Geotechnical Investigation

5640 Bank Street, 7107 Marco Street, and 7041 Mitch Owens Road, City of Ottawa, Ontario

Ref: BAE-1241.3

Prepared For

Alium Investments Ltd.

August 9th, 2013
(Updated April, 2014)

BAE & Associates Environmental Inc.
RR#1 Oro Station, ON L0L 2E0
Phone 705 715 1881
Fax 705 487 5600
envsol@rogers.com

Providing Environmental Solutions Since 1997!

TABLE OF CONTENTS

APPENDICES

LIST OF ACRONYMS AND ABBREVIATIONS

ACM	Asbestos Containing Materials
AEC	Area of Environmental Concern
a.k.a.	Also Known As
APEC	Area of Potential Environmental Concern
AST	Aboveground Storage Tank
BH	Borehole
BTEX	Benzene, Toluene, Ethylbenzene and Xylenes
CCEA	Central Canada Exhibition Association
CFC	Chlorofluorocarbon
CNSC	Canadian Nuclear Safety Commission
COC	Contaminant of Concern
COPC	Contaminant of Potential Concern
CSA	Canadian Standards Association
CSFL	Contaminated Site on Federal Land
CWAC	Canadian Women’s Army Corporation
CWS	Canada Wide Standards
DSS	Designated Substance Survey
DSHMS	Designated Substance and Hazardous Materials Survey
ESA	Environmental Site Assessment
FIP	Fire Insurance Plan
FOI	Freedom of Information
HCFC	Hydro chlorofluorocarbon
HLUI	Historical Land Use Inventory
HVAC	Heating Ventilation and Air Conditioning
LCP	Lead-Containing Paint
masl	Metres Above Sea Level
mbgs	Metres Below Ground Surface
MOE	Ministry of the Environment
MOL	Ministry of Labour
MSDS	Material Safety Data Sheet
MW	Monitoring Well
ODS	Ozone Depleting Substance
OHSA	Occupational Health and Safety Act
PAH	Polycyclic Aromatic Hydrocarbon
PCB	Polychlorinated Biphenyls
PHC	Petroleum Hydrocarbon
RSC	Record of Site Condition
SAR	Sodium Absorption Ratio
SCS	Site Condition Standard
TPH	Total Petroleum Hydrocarbons
TSSA	Technical Standards and Safety Authority
UST	Underground Storage Tank
UFFI	Urea Formaldehyde Foam Insulation
VOC	Volatile Organic Compounds

1.0 INTRODUCTION

1.1 Purpose

This report presents the results of a Geotechnical Investigation prepared by BAE & Associates for the proposed commercial development at 5640 Bank Street, 7107 Marco Street, and 7041 Mitch Owens Road, City of Ottawa (Greely), for Alium Investments Inc. The purpose of the assessment was to further delineate the fill (clay) conditions at the Site and provide recommendations for the geotechnical aspects of the proposed construction.

1.2 Scope of Services

The scope of work included the following:

- Review of available data pertinent to the site.

- Conduct a subsurface investigation.

- Conduct basic laboratory testing of select soils.

- Perform a geotechnical analysis regarding the proposed construction, using the information obtained from the subsurface investigation and laboratory testing.

- Prepare this report of our findings, conclusions, and tentative recommendations for the geotechnical aspects of the proposed construction.

1.3 Authorisation

This assessment was performed and the report prepared in general accordance with and authorisation from Alium Investments Inc. to proceed with the work.

1.4 Standard of Care

The services performed by BAE were conducted in a manner consistent with the level of care and skill ordinarily exercised by members of the geotechnical profession practising contemporaneously under similar conditions in the locality of the project. No other warranty, expressed or implied, is made.

Limitations of this report are discussed in Appendix A. These limitations further explain the realities of geotechnical engineering and the limitations that exist in evaluating geotechnical issues.

This report has been prepared for the exclusive use of Alium Investments Inc. with specific application to the proposed project.

2.0 PROJECT and PROPERTY DESCRIPTION

2.1 Proposed Development

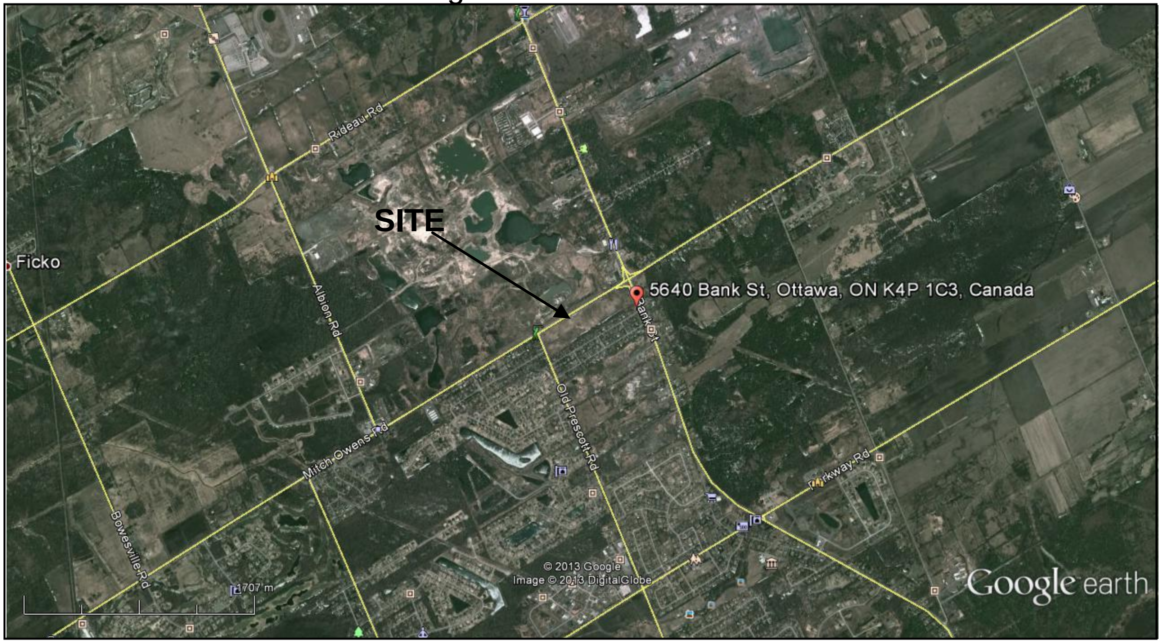
It is understood that the proposed development will consist of multiple commercial slab on grade structures. If the locations of the assumed loadings, proposed structures, floor elevations, or any other site features change BAE should be notified so that the changes can be reviewed to determine if the recommendations presented in this report are still applicable.

The legal description of the property is *Part of Lot 1, Concession 5, Geographic Township of Osgoode, City of Ottawa.* The Site encompasses three municipal addresses - 5640 Bank Street, 7107 Marco Street, and 7041 Mitch Owens Road.

The subject 13 hectare (32 acre) Site is located on an irregular parcel of land situated at the northern periphery of the Community of Greely, at the southwest corner of the intersection of Bank Street and Mitch Owens Road. The northern portion of the Site has an approximate frontage of 653m on the south side of Mitch Owens Road. The western portion of the Site has an approximate frontage of 200m on the east side of Old Prescott Road. The eastern portion of the Site has an approximate frontage of 150m on the west side of Bank Street. The southern portion of the Site has an approximate frontage of 35m on the north side of Marco Street and 720m backing onto the back of single family residential houses located along Marco Street.

The Site is currently vacant, and was previously utilized as a gravel pit (below water table in the central part of the site). It is understood that much of the gravel pit has been backfilled with a fill derived from pond excavations at a site near Airport Parkway and Hunt Club Road to the north. There are no significant environmental concerns from the current onsite operations. Figure 1 shows the Site location, Figure 2 shows the Site layout, and Figure 3 is the Site Plan of Survey.

Figure 1: Site Location



2.2 Land Use

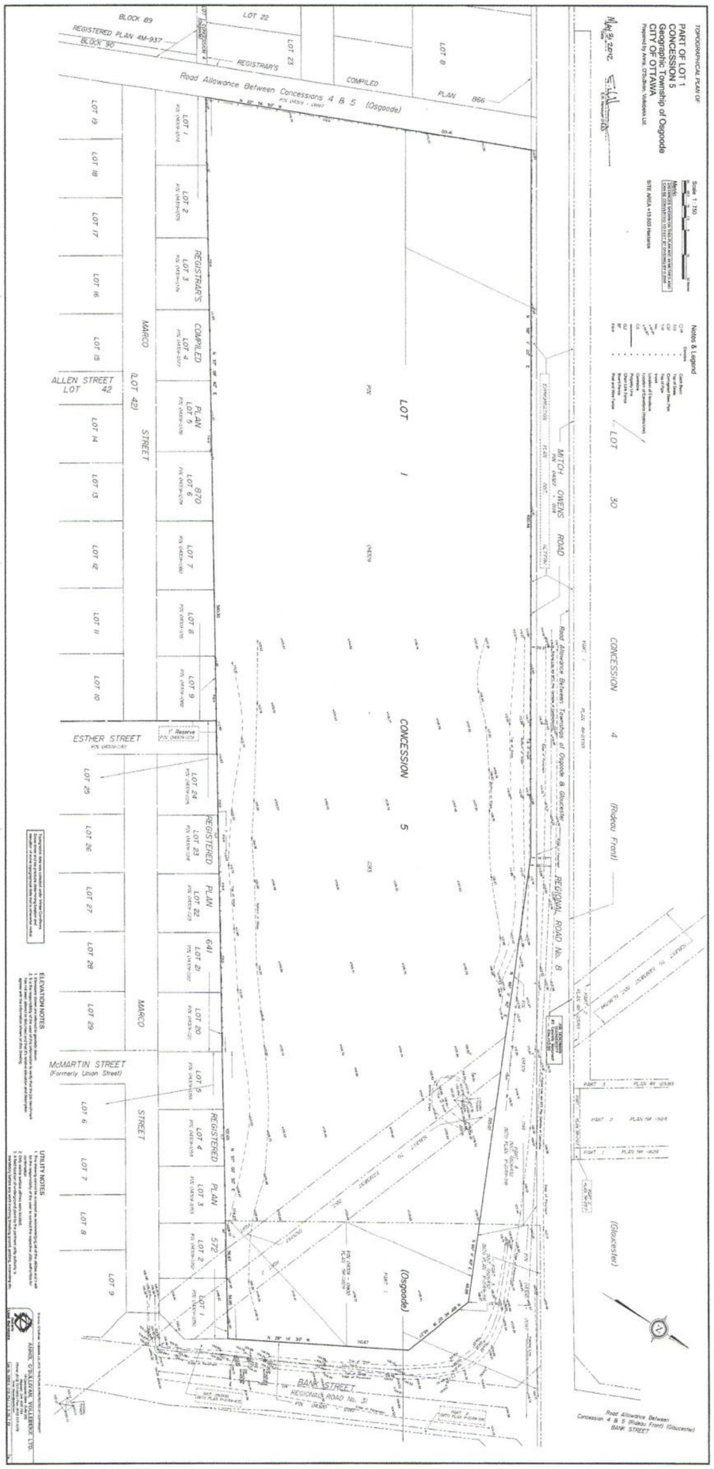
The Site is currently vacant, and was previously utilized as a gravel pit (below water table in the central part of the site). As part of this investigation, BAE reviewed city directories, available fire insurance maps and aerial photographs. The information gathered confirmed that the Site previously operated as an aggregate extraction operation (aggregate pit). Prior to this it was vacant land.

Information concerning the surrounding land use in the vicinity of the subject property was obtained from documented information as well as several site visits. Properties in close proximity to the site are predominantly used as residential and aggregate extraction. Lands to the south and west of the site are in residential use, lands to the north remain in aggregate extractive use, and lands to the east are undeveloped except for a school to the immediate northeast and some scattered commercial properties.

Visual observation of the adjacent properties, to the extent possible, did not reveal the presence of any structures, equipment or materials of concern. There was no evidence of staining, stressed vegetation, odours or environmental concerns currently associated with any of the neighbouring properties.

Figure 2: Site Layout

Figure 3: Site Plan of Survey



2.3 Geology and Physiography

The subject lands are located within the North Gower Drumlin Field physiographic region of southern Ontario, a drumlin field occupying much of the southern periphery of the City of Ottawa. According to Ontario Geological Survey Map 2556, the upper soils across the site mainly consist of glaciofluvial ice contact deposits of gravel and sand associated with the mapped abandoned raised beach, with glaciomarine deposits of sand and gravel mapped to the immediate west.

According to local water well records, mainly from wells located at the residential homes to the immediate south and west, the thickness of the undisturbed overburden in the close vicinity of the site is 15 to 27m. The thickest portion of the overburden appears to be along the north-south axis of the mapped abandoned raised beach trending through the centre of the site. The overburden is reported to consist primarily of granular deposits of sand and/or gravel.

2.4 Groundwater and Surface Water

The bedrock is the most commonly utilized source of potable groundwater in the area. Of the 92 reported wells within the same Township lot as the proposed development (i.e. Lot 1, Concession 5, Osgoode), 78 wells (85%) are reported to be completed in the bedrock. The remaining 14 wells are completed in gravel deposits in the lower overburden.

Groundwater is estimated to flow south toward North Castor River which is located approximately 1.5km to the south of the subject property. This information coincides with information taken from aerial photographs, which indicate that, the topography and hence the groundwater flows in the same direction toward this area.

2.5 Other Services

The Site was serviced at the road with hydro, telephone and gas services. Sanitary sewers and municipal water are not available in this area.

3.0 INVESTIGATION AND TESTING

3.1 Subsurface Investigation

The field investigation to determine the engineering characteristics of the subsurface materials included a reconnaissance of the project site, drilling of borings, performing standard penetration tests and obtaining disturbed split-barrel samples.

The drilling consisted of 31 test borings at the locations depicted on the Site Plan Appendix C and D. The onsite drilling was carried out in April and July, 2013, by Canadian Soil Drilling using a CME 45 mobile mounted drill rig with a 4in diameter, hollow stem auger and split-spoon sampler drill rig with continuous-flight augers.

Borehole locations were selected to maximise property and proposed structure coverage as per the updated site plan, as well as determined by site accessibility. All the boreholes were backfilled upon completion of fieldwork.

The following table is a summary of the Borehole depths and subsurface conditions encountered.

Table 1: Sub-Surface Conditions / Borehole Record

BH#	Depth In Feet	Sub-Surface Conditions	Density AR/BR = Auger Refusal/Boulders/Bedrock	Water Table (Estimate)
1	30	granular with cobble/boulders	medium	22’
2	2	Auger refusal @ 2’ boulder/BR	AR@2’	n/a
3	15	0-4 till, 4-12 grey clay, 12-15 grey granular	Soft to 4-15’	8’
4	6	Auger refusal @ 6’ boulder/BR	Semi-compact to hard	n/a
5	17	granular with cobble/boulders	Semi-compact	9’
6	25	0-5 till, 5-18 moist clay, 18-25 sand/stone some clay	Soft, BR @ 25’	12
7	27	0-27 gray sandy clay	Soft, BR @ 27’	6-8’
8	35	0-29 gray wet clay, 29-35 granular	Soft to 30’	6-8’
9	40	0-16 wet clay, 16-34 fine sand/silt with clay, 34-40 course silty sand	Soft	8’

10	0	COULD NOT ACCESS WITH DRILL – TOO WET/SOFT		
11	15	0-7 wet clay, 7-15 clay with pebbles/stones, shale BR@15	Soft to7’, stiff 7-15’, BR@15	8’
12	40	0-16 wet clay, 16-34 fine sand/silt with clay, 34-40 course silty sand	Soft	8’
13	40	0-16 wet clay, 16-34 fine sand/silt with clay, 34-40 course silty sand	Soft	8’
14	7	0-2 shale cover, 2-7 clay, BR@7	Medium, AR@7	n/a
15	7.5	0-7.5 brown till, AR@7.5	AR@7.5	n/a
16	17.5	0-4 shale cover, 4-10 wet silty clay, 10-17 silty clay with shale pcs.	AR@17.5	5-6’
17	20	0-15grey clay, 15-20 brown silty clay with shale pebbles	Soft to 15, medium beyond	11’
18	20	0-15grey clay, 15-20 brown silty clay with shale pebbles	Soft to 15, medium beyond	11’
19	25	0-5 till, 5-18 wet soft clay, 18-25 wet sand/silt/stone with shale pcs.	Soft to 18, medium beyond	8’
20	10	0-7 brown till, 7-10 till with stone	Medium to stiff	n/a
21	25	0-17 wet sandy clay, 17-25 course sand/stone	Soft to medium	12
22	4	0-4 brown to grey compact till	Stiff AR@4	n/a
23	25	0-18 grey clay, 18-25 brown sand/ gravel	Soft to 17, medium beyond	17’
24	10	0-10 granular/cobble/boulder	Medium AR@10	n/a
25	10	0-10 granular/cobble/boulder	Medium	n/a
26	7.5	0-7.5 till	Medium AR@7.5	n/a
27	8	0-8 till	Medium AR@8	n/a
28	9	0-9 granular/cobble/boulder	Medium AR@9	n/a
29	12	0-9 brown till, 9-12 granular/cobble/ boulder	Medium AR@12	n/a
30	6	0-6 granular/cobble/ boulder	Medium AR@6	n/a
31	4	0-4 granular/cobble/ boulder	Medium AR@4	n/a

Soil samples were obtained at selected intervals in the soil test borings. Undisturbed soil samples were obtained in general accordance with ASTM D-1587 (Thin-Walled Tube Sampling of Soils) using a standard split-spoon sampler. A split-spoon sampler is a 5cm O.D. tube that is driven into the soil to be sampled that can be split open lengthways for easy removal and visual inspection of the soil obtained. Disturbed soil samples were obtained in general accordance with ASTM D-1586 (Penetration Test and Split-Barrel Sampling of Soils). All samples were identified according to project number, boring number and depth, encased in polyethylene plastic wrapping to protect against moisture loss, and transported to our laboratory in special containers.

