HOME

PUAR Engineering Consultants Inc

#200 – 100 Park Royal South Tower
W.Vancouver, BC, Canada V7T 1A2
Fax: 604-922-5054; Tel: 604-913-7827

Puar Engineering
CONSULTANTS

March 14, 2008

Project Number: 07-2-256

NICK EBRAHIM
3707 Dollarton Highway
North Vancouver, BC / V7G 1A1
Ph: 604-987-8313; Fax: N/A
e-mail: nebrahim2001@yahoo.ca

Attention: Mr Nick Ebrahim

**Re: GEOTECHNICAL INVESTIGATION REPORT
PROPOSED SUBDIVISION
3707 DOLLARTON HWY - NORTH VANCOUVER, BC
LOT 1, BLOCK K, DISTRICT LOT 230, PLAN 7990**

As requested, we have carried out a geotechnical investigation for the above site. This report summarizes the results of our field investigation and presents geotechnical recommendations for input to the design and construction of the proposed subdvision. Please refer to the attached Statement of Use of Study and Limitations (page 16) prior to reviewing the document.

We are pleased to be of assistance to you on this project and we trust that this information meets with your approval. Please feel free to contact us, if you have any questions or need further clarification.

PUAR ENGINEERING CONSULTANTS INC

Per:

**Surinder S. Puar, M.A.Sc., P.Eng.
Principal**

TABLE OF CONTENTS

PART A – FACTUAL INFORMATION

1.0 SITE DESCRIPTION & PROPOSED DEVELOPMENT

Puar Engineering Consultants Inc's (herein referred to as "PECI") information about the site and proposed development is based on:

- our ongoing phone and e-mail correspondence with the project team: Nick Ebrahim, Bob Heaslip, Webster Engineering (Civil Eng.), Bill Curtis Design (architectural design), Forma Design (landscape design), and PGL (Environmental),
- site observations during our recent visits and geotechnical field investigation,
- our experience in the vicinity of the site and data from the Geological Survey of Canada ("GSC"), and
- Webster Engineering's most recent conceptual site plan, which includes Hobbs Winter Macdonald's topographic survey information.

The subject site is located on the south side of Dollarton Highway. The lot, which is approximately 6900 m^2 (1.7 acres) in plan area, is currently occupied by a wood-frame, single-family residence. The subject property is bounded by an existing house to the west and by a recent 6-parcel subdivision development to the east. Burrard Inlet bounds the site to the south. Site topography slopes down toward the south and east at approximate angles of 10° to 25° and 0° to 20°, respectively. The foreshore of Burrard Inlet is herein referred to as the "subject slope". Sparse Coniferous and Deciduous tree growth was observed across the site. A small creek, which is located directly adjacent the west property line, traverses the southwest quadrant of the site

It is understood that the proposed development consists of subdivision of the property into seven parcels. The proposed homes are expected to include two-storeys (with basement). At this time, the proposed individual house floor elevations have not been estimated. It is expected that the development will include a westward continuation of the 3 m (10') wide foreshore walkway that was included as part of the neighbouring 6-lot subdivision.

It is understood that a District of North Vancouver Restrictive Covenant (for sloping sites per Master Requirement SPE105) is applicable to the site. The proposed development may require approvals from the Vancouver Port Authority (VPA), Burrard Environmental Review Committee (BERC), and the federal Department of Fisheries & Oceans (DFO).

2.0 SUBSURFACE CONDITIONS

2.1. Site Geology

Based on our experience in the vicinity of the site and information available from the Geological Survey of Canada (GSC), near-surface, native soils in the vicinity of the site were expected to consist of sand to cobble gravel (Capilano Sediments) overlying sand to silt, till-like (Vashon Drift) deposits.

2.2. Field Investigation and Observed Subsurface Conditions

Field Investigation

PECI carried out an investigation consisting of three test-pits that were put down using a large tracked excavator at the locations shown on Figure A-1. An occupied house is present on the subject property. The number of test-pits were limited due to the presence of active utilities, root structures (trees), landscaping amenities that could not be disturbed or damaged, and the existing house (southeast quadrant).

