Amec Foster Wheeler Environment & Infrastructure
a division of Amec Foster Wheeler Americas Limited
Unit 110, 18568 96th Ave
Surrey, British Columbia
Canada, V2T 4C3
Tel +1 (604) 295-8657
Fax: +1 (604) 295-8658

Geotechnical Site Assessment

Proposed Substation Addition
South Substation Switchgear Replacement Project, Esquimalt Graving Dock,
Esquimalt, BC

Submitted to:

Public Works and Government Services Canada
Vancouver, BC

Submitted by:

Amec Foster Wheeler Environment & Infrastructure,
a division of Amec Foster Wheeler Americas Limited

Surrey, BC

04 December 2015

Amec Foster Wheeler File: KA21133

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1.0 INTRODUCTION

Further to our proposal of October 6, 2015, and preliminary memo of November 17, 2015, Amec Foster Wheeler Environment & Infrastructure (Amec Foster Wheeler) presents this geotechnical site assessment report for the proposed addition to the South Substation at the Esquimalt Graving Dock, Esquimalt, BC.

The proposed addition will be a two storey structure with a plan area of approximately 15 metres by 17 metres. The lower floor level will be approximately 2.5 m below the present ground surface. The existing substation is located on the adjacent site to the east. The existing substation and proposed addition will be connected by duct banks and tunnels and will be required to work together to provide power to the graving dock facilities. The existing and proposed facilities are located on the graving dock jetty as shown on Figure 1. The graving dock was originally constructed in 1926 and has been improved over the years as described in Section 3.0 of the report

2.0 FIELD WORK

We visited the site, which was occupied by a light metal framed maintenance building, on October 15, 2015, to observe site access and work with Public Works and Government Services Canada (PWGSC) personnel to determine a borehole location.

One borehole (BH15-01) was drilled at the location shown in Figure 2, using a sonic drill rig operated by Drillwell Ltd. on October 26 and 27, 2015. Prior to drilling, the surface concrete slab was cored and the borehole was advanced to 2.1 m depth by a Hydrovac rig in order to avoid the underground utilities and ground anchors related to the adjacent sheet pile wall. The test hole was monitored by Amec Foster Wheeler, who classified the encountered soils, maintained detailed logs of test holes and recorded groundwater conditions. Representative soil samples were obtained and classified in general accordance with the Modified Unified Soil Classification System. The soil samples were submitted to Amec Foster Wheeler's Surrey materials testing laboratory to confirm field classifications and conduct tests such as moisture content, fines content, grain size analysis and Atterberg Limits.

3.0 REVIEW OF EXISTING INFORMATION

Many geotechnical investigations have been carried out over the years at the Esquimalt Graving Dock. The two investigations referred to below are most relevant because they contain soils information from the vicinity of the substation site. These are:

- Esquimalt Graving Dock and South Landing Wharf, Site Investigation and Liquefaction Assessment, Final Report, Klohn Crippen Consultants Ltd, June 2002;
- Geotechnical Exploration, Southside Substation Building, Esquimalt Graving Dock, Terra Engineering Ltd, March 1997.

Boreholes BK 02-08 and BK 02-10 from the Klohn Crippen report and DCPT-1 and DCPT-2/TH-2 from the Terra report are shown on Figure 2. Borehole logs from these reports are included in Appendix B.

Original construction of the graving dock, bordering on and extending into the east shore of Esquimalt Harbour, occurred from 1921 to 1926. We understand that a coffer dam was constructed along the south and west sides of the site so that construction could be done under dry conditions. Along the south side, the concrete foundation wall of the graving dock is based on bedrock and backfilled with a clay liner. The space between the clay liner and coffer dam was backfilled with “rubble fill”.

From 1926 to 1940, the top of the fill was used as a roadway, about 15 m wide, along the south side of the graving dock. Since 1940, the south side of the graving dock has undergone a series of changes including expansion of the fill zone and construction of various wharves. The last upgrade was carried out in 1984 to 1985.

Presently, as described in the 2002 Klohn Crippen report, in the vicinity of the proposed substation addition there is a 40 m wide concrete apron along the south side of the graving dock. The fill beneath the apron is retained on the southern/seaward side by a 6 to 9 metre deep sheet pile wall. There are shallow steel anchor rods running from the top of the sheet piles to a retaining wall located 25 m north of and parallel to the sheet piles. The fill on the south side of the sheet pile wall is not retained and slopes downward into the water. A 25 m wide concrete wharf, reportedly supported on steel pipe piles driven to refusal, abuts the sheet pile wall on the south side of the concrete apron. A 35 m wide timber wharf, supported on timber piles, abuts the south side of the concrete wharf.

The proposed substation addition site is located in the filled area between the sheet pile wall and the retaining wall to the north. This area, between the sheet pile wall and retaining wall, is occupied by numerous structures, from the substation to the west end of the jetty.

4.0 SITE CONDITIONS

As described above, the site is located on a concrete apron that is supported on backfill of the graving dock foundation wall. The fill is retained on the south side by a sheet pile wall. The roadway adjacent to the graving dock is located beyond a small retaining wall on the north side. To the east is the existing substation building and to the west is another building believed to be a light structure supported on top of the concrete apron.

The site is presently occupied by a light metal maintenance building likewise supported on the concrete apron.

4.1 Subsurface Conditions

The soil conditions encountered at the borehole consist of the following:

- **Fill – Sand and Gravel with trace cobbles and broken rock**: This material was encountered between 0.4 m and 11.6 m except between 3.2 m and 6.7 m, where silty clay fill was present;
- **Fill – silty Clay**: A 4.5 m thick layer of silty clay fill, with trace sand and shell fragments, was encountered. It is inferred that the fill was comprised of dredged material from the vicinity;
- **Sand**: Loose fine grained sand, with some silt and shells, was encountered between 11.6 m and 14 m depth;
- **Clay**: Underlying the sand is native, soft, medium plastic, silty clay, trace sand and shells. The moisture content was in the range of 28% to 34%. Atterberg plastic limits were about 17% and liquid limits from 32% to 37%. The natural moisture content is near the liquid limit for the soil;
- **Bedrock**: Gneiss was encountered below 25.3 m depth. Thin veneers of dense sand then glacial till were found between the clay and underlying bedrock. The thickness of these materials was 0.3 m and 0.5 m respectively. The borehole was terminated at a depth of 27.1 m.

As well, other boreholes were drilled in the vicinity during previous geotechnical investigations. Klohn Crippen’s BK02-08 was drilled from the concrete apron near the northwest corner of the proposed addition. Coarse sand and gravel fill was encountered to a depth of 1.2 m, then clay fill to a depth of 5.8 m. Coarse sand and gravel fill was encountered again from 5.8 to 8.8 m, below which was Silt (identified as fill) to a depth of 12.8 m. Native clay was encountered at a depth of 12.8 m and bedrock at 19.2 m. At BK02-10, the fill was comprised of sand and gravel to 2.6 m, clay fill from 2.6 to 6.1 m, sand and gravel from 6.1 to 9.9 m and sand fill from 9.9 to 12.2 m. Clay was encountered at 12.2 m and bedrock at 23.6 m.

The Terra Engineering test holes appear to have met refusal in the coarse granular fill at a depth of about 7.6 m and did not extend down into the native clay and bedrock. The shallow fill above 6.4 m consisted of relatively clean sand instead of the silt and clay noted in the other locations.

In general, the results of the boreholes show:

- The fill is of variable composition and extends to a depth of 11.6 to 12.8 m;
- Portions of the fill, consisting of loose to compact sand and gravel and non-plastic silt are expected to be liquefiable during the design earthquake;
- The original sea bottom (encountered 11.6 to 12.8 m below the top of the concrete apron) is comprised of native intermediate plastic clay with intermittent loose sand/silt at the top of the clay; and
- Bedrock, with a thin overlying veneer of dense sand/glacial till is present beneath the clay. The bedrock surface appears to slope down toward the southwest.

Ground water was observed at a depth of 3.1 m below the top of the concrete apron at the time of drilling. Groundwater elevation is expected to vary with the tide. A standpipe piezometer, installed by Klohn Crippen in BK02-08 was read over several days in 2002 and confirmed that water level did indeed vary with the tide. This standpipe has since been decommissioned.

5.0 DISCUSSION AND RECOMMENDATIONS

We understand that the existing substation, constructed in about 1998, has heavily reinforced and thickened perimeter footings and foundation walls and a concrete slab. The substation is reported to be in good functional condition and will be connected to the new addition by duct banks and tunnels. In 1997, a previous building on that site, founded on conventional shallow spread footings, was reported by Terra to be in poor condition and exhibiting signs of differential settlement.

