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

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

As requested, we have performed a review of the project to confirm that our original Geotechnical Investigation Report (dated October 2, 2008) is applicable as originally intended.

This report summarizes the results of our updated field investigation and analysis information. Estimated post-construction site geometries have been considered in our follow-up stability analyses. Geotechnical recommendations for input to the design and construction of the proposed subdivision are presented. The District of North Vancouver Restrictive Covenant (for sloping sites per Master Requirement SPE105) is addressed, also. Please refer to the attached Statement of Use of Study and Limitations (page 18) 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:

PROFESSIONAL
PROVINCE OF
S.S. PUAR
BRITISH COLUMBIA
ENGINEER

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:

- From 2007 through 2009, our phone and e-mail correspondence with the project team: Nick Ebrahim (Client), Bob Heaslip (planning), Webster Engineering (Civil Eng.), Bill Curtis Design (architectural design), Forma Design (landscape design), and PGL (environmental),
- site observations during our most recent visits (in 2011),
- our 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 and site sections in digital format (dated September 11, 2008)
- Hobbs Winter Macdonald's topographic survey information, which is presented in Webster Engineering's submissions.

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 two single-family residences. 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 west/ southwest extremity 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, preliminary basement floor elevations of 10 m and 24 m (Geodetic Datum) have been estimated, based on our correspondence with Bill Curtis Design. 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. The proposed development would involve 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. Surficial Geology

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

2.2. Field Investigation and Observed Subsurface Conditions

Field Investigation

PECI carried out multiple site investigations. The first investigation consisted of three test-pits (TP08-1 to -3) that were put down using a large tracked excavator at the locations shown on Figure A-1B. Two hand-dug test-pits (TP08-4 and -5) were put down during a follow-up investigation; TP08-4 was put down at the north extremity where utilities are present and TP08-5 was put down in an area that was considered inaccessible to the excavator. An occupied house (3715 Dollarton Hwy) is present near the central segment of the southern half of the property. The number of test-pits put down using the excavator were limited primarily due to the presence of active utilities, root structures (trees), landscaping amenities that could not be disturbed or damaged, and the above-noted existing house (southeast quadrant). These investigations were augmented by a follow-up investigation consisting of two hand-dug test-pits. Additional information was collected during a site reconnaissance consisting of visual observations of the slope. Our investigation of the subject property was augmented by auger-drill and test-pit data from the neighbouring subdivision to the east, where 9 solid-stem auger-holes, 2 Dynamic Cone Penetrometer Tests (DCPT), and 5 hand-dug test-pits were put down.

Test-pits on the subject property 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 test-holes were backfilled with spoil from the test-pits.

Subsurface Conditions

The observed soil conditions were in conformance with the above-described information from the GSC and our subsurface data from the neighbour property. Detailed descriptions of the soil encountered at the test-holes are provided in the attached test-hole logs. Soil layer densities are inferred based on the observed resistance to penetration by the excavator. Soil observations were correlated with the data from the neighbouring site to confirm densities (ie. based on previous blow-count magnitudes).

In general, stratigraphy across the two properties is very consistent. 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 sand deposit ("UNIT 3").

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 at test-pit TP08-2 to 0.5 m at TP08-3.

Sand (UNIT 2)
All of the test-pits encountered the above-described loose to compact sand underlying the capping organic layer. This deposit was typically less than about 1.5 m thick and consisted of light brown to red-brown sand to silty sand containing variable silt and organic content. At TP08-3, this deposit transitioned to a fine sand that had no perceptible organic content. The upper loose zone containing trace to some organics ranged in thickness from 0.8 m (at TP08-1) to 1.3 m (at TP08-2). The non-organic segment was encountered at TP08-3 at a depth of 1.4 m, where it extended to a depth of 3.4 m.

Sand to Silty Sand Till (UNIT 3)
All of the test-pits were terminated in the above-described sand to silty sand till 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 3.7 m. Test-pits TP08-4 and -5 were terminated at depths of about 1 m.

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, brown, moist Topsoil (Organics and Silty Sand).
0.2 – 3.4 m	0.8 to 2.9 m	2	Loose to compact, light brown to red-brown, moist, fine to medium SAND, some silt to silty, trace to some 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.

Surface and Subsurface Water
With the exception of the creek flow, 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. 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

The currently proposed development and site conditions are generally in conformance with the conditions under which we prepared our original report (dated October 8, 2008).

In general, based on the results of our geotechnical field investigation, the site is judged to be safe for the intended use based on the currently proposed development parameters described above and our stability analyses (defined below). This conclusion is also contingent upon the implementation of the following recommendations.

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. 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.

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 crest/ facade is judged to be sufficient, provided that proper drainage practices are implemented and all foundations are placed on the sand till.

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 sand (till). Based on the information gathered at our test-pit locations, it is estimated that the upper approximately 1 m to 3 m or more of the soil profile contains such unsuitable materials.

The native, silty (till) 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. 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 deposit, the upper topsoil and loose to compact, overburden sand 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 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.

The north cutslopes for Lots 4 to 7 may require consideration of excavation shoring. 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 (overburden) 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, the Geotechnical Engineer-of-Record ("G.E.R") should be notified immediately to confirm suitable 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, test-pit data, nearby drill-hole data, as well as available geological data. Our site reconnaissances 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 reconnaissances, 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 the footprint of any of the proposed lots and associated amenities.

It is envisioned that topography across the majority of the property is at or near pre-development grades. 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.

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 #3715), where topography is generally steeper (about 30°, typical).

Slope Stability Analysis, Discussion, and Results

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 (Bishop's Method) using the computer code ReSSA. Soil layers were extrapolated based on information gathered at the test-pits and our experience in the vicinity. The native, very dense, sand (till) was assumed to be present across the respective footprints of the proposed residences. Topographic survey data was used to develop the slope geometry for input to the stability model. Webster Engineering's site profiles (September 11, 2008) indicated a worst-case topographic profile sloping at about 20° (avg.). It is envisioned that the thickness of the comparatively weak Beach Sand is conservatively modelled (ie. greater than the actual thickness). Two sets of analyses corresponding to pre- and post-construction site geometries are addressed below. Post-construction geometry accounts for the estimated basement excavations at the upper and lower lots; structural loading is represented as a uniform 20 kPa pressure across the excavated segment.

Strength Parameters

Based on the above, the till, is estimated to be "very dense" with corresponding SPT blow-counts in excess of 50 per foot and a seismic shearwave velocity in excess of 1000 m/s. For our base-case analyses, generally conservative strength parameters were applied to the till ($\Phi = 37°$; c =

10 kPa), as well as the beach sand and overburden ($\Phi = 30°$; c = 0) materials. Unit weights of 19 kN/m^3 (Overburden and Beach Sand) and 20 kN/m^3 (Till) were applied.

Pore-water Pressures

No indications of seepage were observed in the test-pits. No seepage discharge was observed at any point across the sloping property. Our experience with excavations in the vicinity indicate that a low proportion of the silty till contains water-bearing lenses. Based on our experience with the till deposit, Storage Capacity is also estimated to be very low (ie. less than 0.01). Furthermore, it is envisioned that the low-permeability till would limit the hydraulic connection between surface water and deeper segments of the slope profile.

In the development of our slope geometries, the comparatively weak yet permeable overburden material is (conservatively) assumed to be uniformly 3 m thick across the site. (Actual overburden thickness is expected to average less than 1.5 m across the site). In our analyses, we have conservatively assessed a piezometric surface simulating perched water (interflow) on the surface of the till. However, it is envisioned that drainage systems at existing and future access paths, landscape retaining walls, and house perimeter drains would cut-off any such perched flows.

Seismic Loading

Current state-of-the-practice for the analysis of slopes under seismic loading is to utilize a pseudo-static procedure that simulates a mass of soil subject to an horizontal inertial force. The British Columbia Building Code (2006) estimates a peak ground acceleration of 0.44g with a 2475-year return period for the North Vancouver area. As described below, we have estimated a Site Class "C" for the subject property. As such, firm ground motions have not been subject to modification (attenuation or amplification). For comparison purposes, we have also analyzed the slope with a reduced pseudo-static coefficient equal to one-half the applicable maximum (0.44g), as suggested by Kramer [Kramer, S.L. (1995). "Geotechnical Earthquake Engineering". Prentice Hall, New York]. The reduced coefficient (0.22g) corresponds closely to ground motions with a 475-year return period.

Failure Modes

As expected, translational soil block movements were found to be inconsequential in our assessment. The limited thickness and surface slope of this deposit, as well as mitigating factors in the site development process (eg. surface drainage) resulted in our focussing on deep-seated rotational slope failures that could impact the proposed residences.

Results

The attached Figures A-3 and A-4 show graphic results corresponding to four of the eight load cases (Cases A-1, A-2, B-1, and B-2) summarized in Table 2 below.

As indicated in Table 2, minimum factors of safety for static and pseudo-static (seismic) load cases for both pre- and post-construction geometries were estimated to be 2.3 and 1.3, respectively. These conservative lower bound estimates exceed the generally accepted factors of safety for static and pseudo-static loading (ie. 1.5 and 1.1, respectively). Furthermore, most analyses were based on the application of the full seismic load (A_{max}=0.44g). As noted above, it is estimated that our consideration of soil strengths, potential pore-pressures, and soil stratigraphy are generally conservative.

Table 2 – Slope Stability Analysis Models

Stability Model*	Description of Model	Minimum Factor of Safety
	Pre-Construction	
Case A-1 ("***Base Case***")	Pre-Construction; A_{max}=0.44g Till: Φ=37°; c=10 kPa Overburden & Beach Sand: Φ= 30°; c=0	$(F.S.)_{min} = 1.3$
Case A-2	Base Case (ie. Case A-1) with Amax=0.22g	$(F.S.)_{min} = 1.7$
Case A-3	Base Case with Till: Φ=37°; c=20 kPa	$(F.S.)_{min} = 1.5$
Case A-4	<u>STATIC LOADING</u>: Base Case – No Seismic Load	$(F.S.)_{min} = 2.3$
	Post-Construction	
Case B-1	Post Construction Base Case; Failure Surface that impacts Lower lot	$(F.S.)_{min} = 1.6$
Case B-2	Post Construction Base Case (ie. Case B-1); Alternate Failure Surface that impacts Lower lot	$(F.S.)_{min} = 1.4$
Case B-3	Post Construction Base Case; Failure Surface that impacts Upper lot	$(F.S.)_{min} = 1.6$
Case B-4	<u>STATIC LOADING</u>: Base Case – No Seismic Load	$(F.S.)_{min} = 2.3$

<u>Recommendations and Conclusions</u>

Based on the above, slope stability conditions are judged to be comparatively favourable. We have addressed slope stability under the general guidance of the Association of Professional Engineers & Geoscientists of BC's (APEGBC) "Guidelines for Legislated Landslide Assessments for Proposed Residential Development in British Columbia" (2006). Subsurface conditions were observed to be in accordance with those observed at the neighbouring subdivision to the east. It is judged that the subject site has a comparatively low level of geotechnical complexity, as well as favourable hydrogeological conditions. Furthermore, in our experience, the low-permeability sand till extends well below the failure surfaces that were estimated in our analyses. Based on these factors, a risk assessment was not considered to be warranted.

Our stability review is premised upon the assumption that residence foundations will be constructed on the undisturbed till subgrade and that our mitigative recommendations below will be implemented. To-date, limitations on stormwater retention features directly downslope of Lots 4 to 7 have already been applied to the preliminary stormwater management design (by Webster Engineering) for the proposed development. Limits on retention features have been applied because it is judged such retention features could over time cause softening of the upper/ surface segment of the till on the generally steeper slope segment below these lots 4 to 7.

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. It is assumed that proper drainage practices will be implemented and all foundations will be placed on the very dense till deposit. Due to the steepness of the site's southern extremity, it is suggested that consideration be given to construction of erosion control features that protect against weathering and wave erosion across the lower foreshore. To retard and reduce the size and scale of these potential shallow (translational) failures in the surficial soils, we recommend that the following measures be incorporated for the areas downslope of the building footprints:

- no discharge of water on the site shall occur directly south of 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, and
- 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 sand till across the northern extent of foundations. However, due to sloping grades, overexcavation into the till deposit is expected to be required.

5.2 Static Bearing Pressure

The above-described native, very dense, sand till is considered suitable for support of shallow foundations consisting of strip and pad footings. A preliminary factored (ie. 0.5) bearing resistance/ capacity of 350 kPa at Ultimate Limit State (ULS) would be applicable for the sand (till), subject to field confirmation by PECI. A bearing resistance/ capacity of 200 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.

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, sand (till) should step no steeper than 2.0 Horizontal to 1.0 Vertical (2H:1V).

5.3 Seismic Design – Geotechnical Considerations

Seismic design philosophy has changed to collapse prevention (in NBCC 2005) from the previously addressed conditions of life-safety and moderate damage. In the NBCC 1995, a Foundation Factor representing one of four ground condition profiles was utilized to characterize the potential modification of firm ground motions as they propagate through the generalized soil profile. The current code includes the use of spectral acceleration to define hazard. Compared to NBCC 1995, current design ground motions have a reduced probability of occurrence of 2% in 50 years, or a return period of 2475 years. As shown in Table 3, 5% damped horizontal spectral acceleration values are defined for a range of periods. The four spectral values forming the Hazard Spectra illustrate the range and period dependence of seismic hazard.

NBCC 2005 assigns the ground profile to one of six site classes, which are defined based on the following soil parameters within the upper 30 m of the soil profile: Shear Wave Velocity (V_s), SPT blow count (N_{60}), Undrained Shear Strength (S_u), Plasticity Index (P.I.), and/or Natural Moisture Content (w_n). Different soil conditions affect the ground motion by increasing or decreasing the amplitude of frequency components present in the firm ground motion.

Table 3 – Surface Spectral Response – Local Ground Motions

Period (sec)	0.2	0.5	1.0	2.0
Spectral Acceleration* (g)	0.88	0.61	0.33	0.17

From a geotechnical stand-point, it is judged that surface spectral response would be expected to correspond most closely to Site Class C. Hence, the tabulated ground motions would not be modified (ie. F_a=1.0), and Spectral Response would correspond to the tabulated values. The locally applicable Peak Ground Acceleration (PGA) would be 0.44 g.

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 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).

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, sand to silty sand (till) subgrade. A friction factor of 0.35 may be applied between the concrete and till 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.

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 4 (below).

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.

Table 4 – 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

10.0 REVIEW

Subsurface conditions will require verification during construction in accordance with the District of North Vancouver's Letters of Assurance. Our design information is based on test-pit data, our local experience and local geological data. Field reviews will be required to confirm that soil conditions are consistent with those estimated in this document. In accordance with the District of North Vancouver's Letters of Assurance, the Geotechnical Engineer-of-Record ("G.E.R.") shall be contacted to carry out field reviews for the following items:

- temporary excavation stability,
- foundation and retaining wall subgrades,
- stability of permanent slopes,
- excavation shoring,
- foundation wall backfill,
- 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, the G.E.R. 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
PROVINCE OF
S.S. PUAR
BRITISH COLUMBIA
ENGINEER

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

F:\PECI\PROJECTS\07-2-256 - 3707 Dollarton Hwy NVan - Ebrahim\1 - Reporting & Correspondence\07-2-256 - UPDATED Geotechnical Inv Report - 11Sept04.doc
Distribution: fax _; mail __; courier _; e-mail _; pick-up _;

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.

