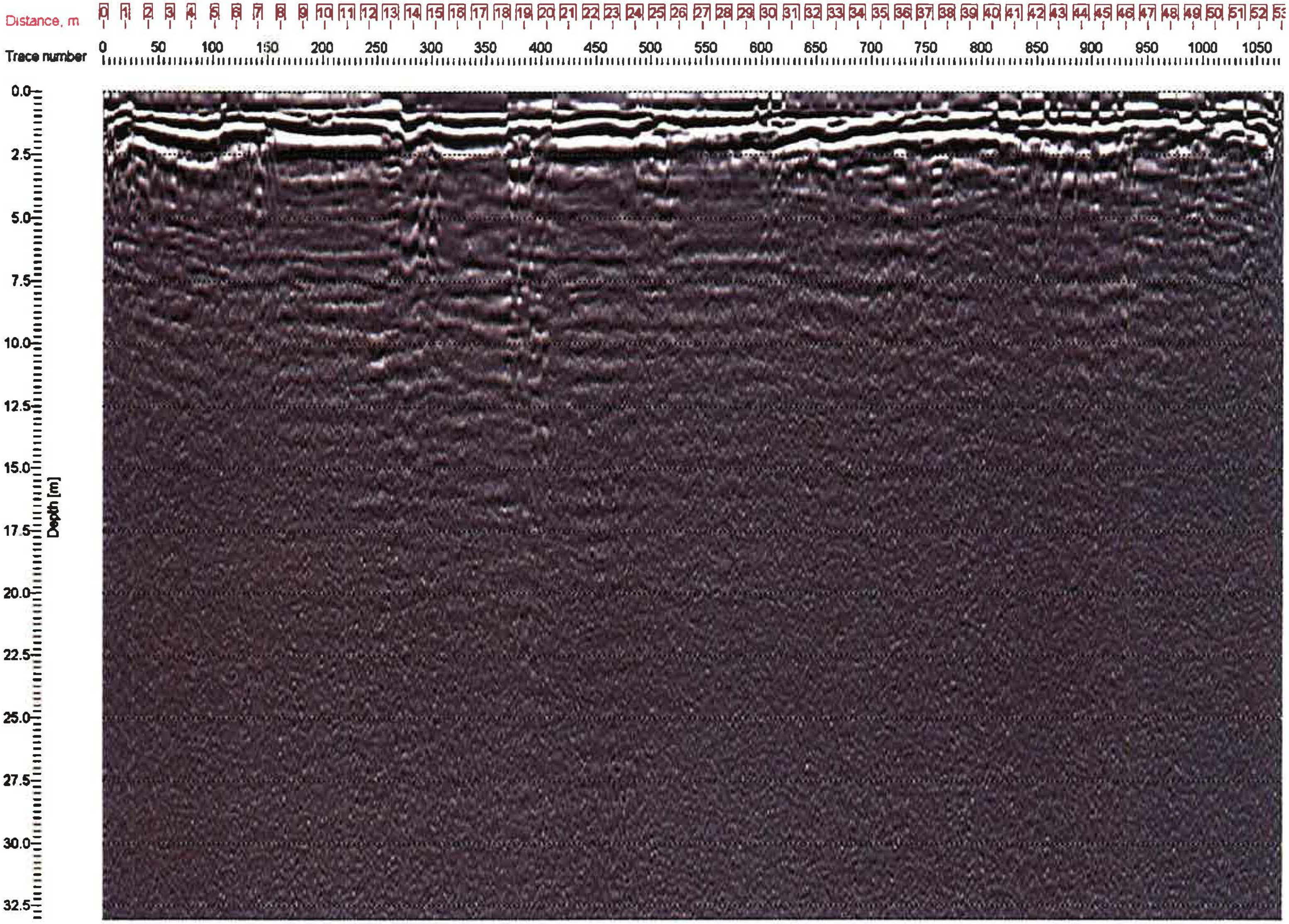
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-20
Antenna type:	250 MHz
Distance between antennas:	0,31 m
Date:	20 february 2008