The borehole completed for this investigation, drilled close to and north of the sheet pile wall, encountered 11.6 m of uncontrolled fill of variable composition, portions of which have been in place for more than 80 years. Some future settlement of the fill is possible due to consolidation over time as well as the action of water rising and falling with the tide. However, given that this fill has been in place for many years, we expect only minor future settlement will occur. We noted that there was a 40 mm thick void between the bottom of the concrete slab and underlying soil at the borehole.

As well, portions of the fill may be subject to liquefaction during the design earthquake. Some vertical and lateral movements may be possible due to liquefaction.

The effect of differential settlement within the building can be mitigated by founding the structure on a raft slab, similar to the foundation of the adjacent structure. Based on a review of the soil conditions, prior geotechnical reports, and on the performance of the adjacent raft foundation, it is our opinion that this site is suited for a shallow foundation comprised of a raft. However, the structure would still be subject to displacements related to liquefaction of underlying fill during the design earthquake. If such displacements are unacceptable, deep ground improvement or a deep foundation could be considered.

Static and seismic movements beneath the building could be reduced by installing a grid of stone columns into the native clay, beneath the building site.

As well, a potential foundation system could consist of piles that extend to bedrock. If a piled foundation is used, the piles should be designed to resist lateral forces due to liquefaction as well as down-drag forces due to settlement of the fill. Both piles and stone columns are intrusive methods requiring large equipment that may affect adjacent buildings and infrastructure during construction. As well, we understand that the new and existing facilities need to function together. It may not be desirable for the addition and existing substation to have different foundation types that respond differently to various types of ground movements.

Recommendations regarding the above items and other geotechnical aspects of construction are contained in the following sections.

5.1 Foundations

We understand that the lower floor level will be about 2.5 m below the present surface elevation, about 3.5 m geodetic, and the base of the foundation will be about 3.0 m below the present surface. The applied pressure due to the new foundation is expected to be close to the pressure exerted by the excavated material so that the net additional pressure due to the building, if any, is minor.

An allowable bearing capacity in serviceability limit state (SLS) of 60kPa and a factored bearing resistance of 80kPa in ultimate limit state (ULS) should be used for design of shallow foundations.

The subgrade surface at a depth of 3.0 m may be soft and exhibit deflection due to construction activity, considering that clay fill was encountered at a depth of 3.2 m. Over-excavation of the existing subgrade to a depth of about 1.0 m, placement of a geotextile, then placement of 75 mm minus coarse granular fill may be required in order to achieve a dense subgrade suitable for support of the raft foundation.

The underside of foundations should have at least 450 mm of cover for the purpose of frost protection. Considering the expected foundation depths discussed above, frost heave of foundations is not expected to be of concern.

5.2 Excavation and Dewatering

A foundation excavation in the order of three metres deep will be required for the building foundation. Excavations should be sloped or shored in accordance with Worksafe BC regulations.

Groundwater was encountered at a depth of 3.2 m below the concrete apron during drilling. Higher groundwater levels, above the excavation bottom, can be expected during high tide. Accordingly the contractor should be prepared with pumps to dewater the excavations as required during construction.

With regard to permanent dewatering, the building should be provided with conventional perimeter foundation drains to lower the groundwater elevation in the building footprint and avoid buoyant forces beneath the building. The drain pipe should be at least 300 mm below the top of the lower floor slab. The lower floor will be below design high water elevation of 1.53 m geodetic. We understand that the building will be waterproofed, nevertheless, the client should consider installing a sump/pump system to dewater the building during high water events.

Excavation may be required below the elevation of the base of the foundation of the existing substation. In this case, support of the exposed soil beneath the foundation and/or underpinning of the foundation may be required.

Temporary support of existing duct banks and/or tunnels may be required during construction as well.

5.3 Backfill

Backfill beneath foundations should consist of clean, free draining granular fill compacted in 200 mm thick lifts to at least 100% of Standard Proctor Maximum Dry Density (SPMDD). Backfill of foundation walls, expected to be beneath a paved or concrete surface should likewise be compacted to 100% of SPMDD. Portions of the existing granular fill may be suitable for re-use and should be salvaged as directed by the Geotechnical Engineer.

Quality Assurance geotechnical inspection and compaction testing should be carried out by Amec Foster Wheeler to confirm that specification requirements are satisfied.

5.4 Design Parameters

We recommend the following parameters for design of foundation and retaining walls and other structural elements:

Unit weight of backfill	$\gamma = 20$ kN/m^3
Angle of internal friction for wall backfill	$\phi' = 36^o$
Coefficient of Active Earth Pressure	$K_a = 0.26$
Coefficient of Passive Earth Pressure	$K_p = 3.0$
Coefficient of At Rest Earth Pressure	$K_o = 0.41$
Coefficient of Seismic Earth pressure	$K_{ae} = 0.56$
Coefficient of sliding friction	$\mu = 0.5$
Modulus of Subgrade Reaction	25 MN/m^3

5.5 Existing Anchors

Existing anchors cross the site at regular spacings from south to north. We understand that these anchors are about one metre below the ground surface and connect the top of the sheet piling south of the building with a retaining wall north of the building. Some of these anchors will be removed to enable construction of the new building.

We understand that the anchors are required to resist differential earth pressure between the seaward and landward sides of the sheet piling. The fill surface on the seaward side of the piling is believed to be about 2.0 m below the surface of the concrete slab.

In our opinion, if the ground level on both sides of the sheet piling is at the same elevation, the anchors may not be required. In this case, the south side of the building should be backfilled to the same elevation as the rock fill on the seaward side of the sheet piling. This may require the addition of fill on the seaward side of the sheet piles and/or a lowered surface between the building and sheet pile wall.

The structural engineer may desire to connect the anchors with the raft foundation of the new building. In this case, the connection between the anchors and the foundation should be designed to disengage at a load much smaller than the frictional resistance at the bottom of the foundation.

During construction, anchors may be encountered by excavation slopes along the east and west perimeters. Soil around the anchors should be excavated with care and the anchors exposed and clearly marked in the field. The contractor should replace/repair anchors damaged by excavation activities.

5.6 Seismic Considerations

The National Building Code of Canada (NBCC) requires that structures be designed to resist collapse when subjected to “strong shaking”, defined as ground motions with a return period of 1 in 2,475 years (or two percent probability of exceedance in 50 years).

A secondary objective of the code is to limit damage to buildings caused by low to moderate shaking. NBCC has adopted the use of foundation factors dependent on analysis of ground motion histories adjusted for local site conditions, characterized based on the average shear wave velocity and relative density of the earth materials in the uppermost 30 m. Based on the presence of loose liquefiable sand and soft clay underlying fill, the site should be classified as **Site Class F** in conformance with Table 4.1.8.4 A of the 2010 NBCC. Considering the presence of soft native clay at depth, the F_a and F_v values associated with Site Class E should be used in structural design. Site specific seismic analysis was not carried out for this project.

The geotechnical characterization of seismic site response is based on published ground motions and assumed subsurface stratigraphy and does not take into account potential focusing effects of topography. If it is found that seismic forces govern the design and small changes in the values used significantly alter the design requirements, site specific analysis may be warranted.

Peak Ground Acceleration (PGA) of 0.595 g should be used for this site.

The sand at the elevations between 11.6 m and 14 m is likely liquefiable during the design earthquake. Portions of the fill are expected to be liquefiable as well. The fill composition appears to be variable in the vicinity of the site.

Terra Engineering encountered loose sand in the upper 6.4 m in their test holes on the east side of the existing substation. This material is expected to be liquefiable below the water table as discussed in their report.

In their BK 02-08, near the northwest corner of the site, Klohn Crippen encountered dense, coarse granular fill from 5.8 to 8.8 m, with $(N1)_{60}$ values exceeding 30, between zones of fine grained low plastic silt and clay fill. Their analysis indicated that the fill would not liquefy at that location. Further to the southeast, at BK 02-10, the sand and gravel fill was found to be loose to compact and expected to liquefy from depths of about 6.1 to 12.2 m.

Preliminary calculations, using cyclic stress ratio (CSR) formulated by Seed and Idriss (1971), volumetric reconsolidation strains by Wu (2002) and lateral displacement Index by Zhang and Robertson (2004), the vertical settlement and lateral displacement at BH15-01 were estimated to be 260 mm and 110 mm, respectively, during the one in 2475 years design earthquake. It should be noted that the lateral displacement will be restrained by the docking facilities such as piles. We estimate the differential settlement to be about 140 mm by comparing the available data in borehole BH15-01 and BK 02-08. The differential lateral displacement can be estimated as 50% of the total (55mm). We can perform a more precise site specific seismic analysis as requested if the above estimated settlement impacts the design.