During the sampling procedures, standard penetration tests were performed in the borings in conjunction with the split-barrel sampling. The standard penetration value (N) is defined as the number of blows of a 63.5kg hammer, falling 75cm, required to advance the split-spoon sampler one-foot into the soil (ASTM D-1585). The sampler is lowered to the bottom of the drill hole and the number of blows recorded for each of the three successive increments of six inches penetration. The "N" value is obtained by adding the second and third incremental numbers. The results of the standard penetration test indicate the relative density and comparative consistency of the soils, and thereby provide a basis for estimating the relative strength and compressibility of the soil profile components.

Water level observations were made during the boring operations and the results are noted on the subsurface Table 1. In relatively pervious soils, such as sandy soils, the indicated elevations are considered reliable ground water levels. In relatively impervious soils such as clays and silty clays, the accurate determination of the ground water elevation may not be possible even after several days of observation. Seasonal variations, temperature and recent rainfall conditions may influence the levels of the ground water table and volumes of water will depend on the permeability of the soils.

A field log was prepared for each boring. Each log contained information concerning the boring method, samples attempted and recovered, indications of the presence of various materials such as silt, clay, gravel or sand and observations of ground water. It also contained an interpretation of subsurface conditions between samples. Therefore, these logs included both factual and interpretative information. On completion of each borehole, the void was filled in with the existing, removed soil and were all sealed with an impermeable covering.

3.2 Laboratory Testing

Limited laboratory tests were carried out on a number of selected soil samples in order to acquire necessary information with regards to the physical and mechanical properties of the soil layers and further on to evaluate and determine the parameters required for the calculations. All phases of the laboratory-testing program were performed in general accordance with the applicable ASTM Specifications.

Representative soil samples from each stratum were submitted to Peto MacCallum Ltd. for soil classification and grain size analysis. Sample 1 is representative of the surficial till stratum. Sample 2 is representative of the granular stratum beneath the clay fill. Sample 3 is representative of the brown clay with silt stratum. Sample 4 is representative of the grey clay fill stratum. The results of the analysis are included as Appendix F Soil Classification – Grain Size Analyses.

4.0 SUBSURFACE CONDITIONS

4.1 Stratigraphy

Detailed descriptions of the geotechnical conditions encountered in the thirty one (31) boreholes are located in Table 1 above. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted.

It should be noted that the soil boundaries indicated on the borehole records are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for purpose of geotechnical design and should not be interpreted as exact planes of geological change.

The initial review of the first seven (7) borehole logs inclusive indicates that the site is generally covered with native overburden overlying approximately 7 to 8 meters of grey clay fill followed by a native deposit of sand and gravel.

Upon review of the thirty one (31) additional boreholes completed at the site, approximately half of the boreholes indicated the deep non native clay layer, with the remaining boreholes showing no signs of non native clay. Appendix D shows the Clay delineation area. Where the clay is located it is as deep as 2 to 8m (7 to 29 ft), however where there is no clay, the soil is desirable, consisting of mostly native gravel, sand, cobble and boulders.

4.2 Topsoil/Overburden

Each borehole indicated a 50mm to 100mm of overburden (topsoil, organics).

4.3 Grey Clay

Below the overburden layer a thick layer of Grey Clay was generally encountered to a depth of approximately 7 to 8 meters. The clay was generally moist, wet, mixed with traces of silt, sand and gravel. The clay had originated just north of the site near the intersection of Airport Parkway and Hunt Club Road, when new retention ponds were constructed, the grey clay was relocated to the subject site and used to fill in the gravel pit.

4.4 Sand and Gravel

Below the clay is the bottom of the gravel pit. The sand and gravel layer has traces of silt and clay. The gravel pit operations did not extend all the way to the Bank Street along the eastern edge. Below the overburden, a thin layer of clay, the sand and gravel, there is a seam of eastern shale bedrock. This extends across the far eastern portion and is located approximately 3.5m below grade.

Detailed description of the type of soil layers encountered during drilling is given in the borehole information, Table 1 and the Borehole Records found in Appendix E.

4.5 Groundwater

Groundwater was encountered at observable levels in all but one of the borehole locations. These measurements indicate that the groundwater table at the site is at between 3.6m to 6.0m below grade. These may fluctuate with seasonal climatic variations and changes in the land use. Low permeability soils will require several days or longer for groundwater to enter and stabilise in the test borings.

5.0 RECOMMENDATIONS

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from our soil test borings and laboratory tests, and our experience with similar projects. Because the test borings represent a very small statistical sampling of subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different from those indicated by the soil test borings. In these instances adjustments to design and construction may be necessary.

This geotechnical report is based on the project information developed by BAE and the assumptions stated in this report. Changes in the proposed location or design of the structures can have significant effects on the conclusions and recommendations of the geotechnical report. BAE should be contacted in the event of such changes.

5.1 Site Preparation

Topsoil and overburden as well as other debris noted at or below the existing ground surface should be removed as part of the site preparation for the proposed construction area. In all new fill and excavation areas, ie; the clay fill area, vegetation, topsoil, roots and other deleterious materials (typically 1.5 to 3cm), deemed unsuitable shall be removed from the proposed construction areas, and replaced with controlled fill. Site clearing, grubbing and stripping will need to be performed only during dry weather conditions. Operation of heavy equipment on the site during wet conditions could result in excessive rutting and mixing of organic debris with the underlying soils especially with low permeability soils like the clays discovered in the drilling.

Due to their physical properties, these types of soils are very sensitive to traffic when allowed to get saturated, as they hold onto the water and will become increasingly difficult to control. ***Extreme care*** must be taken when exposing these types of soils, both to elements such as freezing and excessive wetting or heavy equipment traffic, especially rubber tire equipment.

5.2 Excavations

Temporary construction slopes should be designed and excavated in strict compliance with the rules and regulations of the Provincial Statute - Occupational Health and Safety Act, R.S.O. 1990, c. O.1, as amended Ontario Regulations 213/91 - Regulations for Construction Projects. This document was prepared to better insure the safety of workers entering trenches or excavations, and requires that all excavations conform to the new OSHA guidelines.

The contractor is solely responsible for protecting excavations by shoring, sloping, benching or other means as required to maintain stability of both the excavation sides and bottom. BAE does not assume any responsibility for construction site safety or the activities of the contractor.

For this site, the overburden soil encountered in our exploratory borings consisted of silty till. We anticipate that OSHA will classify these materials as Type 4 to Type 3. OSHA recommends a maximum slope inclination of 9 Horizontal: 5 Vertical for these type soils. Excavation construction slopes should be closely observed for signs of mass movement, such as tension cracks near the crest, bulging at the top of the slope, etc.

5.3 Structural Fill (Engineered Fill)

It is recommended that structural fills be constructed as controlled well-compacted engineered fills. Structural engineered fill should be inorganic, low plastic clay, sand, or gravel. Any existing soils with a high organic content (browns) are suitable for reuse as fill in landscaping areas only. It is recommended that only granular fill be used within the building footprint and within 1.5m of the building footprint. The intent of these recommendations is to reduce the potential for consolidation and settlement of new fills.

Laboratory testing should be performed on the fill materials to determine the appropriate moisture-density relationship of the fill being placed. Adjustments to the soil moisture by wetting or drying should be made as needed during fill placement. During grading operations, representative samples of the proposed imported structural fill materials should be periodically checked via laboratory testing. A representative from BAE or approved other, should be on site to monitor excavation and grading operation as well as the suitability of fill materials. Suitable fill material should be placed in thin lifts (lift thickness depends on type of compaction equipment, but in general, lifts of 200mm loose measurements are recommended). The soil should be compacted by the necessary compaction equipment to meet the specified compaction recommendations.

Sheepsfoot rollers may be required to adequately compact fine-grained fill material (silts and clay). If the fill material is granular (sands and gravels) with less than 10% clays and silts, smooth-drum vibratory compactors should be used. In addition, a smooth-drum roller should be provided to “seal” the fill at the end of each workday to reduce the impact of precipitation. In areas undergoing removal of seepage water, the engineered fill should be limited to well-graded sand and gravel or crushed stone.

Within small excavations, such as in utility trenches (less than 60cm in width), around manholes or behind retaining walls, we recommend the use of diesel plate tampers or “jumping jack” compactors to achieve the specified compaction. Loose lift thickness of 100mm is recommended in small area fills.

A qualified field representative should periodically observe fill placement operations and perform field density tests at various locations throughout each lift, including trench backfill, to indicate if the specified compaction is being achieved.

TABLE 2: STRUCTURAL FILL PLACEMENT GUIDELINES

Areas of Fill Placement	**Compaction Recommendation (ASTM D698-Standard Proctor)**	**Moisture Content (Percent of Optimum moisture)**
Granular cushion beneath Floor Slab and over Footings	98%	As necessary to obtain density
Structural fill supporting Footings	98%	-1 to +3 percent
Structural fill placed within 1.5m beyond the perimeter of the building pad	98%	-1 to +3 percent
Grade-raise fill placed within 30cm of the base of the pavement	98%	-1 to +3 percent
Structural fill placed below the base of the Pavement Soil Subgrade	95%	-1 to +3 percent
Utility Trenches - Within building and pavement areas	98%	-1 to +3 percent
Beneath Landscaped/Grass Areas	92%	As necessary to obtain density

The fill soils should be relatively free of organic materials (less than about two hundredths of a percent by weight) and other deleterious material. In addition, the soils should preferably not contain particle sizes larger than 75mm.

5.4 Foundation Design

Footings should be founded on undisturbed brown or grey sand and gravel with traces of silt and clay found or on engineered fill. Piles and or caissons may also be a feasible option for the deeper clay areas (8m +).

Based on the results of the soil test borings, laboratory testing and our engineering evaluation, it is our opinion that the subsurface conditions where clay fill is present are not suitable for supporting the proposed structure on a conventional shallow foundation. However when the clay is removed and engineered fill is used to backfill these areas, foundations may be constructed as follows: for spread or continuous footings bearing on the natural or engineered fill layers, 150 kPa for an Ultimate Limit State (ULS) and 100 kPa for Serviceability Limit State (SLS) can be used. . The net allowable bearing pressures refer to the bearing pressure at foundation level in excess of the surrounding overburden pressure and do not include footing weight, backfill weight, or slab weight.

Piles or Caissons may be used in the deep clay, areas specifically located in the CR 13 and 14 units, along the south edge, as well as anywhere else on the site that has deep areas of non native clay.

Footings should have minimum dimensions in accordance with the applicable building codes. All footings should be located so that the smallest lateral clear distance between footings will be at least equal to the difference in their bearing elevations. If this distance cannot be maintained, the lower footing should be designed to account for the load imparted by the upper footing. The recommended soil bearing capacity includes a factor of safety of at least 3 against shear failure. It is possible that some soils at the site will have an allowable soil bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations should be performed by a BAE representative or acceptable substitute, during footing construction to aid in the identification of such soils and bearing capacities. After the evaluations and any required remedial measures are performed, concrete should be placed as quickly as possible to avoid exposure of the foundation sub-soils to wetting, drying or freezing. If soils in the areas of foundation support are subjected to such conditions, the footings should be re-evaluated.

The frost line in the Ottawa area is 1.2 metres below grade. All foundations in unheated areas, including the footings for retaining walls, should be provided with a minimum of 1.2 metres of soil cover to minimize the potential for frost related movements

When footings or foundations are excavated, a qualified inspector should re-evaluate the soil to ensure stability, and to make recommendations that might include the use of rebar, or widening footings.

Table 3: Factored ULS Bearing Resistance of Engineered Fill

Founding Soil	Footing Width (m)	Footing Depth (m)	Factored ULS Resistance (kPa)
Engineered Fill	1.0	0.6	150
	1.0	1.5	310
	0.6	0.6	130
	0.6	1.5	290

5.5 Floor Slab Subgrade Preparation

The soil subgrade in the areas of concrete slab-on-grade support is often disturbed during foundation and superstructure construction. Additionally, floor slab areas are often disturbed by construction equipment traffic between the time of initial grading and final pavement construction. The subgrade should be excavated to the design depth of the bottom of slab gravels. To prepare the subgrade, the top 20cm of the subgrade should be compacted to a minimum of 98% of the maximum dry density as determined by ASTM D698-91, Standard Proctor Moisture-Density Relationship. The moisture content should also be controlled to -1 to +3% of the optimum.

The final subgrade should be proof-rolled and evaluated by a representative of BAE immediately prior to placement of the engineered fill to detect any localised areas of instability or soft areas. If unstable soils are encountered which cannot be adequately densified in place, such soils should be removed and replaced with well-compacted fill material placed in accordance with the *Structural Fill* section of this report. The subgrade should be graded to a shallower slope than five horizontal to one vertical (5H: 1V) prior to receiving general engineered fill material to reduce the effects of differential fill thicknesses. The prepared subgrade should be protected from drying, excessive moisture, and freezing.

5.6 Floor Slab Design

The recommendations provided are based on the assumption that the average net floor slab load will not exceed 750 psf, and that the maximum concentrated net floor slab load will be less than 1500 psf. The recommended bearing capacity of the floor slab is 2000 psf. Should a greater bearing capacity be required, BAE should review the recommendations presented in this report. The granular cushion beneath the floor slab, should be free draining, well graded and compacted by vibration prior to pouring the floor slab. A minimum of 400mm of granular fill should be provided below the slab. The granular fill should be compacted according to the recommendations given in Structural Fills section of this report. The recommended minimum gravel thicknesses are required to promote uniform distribution of floor loads to the subgrade, and to bridge over newly constructed fill areas such as utility trenches. Thicker gravel courses may be required for structural considerations. A vapour barrier should be placed beneath the concrete slab.

The slab-on-grade unit should be allowed to float independently of all load-bearing walls and columns. Floating the floor slab independent from the wall and column loads with movable and/or expansion joints will be critical in minimising the potential cracking, which can occur along, and around the proposed foundation system. In regards to the wall/floor structural detail, expansion joints and gap spacing are recommended at the wall/floor connection. A half-inch gap for movement between the floor slab and insulation board is recommended along with a bond break that allows independent movement between the floor slab and masonry block wall. A 100mm thick granular cushion is also recommended between the floor slab and top of column pad and wall footings. Resting the floor slab on top of column pads and wall footings is not recommended. Assuming the previously mentioned recommendations are performed, the risk associated with floor slab cracking will be reduced.