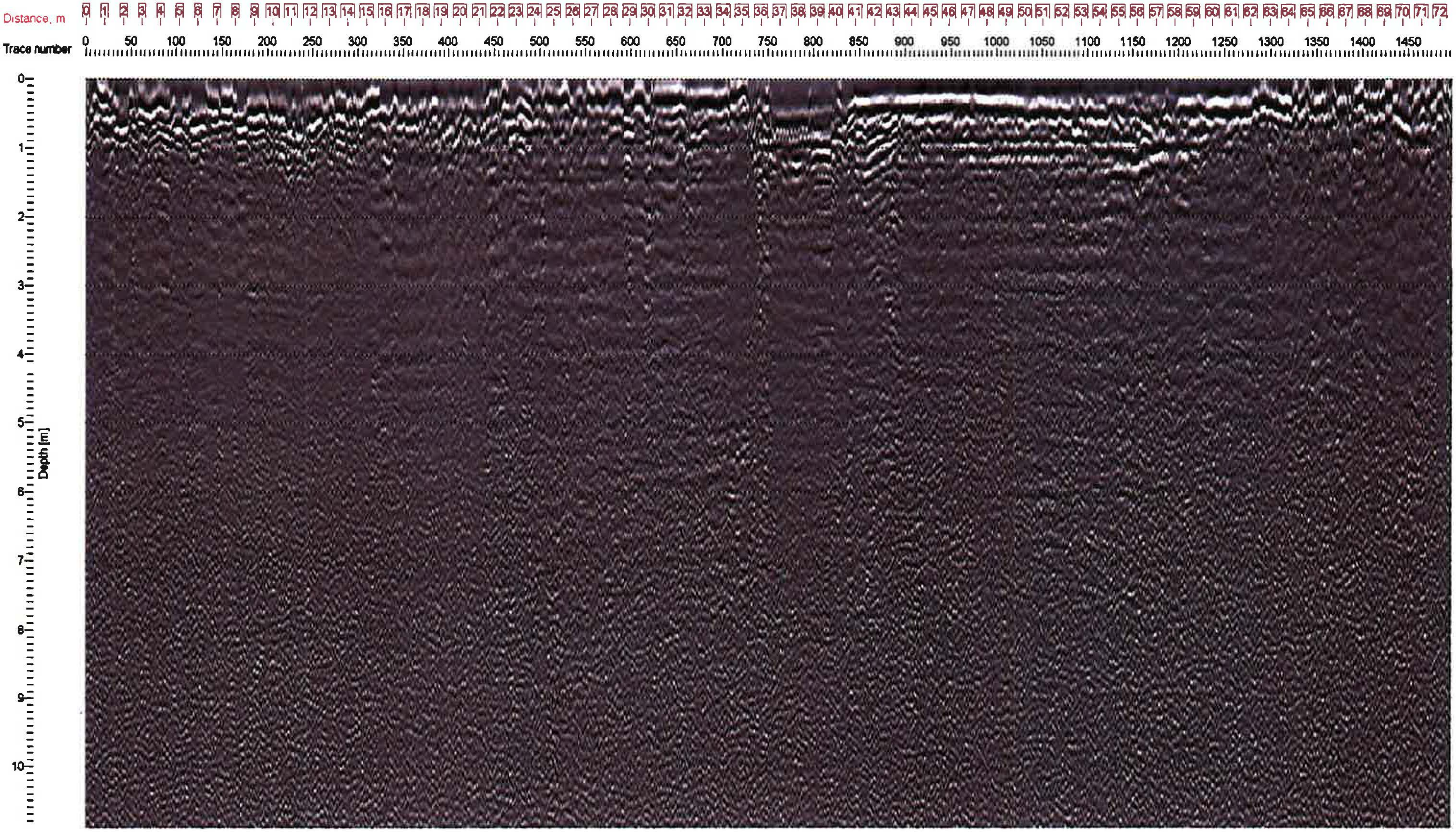
Radar anomaly

Place: High Commission of Canada, Barbados
Project: Ground penetrating radar data
053-P015952-0170-SC-0001-00
Line: L-21
Antenna type: 100 MHz
Distance between antennas: 0,50 m
Date: 20 february 2008



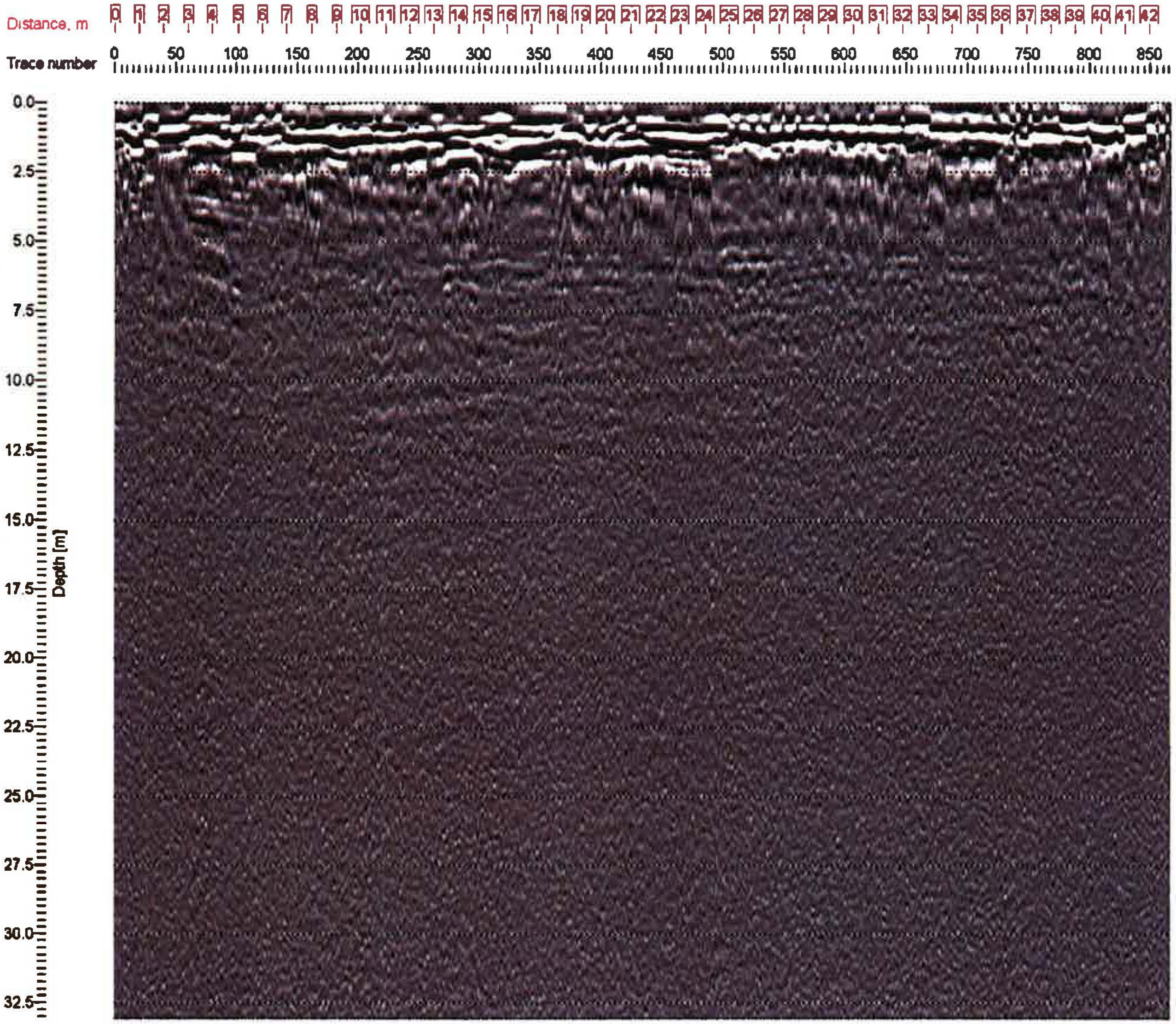
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-21
Antenna type:	250 MHz
Distance between antennas:	0,31 m
Date:	20 february 2008