5.7 Ground Improvement

In order to avoid the risk of displacements due to liquefaction, the building footprint could be densified prior to construction. Densification could consist of the construction of a grid of stone columns, typically installed at 2.5 m spacings on a triangular grid. The stone columns are constructed by extending a vibrating mandrel (vibroflot) into the soil down into the native clay. As the surrounding ground densifies, crushed rock fill is placed into the resulting space, creating a dense gravel column. The vibroflot densifies the crushed stone in the resulting column.

Stone columns and other ground improvement techniques typically involve the use of large vibratory equipment that could cause damage to existing facilities if not done in a conscientious manner. Care must be taken so that the stone column installation does not cause settlement beneath adjacent structures. Vibration and settlement of adjacent structures should be monitored during the installation. We expect that stone column installation within about three metres of existing structures is possible without damaging the structures. These installations may not be attractive if vibration sensitive equipment is operating in the existing substation.

Stone column installation will result in a large quantity of water which will have to be collected and possibly treated before disposal. A settlement pond for such water and for water related to deep excavations may be required on the site as part of the planned construction.

Stone column installation is expected to cost in the order of $20.00 per m^3 of ground improved. Mobilization of equipment would be additional and for a small site mobilization will be significant portion of the cost. Additional cost could also occur due to access difficulties on a site with restricted space and ongoing operations requiring the work to be done in specified windows of time.

As stated in Section 5.0, densification of the ground beneath the addition may not be feasible if the ground beneath the existing substation is not densified as well. We understand that renovation is planned for the existing substation following construction of the new addition. If that renovation requires new foundations, ground improvement in sequence beneath both sites may be an attractive option.

5.8 Deep Foundations

Steel pile piles with hardened tips, based on bedrock, are also possible for the proposed addition. A structure founded on piles can be designed so that it is not susceptible to static or seismic ground displacements. However, as with ground improvement, it may not be feasible for one part of the substation facility to be on piles and the other part to be on a shallow foundation. The comments in Section 5.7 regarding vibrations and large equipment apply to pile driving as well.

Piling may be attractive if new foundations are to be constructed on both new and existing substation sites. We can provide parameters for piles driven to bedrock if desired. Lateral pile analysis will be required as part of the pile design.

6.0 LIMITATIONS AND CLOSURE

This memo has been prepared for the exclusive use of PWGSC and their appointed agents for the specific application to the development described within this memo. Any use which a third party makes of this memo, or any reliance on or decisions made based on it are the responsibility of such third parties. Amec Foster Wheeler accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this memo. It has been prepared in accordance with general accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

Amec Foster Wheeler trusts this meets your immediate requirements. If you have any questions or require further information, please contact us.

Respectfully submitted,

Amec Foster Wheeler Environment & Infrastructure,
a division of Amec Foster Wheeler Americas Limited

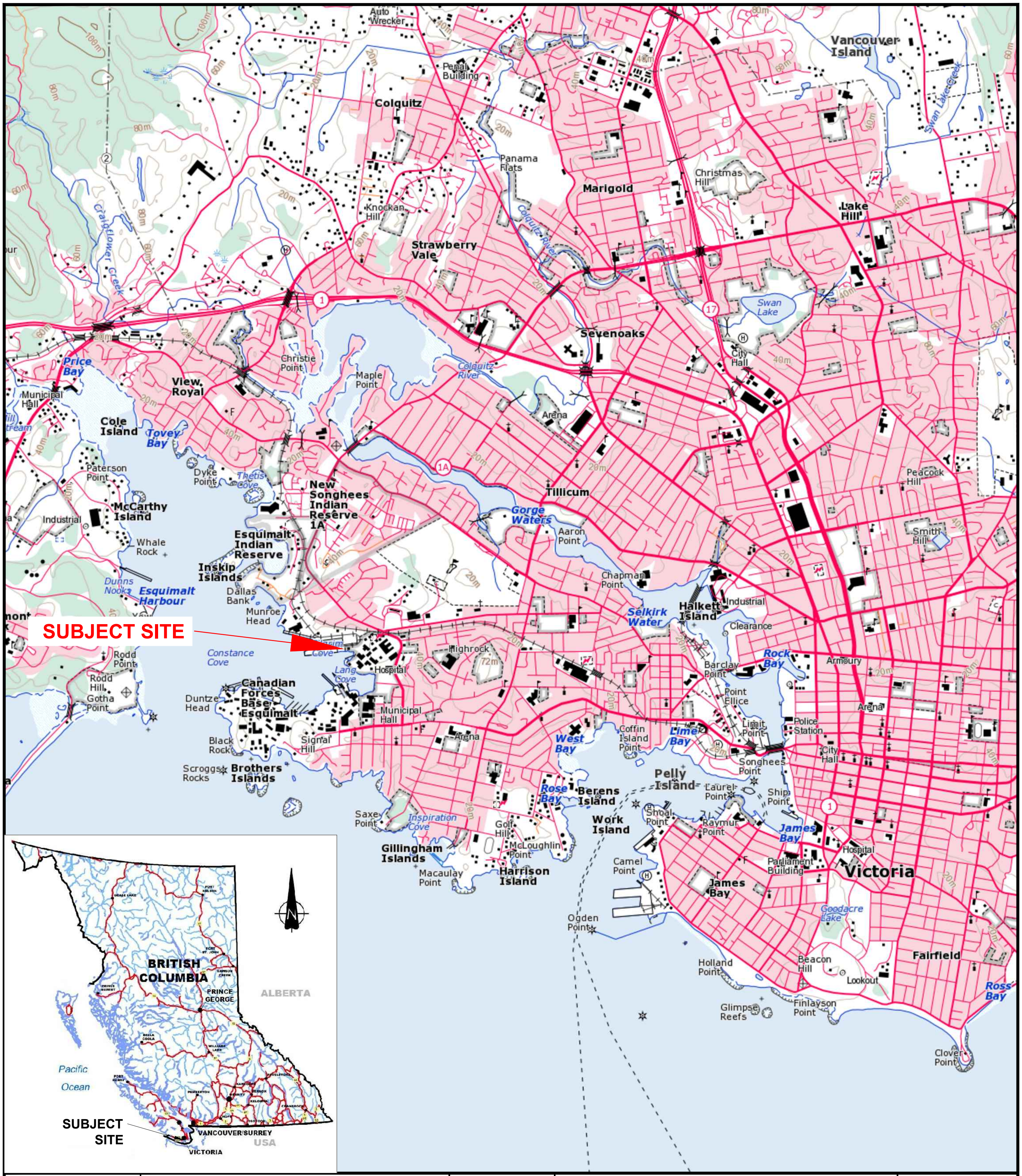
R. J. FORSYTH
4/XII/2015

RJ Forsyth, M.B.A., P.Eng.
Associate Geotechnical Engineer
Manager, Surrey Office

Reviewed by

Darryl Hawkes, P.Eng.
Associate Geotechnical Engineer

FIGURES



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Environment & Infrastructure

CLIENT AND LOGO:

PWGSC

Amec Foster Wheeler
Environment & Infrastructure
Unit 110, 18568 - 96 Avenue
Surrey, BC V4N 3P9
Tel. 604-295-8657 Fax 604-295-8658

