AGENDA ADDENDUM

REGULAR MEETING OF COUNCIL

Monday, December 9, 2013
7:00 p.m.
Council Chamber, Municipal Hall
355 West Queens Road,
North Vancouver, BC

Council Members:
Mayor Richard Walton
Councillor Roger Bassam
Councillor Robin Hicks
Councillor Mike Little
Councillor Doug MacKay-Dunn
Councillor Lisa Muri
Councillor Alan Nixon

NORTH VANCOUVER
DISTRICT

www.dnv.org

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District of North Vancouver
355 West Queens Road
North Vancouver, BC, Canada V7N 4N5
604-990-2311
www.dnv.org

REGULAR MEETING OF COUNCIL

7:00 p.m.
Monday, December 9, 2013
Council Chamber, Municipal Hall
355 West Queens Road, North Vancouver

AGENDA ADDENDUM

THE FOLLOWING LATE ITEMS ARE ADDED TO THE PUBLISHED AGENDA

9. REPORTS FROM COUNCIL OR STAFF

9.8. Remedial Action Requirements – 1576 and 1582 Merlynn Crescent
File No.

9.8.1. Remedial Action Requirements – 1576 and 1582 Merlynn Crescent

Memo: David Stuart, Chief Administrative Officer

9.8.2. Remedial Action Requirements – 1576 Merlynn Crescent: Unsafe Condition

Recommendation:
THAT

1. Council declares, pursuant to section 73 of the *Community Charter*, SBC 2003 c. 26, that the property, legally described as:

   1576 Merlynn Crescent, PID: D-9772-20, Lot 20, Block D Westlynn Plan 9772

(the “Property”) is in and creates an unsafe condition due to slope stability.

2. Council hereby imposes the following remedial action requirements (the “Remedial Action Requirements”) on Mr. Mostafa Madaninejad and Ms. Fatemeh Khosravi-Amiri the registered owners of the Property (the “Owners”) to address and remediate the above unsafe condition:

   1. Select a remediation plan option and indicate to the District in writing the selected option by January 15, 2014 and submit all necessary permit applications to the District by February 15, 2014.

   2. Complete the work in accordance with the selected remediation plan and issued permits by April 30, 2014.

3. The Owner's Qualified Professional must provide a report to the District within three weeks following completion of the work, certifying the safe condition of the slope.

4. Council hereby directs that in the case of failure of the Owner to comply with the Remedial Action Requirements, then:

   a. The District, its contractors or agents may enter the Property and may carry out the following remedial actions:

      I. Generally restore the Property to a safe condition (Option A: 1582 Remediation Plan and Option A: 1576 Remediation Plan) to the satisfaction of the Chief Building Official; and,

      II. For the foregoing purposes may retain the services of a professional engineer to provide advice and certifications;

   b. The charges incurred by the District in carrying out the aforementioned remedial actions will be recovered from the Owner as a debt; and,

   c. If the amount due to the District under 4(b) above is unpaid on December 31st in any year then the amount due shall be deemed to be property taxes in arrears under section 258 of the *Community Charter*.

9.8.3. Remedial Action Requirements – 1582 Merlynn Crescent: Unsafe Condition

Recommendation:
THAT

1. Council declares, pursuant to section 73 of the *Community Charter*, SBC 2003 c. 26, that the property, legally described as:

   1582 Merlynn Crescent, PID: D-9771-20, Lot 21, Block D Westlynn Plan 9772

(the "Property") is in and creates an unsafe condition due to slope stability.

2. Council hereby imposes the following remedial action requirements (the "Remedial Action Requirements") on Mr. William allace and Mrs. Patricia Wallace, the registered owners (the "Owners") to address and remediate the above unsafe condition:

   1. Select a remediation plan option and indicate to the District in writing the selected option by January 15, 2014 and submit all necessary permit applications to the District by February 15, 2014.

2. Complete the work in accordance with the selected remediation plan and issued permits by April 30, 2014.

3. The Owner's Qualified Professional must provide a report to the District within three weeks following completion of the work, certifying the safe condition of the slope.

4. Council hereby directs that in the case of failure of the Owner to comply with the Remedial Action Requirements, then:

   a. The District, its contractors or agents may enter the Property and may carry out the following remedial actions:

      I. Generally restore the Property to a safe condition (Option A: 1576 Remediation Plan and Option A: 1576 Remediation Plan) to the satisfaction of the Chief Building Official; and,

      II. For the foregoing purposes may retain the services of a professional engineer to provide advice and certifications;

   b. The charges incurred by the District in carrying out the aforementioned remedial actions will be recovered from the Owner as a debt; and,

   c. If the amount due to the District under 4(b) above is unpaid on December 31st in any year then the amount due shall be deemed to be property taxes in arrears under section 258 of the *Community Charter*.

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NORTH VANCOUVER
DISTRICT

Memo

December 6, 2013
File:

TO: Mayor Richard Walton and Council

FROM: David Stuart, Chief Administrative Officer

SUBJECT: Remedial Action Requirements - 1576 and 1582 Merlyn Crescent

In the reports from Engineering regarding the above, staff have not made any reference to other options with respect to financing the works. The owners in both instances are elderly and appear to be on fixed incomes with limited capacity to finance the works. Finance and legal staff will be present when these items are considered should Council wish to explore financing options involving the District.

David Stuart

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AGENDA INFORMATION

❒ Regular Meeting Date:____________
❒ Workshop (open to public) Date:____________

Dept. Manager	GM/ Director	CAO

The District of North Vancouver
REPORT TO COUNCIL

December 5, 2013
File:

AUTHOR: Michelle Weston

SUBJECT: **Remedial Action Requirements - 1576 Merlynn Crescent: Unsafe Condition**

RECOMMENDATION:

That Council pass the following Resolutions:

1. Council declares, pursuant to section 73 of the *Community Charter*, SBC 2003 c. 26, that the property, legally described as:

   1576 Merlynn Crescent, PID: D-9772-20, Lot 20 Block D Westlynn Plan 9772

   (the "Property") is in and creates an unsafe condition due to slope stability.

2. Council hereby imposes the following remedial action requirements (the "Remedial Action Requirements") on, Mr. Mostafa Madaninejad and Ms. Fatemeh Khosravi-Amiri the registered owners (the "Owners") to address and remediate the above unsafe condition:

   1. Select a remediation plan option and indicate to the District in writing the selected option by January 15, 2014 and submit all necessary permit applications to the District by February 15, 2014.

   2. Complete the work in accordance with the selected remediation plan and issued permits by April 30, 2014.

   3. The Owner's Qualified Professional must provide a report to the District within 3 weeks following completion of the work, certifying the safe condition of the slope.

   4. Council hereby directs that in the case of failure of the Owner to comply with the Remedial Action Requirements, then:

a. the District, its contractors or agents may enter the Property and may carry out the following remedial actions:

   i. generally restore the Property to a safe condition (Option A: 1582 Remediation Plan and Option A: 1576 Remediation Plan) to the satisfaction of the Chief Building Official; and

   ii. for the foregoing purposes may retain the services of a professional engineer to provide advice and certifications;

b. the charges incurred by the District in carrying out the aforementioned remedial actions will be recovered from the Owner as a debt; and

c. if the amount due to the District under 4(b) above is unpaid on December 31st in any year then the amount due shall be deemed to be property taxes in arrears under section 258 of the *Community Charter*.

REASON FOR REPORT:

To address an unsafe condition related to slope stability on the property of 1576 Merlynn Crescent by ordering remedial action requirements to restore the slope to a safe condition to mitigate landslide risk.

BACKGROUND:

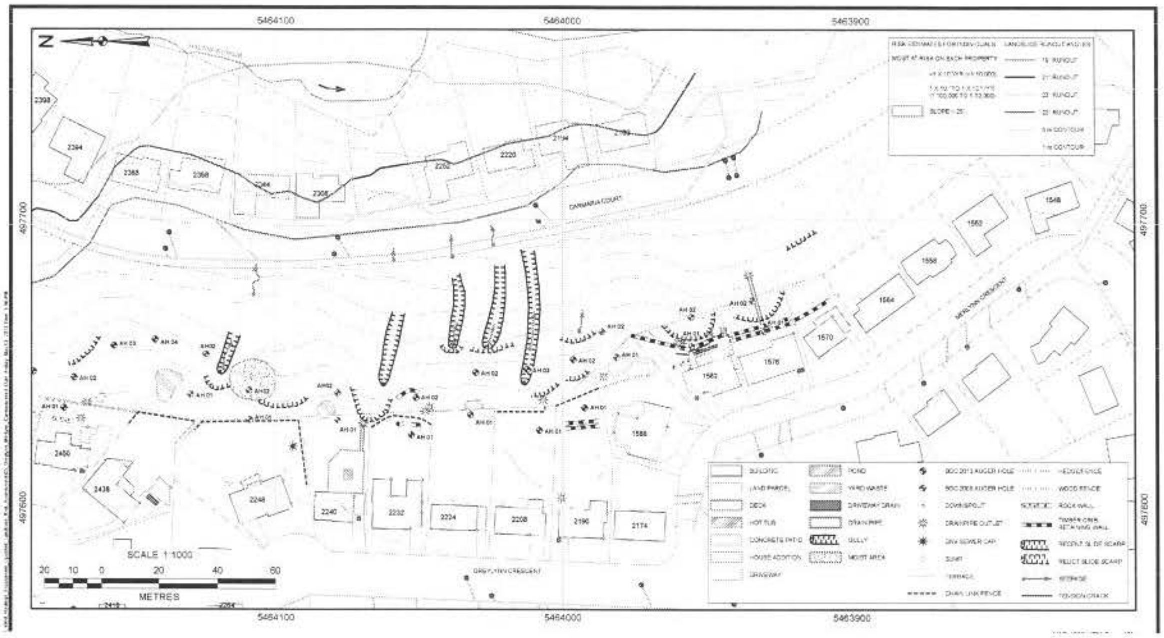
The District's adopted landside risk tolerance for existing development is 1:10,000 for Tolerable properties and 1:100,000 for Broadly Acceptable properties. The District has approximately 110 properties where landslide risks meet existing development but exceed the criteria for new development.

1576 Merlynn Crescent was rated as Tolerable during the 2008 Landslide Risk Assessment. The District retained Horizon Engineering to evaluate the slope condition of the property in 2013 and other adjacent properties of the crest of the escarpment. Horizon Engineering rated the Landslide Hazard Likelihood rating as High and Qualitative Risk Rating as Very High for 1576 Merlynn Crescent (Attachment A). The Property was reevaluated in a Quantitative Risk Assessment by BGC Engineering in 2013. According to the District risk criteria, the property still falls within the Tolerable range as the landslide runout path is predicted to impact Carmaria Court Road and Utilities infrastructure and not a home. The landslide risk potential for loss of life is limited to the potential for the landslide to impact one of the Carmaria Court residents driving a car on the road. Nine homes are accessed from Carmaria Court and would be inaccessible if a landslide blocks the road. The District staff have requested the Owners to mitigate the risk of landslide based on the potential of the landslide impacting the road and causing potential injury to drivers on the road. Engineering staff and BGC

Engineering met with Carmaria Court homeowners on May 23, 2013 to discuss and disclose the landslide risk.

This property is not connected to the storm network. The District has developed cost estimates and a rear yard option to provide storm connections to the properties along Merlynn Crescent at the crest of the slope. The District will continue to work with homeowners on the crest of the slope to obtain access for a rear yard storm connection in 2014.

Location of Properties



Quantiative Risk Assessment BGC 2013

Both geotechnical consultants retained by the District provided the same recommendation of removing the fill load and the removal/replacement of the retaining walls on the property for landslide mitigation.

The Owners were provided copies of geotechnical reports relating to the slope stability of the property on May 23, 2013 and met with BGC Engineering and District Staff to interpret reports. At that time the property owners were requested to voluntarily:

"Submit a plan, prepared by a Qualified Professional retained by you, to address and remediate the unsafe slope stability condition by removing backyard fill and the retaining wall on the Property (the "Remediation Plan"), acceptable to the District's General Manager, Parks and Engineering Services, (the "General Manager"), by no later than August 1, 2013; and,

Perform the remedial work required by the Remediation Plan. This work must be commenced within 30 days of the approval of the Remediation Plan by the General Manager and must be completed in accordance with the Remediation Plan and to the satisfaction of the General Manager by no later than October 15, 2013."

The Owners complied with this request and retained Horizon Engineering to develop remediation plan. The District received the remediation plan (Attachment B) on November 15, 2013 and notified the homeowners that all of the presented options were acceptable with Option A meeting the remediation requirements.

ANALYSIS:

The landslide risk to residents using Carmaria Court road creates an unsafe condition. The remediation order is needed to insure that the risk of landslide impacting the road is mitigated.

The Owners are currently obtaining price estimates from contractors on the scope of work for each remediation plan option. The cost of the remediation to each property is estimated to start at $75,000-$100,000 based on the amount of fill needed to be removed from the slope and the difficulty of access to the rear yards. The Owners have indicated limited financial ability to be able to fund the remediation needed on the Property.

An alternative of a debris fence being constructed at the base of the slope was explored. Preliminary cost estimates to design and install the fence start at $150,000. Installation of a fence would not stop the impending landslide from occurring and clean-up costs would be additional once the landslide occurred.

EXISTING POLICY:

Section 72 of the Community Charter authorizes local governments to impose "remedial action requirements" with respect to hazardous conditions and declared nuisances. Council can require a person to remove, demolish, alter, or otherwise deal with the matter in accordance with the directions of Council or a person authorized by Council.

Section 73 of the Charter specifically authorizes local councils to impose a remedial action requirement where council considers a "matter or thing is in or creates an unsafe condition or the matter or thing contravenes the provincial building regulations or a bylaw under section 8(3)(1) of Division 8 [building regulation] of this Part."

The resolution imposing a remedial action requirement must specify a time by which the required action must be taken which must be at least 30 days after notice of the order is sent. If the person wishes to appeal, they have 14 days to request reconsideration by Council.

If the remedial action requirements are not completed within the time permitted, the District can complete the requirements at the expense of the property owner (per s. 17 of the Charter). If the costs are unpaid at the end of the year, they may be added to the property taxes (s. 258).

Timing/Approval Process:

The District has requested the homeowners notify the District of a decision on which alternative is chosen by January 15, 2014. The Community Charter requires that the deadline cannot be earlier than 30 days after the notice of the remedial action requirements is sent to the owner. The work should be completed by April 30, 2014.

Concurrence:

The Municipal Solicitor has reviewed the recommendations.

Financial Impacts:

In the case of default, the District may undertake the remedial action requirements at the expense of the owner and recover the costs as a debt (s. 17 of the Charter). If the debt remains unpaid on December 31, the amount may be added to the property taxes (s. 258 of the Charter).

The homeowners, as seniors have indicated a limited financial ability to carry out the remediation. In recognition of the financial limitations of the homeowners, the District has provided $2,000 in geotechnical assistance towards development of the remediation plan, has waived permit fees and is providing a location to dump fill for the remediation. The District has offered to tarp the property to lessen the risk of landslide prior to the remediation. This offer has not been accepted by the homeowners of 1576 Merlynn Crescent.

Conclusion:

A remedial action order is required from Council to ensure that remediation to mitigate landslide risk is addressed.

Michelle Weston
Section Manager, Public Safety

REVIEWED WITH:					
❑ Sustainable Community Dev.	____	❑ Clerk's Office	____	External Agencies:	
❑ Development Services	____	❑ Communications	____	❑ Library Board	____
❑ Utilities	____	❑ Finance	____	❑ NS Health	____
❑ Engineering Operations	____	❑ Fire Services	____	❑ RCMP	____
❑ Parks & Environment		❑ ITS		❑ Recreation Com.	

❑ Economic Development	____	❑ Solicitor	____	❑ Museum & Arch.	____
❑ Human resources	____	❑ GIS	____	❑ Other:	____

Attachment A

HORIZON ENGINEERING INC

Unit 1
2433 |
North Vancouver, BC
Canada V7H 0A1

Fax 604-990-0583
www.horizoneng.ca

April 4, 2013

Our File: 112-3072

DISTRICT OF NORTH VANCOUVER
355 West Queens Road
North Vancouver, BC V7N 4N5

Attention: Michelle Weston

Re: Slope Stability Assessment
West Hastings Escarpment, North Vancouver, BC
Geotechnical Investigation Report

1.0 INTRODUCTION

This document reports on the results of the geotechnical assessment carried out at the West Hastings Escarpment in North Vancouver and provides geotechnical comments and recommendations regarding slope stability. The scope of this assessment included a general site reconnaissance, subsequent detailed site investigation at three areas of concern, slope stability analyses of these selected areas, and preliminary runout analyses and risk assessment. This report is prepared in conformance with our proposed scope of services, dated May 4, 2012. Authorization to proceed was received on May 11, 2012. Subsequently, the scope of services was increased to include more detailed runout analyses and risk assessment for selected properties located at the toe of the subject slope.

2.0 SITE DESCRIPTION

The West Hastings Escarpment is located in the Westlynn Terrace area of North Vancouver, as shown on Figure 1 (attached following the text of this document) and is approximately 500 metres (1,600 feet) in length (north-south) and approximately 40 to 60 metres (130 to 200 feet) in width (east-west). This area is bounded by residential properties off of Merlynn Crescent, Greylynn Crescent, and Lauralynn Drive to the west, Carmaria Court with residential properties and Hastings Creek beyond to the east, and residential developments to the north and south. This area is also known as Hastings Park and is currently undeveloped and forested.

Topography within the park generally slopes down from west to east and comprises moderate to steep upper slopes and gentle to moderate lower slopes, with an existing Lock Block retaining wall that retains a road cut on the west side of Carmaria Court at the south portion of the site. Topography west of the site is generally flat and sloping gently down to the south, while topography east of the site is generally flat to gently sloping down to the south across Carmaria Court and adjacent building areas and then moderately to steeply sloping down to Hastings Creek.

At the times of our site visits, the properties at the crest of the West Hastings Escarpment were generally developed with one to two storey houses at the central portion of the sites. The west portions of the properties were generally developed with both soft and hard landscaping. The back yard areas situated at the east portions of these properties were typically developed with soft landscaping from the houses to the slope crest, with the balance consisting of undeveloped forested terrain. Some properties were noted to have wood retaining walls near the crest of the slope. The properties at the crest of the West Hastings Escarpment slope that were reviewed as part of the current assessment include:

- 1552, 1558, 1564, 1570, 1576, 1582, and 1588 M erlynn Crescent,
- 2190, 2208, 2224, 2232, 2240, and 2248 Grey lynn Crescent, and
- 2438, 2450, 2474, 2486, 2498, 2510, 2526, 2542, 2558, 2574, 2590, and 2602 Lauralynn Drive.

At the toe of the slope, all properties on Carmaria Court (i.e., 2180 through 2424 Carmaria Court) were considered with respect to the effects of upslope conditions.

3.0 BACKGROUND INFORMATION

3.1 Reference Documents

We have read and interpreted the following reports that were provided to us for relevant background information:

- 'Westlynn and Pemberton Heights Escarpments: Preliminary Landslide Hazard Assessment' report prepared by BGC Engineering Inc., dated Novem ber 29, 2007
- 'District of North Vancouver: 2009 Landslide Risk Assessment For Select Escarpment Slopes' report prepared by BGC Engineering Inc., dated January 4, 2010
- 'District of North Vancouver: Landslide Risk Summary' report prepared by BGC Engineering Inc., dated November 12, 2010

Based on the above published information by BGC Engineering, the properties at the crest of the Hastings Park slopes for which a landslide hazard is identified are understood to have previously assessed risk levels of "Broadly Acceptable" (i.e., 1588 Merlynn Crescent, 2240 and 2448 Greylynn Crescent, and 2438, 2558, 2574, 2590, and 2602 Lauralynn Drive) or "Tolerable" (i.e., 1576 and 1582 Merlynn Crescent) per the District of North Vancouver's Risk Tolerance Criteria.

It should be noted that Horizon Engineering has previously issued the following documents pertaining to properties that are w ithin the subject site:

- Geotechnical Comments - Proposed Foundation Drainage Discharge at 2498 Lauralynn Drive, North Vancouver, BC - Site Reconnaissance July 6, 2012 (dated July 11, 2012,
- Geotechnical Comments - Linear Ground Depressions at 1582 Merlynn Crescent, North Vancouver, BC (dated April 27, 2012),
- Geotechnical Comments - Slope Stability Reconnaissance at 1570, 1576, and 1588 Merlynn Crescent, North Vancouver, BC (dated May 22, 2012), and

- Geotechnical Investigation Report - Landslide Investigation and Remediation at 2248 Greylynn Crescent, North Vancouver, BC (dated May 24, 2008, which pertains to a landslide caused by an upslope water main break).

The District of North Vancouver's online GeoWeb Geographical Information System was referenced to obtain aerial photos, building footprint locations, and topographic contours, the latter of which is understood to be based on aerial LiDaR (Light Detection and Ranging) mapping. Survey data collected by the District of North Vancouver in March, 2013 was also referenced, as described in Section 6.3.

3.2 Geological Survey of Canada

Based on information provided by the Geological Survey of Canada, the subsurface materials at the subject site are expected to be Capilano Sediments, comprising "raised deltaic and channel fill medium sand to cobble gravel up to 15 metres thick deposited by proglacial streams and commonly underlain by silty to silty clay loam" (Geological Survey of Canada: Surficial Geology of Vancouver, Map 1486A). These expected soil conditions have been previously observed in the general vicinity of the subject site and have generally been found to be in a dense to very dense / very stiff to hard state.

3.3 Seismic Hazard Calculation

Based on published information in the 2012 edition of the British Columbia Building Code (Division B - Appendix C), seismic events with 2% and 10% probabilities of exceedance in 50 years for the subject site would have peak ground accelerations of 0.429g and 0.226g, respectively, where g is the gravitational acceleration. This peak ground acceleration is for firm ground conditions and assumed to have no vertical acceleration component. The published 5% damped horizontal spectral acceleration values for North Vancouver for different natural periods associated with the aforementioned peak ground accelerations are presented in T able 1.

Table 1: 2012 BCBC Design Ground Motions

Probability of Exceedance in 50 Years	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)
2%	0.866	0.603	0.322	0.169
10%	0.456	0.314	0.166	0.085

3.4 District of North Vancouver

Based on the District of North Vancouver's online GeoWeb Geographical Information System, the houses on the subject properties were constructed between 1958 and 1978 (85% were constructed in 1958). The only property for which a storm sewer connection is listed or shown is 1582 Merlynn Crescent; the balance of the properties are not listed as being connected to the municipal storm sewer, which is shown graphically to exist on Merlynn Crescent.

None of the subject properties west of the West Hastings Escarpment are identified as being within Natural Environment, Creek Hazard, or Streamside Protection Development Permit Areas; however, the east portions of some of these properties are identified as being within a Slope Hazard Development Permit Area.