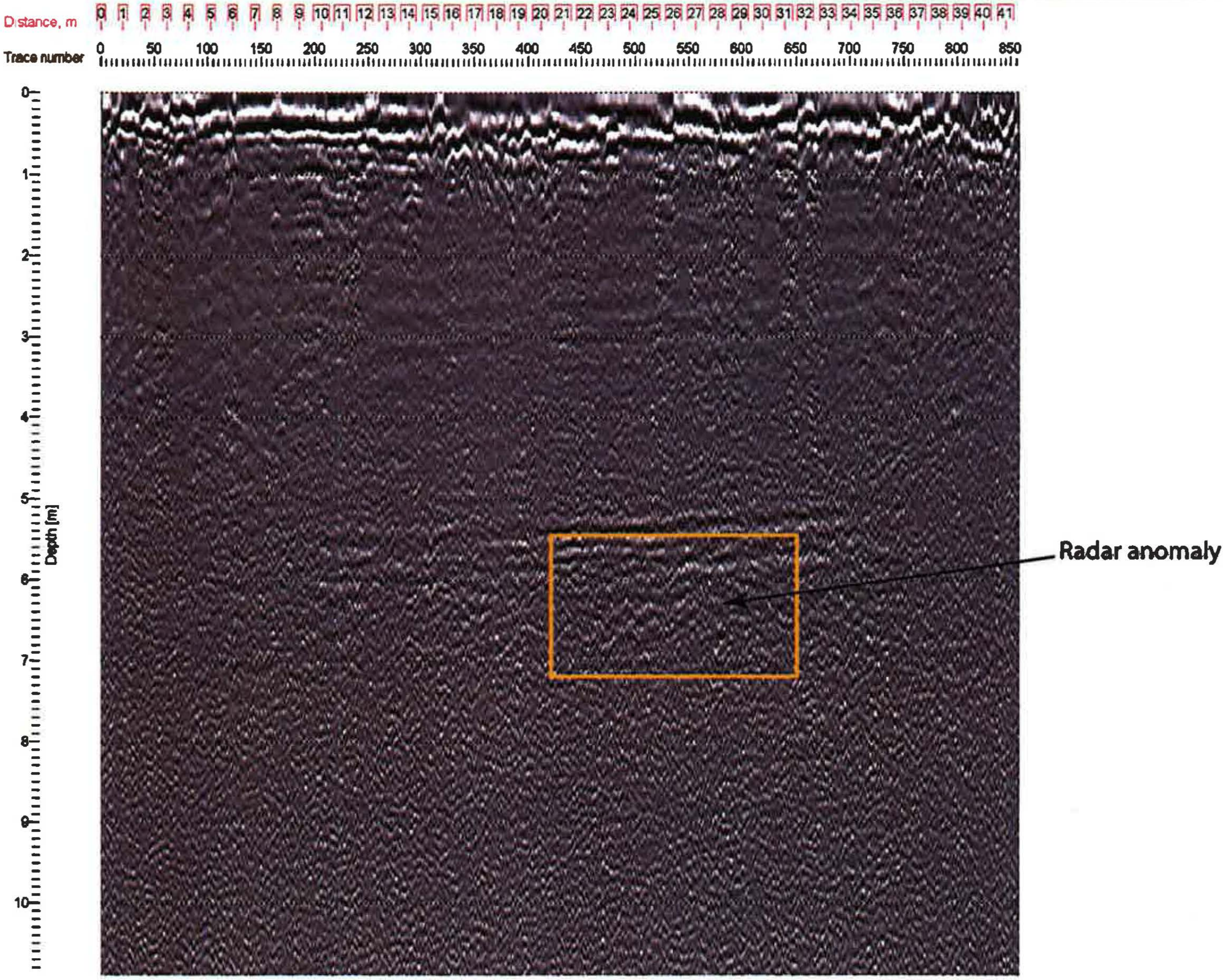
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-22
Antenna type:	100 MHz
Distance between antennas:	0,50 m
Date:	20 february 2008