DWN BY: EDS

CHK'D BY: BF

DATUM: NAD 83

PROJECTION: UTM Zone 10U

SCALE: NTS

PROJECT:
GEOTECHNICAL ASSESSMENT
SOUTH SUBSTATION ADDITION
ESQUIMALT GRAVING DOCK

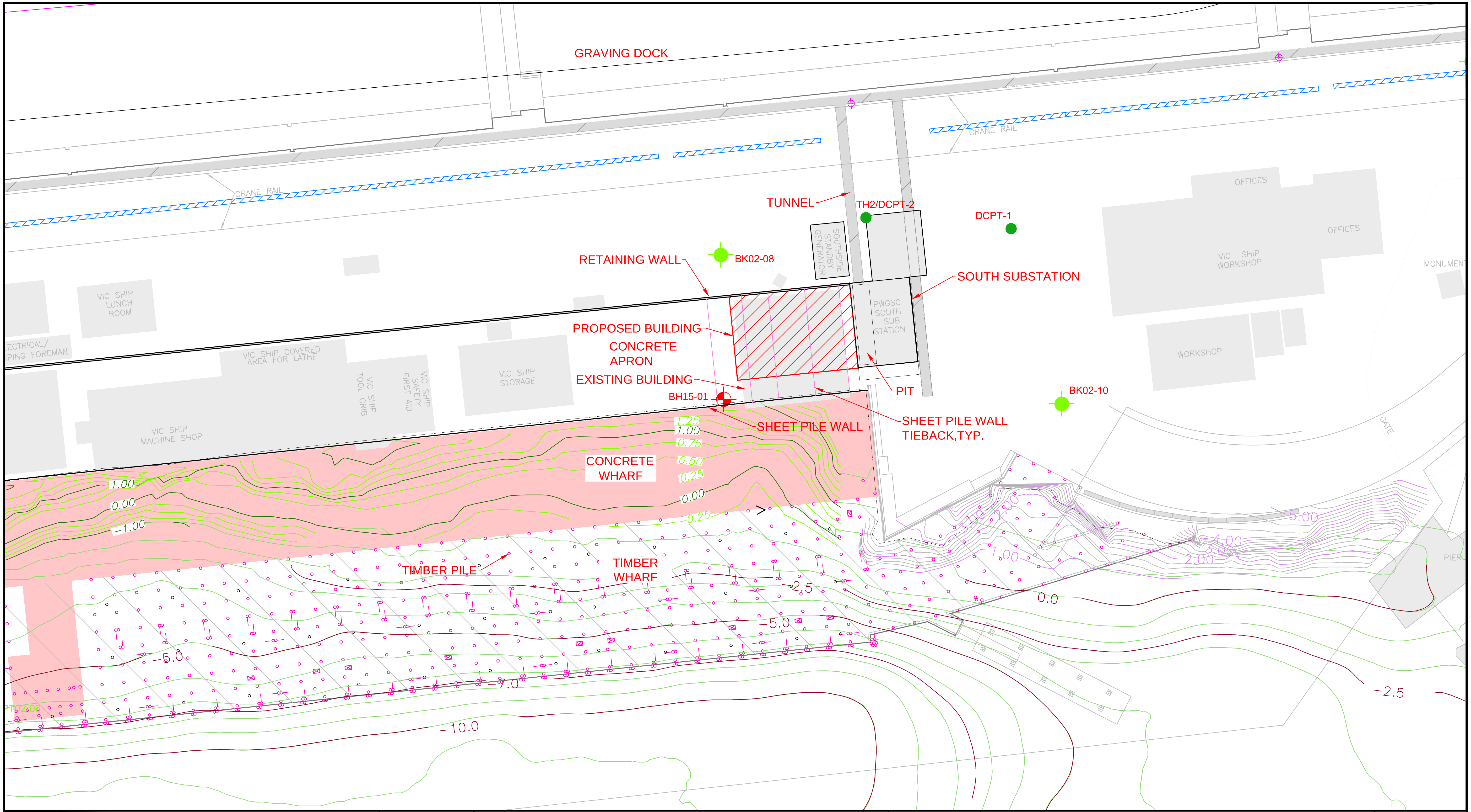
TITLE:
SITE LOCATION PLAN

DATE: NOV 2015

PROJECT NO: KA21133

REV. NO: A

FIGURE NO: 01



LEGEND

BH15-01 ⊕ BOREHOLE (AMECFW OCT/15)

BK02-08 ● CPT02-08 ● TEST HOLES (KC 2002)

DCPT-1 ● TEST HOLE (TERRA 1997)

REFERENCE DRAWING:
PAC#351825-V1-PWGSC-BASE_MAP-TURKAN_(31_MARCH_S013).CAD

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CLIENT AND LOGO:
PWGSC

Amec Foster Wheeler
Environment & Infrastructure
Unit 110, 18568 - 96 Avenue, Surrey, BC V4N 3P9
Tel. 1-604-295-8657 Fax 1-604-295-8658

DWN BY: EDS
CHK'D BY: BF
DATUM: NAD 83
PROJECTION: UTM Zone10U
SCALE: AS SHOWN

PROJECT:
GEOTECHNICAL ASSESSMENT
SOUTH SUBSTATION ADDITION
SOUTH JETTY - ESQUIMALT GRAVING DOCK,

TITLE:
BOREHOLE LOCATION PLAN

DATE: DEC 2015
PROJECT NO: KA21133
REV. NO: A
FIGURE NO: 002

APPENDIX A
Borehole Logs

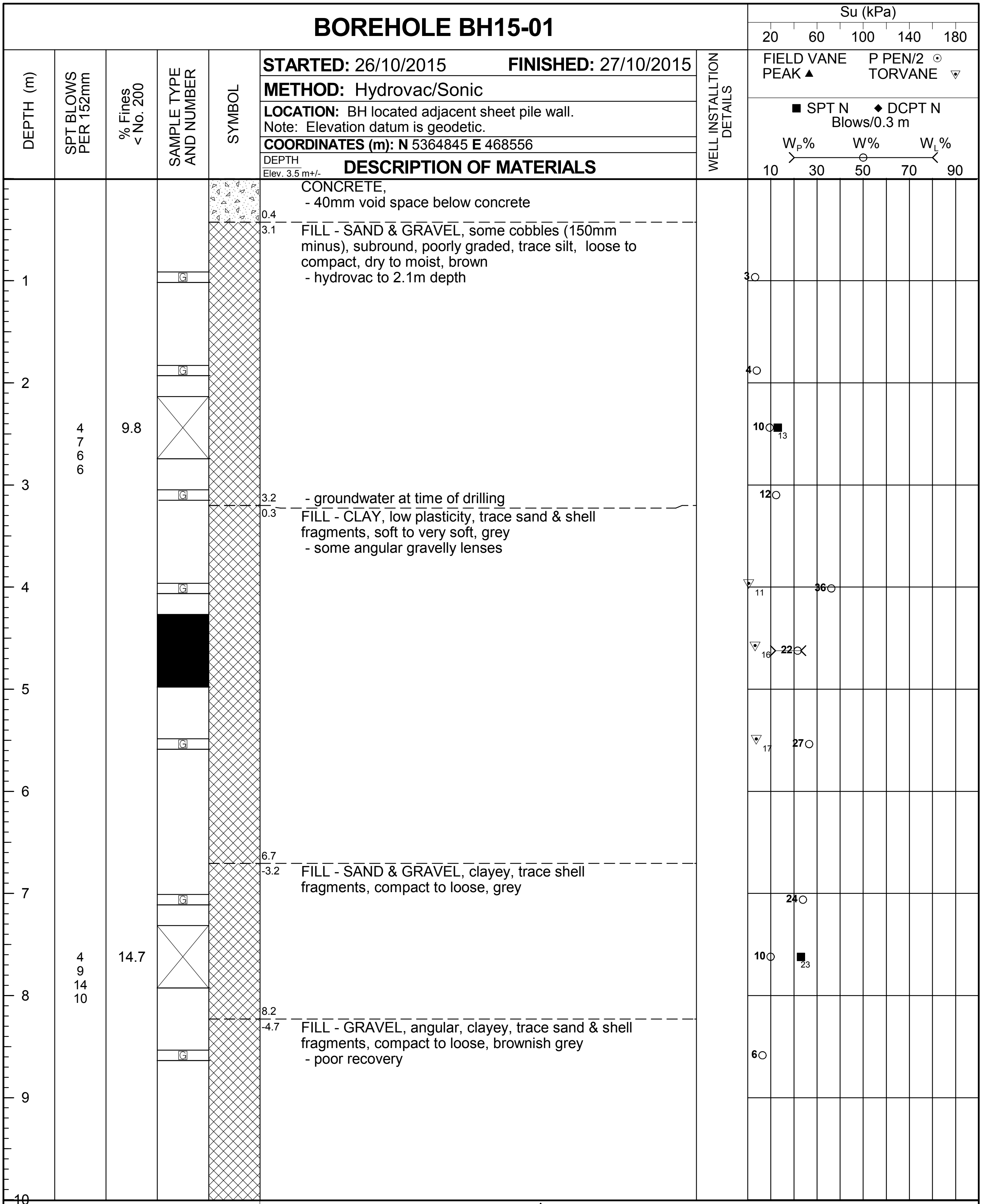
BOREHOLE BH15-01

STARTED: 26/10/2015 **FINISHED:** 27/10/2015

METHOD: Hydrovac/Sonic

LOCATION: BH located adjacent sheet pile wall.
Note: Elevation datum is geodetic.