4.0 SITE INVESTIGATIONS

4.1 Previous Site Investigations

4.1.1 Geotechnical Reconnaissances at 2248 Greylynn Crescent

On April 12, 2006, Ms Karen Savage, P.Eng. and Mr Robert Ng, P.Eng. of Horizon Engineering attended 2248 Greylynn Crescent and the adjacent slope, accompanied by Mr Ariel Estrada, P.Eng. of the District of North Vancouver. This initial site visit was carried out in order to provide recommendations pertaining to public safety following a landslide that occurred on April 9, 2006, which was caused by an upslope water main break. A subsequent site reconnaissance was carried out on April 20, 2006 by the above Horizon engineers to collect landslide geometry measurements, observe surficial soil characteristics, and develop remediation strategies.

4.1.2 Geotechnical Reconnaissance at 1582 Merlynn Crescent

On April 27, 2012, Mr Robert Ng, P.Eng. of Horizon Engineering attended 1582 Merlynn Crescent to carry out a geotechnical reconnaissance to make observations and collect measurements pertaining to linear ground depressions that were reported at the site. A reconnaissance of the upper slope adjacent to the property was also carried out during this site visit.

4.1.3 Foundation Drainage Reconnaissance at 2498 Lauralynn Drive

On July 6, 2012, Mr Robert Ng, P.Eng. and Ms Pamela Bayntun, P.Eng. of Horizon Engineering attended 2498 Lauralynn Drive to carry out a geotechnical reconnaissance with regard to foundation drainage discharge near the subject slope crest. Observations of topography, surficial soil conditions, erosion, slope stability, and existing drainage conditions were collected during the site visit.

4.1.4 Slope Stability Reconnaissance at 1570, 1576, and 1588 Merlynn Crescent

On April 27, 2012, Mr Robert Ng, P.Eng. and Ms Pamela Bayntun, P.Eng. of Horizon Engineering attended 1570, 1576, and 1588 Merlynn Crescent to carry out a geotechnical reconnaissance with regard to slope stability. Observations of topography, surficial settlement, surficial soil conditions, and slope stability were collected during the site visit. A reconnaissance of the upper slope adjacent to the properties was also carried out during the site visit.

4.2 Geotechnical Reconnaissance

On May 9, 2012, Ms Pamela Bayntun, P.Eng. of Horizon Engineering attended the subject site to carry out a geotechnical reconnaissance and to carry out a peer review of the concurrent geomorphological site investigation. The portions of the accessible subject properties at the crest of the West Hastings Escarpment were assessed, and observations of topography, slope stability conditions, drainage, and groundwater seepage were made. Several properties were inaccessible; however, observations from adjacent properties were made wherever possible. A subsequent geotechnical reconnaissance was carried out on January 24, 2013 by Ms Pamela Bayntun and Ms Karen Savage, P.Eng. of Horizon Engineering to 'ground truth' preliminary results of the slope stability analyses.

After issuing a draft version of this report, our scope of services was increased as described in Section 6.3. The increased scope warranted an additional geotechnical and geomorphological reconnaissance to refine the landslide hazards at the site, which was carried out on March 13, 2013 by Ms Pamela Bayntun, P.Eng. of Horizon Engineering and Mr Pierre Friele, M.Sc., P.Geo. of Cordilleran Geoscience.

4.3 Geomorphological Site Investigation

In order to obtain an understanding of the potential natural hazards at the subject site, a geomorphological site investigation was carried out concurrently with the May 9, 2012 geotechnical reconnaissance by Mr Pierre Friele, M.Sc., P.Geo. of Cordilleran Geoscience. This involved conducting traverses of the sloping terrain within the site and providing peer review to aspects of the geotechnical assessment. As described above, Mr Friele re-attended the site on March 13, 2013 to refine the landslide hazards at the site.

4.4 Subsurface Investigations

During the geotechnical reconnaissances and the geomorphological site investigation, multiple suspected active or ancient landslide scarps were identified within the subject site at three main locations, as shown on Figure 2 and as further described in Section 4.5 below. The first suspected landslide scarp intersects 1564 to 1582 Merlynn Crescent, the second intersects 2190 and 2208 Greylynn Crescent, and the third intersects 2574 to 2590 Lauralynn Drive. These three areas of concern were the focus of the subsurface investigations described below, as well as subsequent slope stability analyses, which are described in Section 5.0. It should be noted that the suspected ancient landslide scarp shown on Figure 2 intersecting 1552 and 1558 Merlynn Crescent appeared to be inactive and had been previously stabilized by retention at the toe of the slope; therefore, further analysis of this area was not judged to be required at this tim e.

4.4.1 WildCat Dynamic Cone Penetration Tests

On July 24, 2012 Mr Adam Jessop of Horizon Engineering and Mr Ben Tam of HE Testing attended the subject site to carry out the first portion of the subsurface investigation. One WildCat Dynamic Cone Penetration Test, labelled WCT12-1, was advanced at the east portion of 1576 Merlynn Crescent. On August 1, 2012 Mr Adam Jessop and Ms Alisa Andreeva of Horizon Engineering attended the subject site to carry out the second portion of the subsurface investigation. One WildCat Dynamic Cone Penetration Test, labelled WCT12-2, was advanced at the east portion of 2190 Greylynn Crescent, while a second WildCat Dynamic Cone Penetration Test, labelled WCT12-3, was advanced at the central portion of 2574 Lauralynn Crescent. WildCat Dynamic Cone Penetration Tests were advanced to depths of approximately 0.8 to 5.1 metres (2 feet 7 inches to 16 feet 9 inches) below existing grades.

Based on the WildCat DCPT sounding data, the compactness of the subsurface materials at these locations was determined ed to be:

- WCT12-1

0 - 3.0 metres (0 - 9 feet 10 inches) depth	very loose to loose
3.0 - 5.0 metres (9 feet 10 inches - 16 feet 5 inches)	loose to compact
5.0 - 5.1 metres (16 feet 5 inches - 16 feet 9 inches)	dense to very dense

- WCT12-2

0 - 0.2 metre (0 - 8 inches) depth	very loose to loose
0.2 - 0.4 metre (8 inches - 1 foot 4 inches)	compact
0.4 - 0.8 metre (1 foot 4 inches - 8 feet 7 inches)	dense to very dense

- WCT12-3

0 - 0.9 metre (0 - 3 feet) depth	very loose
0.9 - 1.0 metre (3 feet - 3 feet 3 inches)	compact
1.0 - 1.1 metre (3 feet 3 inches - 3 feet 7 inches)	very dense

WildCat test hole locations are approximately shown on Figure 2 and detailed descriptions of the inferred soil compactness encountered at the WildCat penetration test locations are provided on the attached logs. This investigation was to have included manually-excavated test pits but was curtailed due to the presence of a bear.

4.4.2 Test Pits

On January 10, 2013, Ms Alisa Andreeva and Mr Clive Clarke of HE Testing attended the subject site to carry out the third and final portion of the subsurface investigation. Three manually excavated test pits, labelled TP13-1 through TP13-3, were advanced on the sloping terrain east of 2190 Merlynn Crescent to depths of approximately 0.9 to 1.4 metre (3 feet to 4 feet 7 inches) below existing grades. Test pit locations are approximately shown on Figure 2.

The soil stratigraphy encountered at the test pit locations was found to comprise:

- TP13-1

0 - 0.5 metre (0 - 1 foot 9 inches) depth	topsoil
0.5 - 1.0 metre (1 foot 9 inches - 3 feet 4 inches)	sandy silt
1.0 - 1.1 metre (3 feet 4 inches - 3 feet 6 inches)	sand

- TP13-2

0 - 0.3 metre (0 - 1 foot) depth	topsoil
0.3 - 1.4 metre (1 foot - 4 feet 6 inches)	sandy silt to silty sand
1.4 - 1.5 metre (4 feet 6 inches - 4 feet 7 inches)	sand

- TP13-3

0 - 0.2 metre (0 - 6 inches) depth	topsoil
0.2 - 0.9 metre (6 inches - 3 feet)	sandy silt

The silty sand to sandy silt was observed to be reddish brown and was inferred to be compact / stiff. The sand was observed to be grey, fine to medium grained, and was inferred to be very dense.

Detailed descriptions of the soil encountered at the test pit locations are provided on the attached logs.

It is noteworthy that the unweathered soil exposed in a landslide scar in 2006 (described in Section 7.3.2) was observed to comprise glacial till-like sand that was inferred to be very dense.

4.5 Slope Assessment

A visual assessment of the ground conditions on the sloping terrain within the subject site was carried out in an effort to identify any ancient, existing, or potential slope stability problems.

Anthropogenic topographic alterations that were observed to have affected the slope include filling at the east portions of properties both at the slope crest and at Carmaria Court near the slope toe, as well as excavation at the Carmaria Court road cut. In addition, a Lock Block retaining wall was observed immediately west of Carmaria Court at the south portion of the subject site, which retains the road cut and which we understand was constructed in 1996 to stabilize a shallow slope failure on the slope to the west. It was also noted that the slope located east of 2248 Greylynn Crescent that was remediated following the 2006 landslide event (caused by a District of North Vancouver water main failure) had been revegetated, and no further signs of slope instability were noted in this area.

During the geotechnical reconnaissance of the east portions of the properties at the crest of the subject slope and the adjacent District of North Vancouver property to the east, multiple signs of slope movement were observed, as shown in the photographs provided on Figures 3 through 8. These signs included tension cracks and bulging and failing of existing retaining walls (Photographs 1 and 2 on Figure 3, respectively). In addition, linear topographic features were noted, which may correspond to either ancient landslide scarps and / or anthropogenic landscaping features (Photographs 3 and 4 on Figure 4, respectively). Also, pistol butted trees, ground settlement, and a recent landslide scar (estimated to be approximately one to two years old) were observed at the locations shown on Figure 2. Although detailed reconnaissance of each house at the crest of the slope was beyond the current scope, no obvious signs that would indicate movement of the subject houses were noted, including noticeable exterior cracking, noticeable foundation settlement, or signs of slope instability immediately adjacent to the west sides of the houses.

Significant fill materials were observed to be present near the crest of the slope at many of the subject properties, as indicated on Figure 2. At some locations, retaining walls or large stumps at the crest of the slope retained fill materials (Photograph 5 on Figure 5), and yard waste was observed at many locations to be dumped at or over the crest of the slope (Photograph 6 on Figure 5). Household debris was also observed at several locations to be dumped at or over the crest of the slope. The expected presence of fill materials at the crest of the slope is supported by the observation of loose to very loose soil within the upper portions of WildCat Penetration Test holes, as well as by the observation of local oversteepening of the slope at the slope crest. Using handheld equipment, the gradient of the upper slope was measured to vary from about 26 to 39 degrees, and locally as steep as approximately 53°.

Multiple first growth stumps (expected to be of the order of 500 years old) were observed to be present on the subject slope, including at some areas of the upper, middle, and lower portions of the slope (Photographs 5 and 12 on Figures 5 and 8, respectively). Some of these stumps were observed to be decomposing, and at least one stump located below 1576 Merlynn Crescent was observed to be lying on its side, which suggests that it may have been pushed over the crest of the slope during original site preparation (Photograph 11 on Figure 8).

4.6 Surface and Groundwater Conditions

During the geotechnical reconnaissance, drain pipes were observed at nine properties located at the crest of the slope, which were directed onto the upper portion of the sloping terrain or onto the portions of the properties located immediately west of the slope crest. These properties include (but are not limited to) the following:

- 1582 Merlynn Crescent,
- 1588 Merlynn Crescent,
- 2208 Greylynn Crescent,
- 2224 Greylynn Crescent,
- 2240 Greylynn Crescent,
- 2486 Lauralynn Drive,
- 2498 Lauralynn Drive (downspouts and foundation drainage),
- 2510 Lauralynn Drive (downspouts), and
- 2526 Lauralynn Drive.

Observations were limited by dense vegetation. These drain pipes included 'Big O' or PVC type drain pipes and ceramic drain tiles that are envisaged to provide drainage for foundations, landscaping, and retaining walls (Photograph 7 on Figure 6). Several properties were observed to be directing rainwater downspouts onto the ground (Photograph 8 on Figure 6), and landscaping water features were observed to be located at the crest of the slope at 2526 and 2558 Lauralynn Drive (Photograph 9 on Figure 7). No signs of erosion or concentrated water flow were observed in these areas. The only evidence of concentrated surface water flows were observed downslope of 2248 Greylynn Crescent and 2602 Lauralynn Drive, where we understand that upslope water main breaks in recent years resulted in erosion of the subject slope.

At the times of our site investigations, no groundwater discharge was observed on the upper portions of the subject slope with the exception of minor seepage observed at the slope break located downslope of 2542 Lauralynn Drive. However, significant groundwater discharge was observed on May 9, 2012 during the geotechnical reconnaissance at the toe of the slope immediately west of Carmaria Court and particularly at the north portion of the slope, as shown on Figure 2 and Photograph 10 on Figure 7.

Moist soil conditions were generally observed within the surficial soil; however, seepage was observed at a depth of 1.1 metre (3.5 feet) below existing grade at the location of test pit TP13-2. It is envisaged that the groundwater table is located within the near-surface materials and may be perched on the dense to very dense sand materials as described in Section 4.4.

5.0 SLOPE STABILITY ANALYSIS

5.1 General

A commercially available limit equilibrium slope stability analysis program (XStabl, version 5.204) was used to carry out the analyses for the selected slope profiles under both static and design seismic ground conditions. A Bishop's method of analysis was used to search for the most critical potential circular failure surfaces that could influence the modelled portions of the subject slope.

For the purpose of communicating the comparative stability of a slope, a Factor of Safety may be determined for a given slope condition. A Factor of Safety is based on the ratio of resisting forces to driving forces, where the resisting forces help to stabilize a slope and the driving forces contribute to instability. A Factor of Safety greater than 1.0 would indicate that the slope is more likely to be stable, while a Factor of Safety less than 1.0 would indicate that the slope is likely to be unstable.

In accordance with the District of North Vancouver's document regarding "Natural Hazards Risk Tolerance Criteria" (File: 11.5225.00/000.000; dated November 10, 2009) the following slope stability criteria is presented:

I) For re-developments involving an increase to gross floor area on the property of less than or equal to 25%:
   a) under static conditions the slope stability Factor of Safety must be greater than 1.3; and

   b) under non-static conditions (e.g. for earthquake ground motions) the slope stability Factor of Safety must be greater than 1.0 or predicted ground displacement must be less than 0.15 metre with a 1:475 annual chance of exceedance.

ii) For new developments and for re-developments involving an increase to gross floor area on the property of greater than 25%:
   a) under static conditions the slope stability Factor of Safety must be greater than 1.5; and
   b) under non-static conditions (e.g. for earthquake ground motions) the slope stability Factor of Safety must be greater than 1.0 or predicted ground displacement must be less than 0.15 metre with a 1:2,475 annual chance of exceedance.

Since no development is currently proposed, the analyses were based on a minimum slope stability Factor of Safety of 1.3 under static conditions and 1.0 under seismic conditions. The design seismic condition was based on a seismic event with a 1:475 annual chance of exceedance, which is a 10% probability of exceedance in 50 years.

5.2 Slope Stability Models

The District of North Vancouver provided the topographic map shown on Figure 2, which we understand was developed using LiDAR technology, and which was judged to be suitably detailed for use in the slope stability analyses. It should be noted that we are not in a position to validate all of the slope angles and topographic features shown on this map; however, selected slope angles and features were confirmed during the geotechnical reconnaissances and the topographic information provided by the District of North Vancouver appeared to be reasonably representative. If more detailed, reliable, and/or accurate topographic survey data becomes available in the future, it may be beneficial to refine the following slope stability analyses if there are significant slope geometry differences.

The locations and elevations of existing houses included in the slope stability models were estimated from aerial photographs acquired from the District of North Vancouver's GeoWeb mapping application and from site observations and measurements by Horizon Engineering.

Three slope profiles (Profiles A, B, and C) were selected for slope stability analyses through the subject slope, the locations of which are shown on Figure 2 and slope profiles for which are shown on Figures 9, 10, and 11, respectively. These slope profile locations were selected because observations were made in these areas of concern that indicated potential slope instability, as described in Section 4.5. It should be noted that a fourth slope profile was prepared (Profile D, shown on Figure 12) due to the presence of localised fill and observed signs of potential slope instability at the slope crest; however, site specific site investigation and slope stability analyses were not carried out on this slope profile due to budget constraints. Based on the results of the slope stability analyses discussed below, we do not expect that slope stability analyses of Profile D would yield less favourable results than those determined for Profiles A, B, and C.

Three generalized soil types were used in the slope stability models, consisting of a natural, weathered, sandy soil, a natural, unweathered, sandy soil, and sand fill. Based on the soil conditions observed during the subsurface investigation and our experience in the vicinity of the site, the weathered soil near the surface was considered to be cohesionless and approximately 1 to 2 metres (3 to 6 feet) thick. The thickness of fill materials on the slope profile was inferred based on the subsurface investigation results, retaining wall heights, and topography. The unweathered soil at depth may be considered to have a nominal amount of apparent cohesion resulting from in-situ effects such as matric suction, soil aging, or cementation.

As described in Section 4.6, groundwater discharge could be expected near the surface, perched on the dense to very dense sand materials (which is judged to be a conservative estimate), as well as at Hastings Creek at the toe of the slope. A phreatic surface has been included in the slope stability models to represent these conditions.

Vertical, uniform surcharge pressures of 100 and 200 psf (5 and 10 kPa) were conservatively applied to the slope stability models to represent existing one-storey building additions (i.e., Profile A) and two-storey houses.

The observed soil conditions were correlated with estimated soil strength parameters from the WildCat test results and available published information for inferred soil types and from previous projects in the vicinity of the subject site. Sensitivity analyses were carried out to refine these modelled soil strengths based on observed site conditions. The soil parameters used in this slope stability analysis are presented in Table 2.

Table 2: Soil Parameters Used in Slope Stability Analyses

Soil Type	Estimated Unit Weight		Cohesion		Friction Angle
	(pcf)	(kN/m^3)	(psf)	(kPa)	(degrees)
sand fill	120	19	0	0	33
weathered sand	120	19	0	0	33
unweathered sand	130	20	100	5	42

Both shallow, surficial failures and deep-seated failure surfaces were investigated as part of the slope stability analyses. Potential failure surfaces were modelled at the upper portion of the slope in addition to the overall slope. Additional analyses whereby the stability of global failures that could intersect the existing houses at the crest of the slope were also carried out.

5.3 Static Condition Analysis

5.3.1 Profile A

As presented on Figure 13, the potential critical overall slope failure surface on Profile A (daylighting at the crest of the slope, and therefore not intersecting the existing house and addition footprint areas) was determined to be marginally stable under static conditions, with a Factor of Safety of approximately 1.2, while the potential critical upper slope failure surface was determined to be unstable under static conditions, with a Factor of Safety of approximately 0.9. Since both of these critical failure surfaces are expected to terminate within the fill materials comprising the retaining wall that was observed to be bulging (i.e., slowly failing) and due to the observed slope angle and loose soil condition in the upper portions of the soil profile as previously described, this shallow failure mechanism is expected to be probable (and ongo ing if site conditions are not improved).

It is likely, and born out by sensitivity analyses varying cohesion of the fill and unweathered soil, that root mass cohesion is contributing to current local slope stability and an actual Factor of Safety higher than 0.9. Decreases in root mass cohesion, resulting from decomposition, frost heave, or significant rainfall events could be slow or sudden but would be expected to be associated with ongoing slope movement, which may also be slow or sudden.

The potential critical failure surface intersecting the existing house (specifically, the addition at the southeast portion of the building) was determined to be stable under static conditions, with a Factor of Safety of approximately 1.5, which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

5.3.2 Profile B

As presented on Figure 14, the potential critical overall slope failure surface on Profile B (daylighting at the crest of the slope, and therefore not intersecting the existing house footprint area) was determined to be stable under static conditions, with a Factor of Safety of approximately 1.4. Although this meets the District of North Vancouver Risk Tolerance Criteria, this critical failure surface is expected to terminate in the vicinity of an observed linear topographic feature as previously described (which may represent an ancient scarp), this location should be monitored, as described more fully in Section 6.4, if site conditions are not improved. It should be noted that these analyses for Profile B assume that there is no pre-existing subsurface weakened zone along a surface coincident with the linear topographic feature previously described in Section 4.5.

The potential critical failure surface intersecting the existing house was determined to be stable under static conditions, with a Factor of Safety of approximately 1.5, which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

5.3.3 Profile C

As presented on Figure 15, the potential critical overall slope failure surface on Profile C (daylighting below the crest of the slope, and therefore not intersecting the existing house footprint area) was determined to be stable under static conditions, with a Factor of Safety of approximately 1.4, while the upper slope was determined to be unstable under static conditions, with a Factor of Safety of approximately 0.9 (which ignores root mass cohesion). Since no obvious indicator signs of existing slope instability were noted near the termination zone of the overall slope critical failure surface, this shallow failure mechanism is expected to be improbable, as these analyses predict. However, smaller-scale failures, such as that predicted for the upper slope, are expected to be probable (and ongoing if site conditions are not improved) as a result of expected loose soil conditions within the fill materials and local oversteepening of the slope.

The potential critical failure surface intersecting the existing house was determined to be stable under static conditions, with a Factor of Safety of approximately 1.6, which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

5.4 Seismic Condition Analysis

5.4.1 General

As described in Section 5.1 and in accordance with the District of North Vancouver's document regarding "Natural Hazards Risk Tolerance Criteria", the seismic slope stability analyses would be based on a seismic event with a 1:475 annual chance of exceedance, which is a 10% probability of exceedance in 50 years. As described in Section 3.3, a seismic event with a 10% probability of exceedance in 50 years for the subject site would have a peak ground acceleration of 0.226g, where g is the gravitational acceleration. Based on the aforementioned published information, the design seismic event would not be expected to have a vertical acceleration component; therefore, the vertical seismic acceleration coefficient was set at zero.

It should be noted that in the seismic condition analyses, although the fill materials were assumed to be removed as recommended in Section 6.4 below (and were modelled as having been removed), critical failure surfaces were found to be prevalent in the weathered sand stratum. As described below, the potential critical failure surfaces intersecting the existing houses on the three analysed slope profiles were determined to have Factors of Safety of at least unity when modelled as being subjected to the design seismic conditions. Factors of Safety less than unity might be expected if these fill materials are not removed.

5.4.2 Profile A

As presented on Figure 13, the potential overall slope critical failure surface on Profile A (daylighting at the crest of the slope, and therefore not intersecting the existing house and addition) was determined to be stable under design seismic conditions, with a Factor of Safety of approximately 1.0, while the upper slope was determined to be unstable under design seismic conditions, with a Factor of Safety of approximately 0.7. This upper slope failure mechanism should be expected as a result of a seismic event due to the observed slope angle and loose to compact soil conditions in the weathered, natural sand at the upper portions of the soil profile, even after fill materials are removed.