Test-pits extended to a maximum depth of 3.7 m below the existing ground surface in order to help characterize the subsurface soils for input to our geotechnical review. All augerholes were backfilled with spoil from the test-pits. A member of our engineering staff logged the test-holes.

Subsurface Conditions

The observed soil conditions were in conformance with the above-described information from the GSC. Soil layer densities are inferred based on the observed resistance to penetration by the excavator.

In general, a thin layer of topsoil (organics and silty sand) ("UNIT 1") was encountered overlying the native soil stratigraphy, which generally consisted of loose sand ("UNIT 2") overlying a till-like sand deposit ("UNIT 3").

Table 1 – Generalized Soil Stratigraphy

Depth Range	Layer Thickness (m)	UNIT No.	Soil Description
0 – 0.5 m	0.2 to 0.5 m	1	Loose, moist, brown TOPSOIL (Organics and Silty Sand).
0.2 – 3.4 m	0.8 to 2.0 m	2	Loose, moist, light brown to red-brown, fine to medium SAND, trace to some silt and organics, trace gravel.
1.5 – 3.7 m	To End of Test-pit	3	Very Dense light brown to grey, moist sand to silty sand (till-like) containing trace gravel and clay with undefined cobble and boulder content.

Topsoil (UNIT 1)

In general, a thin layer of loose silty sand and organics (topsoil) was encountered overlying the native soil stratigraphy across the site. This surface layer ranged in thickness from 0.2 m (8") at test-pit TP08-2 to 0.5 m (1.5') at TP08-3.

Sand (UNIT 2)

All of the test-pits encountered the above-described loose sand Native soils underlying the upper organic and fill materials primarily consisted of a comparatively thin layer of loose, light brown, moist, mottled sandy silt to red-brown silty sand. This layer ranged in thickness from 0.2 m (8") to 0.8 m (2.4').

Till-like Sand to Silty Sand (UNIT 3)

All of the test-pits were terminated in the above-described till-like sand to silty sand deposit. Test-pits TP08-1 and -2 were terminated in this deposit at depths of 1.5 m to 1.8 m, respectively. Test-pit TP08-3 was terminated at a depth of 3.4 m.

Surface and Subsurface Water

With the exception of the creek flows, no surface or subsurface water was observed upslope or east of the creek during our investigation. No water was observed within the test-pits during excavation. Different subsurface conditions may be encountered at the time of construction. It should be noted that the impacts of water vary seasonally and in response to rainfall and snowmelt events, and/or special meteorological conditions. Groundwater conditions may also be altered by construction activity on or in the vicinity of the project site. Subsurface water conditions in the area may also be influenced by the nature of the upslope surface water drainage systems.

PART B – GEOTECHNICAL RECOMMENDATIONS

1.0 GENERAL

In general, based on the results of the geotechnical field investigation for the proposed development, the site is judged to be safe for the intended use, assuming the following recommendations are implemented.

The UNIT 2 sands are judged to be unsuitable for house foundation support. However, this sand layer may be useful, to a limited extent, for source control of stormwater flows. Limited application of stormwater source controls (eg. infiltration trenches, absorbent landscape, etc) is anticipated. However, PECI's input would be required when determining the geotechnical feasibility of infiltration features that accommodate high flow sources such as house roofs. Stability considerations may govern such applications.

Subsurface conditions will require verification during PECI's construction field reviews, as an investigation of limited depth (ie. test-holes reached a maximum depth of 1.5 m) was carried out due to limited access for testing equipment. It should be noted that without test-pits that extend beyond the depth of the proposed structure's foundations, extrapolation is required to estimate soil parameters. Our design information is based on the test-pit information, our experience and local geological data. At the time of construction, delays (adjustments in structural and geotechnical design, etc.) may result, if actual subsurface conditions are not found to be in accordance with the assumptions applied in the preliminary design parameters below. In such a case, it is suggested that contingencies be in place to accommodate potential construction anomalies.