COORDINATES (m): N 5364845 **E** 468556



DESCRIPTION OF MATERIALS

CONCRETE,
- 40mm void space below concrete

FILL - SAND & GRAVEL, some cobbles (150mm minus), subround, poorly graded, trace silt, loose to compact, dry to moist, brown
- hydrovac to 2.1m depth

- groundwater at time of drilling

FILL - CLAY, low plasticity, trace sand & shell fragments, soft to very soft, grey
- some angular gravelly lenses

FILL - SAND & GRAVEL, clayey, trace shell fragments, compact to loose, grey

FILL - GRAVEL, angular, clayey, trace sand & shell fragments, compact to loose, brownish grey
- poor recovery

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Surrey, B.C. V4N 3P9

PROJECT NO.: KA21133

PROJECT: South Substation Switch Gear Building

LOCATION: Esquimalt Graving Dock - South Jetty

LOGGED BY: EDS **REVIEWED BY:** BF

SHEET 1 OF 3 **BOREHOLE No. BH15-01**

BOREHOLE BH15-01

STARTED: 26/10/2015 **FINISHED:** 27/10/2015

METHOD: Hydrovac/Sonic

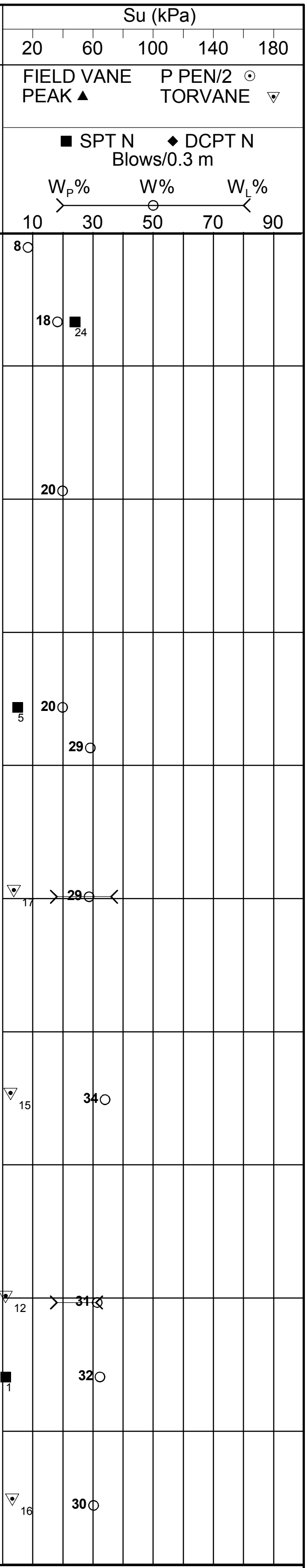
LOCATION: BH located adjacent sheet pile wall.
Note: Elevation datum is geodetic.

COORDINATES (m): N 5364845 **E** 468556

DEPTH Elev. 3.5 m+/-	DESCRIPTION OF MATERIALS
10.4 -6.9	FILL - SAND & GRAVEL, silty, trace shell fragments, compact to loose, grey, occasional cobble/small boulder
11.6 -8.1	SAND, fine grained, trace silt, shells throughout, loose, grey, - poor recovery
13.9 -10.4	CLAY, intermediate plasticity, trace sand partings & shells, very soft to soft, grey

SPT BLOWS PER 152mm	% Fines < No. 200
4 7 17 9	20.5
	8.5
8 3 2 2	
WH 1 WH 1	

DEPTH (m): 11, 12, 13, 14, 15, 16, 17, 18, 19, 20



PROJECT NO.: KA21133

PROJECT: South Substation Switch Gear Building

LOCATION: Esquimalt Graving Dock - South Jetty

LOGGED BY: EDS **REVIEWED BY:** BF

SHEET 2 OF 3 **BOREHOLE No. BH15-01**

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18568 - 96 Avenue, Unit 110
Surrey, B.C. V4N 3P9

BOREHOLE BH15-01

STARTED: 26/10/2015 **FINISHED:** 27/10/2015

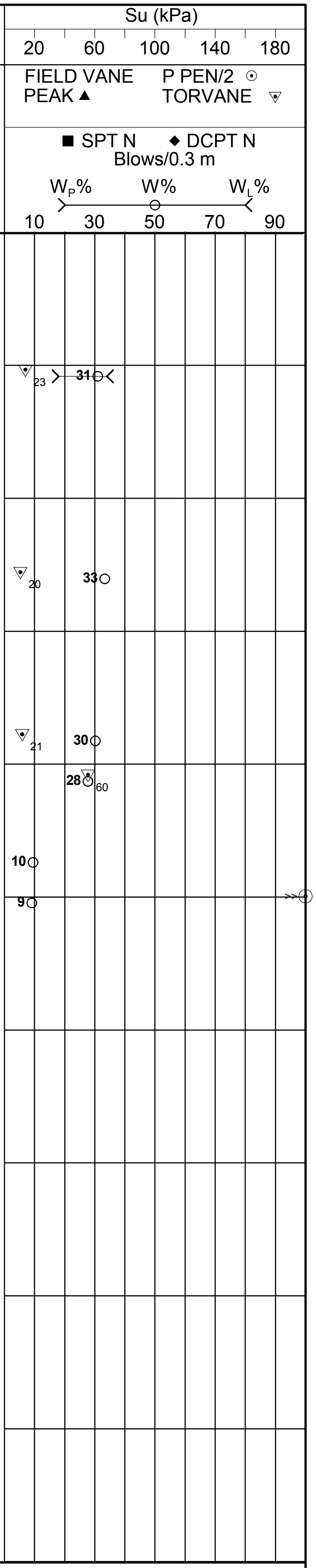
METHOD: Hydrovac/Sonic

LOCATION: BH located adjacent sheet pile wall.
Note: Elevation datum is geodetic.

COORDINATES (m): N 5364845 **E** 468556

DEPTH (m) | SPT BLOWS PER 152mm | % Fines < No. 200 | SAMPLE TYPE AND NUMBER | SYMBOL | WELL INSTALLTION DETAILS

DEPTH Elev. 3.5 m+/-	DESCRIPTION OF MATERIALS	SAMPLE TYPE AND NUMBER
	CLAY, intermediate plasticity, trace sand partings & shells, very soft to soft, grey *(continued)*	
	- trace gravel below 21m	G (21.0)
		G (22.6)
	- some gravel, firm below 24m	G (23.8) G (24.1)
24.5 -21.0	SAND, well graded, gravelly (fine), trace silt, grey	G (24.7)
24.8 -21.3	GLACIAL TILL - Clay, gravelly, very hard, grey	G (25.0)
25.3 -21.8	BEDROCK, Gneiss, strong, greenish grey - full recovery with sonic core	
27.1 -23.6	End of Borehole - backfill with bentonite chips for full depth and concrete plug at surface. - concrete core 0 - 0.4 m - Hydrovac 0.4 - 2.1 m - Sonic drill 2.1 - 27.1 m	



PROJECT NO.: KA21133

PROJECT: South Substation Switch Gear Building

LOCATION: Esquimalt Graving Dock - South Jetty

LOGGED BY: EDS **REVIEWED BY:** BF

SHEET 3 OF 3 **BOREHOLE No. BH15-01**

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Environment & Infrastructure
18568 - 96 Avenue, Unit 110
Surrey, B.C. V4N 3P9

APPENDIX B
Borehole Logs by Others

BECKER TEST HOLE LOG

STARTED: Mar 26, 2002 **FINISHED:** Mar 27, 2002

DRILL RIG MODEL: BECKER HAV180

GROUND ELEV. (m): 4.4

COORDINATES (m): N 468556.3 E 5364864.2

BECKER PENETRATION TEST

Casing O.D.: 170 mm	CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated	OPEN ENDED (blows/0.3m)
● SPT N	BOUNCE PRESS (kPa)

$W_P\%$ × – – – – W% ○ – – – – × $W_L\%$

20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.
1		Grab	1
2		Grab	2
3	2,0,0	SPT	2b
4		Grab	3
5			
6	5,8,24	SPT	4
		Grab	5
7			
8			
		Grab	6
9	2,2,4	SPT	7
10		Grab	8
11		Grab	9
12		Grab	10
13		Grab	11
14			
15		Shelby	12
16		Grab	13
17		Grab	14
18			
19		Grab	15
20			

DESCRIPTION OF MATERIALS

0.2 / 4.2 ASPHALT (5 cm) and CONCRETE (10 cm).

SAND and GRAVEL (SW, GW), fine to coarse, silty, angular to subrounded, grey to black, moist, trace seashells (FILL).

