# Document Review Services
# Marine Drive Binwall Retaining Structures
## City of White Rock
## White Rock, British Columbia

June 12, 2013

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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Submitted To:
Mr. Glen Gaz
BNSF Railway Company
2454 Occidental Avenue, Suite 2d
Seattle, Washington 98134

By:
Shannon & Wilson, Inc.
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21-1-21664-001

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June 12, 2013

Mr. Glen Gaz
BNSF Railway Company
2454 Occidental Avenue, Suite 2d
Seattle, WA  98134

**RE: DOCUMENT REVIEW SERVICES, MARINE DRIVE BINWALL RETAINING STRUCTURES, CITY OF WHITE ROCK, WHITE ROCK, BRITISH COLUMBIA**

Dear Mr. Gaz:

This letter report presents the results of geotechnical review services we completed for three existing steel binwall retaining structures located along Marine Drive between Cypress and Foster Streets in the City of White Rock, British Columbia (BC) (Figure 1).  Our services included reviewing wall stability reports by Levelton Consultants and EBA Engineering Consultants.  We reviewed a report by Levelton Consultants dated 20 June 2011, a report by EBA Engineering Consultants dated September 1994, and slope inclinometer data from Levelton Consultants dated 2011 and 2013.  Our services also included review of BNSF station maps.  At BNSF’s request, we also reviewed historical documents and agreements pertaining to wall ownership and maintenance responsibility, including a copy of an indenture between BNSF and the City of Surrey dated 14 August 1950.  We provided these services in general accordance with our proposal for services dated 26 March 2013.  You authorized our services via signed request for task order dated 2 April 2013.

## SITE DESCRIPTION

The project site is located along Marine Drive, between Cypress and Foster Streets in White Rock, British Columbia (Figures 2 and 3).  Marine Drive traverses a relatively steep hillside in that area, locally referred to as “the hump.”  Three steel binwall retaining structures support portions of Marine Drive within that area.  Those three retaining structures are located at approximate Mileposts 121.58, 121.70, and 121.76 of the New Westminster Subdivision, Line Segment 56.  Marine Drive reaches a maximum elevation of approximately 65 feet above the BNSF Railway Company (BNSF) tracks in the project area, and the three retaining structures are

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approximately 14 to 20 feet tall. The slope between the BNSF tracks and Marine Drive varies from about 1.75 Horizontal to 1 Vertical (1.75H:1V) to 1H:1V and is heavily vegetated.

## PROJECT HISTORY

We received an e-mail from you on 14 March 2013, in which you asked us to review a report entitled *Geotechnical Assessment Report, Revision 1, Retaining Wall & Slope Stability Review, Hump Hillside, Marine Drive, White Rock, BC*, by Levelton Consultants, Ltd. (Levelton), dated 20 June 2011 (Appendix A). You arranged a meeting for the afternoon of 19 March 2013 with Paul Slack, Manager of Municipal Operations for the City of White Rock (the City), who had commissioned the Levelton study. During that meeting, Paul Slack indicated the City was interested in developing a vegetation management plan for the slope, and potentially repairing or replacing the three retaining structures. He also indicated the walls could be partially or fully within BNSF right-of-way, and that BNSF may own the walls. During our meeting, we requested a copy of a 1994 report by EBA Engineering Consultants, Ltd. (EBA), which also was commissioned by the City, and is referenced in Levelton's June 2011 report. We also requested that the City arrange to have Levelton collect current readings from inclinometer casings they had installed in Marine Drive in 2011.

We received an e-mail from Paul Slack on 19 March 2013, which included a copy of EBA's report entitled *Slope Stability Assessment Report, Marine Drive between Foster Street and Cypress Street, White Rock, B.C.*, dated September 1994 (Appendix B). We received an e-mail from Paul Slack on 25 April 2013, which included a letter by Levelton, entitled *Summary of Inclinometer Readings – April 2013, Hump Hillside, Marine Drive, White Rock British Columbia*, dated 25 April 2013 (Appendix C).

During a visit to the BNSF Engineering Department office on 25 March 2013, we received copies of a station map entitled *Great Northern Railway, White Rock No. 1, New Westminster District, B.C., Sec's 10 & 11, T.1.*, dated August 1964, revised August 1966 (Figures 2 and 3). That document includes BNSF right-of-way boundaries, platted City street and parcel boundaries, and other features of interest to BNSF and the City, including three retaining walls in the vicinity of Marine Drive.