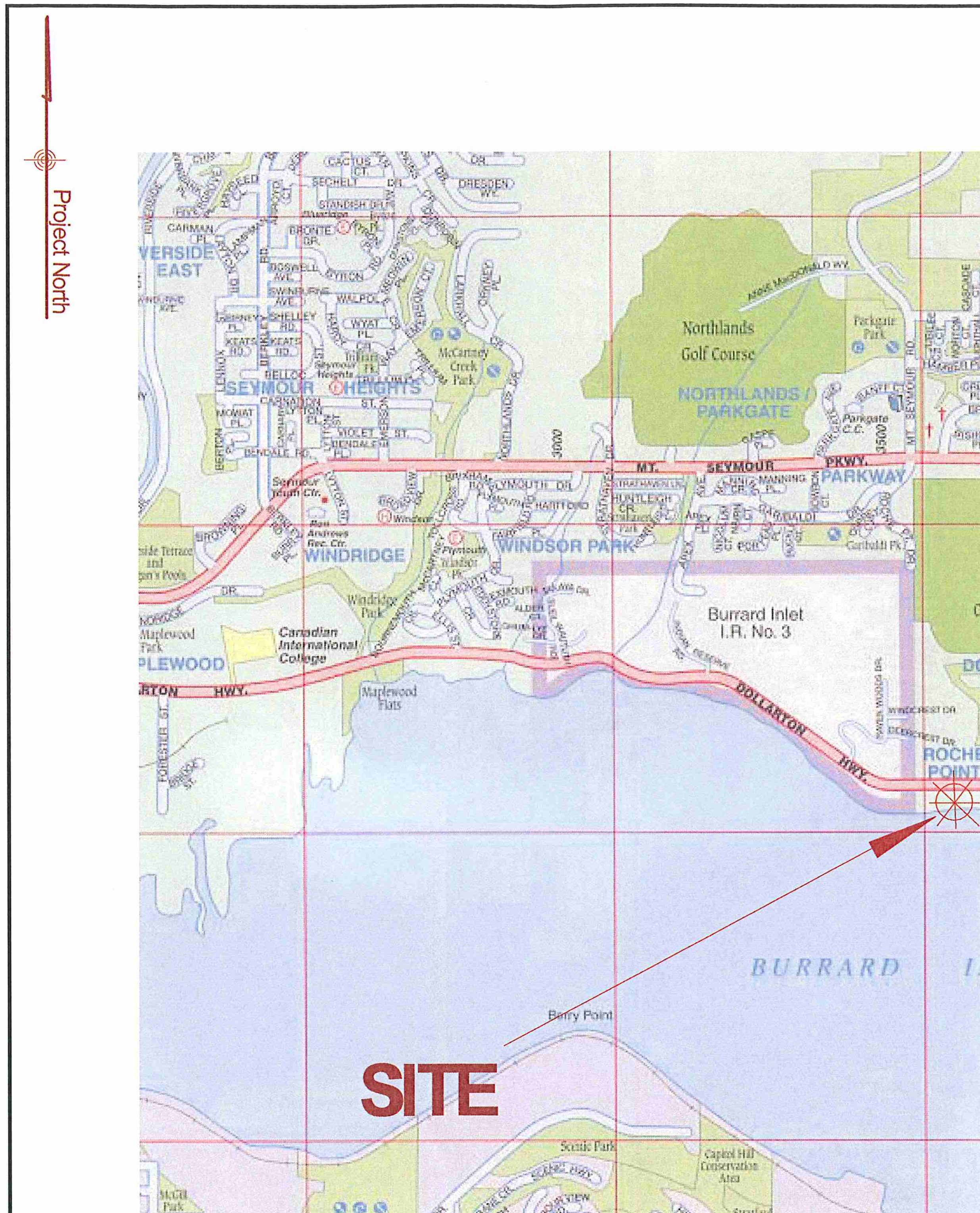
II. USE OF REPORT

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.

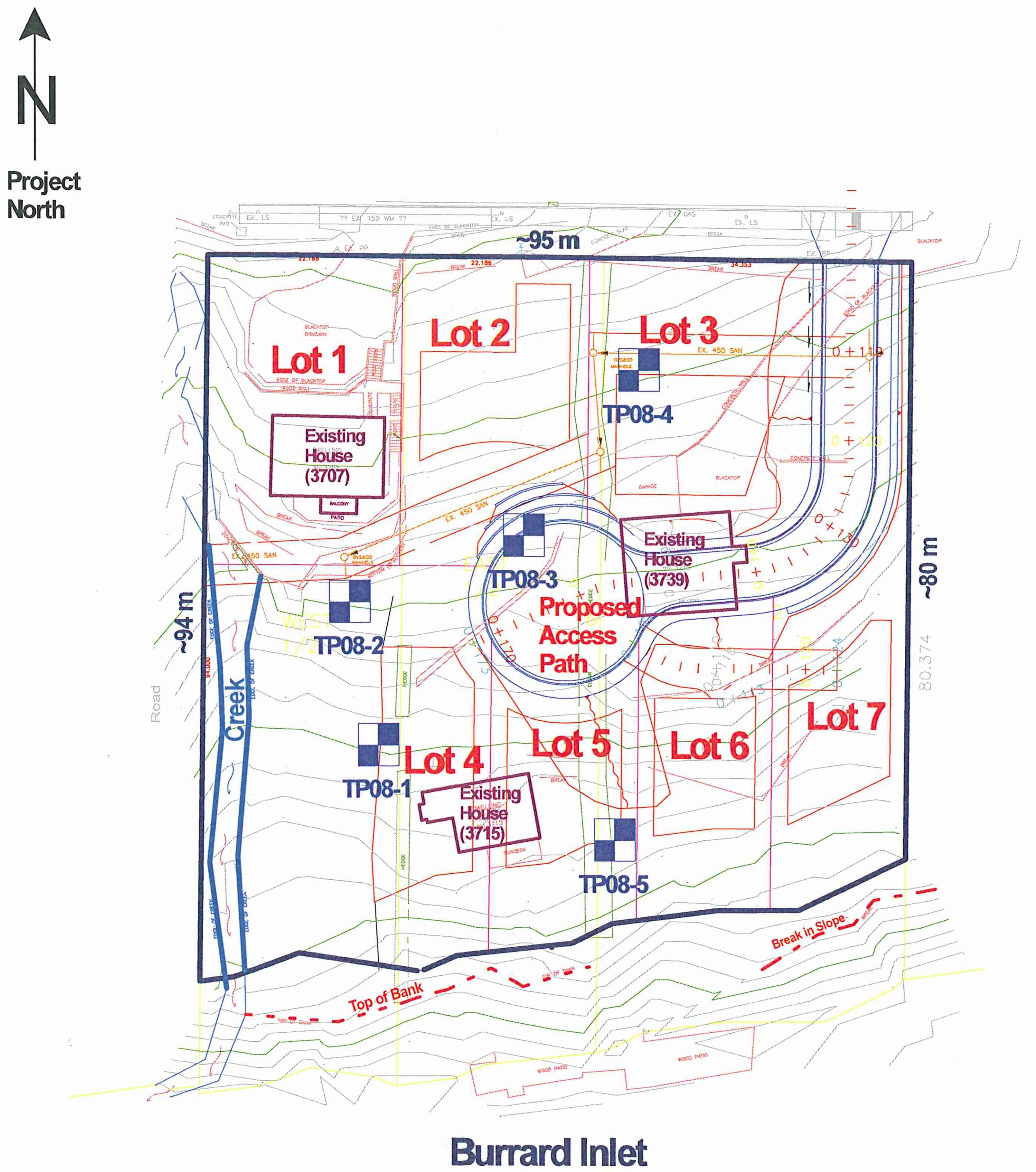
III. INTERPRETATION OF THE REPORT

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.

APPENDIX A – FIGURES



Puar Engineering CONSULTANTS	CLIENT: EBRAHIM			TITLE: Key Plan	
	PROJECT: Proposed Subdivision 3707 Dollarton Hwy - North Vancouver, BC				
	PROJECT NO. 07-2-256	DFTR. wsp	DSGN. sp	DATE 08Oct	SCALE: NTS	DWG. NO. A-1B

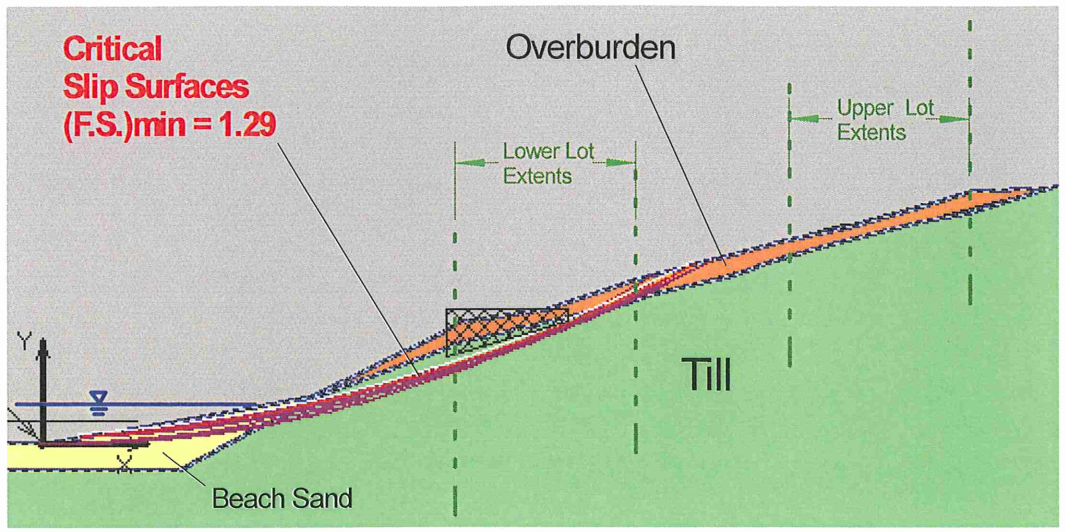


LEGEND

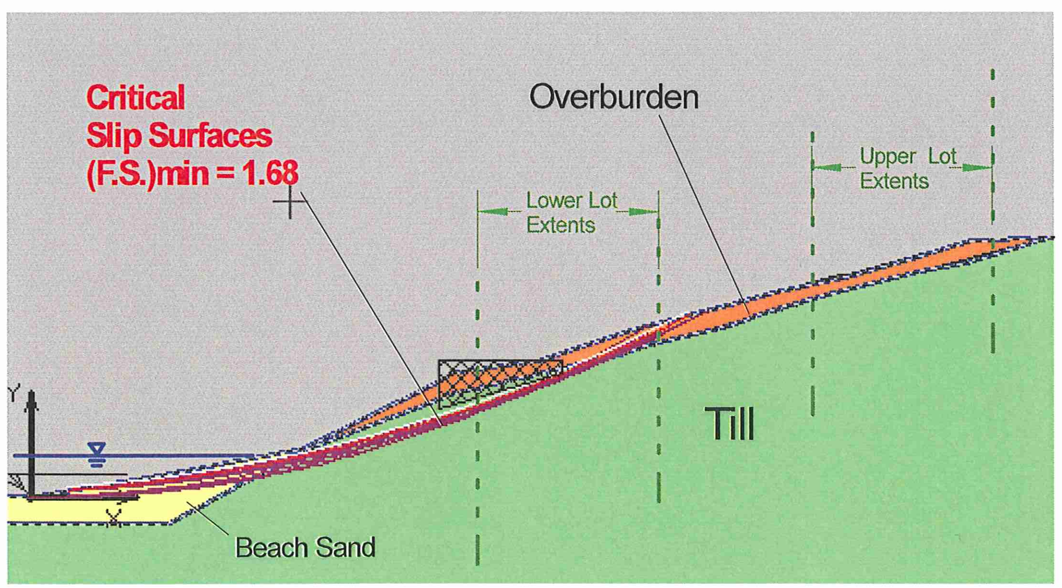
Test-pit

Drawing Not to Scale

Reference Drawing: Webster Engineering - Civil Site Plan (dated 08Feb15)	CLIENT: EBRAHIM	TITLE: Updated 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: 08Oct · SCALE: NTS · DWG NO.: A-2B

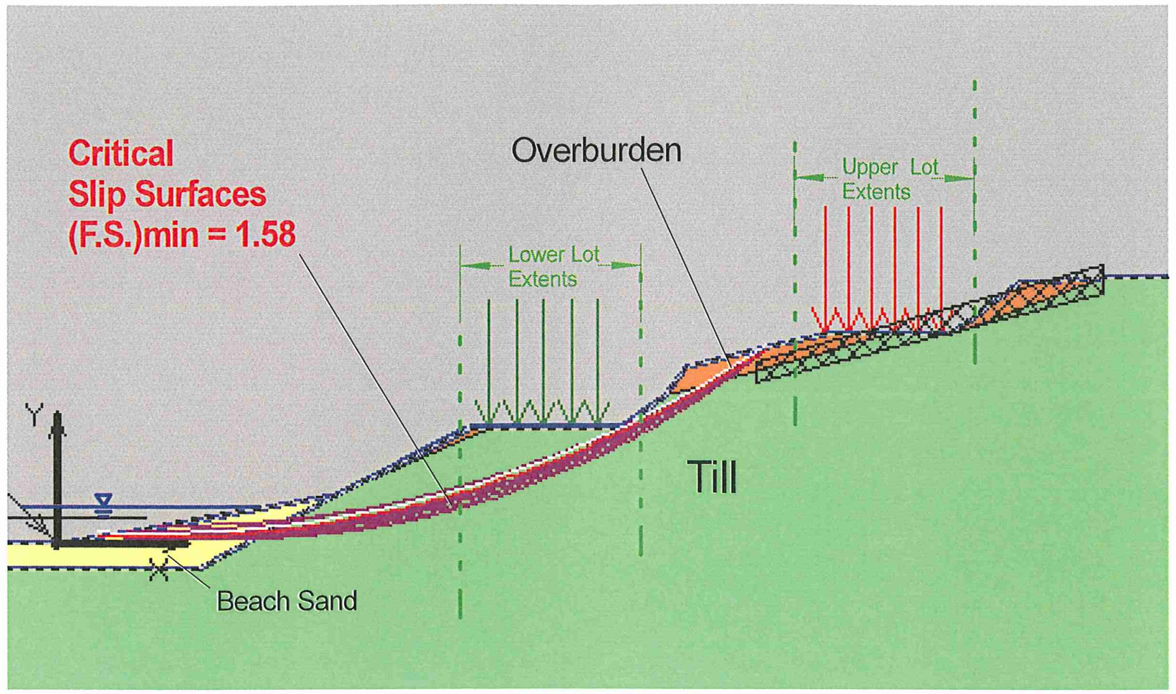


Case A-1 - Pre-Excavation

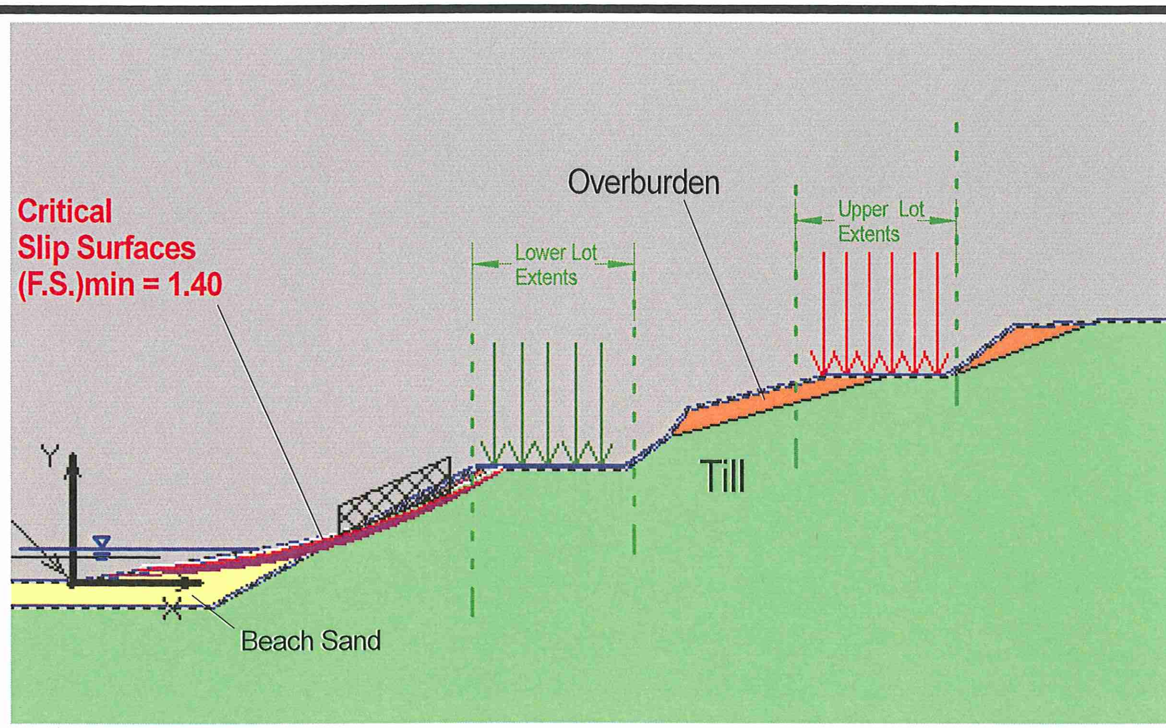


Case A-2 - Pre-Excavation; Reduced Seismic Loading (Amax=0.22g)

Reference Drawing	CLIENT: EBRAHIM	TITLE: Slope Stability Profiles - Case A
Puar Engineering Consultants	PROJECT: Proposed Subdivision, 3707 Dollarton Hwy - North Vancouver, BC	
	PROJECT NO.: 07-2-256 · DFTR.: wsp · DSGN.: sp	DATE: 08Oct · SCALE: NTS · DWG NO.: A-3



Case B-1 - Post-Construction



Case B-2 - Post-Construction (Alternative Failure Scenario)

Reference Drawing	CLIENT: EBRAHIM			TITLE: Slope Stability Profiles - Case B		
Puar Engineering CONSULTANTS	PROJECT: Proposed Subdivision 3707 Dollarton Hwy - North Vancouver, BC					
	PROJECT NO. 07-2-256	DFTR. wsp	DSGN. sp	DATE 08Oct	SCALE: NTS	DWG NO. A-4

APPENDIX B – TEST-HOLE LOGS

Project No. 07-2-256

Log of Test-hole TP08-1

Figure No. B-1

Project: Proposed Subdivision

Sheet No. 1 of 1

Location: 3701 Dollarton Hwy - N.Van., BC

Date Drilled: 3/10/08

Drill Type: Tracked Exc. (300)

Datum: G.S.

Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test

Combustible Vapour Reading
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at % Strain at Failure
Penetrometer

GWL	SYMBOL	SOIL DESCRIPTION	ELEV. m	DEPTH	N Value 20 40 60 80 / Shear Strength MPa 0.1 0.2	Combustible Vapour Reading (ppm) 250 500 750 / Natural Moisture Content % Atterberg Limits (% Dry Weight) 10 20 30	SAMPLES	Natural Unit Weight kN/m³
		Loose, brown, moist Organics and Silty Sand (Forest Litter to TOPSOIL)		0				
		Loose, light brown to red-brown, fine to medium, moist SAND, trace to some gravel, silt and organics (amorphous and rootlets)		1				
		Dense to Very Dense, Light brown to grey, moist SAND (till-like), some gravel and silt; undefined proportion of cobbles and boulders						
		END TEST-PIT						

CAN LOG 07-2-256.GPJ CAN EVAL.GDT 9/29/08

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#200-100 Park Royal South
West Vancouver, BC / V7T1A2
T: 604-913-7827
F: 604-922-5054

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Time	Water Level (m)	Depth to Cave (m)

Project No. 07-2-256

Log of Test-hole TP08-2

Figure No. B-2

Project: Proposed Subdivision

Sheet No. 1 of 1

Location: 3701 Dollarton Hwy - N.Van., BC

Date Drilled: 3/10/08

Drill Type: Tracked Exc. (300)

Datum: G.S.

Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test

Combustible Vapour Reading
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at % Strain at Failure
Penetrometer

GWL	SYMBOL	SOIL DESCRIPTION	ELEV. m	DEPTH	N Value 20 40 60 80 / Shear Strength MPa 0.1 0.2	Combustible Vapour Reading (ppm) 250 500 750 / Natural Moisture Content % Atterberg Limits (% Dry Weight) 10 20 30	SAMPLES	Natural Unit Weight kN/m³
		Loose, brown, moist Organics and Silty Sand (Forest Litter to TOPSOIL)		0				
		Loose, light brown to red-brown, fine to medium, moist SAND, trace to some gravel, silt and organics (amorphous and rootlets)		1				
		Dense to Very Dense, Light brown to grey, moist SAND (till-like), some gravel and silt; undefined proportion of cobbles and boulders						
		END TEST-PIT						

CAN LOG 07-2-256.GPJ CAN EVAL.GDT 9/29/08

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West Vancouver, BC / V7T1A2
T: 604-913-7827
F: 604-922-5054

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Time	Water Level (m)	Depth to Cave (m)

Project No. 07-2-256

Log of Test-hole TP08-3

Figure No. B-3

Project: Proposed Subdivision

Sheet No. 1 of 1

Location: 3701 Dollarton Hwy - N.Van., BC

Date Drilled: 3/10/08

Drill Type: Tracked Exc. (300)

Datum: G.S.

Auger Sample		Combustible Vapour Reading	
SPT (N) Value		Natural Moisture	
Dynamic Cone Test		Plastic and Liquid Limit	
Shelby Tube		Undrained Triaxial at % Strain at Failure	
Field Vane Test		Penetrometer	

GWL	SYMBOL	SOIL DESCRIPTION	ELEV. m	DEPTH	N Value 20 40 60 80 / Shear Strength MPa 0.1 0.2	Combustible Vapour Reading (ppm) 250 500 750 / Natural Moisture Content % Atterberg Limits (% Dry Weight) 10 20 30	SAMPLES	Natural Unit Weight kN/m³
		Loose, brown, moist Organics and Silty Sand (Forest Litter to TOPSOIL)		0				
		Loose, light brown to red-brown, fine to medium, moist SAND, trace to some gravel, silt and organics (amorphous and rootlets)		1				
		Compact, light brown, moist, fine SAND, some silt to silty, trace gravel		2, 3				
		Dense to Very Dense, Light brown to grey, moist SAND (till-like), some gravel and silt; undefined proportion of cobbles and boulders						
		END TEST-PIT						

Time	Water Level (m)	Depth to Cave (m)

CAN LOG 07-2-256.GPJ CAN EVAL.GDT 9/29/08

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Project No. 07-2-256

Log of Test-hole TP08-4

Figure No. B-4

Project: Proposed Subdivision

Sheet No. 1 of 1

Location: 3701 Dollarton Hwy - N.Van., BC

Date Drilled: 9/26/08

Drill Type: Hand-dug

Datum: G.S.

Auger Sample		Combustible Vapour Reading	
SPT (N) Value		Natural Moisture	
Dynamic Cone Test		Plastic and Liquid Limit	
Shelby Tube		Undrained Triaxial at % Strain at Failure	
Field Vane Test		Penetrometer	

GWL	SYMBOL	SOIL DESCRIPTION	ELEV. m	DEPTH	N Value 20 40 60 80 / Shear Strength MPa 0.1 0.2	Combustible Vapour Reading (ppm) 250 500 750 / Natural Moisture Content % Atterberg Limits (% Dry Weight) 10 20 30	SAMPLES	Natural Unit Weight kN/m³
		Loose, brown, moist Organics and Silty Sand (Forest Litter to TOPSOIL)		0				
		Loose, light brown to red-brown, fine to medium, moist SAND, trace to some gravel, silt and organics (amorphous and rootlets)						
		Dense to Very Dense, Light brown to grey, moist SAND (till-like), some gravel and silt; undefined proportion of cobbles and boulders		1				
		END TEST-PIT						

Time	Water Level (m)	Depth to Cave (m)

CAN LOG 07-2-256.GPJ CAN EVAL.GDT 9/29/08

Puar Engineering Consultants Inc
#200-100 Park Royal South
West Vancouver, BC / V7T1A2
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Puar Engineering CONSULTANTS

Log of Test-hole TP08-5

Project No. 07-2-256

Figure No. B-5

Sheet No. 1 of 1

Project: Proposed Subdivision

Location: 3701 Dollarton Hwy - N.Van., BC

Date Drilled: 9/26/08

Drill Type: Hand-dug

Datum: G.S.

Auger Sample	◫
SPT (N) Value	○ ⊠
Dynamic Cone Test	—
Shelby Tube	□
Field Vane Test	S
Combustible Vapour Reading	□
Natural Moisture	×
Plastic and Liquid Limit	├─○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	SOIL DESCRIPTION	ELEV. m	DEPTH	N Value (20 / 40 / 60 / 80); Shear Strength MPa (0.1 / 0.2)	Combustible Vapour Reading (ppm) (250 / 500 / 750); Natural Moisture Content %, Atterberg Limits (% Dry Weight) (10 / 20 / 30)	SAMPLES	Natural Unit Weight kN/m³
		Loose, brown, moist Organics and Silty Sand (Forest Litter to TOPSOIL)		0				
		Loose, light brown to red-brown, fine to medium, moist SAND, trace to some gravel, silt and organics (amorphous and rootlets)		1				
		Dense to Very Dense, Light brown to grey, moist SAND (till-like), some gravel and silt; undefined proportion of cobbles and boulders						
		END TEST-PIT						

CAN LOG 07-2-256.GPJ CAN EVAL.GDT 9/29/08

Time	Water Level (m)	Depth to Cave (m)

Puar Engineering Consultants Inc
#200-100 Park Royal South
West Vancouver, BC / V7T1A2
T: 604-913-7827
F: 604-922-5054

Puar Engineering CONSULTANTS

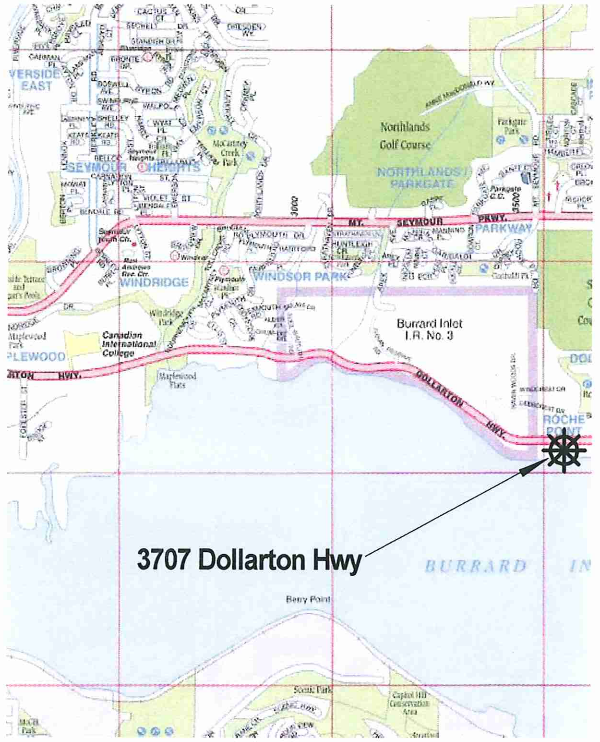
SEGMENTAL, REINFORCED-EARTH RETAINING WALL DESIGN - ROAD 'A'

Project:

PROPOSED SUBDIVISION 3707 DOLLARTON HWY - N.VANCOUVER, BC

Client:

NICK EBRAHIM



Drawing List:

PUAR ENGINEERING CONSULTANTS INC

Puar Engineering CONSULTANTS

1.0 GENERAL

1.1 Project Team, Reference Documents, and Proposed Development
- Puar Engineering Consultants Inc's (herein referred to as "PECI") is the geotechnical design consultant.
- PECI's Geotechnical Investigation Report (dated 11Sept04)
- Civil design drawings by Creus Engineering Ltd ("Creus") (drawings via email 11Sept21)
- Landscape design drawings by Forma Design (drawings via email 12Feb07)

- The proposed Allan Block segmental retaining walls will provide stabilization for the the grade difference adjacent the east and south edge of Road 'A' via one- to two-level terraces. PECI's design geometries are based on Creus Engineering's section geometries (Dwg.#DF 8734).

1.2 Estimated Subsurface Conditions
- Based on local experience, preliminary site observations (including soil exposures), and information from the Geological Survey of Canada (GSC), the native materials in the general vicinity are expected to consist of compact sand overlying a dense, till-like sand to silty sand deposit.
- Limited Interflow/ groundwater flow may perch on glaciated deposits.

1.3 Design Parameters

a.) Bearing Capacity of i.) till-like sand Subgrade - factored bearing resistance/ capacity of 300 kPa (6270 psf) at Ultimate Limit State (ULS); ii.) compact sand subgrade - factored bearing resistance/ capacity of 240 kPa (5000 psf) at Ultimate Limit State (ULS)

b.) LIVE LOAD: 12kPa; DEAD LOAD: None.

1.4 Site Drainage
- The internal drainage system of the retaining wall system is designed to remove incidental water that surficially infiltrates the backfill of the walls. Consequently, it is imperative that site grading and drainage systems (storm sewers, roof downspouts, curb gutters, etc.) be suitably designed and constructed, such that all flow collection points and outlets are outside the retaining wall area.
- Site grading shall be designed to route surface water around and away from the walls.

1.5 Other Loading Considerations
- Walls have not been designed to accommodate temporary construction surcharge loading (material or soil stockpiles, etc.). Such additional temporary loading may induce damage or failure of the walls.
- Walls are designed to support light-duty passenger vehicles.

2.0 DESIGN & CONSTRUCTION RECOMMENDATIONS

2.1 General
- The attached design drawings correspond to the shown wall heights only. These heights shall not be exceeded without confirmation by PECI.
- The Contractor shall ensure that the Allan Block Technical Specifications Manual shall be adhered to with respect to construction items not discussed within the text of this report. No recommendations within the Allan Block manual shall take precedence over the recommendations within PECI's design.

2.2 Site Preparation

a.) All disturbed and loose soil, fill, and organic-rich material (topsoil, etc.) shall be stripped across the footprints of the proposed retaining walls (ie. across facade and extent of the geogrid reinforcement).

b.) Temporary excavation slopes shall be constructed no steeper than 1 Vertical to 1 Horizontal (3V:2H), or as recommended by PECI on site. If significant seepage flows are observed at the excavation face, PECI shall be notified immediately.

c.) The exposed excavation slopes shall be covered by 6-mil polyethylene sheeting that is suitably secured (to resist wind action, etc.). The sheeting shall be removed prior to placing any backfill.

2.3 Wall Preparation

a.) Allan Blocks shall be installed in accordance with the manufacturer's recommendations.

b.) All Allan Blocks shall be filled with ¾" (19mm) clear crushed gravel, or concrete, at every lift as per the manufacturer's recommendations.

2.4 Fill Selection and Placement

a.) WALL BASE LAYER: The base layer for the walls shall consist of a minimum 6" (150 mm) thick layer of ¾" (19mm) clear crushed gravel or ¾"-minus sand and gravel containing less than 5% passing the No 200 US sieve size. If over-excavation is required to expose a suitable subgrade, the base layer shall be thickened, accordingly, as approved in the field by PECI.

b.) WALL BACKFILL WITHIN GEOGRID ZONE: Engineered Fill within the area containing geogrid reinforcement shall consist of imported, clean, granular material such as ¾" (19 mm) clear crushed gravel or 3" (75 mm) minus sand and gravel containing less than 5% passing the No 200 US sieve size or approved equivalent.

c.) WALL BACKFILL BEYOND GEOGRID ZONE: Engineered Fill beyond the area containing geogrid reinforcement shall consist of either one of the above-mentioned materials, or 6" (150 mm) minus sand and gravel containing less than 10% passing the No 200 US sieve size.

d.) SEPARATION LAYER BETWEEN WALL BACKFILL AND PAVEMENT BASE FILL: A layer of Amoco 4506 Non-woven Filter Fabric shall be used as a separation layer between the pavement base fill and the wall backfill below.

e.) All Engineered Fill shall be placed and compacted in 1' (300 mm) thick (maximum) lifts to greater than 95% of its Modified Proctor Maximum Dry Density (per ASTM D1557).

f.) Suitable (ie. comparatively light-weight) compaction equipment shall be used to compact Engineered Fill. To prevent excessive wall face movement, compaction equipment shall be operated parallel to the wall facades; compaction shall start at the facade and work away from the walls.

2.5 Drainage

a.) 4" (100 mm) diameter perforated PVC pipe surrounded by a minimum 6" (150 mm) thick layer of ¾" (19 mm) clear crushed gravel shall be placed behind the walls, as shown on the design drawings.

b.) A drainage zone behind the wall facades shall consist of a minimum 12" (300 mm) wide zone of ¾" (19 mm) clear crushed gravel; this drainage zone shall extend to within 12" (300 mm) of the top of the walls. A minimum 6" (150 mm) thick layer of pea (or birdseye) gravel shall be placed over the clear crushed gravel prior to placing other landscape (surface) fills (eg. topsoil, etc.).

c.) all segments of the perforated PVC drain lines should be accessible via cleanouts. As such, the spacing of cleanouts will vary.

d.) drainage should be conveyed to an approved discharge point.

2.6 Geogrid Reinforcement

a.) Geogrid shall be Mirafi Miragrid 7XT (or approved polyester equivalent)

b.) Minimum geogrid lengths are indicated in the attached design drawings.

c.) Geogrid lengths in the field shall correspond to the next tallest design section, as shown in the attached drawings (eg. for a wall segment that has an exposed height of 1.6 m, the geogrid lengths for the 2.0 m high design segment would be applied)

d.) The orientation (ie. maximum strength direction) of the geogrid shall be verified by the contractor.

e.) The geogrid reinforcement shall be attached to the concrete fascia by means of the friction connection. Once attached to the concrete fascia, the geogrid reinforcement shall be pulled taut toward the back of the reinforced fill zone, in order to remove all slack prior to fill placement (on top of it).

f.) Construction equipment shall not be operated directly on the geogrid. A minimum thickness of 150 mm of the backfill shall be placed over the geogrid prior to allowing any light-weight construction equipment on the geogrid.

2.7 Guard Rails

a.) Vehicle Barriers and Posts shall be in accordance with Section 312 of BC Ministry of Transportation & Highways ("M.o.T.H.") document "2006 Design Build Standard Specifications for Highway Construction" (Dwg 1-SP312).

b.) Pedestrian Safety Barriers shall be in accordance with Drawing No. SSD-R.31 of MMCD Supplementary Specifications and Standard Drawings Schedule D.2 per District of North Vancouver bylaw document "DISTRICT OF NORTH VANCOUVER DEVELOPMENT SERVICING BYLAW NO. 7388, 2005".

3.0 FIELD REVIEW

- During construction, PECI shall be contacted by the Contractor to attend the site to observe the following aspects of each wall segment:

a.) subgrade preparation to confirm that native material meets criteria (as outlined in Points 1.2 and 1.3, above),

b.) backfill selection and densification,

c.) geogrid reinforcement, and

d.) drainage installation.