Based on the results of our slope stability review, the assumed setback of the proposed residence foundations – in particular, Lots 4 through 7 -- from the slope facade is judged to be sufficient, provided that proper drainage practices are implemented and all foundations are placed on the till-like sand. Furthermore, to maintain stability suitable protection against surficial erosion of the slope should be incorporated in the design. Due to the steepness of the site's southern extremity, it is suggested that consideration be given to construction of erosion control facilities that protect against weathering and wave erosion across the lower foreshore.

Large boulders and/or bedrock were not encountered within the test-pits. However, contingencies should be in place for splitting and/or blasting if such materials are encountered during excavation.

2.0 SITE PREPARATION

It is recommended that all deleterious soils (eg. material containing organics) and all loosened/ softened soils underlying any settlement-sensitive structures be removed to expose the native, undisturbed, very dense till-like sand. Based on the information gathered at our test-pit locations, it is estimated that the upper approximately 1.5 m to 3.4 m or more of the soil profile contains such unsuitable materials.

We envision that the excavations for the residences may extend below the local groundwater level at the time of construction. Accordingly, an allowance for sump-pumping of excavation water should be made. Vacuum dewatering is not recommended to control groundwater discharge. PECI should be contacted to confirm that the rate of pumping will not be detrimental to (ie. induce settlement of) surrounding structures and that subsurface material is not being 'washed' out (ie. eroded) from upslope areas within the seepage flows. Collected water should be discharged off-site in a manner approved by the District of North Vancouver.

The native, till-like silty subgrade is expected to be susceptible to disturbance by rainfall, runoff, seepage and/or construction traffic. Foundation subgrade areas should be protected from freezing. Groundwater and rainwater runoff should be directed to temporary sumps and subgrade areas should be kept free of standing water.

If construction is to commence in traditionally wet months, then stripping should be completed in stages. It is suggested that subgrade areas be protected by a 'working mat' consisting of a 100-mm (4") (maximum) thick layer of 12 mm (1/2") clear crushed gravel.

We envision that the material encountered in the test-holes may be excavated using conventional excavation equipment. Very dense till-like material and large boulders or bedrock may be encountered. It is suggested that a provision for splitting of boulders and/ or blasting be considered in the construction budgeting.

Consideration may be given to stockpiling and re-use of select excavated materials for landscaping or general fill in areas where significant post-construction settlements are acceptable and where drainage is not a consideration.

3.0 TEMPORARY EXCAVATION STABILITY

Unshored Excavation Slopes

Above the native, till-like soils, the upper topsoil and loose to compact, native deposits may become unstable, if they are cut steeper than 1 vertical to 1 horizontal (1V:1H). This could potentially result in undermining of any facilities adjacent the crest of the respective cut. The presence of groundwater seepage may result in the need for more shallow slopes (eg. 3V:4H). As a general guide, unshored excavation cut-slopes less than 2.5 m high should be no steeper than 1V:1H.

When determining whether excavation shoring will be required, some of the key considerations when developing preliminary estimates of excavation slope geometries include:

- the existing site grade and neighbouring grades,

- the founding depth of neighbouring structures (including utilities) and landscaping,
- the required work-space offset (typically 0.6 m) between the edge of proposed foundations and the base of the excavation,
- the width of proposed footings and the thickness of the proposed foundation walls, and
- the presence of large coniferous trees (and root structure).

Temporary excavation shoring measures may be required, if significant sloughing of excavation slopes occurs and/or slopes steeper than 1V:1H through the upper soils are required.

Other Excavation Preparation Considerations

Cobble-sized (75-mm diameter, or larger) material should be removed from the facade of all excavation cut-slopes. Grade adjacent the excavation should be sloped to direct surface runoff away from the excavation slopes. As required, a swale (cut-off trench) should be constructed back of the crest of all slopes to minimize flow of surface water along the face of the slope.