The potential critical failure surface intersecting the existing house and addition footprint areas once the fill was removed was determined to be stable under design seismic conditions, with a Factor of Safety of approximately 1.0, which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

5.4.2 Profile B

As presented on Figure 14, the potential overall slope critical failure surface on Profile B (daylighting at the crest of the slope, and therefore not intersecting the existing house) was determined to be unstable under design seismic conditions, with a Factor of Safety of approximately 0.9.

Although the potential critical failure surface intersecting the existing house footprint area was modelled to have a Factor of Safety of approximately 0.9 when subjected to the design seismic event, the predicted slope displacement along the critical slip surface was estimated to be less than 1 cm (less than 0.5 inch), which is considered to be within the range allowed by the District of North Vancouver's Risk Tolerance Criteria. This calculation was carried out in accordance with standard practice, based on the "Slope Displacement - Method 1" approach from Appendix E of APEGBC's "Guidelines for Legislated Landslide Assessments for Proposed Residential Developments in BC" document, dated May 2010.

As noted above, these analyses for Profile B assume that there is no pre-existing subsurface weakened zone along a surface coincident with the linear topographic feature previously described in Section 4.5.

5.4.3 Profile C

As presented on Figure 15, the potential overall slope critical failure surface on Profile C (daylighting below the crest of the slope, and therefore not intersecting the existing house) was determined to be stable under design seismic conditions, with a Factor of Safety of approximately 1.0, which is allowable per the District of North Vancouver's Risk Tolerance Criteria. The upper slope was determined to be unstable under design seismic conditions, with a Factor of Safety of approximately 0.7. This failure mechanism should be expected as a result of the design seismic event due to expected loose to compact soil conditions in the weathered, natural sand at the upper portions of the soil profile, even after fill materials are removed.

The critical failure surface intersecting the existing house footprint area once the fill was removed was determined to be stable under design seismic conditions, with a Factor of Safety of approximately 1.0., which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

6.0 RUNOUT ANALYSES AND RISK ASSESSMENT

As described in Section 1.0, the original scope of this assessment included preliminary runout analyses and risk assessment for properties at the toe of the subject slope, which are described in Sections 6.1 and 6.2. Subsequently, the scope of services was increased to include more detailed runout analyses and risk assessment for selected properties located at the toe of the subject slope, as described in Sections 6.3 and 6.4. Comprehensive runout analyses and risk assessment were beyond the current scope and have not been carried out. Recommendations for such comprehensive analyses are provided in Section 6.5.

6.1 Preliminary Runout Analyses

As previously discussed, downslope movement of the fill and weathered sand materials should be expected to continue if not remediated. In order to assess the landslide risk to Carmaria Court properties at the toe of the slope, preliminary runout analyses were carried out using available information. Topographic data shown on Figure 2 was used, and the locations and elevations of existing houses were estimated from aerial photographs acquired from the District of North Vancouver's GeoWeb mapping application (subsequently refined by surveying for the detailed runout analyses, as described in Section 6.3). The angle between the west side of each house and the relevant slope crest was estimated, which were estimated to range from approximately 16 to 24 degrees.

6.2 Preliminary Risk Assessment

As discussed in Section 4.5, no obvious signs that would indicate movement of the subject houses at the crest of the subject slope were noted. Accordingly, static-condition slope stability analyses (described in Section 5.3) indicate that the potential critical failure surfaces intersecting the existing houses in the three areas of concern were determined to be stable (i.e., with Factors of Safety greater than 1.3). As a result, slope failure mechanisms that could impact the houses at the crest of the slope are expected to be improbable and therefore are not judged to warrant risk assessment.

A preliminary "Landslide Hazard Likelihood Rating" was estimated for each property based on Table 2 of BGC Engineering's "Geotechnical Stability Study: Partial Risk Analysis" (April 2009), which is a "...qualitative measure of likelihood of occurrence of a harmful or potentially harmful landslide". The preliminary Landslide Hazard Likelihood Ratings for the subject properties were estimated based on the information and observations previously described in this report, and were estimated to range from "low" to "high".

The "Spatial Probability Rating" was estimated for each property based on Table 4 of the aforementioned BGC Engineering report, which is based on the angle between each house and the relevant slope crest above, as described in Section 6.3.3. It should be emphasized that there were significant uncertainties in the estimated preliminary Spatial Probability Ratings at this stage: precision of house locations (both lateral positions and elevations), and accuracy and detail of topography (as discussed in Section 5.2), both for determining crest elevation and with regard to the presence or absence of microtopography that could affect landslide runout or catchment. Spatial Probability Rating designations are only separated by two degrees in slope angle (i.e., "high" is greater than 23 degrees, while "low" is between 19 and 21 degrees); therefore, the preliminary runout analysis is judged to be a general approximation only. We understand that a "not rated" designation, based on the source table, could be referred to as "very low" Spatial Probability Rating. The preliminary Spatial Probability Ratings for the subject properties were estimated to range from "very low" to "high".

A "Preliminary Qualitative Risk Rating" estimate of partial landslide risk for each property was determined by multiplying the preliminary Landslide Hazard Likelihood Rating and the preliminary Spatial Probability Rating for each property in accordance with Table 5 of the aforementioned BGC Engineering report. The resulting Preliminary Qualitative Risk Ratings were estimated to range from "very low" to "very high".

6.3 Detailed Runout Analyses

The Preliminary Qualitative Risk Rating based on the aforementioned preliminary runout analysis ranged from "very low" to "very high", suggesting that multiple properties warranted more detailed analyses. Subsequently, following presentation of the preliminary risk assessment results to the District of North Vancouver in the draft version of this report, our scope of services was increased to include detailed runout analyses and risk assessment for selected properties located at the toe of the subject slope such that risk for these properties could be more accurately estimated. It should be noted that these assessments are not comprehensive, as they do not account for microtopography (which may not be reflected in the LiDaR topographic data), nor do they account for fill volumes.

In order to carry out detailed runout analyses, accurate locations and elevations of the subject houses and the relevant slope crests were required and were subsequently surveyed by the District of North Vancouver. The expected landslide path that could affect each of the subject Carmaria Court houses was estimated based on the LiDaR topography by drawing potential landslide paths from the crest of the slope to Carmaria Court below, crossing contours perpendicularly (as shown on Figure 2). The surveyed elevation difference between the west side of each downslope house and the slope crest at the top of the landslide path was used with the graphically-determined horizontal length of the estimated landslide path to calculate an angle for each Carmaria Court property. These angles were estimated to range from approximately 18 to 25 degrees, and these values are shown along with the resulting Spatial Probability Ratings in Table 3 below.

6.4 Detailed Risk Assessment

In order to carry out a detailed risk assessment for the subject Carmaria Court properties of concern and refine the Landslide Hazard Likelihood Rating, an additional geotechnical and geomorphological site reconnaissance was carried out on March 13, 2013 by Mr Pierre Friele, M.Sc., P.Geo. of Cordilleran Geoscience and Ms Pamela Bayntun, P.Eng. of Horizon Engineering, as described in Sections 4.2 and 4.3. A traverse of the sloping terrain near the slope crest was carried out in order to refine the Landslide Hazard Likelihood Rating for each area at the crest of the slope that could affect the subject houses of concern on Carmaria Court below. The resulting Landslide Hazard Likelihood Ratings are provided in Table 3 below, which were estimated to range from "low" to "high".

A Preliminary Qualitative Risk Rating estimate of partial landslide risk for each property on Carmaria Court was determined by multiplying the Landslide Hazard Likelihood Rating and the Spatial Probability Rating for each property, as previously described. The resulting Qualitative Risk Ratings were estimated to range from "very low" to "very high".

Table 3: Partial Landslide Risk Analysis

Carmaria Court Address	Relevant Properties at Crest of Slope	Angle Between House and Slope Crest Along Estimated Landslide Path	Upslope Observations Supporting Landslide Likelihood Rating	Landslide Hazard Likelihood Rating	Spatial Probability Rating	Qualitative Risk Rating
2180	1576, 1582, & 1588 Merlynn Crescent	24.7	• tension cracks at 1582 Merlynn • bulging retaining wall at 1576 Merlynn • fill materials near crest • pistol-butted trees on slope • suspected ancient landslide scarp • slopes steeper than 35°	HIGH	HIGH	VERY HIGH
2194	1588 Merlynn Crescent, 2190 & 2208 Greylynn Crescent	24.4	• minor settlement of fill materials at crest at 1588 Merlynn Crescent • significant fill at 2190 Greylynn Crescent crest • slopes flatter than approximately 35°	MODERATE (LOW IF FILL REMOVED AT CREST)	HIGH	HIGH (MODERATE IF FILL REMOVED AT CREST)
2220	2224 & 2232 Greylynn Crescent	21.5	• significant fill materials at crest • fill settlement at 2232 Greylynn Crescent • slopes steeper than 35°	HIGH	MODERATE	HIGH
2252	2232 & 2240 Greylynn Crescent	20.7	• significant fill materials at crest • fill settlement at 2232 Greylynn Crescent • slopes steeper than 35°	HIGH	LOW	MODERATE
2306	2240 & 2248 Greylynn Crescent	23.1	• fill materials at crest • pistol-butted trees on upper slope • slopes steeper than 35°	MODERATE	HIGH	HIGH

2344	2248 Greylynn Crescent & 2438 Lauralynn Drive	20.9	• fill materials at crest • pistol-butted trees on upper slope • slopes steeper than 35°	MODERATE	LOW	LOW
2358	2438 & 2450 Lauralynn Drive	19.4	• fill materials generally located behind crest on nearly flat ground • slopes flatter than 35°	LOW	LOW	VERY LOW
2388	2450 Lauralynn Drive	23.01	• no fill materials observed at crest • slopes flatter than 35°	LOW	**HIGH**	MODERATE
2394	2462, 2474, 2486, 2498, 2510, & 2526 Lauralynn Drive	17.5	• bulging retaining walls at 2462 Lauralynn Drive • linear topographic feature at crest • fill materials at crest • pistol-butted trees at crest • slopes steeper than 35°	**HIGH**	VERY LOW*	MODERATE*
2398	2510 & 2526 Lauralynn Drive	19.1	• significant fill materials at crest • slopes flatter than 35°	MODERATE (LOW IF FILL REMOVED AT CREST)	LOW	LOW (VERY LOW IF FILL REMOVED AT CREST)
2404	2526 Lauralynn Drive	19.1	• significant fill materials at crest • lower slopes steeper than 35°	MODERATE	LOW	LOW
2410	2526 Lauralynn Drive	20.4	• significant fill materials at crest • lower slopes steeper than 35°	MODERATE	LOW	LOW
2412	2526 & 2542 Lauralynn Drive	19.1	• fill materials at crest • potential recent slide area on upper slope (seepage and lack of vegetation observed) • slopes steeper than 35°	**HIGH**	LOW	MODERATE
2416	2542, 2558, 2574, 2590, 2602 Lauralynn Drive	17.5	• fill materials at crest • pistol-butted trees on slope • suspected ancient landslide scarp • recent landslide observed on upper slope • slopes steeper than 35°	**HIGH**	VERY LOW*	MODERATE*

2420	2558, 2574, 2590, 2602 Lauralynn Drive	17.9	• fill materials at crest • pistol-butted trees on slope • suspected ancient landslide scarp • recent landslide observed on upper slope • slopes steeper than 35°	HIGH	VERY LOW*	MODERATE*
2424	2590 & 2602 Lauralynn Drive	20.4	• fill materials at crest • pistol-butted trees on slope • suspected ancient landslide scarp • recent landslide observed on upper slope • slopes steeper than 35°	HIGH	LOW	MODERATE

* No designation for "very low" Spatial Probability Rating is provided in the source table; therefore, designations for "low" Spatial Probability Rating were deferred to when determining Qualitative Risk Ratings.

6.5 Risk Assessment Summary

As described in Table 3, all of the Carmaria Court properties are estimated to have Qualitative Risk Ratings of "moderate", "low", or "very low", with the exception of the following four properties, which are estimated to have Qualitative Risk Ratings of "high" or "very high" and are therefore judged to warrant comprehensive risk assessment (further mitigation recommendations are provided in Section 7.4):

- 2180 Carmaria Court,
- 2194 Carmaria Court,
- 2220 Carmaria Court, and
- 2306 Carmaria Court.

It is noteworthy that the property at 2194 Carmaria Court could see a reduction in Landslide Hazard Likelihood Rating from "moderate" to "low" if the fill materials currently present at the crest of the slope above (at 1588 Merlynn Crescent and 2190 Greylynn Crescent) are removed. This reduction in Landslide Hazard Likelihood Rating would, in turn, reduce the current Qualitative Risk Rating from "high" to "moderate" and therefore negate the recommendation for comprehensive risk assessment.

If comprehensive risk assessment highlights microtopography that could affect the Spatial Probability Rating at any Carmaria Court properties, then additional comprehensive risk assessment may be warranted, as microtopography was not expressly considered in the current assessment, as described in Section 6.3. Microtopography should be assessed during the comprehensive risk assessment at all portions of the subject slope, as variations in topography that may not be reflected in the LiDaR topographic data (and therefore may not have influenced the estimated potential landslide paths shown on Figure 2) could have a positive or negative influence on the Spatial Probability Ratings by lengthening or shortening these landslide paths, or by affecting the relevant slope crest location. In particular, it is judged that Spatial Probability Ratings and therefore Qualitative Risk Ratings could be vulnerable to increases due to microtopography above the following addresses:

- 2252 Carmaria Court,
- 2412 Carmaria Court,
- 2416 Carmaria Court,
- 2420 Carmaria Court, and
- 2424 Carmaria Court.

Surveying of the slope in these areas is recommended, as is further review of landslide hazards, as described in Section 7.4.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 General

Based on the results of the site investigations and subsequent slope stability analyses, it is concluded that the subject site has been and is currently affected by both ancient and active slope instability. The following recommendations should be initiated as soon as possible to improve the slope stability and safety of residents living above and below the subject slope, as well as users of the park and its adjacent roads and creek.

7.2 Ancient Landslide Activity

As described in Section 4.5, multiple suspected ancient landslide scarps were identified within the subject site. The geologic origin of the Westlynn Terrace area is a glacial outwash deposit, which was laid down by proglacial streams as upslope glacial ice melted. For the last 10,000 years, Hastings Creek has been eroding these materials, which could be expected to slough toward the creek channel as the slopes are undercut by erosion. It should be noted that this sloughing would have been more prevalent at the beginning of the Capilano geologic era, when the subject deposits were younger and saturated. Within the current geologic era, this type of movement would be expected to be limited to the creek bank.

At least three suspected ancient landslide scarps are evident on the contours of the topographic map of the subject site, which have crests coincident with the current slope crest, as shown on Figure 2. In addition, the previously noted linear topographic features may be evidence of ancient scarps. These topographic features and more recent tension cracks are noted to be concentric with the suspected ancient landslide scarps at the south portion of the subject site, which may or may not be coincidental.

As described in Section 4.5, multiple first growth stumps (expected to be of the order of at least 500 years old) were observed to be present on the subject slope, including at some areas of the upper, middle, and lower portions of the slope. The presence of such large, intact, and upright stumps suggests that significant landslide activity has not affected the subject slope since these trees existed. Therefore, we expect that the aforementioned ancient landslides occurred more than approximately 500 years ago and the topography we see today could be considered "global equilibrium" - that is, until or unless a failure of upslope water infrastructure triggers a landslide or Hastings Creek erodes the slopes enough to result in further large scale landslides (which is not expected in the foreseeable future). We do not expect that naturally-caused large-scale, global slope stability problems such as those that occurred earlier in this era would affect the subject slopes at this time.

7.3 Recent and Ongoing Landslide Activity

7.3.1 General

Based on the signs of recent slope movement described in Section 4.5 and the results of static slope stability analyses described in Section 5.3, we conclude that recent and ongoing creep movement of the near-surface, weathered sand and fill materials has been occurring within the subject slope above Carmaria Court. We envisage that under natural conditions (i.e., had development or placement of fill materials at the crest of the slope not occurred), movement of the near-surface, weathered materials would be minimal. However, the significant fill materials and concentrated surface water being introduced at the upper portions of the slope are judged to be increasing slope movement. Fill materials that are acting as a surcharge load at the crest of the slope are envisaged to include large stumps, logs, and, soil pushed over the crest in the 1950's and 1960's during original site preparation (during which time bulldozers, not excavators, were the common site preparation equipment), yard and household debris dumped at the crest by previous and current home owners, and soil purposefully retained at the crest to provide flat back yards. In addition, other surcharge loads would include structures including building additions and sheds that are present near the slope crest. Some first growth stumps and aged logs appear to be locally integral to crest slope stability; however, these stumps appear to be decomposing to the point where this root mass cohesion contribution to slope stabilization may be approaching zero.

Without remediation, downslope movement of these weathered sand and fill materials should be expected to continue and may worsen if fill volumes and directed drainage accumulates and retention structures (natural and man-made) decompose.

7.3.2 Landslides Caused by Water Main Rupture

As referenced in Section 3.1, a landslide occurred in 2006 on the subject slope below 2248 Greylynn Crescent, as shown on Figure 2. This landslide occurred as a result of an upslope water main rupture, which entrained the surficial soils near the crest of the slope and resulted in significant erosion. The entrained materials were mobilized to Carmaria Court below and impacted the nearby residential properties. Remediation of the landslide scar comprised fill placement for erosion protection, revegetation, and construction of a small segmental retention structure on the slope to minimize and retain erosion protection materials.

We understand that the aforementioned water main rupture may have resulted from a short term increase in operating pressure within the water service utility in conjunction with aging infrastructure, which may comprise asbestos concrete pipe (a material which is expected to experience ongoing material degradation over time). Although we understand that the operating pressure within the utility has since been reduced, we envisage that the aging infrastructure may be susceptible to rupture in the future, possibly even without an increase in operating pressure. Therefore, we recommend that the water main pipes upslope of the subject site be replaced with a suitable material. In the mean time, we recommend that the fill materials near the crest of the subject slope are removed and site drainage be connected to the municipal system, as recommended in Section 6.4. This would minimize the water main rupture-induced landslide hazard to the Carmaria Court residential properties below, as well as minimize the potential slope remediation costs that might otherwise be incurred in the event of a future water main rupture.

It is noteworthy that, as described in Section 4.6, evidence of concentrated surface water flow was also observed downslope of 2602 Lauralynn Drive. At the time of our site reconnaissance, the property owner informed us of an upslope water main break that occurred in 2011. A landslide scar was observed mid-slope in this area (as shown on Figure 2), which was estimated to be approximately one to two years old based on the amount of vegetation that had grown over the scar. Based on this estimate and the landslide location, we envisage that it may have been caused by the aforementioned 2011 upslope water break. Minor surficial erosion was noted on the lower slope below; however, no evidence of landslide debris was observed at the lower portion of the slope or near Carm aria Court.

It should be noted that the discussions within this report regarding runout analysis, risk assessment, and slope stability management do not specifically consider the potential for water main rupture-induced landslides.

7.4 Recommended Mitigative Measures and Comprehensive Risk Assessment

Where the landslide Qualitative Risk Ratings are estimated to be "high" or "very high" as described in Section 6.4 (i.e., 2180, 2194, 2220, and 2306 Carmaria Court), we recommend that mitigation of the landslide risk is carried out. Based on the current risk assessment, mitigation of the landslide risk is recommended at the following properties at the crest of the slope:

- 1576 Merlynn Crescent
- 1582 Merlynn Crescent
- 1588 Merlynn Crescent
- 2190 Greylynn Crescent
- 2232 Greylynn Crescent
- 2240 Greylynn Crescent
- 2248 Greylynn Crescent

We recommend that property owners of the above listed Merlynn and Greylynn Crescent properties, as well as the owners of the properties at 2180, 2194, 2220, and 2306 Carmaria Court be notified of the potential landslide risk as described in this report. We recommend that mitigative works be undertaken as soon as possible, designed and field-reviewed by individually hired qualified professionals.

Removal of the crest fill materials at these properties would be expected to reduce the Landslide Hazard Likelihood Ratings at the downslope Carmaria Court properties; however, reduction to acceptable levels may not be possible without removal of all near-surface, weathered soil (i.e., the potential sliding mass), which may not be feasible. However, removal of crest fill materials may reduce the travel angle and, hence, the Spatial Probability Ratings. Further comprehensive assessments at the subject properties at risk are recommended.

The comprehensive risk assessments should be carried out using detailed topographic information to highlight microtopography, which we envisage would be obtained by surveying the slope above the aforementioned four Carmaria Court properties. Each comprehensive risk assessment should include a vulnerability assessment, which would require characterization of the potential landslide affecting each house (i.e., potential volume, depth of debris, velocity of impact, etc.). Re-assessment of the Spatial Probability Rating and Qualitative Risk Rating for each property should follow. If comprehensive risk assessments indicate an unacceptable risk to any Carmaria Court properties, construction of a mitigative structure such as a debris catchment berm, retaining wall, or debris fence may be required.

7.5 Slope Stability Management

As described in Section 7.3.1, downslope movement of the weathered sand and fill materials on the subject slopes should be expected to continue and may worsen if slope conditions do not improve at the crest of the slope. The following recommendations are provided with respect to improving the stability of the slopes within and adjacent to the West Hastings Escarpment, and pertain to all properties located near the slope crest:

- Fill materials and associated retaining walls at and near the crest of the slope should be removed, including retained fills, yard debris, and fill materials that have been pushed or dumped onto the upper portions of the slope. Fill removal and slope recontouring at private property should be carried out under the direction of a qualified geotechnical engineer. It is noteworthy that retaining walls were observed near the crest of the slope at the following properties:
    - 1570 Merlynn Crescent
    - 1576 Merlynn Crescent (observed to be bulging)
    - 1582 Merlynn Crescent (fence above observed to be bowed)
    - 2190 Greylynn Drive (located behind crest)
    - 2208 Greylynn Drive (located behind crest)
    - 2462 Lauralynn Drive (observed to be failing)
    - 2498 Lauralynn Drive
    - 2542 Lauralynn Drive
    - 2590 Lauralynn Drive (observed to be failing)

- No additional surcharge loads, such as fill, retaining walls, or other structures, should be placed on the slope without suitable engineering recommendations regarding slope stability. If property owners want to extend their back yards following fill removal, this could be attained by constructing decks or retaining walls founded upon the unweathered soil at depth and utilizing lightweight or reinforced fill materials to restore grades. Any proposed development at the crest of the slope should undergo site specific geotechnical analysis and design by a suitably qualified professional adhering to the District of North Vancouver's requirements.