5.7 Pavement Subgrade Preparation

Due to the depth of the clay in the parking areas it is possible to bridge the clay and only excavate to a depth that would allow approximately 600 to 800mm of bridging material (in this case preferably rip rap, gabion basket stone, or any stone approximately 100-200mm in size), specified thickness of Granular “B” and Granular “A” and pavement. A few items to keep in mind when designing this structure:

1. The thickness of the granular material must be sufficient to develop acceptable pressure distribution over the wet soils.
2. The backfill material must be able to withstand the wheel load without rutting.
3. The compaction of the backfill material should be in accordance with the Standard Specifications.

The subgrade should be proof rolled with a fully loaded dump truck, scraper, or similar rubber-tired equipment weighing at least 25 tons or a 10-ton vibratory steel drum roller (with vibration off). Do not use vibratory rollers to proofroll materials containing significant amounts (>10%) of fines if the subgrade materials are wet or near groundwater levels, since vibratory rollers tend to wick water to the surface.

A representative of BAE or equivalent qualified person, should observe Proofrolling operations. Unstable and unsuitable soils, which are revealed by proof rolling and which cannot be adequately densified in-place, should be removed under the direction of the BAE representative. It may be necessary to perform selective removal of soft, wet soils and/or stabilise existing soft soils in-place. If required, the methods of stabilisation will typically include incorporating a lift of crushed stone materials or a geosynthetic over the soft soils. The identification of areas that may require undercutting and/or stabilisation should be based on the actual conditions at the time of construction, and will depend on the location of the soft area.

The subgrade should be compacted to a minimum of 98% of the maximum proctor density of ASTM D-698-91, Standard Proctor Moisture-Density Relationship. The moisture content should also be controlled to -1 to +3% of the optimum. The subgrade should be tested by a representative of BAE and approved for placement of select fill.

5.8 Pavement Design

All of the topsoil and any fill or excessively wet materials within the proposed driveway and parking lot areas should be subexcavated and the areas brought to grade using compacted Granular B. Based on the anticipated traffic loadings, it is recommended that flexible pavements for passenger vehicle parking areas be designed for a maximum Benkelman beam rebound of *2.5* millimetres. Driveways and truck parking areas should be designed for a maximum Benkelman beam rebound of *1.5* millimetres. To achieve these criteria, the pavement structures should consist of the following constructed on a properly prepared engineered fill subgrade:

Table 4: Recommended Pavement Structure Thickness for Surface Parking Areas

Pavement Layer	**Compaction Requirements**	**Computed Pavement Structure**	
		Parking Areas (light Duty)	Access Roads (heavy duty)
Asphaltic Concrete	92.0 to 96.5% Maximum relative density	65mm HL3	90mm
OPSS Granular “A” Base	98% SPMDD	150mm	150mm
OPSS Granular “B” Subbase	98% SPMDD	300mm	450mm

The above-noted pavement structures are not intended to support construction traffic. The pavement subgrade should be thoroughly proofrolled with heavy machinery prior to pavement construction to identify any areas requiring remedial work.

The Granular A base and Granular B subbase should be uniformly compacted to at least 98 per cent of standard Proctor maximum dry density. To preserve the integrity of the completed pavement structure, perforated stub drains should be provided at subgrade level at any catchbasin locations; otherwise, grading should direct surface and subsurface water to perimeter ditches with inverts at least *0.5* metres below subgrade level.

It is recommended that placement of the sheet asphalt be deferred for one year following placement of the binder asphalt to minimise the detrimental effects of potential differential settlement of the service trench backfill.

Surface drainage around the pavement and proper maintenance are also important to long-term performance. Curbs should be backfilled as soon as possible after construction of the pavement. Backfill should be compacted and should be sloped to prevent water from ponding and infiltration under the pavement. All pavement joints should be caulked and any cracks should be quickly patched or sealed to prevent moisture from reaching and softening the subgrade.

5.9 Drainage and Groundwater Considerations

The site should be graded to provide positive drainage to reduce storm water infiltration. A minimum gradient of one percent for asphalt areas should be maintained. A three percent gradient should be maintained for landscaped areas immediately adjacent (within 3m) to the building. In general, water should not be allowed to collect near the surface of the foundation or floor slab areas of the structures during or after construction. If water were allowed to accumulate next to the foundation, it would provide an available source of free water to the expansive soil underlying the foundation. Similarly, surface water drainage patterns or swales must not be altered so that runoff is allowed to collect next to the foundation.

Temporary drainage provisions should be established, as necessary, to minimise water runoff into the construction areas. Since soils generally tend to soften when exposed to free water, provisions should be made to remove seepage water from excavations, should it occur. Also, undercut or excavated areas should be sloped toward one corner to facilitate the collection and removal of rainwater or surface runoff. Adequate protection against sloughing of soils should be provided for workers and inspectors entering the excavations. This protection should meet O.S.H.A. and other applicable building codes.

Ground water seepage was encountered in our borings during drilling, and groundwater should not be encountered during the shallow excavations, but will be found approximately 3.6m below grade. If minor ground water seepage is encountered within the proposed building foundation, utility trenches and grading excavations at the time of construction, especially after periods of heavy precipitation, small quantities of seepage may be handled by conventional sump and pump methods of dewatering.
Steel casing should be on hand during piling operations to prevent seepage and sloughing of the sidewalls. The piles should be concreted immediately following inspection to reduce the potential for sloughing. Some pumping of collected water may be required during underground utility construction.

A permanent dewatering trench with an access well would be a way to minimize water issues on site, especially for long term water management. An ideal location would be moving towards the proposed retention pond, approximately 450mm below grade, ±100mm size stones lining the trench. An access well would allow for routine maintenance over the years, as well as emergency access. In addition to the trench, additional sock wrapped drainage tile should be used abundantly under the parking structures, extending outward from the catch basins as well as along all foundation, and retaining wall structures. This will minimise long term undesirable water settlement across the property.

5.10 Earthquake Conditions

The subsoil and groundwater information at this site has been examined in relation to Section 4.1.8.4. of the OBC 2006. The subsoil at the structure location will generally consist of fill. The shallow foundations will be set on the engineered fill. The reported undisturbed N-Values for the soil below the founding levels ranged from 36 to 71.

Based on the subsurface soil conditions encountered during our geotechnical investigations, the Site Class for this site is “C” as per Table 4.1.8.4.A, Site Classification for Seismic Response, OBC 2006.

5.10 Septic and Tile Bed Areas

For the septic and tile bed areas, the clay should be removed and replaced with free draining, engineered fill capable of supporting the septic and tile bed specifications.

6.0 ADDITIONAL SERVICES

The recommendations presented in this report are contingent on BAE observing and/or monitoring:

- Proofrolling and fill Subgrade conditions;
- Backfilling and compaction of excavations;
- Suitability of borrow materials;
- Fill placement and compaction;
- Foundation subgrades; and
- Compliance with the geotechnical recommendations.

7.0 CLOSURE

We trust that this report will assist you in the design and construction of the proposed project. Should you have any questions, please do not hesitate to contact us. This report was completed by Brian A. Emms, C.E.T. and Sarah Heino, Geotechnologist and reviewed by G. Jan Van Iterson, P. Eng.

Respectfully submitted,
BAE & Associates Environmental Inc.

BRIAN EMMS
N° 066869

G. J. VAN ITERSON

Brian A. Emms, C.E.T.
Senior Env. Technologist

G. Jan Van Iterson, P. Eng.
Associate

APPENDIX A
LIMITATIONS

This report was prepared for the exclusive use of Alium Investments Inc. for the geotechnical aspect of the proposed development described in Section 2. The report may not be relied upon by any other person or entity without the written permission of BAE. This report was prepared in accordance with current, generally accepted geotechnical engineering practices. No other warrantee is provided.

BAE should be allowed the opportunity to review the geotechnical aspects of plans and specifications prior to construction, to allow confirmation of the correct interpretation of the recommendations provided in this report. Foundation, earthworks, underground construction, and pavement construction should be undertaken only with full time monitoring by qualified personnel. BAE can provide these services on request.

The conclusions and recommendations submitted in this report are based upon the data obtained from a limited number of widely spaced subsurface explorations. The nature and extent of variations between these explorations may not become evident until construction or further investigation. If variations or other latent conditions do become evident, it will be necessary to re-evaluate the recommendations of this report. The recommendations contained herein are not intended to dictate construction methods or sequences. Instead, they are furnished solely to help designers identify potential construction problems related to foundation and earth plans and specifications, based upon findings derived from sampling. Depending upon the final design chosen for the project, the recommendations may also be useful to personnel who observe construction activity.

Potential contractors for the project must evaluate potential construction problems on the basis of their review of the contract documents, their own knowledge of and experience in the local area, and on the basis of similar projects in other localities, taking into account their own proposed methods and procedures.

APPENDIX B
PROJECT PHOTOGRAPHS

Area of BH 18 representative of central area of Site, note heavy vegetation

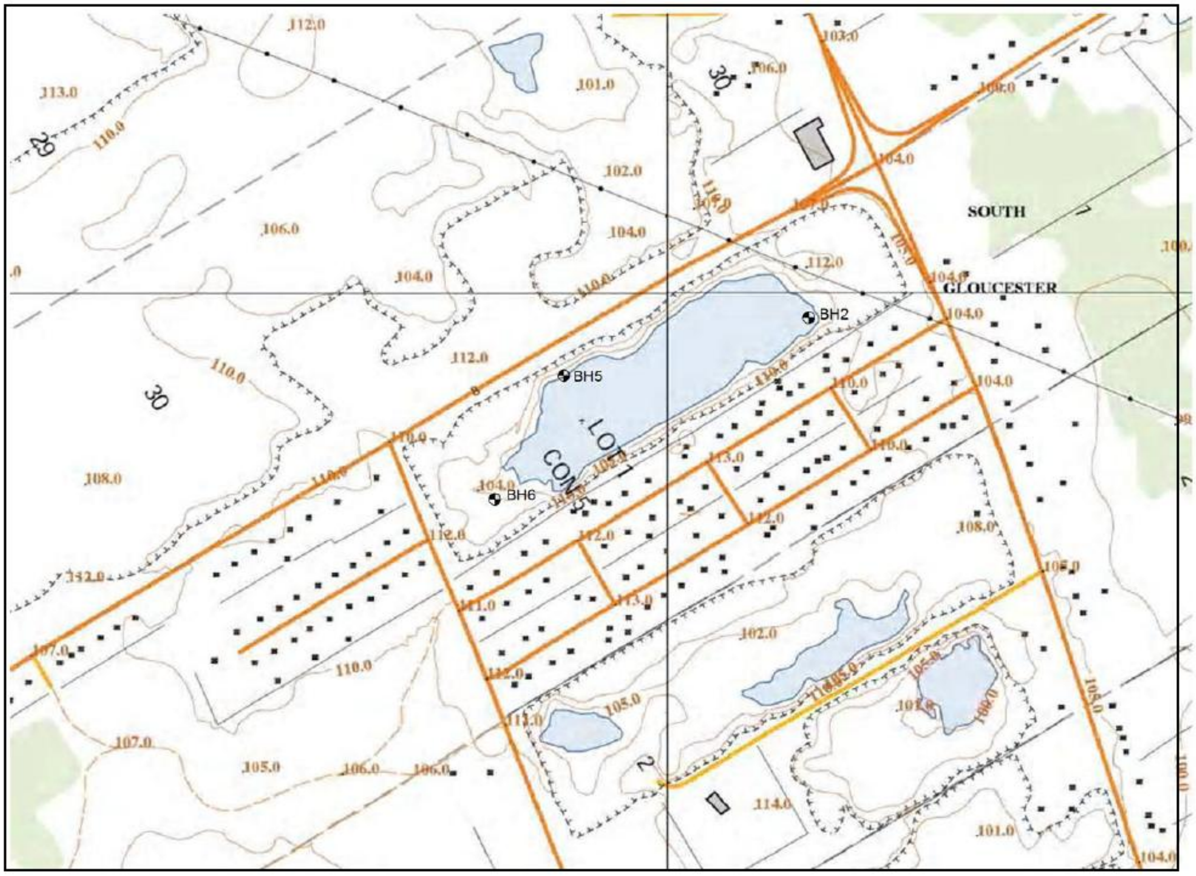
General surface conditions at East side of property, note boulders and heavy vegetation

Looking SE at area of BH 27, start of increased incline to Marco Street

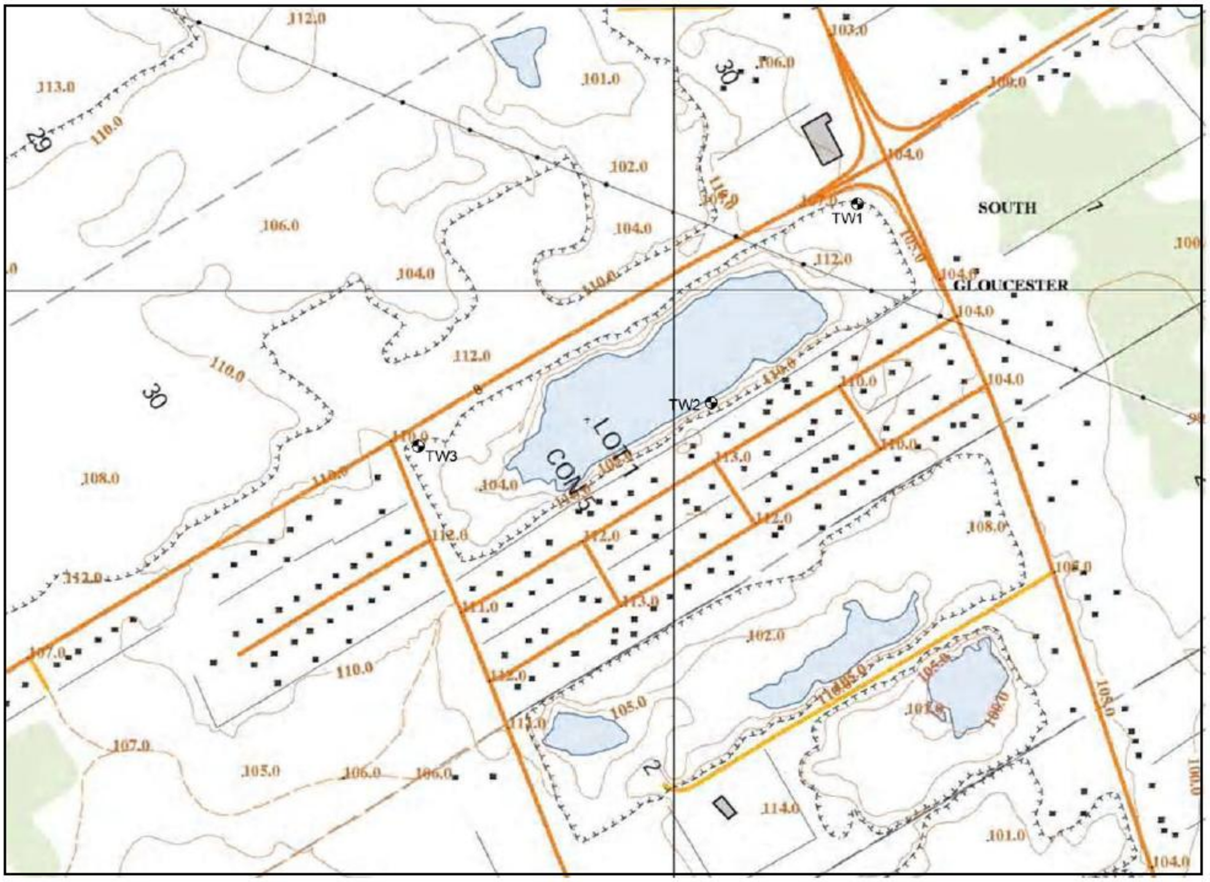
Looking west along bottom of slope and tree line, note incline of slope is parallel with tree tops