4.0 REFERENCE DRAWINGS - PRIORITY

The Geometries and Site Data shown on Civil and Landscape Architectural Design Drawings Take Precedence Over the Information Shown in Puar Engineering's Drawings. If any Conflict/ Discrepancy is Observed Between PECI's Design Drawings and those of Creus Engineering and/or Forma Design and Puar, it Shall be Reported Immediately to PECI.

Revisions

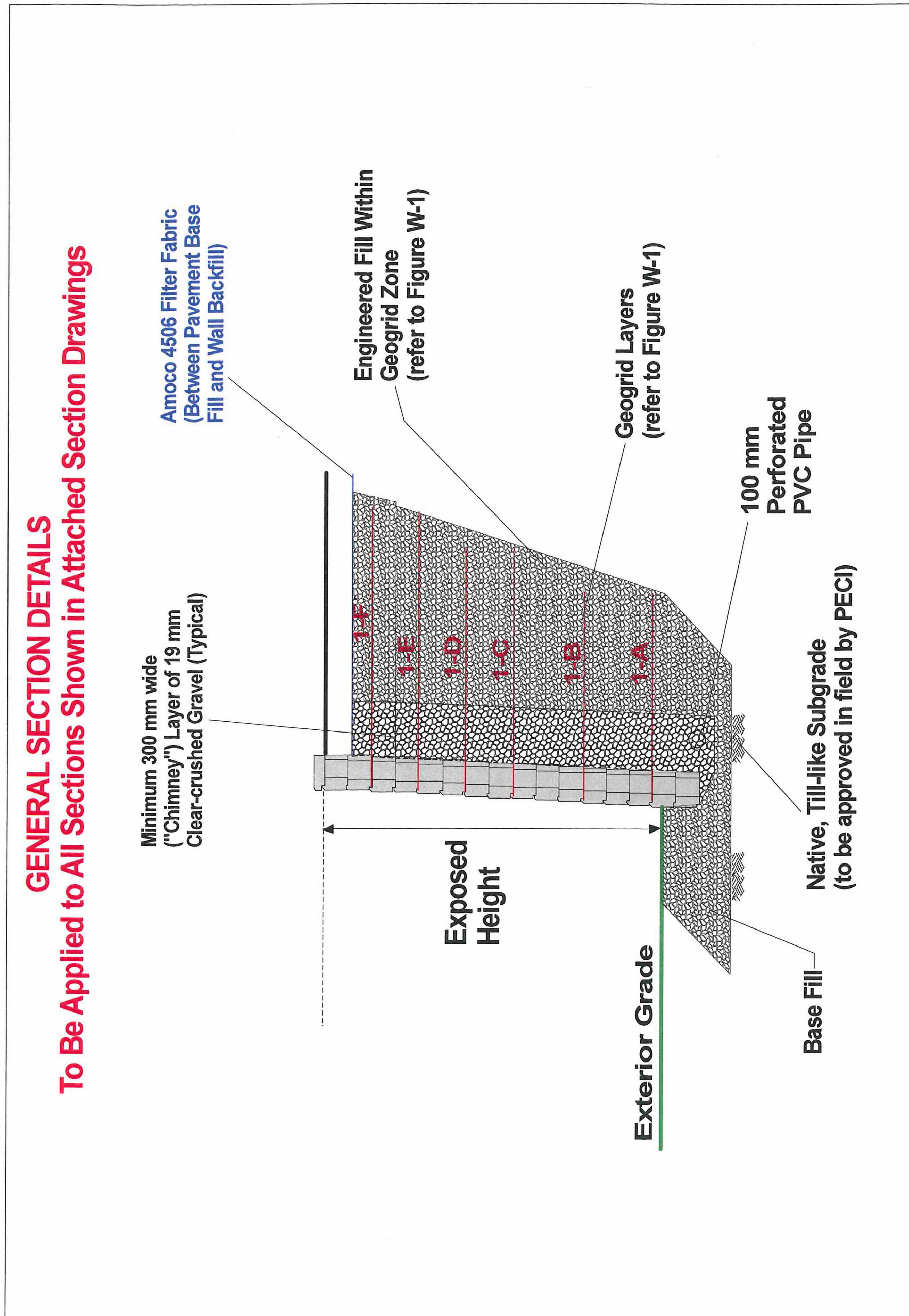
CLIENT: EBRAHIM
3707 Dollarton Hwy, North Vancouver, BC
Proposed Segmental Retaining Wall Systems

Reference Drawings

ROAD 'A' SEGMENTAL RETAINING WALL DESIGN & CONSTRUCTION RECOMMENDATIONS

Puar Engineering CONSULTANTS

Scale:	Job No:	Date:	FIGURE:	Rev:	Dwn:	Chk:
n/a	07-2-256	12Feb	W-1		wsl	sp



Puar Engineering
CONSULTANTS

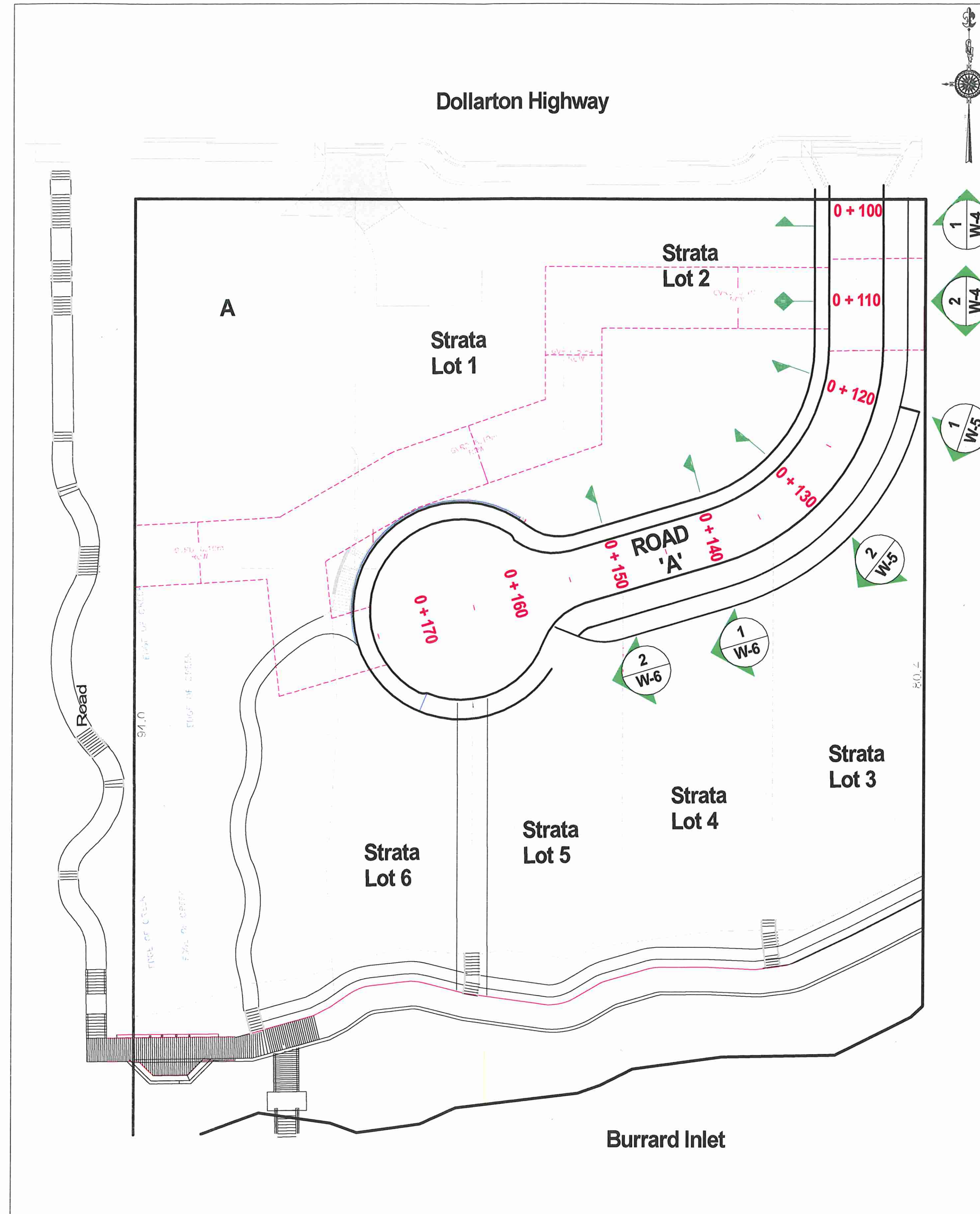
EBRAHIM
Proposed Allan Block Retaining Wall Systems
3707 Dollarton Hwy - N. Vancouver, BC

07-2-256 | wsl | sp

GENERAL SECTION DETAILS

Feb/12 | NTS | W-3

REFERENCE



Reference Drawing
Webster Engineering - Civil Site Plan
Dwg. DF 8728 (emailed 11Sept21)

Puar Engineering
CONSULTANTS

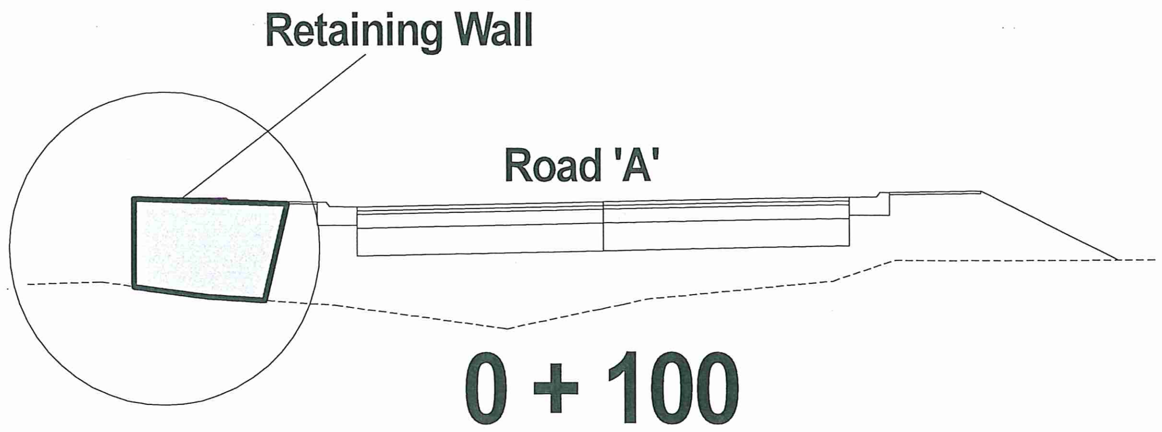
CLIENT: Nick Ebrahim

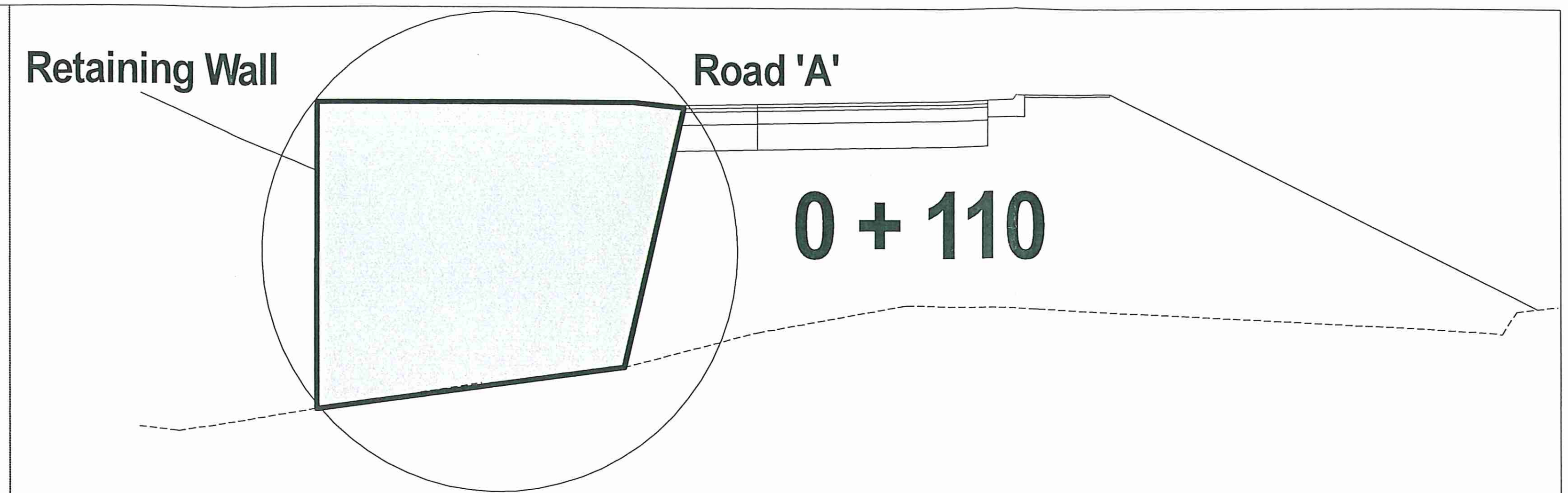
PROJECT: Proposed Subdivision
3707 Dollarton Hwy - North Vancouver, BC

PROJECT NO.	DFTR.	DSGN.
07-2-256	wsl	sp

TITLE: RETENTION PLAN
ROAD 'A'

DATE	SCALE:	DWG NO.
12Feb	1:500	W-2





Geogrid Lengths:
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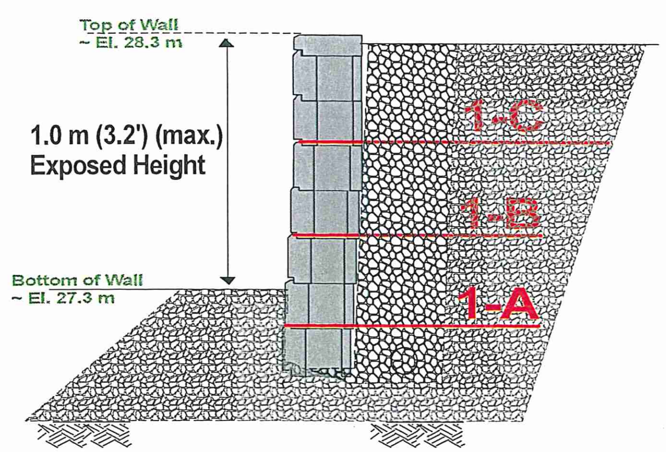
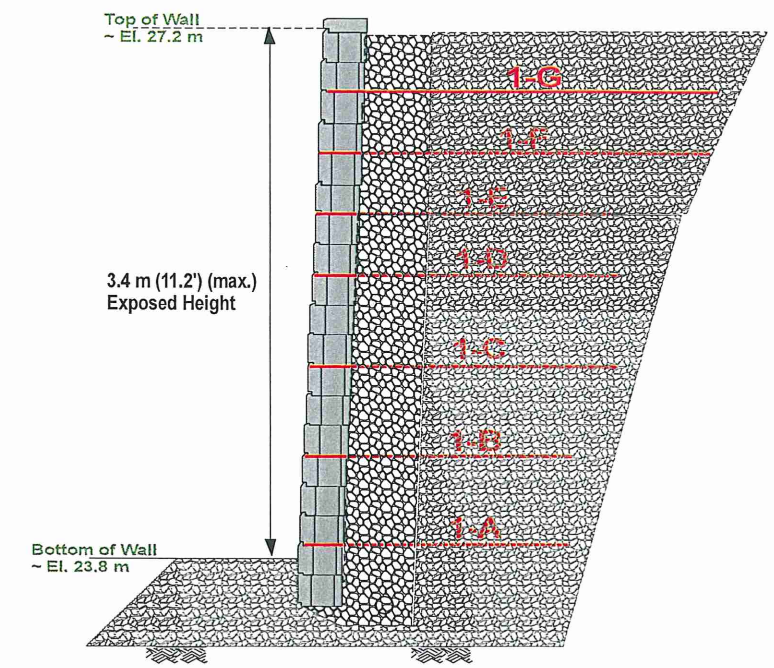
1-B = 1.2 m
1-C = 1.6 m



Geogrid Lengths:
1-A = 2.8 m
1-B = 2.8 m
1-C = 3.2 m
1-D = 3.2 m
1-E = 3.2 m
1-F = 4.3 m
1-G = 4.3 m



Notes:
1.) Refer to Figure W-1 (for Specifications).
2.) Refer to Figure W-3 for Typical Section Details (Engineered Fill, Subgrade, etc).
3.) The Geometries and Site Data shown on Civil and Landscape Architectural Design Drawings Take Precedence Over the InformationShown in Puar Engineering's Drawings. If any Conflict/ Discrepancy is Observed Between PECI's Design Drawings and those ofCreus Engineering and/or Forma Design and Puar, it Shall be Reported Immediately to PECI.
4.) Refer to Forma Design (Landscape) and Creus Engineering (Civil) Design Drawings for Spatial and Elevation Information