- A review of existing structures near the crest of the slope should be carried out by the District of North Vancouver to determine if they were permitted. The observed structures in question include, but are not limited to, the following:

- house addition at 1576 Merlynn Crescent (suspected to be an enclosure beneath a deck),
- two garden sheds at 2208 Lauralynn Drive,
- garden shed at 2462 Lauralynn Drive, and
- deck at 2498 Lauralynn Drive.

- Intercepted water from all houses and hard landscaped surfaces, including rainwater leaders and perimeter drainage, should either be connected to the District of North Vancouver's storm sewer system or another suitable dispersion system. If connection to the municipal storm sewer is not possible, intercepted water should be managed by a system designed by a qualified geotechnical engineer.

- Landscaping water features (such as those observed at 2526 and 2558 Lauralynn Drive) and other potential sources of water near the crest of the slope should be repaired or removed if leakage is observed or suspected.

- Vegetation on the slope should be retained where possible in an effort to reduce surface erosion and soil ravelling.

- The existing slope geometry should not be steepened.

- Excavation work at the toe of the slope should not be carried out without prior review and recommendations from a geotechnical engineer.

Should there be any observed signs of increased ground movement such as recent settlement or new / widened / extended tension cracks, these areas should be immediately reviewed by a qualified professional engineer.

We recommend that a public education and reporting program be initiated to provide property owners at the crest of the subject slope with information regarding slope stability, with emphasis on increased vigilance in areas near the crest and toe of the subject slope. We recommend that this program include the following:

- a brief explanation of slope stability issues and potential risks to properties at the crest and toe of the slope,

- instructions not to dump yard waste or fill onto the upper portions of a slope, or to stockpile materials near the crest (we recommend that an enforcement system is adopted in this regard),

- instructions regarding disposal of intercepted water, as described above,

- information regarding development near the slope crest (including house additions, sheds, decks, hot tubs, etc.) and the associated permitting process required, and

- recommendations pertaining to monitoring their property for signs of slope instability (including tension cracks, ground settlement, foundation cracks, leaning trees, displaced fences, etc.) and reporting any such signs to the District of North Vancouver and a qualified geotechnical engineer. Installation of stake lines parallel to the slope crest are recommended as a simple and effective means of visual slope stability monitoring.

Consideration could be given to including reporting as an element of the monitoring program. If there is a lack of confidence that this monitoring program will be effective, consideration could be given to installing inclinometer(s) in deep drillhole(s) at select locations near the crest of the West Hastings Escarpment slope. These inclinometers could be monitored on an annual basis by a suitably qualified party. In addition, installation of these drillholes would have the benefit of confirming soil strengths at depth, particularly in the areas of concentric topographic features, as described above.

8.0 CLOSURE

This report has been prepared for the sole use the District of North Vancouver and other consultants for this project. Any use or reproduction of this report for other than the stated intended purpose is prohibited without the written permission of Horizon Engineering Inc.

We are pleased to be of assistance to you on this project and we trust that our comments and recommendations are both helpful and sufficient for your current purposes. If you would like further details or require clarification of the above, please do not hesitate to contact us.

For
HORIZON ENGINEERING INC

Karen E. Savage, P.Eng.
President

For
HORIZON ENGINEERING INC

Pamela Bayntun, P.Eng.
Project Engineer

Attachments:

Figure 1	Site Location Plan
Figure 2	Site and Test Hole Location Plan
Figure 3	Photographs 1 and 2
Figure 4	Photographs 3 and 4
Figure 5	Photographs 5 and 6
Figure 6	Photographs 7 and 8
Figure 7	Photographs 9 and 10
Figure 8	Photographs 11 and 12
Figure 9	Slope Profile A
Figure 10	Slope Profile B
Figure 11	Slope Profile C
Figure 12	Slope Profile D
Figure 13	Slope Profile A - Slope Stability Assessment Results
Figure 14	Slope Profile B - Slope Stability Assessment Results
Figure 15	Slope Profile C - Slope Stability Assessment Results

Test Pit Logs (TP13-1 through TP13-3)
Wildcat Cone Penetration Data & Results (WCT12-1 through WCT12-3)

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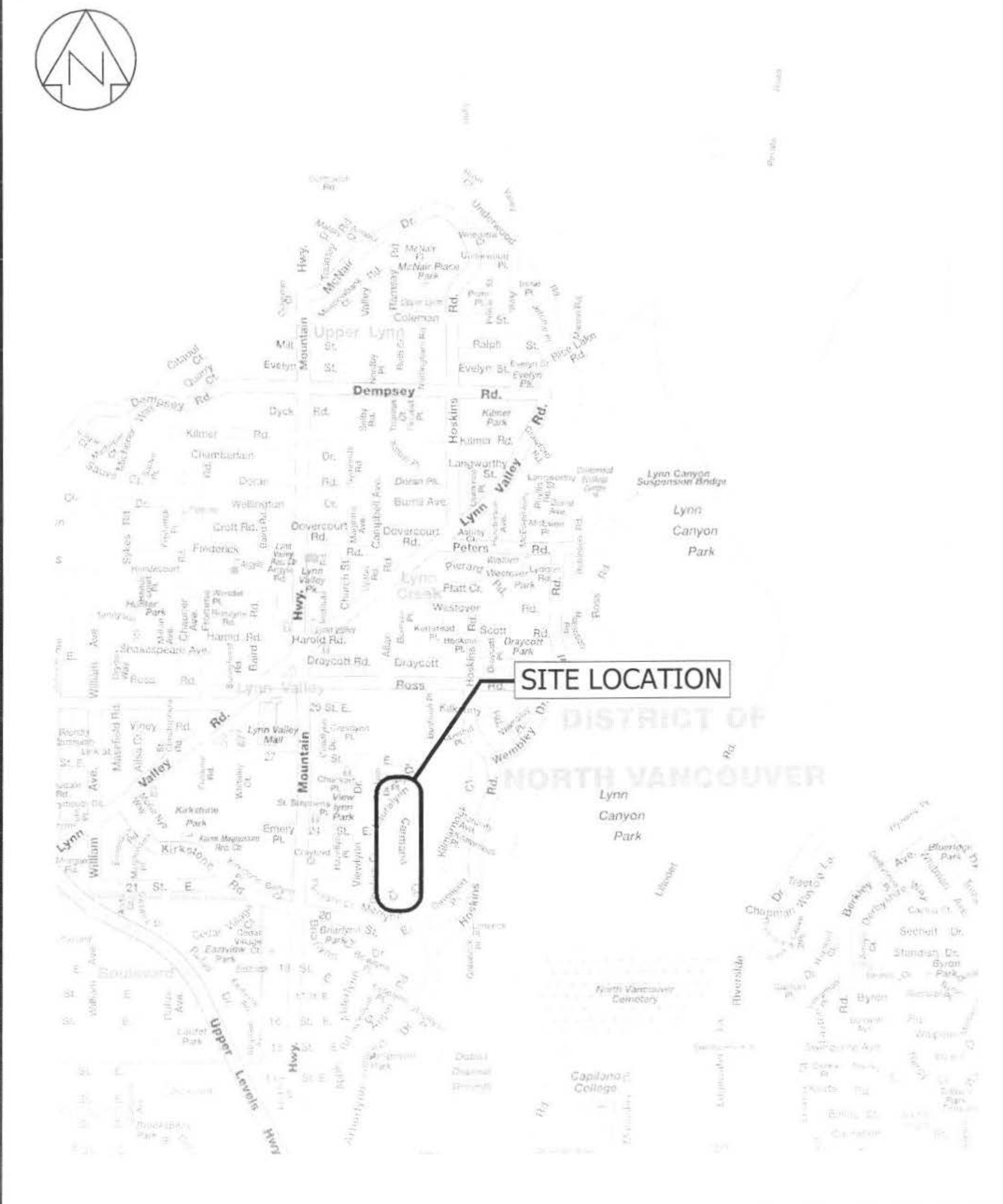


Apr 13, 2013 PM, Bronson, N:\2012 Projects\112-3072 DNV Hostings Park\drawings\112-3072 Site Location & Test Hole Plans R0.dwg

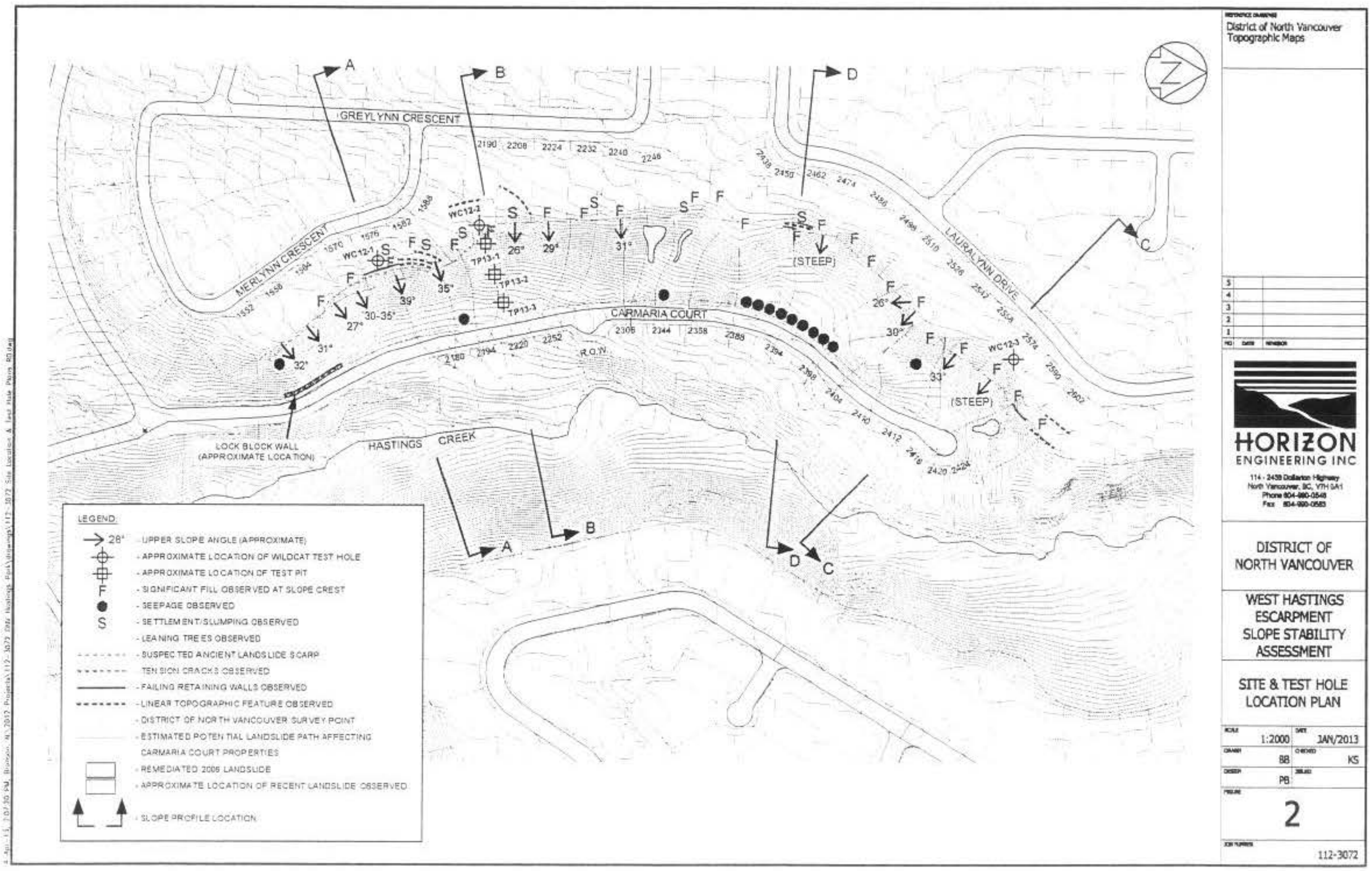
DISTRICT OF NORTH VANCOUVER

WEST HASTINGS ESCARPMENT
SLOPE STABILITY ASSESSMENT

SITE LOCATION
PLAN

HORIZON
ENGINEERING INC

Scale	Job No	Date	FIGURE
NTS	112-3072	JAN/2013	1
Des: PB	Dwn: BB	Chk: KS	Rev:



GREYLYNN CRESCENT
MERLYNN CRESCENT
CARMARIA COURT
LAURALYNN DRIVE
HASTINGS CREEK
LOCK BLOCK WALL
(APPROXIMATE LOCATION)
(STEEP)
LEGEND:
- UPPER SLOPE ANGLE (APPROXIMATE)
- APPROXIMATE LOCATION OF WILDCAT TEST HOLE
- APPROXIMATE LOCATION OF TEST PIT
- SIGNIFICANT FILL OBSERVED AT SLOPE CREST
- SEEPAGE OBSERVED
- SETTLEMENT/SLUMPING OBSERVED
- LEANING TREES OBSERVED
- SUSPECTED ANCIENT LANDSLIDE SCARP
TENSION CRACKS OBSERVED
- FAILING RETAINING WALLS OBSERVED
- LINEAR TOPOGRAPHIC FEATURE OBSERVED
- DISTRICT OF NORTH VANCOUVER SURVEY POINT
- ESTIMATED POTENTIAL LANDSLIDE PATH AFFECTING CARMARIA COURT PROPERTIES
- REMEDIATED 2005 LANDSLIDE
- APPROXIMATE LOCATION OF RECENT LANDSLIDE OBSERVED
- SLOPE PROFILE LOCATION
District of North Vancouver Topographic Maps
HORIZON ENGINEERING INC
114 - 2438 Dollarton Highway
North Vancouver, BC, V7H 0A1
Phone 604-990-0546
Fax 604-990-0583
DISTRICT OF NORTH VANCOUVER
WEST HASTINGS ESCARPMENT SLOPE STABILITY ASSESSMENT
SITE & TEST HOLE LOCATION PLAN
1:2000
JAN/2013
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KS
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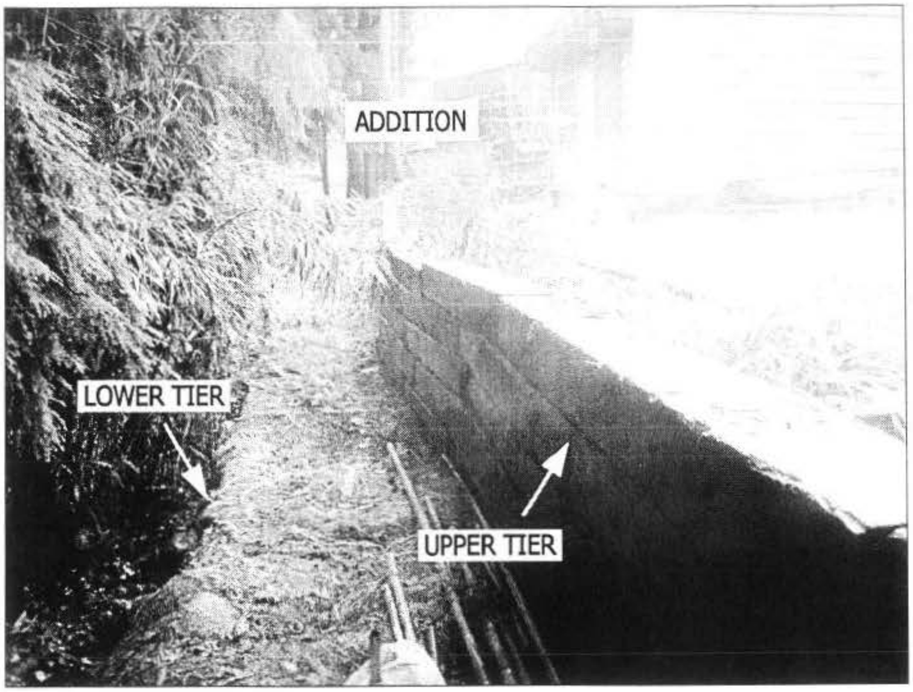
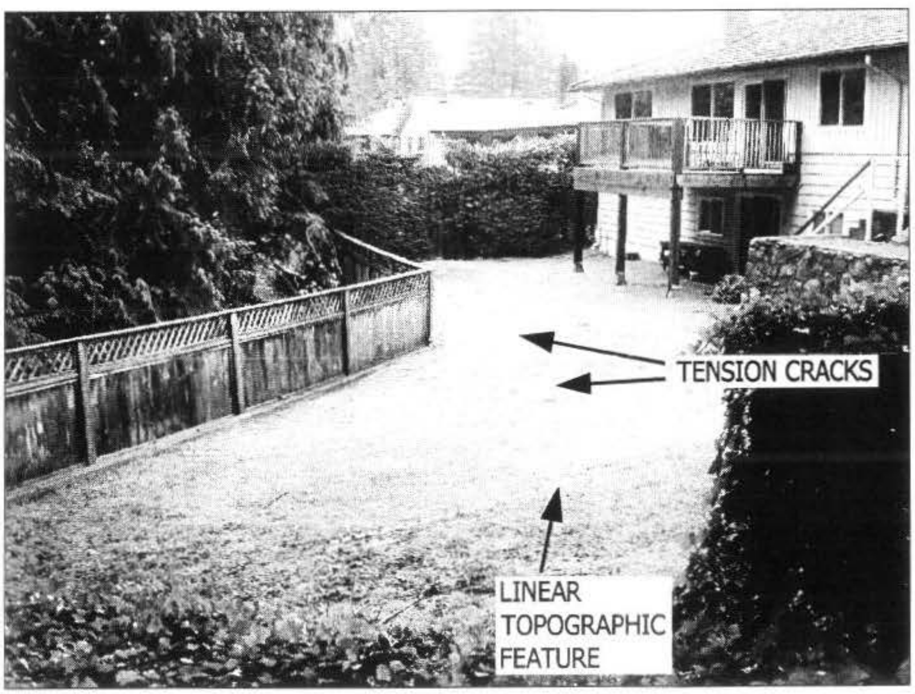
2
112-3072



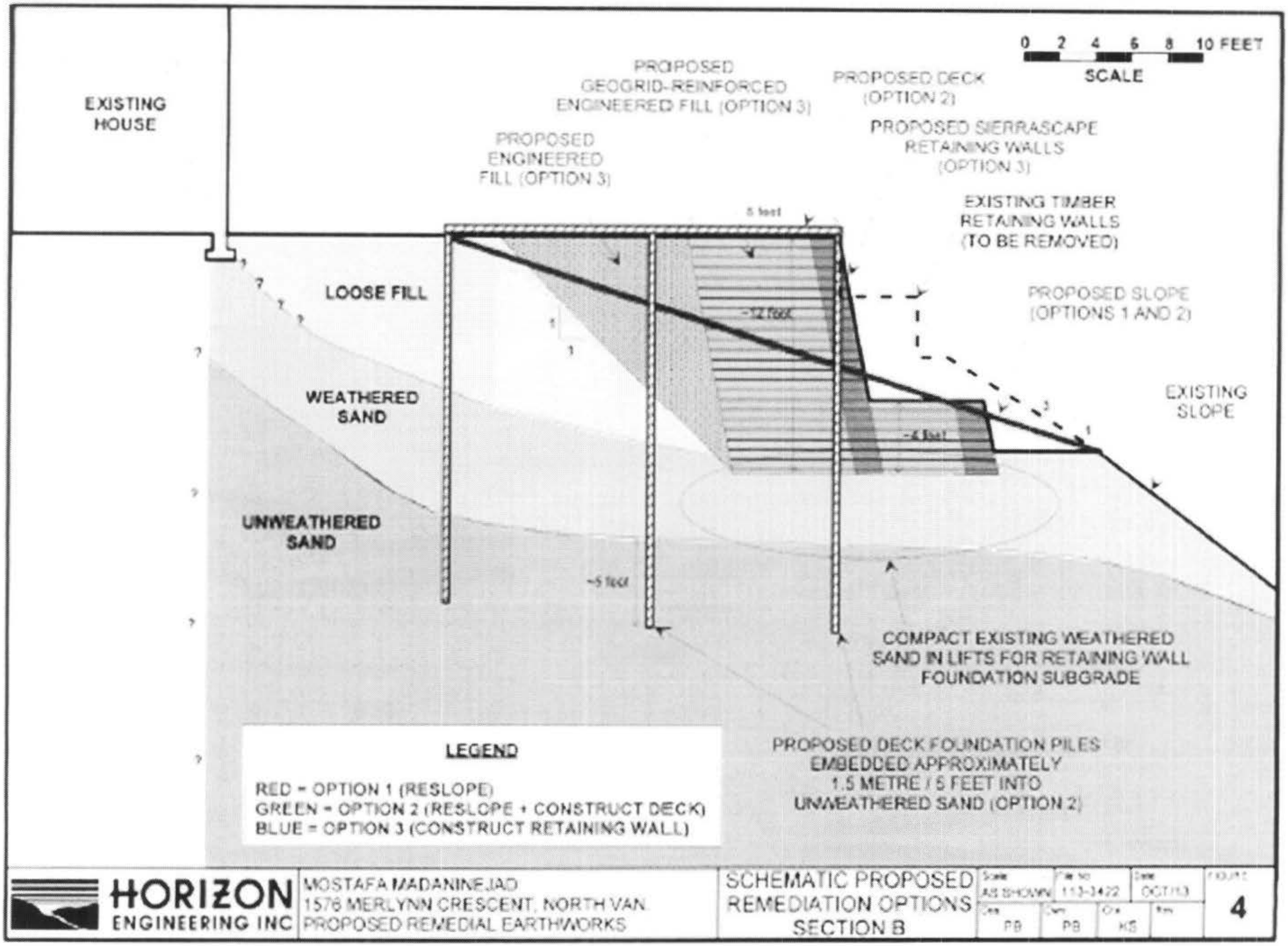
PHOTOGRAPH 1. BULGING TIMBER RETAINING WALL OBSERVED AT 1576 MERLYNN CRESCENT.



PHOTOGRAPH 2. LINEAR TOPOGRAPHIC FEATURE AND TENSION CRACKS OBSERVED AT 1582 MERLYNN CRECENT.