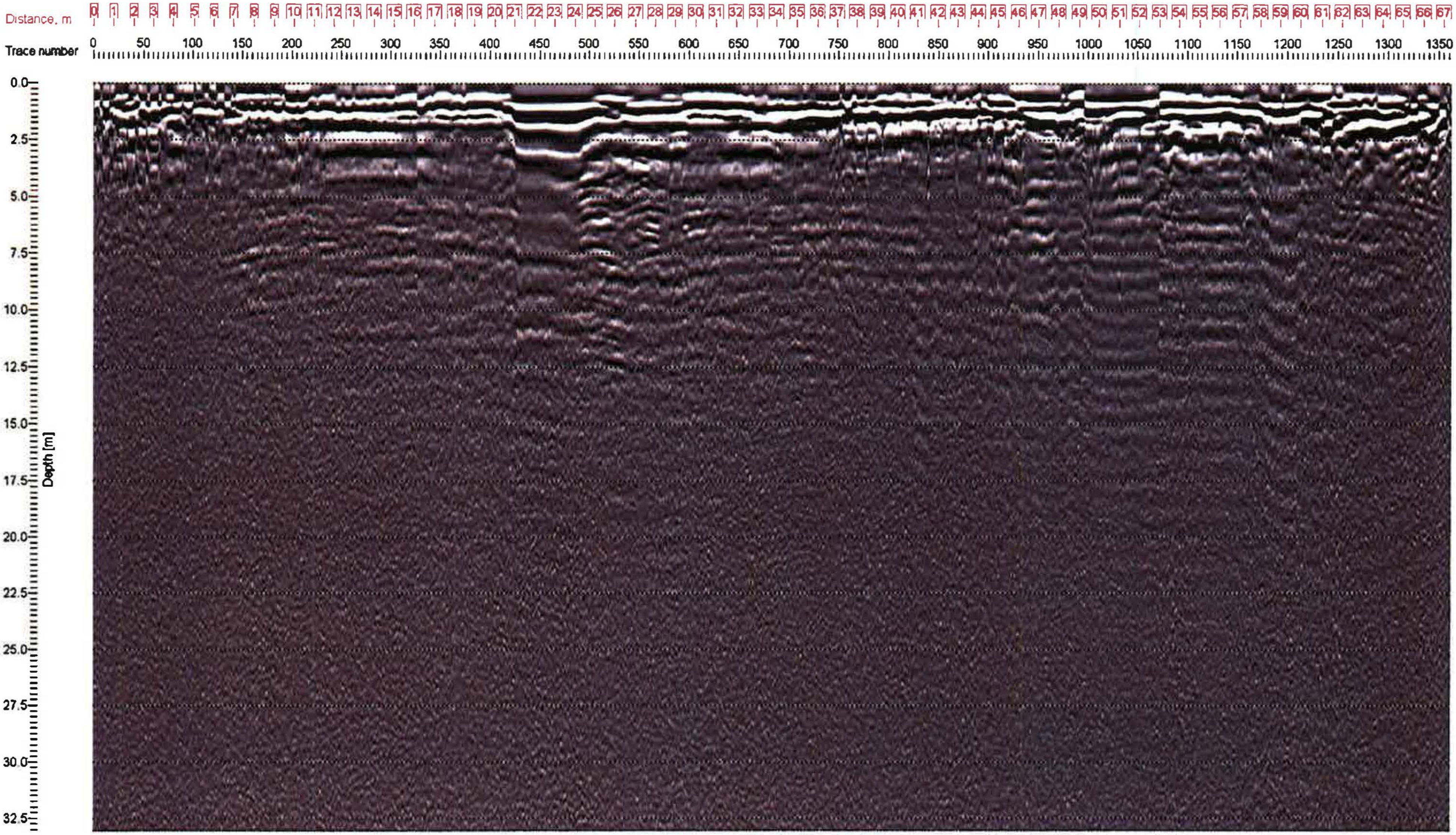
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-22
Antenna type:	250 MHz
Distance between antennas:	0,31 m
Date:	20 february 2008



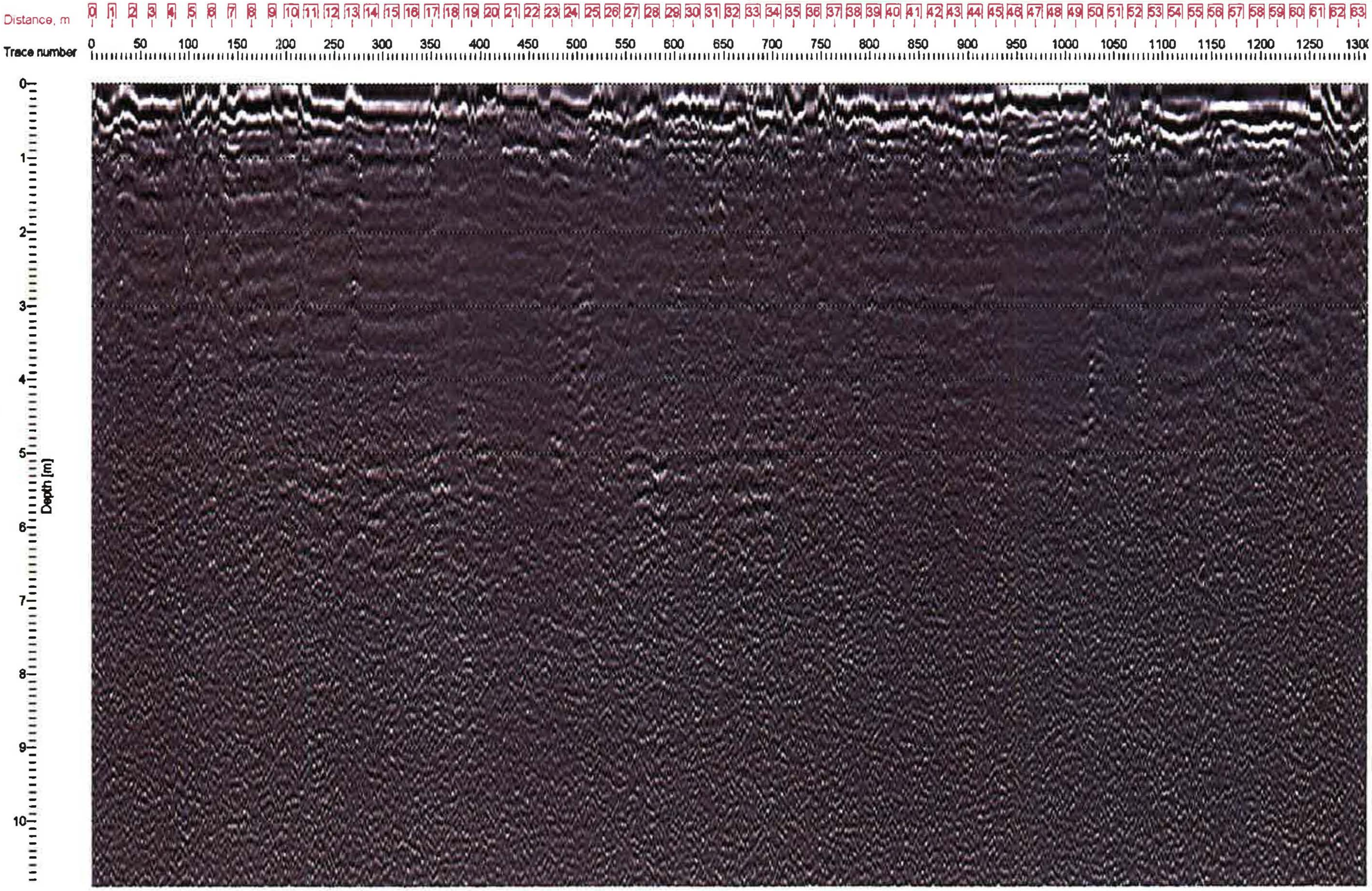
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-23
Antenna type:	100 MHz
Distance between antennas:	0,50 m
Date:	20 february 2008