If the excavation walls are subject to seepage flows, they may require more shallow grading. If significant seepage is observed during construction, PECI should be notified immediately to confirm the suitability of the above excavation preparation recommendations. Unshored excavation slopes should be protected by a layer of 6-mil polyethylene sheeting securely tied to resist wind action. The stockpiling or storage of excavation spoil, construction materials, or heavy equipment should not be permitted within 5 m (16') of the crest of excavation slopes.

4.0 SLOPE STABILITY CONSIDERATIONS

Field Investigation

PECI performed an assessment of the surficial stability of the slope that was based on visual observations, limited test-pit data, nearby drill-hole data, as well as available geological data.

Our site reconnaissance included observation of soil exposures, surface and subsurface water, and signs of slope 'creep' to aid our understanding of the geology of the slope. During our site reconnaissance, no signs of imminent failure (eg. tension cracks in the ground) were observed in the vicinity of the proposed development. No signs of historic, large-scale, deep-seated failures were observed. Additionally, no indications of surficial instability or colluvium (loose, gravity deposited material from upslope areas) were observed in or directly adjacent to the footprint area of the proposed development.

Site topography slopes down toward the south and east at approximate angles of 10° to 25° and 0° to 20°, respectively. Vegetation across the subject slope consists mostly of a low-density growth of coniferous and deciduous trees with generally light to moderate undergrowth. No seepage was observed across the façade of the slope during our recent site visits.

Some coniferous trees with curved bases and leaning deciduous trees were observed on the lower reaches of the slope. In particular, the number of such trees was proportionally higher across the lower approximate 10 m of the slope, where topography is generally steeper. The majority of trees across the site were observed to be vertical. A limited number of the trees had curved bases ("pistol butt") and/or were not vertical across the lower foreshore (ie. south of the existing house), where topography is generally steeper (about 30 to 45 degrees, typical). No seepage was observered across the foreshore.

Slope Stability Analysis

The static and dynamic (seismic) stability of the slope -- as it pertains to a slope failure that could significantly impact the proposed development -- was assessed, in terms of limit equilibrium stability. The native, very dense, till-like sand was assumed to be present across the respective footprints of the proposed residences.

Recommendations and Conclusions

Based on the above observations, the assumed/ estimated setback of the proposed residence foundations – in particular, Lots 4 through 7 -- from the slope facade is judged to be sufficient, provided that proper drainage practices are implemented and all foundations are placed on the till-like sand. Furthermore, to maintain stability suitable protection against surficial erosion of the slope should be incorporated in the design. Due to the steepness of the site's southern extremity, it is suggested that consideration be given to construction of erosion control facilities that protect against weathering and wave erosion across the lower foreshore.

Based on the results of our analyses, the estimated factors of safety for the static and dynamic load conditions were judged to be acceptable, provided our recommendations (noted above and below) are implemented.

In the future, the potential exists for localized, surficial soil instability, if basic measures are not maintained and adopted to mitigate development-related changes in the environment. To retard and reduce the size and scale of these potential shallow failures in the surficial soils, we recommend that the following measures be incorporated for the areas downslope of the building footprints:

- No concentrated run-off or other discharge shall be directed toward the slope.
- no discharge of water on the site shall occur north the residences.
- no stockpiling or dumping of materials on the sloping areas,
- no unauthorized removal, excessive thinning, or other alteration of the existing vegetation, unless it poses a fall-hazard or some other significant risk,
- absorbent vegetation should be planted across zones of exposed soil.

Energy dissipating systems should be employed at the discharge point of all drainage outlets.

5.0 PRELIMINARY FOOTING DESIGN

5.1 General

Subsurface conditions will require confirmation during construction to ensure that conditions in the field are in accordance with our current recommendations.

Based on the above observations and our past experience with local soil conditions, it is envisioned that basement excavations would result in removal of most of the soils overlying the till-like sand across the northern extent of foundations. However, due to sloping grades, overexcavation into the till-like soils is expected to be required.

5.2 Static Bearing Pressure

The above-described native, very dense, till-like sand is considered suitable for support of shallow foundations consisting of strip and pad footings. A preliminary maximum, factored (ie. 0.5) bearing resistance/ capacity of 200 kPa (4200 psf) at Ultimate Limit State (ULS) would be applicable for the subject till-like sand, subject to field confirmation by PECI.