DISTRICT OF NORTH VANCOUVER	PHOTOGRAPHS 1 AND 2	HORIZON ENGINEERING INC
WEST HASTINGS ESCARPMENT SLOPE STABILITY ASSESSMENT		Scale: NTS; Job No: 112-3072; Date: JAN/2013; FIGURE: 3; Des: PB; Dwn: BB; Chk: KS; Rev:

Attachment B


EXISTING HOUSE
PROPOSED GEOGRID-REINFORCED ENGINEERED FILL (OPTION 3)
PROPOSED DECK (OPTION 2)
0 2 4 6 8 10 FEET
SCALE
PROPOSED ENGINEERED FILL (OPTION 3)
PROPOSED SIERRASCAPE RETAINING WALLS (OPTION 3)
EXISTING TIMBER RETAINING WALLS (TO BE REMOVED)
LOOSE FILL
PROPOSED SLOPE (OPTIONS 1 AND 2)
WEATHERED SAND
EXISTING SLOPE
UNWEATHERED SAND
COMPACT EXISTING WEATHERED SAND IN LIFTS FOR RETAINING WALL FOUNDATION SUBGRADE
LEGEND
RED = OPTION 1 (RESLOPE)
GREEN = OPTION 2 (RESLOPE + CONSTRUCT DECK)
BLUE = OPTION 3 (CONSTRUCT RETAINING WALL)
PROPOSED DECK FOUNDATION PILES EMBEDDED APPROXIMATELY 1.5 METRE / 5 FEET INTO UNWEATHERED SAND (OPTION 2)
HORIZON ENGINEERING INC
MOSTAFA MADANINEJAD
1576 MERLYNN CRESCENT, NORTH VAN.
PROPOSED REMEDIAL EARTHWORKS
SCHEMATIC PROPOSED REMEDIATION OPTIONS SECTION B
AS SHOWN
113-3422
OCT/13
4
PB
PB
KS

9.8.3

AGENDA INFORMATION	
❐ Regular Meeting	Date:__________
❐ Workshop (open to public)	Date:__________

Dept. Manager	~~GM/~~ Director	CAO

The District of North Vancouver
REPORT TO COUNCIL

December 5, 2013
File:

AUTHOR: Michelle Weston

SUBJECT: **Remedial Action Requirements - 1582 Merlynn Crescent: Unsafe Condition**

RECOMMENDATION:

That Council pass the following Resolutions:

1. Council declares, pursuant to section 73 of the *Community Charter*, SBC 2003 c. 26, that the property, legally described as:

   1582 Merlynn Crescent, PID: D-9772-21, Lot 21 Block D Westlynn Plan 9772

   (the "Property") is in and creates an unsafe condition due to slope stability.

2. Council hereby imposes the following remedial action requirements (the "Remedial Action Requirements") on Mr. William Wallace and Mrs. Patricia Wallace, the registered owners (the "Owners") to address and remediate the above unsafe condition:

   1. Select a remediation plan option and indicate to the District in writing the selected option by January 15, 2014 and submit all necessary permit applications to the District by February 15, 2014.

   2. Complete the work in accordance with the selected remediation plan and issued permits by April 30, 2014.

   3. The Owner's Qualified Professional must provide a report to the District within 3 weeks following completion of the work, certifying the safe condition of the slope.

   4. Council hereby directs that in the case of failure of the Owner to comply with the Remedial Action Requirements, then:

a. the District, its contractors or agents may enter the Property and may carry out the following remedial actions:

   i. generally restore the Property to a safe condition (Option A: 1582 Remediation Plan and Option A: 1576 Remediation Plan) to the satisfaction of the Chief Building Official; and

   ii. for the foregoing purposes may retain the services of a professional engineer to provide advice and certifications;

b. the charges incurred by the District in carrying out the aforementioned remedial actions will be recovered from the Owner as a debt; and

c. if the amount due to the District under 4(b) above is unpaid on December 31st in any year then the amount due shall be deemed to be property taxes in arrears under section 258 of the *Community Charter*.

REASON FOR REPORT:

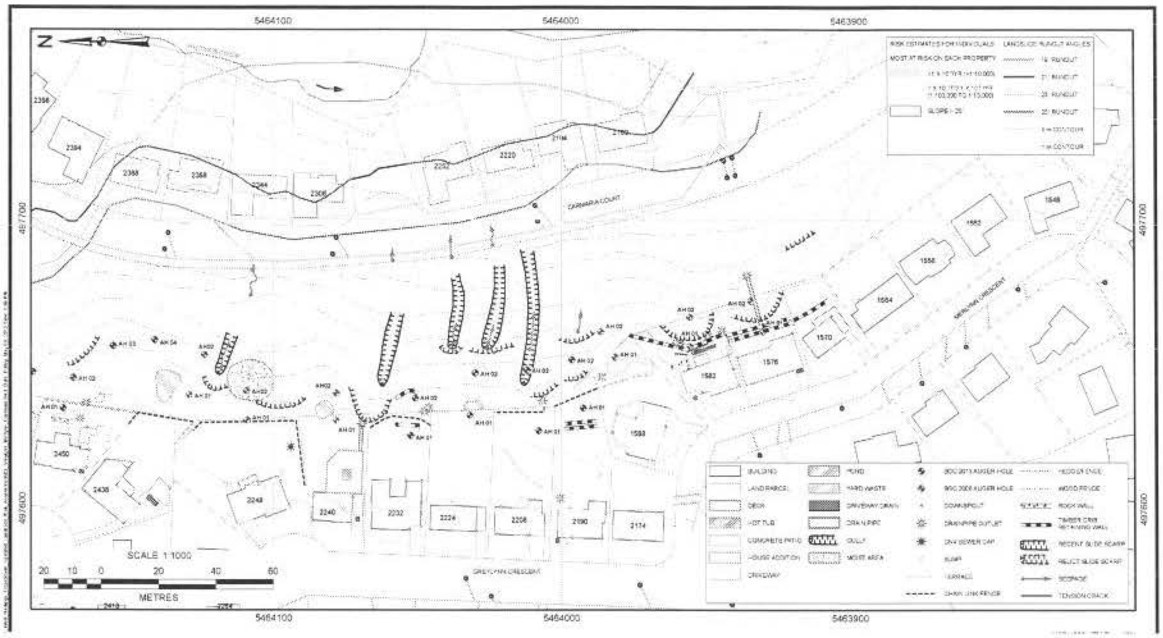
To address an unsafe condition related to slope stability on the property of 1582 Merlynn Crescent by ordering remedial action requirements to restore the slope to a safe condition to mitigate landslide risk.

BACKGROUND:

The District's adopted landside risk tolerance for existing development is 1:10,000 for Tolerable properties and 1:100,000 for Broadly Acceptable properties. The District has approximately 110 properties where landslide risks meet existing development but exceed the criteria for new development.

1582 Merlynn Crescent was rated as Tolerable during the 2008 Landslide Risk Assessment. The Owner of 1582 Merlynn Crescent contacted the District in April of 2013 with concerns over changes in slope conditions of the property, primarily the presentation of new tension cracks in the rear yard. The District retained Horizon Engineering to evaluate the slope condition of the property and other adjacent properties of the crest of the escarpment. Horizon Engineering rated the Landslide Hazard Likelihood rating as High and Qualitative Risk Rating as Very High for 1582 Merlynn Crescent (Attachment A). The Property was reevaluated in a Quantitative Risk Assessment by BGC Engineering in 2013. According to the District risk criteria, the property still falls within the Tolerable range as the landslide runout path is predicted to impact Carmaria Court Road and Utilities infrastructure and not a home. The landslide risk potential for loss of life is limited to the potential for the landslide to impact one of the Carmaria Court residents driving a car on the road. Nine homes are accessed from Carmaria Court and would be inaccessible if a landslide blocks the road. The District staff have requested the Owners to mitigate the risk of landslide based on the potential of the landslide impacting the road and causing potential injury to drivers on the road. Engineering staff and BGC Engineering met with Carmaria Court homeowners on May 23, 2013 to discuss and disclose the landslide risk.

Location of Properties



Quantiative Risk Assessment BGC 2013

Both geotechnical consultants retained by the District provided the same recommendation of removing the fill load and the removal/replacement of the retaining walls on the property for landslide mitigation.

The Owners were provided copies of geotechnical reports relating to the slope stability of the property on May 23, 2013 and met with BGC Engineering and District Staff to interpret reports. At that time the property owners were requested to voluntarily:

"Submit a plan, prepared by a Qualified Professional retained by you, to address and remediate the unsafe slope stability condition by removing backyard fill and the retaining wall on the Property (the "Remediation Plan"), acceptable to the District's General Manager, Parks and Engineering Services, (the "General Manager"), by no later than August 1, 2013; and,

Perform the remedial work required by the Remediation Plan. This work must be commenced within 30 days of the approval of the Remediation Plan by the General Manager and must be completed in accordance with the Remediation Plan and to the satisfaction of the General Manager by no later than October 15, 2013."

The Owners complied with this request and retained Horizon Engineering to develop remediation plan. The District received the remediation plan (Attachment B) on November 15, 2013 and notified the homeowners that all of the presented options were acceptable with Option A meeting the remediation requirements.

ANALYSIS:

The landslide risk to residents using Carmaria Court road creates an unsafe condition. The remediation order is needed to insure that the risk of landslide impacting the road is mitigated.

The Owners are currently obtaining price estimates from contractors on the scope of work for each remediation plan option. The cost of the remediation to each property is estimated to start at $75,000-$100,000 based on the amount of fill needed to be removed from the slope and the difficulty of access to the rear yards. The Owners have indicated limited financial ability to be able to fund the remediation needed on the Property.

An alternative of a debris fence being constructed at the base of the slope was explored. Preliminary cost estimates to design and install the fence start at $150,000. Installation of a fence would not stop the impending landslide from occurring and clean-up costs would be additional once the landslide occurred.

EXISTING POLICY:

Section 72 of the Community Charter authorizes local governments to impose "remedial action requirements" with respect to hazardous conditions and declared nuisances. Council can require a person to remove, demolish, alter, or otherwise deal with the matter in accordance with the directions of Council or a person authorized by Council.

Section 73 of the Charter specifically authorizes local councils to impose a remedial action requirement where council considers a "matter or thing is in or creates an unsafe condition or the matter or thing contravenes the provincial building regulations or a bylaw under section 8(3)(1) of Division 8 [building regulation] of this Part."

The resolution imposing a remedial action requirement must specify a time by which the required action must be taken which must be at least 30 days after notice of the order is sent. If the person wishes to appeal, they have 14 days to request reconsideration by Council.

If the remedial action requirements are not completed within the time permitted, the District can complete the requirements at the expense of the property owner (per s. 17 of the Charter). If the costs are unpaid at the end of the year, they may be added to the property taxes (s. 258).

Timing/Approval Process:

The District has requested the homeowners notify the District of a decision on which alternative is chosen by January 15, 2014. The Community Charter requires that the deadline cannot be earlier than 30 days after the notice of the remedial action requirements is sent to the owner. The work should be completed by April 30, 2014.

Concurrence:

The Municipal Solicitor has reviewed the recommendations.

Financial Impacts:

In the case of default, the District may undertake the remedial action requirements at the expense of the owner and recover the costs as a debt (s. 17 of the Charter). If the debt remains unpaid on December 31, the amount may be added to the property taxes (s. 258 of the Charter).

The homeowners, as seniors have indicated a limited financial ability to carry out the remediation. In recognition of the financial limitations of the homeowners, the District has provided $2,000 in geotechnical assistance towards development of the remediation plan, has waived permit fees and is providing a location to dump fill for the remediation. The District has offered to tarp the property to lessen the risk of landslide prior to the remediation. This offer has been accepted by the homeowners of 1582 Merlynn Crescent.

Conclusion:

A remedial action order is required from Council to ensure that remediation to mitigate landslide risk is addressed.

Michelle Weston
Section Manager, Public Safety

REVIEWED WITH:

☐ Sustainable Community Dev. ____	☐ Clerk's Office ____	External Agencies:
☐ Development Services ____	☐ Communications ____	☐ Library Board ____
☐ Utilities ____	☐ Finance ____	☐ NS Health ____
☐ Engineering Operations ____	☐ Fire Services ____	☐ RCMP ____
☐ Parks & Environment	☐ ITS	☐ Recreation Com.

☐ Economic Development	____	☐ Solicitor	____	☐ Museum & Arch.	____
☐ Human resources	____	☐ GIS	____	☐ Other:	____

Attachment A

HORIZON
ENGINEERING INC

Unit 1
2433 I
North Vancouver, BC
Canada V7H 0A1

Fax 604-990-0583
www.horizoneng.ca

April 4, 2013

Our File: 112-3072

DISTRICT OF NORTH VANCOUVER
355 West Queens Road
North Vancouver, BC V7N 4N5

Attention: Michelle Weston

Re: Slope Stability Assessment
West Hastings Escarpment, North Vancouver, BC
Geotechnical Investigation Report

1.0 INTRODUCTION

This document reports on the results of the geotechnical assessment carried out at the West Hastings Escarpment in North Vancouver and provides geotechnical comments and recommendations regarding slope stability. The scope of this assessment included a general site reconnaissance, subsequent detailed site investigation at three areas of concern, slope stability analyses of these selected areas, and preliminary runout analyses and risk assessment. This report is prepared in conformance with our proposed scope of services, dated May 4, 2012. Authorization to proceed was received on May 11, 2012. Subsequently, the scope of services was increased to include more detailed runout analyses and risk assessment for selected properties located at the toe of the subject slope.

2.0 SITE DESCRIPTION

The West Hastings Escarpment is located in the Westlynn Terrace area of North Vancouver, as shown on Figure 1 (attached following the text of this document) and is approximately 500 metres (1,600 feet) in length (north-south) and approximately 40 to 60 metres (130 to 200 feet) in width (east-west). This area is bounded by residential properties off of Merlynn Crescent, Greylynn Crescent, and Lauralynn Drive to the west, Carmaria Court with residential properties and Hastings Creek beyond to the east, and residential developments to the north and south. This area is also known as Hastings Park and is currently undeveloped and forested.

Topography within the park generally slopes down from west to east and comprises moderate to steep upper slopes and gentle to moderate lower slopes, with an existing Lock Block retaining wall that retains a road cut on the west side of Carmaria Court at the south portion of the site. Topography west of the site is generally flat and sloping gently down to the south, while topography east of the site is generally flat to gently sloping down to the south across Carmaria Court and adjacent building areas and then moderately to steeply sloping down to Hastings Creek.

At the times of our site visits, the properties at the crest of the West Hastings Escarpment were generally developed with one to two storey houses at the central portion of the sites. The west portions of the properties were generally developed with both soft and hard landscaping. The back yard areas situated at the east portions of these properties were typically developed with soft landscaping from the houses to the slope crest, with the balance consisting of undeveloped forested terrain. Some properties were noted to have wood retaining walls near the crest of the slope. The properties at the crest of the West Hastings Escarpment slope that were reviewed as part of the current assessment include:

- 1552, 1558, 1564, 1570, 1576, 1582, and 1588 Merlynn Crescent,
- 2190, 2208, 2224, 2232, 2240, and 2248 Greylynn Crescent, and
- 2438, 2450, 2474, 2486, 2498, 2510, 2526, 2542, 2558, 2574, 2590, and 2602 Lauralynn Drive.

At the toe of the slope, all properties on Carmaria Court (i.e., 2180 through 2424 Carmaria Court) were considered with respect to the effects of upslope conditions.

3.0 BACKGROUND INFORMATION

3.1 Reference Documents

We have read and interpreted the following reports that were provided to us for relevant background information:

- 'Westlynn and Pemberton Heights Escarpments: Preliminary Landslide Hazard Assessment' report prepared by BGC Engineering Inc., dated November 29, 2007
- 'District of North Vancouver: 2009 Landslide Risk Assessment For Select Escarpment Slopes' report prepared by BGC Engineering Inc., dated January 4, 2010
- 'District of North Vancouver: Landslide Risk Summary' report prepared by BGC Engineering Inc., dated November 12, 2010

Based on the above published information by BGC Engineering, the properties at the crest of the Hastings Park slopes for which a landslide hazard is identified are understood to have previously assessed risk levels of "Broadly Acceptable" (i.e., 1588 Merlynn Crescent, 2240 and 2448 Greylynn Crescent, and 2438, 2558, 2574, 2590, and 2602 Lauralynn Drive) or "Tolerable" (i.e., 1576 and 1582 Merlynn Crescent) per the District of North Vancouver's Risk Tolerance Criteria.

It should be noted that Horizon Engineering has previously issued the following documents pertaining to properties that are within the subject site:

- Geotechnical Comments - Proposed Foundation Drainage Discharge at 2498 Lauralynn Drive, North Vancouver, BC - Site Reconnaissance July 6, 2012 (dated July 11, 2012,
- Geotechnical Comments - Linear Ground Depressions at 1582 Merlynn Crescent, North Vancouver, BC (dated April 27, 2012),
- Geotechnical Comments - Slope Stability Reconnaissance at 1570, 1576, and 1588 Merlynn Crescent, North Vancouver, BC (dated May 22, 2012), and

- Geotechnical Investigation Report - Landslide Investigation and Remediation at 2248 Greylynn Crescent, North Vancouver, BC (dated May 24, 2008, which pertains to a landslide caused by an upslope water main break).

The District of North Vancouver's online GeoWeb Geographical Information System was referenced to obtain aerial photos, building footprint locations, and topographic contours, the latter of which is understood to be based on aerial LiDaR (Light Detection and Ranging) mapping. Survey data collected by the District of North Vancouver in March, 2013 was also referenced, as described in Section 6.3.

3.2 Geological Survey of Canada

Based on information provided by the Geological Survey of Canada, the subsurface materials at the subject site are expected to be Capilano Sediments, comprising "raised deltaic and channel fill medium sand to cobble gravel up to 15 metres thick deposited by proglacial streams and commonly underlain by silty to silty clay loam" (Geological Survey of Canada: Surficial Geology of Vancouver, Map 1486A). These expected soil conditions have been previously observed in the general vicinity of the subject site and have generally been found to be in a dense to very dense / very stiff to hard state.

3.3 Seismic Hazard Calculation

Based on published information in the 2012 edition of the British Columbia Building Code (Division B - Appendix C), seismic events with 2% and 10% probabilities of exceedance in 50 years for the subject site would have peak ground accelerations of 0.429g and 0.226g, respectively, where g is the gravitational acceleration. This peak ground acceleration is for firm ground conditions and assumed to have no vertical acceleration component. The published 5% damped horizontal spectral acceleration values for North Vancouver for different natural periods associated with the aforementioned peak ground accelerations are presented in T able 1.

Table 1: 2012 BCBC Design Ground Motions

Probability of Exceedance in 50 Years	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)
2%	0.866	0.603	0.322	0.169
10%	0.456	0.314	0.166	0.085

3.4 District of North Vancouver

Based on the District of North Vancouver's online GeoWeb Geographical Information System, the houses on the subject properties were constructed between 1958 and 1978 (85% were constructed in 1958). The only property for which a storm sewer connection is listed or shown is 1582 Merlynn Crescent; the balance of the properties are not listed as being connected to the municipal storm sewer, which is shown graphically to exist on Merlynn Crescent.

None of the subject properties west of the West Hastings Escarpment are identified as being within Natural Environment, Creek Hazard, or Streamside Protection Development Permit Areas; however, the east portions of some of these properties are identified as being within a Slope Hazard Development Permit Area.

4.0 SITE INVESTIGATIONS

4.1 Previous Site Investigations

4.1.1 Geotechnical Reconnaissances at 2248 Greylynn Crescent

On April 12, 2006, Ms Karen Savage, P.Eng. and Mr Robert Ng, P.Eng. of Horizon Engineering attended 2248 Greylynn Crescent and the adjacent slope, accompanied by Mr Ariel Estrada, P.Eng. of the District of North Vancouver. This initial site visit was carried out in order to provide recommendations pertaining to public safety following a landslide that occurred on April 9, 2006, which was caused by an upslope water main break. A subsequent site reconnaissance was carried out on April 20, 2006 by the above Horizon engineers to collect landslide geometry measurements, observe surficial soil characteristics, and develop remediation strategies.

4.1.2 Geotechnical Reconnaissance at 1582 Merlynn Crescent

On April 27, 2012, Mr Robert Ng, P.Eng. of Horizon Engineering attended 1582 Merlynn Crescent to carry out a geotechnical reconnaissance to make observations and collect measurements pertaining to linear ground depressions that were reported at the site. A reconnaissance of the upper slope adjacent to the property was also carried out during this site visit.

4.1.3 Foundation Drainage Reconnaissance at 2498 Lauralynn Drive

On July 6, 2012, Mr Robert Ng, P.Eng. and Ms Pamela Bayntun, P.Eng. of Horizon Engineering attended 2498 Lauralynn Drive to carry out a geotechnical reconnaissance with regard to foundation drainage discharge near the subject slope crest. Observations of topography, surficial soil conditions, erosion, slope stability, and existing drainage conditions were collected during the site visit.

4.1.4 Slope Stability Reconnaissance at 1570, 1576, and 1588 Merlynn Crescent

On April 27, 2012, Mr Robert Ng, P.Eng. and Ms Pamela Bayntun, P.Eng. of Horizon Engineering attended 1570, 1576, and 1588 Merlynn Crescent to carry out a geotechnical reconnaissance with regard to slope stability. Observations of topography, surficial settlement, surficial soil conditions, and slope stability were collected during the site visit. A reconnaissance of the upper slope adjacent to the properties was also carried out during the site visit.

4.2 Geotechnical Reconnaissance

On May 9, 2012, Ms Pamela Bayntun, P.Eng. of Horizon Engineering attended the subject site to carry out a geotechnical reconnaissance and to carry out a peer review of the concurrent geomorphological site investigation. The portions of the accessible subject properties at the crest of the West Hastings Escarpment were assessed, and observations of topography, slope stability conditions, drainage, and groundwater seepage were made. Several properties were inaccessible; however, observations from adjacent properties were made wherever possible. A subsequent geotechnical reconnaissance was carried out on January 24, 2013 by Ms Pamela Bayntun and Ms Karen Savage, P.Eng. of Horizon Engineering to 'ground truth' preliminary results of the slope stability analyses.

After issuing a draft version of this report, our scope of services was increased as described in Section 6.3. The increased scope warranted an additional geotechnical and geomorphological reconnaissance to refine the landslide hazards at the site, which was carried out on March 13, 2013 by Ms Pamela Bayntun, P.Eng. of Horizon Engineering and Mr Pierre Friele, M.Sc., P.Geo. of Cordilleran Geoscience.

4.3 Geomorphological Site Investigation

In order to obtain an understanding of the potential natural hazards at the subject site, a geomorphological site investigation was carried out concurrently with the May 9, 2012 geotechnical reconnaissance by Mr Pierre Friele, M.Sc., P.Geo. of Cordilleran Geoscience. This involved conducting traverses of the sloping terrain within the site and providing peer review to aspects of the geotechnical assessment. As described above, Mr Friele re-attended the site on March 13, 2013 to refine the landslide hazards at the site.

4.4 Subsurface Investigations

During the geotechnical reconnaissances and the geomorphological site investigation, multiple suspected active or ancient landslide scarps were identified within the subject site at three main locations, as shown on Figure 2 and as further described in Section 4.5 below. The first suspected landslide scarp intersects 1564 to 1582 Merlynn Crescent, the second intersects 2190 and 2208 Greylynn Crescent, and the third intersects 2574 to 2590 Lauralynn Drive. These three areas of concern were the focus of the subsurface investigations described below, as well as subsequent slope stability analyses, which are described in Section 5.0. It should be noted that the suspected ancient landslide scarp shown on Figure 2 intersecting 1552 and 1558 Merlynn Crescent appeared to be inactive and had been previously stabilized by retention at the toe of the slope; therefore, further analysis of this area was not judged to be required at this time.