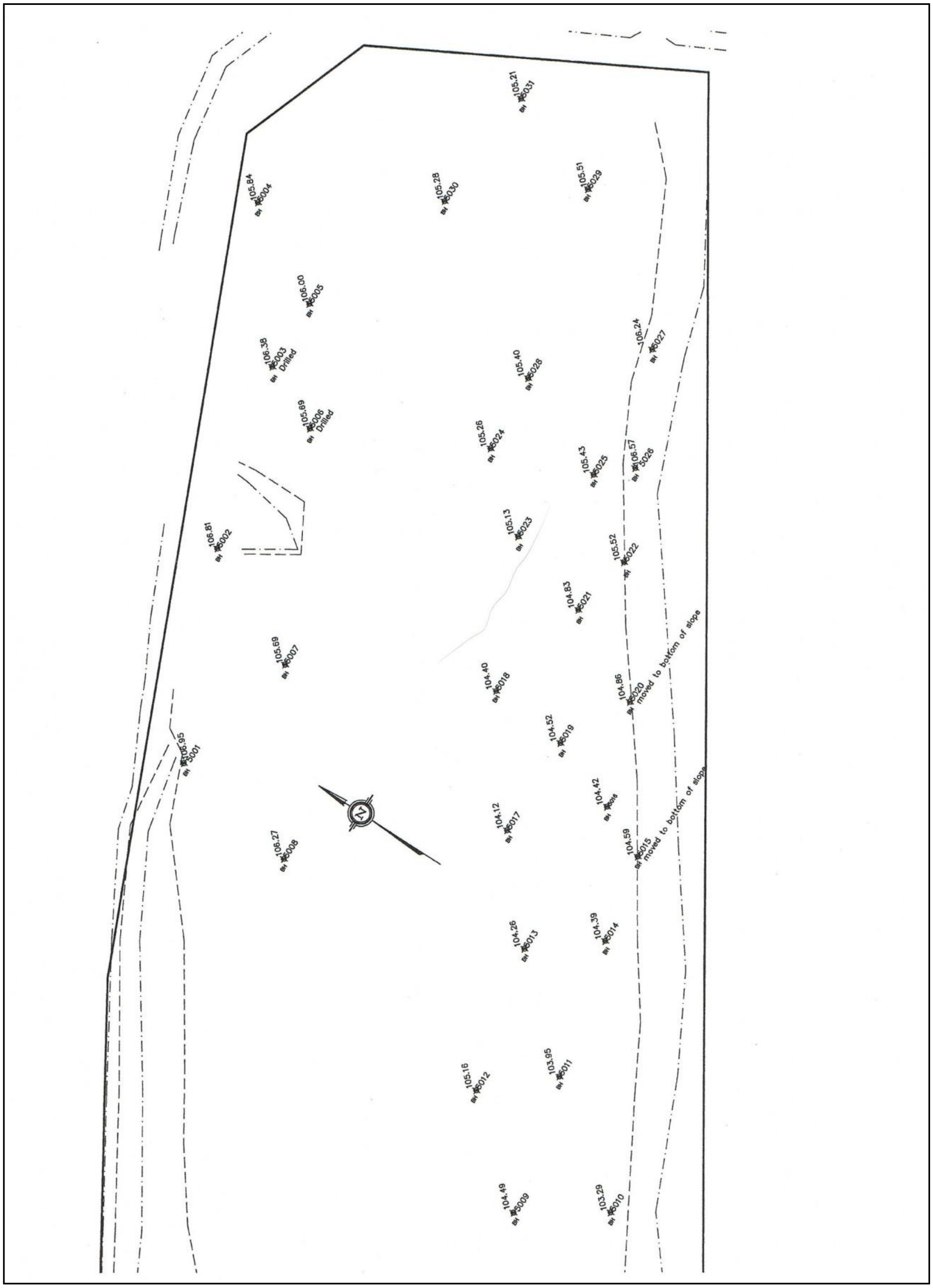
APPENDIX C
BOREHOLE LOCATIONS – April 2013
BOREHOLE LOCATIONS AND SURFACE ELEVATIONS - July 2013

Greely: Borehole Locations April 2012

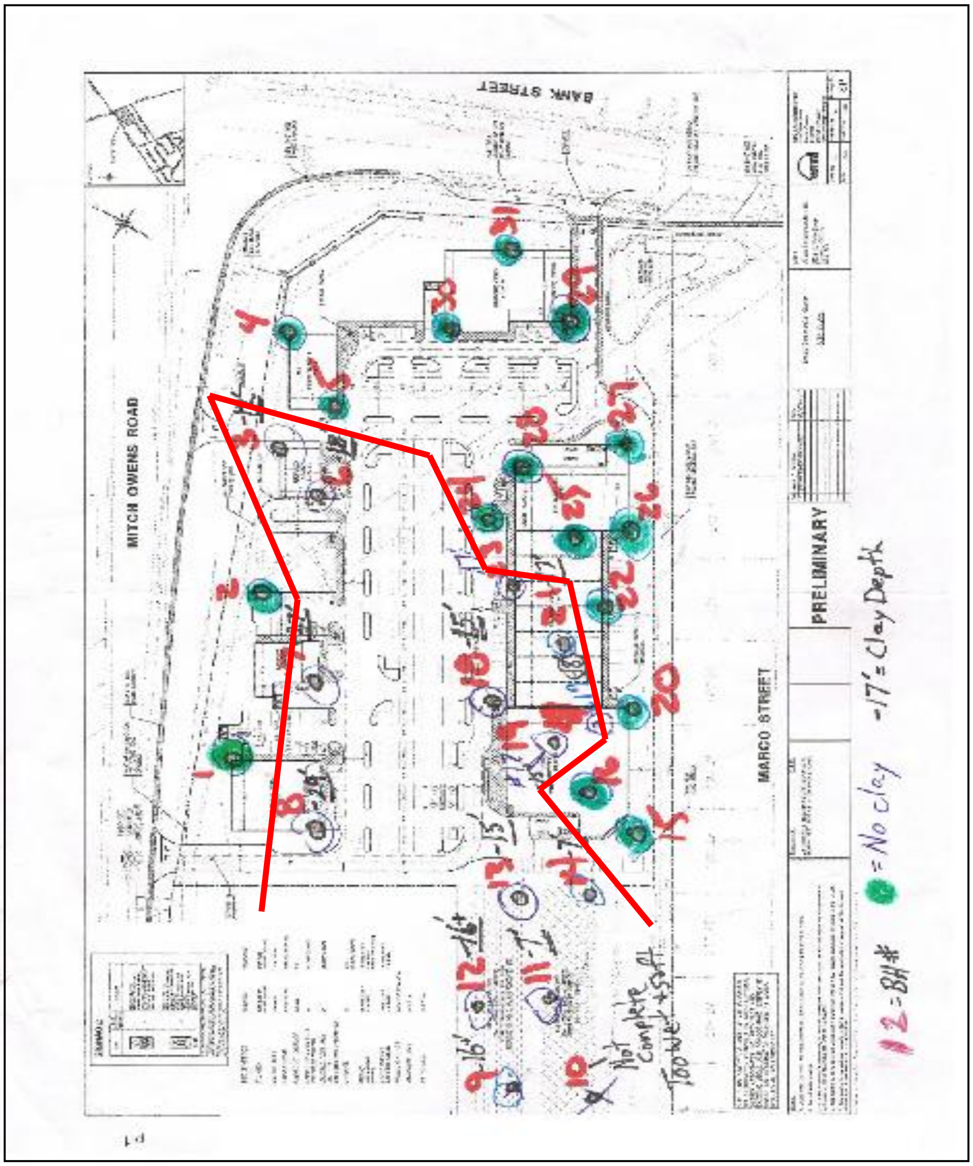


Greely: Monitoring Well Locations April 2012




105.21
BH 5031
105.84
BH 5004
105.28
BH 5030
105.51
BH 5029
106.00
BH 5005
106.38
BH 5003
Drilled
105.40
BH 5028
106.24
BH 5027
105.69
BH 5006
Drilled
105.26
BH 5024
105.43
BH 5025
106.57
BH 5026
106.81
BH 5002
105.13
BH 5023
105.52
BH 5022
104.83
BH 5021
105.69
BH 5007
104.40
BH 5018
104.86
BH 5020
moved to bottom of slope
106.95
BH 5001
104.52
BH 5019
N
104.42
BH 5016
104.12
BH 5017
106.27
BH 5008
104.59
BH 5015
moved to bottom of slope
104.26
BH 5013
104.39
BH 5014
105.16
BH 5012
103.95
BH 5011
104.49
BH 5009
103.29
BH 5010

APPENDIX D
NUMBERED FIELD BOREHOLE LOCATIONS
CLAY DELINEATION


BANK STREET
MITCH OWENS ROAD
MARCO STREET
PRELIMINARY
#2 = BH#
● = No clay
-17' = Clay Depth
Not Complete Too wet +5 ft

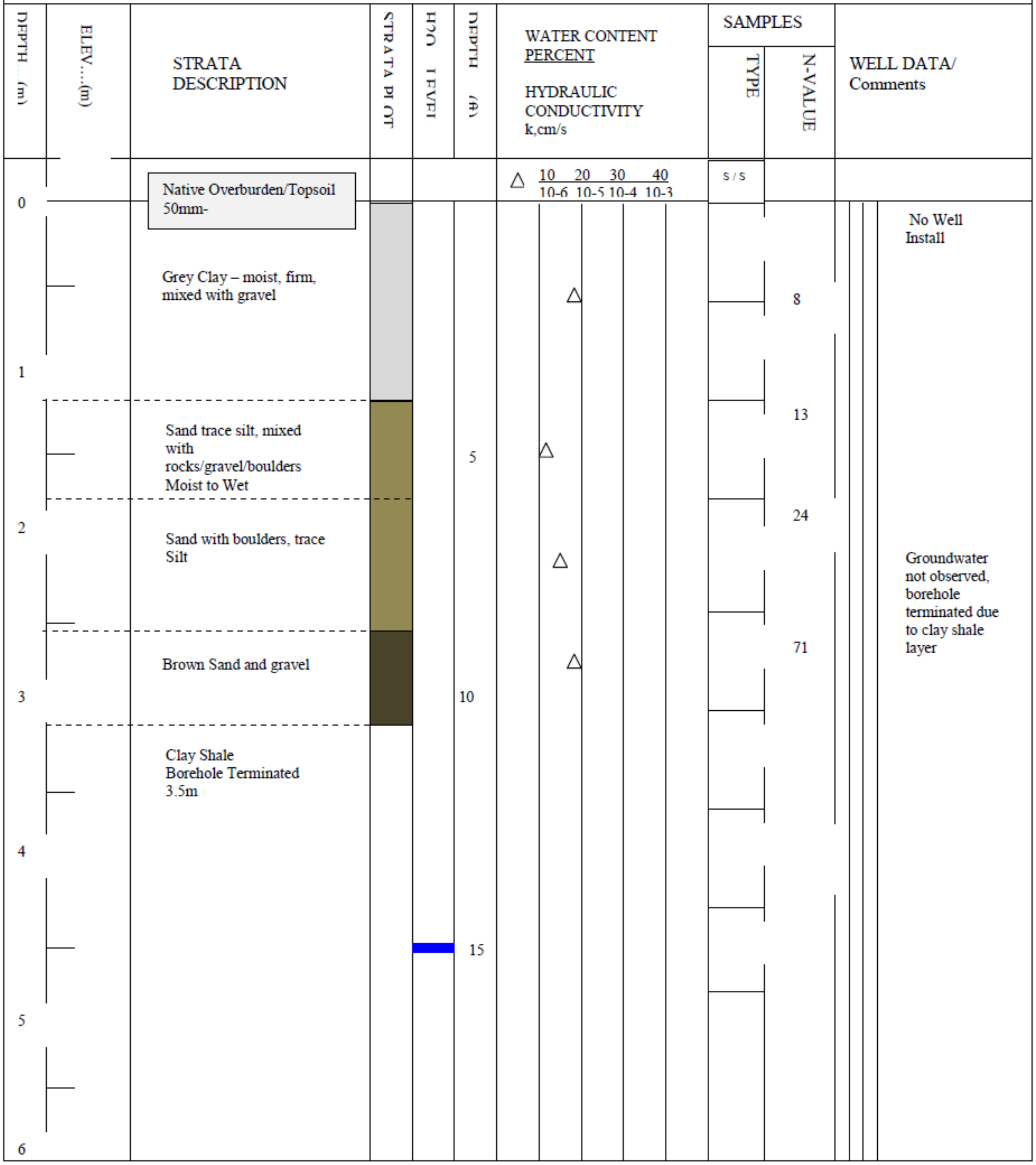
Appendix E
Borehole Records
April 2013 Preliminary Geotechnical (BH1-BH7)
July 2013 Geotechnical – Clay Delineation (BH1-BH23)

BOREHOLE RECORD

BH 1

BAE & Associates

CLIENT: Otis
LOCATION: Greely
DATE: April 4, 2012
Penetration Test Hammer: 63.5kg
Drop: 760mm
Groundwater Level Estimated - 6.0m
Continual Split Spoon Sampling @ 0.45m intervals



Native Overburden/Topsoil 50mm-

Grey Clay – moist, firm, mixed with gravel

Sand trace silt, mixed with rocks/gravel/boulders Moist to Wet

Sand with boulders, trace Silt

Brown Sand and gravel

Clay Shale
Borehole Terminated
3.5m

No Well Install

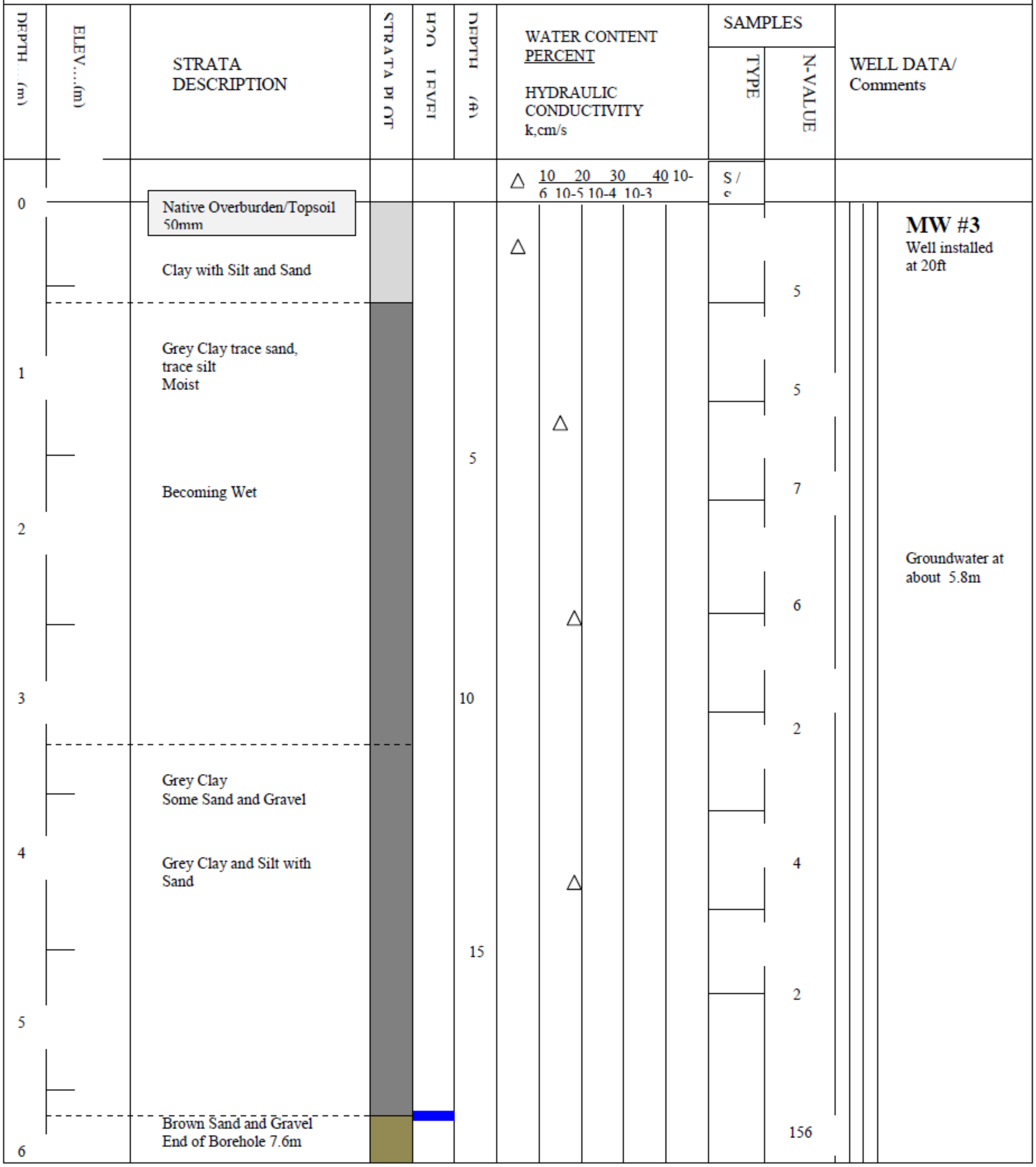
Groundwater not observed, borehole terminated due to clay shale layer