1.2 / 3.2 SILT (ML), low plasticity, clayey, sandy, trace to some gravel, soft to firm, grey, wet, trace seashells (FILL).

5.8 / -1.4 SAND and GRAVEL (SW, GW), fine to coarse, trace to some silt, trace cobble, angular to subangular, grey to black, wet, trace seashells (ROCKFILL).

8.8 / -4.4 SILT (ML), low plasticity, trace to some fine to coarse sand and gravel, soft, grey, wet, trace wood pieces and seashells (FILL).

12.8 / -8.4 CLAY (CI), medium plasticity, trace sand, firm to stiff, grey, wet.

18.3 / -13.9 SAND and GRAVEL (SW, GW), fine to coarse, silty, trace to some clay, occasional cobble, subangular to subrounded, grey, wet.

19.2 / -14.8 End of Becker Drill Hole at 19.2 m

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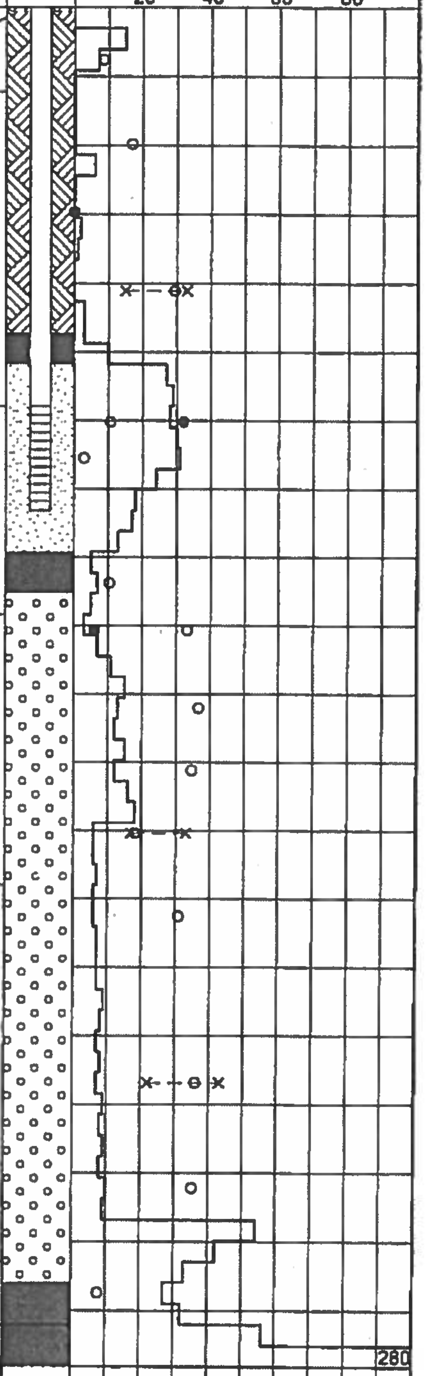
INSTRUMENT DETAILS



KLOHN CRIPPEN

PROJECT NO.: PG9060 01

PROJECT: PWGSC EGD and SLW Site Investigation

LOCATION: Esquimalt, BC

LOGGED BY: TT **CHECKED BY:**

SHEET 1 **OF** 2 **HOLE NO.:** BK02-08

BECKER TEST HOLE LOG

STARTED: Mar 26, 2002 **FINISHED:** Mar 27, 2002

DRILL RIG MODEL: BECKER HAV180

GROUND ELEV. (m): 4.4

COORDINATES (m): N 468556.3 E 5364864.2

BECKER PENETRATION TEST

Casing O.D.: 170 mm	CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated	OPEN ENDED (blows/0.3m)
● SPT N	BOUNCE PRESS (kPa)

$W_P\%$ × — — o — — × $W_L\%$, W% 20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
21						
22						
23						
24						
25						
26						
27						
28						
29						
30						
31						
32						
33						
34						
35						
36						
37						
38						
39						
40						

Notes:

1. Drilling was carried out by Foundex Explorations Ltd. of Surrey, B.C.
2. BPT02-08 was carried out approximately 2 m east of the open ended Becker hole, BOC02-08.
3. Piezometer construction details:

0.00 m - 0.15 m	Concrete mixture;
0.15 m - 4.72 m	Cement-bentonite grout;
4.72 m - 5.18 m	1/4" Coated bentonite tablets;
5.18 m - 7.92 m	Silica filter sand;
7.92 m - 8.53 m	1/4" Coated bentonite tablets;
8.53 m - 18.59 m	Pea gravel;
18.59 m - 19.81 m	Medium bentonite chips;
Surface to 5.79 m	25 mm O.D. PVC pipe;
5.79 m - 7.32 m	No. 10 slot, 25 mm O.D. PVC pipe.

4. The piezometer was subsequently abandoned and backfilled on April 2, 2002.
5. Standard penetration tests (SPT) were conducted using an automatic trip hammer (63.5 kg), AWJ rods and a split-spoon sampler (51 mm O.D., 38 mm I.D.).

KLOHN CRIPPEN

PROJECT NO.: PG9060 01

PROJECT: PWGSC EGD and SLW Site Investigation

LOCATION: Esquimalt, BC

LOGGED BY: TT **CHECKED BY:**

SHEET 2 **OF** 2 **HOLE NO.:** BK02-08

BECKER TEST HOLE LOG

STARTED: Mar 29, 2002 **FINISHED:** Mar 30, 2002

DRILL RIG MODEL: BECKER HAV180

GROUND ELEV. (m): 4.5

COORDINATES (m): N 468605.2 E 5364841.9

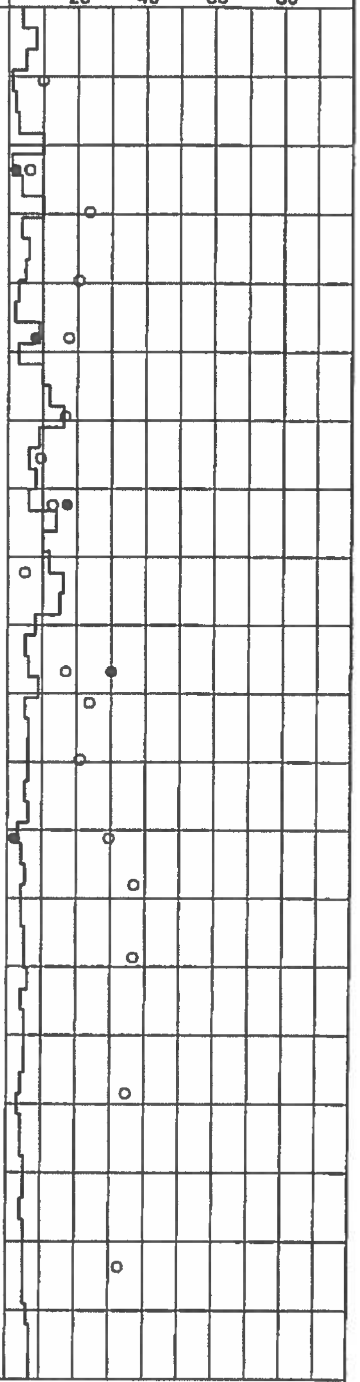
DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					0.1 4.4 ASPHALT (6 cm). GRAVEL (GW), fine to coarse, sandy, fine to coarse sand, some silt, angular to subangular, brown, moist, trace seashells (ROCKFILL).	
1		Grab	1			
2						
	6,1,1	SPT	2			
					2.6 1.9 SILT and CLAY (ML-CI), some fine to coarse sand and gravel, occasional cobble, soft to firm, brown, wet, trace organics and seashells (FILL).	
3		Grab	3			
4		Grab	4			
	5,3,5	SPT	5		- Water returned from cyclone at 4.57 m depth.	
5						
6		Grab	6			
					6.1 -1.6 SAND and GRAVEL (SW, GW), fine to coarse, some silt to silty, occasional cobble, angular to subrounded, brown to grey, wet (FILL/ROCKFILL).	
		Grab	7			
7						
	32,11,6	SPT	8			
8		Grab	9			
9						
	36,13,17	SPT	10			
10		Grab	11		9.9 -5.4 SAND (SP), fine, silty, gravelly, brown to grey, wet, trace seashells (FILL). - Lots of seashells at 10.36 m depth.	
11		Grab	12			
12	0,0,2	SPT	13			
					12.2 -7.7 CLAY (CI), medium plasticity, trace to some sand and gravel, soft to stiff, grey, wet, trace organics and seashells. - Sand and gravel content decreases with depth.	
		Grab	14			
13						
14		Grab	15			
15						
16		Grab	16			
17						
18		Grab	17		- Some medium to coarse sand between 18.1 m and 18.6 m depth.	
19						
20		Shelby	18			