Revisions

CLIENT: EBRAHIM
3707 Dollarton Hwy - North Vancouver, BC
Proposed Segmental Retaining Walls

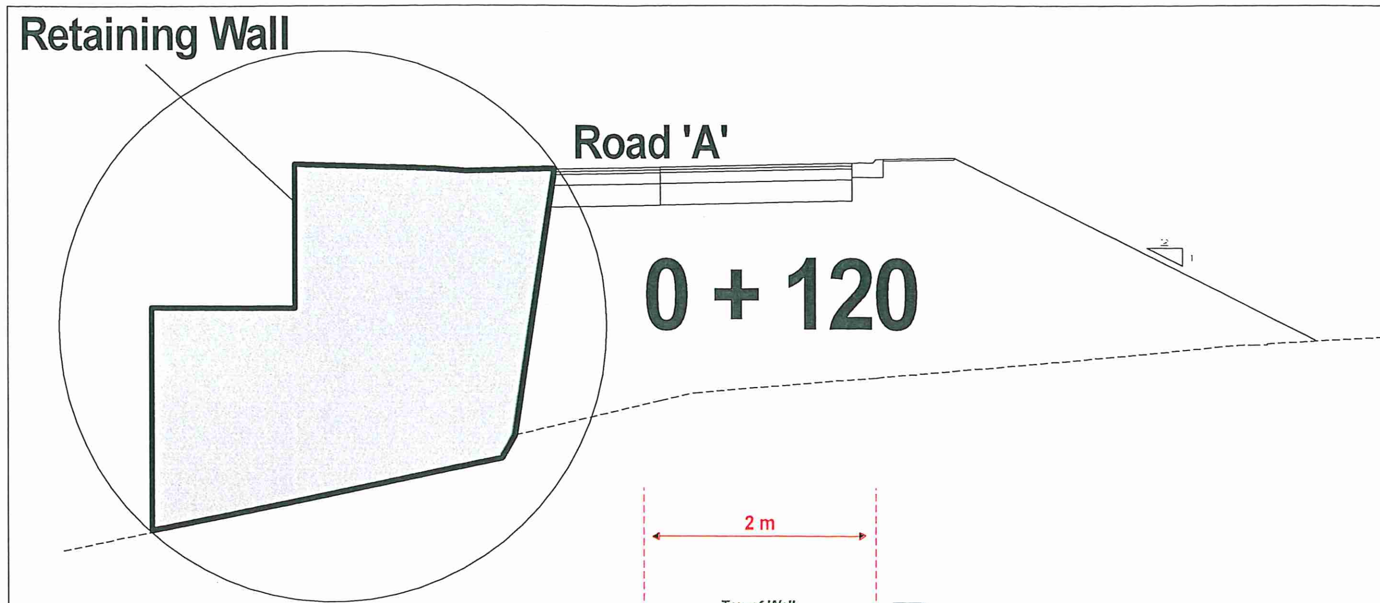
Reference Drawings
Webster Engineering - Civil Site Plan
Dwg. DF 8734 (emailed 11Sept21)

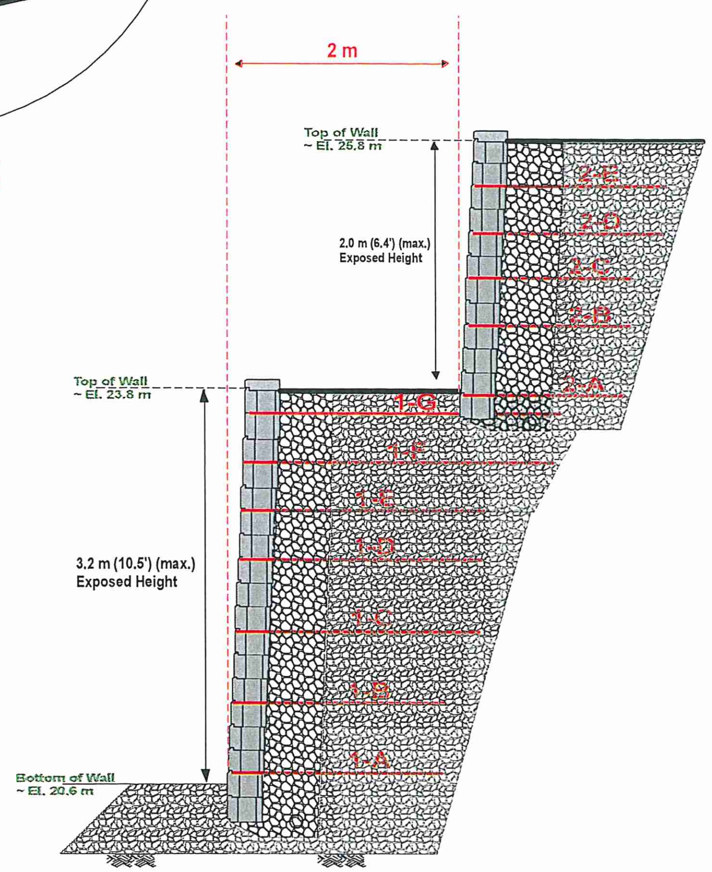
SECTION: 0+100, 0+110 ROAD 'A' - EAST/ SOUTH EDGE TERRACES

Scale: NTS

Puar Engineering
CONSULTANTS

Job No: 07-2-256	Date: 12Feb	FIGURE: W-4
Rev:	Dwn: wsl	Chk: sp

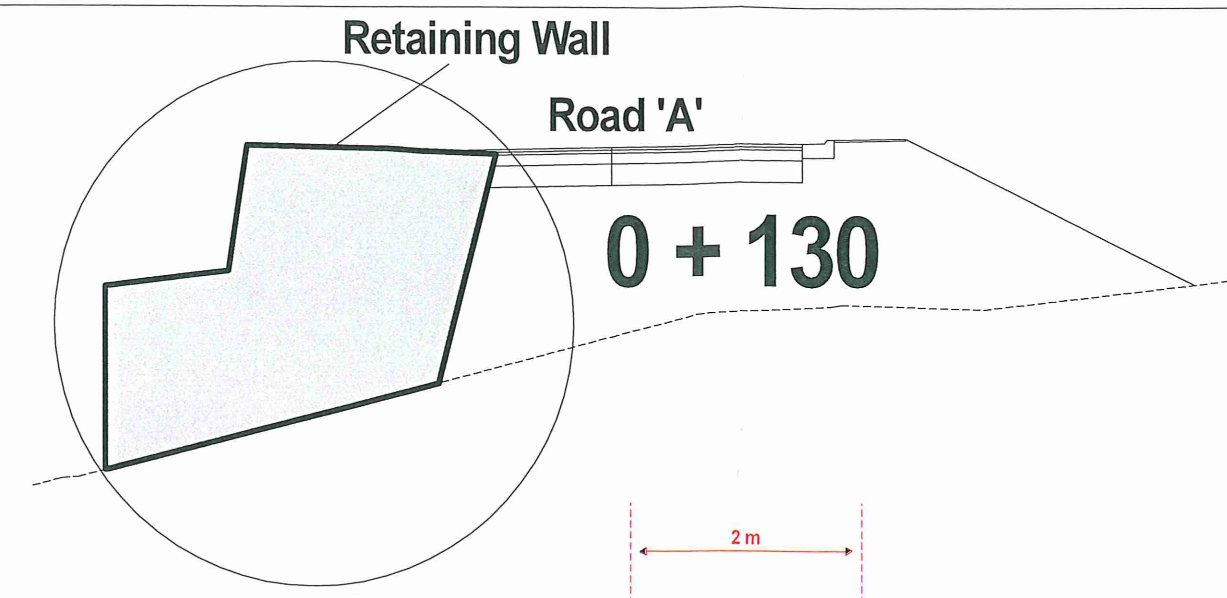


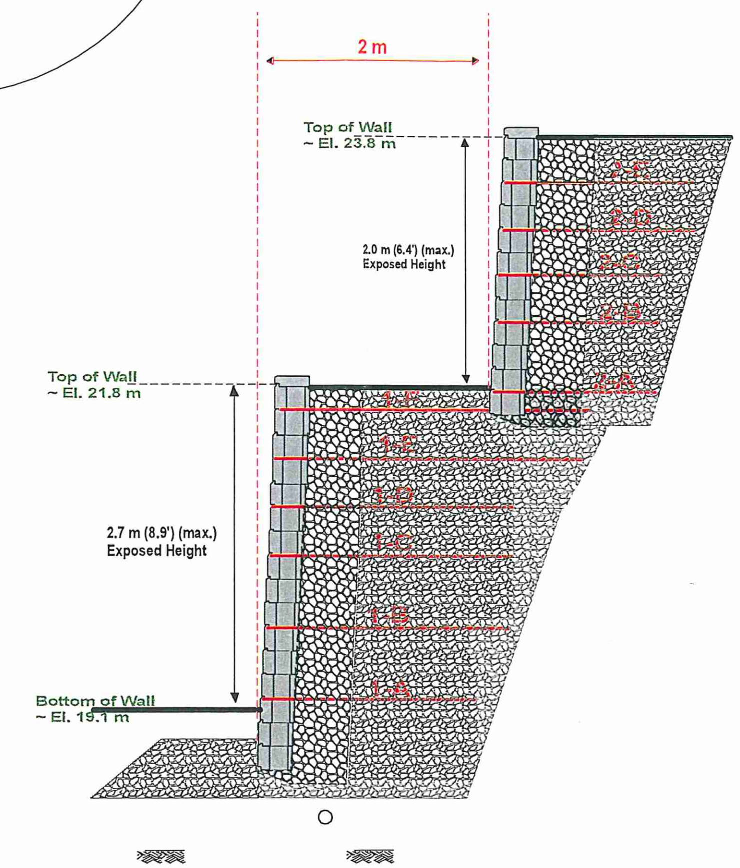


Geogrid Lengths:

1-A = 4.1 m
1-B = 4.1 m
1-C = 4.9 m
1-D = 4.9 m
1-E = 4.9 m
1-F = 5.5 m
1-G = 5.5 m

2-A = 2.2 m
2-B = 2.2 m
2-C = 2.2 m
2-D = 2.8 m
2-E = 2.8 m





Geogrid Lengths:

1-A = 4.1 m
1-B = 4.1 m
1-C = 4.9 m
1-D = 4.9 m
1-E = 4.9 m
1-F = 4.9 m

2-A = 2.2 m
2-B = 2.2 m
2-C = 2.2 m
2-D = 2.8 m
2-E = 2.8 m

Notes:

1.) Refer to Figure W-1 (for Specifications).
2.) Refer to Figure W-3 for Typical Section Details (Engineered Fill, Subgrade, etc).
3.) The Geometries and Site Data shown on Civil and Landscape Architectural Design Drawings Take Precedence Over the InformationShown in Puar Engineering's Drawings. If any Conflict/ Discrepancy is Observed Between PECI's Design Drawings and those ofCreus Engineering and/or Forma Design and Puar, it Shall be Reported Immediately to PECI.
4.) Refer to Forma Design (Landscape) and Creus Engineering (Civil) Design Drawings for Spatial and Elevation Information

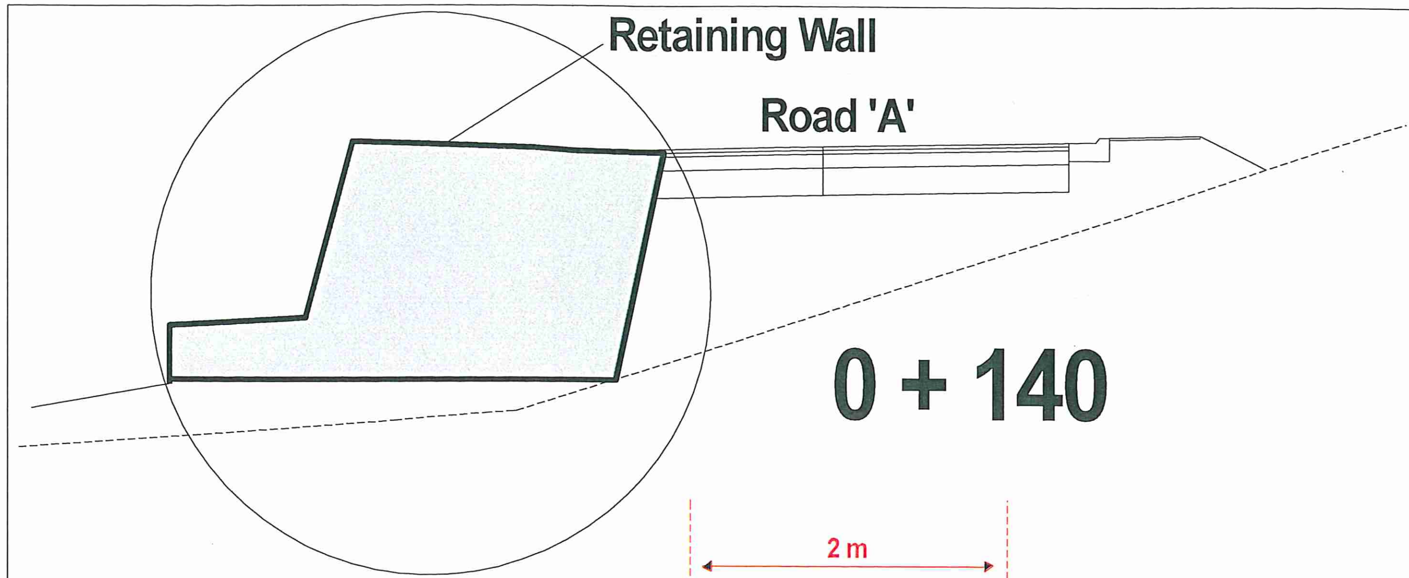
Revisions

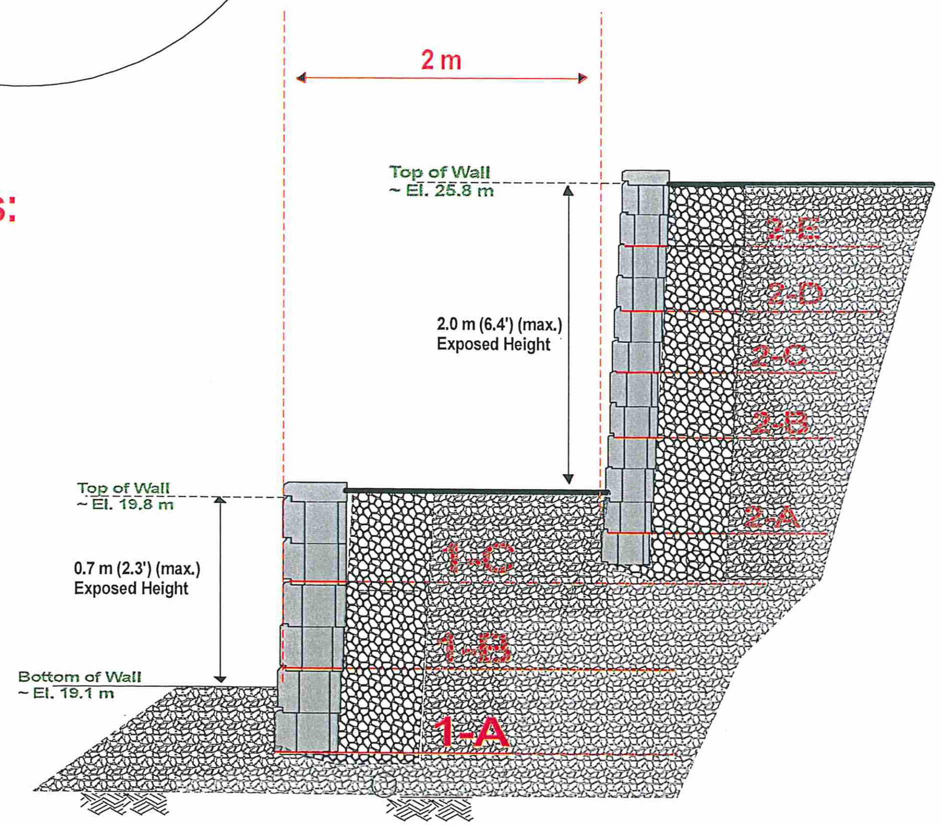
CLIENT: EBRAHIM
3707 Dollarton Hwy - North Vancouver, BC
Proposed Segmental Retaining Walls

Reference Drawings
Webster Engineering - Civil Site Plan
Dwg. DF 8734 (emailed 11Sept21)