BOREHOLE RECORD

BH 2

BAE & Associates

CLIENT: Otis
LOCATION: Greely
DATE: April 4, 2012
Penetration Test Hammer: 63.5kg
Drop: 760mm
Groundwater Level Estimated - 6.0m
Continual Split Spoon Sampling @ 0.45m intervals



Native Overburden/Topsoil 50mm

Clay with Silt and Sand

Grey Clay trace sand, trace silt
Moist

Becoming Wet

Grey Clay
Some Sand and Gravel

Grey Clay and Silt with Sand

Brown Sand and Gravel
End of Borehole 7.6m

MW #3
Well installed at 20ft

Groundwater at about 5.8m

BOREHOLE RECORD

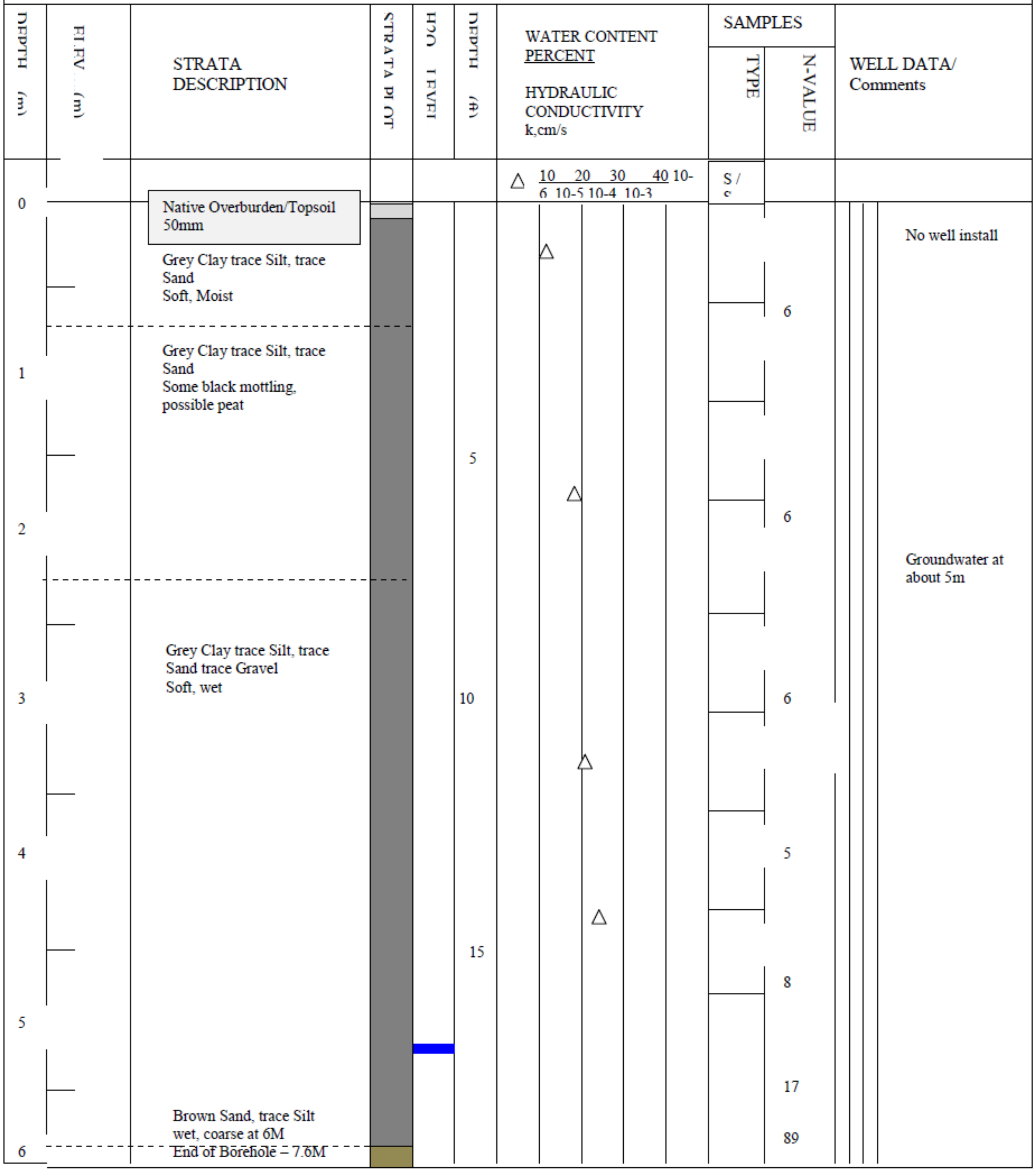
BH 3

BAE & Associates

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: April 4, 2012



Native Overburden/Topsoil 50mm

Grey Clay trace Silt, trace Sand
Soft, Moist

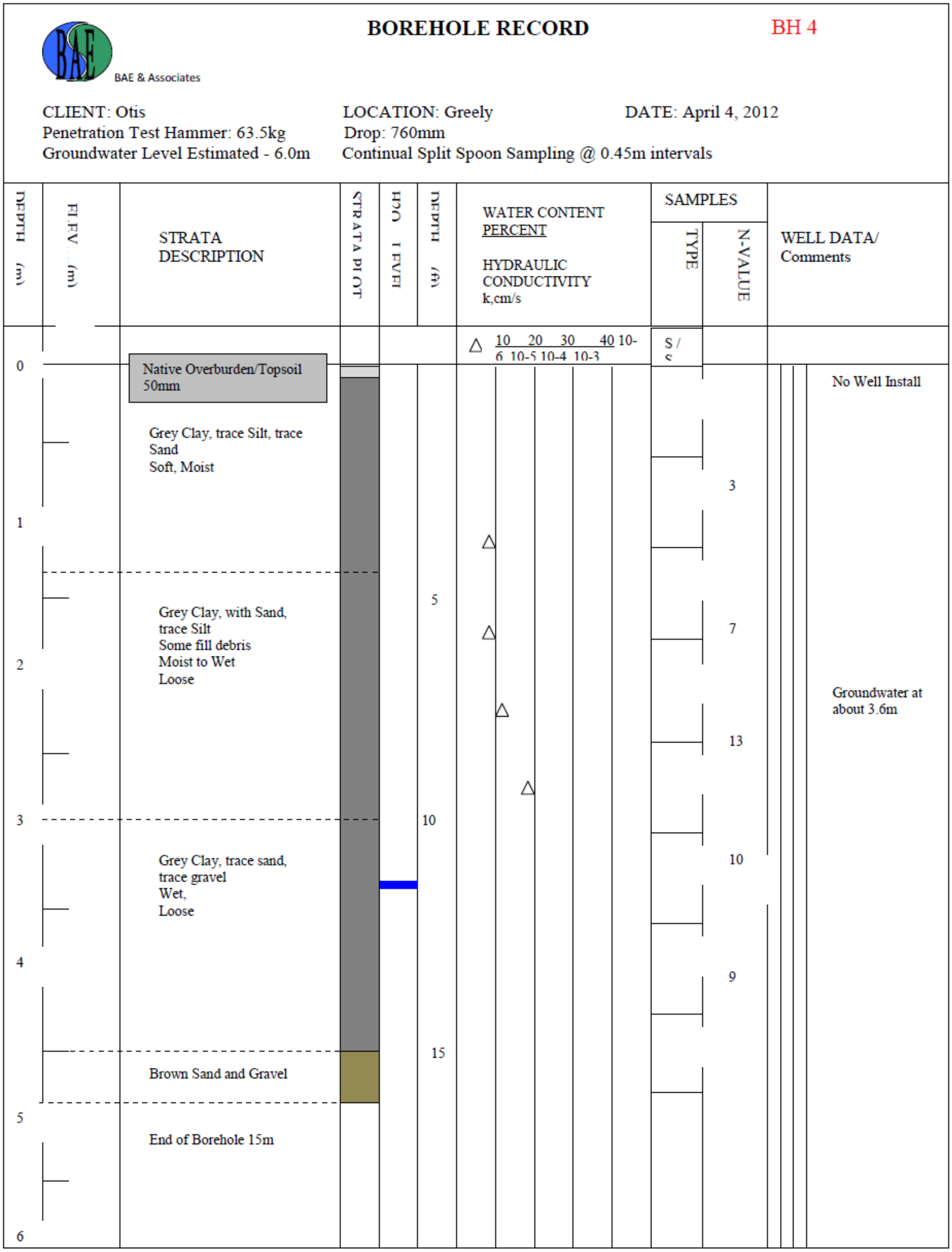
Grey Clay trace Silt, trace Sand
Some black mottling, possible peat

Grey Clay trace Silt, trace Sand trace Gravel
Soft, wet

Brown Sand, trace Silt
wet, coarse at 6M
End of Borehole – 7.6M

No well install

Groundwater at about 5m



BOREHOLE RECORD

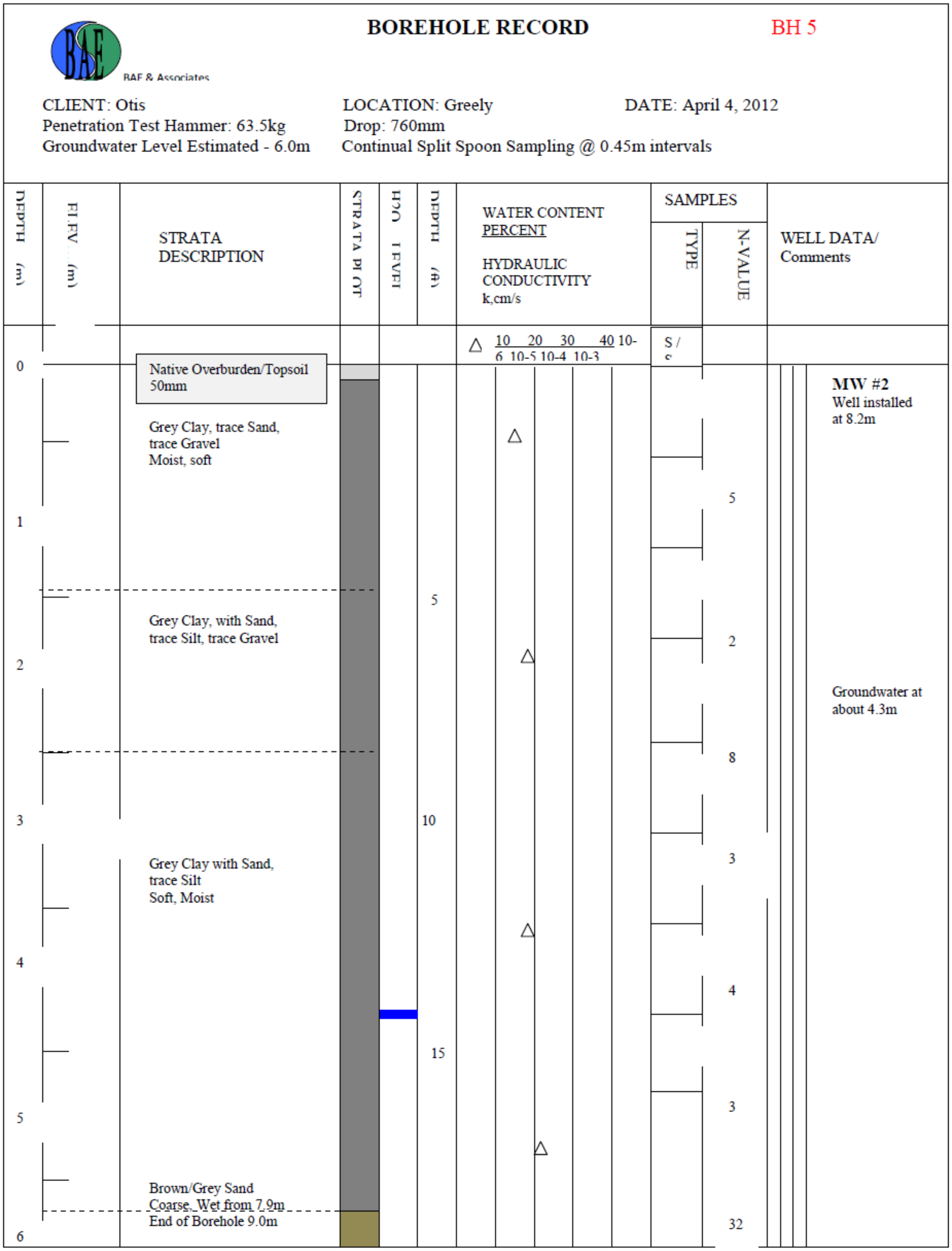
BH 4

BAE & Associates

CLIENT: Otis
LOCATION: Greely
DATE: April 4, 2012
Penetration Test Hammer: 63.5kg
Drop: 760mm
Groundwater Level Estimated - 6.0m
Continual Split Spoon Sampling @ 0.45m intervals

Depth (m)	Strata Description	N-Value
0	Native Overburden/Topsoil 50mm	
0–1.3	Grey Clay, trace Silt, trace Sand Soft, Moist	3
1.3–3.0	Grey Clay, with Sand, trace Silt Some fill debris Moist to Wet Loose	7 13
3.0–4.5	Grey Clay, trace sand, trace gravel Wet, Loose	10 9
4.5–5.0	Brown Sand and Gravel	
5.0	End of Borehole 15m	

WELL DATA/ Comments:
No Well Install
Groundwater at about 3.6m



BAE & Associates

BOREHOLE RECORD

BH 5

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: April 4, 2012

DEPTH (m)	ELEV. (m)	STRATA DESCRIPTION	STRATA PLOT	H2O LEVEL	DEPTH (ft)	WATER CONTENT PERCENT HYDRAULIC CONDUCTIVITY k,cm/s	SAMPLES TYPE	SAMPLES N-VALUE	WELL DATA/ Comments
						△ 10 20 30 40 10-6 10-5 10-4 10-3	S / S		
0		Native Overburden/Topsoil 50mm							MW #2 Well installed at 8.2m
1		Grey Clay, trace Sand, trace Gravel Moist, soft				△		5	
2		Grey Clay, with Sand, trace Silt, trace Gravel			5	△		2	Groundwater at about 4.3m
3		Grey Clay with Sand, trace Silt Soft, Moist			10	△		8 3	
4								4	
5					15	△		3	
6		Brown/Grey Sand Coarse, Wet from 7.9m End of Borehole 9.0m						32	