4.4.1 WildCat Dynamic Cone Penetration Tests

On July 24, 2012 Mr Adam Jessop of Horizon Engineering and Mr Ben Tam of HE Testing attended the subject site to carry out the first portion of the subsurface investigation. One WildCat Dynamic Cone Penetration Test, labelled WCT12-1, was advanced at the east portion of 1576 Merlynn Crescent. On August 1, 2012 Mr Adam Jessop and Ms Alisa Andreeva of Horizon Engineering attended the subject site to carry out the second portion of the subsurface investigation. One WildCat Dynamic Cone Penetration Test, labelled WCT12-2, was advanced at the east portion of 2190 Greylynn Crescent, while a second WildCat Dynamic Cone Penetration Test, labelled WCT12-3, was advanced at the central portion of 2574 Lauralynn Crescent. WildCat Dynamic Cone Penetration Tests were advanced to depths of approximately 0.8 to 5.1 metres (2 feet 7 inches to 16 feet 9 inches) below existing grades.

Based on the WildCat DCPT sounding data, the compactness of the subsurface materials at these locations was determined ed to be:

- WCT12-1

0 - 3.0 metres (0 - 9 feet 10 inches) depth	very loose to loose
3.0 - 5.0 metres (9 feet 10 inches - 16 feet 5 inches)	loose to compact
5.0 - 5.1 metres (16 feet 5 inches - 16 feet 9 inches)	dense to very dense

- WCT12-2

0 - 0.2 metre (0 - 8 inches) depth	very loose to loose
0.2 - 0.4 metre (8 inches - 1 foot 4 inches)	compact
0.4 - 0.8 metre (1 foot 4 inches - 8 feet 7 inches)	dense to very dense

- WCT12-3

0 - 0.9 metre (0 - 3 feet) depth	very loose
0.9 - 1.0 metre (3 feet - 3 feet 3 inches)	compact
1.0 - 1.1 metre (3 feet 3 inches - 3 feet 7 inches)	very dense

WildCat test hole locations are approximately shown on Figure 2 and detailed descriptions of the inferred soil compactness encountered at the WildCat penetration test locations are provided on the attached logs. This investigation was to have included manually-excavated test pits but was curtailed due to the presence of a bear.

4.4.2 Test Pits

On January 10, 2013, Ms Alisa Andreeva and Mr Clive Clarke of HE Testing attended the subject site to carry out the third and final portion of the subsurface investigation. Three manually excavated test pits, labelled TP13-1 through TP13-3, were advanced on the sloping terrain east of 2190 Merlynn Crescent to depths of approximately 0.9 to 1.4 metre (3 feet to 4 feet 7 inches) below existing grades. Test pit locations are approximately shown on Figure 2.

The soil stratigraphy encountered at the test pit locations was found to comprise:

- TP13-1

0 - 0.5 metre (0 - 1 foot 9 inches) depth	topsoil
0.5 - 1.0 metre (1 foot 9 inches - 3 feet 4 inches)	sandy silt
1.0 - 1.1 metre (3 feet 4 inches - 3 feet 6 inches)	sand

- TP13-2

0 - 0.3 metre (0 - 1 foot) depth	topsoil
0.3 - 1.4 metre (1 foot - 4 feet 6 inches)	sandy silt to silty sand
1.4 - 1.5 metre (4 feet 6 inches - 4 feet 7 inches)	sand

- TP13-3

0 - 0.2 metre (0 - 6 inches) depth	topsoil
0.2 - 0.9 metre (6 inches - 3 feet)	sandy silt

The silty sand to sandy silt was observed to be reddish brown and was inferred to be compact / stiff. The sand was observed to be grey, fine to medium grained, and was inferred to be very dense.

Detailed descriptions of the soil encountered at the test pit locations are provided on the attached logs.

It is noteworthy that the unweathered soil exposed in a landslide scar in 2006 (described in Section 7.3.2) was observed to comprise glacial till-like sand that was inferred to be very dense.

4.5 Slope Assessment

A visual assessment of the ground conditions on the sloping terrain within the subject site was carried out in an effort to identify any ancient, existing, or potential slope stability problems.

Anthropogenic topographic alterations that were observed to have affected the slope include filling at the east portions of properties both at the slope crest and at Carmaria Court near the slope toe, as well as excavation at the Carmaria Court road cut. In addition, a Lock Block retaining wall was observed immediately west of Carmaria Court at the south portion of the subject site, which retains the road cut and which we understand was constructed in 1996 to stabilize a shallow slope failure on the slope to the west. It was also noted that the slope located east of 2248 Greylynn Crescent that was remediated following the 2006 landslide event (caused by a District of North Vancouver water main failure) had been revegetated, and no further signs of slope instability were noted in this area.

During the geotechnical reconnaissance of the east portions of the properties at the crest of the subject slope and the adjacent District of North Vancouver property to the east, multiple signs of slope movement were observed, as shown in the photographs provided on Figures 3 through 8. These signs included tension cracks and bulging and failing of existing retaining walls (Photographs 1 and 2 on Figure 3, respectively). In addition, linear topographic features were noted, which may correspond to either ancient landslide scarps and / or anthropogenic landscaping features (Photographs 3 and 4 on Figure 4, respectively). Also, pistol butted trees, ground settlement, and a recent landslide scar (estimated to be approximately one to two years old) were observed at the locations shown on Figure 2. Although detailed reconnaissance of each house at the crest of the slope was beyond the current scope, no obvious signs that would indicate movement of the subject houses were noted, including noticeable exterior cracking, noticeable foundation settlement, or signs of slope instability immediately adjacent to the west sides of the houses.

Significant fill materials were observed to be present near the crest of the slope at many of the subject properties, as indicated on Figure 2. At some locations, retaining walls or large stumps at the crest of the slope retained fill materials (Photograph 5 on Figure 5), and yard waste was observed at many locations to be dumped at or over the crest of the slope (Photograph 6 on Figure 5). Household debris was also observed at several locations to be dumped at or over the crest of the slope. The expected presence of fill materials at the crest of the slope is supported by the observation of loose to very loose soil within the upper portions of WildCat Penetration Test holes, as well as by the observation of local oversteepening of the slope at the slope crest. Using handheld equipment, the gradient of the upper slope was measured to vary from about 26 to 39 degrees, and locally as steep as approximately 53°.

Multiple first growth stumps (expected to be of the order of 500 years old) were observed to be present on the subject slope, including at some areas of the upper, middle, and lower portions of the slope (Photographs 5 and 12 on Figures 5 and 8, respectively). Some of these stumps were observed to be decomposing, and at least one stump located below 1576 Merlynn Crescent was observed to be lying on its side, which suggests that it may have been pushed over the crest of the slope during original site preparation (Photograph 11 on Figure 8) .

4.6 Surface and Groundwater Conditions

During the geotechnical reconnaissance, drain pipes were observed at nine properties located at the crest of the slope, which were directed onto the upper portion of the sloping terrain or onto the portions of the properties located immediately west of the slope crest. These properties include (but are not limited to) the following:

- 1582 Merlynn Crescent,
- 1588 Merlynn Crescent,
- 2208 Greylynn Crescent,
- 2224 Greylynn Crescent,
- 2240 Greylynn Crescent,
- 2486 Lauralynn Drive,
- 2498 Lauralynn Drive (downspouts and foundation drainage),
- 2510 Lauralynn Drive (downspouts), and
- 2526 Lauralynn Drive.

Observations were limited by dense vegetation. These drain pipes included 'Big O' or PVC type drain pipes and ceramic drain tiles that are envisaged to provide drainage for foundations, landscaping, and retaining walls (Photograph 7 on Figure 6). Several properties were observed to be directing rainwater downspouts onto the ground (Photograph 8 on Figure 6), and landscaping water features were observed to be located at the crest of the slope at 2526 and 2558 Lauralynn Drive (Photograph 9 on Figure 7). No signs of erosion or concentrated water flow were observed in these areas. The only evidence of concentrated surface water flows were observed downslope of 2248 Greylynn Crescent and 2602 Lauralynn Drive, where we understand that upslope water main breaks in recent years resulted in erosion of the subject slope.

At the times of our site investigations, no groundwater discharge was observed on the upper portions of the subject slope with the exception of minor seepage observed at the slope break located downslope of 2542 Lauralynn Drive. However, significant groundwater discharge was observed on May 9, 2012 during the geotechnical reconnaissance at the toe of the slope immediately west of Carmaria Court and particularly at the north portion of the slope, as shown on Figure 2 and Photograph 10 on Figure 7.

Moist soil conditions were generally observed within the surficial soil; however, seepage was observed at a depth of 1.1 metre (3.5 feet) below existing grade at the location of test pit TP13-2. It is envisaged that the groundwater table is located within the near-surface materials and may be perched on the dense to very dense sand materials as described in Section 4.4.

5.0 SLOPE STABILITY ANALYSIS

5.1 General

A commercially available limit equilibrium slope stability analysis program (XStabl, version 5.204) was used to carry out the analyses for the selected slope profiles under both static and design seismic ground conditions. A Bishop's method of analysis was used to search for the most critical potential circular failure surfaces that could influence the modelled portions of the subject slope.

For the purpose of communicating the comparative stability of a slope, a Factor of Safety may be determined for a given slope condition. A Factor of Safety is based on the ratio of resisting forces to driving forces, where the resisting forces help to stabilize a slope and the driving forces contribute to instability. A Factor of Safety greater than 1.0 would indicate that the slope is more likely to be stable, while a Factor of Safety less than 1.0 would indicate that the slope is likely to be unstable.

In accordance with the District of North Vancouver's document regarding "Natural Hazards Risk Tolerance Criteria" (File: 11.5225.00/000.000; dated November 10, 2009) the following slope stability criteria is presented:

I) For re-developments involving an increase to gross floor area on the property of less than or equal to 25%:
   a) under static conditions the slope stability Factor of Safety must be greater than 1.3; and

   b) under non-static conditions (e.g. for earthquake ground motions) the slope stability Factor of Safety must be greater than 1.0 or predicted ground displacement must be less than 0.15 metre with a 1:475 annual chance of exceedance.

ii) For new developments and for re-developments involving an increase to gross floor area on the property of greater than 25%:
   a) under static conditions the slope stability Factor of Safety must be greater than 1.5; and
   b) under non-static conditions (e.g. for earthquake ground motions) the slope stability Factor of Safety must be greater than 1.0 or predicted ground displacement must be less than 0.15 metre with a 1:2,475 annual chance of exceedance.

Since no development is currently proposed, the analyses were based on a minimum slope stability Factor of Safety of 1.3 under static conditions and 1.0 under seismic conditions. The design seismic condition was based on a seismic event with a 1:475 annual chance of exceedance, which is a 10% probability of exceedance in 50 years.

5.2 Slope Stability Models

The District of North Vancouver provided the topographic map shown on Figure 2, which we understand was developed using LiDAR technology, and which was judged to be suitably detailed for use in the slope stability analyses. It should be noted that we are not in a position to validate all of the slope angles and topographic features shown on this map; however, selected slope angles and features were confirmed during the geotechnical reconnaissances and the topographic information provided by the District of North Vancouver appeared to be reasonably representative. If more detailed, reliable, and/or accurate topographic survey data becomes available in the future, it may be beneficial to refine the following slope stability analyses if there are significant slope geometry differences.

The locations and elevations of existing houses included in the slope stability models were estimated from aerial photographs acquired from the District of North Vancouver's GeoWeb mapping application and from site observations and measurements by Horizon Engineering.

Three slope profiles (Profiles A, B, and C) were selected for slope stability analyses through the subject slope, the locations of which are shown on Figure 2 and slope profiles for which are shown on Figures 9, 10, and 11, respectively. These slope profile locations were selected because observations were made in these areas of concern that indicated potential slope instability, as described in Section 4.5. It should be noted that a fourth slope profile was prepared (Profile D, shown on Figure 12) due to the presence of localised fill and observed signs of potential slope instability at the slope crest; however, site specific site investigation and slope stability analyses were not carried out on this slope profile due to budget constraints. Based on the results of the slope stability analyses discussed below, we do not expect that slope stability analyses of Profile D would yield less favourable results than those determined for Profiles A, B, and C.

Three generalized soil types were used in the slope stability models, consisting of a natural, weathered, sandy soil, a natural, unweathered, sandy soil, and sand fill. Based on the soil conditions observed during the subsurface investigation and our experience in the vicinity of the site, the weathered soil near the surface was considered to be cohesionless and approximately 1 to 2 metres (3 to 6 feet) thick. The thickness of fill materials on the slope profile was inferred based on the subsurface investigation results, retaining wall heights, and topography. The unweathered soil at depth may be considered to have a nominal amount of apparent cohesion resulting from in-situ effects such as matric suction, soil aging, or cementation.

As described in Section 4.6, groundwater discharge could be expected near the surface, perched on the dense to very dense sand materials (which is judged to be a conservative estimate), as well as at Hastings Creek at the toe of the slope. A phreatic surface has been included in the slope stability models to represent these conditions.

Vertical, uniform surcharge pressures of 100 and 200 psf (5 and 10 kPa) were conservatively applied to the slope stability models to represent existing one-storey building additions (i.e., Profile A) and two-storey houses.

The observed soil conditions were correlated with estimated soil strength parameters from the WildCat test results and available published information for inferred soil types and from previous projects in the vicinity of the subject site. Sensitivity analyses were carried out to refine these modelled soil strengths based on observed site conditions. The soil parameters used in this slope stability analysis are presented in Table 2.

Table 2: Soil Parameters Used in Slope Stability Analyses

Soil Type	Estimated Unit Weight		Cohesion		Friction Angle
	(pcf)	(kN/m^3)	(psf)	(kPa)	(degrees)
sand fill	120	19	0	0	33
weathered sand	120	19	0	0	33
unweathered sand	130	20	100	5	42

Both shallow, surficial failures and deep-seated failure surfaces were investigated as part of the slope stability analyses. Potential failure surfaces were modelled at the upper portion of the slope in addition to the overall slope. Additional analyses whereby the stability of global failures that could intersect the existing houses at the crest of the slope were also carried out.

5.3 Static Condition Analysis

5.3.1 Profile A

As presented on Figure 13, the potential critical overall slope failure surface on Profile A (daylighting at the crest of the slope, and therefore not intersecting the existing house and addition footprint areas) was determined to be marginally stable under static conditions, with a Factor of Safety of approximately 1.2, while the potential critical upper slope failure surface was determined to be unstable under static conditions, with a Factor of Safety of approximately 0.9. Since both of these critical failure surfaces are expected to terminate within the fill materials comprising the retaining wall that was observed to be bulging (i.e., slowly failing) and due to the observed slope angle and loose soil condition in the upper portions of the soil profile as previously described, this shallow failure mechanism is expected to be probable (and ongo ing if site conditions are not improved).

It is likely, and born out by sensitivity analyses varying cohesion of the fill and unweathered soil, that root mass cohesion is contributing to current local slope stability and an actual Factor of Safety higher than 0.9. Decreases in root mass cohesion, resulting from decomposition, frost heave, or significant rainfall events could be slow or sudden but would be expected to be associated with ongoing slope movement, which may also be slow or sudden.

The potential critical failure surface intersecting the existing house (specifically, the addition at the southeast portion of the building) was determined to be stable under static conditions, with a Factor of Safety of approximately 1.5, which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

5.3.2 Profile B

As presented on Figure 14, the potential critical overall slope failure surface on Profile B (daylighting at the crest of the slope, and therefore not intersecting the existing house footprint area) was determined to be stable under static conditions, with a Factor of Safety of approximately 1.4. Although this meets the District of North Vancouver Risk Tolerance Criteria, this critical failure surface is expected to terminate in the vicinity of an observed linear topographic feature as previously described (which may represent an ancient scarp), this location should be monitored, as described more fully in Section 6.4, if site conditions are not improved. It should be noted that these analyses for Profile B assume that there is no pre-existing subsurface weakened zone along a surface coincident with the linear topographic feature previously described in Section 4.5.

The potential critical failure surface intersecting the existing house was determined to be stable under static conditions, with a Factor of Safety of approximately 1.5, which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

5.3.3 Profile C

As presented on Figure 15, the potential critical overall slope failure surface on Profile C (daylighting below the crest of the slope, and therefore not intersecting the existing house footprint area) was determined to be stable under static conditions, with a Factor of Safety of approximately 1.4, while the upper slope was determined to be unstable under static conditions, with a Factor of Safety of approximately 0.9 (which ignores root mass cohesion). Since no obvious indicator signs of existing slope instability were noted near the termination zone of the overall slope critical failure surface, this shallow failure mechanism is expected to be improbable, as these analyses predict. However, smaller-scale failures, such as that predicted for the upper slope, are expected to be probable (and ongoing if site conditions are not improved) as a result of expected loose soil conditions within the fill materials and local oversteepening of the slope.

The potential critical failure surface intersecting the existing house was determined to be stable under static conditions, with a Factor of Safety of approximately 1.6, which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

5.4 Seismic Condition Analysis

5.4.1 General

As described in Section 5.1 and in accordance with the District of North Vancouver's document regarding "Natural Hazards Risk Tolerance Criteria", the seismic slope stability analyses would be based on a seismic event with a 1:475 annual chance of exceedance, which is a 10% probability of exceedance in 50 years. As described in Section 3.3, a seismic event with a 10% probability of exceedance in 50 years for the subject site would have a peak ground acceleration of 0.226g, where g is the gravitational acceleration. Based on the aforementioned published information, the design seismic event would not be expected to have a vertical acceleration component; therefore, the vertical seismic acceleration coefficient was set at zero.

It should be noted that in the seismic condition analyses, although the fill materials were assumed to be removed as recommended in Section 6.4 below (and were modelled as having been removed), critical failure surfaces were found to be prevalent in the weathered sand stratum. As described below, the potential critical failure surfaces intersecting the existing houses on the three analysed slope profiles were determined to have Factors of Safety of at least unity when modelled as being subjected to the design seismic conditions. Factors of Safety less than unity might be expected if these fill materials are not removed.

5.4.2 Profile A

As presented on Figure 13, the potential overall slope critical failure surface on Profile A (daylighting at the crest of the slope, and therefore not intersecting the existing house and addition) was determined to be stable under design seismic conditions, with a Factor of Safety of approximately 1.0, while the upper slope was determined to be unstable under design seismic conditions, with a Factor of Safety of approximately 0.7. This upper slope failure mechanism should be expected as a result of a seismic event due to the observed slope angle and loose to compact soil conditions in the weathered, natural sand at the upper portions of the soil profile, even after fill materials are removed.

The potential critical failure surface intersecting the existing house and addition footprint areas once the fill was removed was determined to be stable under design seismic conditions, with a Factor of Safety of approximately 1.0, which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

5.4.2 Profile B

As presented on Figure 14, the potential overall slope critical failure surface on Profile B (daylighting at the crest of the slope, and therefore not intersecting the existing house) was determined to be unstable under design seismic conditions, with a Factor of Safety of approximately 0.9.

Although the potential critical failure surface intersecting the existing house footprint area was modelled to have a Factor of Safety of approximately 0.9 when subjected to the design seismic event, the predicted slope displacement along the critical slip surface was estimated to be less than 1 cm (less than 0.5 inch), which is considered to be within the range allowed by the District of North Vancouver's Risk Tolerance Criteria. This calculation was carried out in accordance with standard practice, based on the "Slope Displacement - Method 1" approach from Appendix E of APEGBC's "Guidelines for Legislated Landslide Assessments for Proposed Residential Developments in BC" document, dated May 2010.

As noted above, these analyses for Profile B assume that there is no pre-existing subsurface weakened zone along a surface coincident with the linear topographic feature previously described in Section 4.5.

5.4.3 Profile C

As presented on Figure 15, the potential overall slope critical failure surface on Profile C (daylighting below the crest of the slope, and therefore not intersecting the existing house) was determined to be stable under design seismic conditions, with a Factor of Safety of approximately 1.0, which is allowable per the District of North Vancouver's Risk Tolerance Criteria. The upper slope was determined to be unstable under design seismic conditions, with a Factor of Safety of approximately 0.7. This failure mechanism should be expected as a result of the design seismic event due to expected loose to compact soil conditions in the weathered, natural sand at the upper portions of the soil profile, even after fill materials are removed.

The critical failure surface intersecting the existing house footprint area once the fill was removed was determined to be stable under design seismic conditions, with a Factor of Safety of approximately 1.0., which is allowable per the District of North Vancouver's Risk Tolerance Criteria.

6.0 RUNOUT ANALYSES AND RISK ASSESSMENT

As described in Section 1.0, the original scope of this assessment included preliminary runout analyses and risk assessment for properties at the toe of the subject slope, which are described in Sections 6.1 and 6.2. Subsequently, the scope of services was increased to include more detailed runout analyses and risk assessment for selected properties located at the toe of the subject slope, as described in Sections 6.3 and 6.4. Comprehensive runout analyses and risk assessment were beyond the current scope and have not been carried out. Recommendations for such comprehensive analyses are provided in Section 6.5.

6.1 Preliminary Runout Analyses

As previously discussed, downslope movement of the fill and weathered sand materials should be expected to continue if not remediated. In order to assess the landslide risk to Carmaria Court properties at the toe of the slope, preliminary runout analyses were carried out using available information. Topographic data shown on Figure 2 was used, and the locations and elevations of existing houses were estimated from aerial photographs acquired from the District of North Vancouver's GeoWeb mapping application (subsequently refined by surveying for the detailed runout analyses, as described in Section 6.3). The angle between the west side of each house and the relevant slope crest was estimated, which were estimated to range from approximately 16 to 24 degrees.

6.2 Preliminary Risk Assessment

As discussed in Section 4.5, no obvious signs that would indicate movement of the subject houses at the crest of the subject slope were noted. Accordingly, static-condition slope stability analyses (described in Section 5.3) indicate that the potential critical failure surfaces intersecting the existing houses in the three areas of concern were determined to be stable (i.e., with Factors of Safety greater than 1.3). As a result, slope failure mechanisms that could impact the houses at the crest of the slope are expected to be improbable and therefore are not judged to warrant risk assessment.

A preliminary "Landslide Hazard Likelihood Rating" was estimated for each property based on Table 2 of BGC Engineering's "Geotechnical Stability Study: Partial Risk Analysis" (April 2009), which is a "...qualitative measure of likelihood of occurrence of a harmful or potentially harmful landslide". The preliminary Landslide Hazard Likelihood Ratings for the subject properties were estimated based on the information and observations previously described in this report, and were estimated to range from "low" to "high".