A factored bearing resistance/ capacity of 150 kPa would apply for the Serviceability Limit State (SLS). As input to the SLS, it is envisioned that post construction settlements (due to static loading) would be limited to less than 25 mm (1"). It is envisioned that differential settlement would be less than 19 mm over a span of 9 metres.

5.3 Recommended Footing Characteristics

Minimum pad footing dimensions of 0.6 m (2.0') and minimum strip footing widths of 0.45 m (1.5') are recommended. It is recommended that foundations be placed a minimum of 0.45 m (18") below the adjacent final exterior grade for frost protection. Foundations within the undisturbed, native, till-like sand should step no steeper than 2.0 Horizontal to 1.0 Vertical (2H:1V).

5.4 Seismic Considerations

The above allowable, static, design bearing pressures may be increased by 50% for short-term transient loading conditions such as those induced by wind and earthquakes. In accordance with the 2006 edition of the BC Building Code and the above observations, it is judged that Site Class "C" would be representative of the observed ground conditions.

6.0 SLAB-ON-GRADE

Generally, it is recommended that a minimum 150 mm (6") thick drainage layer of compacted 19-mm (3/4") clear crushed gravel be placed beneath all slabs-on-grade. This drainage layer should be separated from the slab-on-grade by a layer of 6-mil polyethylene sheeting (per CAN/CGSB-51.34-M86).

It should be noted that, if any slab-on-grade (eg. garage) is set higher than adjacent below-grade parts of the proposed structure, then structural design of below-grade walls and/or slabs would be required to accommodate lateral loads associated with the upper slab loads.

7.0 PRELIMINARY BASEMENT & LANDSCAPE RETAINING WALL DESIGN

7.1 General

The lateral earth pressure on basement and retaining walls depends upon a number of factors, including the backfill material, surcharge loads, backfill slope, drainage, rigidity of the wall, and method of construction including sequence and degree of compaction.

The lateral pressure estimates below do not include hydrostatic components, as it is envisioned that all retaining (including basement) walls will be suitably drained. If it is not possible to provide continuous drainage behind the walls, hydrostatic pressure must be assumed to act over the depth of the walls up to a level corresponding to locally applicable stormwater and/or groundwater design events; the hydrostatic pressure would be added to the static design lateral earth pressure. It should be noted that "continuous drainage" would be expected to include not only a drainage main, but also a suitable width of free-draining backfill.

The lateral earth pressure estimates provided below assume that the area behind the wall is horizontal and no adjacent structures or surcharges are situated within a horizontal offset from the base of the wall corresponding to a line projected at 3 Vertical to 2 Horizontal (3V:2H) from the base of the wall. Furthermore, the following design parameters are based on the assumption that all applicable walls will be backfilled with clean, granular, free-draining material such as Engineered Fill (described in a later section).

As noted above, landscape retaining walls are suggested to protect the southern slope against surficial erosion. These walls should be constructed by cutting into the existing slope, as opposed to increasing the width of the terraced areas (ie. the overall slope should not become steeper due to the construction of retaining walls).

7.2 Static Design

7.2.1. Unrestrained Condition

For walls that can displace laterally an amount equivalent to 0.2% (min.) of the wall height, the condition is considered to be 'unrestrained'. For the unrestrained condition, we recommend that the wall be designed on the basis of a 6.4 x h (kPa) triangular earth pressure distribution where 'h' is the distance from the ground surface measured in metres. In imperial units this corresponds to 40 x h (psf), where 'h' is measured in feet.

7.2.2. Restrained Condition

If a 'restrained' condition is present (eg. some basement walls) then we recommend that the wall be designed on the basis of a 9.6·h (kPa) triangular earth pressure distribution where 'h' is the distance from the ground surface measured in metres. In imperial units this corresponds to 60·h (psf), where 'h' is measured in feet.