BOREHOLE RECORD

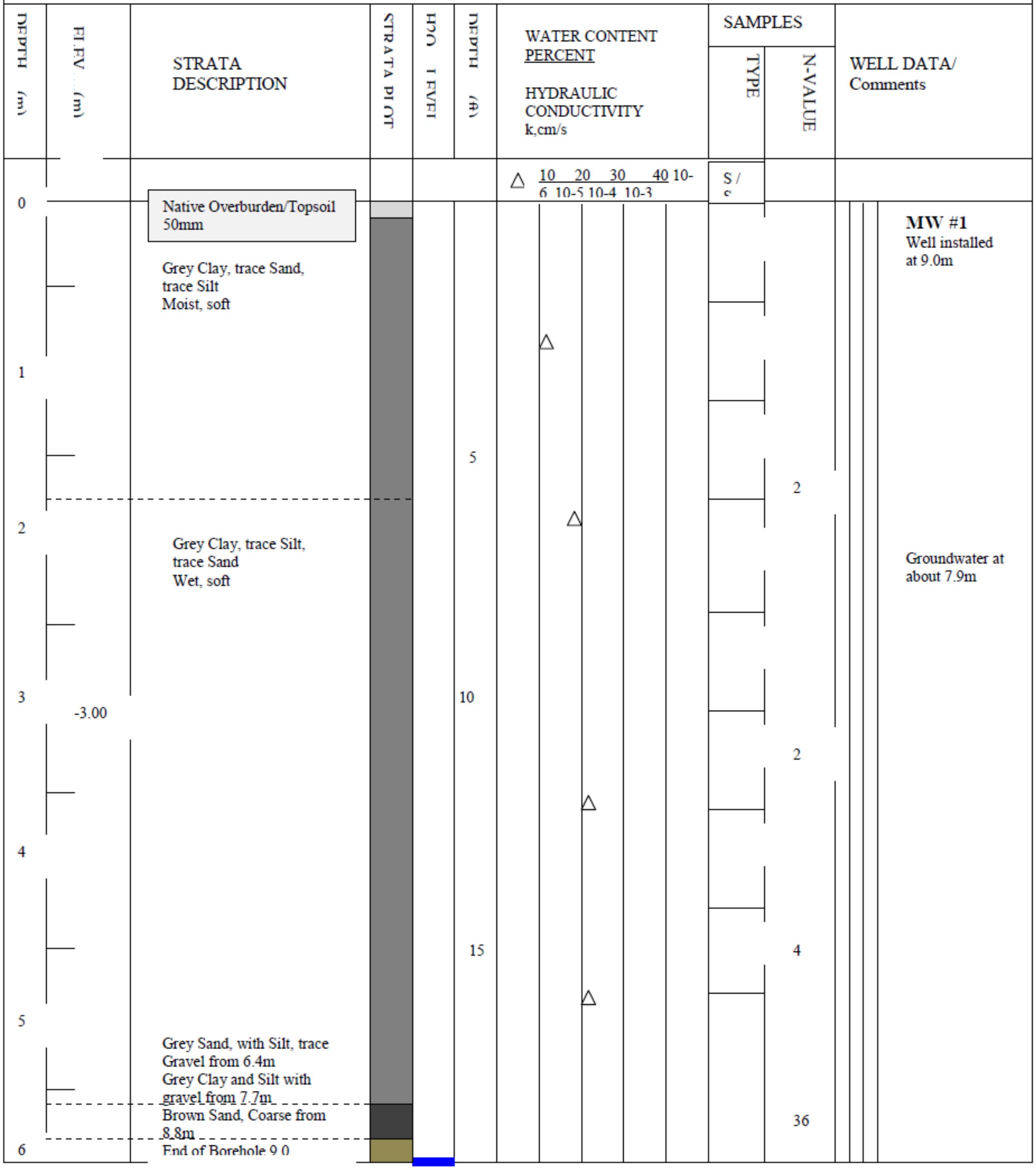
BH 6

BAE & Associates

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: April 4, 2012



Native Overburden/Topsoil 50mm

Grey Clay, trace Sand, trace Silt
Moist, soft

Grey Clay, trace Silt, trace Sand
Wet, soft

Grey Sand, with Silt, trace Gravel from 6.4m
Grey Clay and Silt with gravel from 7.7m
Brown Sand, Coarse from 8.8m
End of Borehole 9.0

MW #1
Well installed at 9.0m

Groundwater at about 7.9m

BOREHOLE RECORD

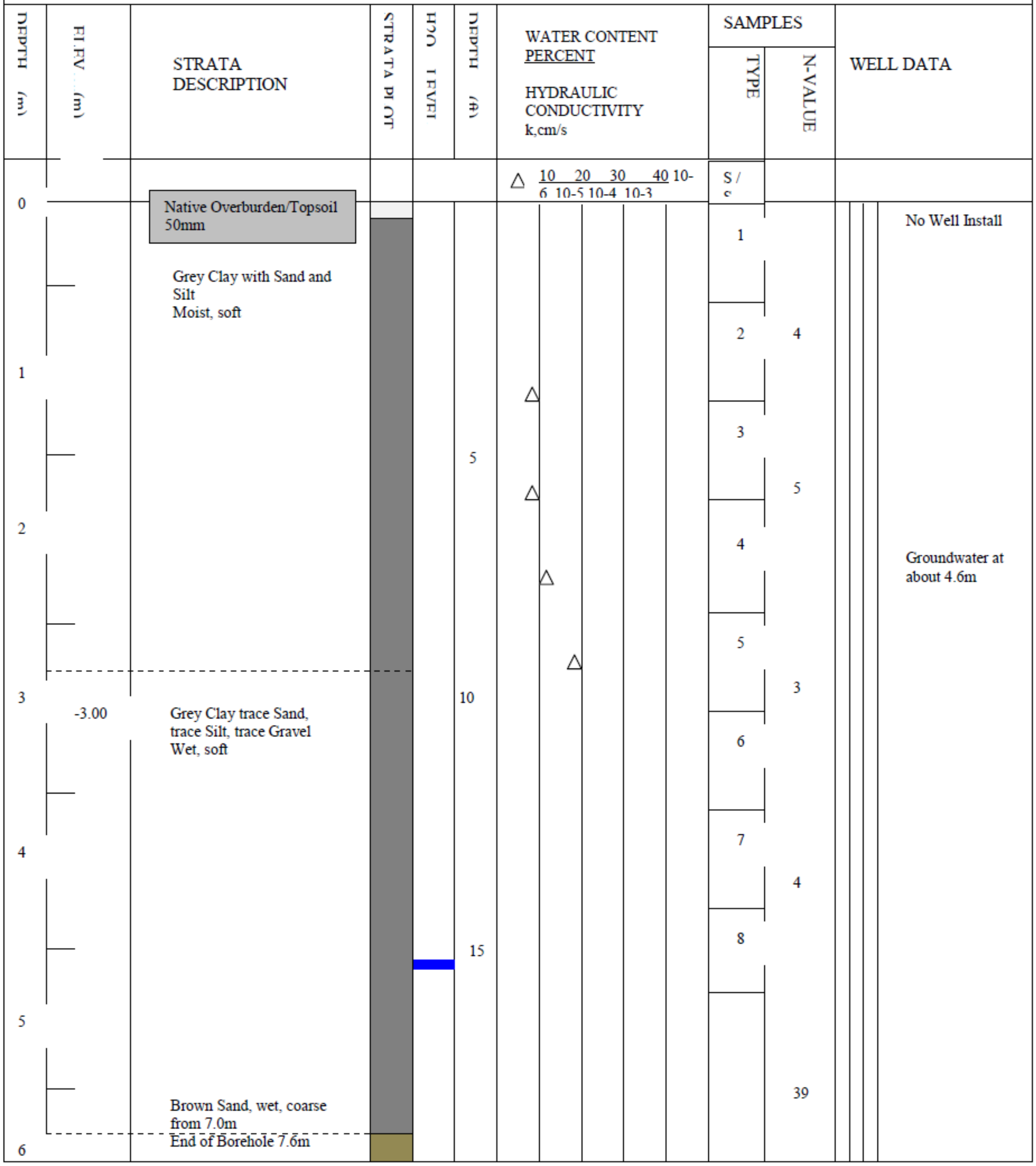
BH 7

BAE & Associates

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: April 4, 2012



BOREHOLE RECORD

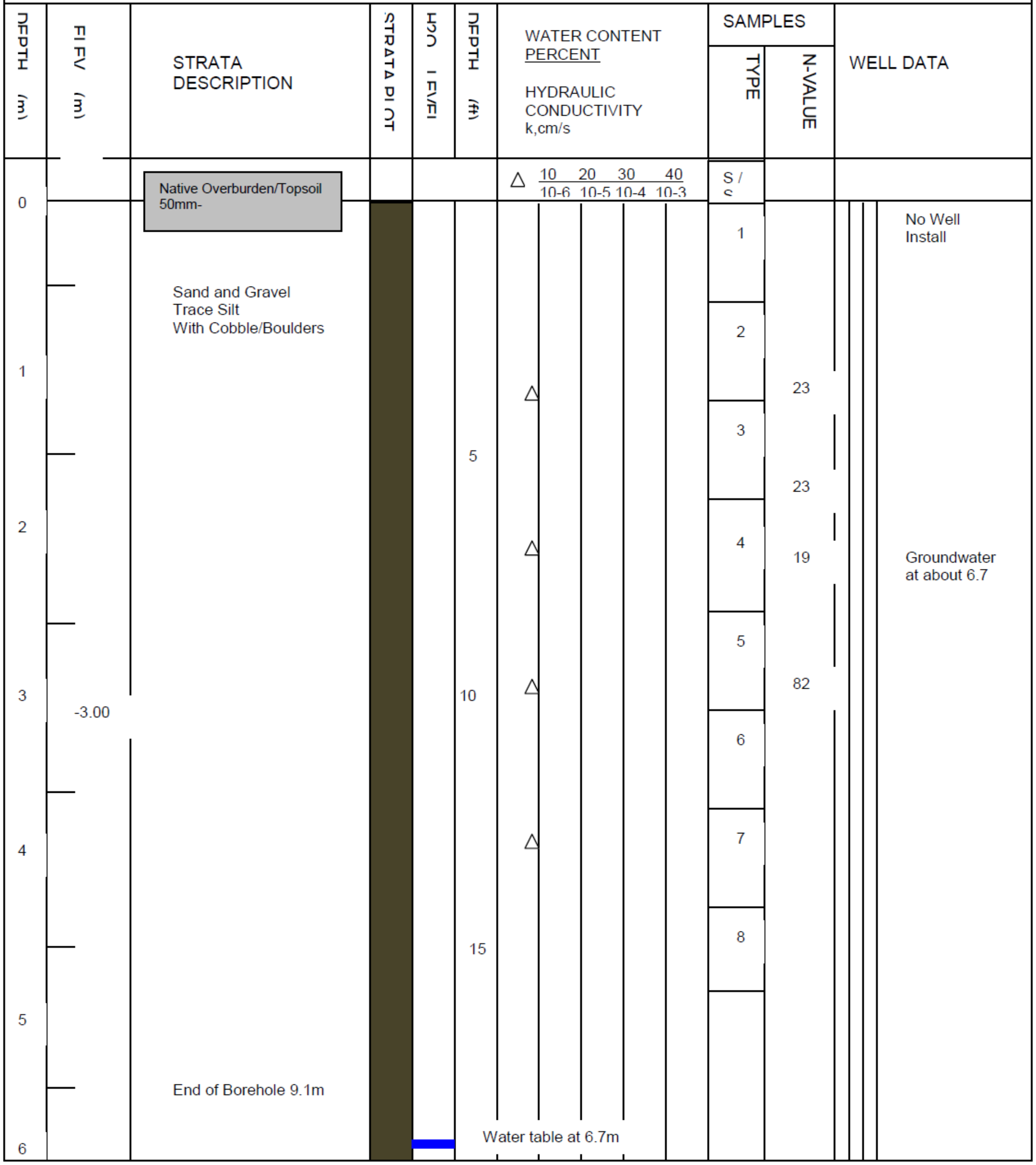
BH 1

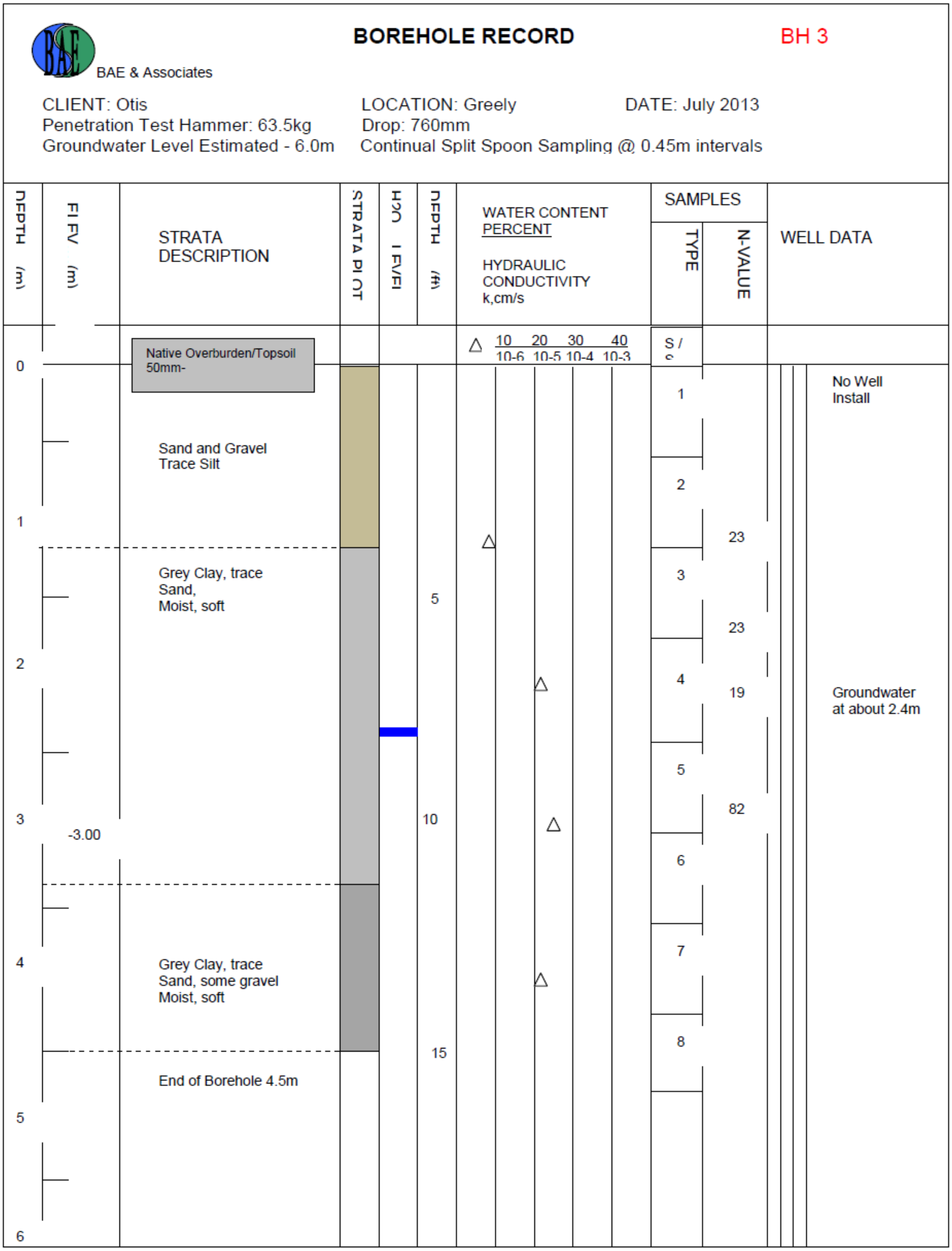
BAE & Associates

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated – 6.7m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: July 2013