The "Spatial Probability Rating" was estimated for each property based on Table 4 of the aforementioned BGC Engineering report, which is based on the angle between each house and the relevant slope crest above, as described in Section 6.3.3. It should be emphasized that there were significant uncertainties in the estimated preliminary Spatial Probability Ratings at this stage: precision of house locations (both lateral positions and elevations), and accuracy and detail of topography (as discussed in Section 5.2), both for determining crest elevation and with regard to the presence or absence of microtopography that could affect landslide runout or catchment. Spatial Probability Rating designations are only separated by two degrees in slope angle (i.e., "high" is greater than 23 degrees, while "low" is between 19 and 21 degrees); therefore, the preliminary runout analysis is judged to be a general approximation only. We understand that a "not rated" designation, based on the source table, could be referred to as "very low" Spatial Probability Rating. The preliminary Spatial Probability Ratings for the subject properties were estimated to range from "very low" to "high".

A "Preliminary Qualitative Risk Rating" estimate of partial landslide risk for each property was determined by multiplying the preliminary Landslide Hazard Likelihood Rating and the preliminary Spatial Probability Rating for each property in accordance with Table 5 of the aforementioned BGC Engineering report. The resulting Preliminary Qualitative Risk Ratings were estimated to range from "very low" to "very high".

6.3 Detailed Runout Analyses

The Preliminary Qualitative Risk Rating based on the aforementioned preliminary runout analysis ranged from "very low" to "very high", suggesting that multiple properties warranted more detailed analyses. Subsequently, following presentation of the preliminary risk assessment results to the District of North Vancouver in the draft version of this report, our scope of services was increased to include detailed runout analyses and risk assessment for selected properties located at the toe of the subject slope such that risk for these properties could be more accurately estimated. It should be noted that these assessments are not comprehensive, as they do not account for microtopography (which may not be reflected in the LiDaR topographic data), nor do they account for fill volumes.

In order to carry out detailed runout analyses, accurate locations and elevations of the subject houses and the relevant slope crests were required and were subsequently surveyed by the District of North Vancouver. The expected landslide path that could affect each of the subject Carmaria Court houses was estimated based on the LiDaR topography by drawing potential landslide paths from the crest of the slope to Carmaria Court below, crossing contours perpendicularly (as shown on Figure 2). The surveyed elevation difference between the west side of each downslope house and the slope crest at the top of the landslide path was used with the graphically-determined horizontal length of the estimated landslide path to calculate an angle for each Carmaria Court property. These angles were estimated to range from approximately 18 to 25 degrees, and these values are shown along with the resulting Spatial Probability Ratings in Table 3 below.

6.4 Detailed Risk Assessment

In order to carry out a detailed risk assessment for the subject Carmaria Court properties of concern and refine the Landslide Hazard Likelihood Rating, an additional geotechnical and geomorphological site reconnaissance was carried out on March 13, 2013 by Mr Pierre Friele, M.Sc., P.Geo. of Cordilleran Geoscience and Ms Pamela Bayntun, P.Eng. of Horizon Engineering, as described in Sections 4.2 and 4.3. A traverse of the sloping terrain near the slope crest was carried out in order to refine the Landslide Hazard Likelihood Rating for each area at the crest of the slope that could affect the subject houses of concern on Carmaria Court below. The resulting Landslide Hazard Likelihood Ratings are provided in Table 3 below, which were estimated to range from "low" to "high".

A Preliminary Qualitative Risk Rating estimate of partial landslide risk for each property on Carmaria Court was determined by multiplying the Landslide Hazard Likelihood Rating and the Spatial Probability Rating for each property, as previously described. The resulting Qualitative Risk Ratings were estimated to range from "very low" to "very high".

Table 3: Partial Landslide Risk Analysis

Carmaria Court Address	Relevant Properties at Crest of Slope	Angle Between House and Slope Crest Along Estimated Landslide Path	Upslope Observations Supporting Landslide Likelihood Rating	Landslide Hazard Likelihood Rating	Spatial Probability Rating	Qualitative Risk Rating
2180	1576, 1582, & 1588 Merlynn Crescent	24.7	• tension cracks at 1582 Merlynn • bulging retaining wall at 1576 Merlynn • fill materials near crest • pistol-butted trees on slope • suspected ancient landslide scarp • slopes steeper than 35°	**HIGH**	**HIGH**	**VERY HIGH**
2194	1588 Merlynn Crescent, 2190 & 2208 Greylynn Crescent	24.4	• minor settlement of fill materials at crest at 1588 Merlynn Crescent • significant fill at 2190 Greylynn Crescent crest • slopes flatter than approximately 35°	MODERATE (LOW IF FILL REMOVED AT CREST)	**HIGH**	**HIGH** (MODERATE IF FILL REMOVED AT CREST)
2220	2224 & 2232 Greylynn Crescent	21.5	• significant fill materials at crest • fill settlement at 2232 Greylynn Crescent • slopes steeper than 35°	**HIGH**	MODERATE	**HIGH**
2252	2232 & 2240 Greylynn Crescent	20.7	• significant fill materials at crest • fill settlement at 2232 Greylynn Crescent • slopes steeper than 35°	**HIGH**	LOW	MODERATE
2306	2240 & 2248 Greylynn Crescent	23.1	• fill materials at crest • pistol-butted trees on upper slope • slopes steeper than 35°	MODERATE	HIGH	HIGH

2344	2248 Greylynn Crescent & 2438 Lauralynn Drive	20.9	• fill materials at crest • pistol-butted trees on upper slope • slopes steeper than 35°	MODERATE	LOW	LOW
2358	2438 & 2450 Lauralynn Drive	19.4	• fill materials generally located behind crest on nearly flat ground • slopes flatter than 35°	LOW	LOW	VERY LOW
2388	2450 Lauralynn Drive	23.01	• no fill materials observed at crest • slopes flatter than 35°	LOW	**HIGH**	MODERATE
2394	2462, 2474, 2486, 2498, 2510, & 2526 Lauralynn Drive	17.5	• bulging retaining walls at 2462 Lauralynn Drive • linear topographic feature at crest • fill materials at crest • pistol-butted trees at crest • slopes steeper than 35°	**HIGH**	VERY LOW*	MODERATE*
2398	2510 & 2526 Lauralynn Drive	19.1	• significant fill materials at crest • slopes flatter than 35°	MODERATE (LOW IF FILL REMOVED AT CREST)	LOW	LOW (VERY LOW IF FILL REMOVED AT CREST)
2404	2526 Lauralynn Drive	19.1	• significant fill materials at crest • lower slopes steeper than 35°	MODERATE	LOW	LOW
2410	2526 Lauralynn Drive	20.4	• significant fill materials at crest • lower slopes steeper than 35°	MODERATE	LOW	LOW
2412	2526 & 2542 Lauralynn Drive	19.1	• fill materials at crest • potential recent slide area on upper slope (seepage and lack of vegetation observed) • slopes steeper than 35°	**HIGH**	LOW	MODERATE
2416	2542, 2558, 2574, 2590, 2602 Lauralynn Drive	17.5	• fill materials at crest • pistol-butted trees on slope • suspected ancient landslide scarp • recent landslide observed on upper slope • slopes steeper than 35°	**HIGH**	VERY LOW*	MODERATE*

2420	2558, 2574, 2590, 2602 Lauralynn Drive	17.9	• fill materials at crest • pistol-butted trees on slope • suspected ancient landslide scarp • recent landslide observed on upper slope • slopes steeper than 35°	**HIGH**	VERY LOW*	MODERATE*
2424	2590 & 2602 Lauralynn Drive	20.4	• fill materials at crest • pistol-butted trees on slope • suspected ancient landslide scarp • recent landslide observed on upper slope • slopes steeper than 35°	**HIGH**	LOW	MODERATE

* No designation for "very low" Spatial Probability Rating is provided in the source table; therefore, designations for "low" Spatial Probability Rating were deferred to when determining Qualitative Risk Ratings.

6.5 Risk Assessment Summary

As described in Table 3, all of the Carmaria Court properties are estimated to have Qualitative Risk Ratings of "moderate", "low", or "very low", with the exception of the following four properties, which are estimated to have Qualitative Risk Ratings of "high" or "very high" and are therefore judged to warrant comprehensive risk assessment (further mitigation recommendations are provided in Section 7.4):

- 2180 Carmaria Court,
- 2194 Carmaria Court,
- 2220 Carmaria Court, and
- 2306 Carmaria Court.

It is noteworthy that the property at 2194 Carmaria Court could see a reduction in Landslide Hazard Likelihood Rating from "moderate" to "low" if the fill materials currently present at the crest of the slope above (at 1588 Merlynn Crescent and 2190 Greylynn Crescent) are removed. This reduction in Landslide Hazard Likelihood Rating would, in turn, reduce the current Qualitative Risk Rating from "high" to "moderate" and therefore negate the recommendation for comprehensive risk assessment.

If comprehensive risk assessment highlights microtopography that could affect the Spatial Probability Rating at any Carmaria Court properties, then additional comprehensive risk assessment may be warranted, as microtopography was not expressly considered in the current assessment, as described in Section 6.3. Microtopography should be assessed during the comprehensive risk assessment at all portions of the subject slope, as variations in topography that may not be reflected in the LiDaR topographic data (and therefore may not have influenced the estimated potential landslide paths shown on Figure 2) could have a positive or negative influence on the Spatial Probability Ratings by lengthening or shortening these landslide paths, or by affecting the relevant slope crest location. In particular, it is judged that Spatial Probability Ratings and therefore Qualitative Risk Ratings could be vulnerable to increases due to microtopography above the following addresses:

- 2252 Carmaria Court,
- 2412 Carmaria Court,
- 2416 Carmaria Court,
- 2420 Carmaria Court, and
- 2424 Carmaria Court.

Surveying of the slope in these areas is recommended, as is further review of landslide hazards, as described in Section 7.4.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 General

Based on the results of the site investigations and subsequent slope stability analyses, it is concluded that the subject site has been and is currently affected by both ancient and active slope instability. The following recommendations should be initiated as soon as possible to improve the slope stability and safety of residents living above and below the subject slope, as well as users of the park and its adjacent roads and creek .

7.2 Ancient Landslide Activity

As described in Section 4.5, multiple suspected ancient landslide scarps were identified within the subject site. The geologic origin of the Westlynn Terrace area is a glacial outwash deposit, which was laid down by proglacial streams as upslope glacial ice melted. For the last 10,000 years, Hastings Creek has been eroding these materials, which could be expected to slough toward the creek channel as the slopes are undercut by erosion. It should be noted that this sloughing would have been more prevalent at the beginning of the Capilano geologic era, when the subject deposits were younger and saturated. Within the current geologic era, this type of movement would be expected to be limited to the creek bank.

At least three suspected ancient landslide scarps are evident on the contours of the topographic map of the subject site, which have crests coincident with the current slope crest, as shown on Figure 2. In addition, the previously noted linear topographic features may be evidence of ancient scarps. These topographic features and more recent tension cracks are noted to be concentric with the suspected ancient landslide scarps at the south portion of the subject site, which may or may not be coincidental.

As described in Section 4.5, multiple first growth stumps (expected to be of the order of at least 500 years old) were observed to be present on the subject slope, including at some areas of the upper, middle, and lower portions of the slope. The presence of such large, intact, and upright stumps suggests that significant landslide activity has not affected the subject slope since these trees existed. Therefore, we expect that the aforementioned ancient landslides occurred more than approximately 500 years ago and the topography we see today could be considered "global equilibrium" - that is, until or unless a failure of upslope water infrastructure triggers a landslide or Hastings Creek erodes the slopes enough to result in further large scale landslides (which is not expected in the foreseeable future). We do not expect that naturally-caused, large-scale, global slope stability problems such as those that occurred earlier in this era would affect the subject slopes at this time.

7.3 Recent and Ongoing Landslide Activity

7.3.1 General

Based on the signs of recent slope movement described in Section 4.5 and the results of static slope stability analyses described in Section 5.3, we conclude that recent and ongoing creep movement of the near-surface, weathered sand and fill materials has been occurring within the subject slope above Carmaria Court. We envisage that under natural conditions (i.e., had development or placement of fill materials at the crest of the slope not occurred), movement of the near-surface, weathered materials would be minimal. However, the significant fill materials and concentrated surface water being introduced at the upper portions of the slope are judged to be increasing slope movement. Fill materials that are acting as a surcharge load at the crest of the slope are envisaged to include large stumps, logs, and, soil pushed over the crest in the 1950's and 1960's during original site preparation (during which time bulldozers, not excavators, were the common site preparation equipment), yard and household debris dumped at the crest by previous and current home owners, and soil purposefully retained at the crest to provide flat back yards. In addition, other surcharge loads would include structures including building additions and sheds that are present near the slope crest. Some first growth stumps and aged logs appear to be locally integral to crest slope stability; however, these stumps appear to be decomposing to the point where this root mass cohesion contribution to slope stabilization may be approaching zero.

Without remediation, downslope movement of these weathered sand and fill materials should be expected to continue and may worsen if fill volumes and directed drainage accumulates and retention structures (natural and man-made) decompose.

7.3.2 Landslides Caused by Water Main Rupture

As referenced in Section 3.1, a landslide occurred in 2006 on the subject slope below 2248 Greylynn Crescent, as shown on Figure 2. This landslide occurred as a result of an upslope water main rupture, which entrained the surficial soils near the crest of the slope and resulted in significant erosion. The entrained materials were mobilized to Carmaria Court below and impacted the nearby residential properties. Remediation of the landslide scar comprised fill placement for erosion protection, revegetation, and construction of a small segmental retention structure on the slope to minimize and retain erosion protection materials.

We understand that the aforementioned water main rupture may have resulted from a short term increase in operating pressure within the water service utility in conjunction with aging infrastructure, which may comprise asbestos concrete pipe (a material which is expected to experience ongoing material degradation over time). Although we understand that the operating pressure within the utility has since been reduced, we envisage that the aging infrastructure may be susceptible to rupture in the future, possibly even without an increase in operating pressure. Therefore, we recommend that the water main pipes upslope of the subject site be replaced with a suitable material. In the mean time, we recommend that the fill materials near the crest of the subject slope are removed and site drainage be connected to the municipal system, as recommended in Section 6.4. This would minimize the water main rupture-induced landslide hazard to the Carmaria Court residential properties below, as well as minimize the potential slope remediation costs that might otherwise be incurred in the event of a future water main rupture.

It is noteworthy that, as described in Section 4.6, evidence of concentrated surface water flow was also observed downslope of 2602 Lauralynn Drive. At the time of our site reconnaissance, the property owner informed us of an upslope water main break that occurred in 2011. A landslide scar was observed mid-slope in this area (as shown on Figure 2), which was estimated to be approximately one to two years old based on the amount of vegetation that had grown over the scar. Based on this estimate and the landslide location, we envisage that it may have been caused by the aforementioned 2011 upslope water break. Minor surficial erosion was noted on the lower slope below; however, no evidence of landslide debris was observed at the lower portion of the slope or near Carmaria Court.

It should be noted that the discussions within this report regarding runout analysis, risk assessment, and slope stability management do not specifically consider the potential for water main rupture-induced landslides.

7.4 Recommended Mitigative Measures and Comprehensive Risk Assessment

Where the landslide Qualitative Risk Ratings are estimated to be "high" or "very high" as described in Section 6.4 (i.e., 2180, 2194, 2220, and 2306 Carmaria Court), we recommend that mitigation of the landslide risk is carried out. Based on the current risk assessment, mitigation of the landslide risk is recommended at the following properties at the crest of the slope:

- 1576 Merlynn Crescent
- 1582 Merlynn Crescent
- 1588 Merlynn Crescent
- 2190 Greylynn Crescent
- 2232 Greylynn Crescent
- 2240 Greylynn Crescent
- 2248 Greylynn Crescent

We recommend that property owners of the above listed Merlynn and Greylynn Crescent properties, as well as the owners of the properties at 2180, 2194, 2220, and 2306 Carmaria Court be notified of the potential landslide risk as described in this report. We recommend that mitigative works be undertaken as soon as possible, designed and field-reviewed by individually hired qualified professionals.

Removal of the crest fill materials at these properties would be expected to reduce the Landslide Hazard Likelihood Ratings at the downslope Carmaria Court properties; however, reduction to acceptable levels may not be possible without removal of all near-surface, weathered soil (i.e., the potential sliding mass), which may not be feasible. However, removal of crest fill materials may reduce the travel angle and, hence, the Spatial Probability Ratings. Further comprehensive assessments at the subject properties at risk are recommended.

The comprehensive risk assessments should be carried out using detailed topographic information to highlight microtopography, which we envisage would be obtained by surveying the slope above the aforementioned four Carmaria Court properties. Each comprehensive risk assessment should include a vulnerability assessment, which would require characterization of the potential landslide affecting each house (i.e., potential volume, depth of debris, velocity of impact, etc.). Re-assessment of the Spatial Probability Rating and Qualitative Risk Rating for each property should follow. If comprehensive risk assessments indicate an unacceptable risk to any Carmaria Court properties, construction of a mitigative structure such as a debris catchment berm, retaining wall, or debris fence may be required.

7.5 Slope Stability Management

As described in Section 7.3.1, downslope movement of the weathered sand and fill materials on the subject slopes should be expected to continue and may worsen if slope conditions do not improve at the crest of the slope. The following recommendations are provided with respect to improving the stability of the slopes within and adjacent to the West Hastings Escarpment, and pertain to all properties located near the slope crest:

- Fill materials and associated retaining walls at and near the crest of the slope should be removed, including retained fills, yard debris, and fill materials that have been pushed or dumped onto the upper portions of the slope. Fill removal and slope recontouring at private property should be carried out under the direction of a qualified geotechnical engineer. It is noteworthy that retaining walls were observed near the crest of the slope at the following properties:
    - 1570 Merlynn Crescent
    - 1576 Merlynn Crescent (observed to be bulging)
    - 1582 Merlynn Crescent (fence above observed to be bowed)
    - 2190 Greylynn Drive (located behind crest)
    - 2208 Greylynn Drive (located behind crest)
    - 2462 Lauralynn Drive (observed to be failing)
    - 2498 Lauralynn Drive
    - 2542 Lauralynn Drive
    - 2590 Lauralynn Drive (observed to be failing)
- No additional surcharge loads, such as fill, retaining walls, or other structures, should be placed on the slope without suitable engineering recommendations regarding slope stability. If property owners want to extend their back yards following fill removal, this could be attained by constructing decks or retaining walls founded upon the unweathered soil at depth and utilizing lightweight or reinforced fill materials to restore grades. Any proposed development at the crest of the slope should undergo site specific geotechnical analysis and design by a suitably qualified professional adhering to the District of North Vancouver's requirements.
- A review of existing structures near the crest of the slope should be carried out by the District of North Vancouver to determine if they were permitted. The observed structures in question include, but are not limited to, the following:

- house addition at 1576 Merlynn Crescent (suspected to be an enclosure beneath a deck),
- two garden sheds at 2208 Lauralynn Drive,
- garden shed at 2462 Lauralynn Drive, and
- deck at 2498 Lauralynn Drive.

- Intercepted water from all houses and hard landscaped surfaces, including rainwater leaders and perimeter drainage, should either be connected to the District of North Vancouver's storm sewer system or another suitable dispersion system. If connection to the municipal storm sewer is not possible, intercepted water should be managed by a system designed by a qualified geotechnical engineer.

- Landscaping water features (such as those observed at 2526 and 2558 Lauralynn Drive) and other potential sources of water near the crest of the slope should be repaired or removed if leakage is observed or suspected.

- Vegetation on the slope should be retained where possible in an effort to reduce surface erosion and soil ravelling.

- The existing slope geometry should not be steepened.

- Excavation work at the toe of the slope should not be carried out without prior review and recommendations from a geotechnical engineer.

Should there be any observed signs of increased ground movement such as recent settlement or new / widened / extended tension cracks, these areas should be immediately reviewed by a qualified professional engineer.

We recommend that a public education and reporting program be initiated to provide property owners at the crest of the subject slope with information regarding slope stability, with emphasis on increased vigilance in areas near the crest and toe of the subject slope. We recommend that this program include the following:

- a brief explanation of slope stability issues and potential risks to properties at the crest and toe of the slope,

- instructions not to dump yard waste or fill onto the upper portions of a slope, or to stockpile materials near the crest (we recommend that an enforcement system is adopted in this regard),

- instructions regarding disposal of intercepted water, as described above,

- information regarding development near the slope crest (including house additions, sheds, decks, hot tubs, etc.) and the associated permitting process required, and

- recommendations pertaining to monitoring their property for signs of slope instability (including tension cracks, ground settlement, foundation cracks, leaning trees, displaced fences, etc.) and reporting any such signs to the District of North Vancouver and a qualified geotechnical engineer. Installation of stake lines parallel to the slope crest are recommended as a simple and effective means of visual slope stability monitoring.

Consideration could be given to including reporting as an element of the monitoring program. If there is a lack of confidence that this monitoring program will be effective, consideration could be given to installing inclinometer(s) in deep drillhole(s) at select locations near the crest of the West Hastings Escarpment slope. These inclinometers could be monitored on an annual basis by a suitably qualified party. In addition, installation of these drillholes would have the benefit of confirming soil strengths at depth, particularly in the areas of concentric topographic features, as described above.

8.0 CLOSURE

This report has been prepared for the sole use the District of North Vancouver and other consultants for this project. Any use or reproduction of this report for other than the stated intended purpose is prohibited without the written permission of Horizon Engineering Inc.

We are pleased to be of assistance to you on this project and we trust that our comments and recommendations are both helpful and sufficient for your current purposes. If you would like further details or require clarification of the above, please do not hesitate to contact us.