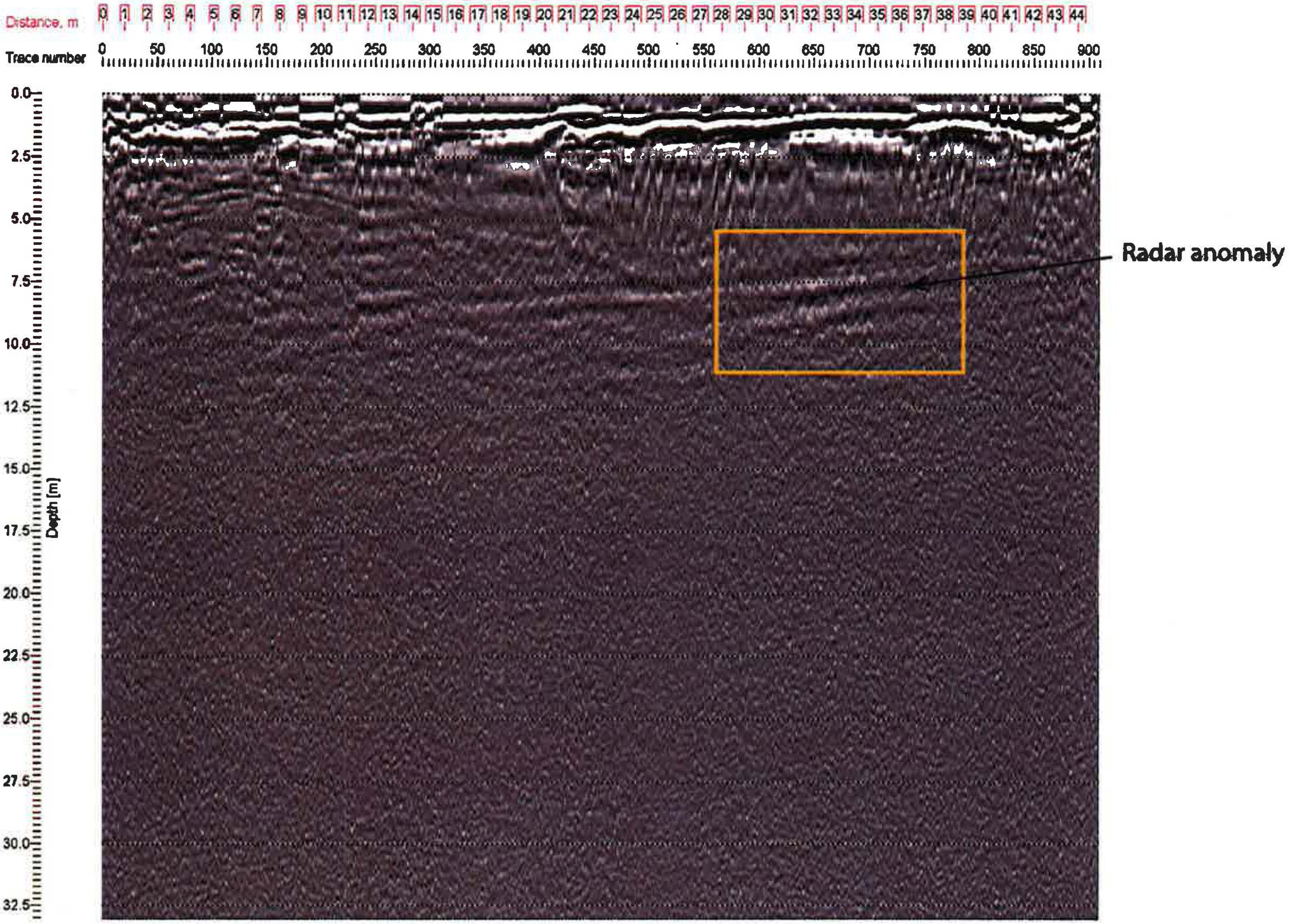
HYDROGÉO-SOL

Place: High Commission of Canada, Barbados
Project: Ground penetrating radar data
053-P015952-0170-SC-0001-00
Line: L-23
Antenna type: 250 MHz
Distance between antennas: 0,31 m
Date: 20 february 2008