BAE & Associates

BOREHOLE RECORD

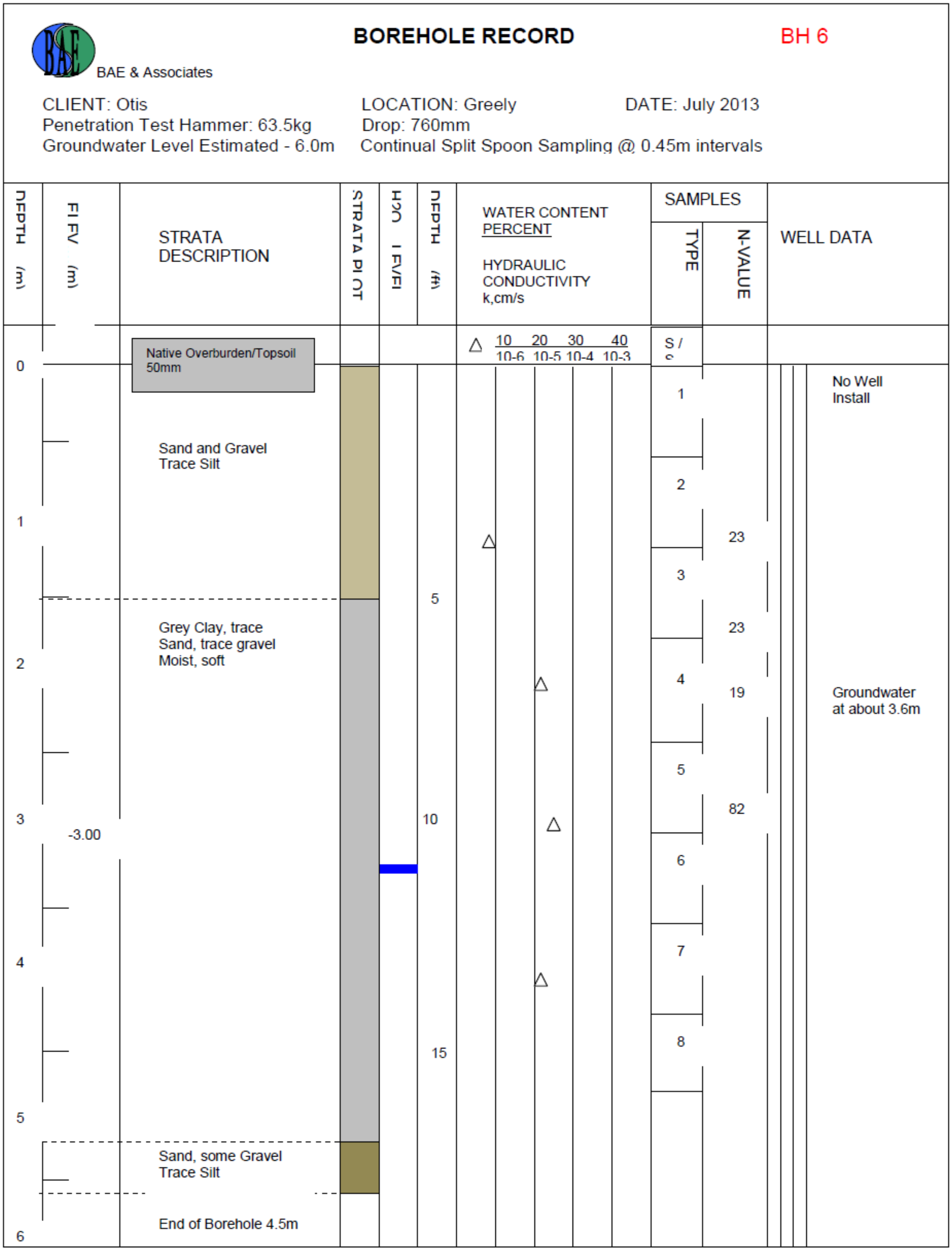
BH 3

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: July 2013

DEPTH (m)	ELEV (m)	STRATA DESCRIPTION	STRATA PLOT	H2O LEVEL	DEPTH (ft)	WATER CONTENT PERCENT HYDRAULIC CONDUCTIVITY k,cm/s	SAMPLES TYPE	SAMPLES N-VALUE	WELL DATA
						△ 10 20 30 40 10-6 10-5 10-4 10-3	S / S		
0		Native Overburden/Topsoil 50mm-							No Well Install
		Sand and Gravel Trace Silt					1		
1							2	23	
		Grey Clay, trace Sand, Moist, soft			5		3	23	
2							4	19	Groundwater at about 2.4m
3	-3.00				10		5	82	
							6		
4		Grey Clay, trace Sand, some gravel Moist, soft					7		
		End of Borehole 4.5m			15		8		
5									
6									