Continued Next Page

BECKER PENETRATION TEST

Casing O.D.: 170 mm	CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated	OPEN ENDED (blows/0.3m)
● SPT N	BOUNCE PRESS (kPa)

$W_P\%$ × – – – ○ W% – – – × $W_L\%$



KLOHN CRIPPEN

PROJECT NO.: PG9060 01

PROJECT: PWGSC EGD and SLW Site Investigation

LOCATION: Esquimalt, BC

LOGGED BY: DT **CHECKED BY:**

SHEET 1 **OF** 2 **HOLE NO.:** BK02-10

BECKER TEST HOLE LOG

STARTED: Mar 29, 2002 **FINISHED:** Mar 30, 2002

DRILL RIG MODEL: BECKER HAV180

GROUND ELEV. (m): 4.5

COORDINATES (m): N 468605.2 E 5364841.9

BECKER PENETRATION TEST

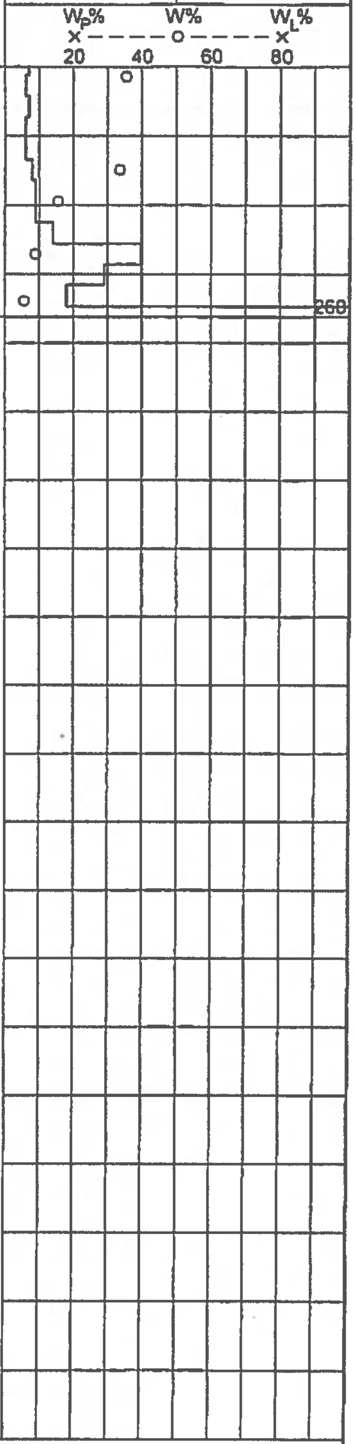
Casing O.D.: 170 mm	CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated	OPEN ENDED (blows/0.3m)
● SPT N	BOUNCE PRESS (kPa)

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
		Grab	19			
21		Grab	20		21.6 - Trace to some sand and gravel at 21.34 m depth.	
22		Grab	21		-17.2 SAND (SP), fine, silty, grey, wet.	
					22.2 -17.8 SAND and GRAVEL (SW, GW), fine to coarse, some silt to silty, occasional cobble, angular to subrounded, grey to black, wet.	
23		Grab	22			
		Grab	23			
24					23.6 -19.1 BEDROCK.	

End of Becker Drill Hole at 23.6 m

Notes:

1. Drilling was carried out by Foundex Explorations Ltd. of Surrey, B.C.
2. BPT02-10 was carried out approximately 2 m north of the open ended Becker hole, BOC02-10.
3. Standard penetration tests (SPT) were conducted using an automatic trip hammer (63.5 kg), AWJ rods and a split-spoon sampler (51 mm O.D., 38 mm I.D.).



21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40

KLOHN CRIPPEN

PROJECT NO.: PG9060 01

PROJECT: PWGSC EGD and SLW Site Investigation

LOCATION: Esquimalt, BC

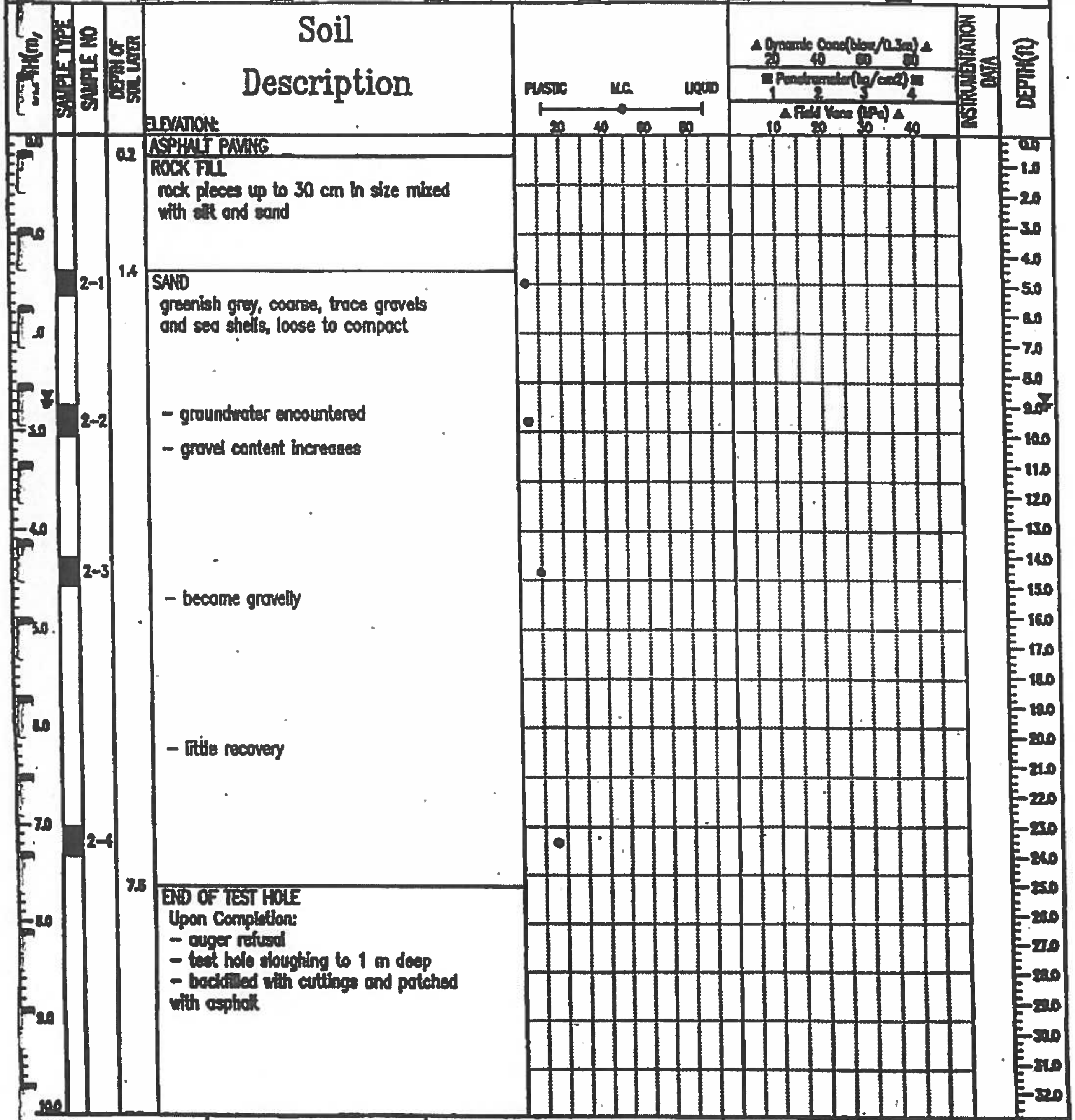
LOGGED BY: DT **CHECKED BY:**

SHEET 2 **OF** 2 **HOLE NO.:** BK02-10

LOG OF AUGER HOLE NO. TH-2

JOB NUMBER: 971-0155
CLIENT: Public Works & Government Services
SITE LOCATION: Esquimalt Graving Dock, Victoria