7.2.3. Compaction-Induced Pressure

If the backfill is to support settlement-sensitive structures, it will require compaction. For this condition, a compaction-induced, uniformly-distributed, lateral earth pressure of 20 kPa can be used in the uppermost approximate 3 m. In imperial units this corresponds to a uniformly-distributed, compaction-induced earth pressure of 400 psf in the top approximate 10 ft.

7.2.4. Base Friction

It is envisioned that sliding resistance for footings would be derived from the native, till-like sand to silty sand subgrade. A friction factor of 0.5 may be applied between the concrete and sand subgrade interface.

7.3 Seismic Design

Seismic loading conditions can be assumed to represent an additional triangular pressure at the top of the wall that decreases to zero at the base of the wall. The seismic surcharge pressure can be assumed to be 3.2·(H-h) kPa, where 'h' is the distance from the top of the wall and 'H' is the total wall height in metres. In imperial units this corresponds to 20·(H-h) (psf), where the measurements are in feet.

The seismic loading is added to the static loading, but the compaction-induced loading represents a superimposed loading condition. Consequently, the maximum lateral earth pressure at any point over the depth of the wall would be the **greater of**:

1.) ***For the Unrestrained Condition:***

- 3.2·(H+h) (kPa) [i.e., the sum of 6.4·h (static) and 3.2·(H-h) (dynamic)], and
- 20 kPa (ie. compaction-induced pressure).

In imperial units, this amounts to:

- 20·(H+h) (psf) [i.e., the sum of 40·h (static) and 20·(H-h) (dynamic)], and
- 400 psf (ie. compaction-induced pressure).

and,

2.) ***For the Restrained Condition:***

- 3.2·H + 6.3·h (kPa) [i.e., the sum of 9.5·h (static) and 3.2·(H-h) (dynamic)], and
- 20 kPa (ie. compaction-induced pressure).

In imperial units, this amounts to:

- 20·H + 40·h (psf) [i.e., the sum of 60·h (static) and 20·(H-h) (dynamic)], and
- 400 psf (ie. compaction-induced pressure).

7.4 Additional Loading - Vehicle Surcharges, etc

PECI should be contacted to confirm changes to the above estimates if any dead and/or live loads will be present adjacent a particular retaining wall. Any loads associated with lateral earth pressure induced by surcharges present across the top of the backfill (such as vehicle loading at paved locations adjacent the subject walls) should be added to the above earth pressure estimates as instructed by PECI.

8.0 FILL MATERIALS

8.1 Engineered Fill

Engineered Fill should consist of select, clean, well-graded granular material with less than 5% fines content and 100% passing a 75 mm (3") sieve. Engineered Fill should be placed in suitable lifts (generally 0.3 m loose thickness, or less) and compacted to the equivalent of 100% (or greater) of its Standard Proctor maximum dry density (per ASTM D698). Field density testing should be carried out on each lift of Engineered Fill placed. Engineered Fill should extend beyond the outer edges of footings a minimum horizontal distance equal to the width of the foundation, plus the thickness of the Engineered Fill.

The clean, native, loose sands (UNIT 2) are envisioned to be suitable for re-use as Engineered Fill in limited areas of the site, as instructed by PECI (on site).

In accordance with the issued Letters of Assurance, field reviews must be conducted by PECI to confirm that fill selection and placement procedures are satisfactory and density test results are representative. In all instances, the engineer responsible for the long-term performance of any settlement-sensitive structure supported on Engineered Fill should be given the opportunity to review the material composition and the achieved level of compaction of each lift. Where a testing agency is retained for additional density confirmation, test results should be forwarded to PECI for review.

8.2 Backfill

In order to minimize lateral earth pressure against below-grade walls, backfill placed against foundation and landscape retaining walls should consist of Engineered Fill as specified in this report. It should be noted that backfill can be expected to experience post-construction settlement of the order of 1% of the total fill height. Therefore, consideration should be given to the design of paved areas and/ or landscaping that spans the outer edge of the backfill.