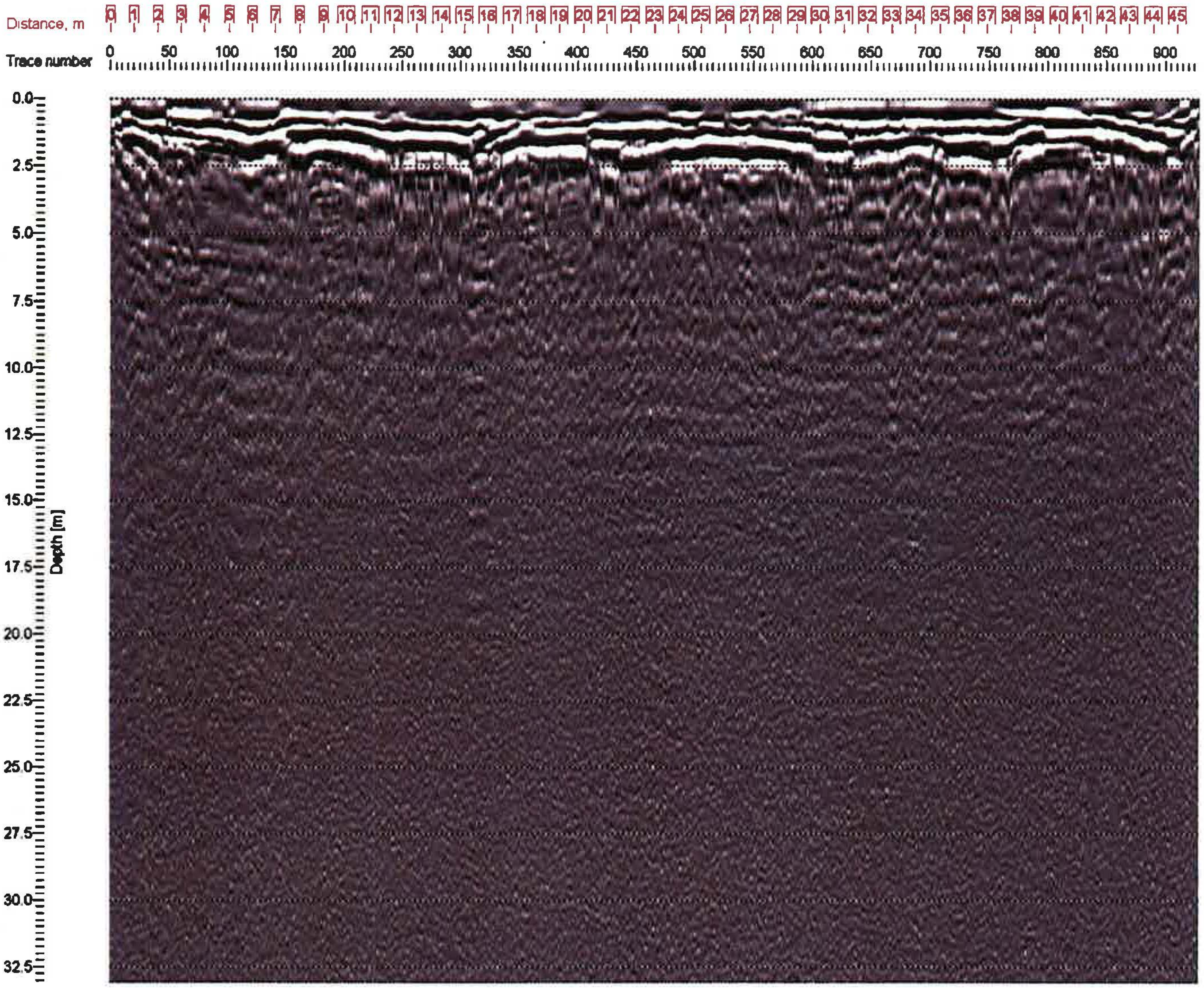
HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-24
Antenna type:	100 MHz
Distance between antennas:	0,50 m
Date:	20 february 2008



HYDROGÉO-SOL

Place:	High Commission of Canada, Barbados
Project:	Ground penetrating radar data 053-P015952-0170-SC-0001-00
Line:	L-25
Antenna type:	100 MHz
Distance between antennas:	0,50 m
Date:	20 february 2008



APPENDIX D

Laboratory Test Results
Current Investigation

Golder Associates Ltd.
32 Steacie Drive
Kanata, Ontario
K2K 2A9

UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: Barbados - Bridgetown - Chancery

Project No.: 10-1121-0089 / 300

Client: J.L. Richards and Associates

Date: April 11, 2011

Rock Description : Coral

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)
C-11-1	1.25-1.35	Mar 23/11	NQ	50.3	1814	10.5
C-11-1	9.91-10.05	Mar 23/11	NQ	49.9	1268	4.7
C-11-2	1.37-1.52	Mar 24/11	NQ	50.5	2287	39.3
C-11-2	17.37-17.48	Mar 24/11	NQ	50.6	2188	51.6
C-11-3	1.29-1.42	Mar 24/11	NQ	50.2	1801	9.4
C-11-3	2.24-2.36	Mar 24/11	NQ	50.3	1794	9.2
C-11-4	11.58-11.73	Mar 24/11	NQ	50.9	1709	19.5
C-11-5	3.66-3.81	Mar 24/11	NQ	49.3	2054	25.0

REMARKS : - Compressive Strength Corrected for L/D Ratio.
- Cores tested in vertical direction.

TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH ASTM D7012 - Method C

SIGNED: C.N.Mangione P.Eng.

POINT LOAD INDEX TEST WORKSHEET

Using modified Marshall apparatus (non-MTO) (ASTM D5731-05)

Project Number: 10-1121-0089 / 300

Date: March 23, 2011

Name: Barbados - Chancery

Core Size: NQ

Diam. : 47.6 mm

Rock Descriptions :

D - Dolomite Sn - Sandstone
L - Limestone G - Granite
M - Marble C - Coral

Moisture Condition :

___ Saturated
_X_ As Received
___ Lab Air Dried
___ Oven Dried

BOREHOLE	DEPTH (m)	Diametral / Axial (D / A)	LENGTH (Axial) DIAM (Diametral) (mm)	EQUIVALENT CORE DIAMETER D_e	FAILURE READING (div)	FAILURE LOAD (lbs)	I_s (MPa)	SIZE CORRECTION FACTOR " F "	$I_{s(50)}$ (MPa)	UNIAXIAL COMPRESSIVE STRENGTH (MPa) ** [$C \times I_{s(50)}$]	Rock Descr.	Moisture content		
												"Moist" Mass (g)	Oven-Dry Mass (g)	W_{nat} (g)
C 11-1	1.37-1.52	D	45.3	45.3	80	854	1.9	0.96	1.8	12	C			
	9.75-9.91	D	42.8	42.8	35	363	0.9	0.93	0.8	6	C			
C 11-2	1.07-1.22	D	49.7	49.7	135	1428	2.6	1.00	2.6	18	C			
	2.44-2.47	D	44.4	44.4	175	1820	4.1	0.95	3.9	27	C			
C 11-3	0.91-1.22	D	48.7	48.7	200	2074	3.9	0.99	3.8	27	C			
C 11-4	11.28-11.38	D	44.3	44.3	83	886	2.0	0.95	1.9	13	C			
	12.80-12.92	D	46.9	46.9	60	635	1.3	0.97	1.2	9	C			
	17.37-17.50	D	43.9	43.9	30	310	0.7	0.94	0.7	5	C			
C 11-5	7.01-7.13	D	44.5	44.5	61	646	1.5	0.95	1.4	9	C			
	14.02-14.14	D	45.0	45.0	150	1579	3.5	0.95	3.3	23	C			
C 11-6	2.90-3.05	D	43.9	43.9	206	2135	4.9	0.94	4.6	32	C			
	15.09-15.24	D	43.5	43.5	115	1224	2.9	0.94	2.7	19	C			

** : Correlation factor "C" found to be approx. 6.9 based on adjacent UCS testing

Calc By : CW
Checked By :

POINT LOAD INDEX TEST - DENSITY WORKSHEET

Unit Weight based on approximate dimension measurements

Project Number: 10-1121-0089 / 300

Date: 23-Mar-11

Name: Barbados - Chancery

Core Size: NQ

Diam. : 47.6 mm

Rock Descriptions :

D - Dolomite	Sn - Sandstone
L - Limestone	G - Granite
M - Marble	C - Coral

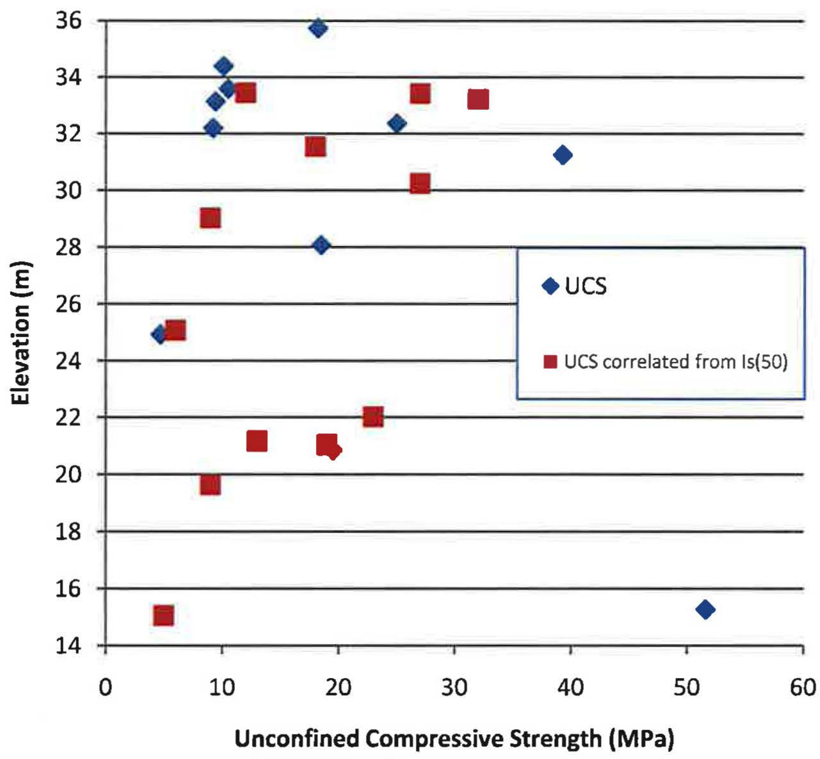
Moisture Condition :
- ___ Saturated
- _X_ As Received
- ___ Lab Air Dried
- ___ Oven Dried