SECTION: 0+120, 0+130 ROAD 'A' - EAST/ SOUTH EDGE TERRACES

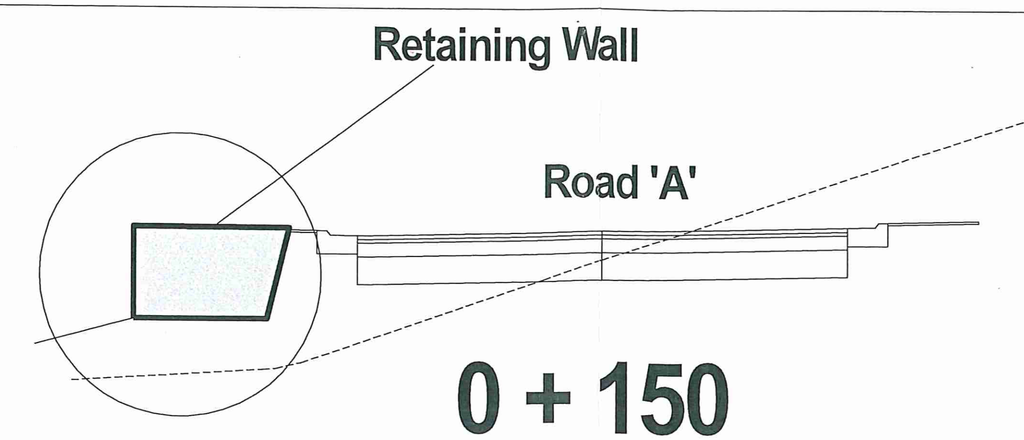
Scale: NTS	Puar Engineering Consultants		
Job No: 07-2-256		Date: 12Feb	FIGURE: W-5
Rev:	Dwn: wsl	Chk: sp	



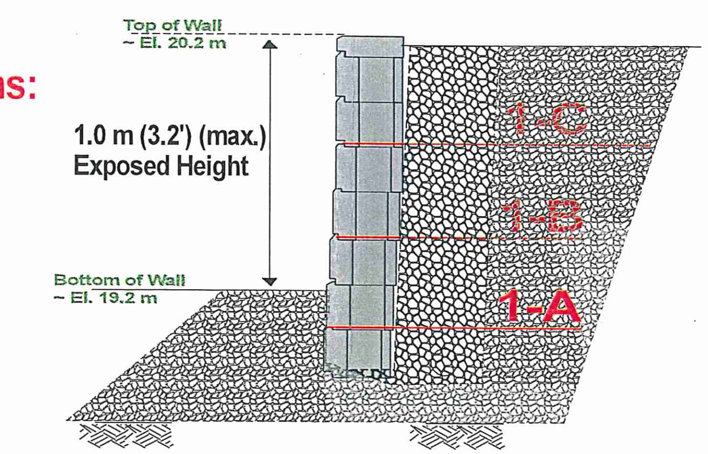


Geogrid Lengths:
1-A = 2.4 m
1-B = 2.4 m
1-C = 2.8 m

2-A = 2.2 m
2-B = 2.2 m
2-C = 2.2 m
2-D = 2.8 m
2-E = 2.8 m



Geogrid Lengths:
1-A = 1.2 m
1-B = 1.2 m
1-C = 1.6 m



Notes:
1.) Refer to Figure W-1 (for Specifications).
2.) Refer to Figure W-3 for Typical Section Details (Engineered Fill, Subgrade, etc).
3.) The Geometries and Site Data shown on Civil and Landscape Architectural Design Drawings Take Precedence Over the InformationShown in Puar Engineering's Drawings. If any Conflict/ Discrepancy is Observed Between PECI's Design Drawings and those ofCreus Engineering and/or Forma Design and Puar, it Shall be Reported Immediately to PECI.
4.) Refer to Forma Design (Landscape) and Creus Engineering (Civil) Design Drawings for Spatial and Elevation Information

Revisions

CLIENT: EBRAHIM
3707 Dollarton Hwy - North Vancouver, BC
Proposed Segmental Retaining Walls

Reference Drawings
Webster Engineering - Civil Site Plan
Dwg. DF 8734 (emailed 11Sept21)

SECTION: 0+140, 0+150
ROAD 'A' - EAST/ SOUTH EDGE TERRACES

Scale: NTS

Puar Engineering CONSULTANTS

Job No: 07-2-256	Date: 12Feb	FIGURE: W-6
Rev:	Dwn: wsl	Chk: sp

1.0 GENERAL

1.1 Project Team, Reference Documents, and Proposed Development
- Puar Engineering Consultants Inc's (herein referred to as "PECI") is the geotechnical design consultant.
- PECI's Geotechnical Investigation Report (dated 11Sept04)
- Civil design drawings by Creus Engineering Ltd ("Creus") (drawings via email 11Sept21)
- Landscape design drawings by Forma Design (drawings via email 12Feb07)

- The proposed Cantilevered Pipe Pile retaining walls will provide stabilization for the the grade difference adjacent the north and west edge of the Road 'A' cul de sac.
- PECI's design geometries are based on Forma Design's Landscape Plan Layout (Dwg. # L1) and Creus Engineering's site section geometries (Dwg.#DF 8741).

1.2 Estimated Subsurface Conditions
- Based on PECI's field investigation, our local experience, and information from the Geological Survey of Canada (GSC), the native materials in the general vicinity are expected to consist of up to 3 m of overburden (including compact sand) overlying the dense, till-like sand to silty sand deposit.
- Limited Interflow/ groundwater flow may perch on glaciated deposits.

1.3 Design Parameters
a.) Bearing Capacity of i.) till-like sand Subgrade - factored bearing resistance/ capacity of 300 kPa (6270 psf) at Ultimate Limit State (ULS); ii.) compact sand subgrade - factored bearing resistance/ capacity of 240 kPa (5000 psf) at Ultimate Limit State (ULS)
b.) LIVE LOAD: None; DEAD LOAD: None.

1.4 Site Drainage
- It is imperative that site grading and drainage systems (storm sewers, roof downspouts, curb gutters, etc.) be suitably designed and constructed, such that all flow collection points and outlets are outside the retaining wall area.
- Site grading shall be designed to route surface water around and away from the walls.

2.0 PRE-CONSTRUCTION REVIEW & OTHER CONSIDERATIONS
It is recommended that no construction take place until the following items have been addressed by the General Contractor and/or Client.

2.1 Utilities & Other Subsurface Elements
Prior to construction, the General Contractor shall determine the locations of all structures and underground services that may be affected by the proposed works; this includes proposed and existing building offsets and depths. The General Contractor shall verify utility information by independently collecting relevant reference drawings and obtaining field verification as required. All work shall be carried out without disturbance to existing utilities. The General Contractor shall notify PECI and the utility companies a minimum of 72 hours before commencing excavation.

2.2 Hoarding
For public safety, the General Contractor shall install hoarding that is adequately braced along all perimeter slopes. Hoarding should meet the minimum of requirements of the District of West Vancouver.

3.0 MATERIALS

3.1 Grout
Non-shrink Cementitious grout shall be "Microsil" grout, or approved equivalent. Grout shall be batched in accordance with the manufacturer's specifications and shall have a minimum compressive strength of 21 MPa (3.0 ksi) after 24 hours and 35 MPa (5.0 ksi) after 28 days.

3.2 Pipe Piles & Internal Reinforcement
Pipe Pile
Piles shall be embedded a minimum of 5.2 m below the proposed finished grade (ie. at El.19.5 m approx.). Schedule 80 steel pipe piles with the following characteristics shall be utilized:
- Length = 7 m to 9 m,
- Size = 100 mm (113 mm Outside Diameter, 96 mm Inside Diameter),
- Alignment = 6V:1H to vertical
- Embedment into Till-like sand 1.5 m (min.) and compact native sand 1.5 m (min.)

Internal Reinforcement
Steel Flatbar reinforcement shall have minimum cross-section dimensions as follows:
- Thickness: 10 mm (3/8") minimum,
- Height: 75 mm (min.); ideally, near-equivalent to internal diameter of pipe pile (ie. 96 mm)

Prior to and during grouting of pipe pile, flat bar orientation shall be confirmed to be as shown on Figure SH-2 (ie. perpendicular to wall alignment).

3.3 Reinforcement
Welded Wire Mesh
Welded Wire Mesh shall be installed in front and behind piping. Welded Wire Mesh CSA-G 30.5M 1983 grade 400 shall be continuous across all shotcrete joints, unless noted otherwise. Minimum mesh lap shall be 2 squares. Shoring shall be two layers of 4 x 4 x 8/8 WWM

Reinforcing Steel
- Horizontal reinforcement shall consist of 20 M rebar spaced at 0.6 m vertically in front and behind pipe piles.
- Reinforcing steel shall be CSA-G 30.12M, grade 400. Minimum bar lap shall be 0.5 m.

3.4 Shotcrete
The shotcrete support membrane shall conform to the material specification of ACI 506.2-95, "Specifications for Materials, Proportioning and Application of Shotcrete", published by the American Concrete Institute. Minimum compressive strength shall be 30 MPa (4.4 ksi) at 28 days, 14 MPa (2.0 ksi) at 3 days and 7 MPa (1.0 ksi) at 24 hours. Admixtures shall only be used if specifically approved by PECI.
Shotcrete Shoring shall be a minimum of 250 mm thick and shall completely contain the specified reinforcing with cover, as noted. A minimum cover of 75 mm shall contain the outer (south) edge of the pipe pile and lateral reinforcement.

3.5 Backfill
Backfill material shall consist of 12 to 19 mm clear crushed gravel.

4.0 INSTALLATION RECOMMENDATIONS

4.1 General
The General Contractor shall carry out regular site reconnaissance around the excavation and perimeter for the express purpose of observing any signs of movement of the soil around the excavation. The General Contractor shall report any observed movement or deterioration to PECI immediately.

4.2 Shoring Extents Confirmation
The total length to be shored should be based on the most recent Civil and Landcape Architectural Design drawings. Based on the above reference drawings, the total length to be shored is currently estimated to be 25 m (82'). This length includes the tail ends of the segment consists of 8.5 m shored at the full height (ie. 2.4 m) across the alignment of the basement; 45-degree tapers would be implemented at the north and south extremities (ie. over lengths of 2.7 m at both ends).

4.3 Shoring Sequencing
The central segment of the wall requires attention to the GVRD sanitary main, which is embedded a minimum distance of 2 m from the central segment of the proposed wall. Based on our observations during initial berm placement, the temporary berm is envisioned to have limited cohesion (from the temporary support standpoint), and it is envisioned that groundwater may not be encountered. As a result, construction scheduling should tentatively be based on cutting and spraying of the full length of one to three rows that are each approximately 1.2 m high/ deep. Conventional splicing and overlaps (for horizontal reinforcing, mesh, membrane, etc) should be assumed.
The following row height parameters apply to the critical section (Section C, refer to Figure SH-5):
- upper row to be cut at 1.2 m depth/ height (max.), and
- remaining rows to be cut at 1.2 m (max.) height.

4.4 Shotcrete
Shotcrete shall be applied in a horizontal or downward vector in such a way that voids behind the reinforcing steel do not develop. Panel edges shall be cleaned prior to shotcreting adjacent panels.

4.5 Site Drainage
General
- Surface water from upslope neighbouring areas may require mitigative measures, as discussed below. Based on our site observations to-date, it is not envisioned that groundwater would be encountered.
- The General Contractor or Shoring Sub-contractor – depending on contractual obligations – is responsible for construction and maintenance of works, such as berms and swales, to collect and divert any site water via pumping or gravity flow. At the end of each day and prior to rainfall events, the site shall be graded to direct run-off away from excavation slopes and shotcrete shoring.

Control of Surface Water
Any surface flows shall be directed to a suitable sediment removal system prior to discharge to the municipally approved outlet. The interior water level shall be maintained below the level of fresh shotcrete.

Control of Groundwater
Suitable measures shall be taken, where necessary, to control groundwater flow at the excavation face during excavation and shotcrete installation. These shall include the installation of materials such as burlap and filter cloth and pea or bird eye gravel to permit drainage without loss of soil. If any erosion or loss of soil occurs, PECI shall be contacted immediately.

4.6 Soil Drainage
Drainage Membrane
Drainage membrane shall be Nudrain WD15 or approved equivalent with 300 mm (min.) lap. Laps shall be sealed with continuous taping.

Horizontal Drains
- A Multi-Flow Drain System should be installed across the base of the wall; drains should 'daylight' at the drainage bio-swale (in front of the wall).
- Horizontal drains shall be utilized to lessen any potential hydrostatic pressure behind the shotcrete, such that 1 row of 1.2 m long, 50 mm perforated PVC weep-holes are set at 2 m (o/c) at a point 1 m above the proposed excavation base. The drains should be routed to the base of the wall, for discharge, as noted above.
- Drainage pipe shall be set in drilled holes at 5° up from horizontal. The gap between horizontal drains and the installation hole at shotcrete face shall be packed with filter cloth or dry packed. Opening size, length and location of drains shall be as indicated on the drawing.

5.0 QUALITY CONTROL & TESTING

5.1 Pipe Piles
A supplier's mill certificate should be provided for pipe piles.

5.2 Shotcrete
The Sub-contractor should provide a minimum of one test shotcrete panel, or as directed by PECI. Shotcrete test panels should be 750 x 750 x 89 mm gunned in the same position as the work. Four cores should be taken from each panel and tested for compressive strength by an approved, qualified testing agency at 24 hours, 3 days and 28 days (two cores). Depending upon our observations during construction, less or more testing may be recommended by PECI.

5.3 Field Review
During construction, PECI shall be contacted by the Contractor to attend the site to observe the following aspects of construction:
a.) installation of pipe piles (length and soil recovery during drilling),
b.) prior to each excavation stage (ie. prior to Stage 1, etc),
c.) lateral reinforcement installation,
d.) shotcrete application,
e.) drainage installation,
f.) backfilling

6.0 REFERENCE DRAWINGS - PRIORITY
The Geometries and Site Data shown on Civil and Landscape Architectural Design Drawings Take Precedence Over the InformationShown in Puar Engineering's Drawings. If any Conflict/ Discrepancy is Observed Between PECI's Design Drawings and those ofCreus Engineering and/or Forma Design and Puar, it Shall be Reported Immediately to PECI.

Revisions

CLIENT: EBRAHIM
3707 Dollarton Hwy, North Vancouver, BC
Proposed Segmental Retaining Wall Systems

Reference Drawings

Road 'A' Cantilever Soldier Pile Wall
DESIGN & CONSTRUCTION RECOMMENDATIONS

Puar Engineering CONSULTANTS

Scale: n/a	Job No: 07-2-256	Date: 12Feb	FIGURE: SH-1
Rev:	Dwn: wsl	Chk: sp	

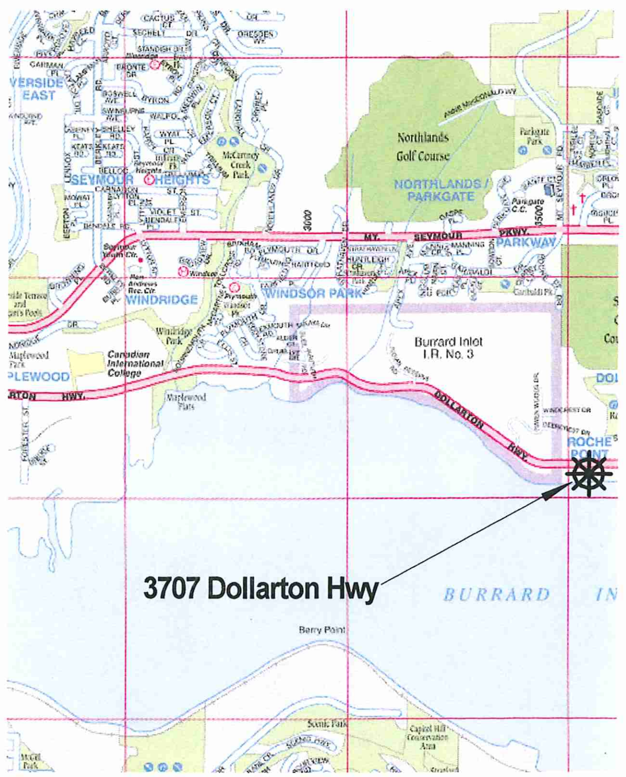
CANTILEVER SOLDIER PILE WALL DESIGN
ROAD 'A' CUL DE SAC

Project:

PROPOSED SUBDIVISION
3707 DOLLARTON HWY - N.VANCOUVER, BC

Client:

NICK EBRAHIM



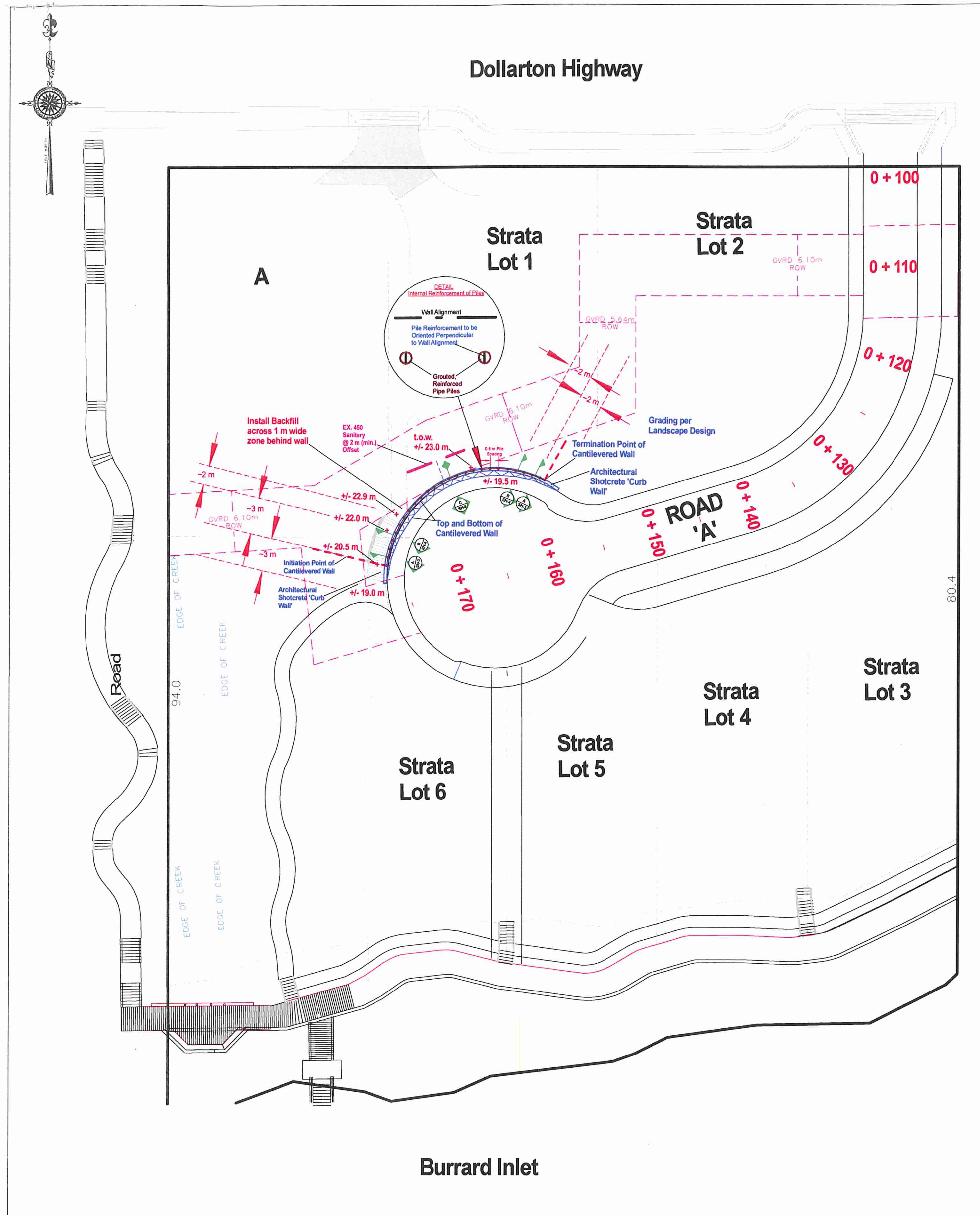
Drawing List:

Figure SH-1 - Cantilever Soldier Pile Wall Design and Construction Recommendations
Figure SH-2 - Cantilever Soldier Pile Wall Plan
Figure SH-3 - Section A - Soldier Pile Wall
Figure SH-4 - Section B - Soldier Pile Wall
Figure SH-5 - Section C - Soldier Pile Wall

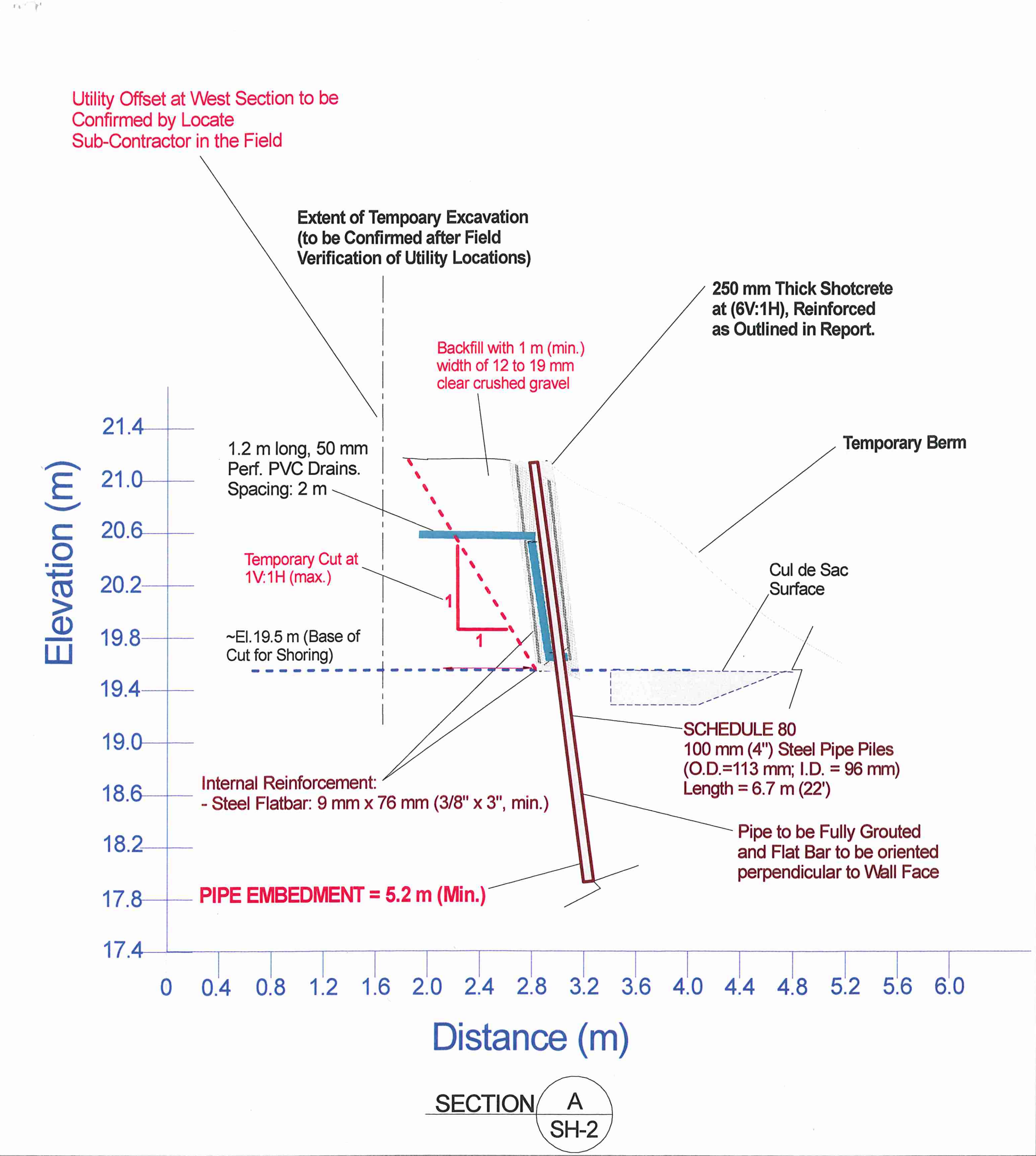
PROFESSIONAL ENGINEER

PUAR ENGINEERING CONSULTANTS INC

Puar Engineering CONSULTANTS

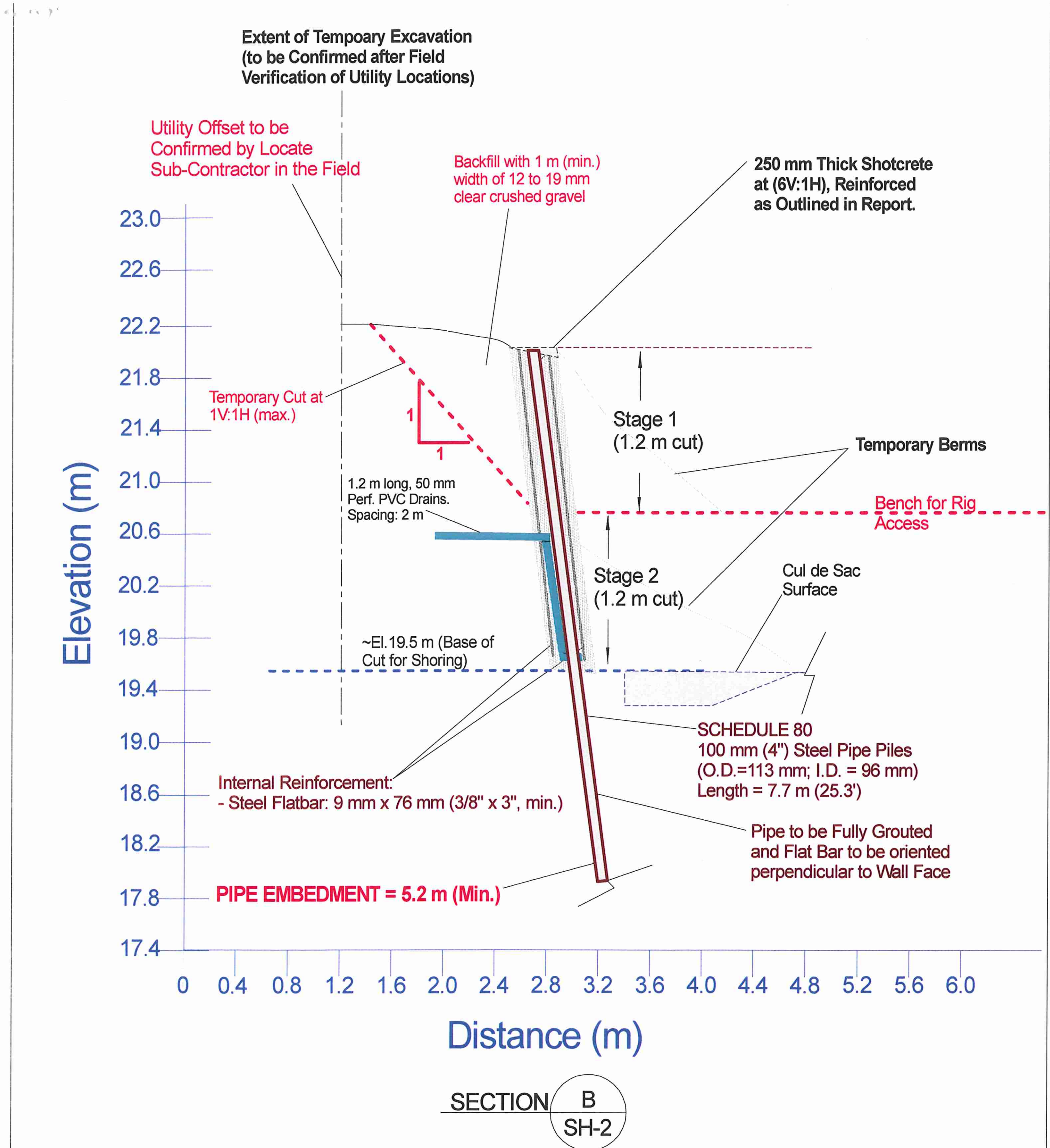


Reference Drawing Webster Engineering - Civil Site Plan Dwg. DF 8728 (emailed 11Sept21)	CLIENT: Nick Ebrahim			TITLE: PLAN: Cantilevered Wall Cul de Sac	
Puar Engineering CONSULTANTS	PROJECT: Proposed Subdivision 3707 Dollarton Hwy - North Vancouver, BC				
	PROJECT NO. 07-2-256	DFTR. wsl	DSGN. sp	DATE 12Feb	SCALE: 1:500	DWG NO. SH-2



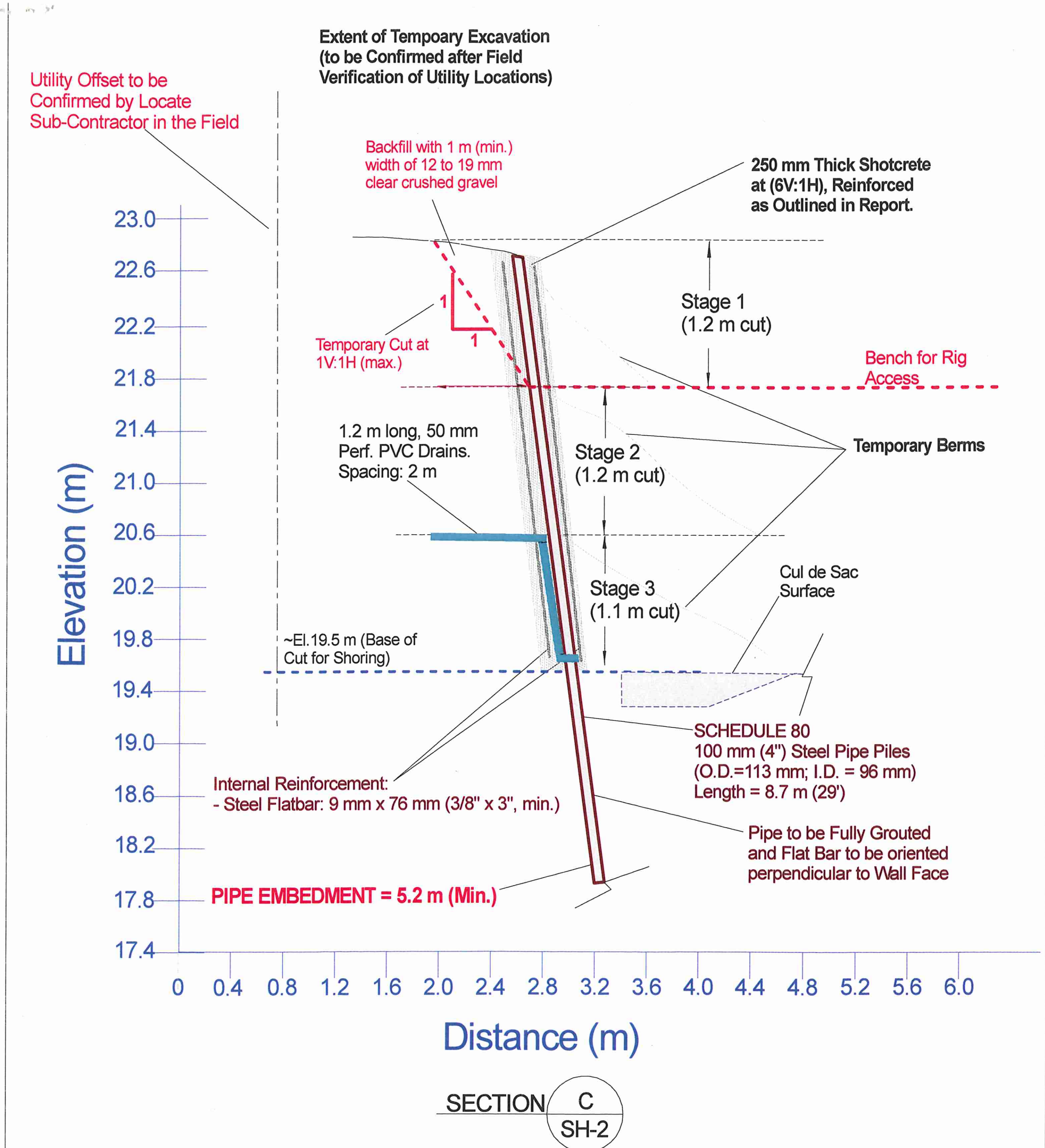
NOTES:
Refer to Figure SH-1 for construction recommendations:
- staging of rows (panelling, etc),
- lateral reinforcement scheme (in front and behind piles)

Reference Drawing	CLIENT: EBRAHIM	TITLE: SECTION A Cantilevered Soldier Pile Wall
Puar Engineering CONSULTANTS	PROJECT: PROPOSED SUBDIVISION 3707 Dollarton Hwy - N. Vancouver, BC	
	PROJECT NO. 07-2-256 / DFTR. wsl / DSGN. sp	DATE Feb/12 / SCALE: N.T.S / DWG NO. SH-3



NOTES:
Refer to Figure SH-1 for construction recommendations:
- staging of rows (panelling, etc),
- lateral reinforcement scheme (in front and behind piles)

Reference Drawing	CLIENT: EBRAHIM	TITLE: SECTION B Cantilevered Soldier Pile Wall
Puar Engineering CONSULTANTS	PROJECT: PROPOSED SUBDIVISION 3707 Dollarton Hwy - N. Vancouver, BC	
	PROJECT NO. 07-2-256 / DFTR. wsl / DSGN. sp	DATE Feb/12 / SCALE: N.T.S / DWG NO. SH-4



NOTES:
Refer to Figure SH-1 for construction recommendations:
- staging of rows (panelling, etc),
- lateral reinforcement scheme (in front and behind piles)

Reference Drawing	CLIENT: EBRAHIM			TITLE: SECTION C Cantilevered Soldier Pile Wall		
Puar Engineering CONSULTANTS	PROJECT: PROPOSED SUBDIVISION 3707 Dollarton Hwy - N. Vancouver, BC					
	PROJECT NO. 07-2-256	DFTR. wsl	DSGN. sp	DATE Feb/12	SCALE: N.T.S	DWG NO. SH-5

PUAR Engineering Consultants Inc

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

Puar Engineering
CONSULTANTS

FACSIMILE/ MAIL TRANSMISSION

Date: December 2, 2011 — **File:** 07-2-256

To: BRIAN FORTIER, P.ENG c/o
DAVID NAIRNE & ASSOCIATES
e-mail: bfortier@davidnairne.com

Phone: 604-984-3503
Fax: 604-984-0627

cc: Nick (nebrahim2001@yahoo.ca) and Teri (thodgins@lonsdalelaw.ca)

Pages To Follow: 5

RE: BEARING AND EARTH PRESSURE/ RESISTANCE PARAMETERS
PROPOSED BRIDGE FOUNDATIONS
AND REINFORCED-CONCRETE RETAINING WALLS
3707 DOLLARTON HWY - NORTH VANCOUVER, BC

1.0 TERMS OF REFERENCE

Further to our ongoing correspondence with David Nairne & Associates (DNA) and our Updated Geotechnical Investigation Report (dated September 4, 2011), the following provides preliminary geotechnical parameters for input to the structural design of the above elements. The ongoing design process can be expected to be iterative in nature. Furthermore, field review confirmation by the respective engineers-of-record will be required during construction.