For
HORIZON ENGINEERING INC

Karen E. Savage, P.Eng.
President

For
HORIZON ENGINEERING INC

Pamela Bayntun, P.Eng.
Project Engineer

Attachments:

Figure 1	Site Location Plan
Figure 2	Site and Test Hole Location Plan
Figure 3	Photographs 1 and 2
Figure 4	Photographs 3 and 4
Figure 5	Photographs 5 and 6
Figure 6	Photographs 7 and 8
Figure 7	Photographs 9 and 10
Figure 8	Photographs 11 and 12
Figure 9	Slope Profile A
Figure 10	Slope Profile B
Figure 11	Slope Profile C
Figure 12	Slope Profile D
Figure 13	Slope Profile A - Slope Stability Assessment Results
Figure 14	Slope Profile B - Slope Stability Assessment Results
Figure 15	Slope Profile C - Slope Stability Assessment Results

Test Pit Logs (TP13-1 through TP13-3)
Wildcat Cone Penetration Data & Results (WCT12-1 through WCT12-3)

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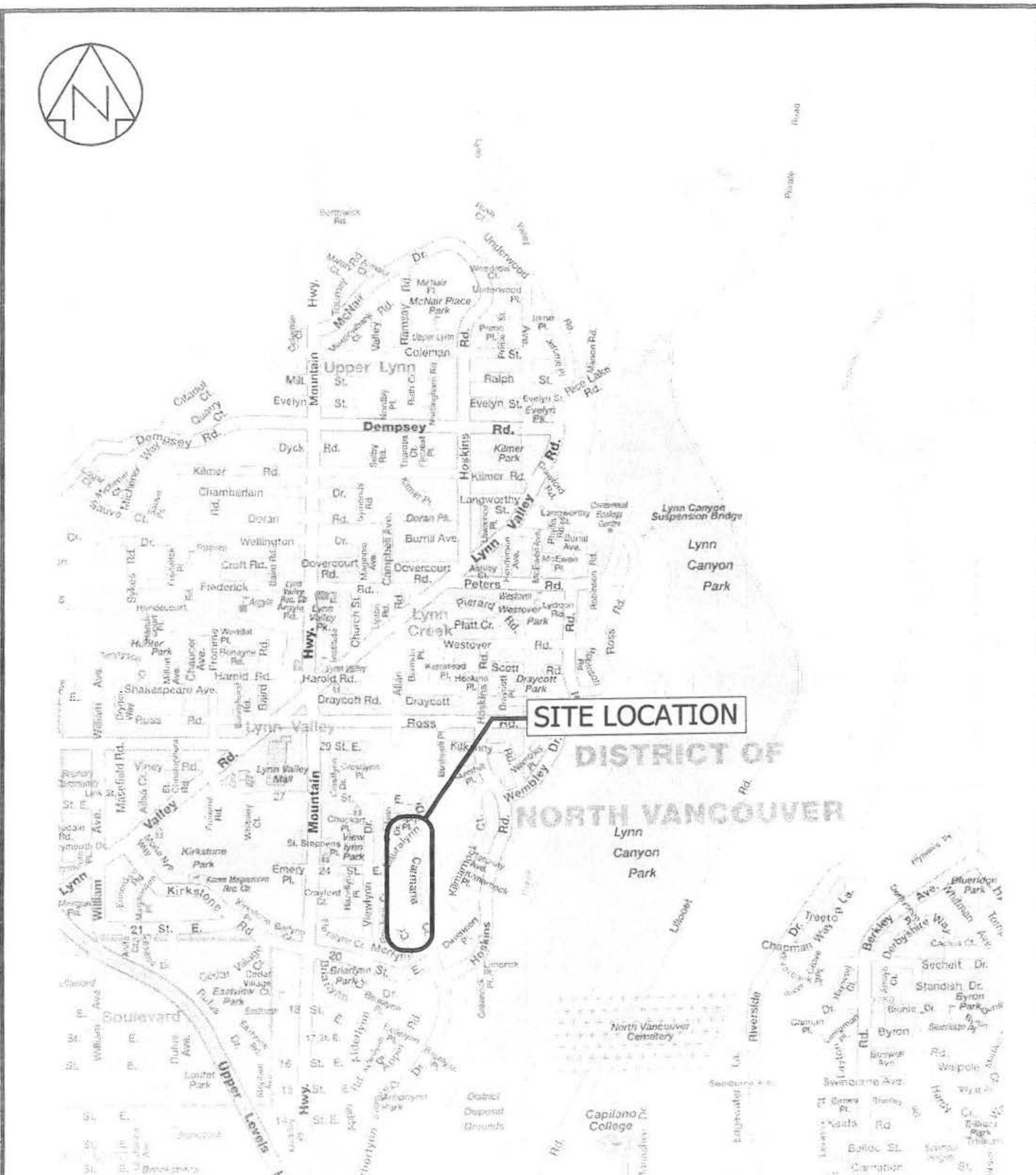

DISTRICT OF NORTH VANCOUVER

WEST HASTINGS ESCARPMENT
SLOPE STABILITY ASSESSMENT

SITE LOCATION PLAN

HORIZON ENGINEERING INC

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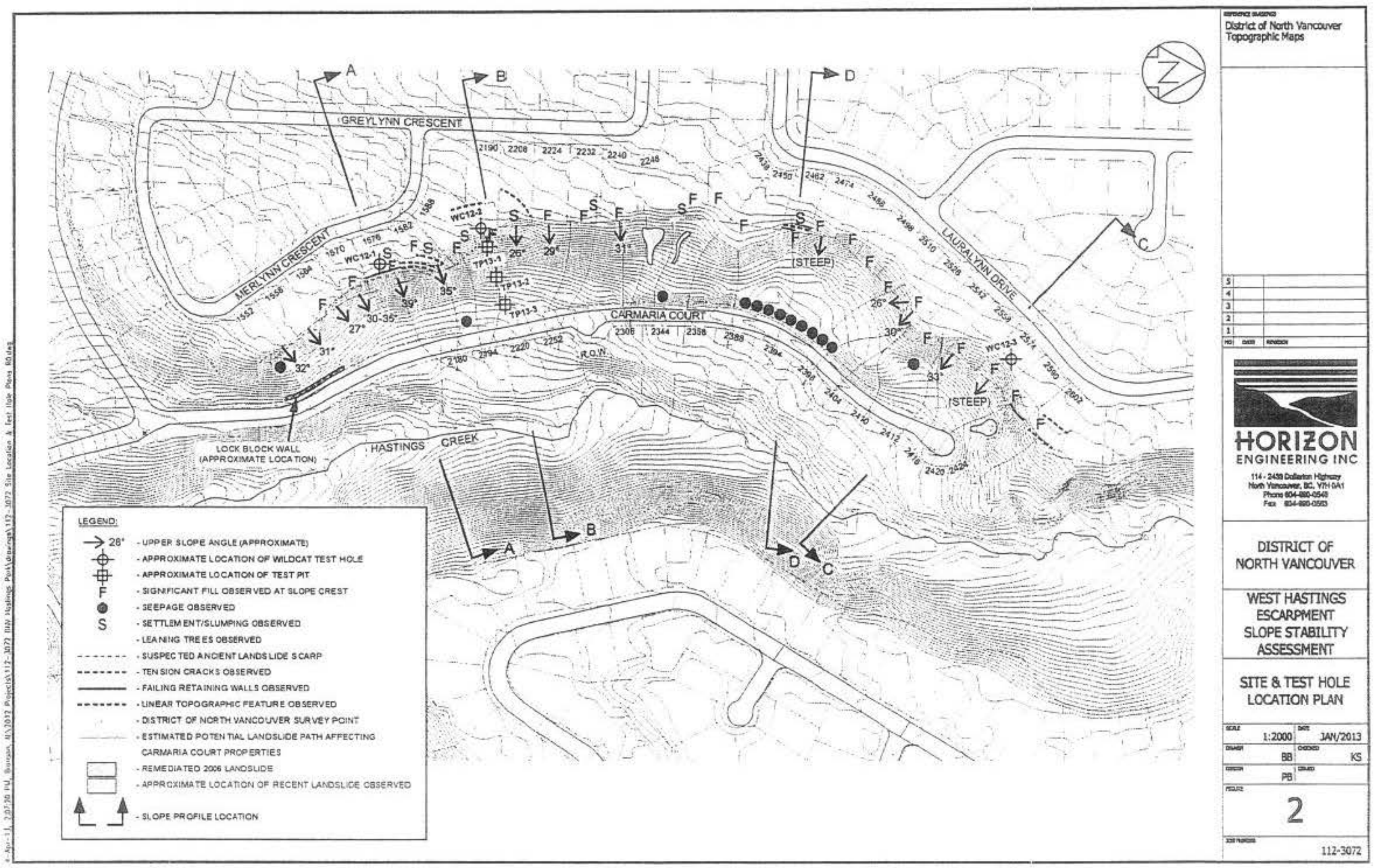
District of North Vancouver Topographic Maps
GREYLYNN CRESCENT
MERLYNN CRESCENT
CARMARIA COURT
LAURALYNN DRIVE
HASTINGS CREEK
LOCK BLOCK WALL (APPROXIMATE LOCATION)
(STEEP)
LEGEND:
26° - UPPER SLOPE ANGLE (APPROXIMATE)
- APPROXIMATE LOCATION OF WILDCAT TEST HOLE
- APPROXIMATE LOCATION OF TEST PIT
F - SIGNIFICANT FILL OBSERVED AT SLOPE CREST
- SEEPAGE OBSERVED
S - SETTLEMENT/SLUMPING OBSERVED
- LEANING TREES OBSERVED
- SUSPECTED ANCIENT LANDSLIDE SCARP
- TENSION CRACKS OBSERVED
- FAILING RETAINING WALLS OBSERVED
- LINEAR TOPOGRAPHIC FEATURE OBSERVED
- DISTRICT OF NORTH VANCOUVER SURVEY POINT
- ESTIMATED POTENTIAL LANDSLIDE PATH AFFECTING CARMARIA COURT PROPERTIES
- REMEDIATED 2006 LANDSLIDE
- APPROXIMATE LOCATION OF RECENT LANDSLIDE OBSERVED
- SLOPE PROFILE LOCATION
HORIZON ENGINEERING INC
DISTRICT OF NORTH VANCOUVER
WEST HASTINGS ESCARPMENT SLOPE STABILITY ASSESSMENT
SITE & TEST HOLE LOCATION PLAN
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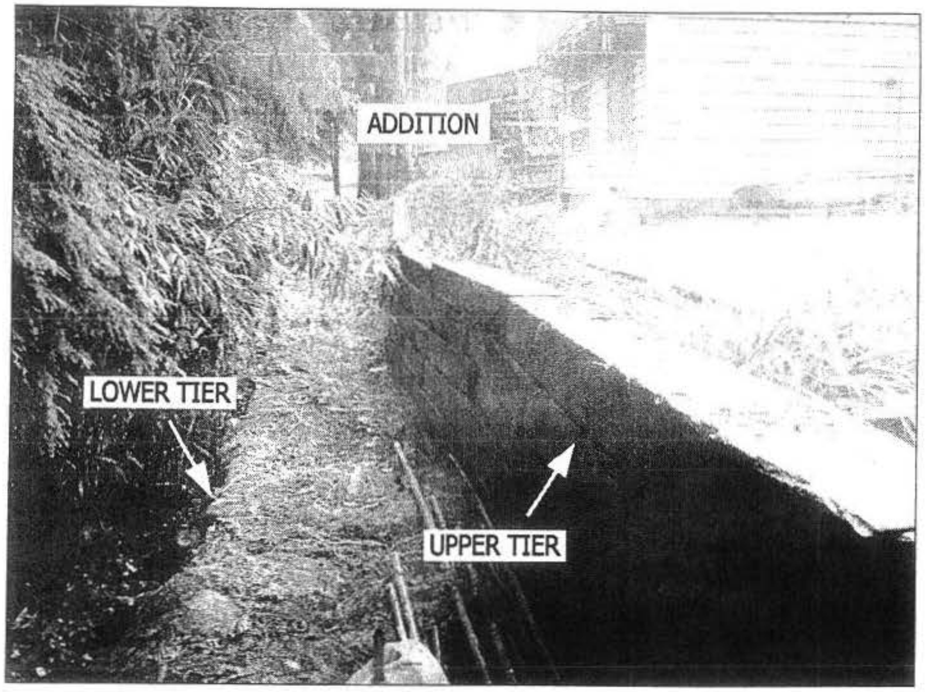
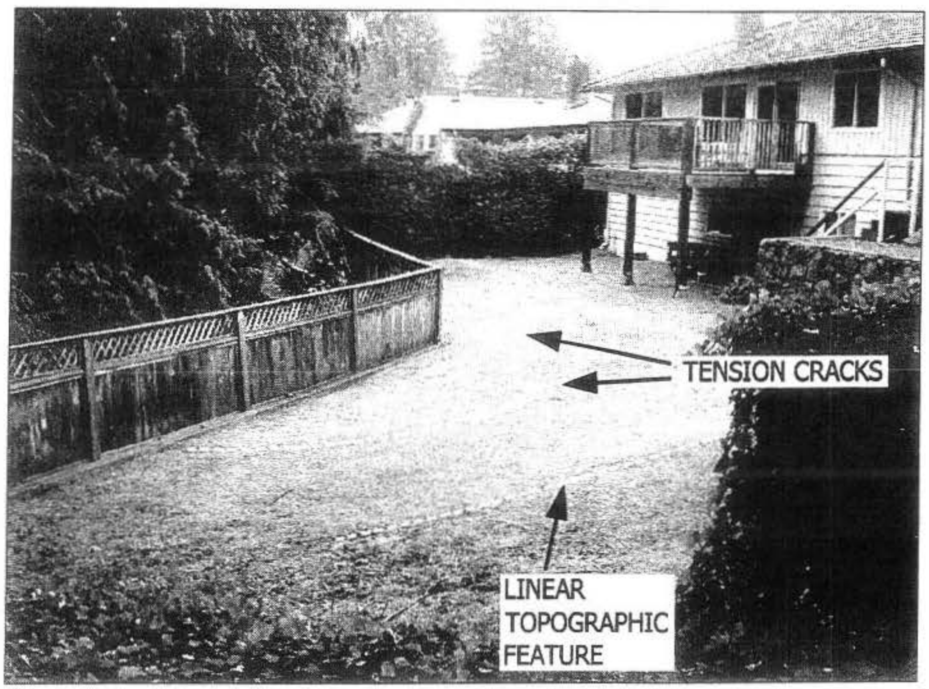


PHOTOGRAPH 1. BULGING TIMBER RETAINING WALL OBSERVED AT 1576 MERLYNN CRESCENT.

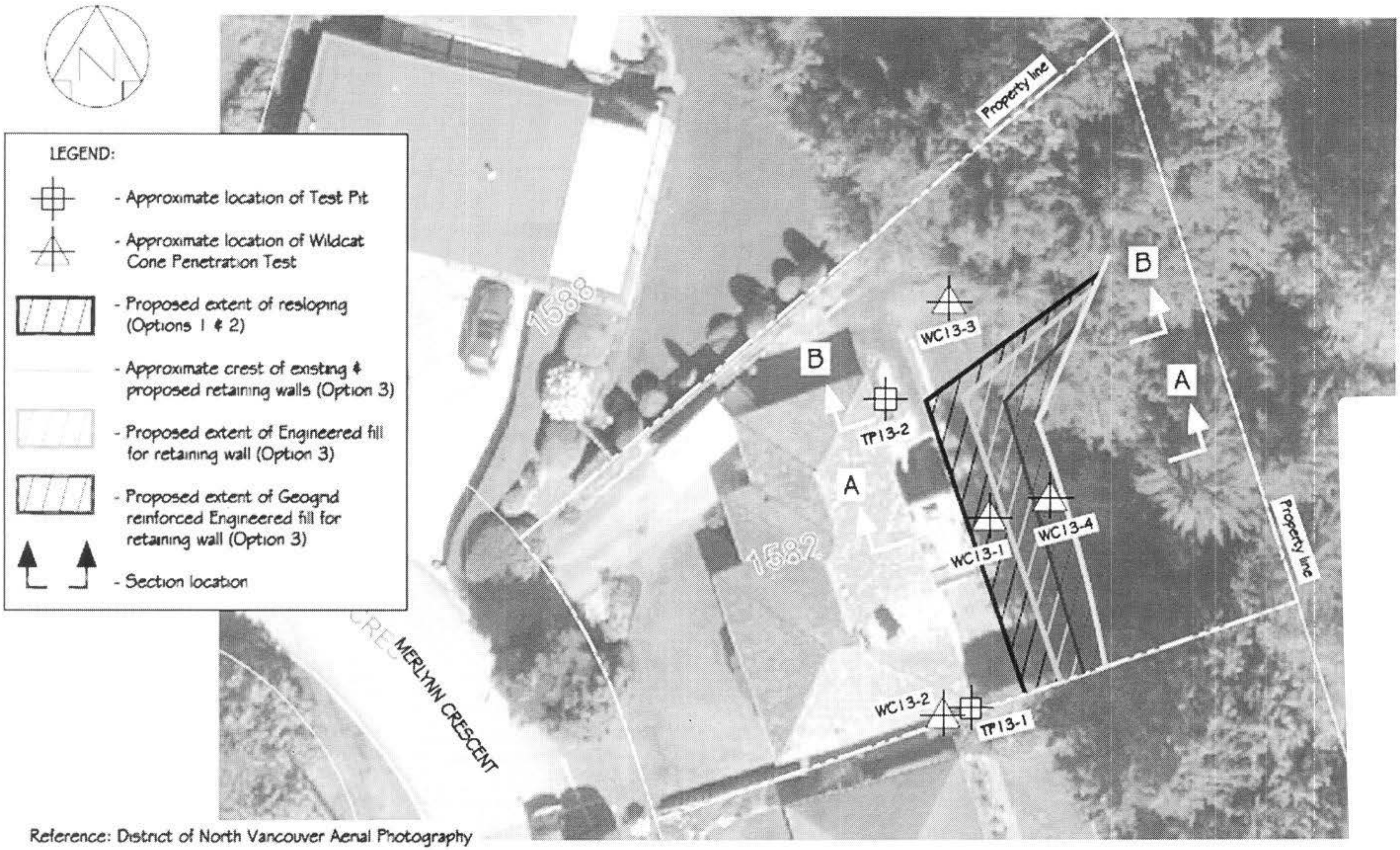


PHOTOGRAPH 2. LINEAR TOPOGRAPHIC FEATURE AND TENSION CRACKS OBSERVED AT 1582 MERLYNN CRECENT.

DISTRICT OF NORTH VANCOUVER	PHOTOGRAPHS 1 AND 2	HORIZON ENGINEERING INC
WEST HASTINGS ESCARPMENT SLOPE STABILITY ASSESSMENT		Scale: NTS; Job No: 112-3072; Date: JAN/2013; FIGURE: 3 Des: PB; Dwn: BB; Chk: KS; Rev:

31-Oct-13, 10:48:13 AM, Bronson, N:\2013 Projects\113-3409 DNV 1582 Merlynn Crescent\Drawings\113-3409 Site Location & Test Hole Plans R1a.dwg

Attachment B



LEGEND:

- Approximate location of Test Pit
- Approximate location of Wildcat Cone Penetration Test
- Proposed extent of resloping (Options 1 & 2)
- Approximate crest of existing & proposed retaining walls (Option 3)
- Proposed extent of Engineered fill for retaining wall (Option 3)
- Proposed extent of Geogrid reinforced Engineered fill for retaining wall (Option 3)
- Section location

Reference: District of North Vancouver Aerial Photography

MR. WILLIAM WALLACE
1582 Merlynn Crescent, North Vancouver, BC

PROPOSED REMEDIAL EARTHWORKS
1582 Merlynn Crescent, North Vancouver, BC

SITE & TEST HOLE LOCATION PLAN

HORIZON ENGINEERING INC

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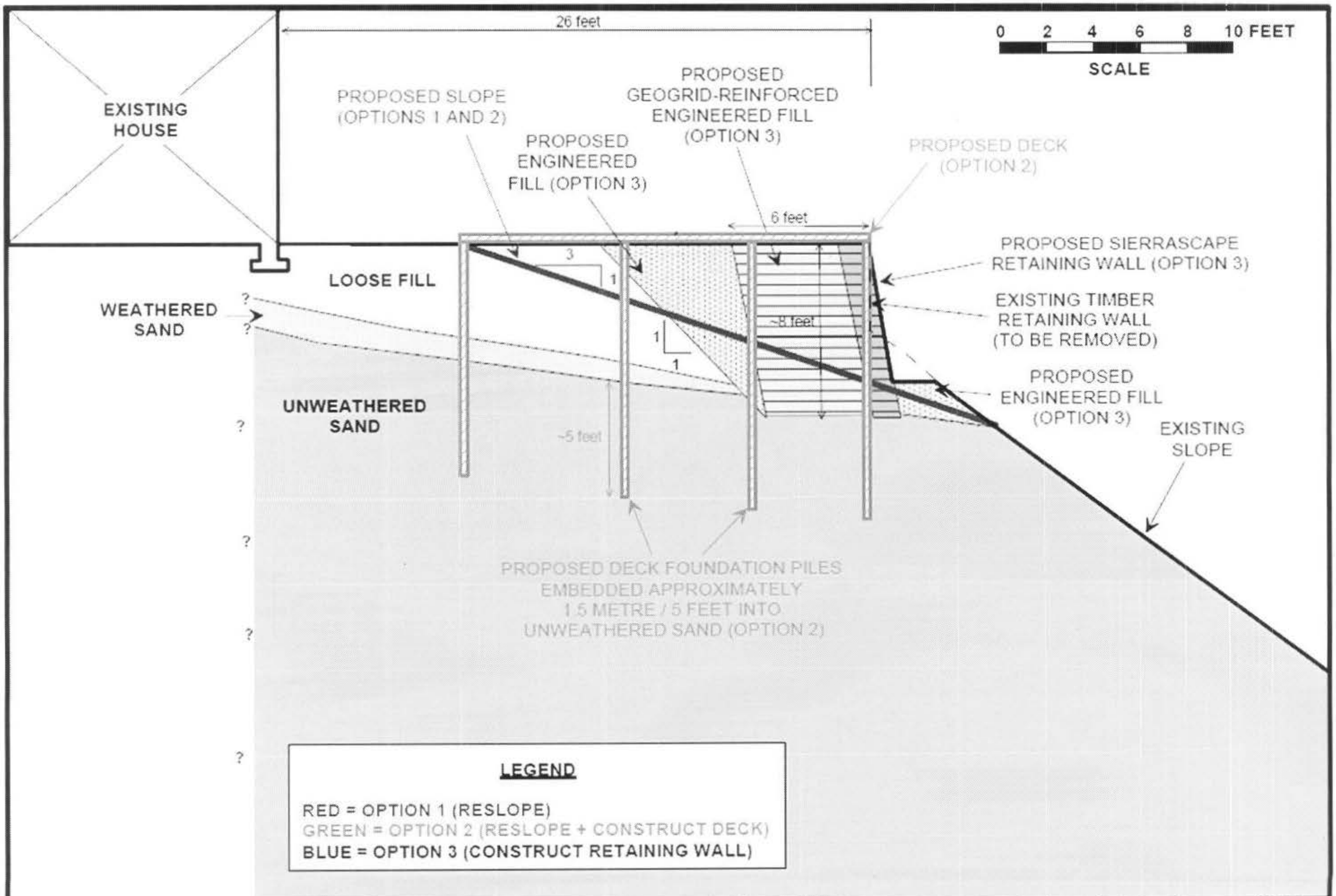

HORIZON ENGINEERING INC

WILLIAM WALLACE
1582 MERLYNN CRESCENT, NORTH VAN.
PROPOSED REMEDIAL EARTHWORKS

SCHEMATIC PROPOSED REMEDIATION OPTIONS SECTION A

Scale	File No	Date	FIGURE
AS SHOWN	113-3409	OCT/13	3
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