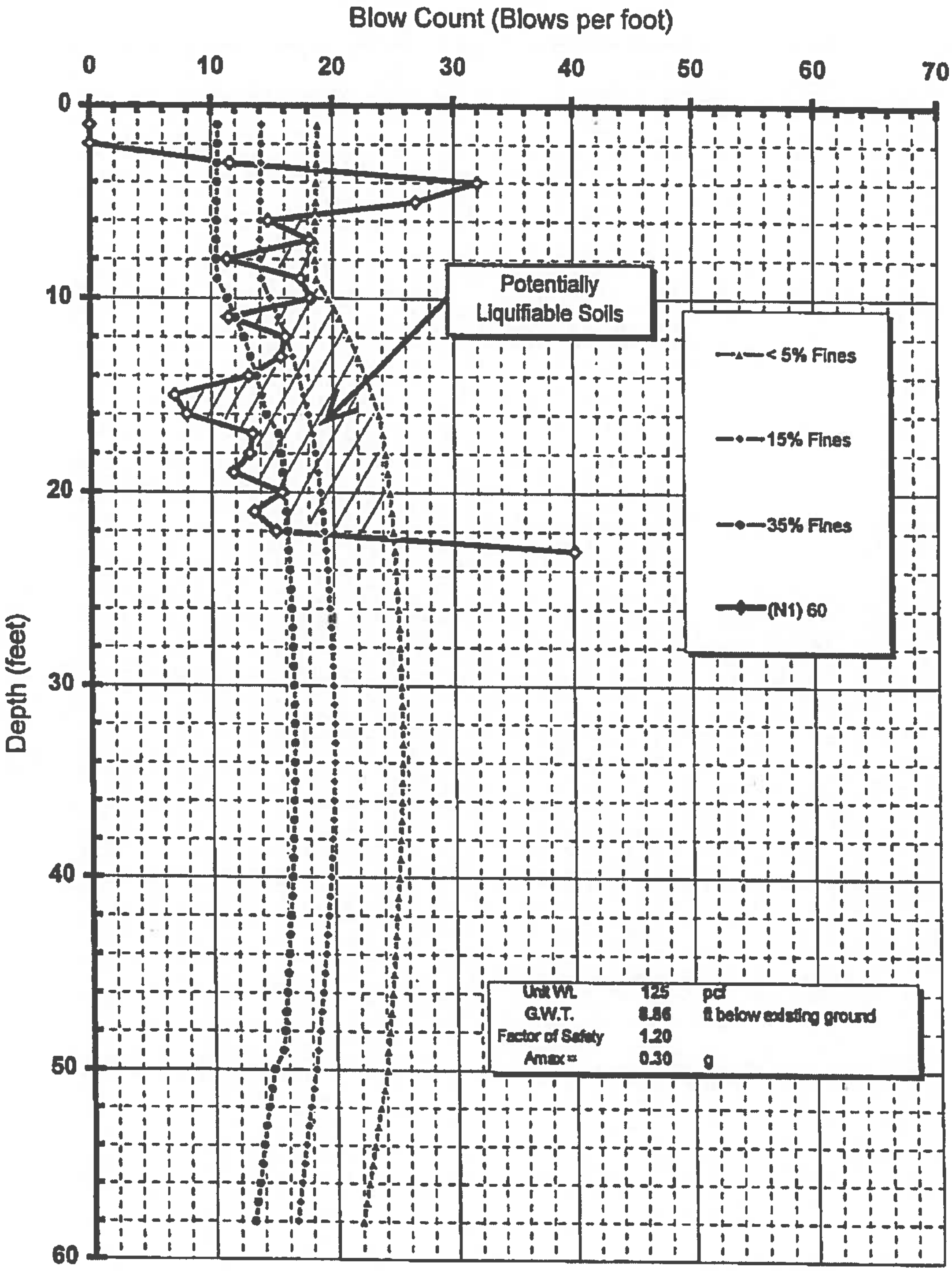
SAMPLE TYPE	■ GRAB SAMPLE	▨ Piston tube				
BACKFILL TYPE	■ BENTONITE	⁘ PEA GRAVEL	▥ SLOUGH	◬ GROUT	▨ DRILL CUTTINGS	▩ SAND



Depth of Soil Layer	Soil Description
	ELEVATION:
0.2	ASPHALT PAVING
1.4	ROCK FILL rock pieces up to 30 cm in size mixed with silt and sand
7.8	SAND greenish grey, coarse, trace gravels and sea shells, loose to compact – groundwater encountered – gravel content increases – become gravelly – little recovery
	END OF TEST HOLE Upon Completion: – auger refusal – test hole sloughing to 1 m deep – backfilled with cuttings and patched with asphalt

LOGGED BY: CZ	TRANSCRIBED BY: CZ	COMPLETION DATE: 03/05/97	EQUIPMENT: Drill Rig
Terra Engineering Ltd. (Vancouver, B.C.)		COORDINATES: N 0 0' 0" W 0 0' 0"	Page 1 of 1

Graph 1: Southside Substation Esquimalt Graving Dock
Dynamic Cone (CPT-1)



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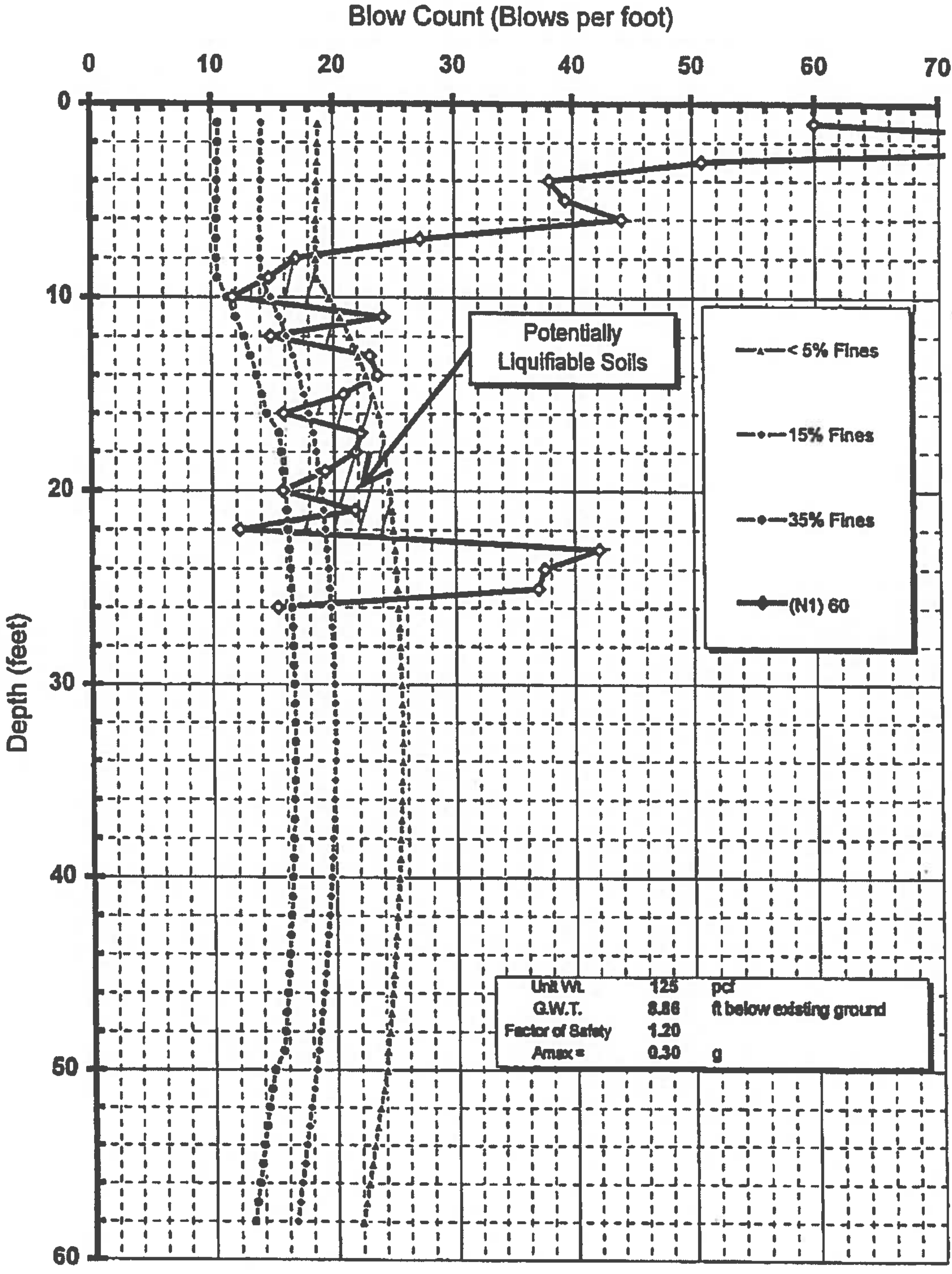
Graph 2: Southside Substation Esquimalt Graving Dock
Dynamic Cone (TH-2)



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