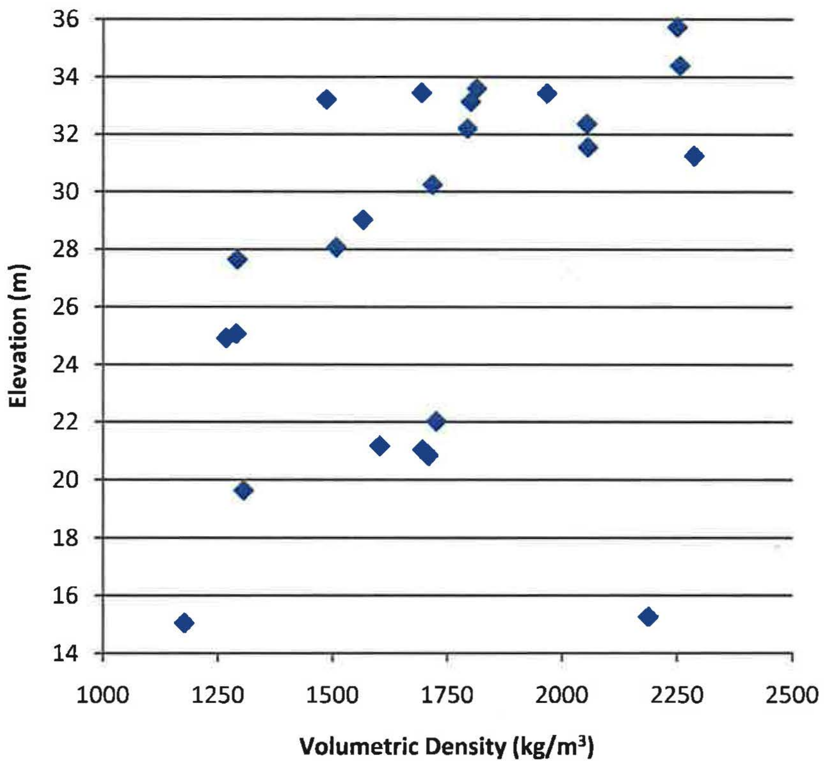
BOREHOLE	DEPTH (m)	AVERAGE DIAMETER (mm)	APPROX. LENGTH (mm)	MASS (g)	APPROX. DENSITY (kg/m³)	ROCK DESCRIPTION
C 11-1	1.37-1.52	50.1	140	467.4	1694	C
	9.75-9.91	50.1	175	445.0	1290	C
C 11-2	1.07-1.22	50.3	175	715.1	2056	C
	2.44-2.47	50.5	100	344.2	1718	C
C 11-3	0.91-1.22	49.4	131	493.8	1967	C
C 11-4	11.28-11.38	50.1	175	552.9	1603	C
	12.80-12.92	50.2	150	387.7	1306	C
	17.37-17.50	49.6	120	273.0	1177	C
C 11-5	7.01-7.13	48.4	215	619.5	1566	C
	14.02-14.14	48.6	144	461.0	1726	C
C 11-6	2.90-3.05	50.5	135	401.7	1486	C
	15.09-15.24	51.2	105	366.7	1696	C

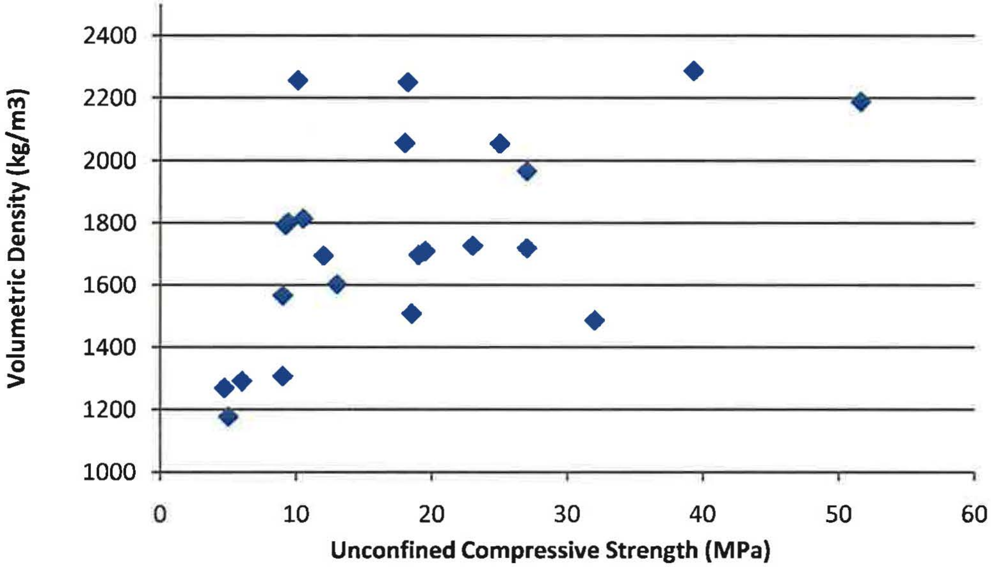
Golder Associates Ltd.

Calc By : CW

Checked By :







GEOTECHNICAL INVESTIGATION FOR CANADIAN
HIGH COMMISSION CHANCERY
BRIDGETOWN, BARBADOS

SUMMARY OF UNCONFINED COMPRESSIVE STRENGTHS AND VOLUMETRIC DENSITIES

Golder Associates

PROJECT No.	10-1121-0089		PHASE No. 1400	
DESIGN	BDG	17/02/2011	SCALE NTS	REV.
CADD			FIGURE D1	
CHECK	CMM			
REVIEW	MSS			