## OBSERVED SITE CONDITIONS

On 19 March 2013, after our meeting with the City, we visited the site with you to observe conditions. Visible portions of the retaining structures appeared to be relatively well aligned, with no obvious large deformations. We observed wood cribbing at the ends of the steel binwalls in several locations. The wood cribbing appeared to be deteriorated and misaligned with the steel walls in some locations. We observed surface settlement of approximately ½ inch and tension cracks in the roadway pavement that were open approximately 1/8 inch in several locations.

## GEOTECHNICAL REVIEW

Levelton's report dated 20 June 2011 (Appendix A) presents a geotechnical assessment of one of the retaining structures, identified as *Subject Bin Retaining Wall (RW1)*. In their report they describe vegetated landslide scarps along the slope and near the base of the retaining wall, as well as tension cracks and localized settlement within Marine Drive. Levelton conducted subsurface explorations in Marine Drive near RW1 and another nearby retaining structure. They installed slope inclinometer casings to measure ground displacement and a monitoring well to measure the depth to groundwater. Levelton conducted stability analyses and concluded that RW1 is marginally stable under static loading conditions, and unstable under seismic loading conditions.

We reviewed the 25 April 2013 letter by Levelton, in which they summarize and interpret inclinometer readings at the site. They concluded that the inclinometers indicate cumulative displacements in "variable and opposite directions in both the A and B orientations." Measured displacements ranged from 0.37 mm to 3.50 mm with the largest displacement measured parallel to Marine Drive. The largest measured downslope displacement was 2.45 mm, with 0.63 mm measured in the upslope direction in the same inclinometer. Similarly, in the other inclinometer, a measured downslope displacement of 2.23 mm is coupled with a measured upslope displacement of 2.77 mm. Levelton suggests the displacements could be attributable to settlement of the loose fill present in the upper slope, or possible shallow slope/wall deformations. In our opinion, the displacements measured to date are inconclusive with regard to a pattern of downslope movement.

We reviewed the September 1994 report by EBA (Appendix B). EBA conducted subsurface explorations, installed monitoring wells in the explorations, collected groundwater measurements, and conducted laboratory testing on collected soil samples. Based on the subsurface conditions they observed in their explorations, EBA conducted stability analyses at the tallest retaining wall along Marine Drive and at a location with no wall. They concluded that the slope and wall are marginally stable under static loading conditions. EBA did not conduct seismic stability analyses.

## STATION MAP REVIEW

The station map (Figure 2) shows the tracks, right of way boundary, city streets, parcel boundaries, utilities, and other railway and City features. Retaining walls shown on the map match the approximate locations of the three walls at the project site. To compare the features on the map to present site features, we overlaid the map onto an aerial photograph (Figure 3). The photo and map do not match exactly, and the features depicted in the map should be considered approximate.

The approximate location of the right-of-way boundary is at the south edge of the sidewalk along the south side of Marine Drive. Though the three retaining structures are not clearly distinguishable in the aerial photo, those on the station map appear to correspond to the approximate locations described in the reports by Levelton and EBA.

There are many notes and markings on the map that allude to the history of the site. Several of the notes reference permits, contracts, and memos. Several of the notes and markings we identified include:

- Marine Drive is labeled Washington Avenue in the map.
- There are notes that indicate Washington Avenue was platted south of present-day Marine Drive, just north of BNSF tracks.
- Notes indicate Washington Avenue was vacated via memo: ***Memo No. 70, March 31, 1927.*** The same note appears to be lined-out elsewhere on the map.
- The area between sidewalk on south side of Marine Drive and lower slope is labeled as: ***Permit to Dist. Of Surrey L.D. Cont. No. 40862.***
- A note on the map indicates: L.D. Cont. No. 40862, Aug. 14, 1950, Ry. Co. grants Dist. of Surrey right to enter upon and construct bulkhead and road support to Washington Ave. (District to furnish conc. Pipe for three culverts).
- Separate note indicates: Established by Agreement 12-17-26 Memo No. 70

We also noted a marking near the station map title block that says *Superseded V/M~1646-4-2, Profile~H11-1, R/W Map~RH11-1.*

## DOCUMENTS PROVIDED BY BNSF

At your request we contacted the firm Jones, Lang, LaSalle to inquire about whether they had copies of any of the above mentioned agreements. They researched their files and provided a copy of an indenture dated August 14, 1950, referred to as agreement No. 40862 in the station map. A copy of that indenture is included as Appendix D of this report.