9.0 PRIVATE PAVED AREAS

It should be noted that if pavement is to be constructed at the site, the following recommendations do not supercede or replace local municipal requirements. The recommendations below apply only to those areas that are not subject to municipal bylaws.

Upon removing all fill, topsoil, loosened, disturbed or otherwise deleterious material from beneath the new pavement footprint areas, we recommend placing the pavement section summarized in Table 2 (below).

Table 2 – Recommended Minimum Asphalt Pavement Structure

MATERIAL	THICKNESS (mm)
Asphaltic Concrete	85
19-mm minus crushed sand and gravel Base course	100
Engineered Fill (as described above) Sub-base course	As Required for Grade Restoration Fill

The prepared subgrade should be adequately crowned and graded to assist in draining of overlying fills. Provision of subsurface drains on the upslope side of proposed paved areas should be considered. The stripped subgrade should be reviewed by PECI prior to placement of base layers to confirm the removal of unsuitable material. Laboratory Gradation and Standard or Modified Proctor tests should be carried out on the base and sub-base course materials.

10.0 REVIEW

Subsurface conditions will require verification during PECI's construction field reviews, as an investigation of limited depth was carried out due to site constraints. Without test-holes that extend beyond the depth of a proposed structure's foundations, extrapolation is required to estimate soil parameters. Our design information is based on test-pit information, our local experience and local geological data only.

In accordance with the District of North Vancouver's Letters of Assurance, PECI is obligated to carry out field reviews for the following items:

- temporary excavation stability (WCB clearance for excavation entry),
- foundation and retaining wall subgrades,
- stability of permanent slopes,
- floor slab subgrades, prior to and after placement of drainage layer,
- Engineered Fill selection and placement, and
- pavement subgrades prior to and after placement of sub-base and base layers.

In the absence of field review of Engineered Fill placement, PECI should review the field density test results.

11.0 CLOSURE

This report has been prepared for the sole use of Nick Ebrahim and other consultants and contractors for the subject development. Any use or reproduction of this report for other than the stated intended purpose is prohibited without the written permission of Puar Engineering Consultants Inc.

We are pleased to be of assistance to you on this project and we trust that this information meets with your approval. Please feel free to contact us, if you have any questions or need further clarification.

PUAR ENGINEERING CONSULTANTS INC
Per:

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
S. PUAR

Surinder Puar, P.Eng.

Attachment: Figure A-1

cc: 1.) Bob Heaslip
2.) Webster Engineering
3.) Forma Design
4.) Bill Curtis Design

Distribution: fax X ; mail __ ; courier__ ; pick-up X ;