2.0 SUBSURFACE CONDITIONS

During our recent test-pitting investigation (Nov.22/11), the observed soil conditions were in conformance with our subsurface data observations during the original investigation.

In the vicinity of the proposed bridge foundations, the stratigraphy generally consisted of 1 m to 1.5 m thickness of the combined UNIT 1 and 2 soil layers (ie. Topsoil and Compact Sand, respectively) overlying the dense UNIT 3 till-like sand deposit.

The following recommendations are based this estimated stratigraphy.

3.0 BEARING CAPACITY

Subject to field verification during construction, the above-described UNIT 3 undisturbed, native, dense, till-like sands are considered suitable for support of foundations, based on the Limit States bearing capacities described below.

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 approved subgrade should step at no more than 1.0 vertical to 2.0 horizontal (1V:2H).

As discussed in our Investigation Report (Sept.4/11), local subsurface water is expected to primarily perch on the till-like sand. Nonetheless, an associated factored (ie. 0.5) bearing capacity/ resistance of 180 kPa (3760 psf) at Ultimate Limit State (ULS) would be applicable to the undisturbed, till-like sand subgrade. A bearing capacity of 100 kPa (2090 psf) would be applicable under the Serviceability Limit State (SLS).

As shown in Table 1, 5% damped horizontal spectral acceleration values are defined for a range of periods for the subject property.

Table 1 – Surface Spectral Response – Local Ground Motions

Period (sec)	0.2	0.5	1.0	2.0
Spectral Acceleration* (g)	0.88	0.61	0.33	0.17

From a geotechnical stand-point, it is judged that surface spectral response would be expected to correspond most closely to Site Class C. Hence, the tabulated ground motions would not be modified (ie. F_a=1.0), and Spectral Response would correspond to the tabulated values. The locally applicable Peak Ground Acceleration (PGA) would be 0.44 g.

4.0 PROTECTION OF BRIDGE FOUNDATIONS

Scour of foundations will require consideration. Ideally, structural spans should allow foundations to be placed at elevations beyond the points on the creek banks corresponding in elevation to the 200-year level creek flows. It should be remembered that the creek flows may also be subject to increased flow levels due to higher levels of runoff as urbanization of upslope areas increases.

Even if foundation supports are placed beyond this flood elevation, protection consisting of suitably sized Rip Rap or alternative protection schemes such as "Eco-Wrap" (by Deltalok) should be implemented (due to considerations such as increased neighbourhood densification).

5.0 SLIDING & PASSIVE RESISTANCE

As discussed with DNA (B.Fortier, P.Eng), structural walls may require inclusion of a foundation 'key' element to increase resistance to sliding forces.

To mobilize the passive resistance of a soil, strain must occur at the structure/soil interface. The tabulated coefficients (in Table 2, below) correspond to allowable structural displacements. These required displacements are summarized as rotation magnitudes, which are quantified as strain as a function of the contact depth (i.e., the embedded depth of the foundation key within the undisturbed, dense, till-like sand subgrade). It is imperative that the Client contact PECI to confirm the presence of the undisturbed, till-like sand at the time of excavation. For design purposes, it is suggested that an effective unit weight of 19 kN/m^3 be used for the dense till-like sand.

Table 2: Design Passive Earth Pressure Coefficients

Allowable Foundation Element Rotation [% Strain as a function of Embedded Depth of Key]	Passive Earth Pressure Coefficient (Kp) Dense Till-like Sand (Resisting Soil Deposit)
0	1.0
2	3.0
4	4.0
6	6.0

6.0 LATERAL PRESSURES

Our lateral pressure estimates assume that backfill does not slope more than 10°. When considering limit states, it should be noted that these lateral pressures are not factored.

6.1 General

The lateral earth pressure on below-grade 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 **include hydrostatic components** to account for elevated groundwater levels. If the foundation is to be drained (eg. suitable quantity of free-draining backfill, pumping, etc), then the estimates below can be reduced by an amount corresponding to the hydrostatic pressure, which is highlighted in italic text. If it is not possible to provide continuous drainage behind the wall, hydrostatic pressure should be assumed to act over the full depth of the wall; the hydrostatic pressure is additive to the static design lateral earth pressure.

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 below).

6.2 Static Design

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 16.2 x h (kPa) (ie. 6.4 x h + *9.8 x h* triangular earth pressure distribution where 'h' is the distance from the ground surface measured in metres). In imperial units this corresponds to 103 x h (psf), where 'h' is measured in feet.

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 19.4·h (kPa) (ie. 9.6·h + *9.8·h*) triangular earth pressure distribution where 'h' is the distance from the ground surface measured in metres. In imperial units this corresponds to 124·h (psf), where 'h' is measured in feet.

6.3 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.

6.4 Base Friction

It is envisioned that sliding resistance for footings would be derived from the undisturbed, till-like sand subgrade. A friction factor of 0.35 may be applied between the concrete and sand subgrade interface.

6.5 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 + 13·h (kPa) [i.e., the sum of 16.2·h (static) and 3.2·(H-h) (dynamic)], and
- 20 kPa (ie. compaction-induced pressure).

In imperial units, this amounts to:

- 20·H + 83·h (psf) [i.e., the sum of 103·h (static) and 20·(H-h) (dynamic)], and
- 400 psf (ie. compaction-induced pressure).

and,

2.) For the Restrained Condition:

- 3.2·H + 16.2·h (kPa) [i.e., the sum of 19.4·h (static) and 3.2·(H-h) (dynamic)], and
- 20 kPa (ie. compaction-induced pressure).

In imperial units, this amounts to:

- 20·H + 104·h (psf) [i.e., the sum of 124·h (static) and 20·(H-h) (dynamic)], and
- 400 psf (ie. compaction-induced pressure).

7.0 REVIEW & CLOSURE

The design process for bridge foundations is expected to be iterative in nature. As such, the above recommendations will be refined, as further consultation with the project team takes place.

Subsurface conditions will require verification during construction in accordance with the District of North Vancouver's Letters of Assurance. Our design information is based on test-pit data, our local experience and local geological data. Field reviews will be required to confirm that soil conditions are consistent with those estimated in this document. In accordance with the District of North Vancouver's Letters of Assurance, the Geotechnical Engineer-of-Record ("G.E.R.") shall be contacted to carry out field reviews for the following items:

- temporary excavation stability,
- foundation and retaining wall subgrades,
- stability of permanent slopes,

- excavation shoring, and
- Engineered Fill selection and placement.

Please feel free to contact us, if you have any questions.

PUAR ENGINEERING CONSULTANTS INC
Per:

Surinder Puar, P.Eng.
Principal

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

Puar Engineering
CONSULTANTS

FACSIMILE/ MAIL TRANSMISSION

Date:	January 9, 2012	**File:**	07-2-256
To:	**JERRY LUM, P.ENG c/o** **DAVID NAIRNE & ASSOCIATES** **e-mail**: jlum@davidnairne.com	**Phone:** **Fax:**	604-984-3503 604-984-0627
To:	**BILL HARRISON** **FORMA DESIGN** **e-mail**: bill@formadesign.ca	**Phone:** **Fax:**	604-986-9193 604-986-7320
cc:	**FRED CIAMBRELLI, P.ENG.** **CREUS ENGINEERING** **e-mail**: fciambrelli@creus.ca	**Phone:** **Fax:**	(604) 313-2996 (604) 987-9071
cc:	Nick (nebrahim2001@yahoo.ca) and Teri (thodgins@lonsdalelaw.ca)		
RE:	**REVISION OF BRIDGE AND ACCESS POINT ELEVATIONS** **PROPOSED SUBDIVISION** **3707 DOLLARTON HWY - NORTH VANCOUVER, BC**		

1.0 TERMS OF REFERENCE

Further to Puar Engineering Consultants Inc's ("PECI") ongoing correspondence with Brian Fortier, P.Eng (of David Nairne & Associates (DNA)) and Bill Harrison (of Forma Design), the following provides suggested preliminary revisions to the bridge layout as well as the structural design concept. Further correspondence between PECI, DNA, and Forma is expected to be required.

2.0 SITE CONDITIONS AND EXISTING CONCEPT

The water channel represents a local drainage 'tributary'. The loading of the channel can be expected to increase as the upslope parts of the neighbourhood are further developed. In our opinion, it may be difficult to estimate a 200-year flood level for the channel.

Topographic profiles in line with the north and south extremities of the bridge deck (refer to Figure B-1, attached) show the primary gully through which flow occurs.

The north extremity of the bridge will govern the potential exposure of the bridge. As shown in the Figure, flooding above elevation 9.0 m should result in channel waters flowing over the crest of the gully. The capacity of the channel, therein should increase significantly (during the flood event).

The currently proposed bridge deck elevation is 7.9 m. DNA has indicated that the deck supports would extend up to 0.5 m below the deck surface. This would result in deck supports at an elevation of about 7.4 m.

The current concept consists of an approximate 17 m span between the east and west pedestrian access points to the bridge deck. Currently, the east and west access points consist of stairs extending down to a west-side landing and the east-side waterfront walkway.

3.0 RECOMMENDATIONS

It is envisioned that decreasing the bridge deck's exposure to channel flows would result in:

1.) Less lateral loading on the bridge during flooding of the channel,
2.) lighter and less invasive installation of bridge footings.

At this point, we envision that a bridge deck surface elevation of 9.7 m would significantly decrease the bridge's exposure to high water levels in the channel.

As discussed with Bill Harrison, the east and west access points would also need to be revised to account for the 1.8 m increase in elevation of the bridge deck. At this point, it appears that the proposed increase in deck elevation would result in less grade change for pedestrians access from upslope areas; a stairway would be required for pedestrians from the waterfront walkway.

We trust the above is sufficient for your current requirements. Please feel free to contact us, if you have any questions.

PUAR ENGINEERING CONSULTANTS INC
Per:

PROFESSIONAL ENGINEER
S. PUAR

Surinder Puar, P.Eng.
Principal

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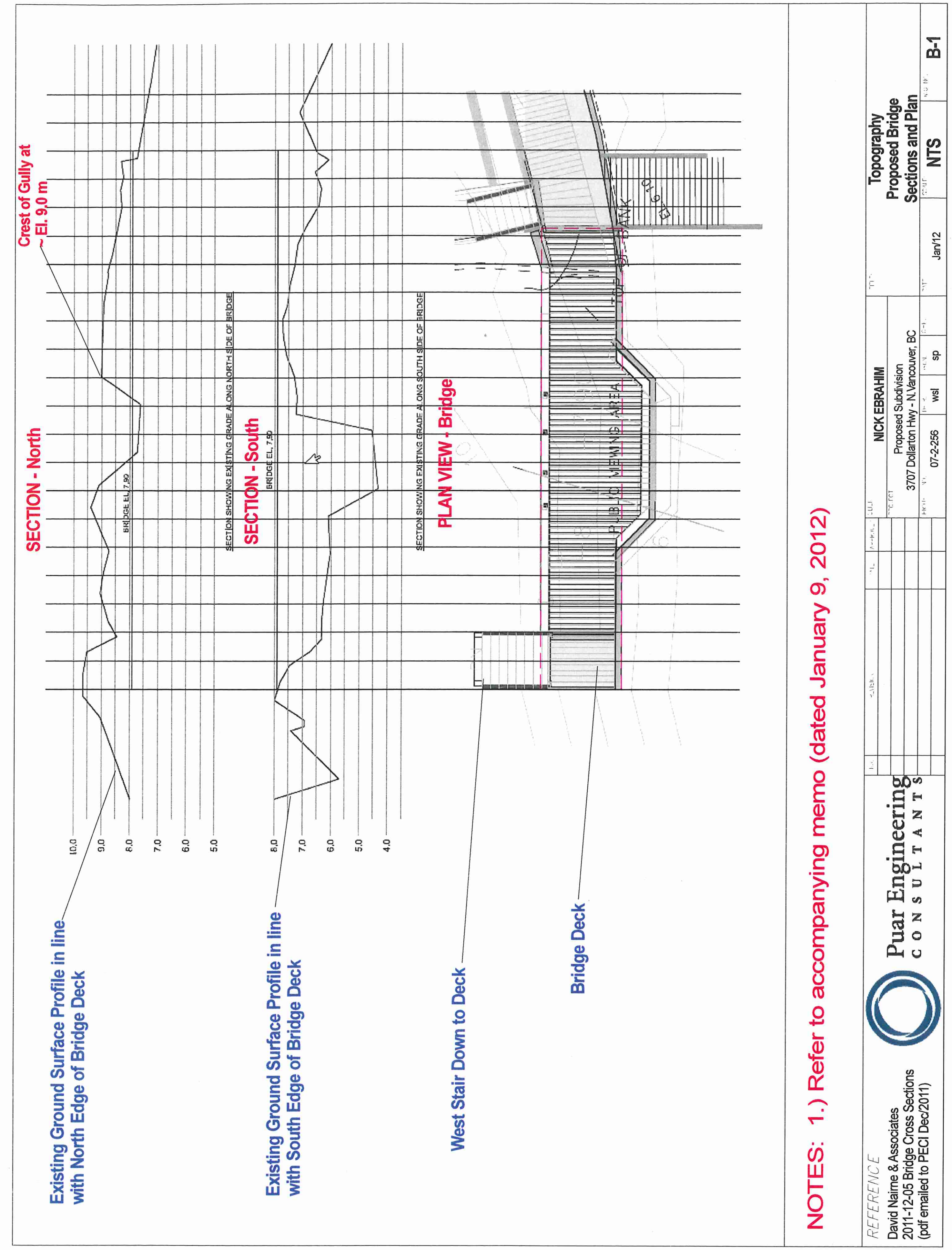

PUAR Engineering Consultants Inc

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

Puar Engineering
CONSULTANTS

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Date: January 10, 2012 **File:** 07-2-256

To: **JERRY LUM, P.ENG c/o**
DAVID NAIRNE & ASSOCIATES
e-mail: jlum@davidnairne.com
Phone: 604-984-3503
Fax: 604-984-0627

To: **BILL HARRISON**
FORMA DESIGN
e-mail: bill@formadesign.ca
Phone: 604-986-9193
Fax: 604-986-7320

cc: **FRED CIAMBRELLI, P.ENG.**
CREUS ENGINEERING
e-mail: fciambrelli@creus.ca
Phone: (604) 313-2996
Fax: (604) 987-9071

cc: Nick (nebrahim2001@yahoo.ca) and Teri (thodgins@lonsdalelaw.ca)

RE: **CONCEPTUAL DESIGN - BRIDGE**
PROPOSED SUBDIVISION
3707 DOLLARTON HWY - NORTH VANCOUVER, BC

Further to our most recent report (issued 12Jan09), the following provides additional recommendations pertaining to the structural design.

As noted in our report, the north extremity of the bridge will govern the potential exposure of the bridge. The underside of the north-most structural support would be expected to be nearest the channel flows.

We proposed a bridge deck elevation of 9.7 m. This would result in the underside of the east-west support beams being set at an elevation of about 9.2 m. As discussed with Brian and Jerry (of DNA), if the deck is designed to cantilever in the northward direction, the underside of the above-noted beam can be offset a greater amount from the channel flow.

Please feel free to contact us, if you have any questions.
PUAR ENGINEERING CONSULTANTS INC
Per:

Surinder Puar, P.Eng.
Principal
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