BAE & Associates

BOREHOLE RECORD

BH 6

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: July 2013

Native Overburden/Topsoil 50mm

Sand and Gravel
Trace Silt

Grey Clay, trace
Sand, trace gravel
Moist, soft

Sand, some Gravel
Trace Silt

End of Borehole 4.5m

TYPE	N-VALUE
1	
2	23
3	23
4	19
5	82
6	
7	
8	

No Well Install

Groundwater at about 3.6m

BOREHOLE RECORD

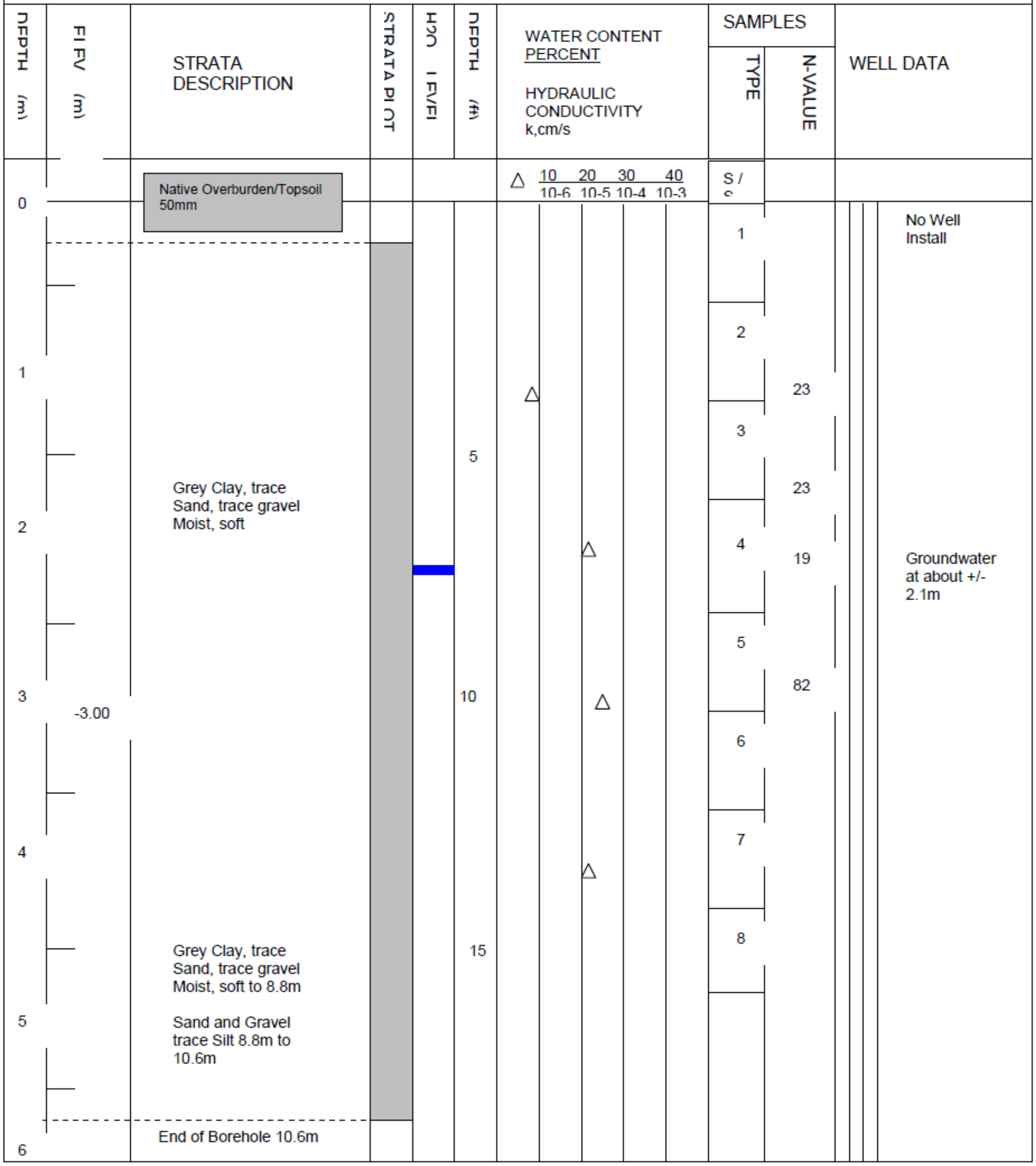
BH 8

BAE & Associates

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: July 2013



BOREHOLE RECORD

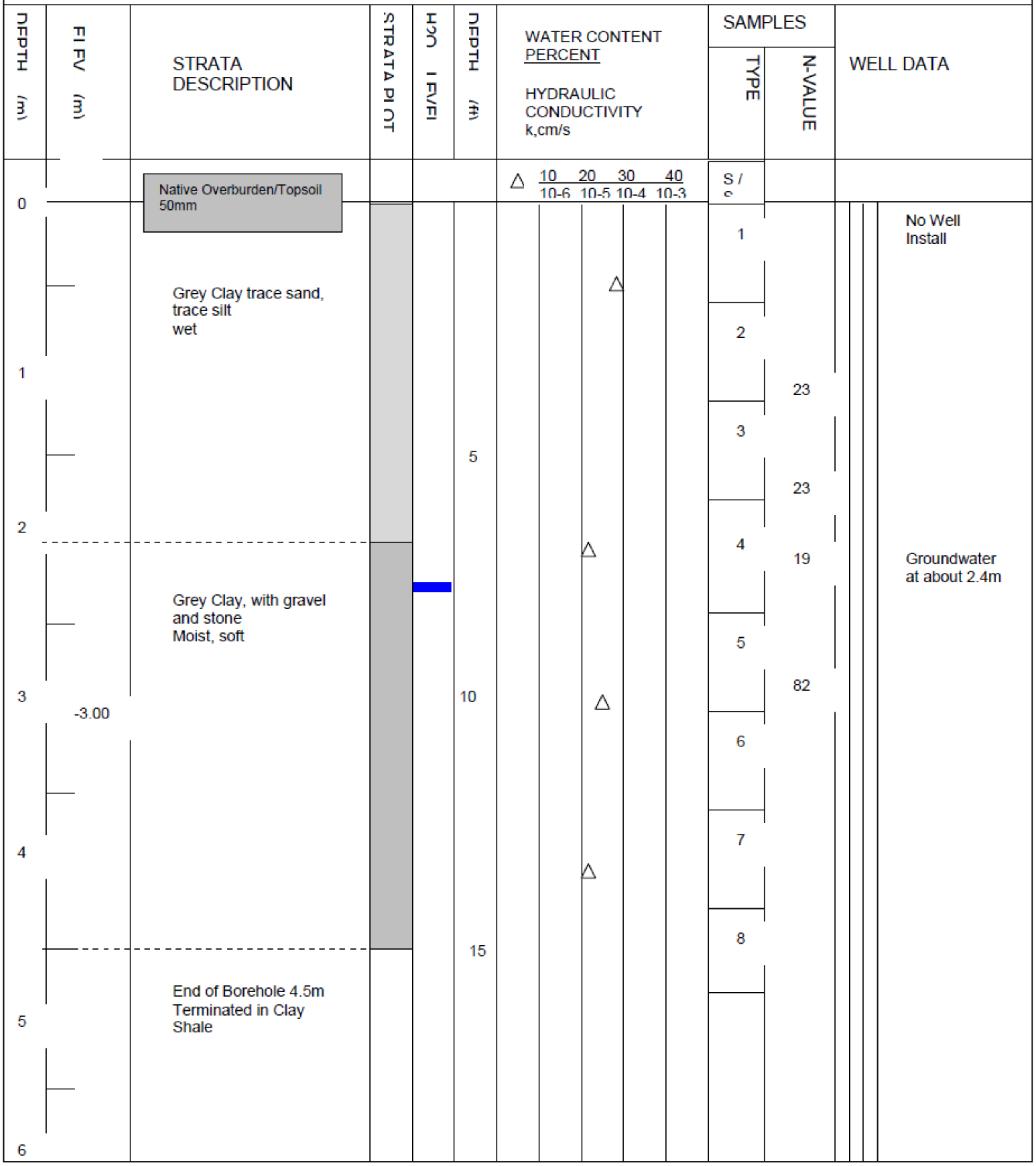
BH 11

BAE & Associates

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: July 2013



BAE & Associates

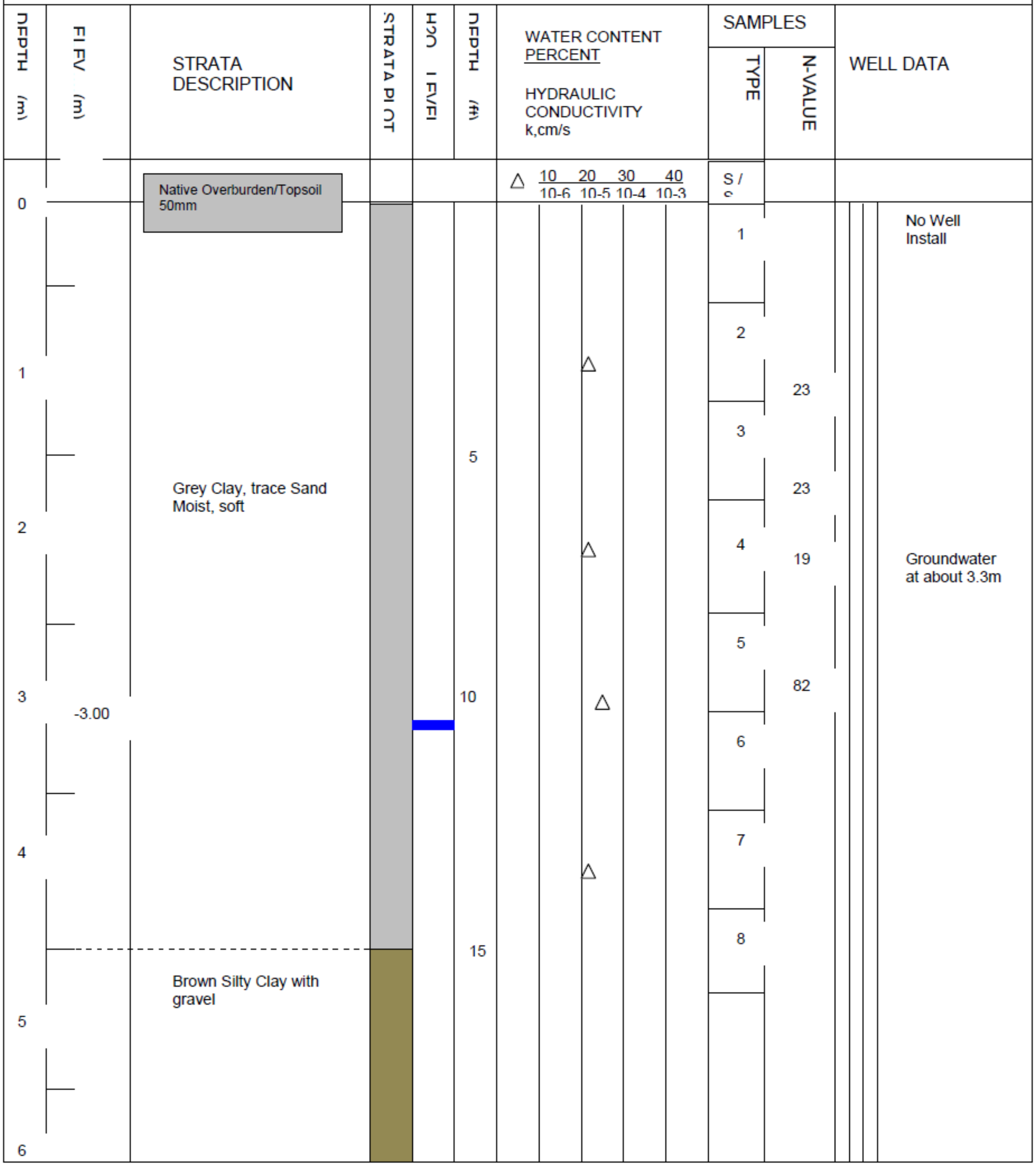
BOREHOLE RECORD

BH 17

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: July 2013



BAE & Associates

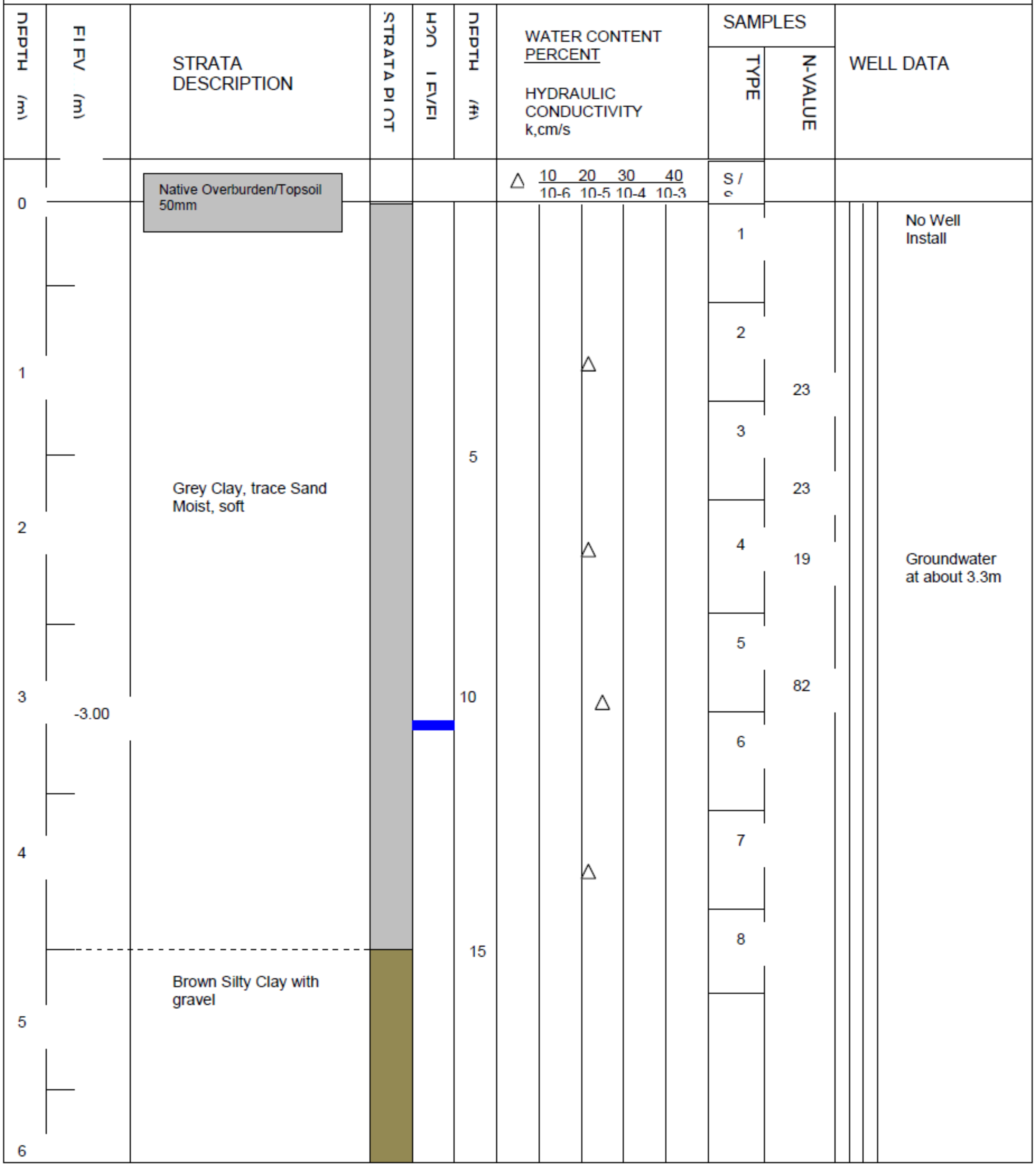
BOREHOLE RECORD

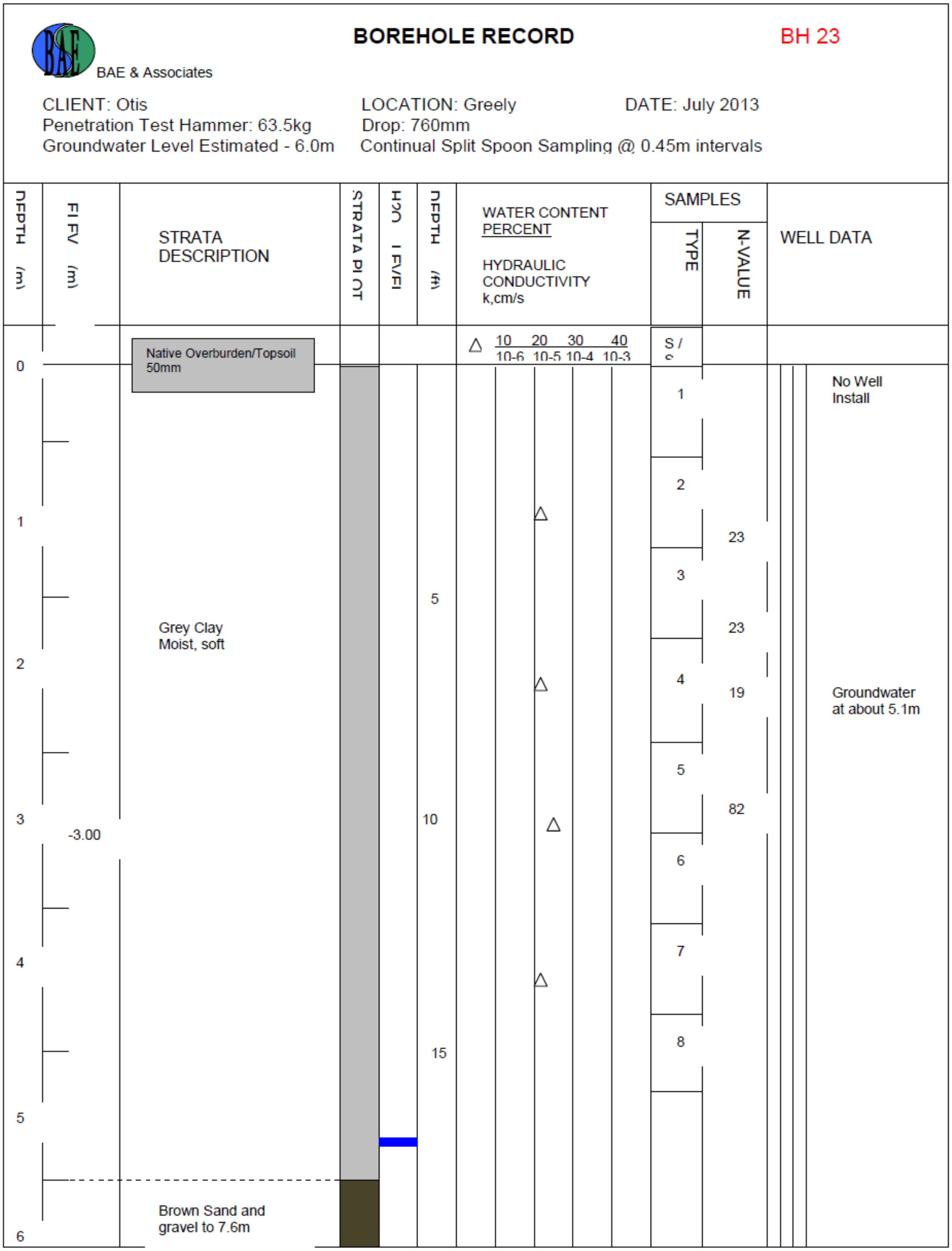
BH 18

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: July 2013





BOREHOLE RECORD

BH 23

BAE & Associates

CLIENT: Otis
Penetration Test Hammer: 63.5kg
Groundwater Level Estimated - 6.0m

LOCATION: Greely
Drop: 760mm
Continual Split Spoon Sampling @ 0.45m intervals

DATE: July 2013

Native Overburden/Topsoil 50mm

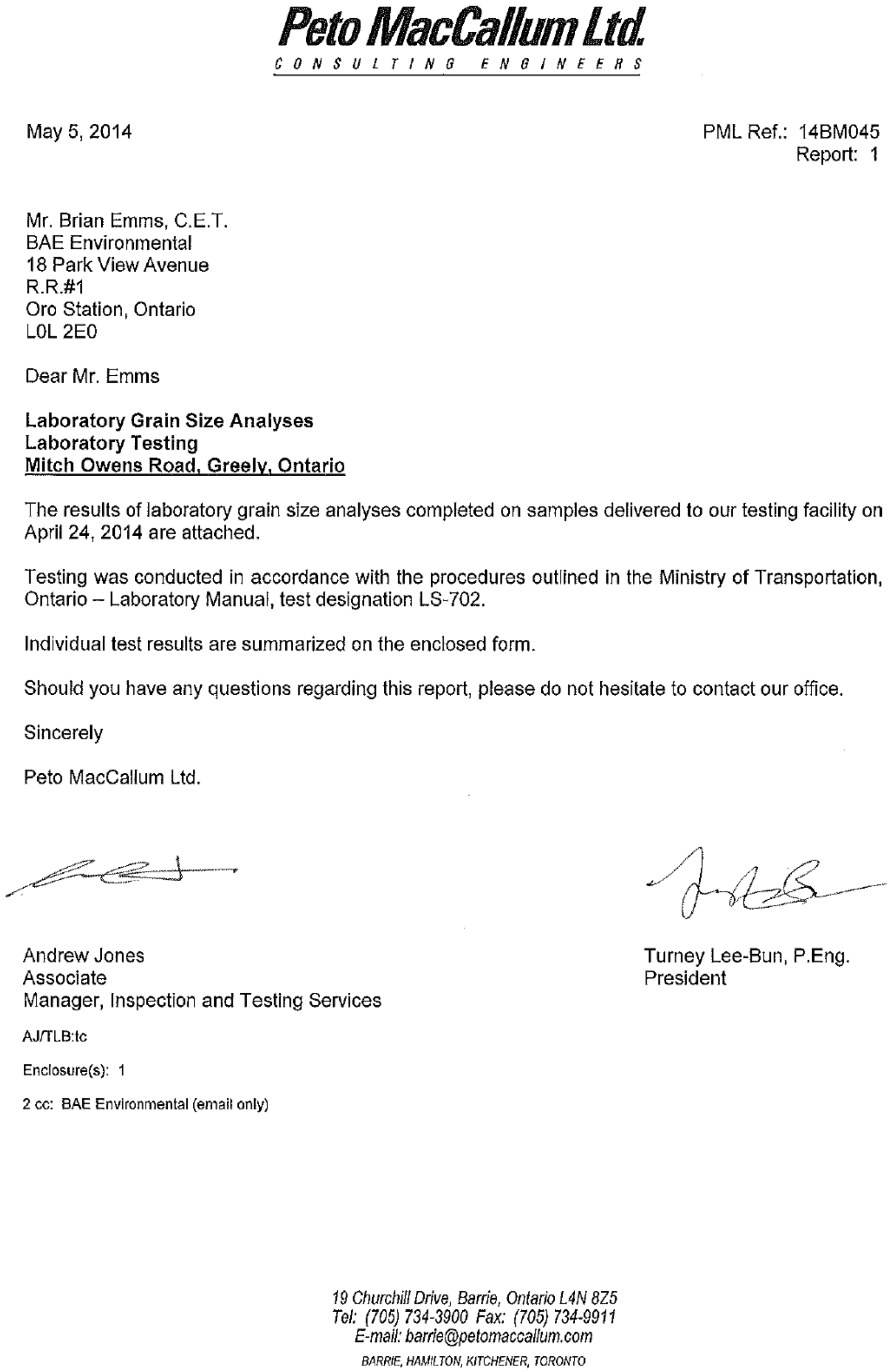
Grey Clay
Moist, soft

Brown Sand and gravel to 7.6m

No Well Install

Groundwater at about 5.1m

Appendix F
Soil Classification – Grain Size Analyses



Peto MacCallum Ltd.
CONSULTING ENGINEERS

May 5, 2014

PML Ref.: 14BM045
Report: 1

Mr. Brian Emms, C.E.T.
BAE Environmental
18 Park View Avenue
R.R.#1
Oro Station, Ontario
L0L 2E0

Dear Mr. Emms

Laboratory Grain Size Analyses
Laboratory Testing
Mitch Owens Road, Greely, Ontario

The results of laboratory grain size analyses completed on samples delivered to our testing facility on April 24, 2014 are attached.

Testing was conducted in accordance with the procedures outlined in the Ministry of Transportation, Ontario – Laboratory Manual, test designation LS-702.

Individual test results are summarized on the enclosed form.

Should you have any questions regarding this report, please do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.

Andrew Jones
Associate
Manager, Inspection and Testing Services

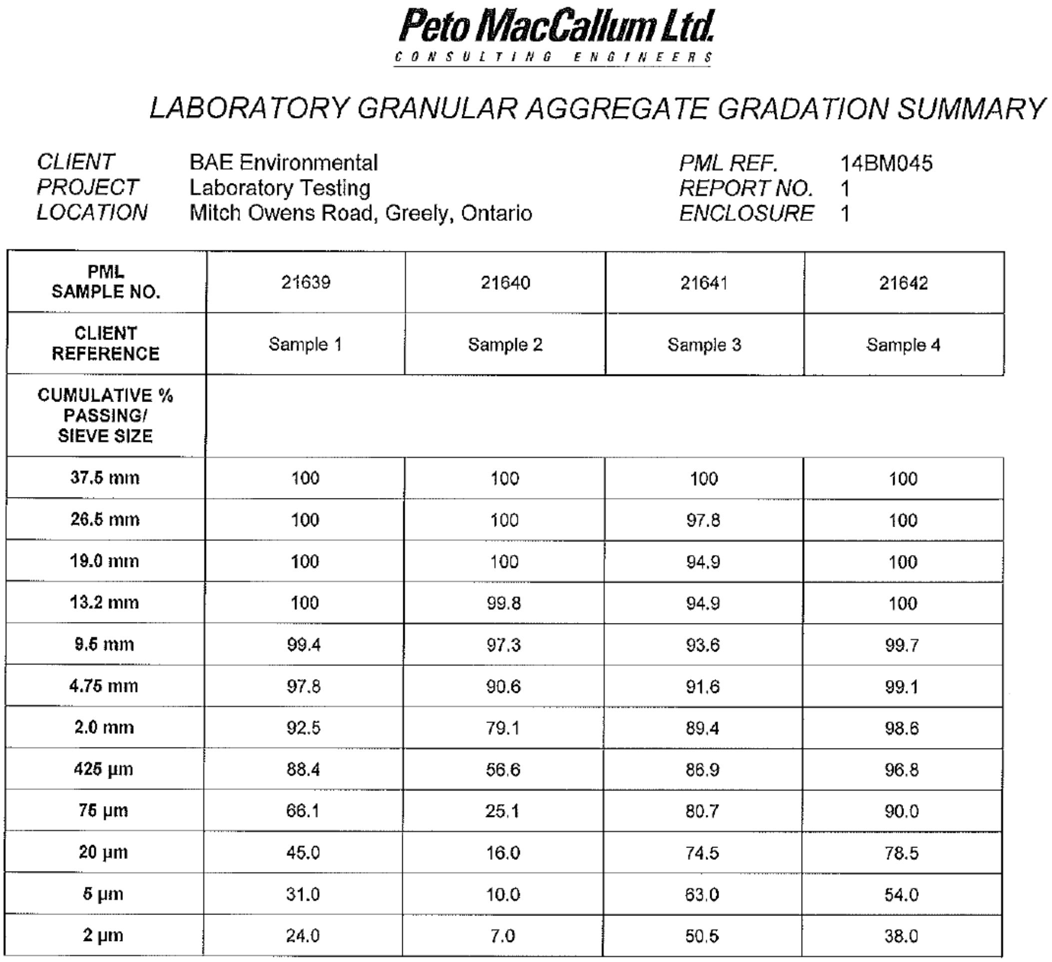
Turney Lee-Bun, P.Eng.
President

AJ/TLB:tc

Enclosure(s): 1

2 cc: BAE Environmental (email only)

19 Churchill Drive, Barrie, Ontario L4N 8Z5
Tel: (705) 734-3900 Fax: (705) 734-9911
E-mail: barrie@petomaccallum.com
BARRIE, HAMILTON, KITCHENER, TORONTO



Peto MacCallum Ltd.
CONSULTING ENGINEERS

LABORATORY GRANULAR AGGREGATE GRADATION SUMMARY

CLIENT	BAE Environmental	PML REF.	14BM045
PROJECT	Laboratory Testing	REPORT NO.	1
LOCATION	Mitch Owens Road, Greely, Ontario	ENCLOSURE	1

PML SAMPLE NO.	21639	21640	21641	21642
CLIENT REFERENCE	Sample 1	Sample 2	Sample 3	Sample 4
CUMULATIVE % PASSING/ SIEVE SIZE				
37.5 mm	100	100	100	100
26.5 mm	100	100	97.8	100
19.0 mm	100	100	94.9	100
13.2 mm	100	99.8	94.9	100
9.5 mm	99.4	97.3	93.6	99.7
4.75 mm	97.8	90.6	91.6	99.1
2.0 mm	92.5	79.1	89.4	98.6
425 µm	88.4	56.6	86.9	96.8
75 µm	66.1	25.1	80.7	90.0
20 µm	45.0	16.0	74.5	78.5
5 µm	31.0	10.0	63.0	54.0
2 µm	24.0	7.0	50.5	38.0

-