STATEMENT OF USE OF STUDY AND LIMITATIONS

I. STANDARD OF CARE

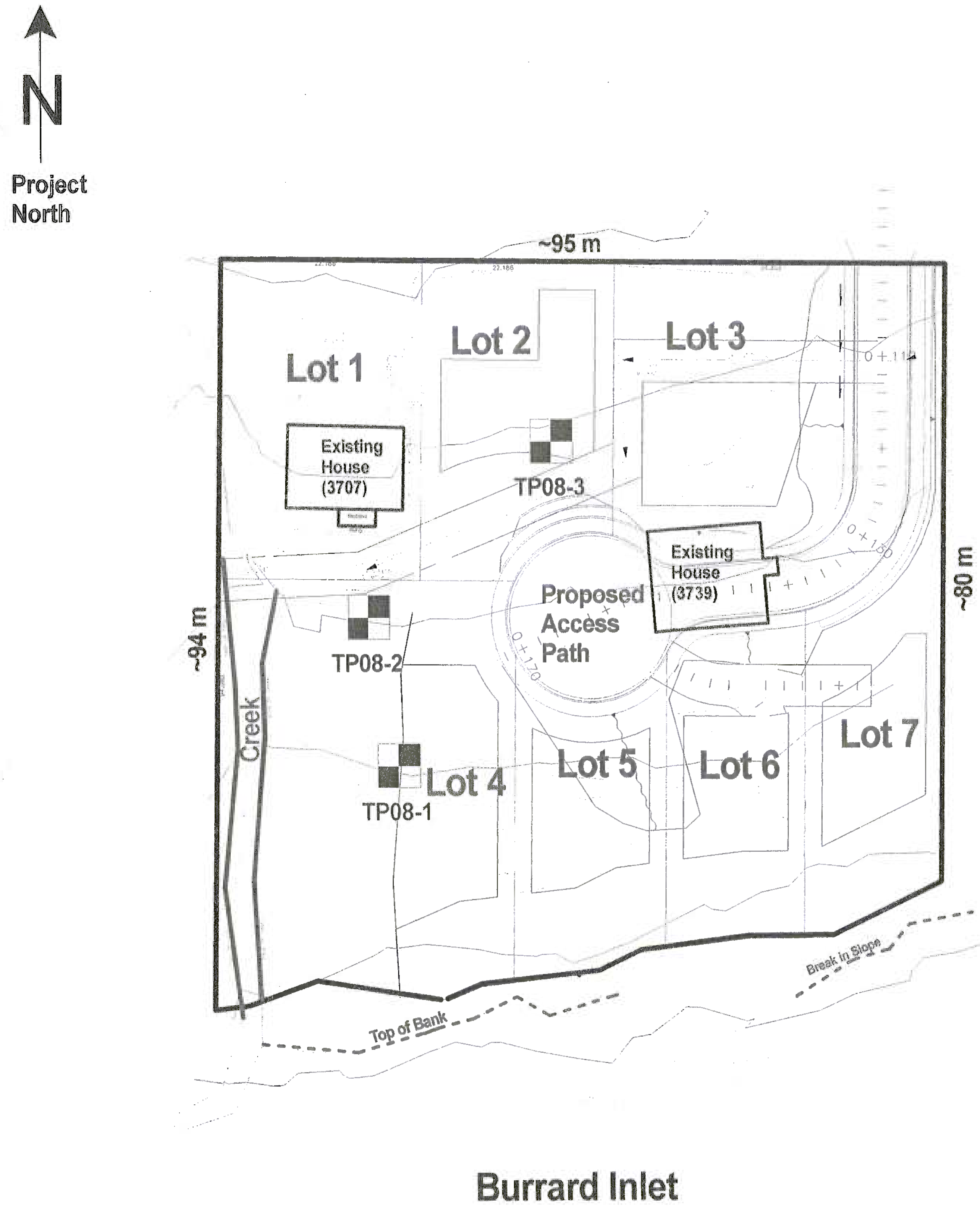
Puar Engineering Consultants Inc (herein referred to as "PECI") has prepared this study and report in accordance with generally accepted engineering consulting practices in this area, subject to applicable time limits and any physical constraints in preparation of this study. No other warranty, expressed or implied, is made.

II. USE OF REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this study are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client and to any other reports, writings, proposals or documents prepared by us for the Client for the subject development, all of which constitute the Report. All opinions, information, and/or recommendations expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without PECI's express written consent. PECI will consent to any reasonable request by the Client to approve the use of this report by other parties as Approved Users. The report, all plans, data, drawings and other documents as well as all electronic media prepared by PECI are considered its professional work product and shall remain the copyright property of PECI, who authorizes only the Client and Approved Users to make copies of the report -- only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of PECI. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

III. INTERPRETATION OF THE REPORT

Any change of site conditions, development plans, or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Classification and identification of soils, rocks, geological units and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to identify some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of and accept this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling.



LEGEND

Test-pit

Drawing Not to Scale

Reference Drawing: Webster Engineering - Civil Site Plan (dated 08Feb15)	CLIENT: Nick Ebrahim			TITLE: Test-hole Plan			
Puar Engineering CONSULTANTS	PROJECT: Proposed Subdivision 3707 Dollarton Hwy - North Vancouver, BC						
	PROJECT NO. 07-2-256	DFTR. wsp	DSGN. sp	DATE 08Mar	SCALE: NTS	DWG NO. A-1	

HOME