That indenture grants the Corporation of the District of Surrey (City) an easement to "construct timber cribs, pile bulk heads, and sloping of lands for the purpose of giving lateral support to Washington Avenue." It also includes a clause in which the City agrees to "maintain the said works at all times and in such manner so as not to create any possible hazard, detriment or interference to the lands and operations of the Railway Company contiguous thereto."

## CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the geotechnical reports by Levelton and EBA, it is our opinion that those studies reached reasonable conclusions given the soil conditions described. Each described tension cracks and ground settlement that are consistent with conditions we observed at the site.

Based on our review of the August 14, 1950 indenture between the City and BNSF, we infer that the City intended to construct retaining structures and regrade the slope to support Washington Avenue, which is present-day Marine Drive. In their report, Levelton states that the three steel binwalls appear to have been constructed by 1963. They do not state whether other retaining structures were visible in earlier photos. We do not have record of what types of retaining structures were constructed and when. It is likely that the three steel binwalls were constructed under the indenture between the City and BNSF, and that binwall stability and maintenance remains the responsibility of the City.

Two previous reports commissioned by the City indicate that the slope and walls are marginally stable under static conditions. One of those reports indicates the walls are unstable under seismic loading.

Both EBA and Levelton recommended that the City complete additional work to improve stability along Marine Drive. We support Levelton's recommendation to replace the wood crib retaining walls adjacent to the steel binwall and to complete a structural examination of the metal facing and connections, as well as a corrosion assessment to estimate the remaining design life of the bin walls. We also support their recommendation that the condition of sanitary and storm sewer mains along Marine Drive be reviewed, or evaluated for the presence of water leakage.

We observed deformation of the wall faces which, in our opinion, is not likely attributable to slope movements or downslope displacement at the base of the wall, but may be due to other factors.

In our opinion the inclinometer measurements to date by Levelton are inconclusive.

Based on our review of the EBA and Levelton reports, and conditions we observed at the site, we recommend the City develop a program to measure the inclinometers periodically to assess the magnitude, direction, and rate of change of movements with time. We recommend the City install additional inclinometers at the other steel binwalls and representative areas of the slope without walls, and monitor them periodically for the same purposes.

We recommend the City develop a draft action plan to improve the stability of the steel binwalls and slope based on their current understanding of the site. This plan could be revised over time as data from existing inclinometers, new borings, and new inclinometers are evaluated.

We recommend that the City monitor the condition of the pavement above and near the three steel binwalls, and that any cracks be sealed such that surface water is not permitted to infiltrate the wall backfill. We recommend that irrigation, sprinkler systems, and other sources of potential uncontrolled water release be prohibited near the top of the three steel binwalls and adjacent areas.

## LIMITATIONS

This letter report was prepared for the exclusive use of BNSF for specific application to the, Marine Drive Binwall Retaining Structures Stability Review Services Project, located in the City of White Rock, British Columbia. It should be made available to prospective contractors and/or the contractor for information on factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the boring logs, or discussions of subsurface conditions included in this report. The purpose of this letter report is to assist BNSF in understanding the geotechnical reports prepared by Levelton and EBA regarding stability of the Marine Drive Binwall Retaining Structures.

The analyses, conclusions, and recommendations presented in this letter report are based on site conditions we observed and our review of geotechnical reports by Levelton and EBA. Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this letter report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this letter report was prepared. We make no other warranty, either express or implied.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples from borings or from interpreting subsurface conditions from existing boring logs. Such unexpected conditions frequently require that additional expenditures be made to attain properly constructed projects. Therefore, some contingency fund is recommended to accommodate such potential extra cost.

This letter report does not include any environmental assessment or evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air on or below or around the site.

Shannon & Wilson has prepared Appendix E, “Important Information About Your Geotechnical/Environmental Report,” to assist you and others in understanding the use and limitations of this letter report.

If you have any questions or comments about the contents of this letter report, please contact Tyler Stephens at (206) 695-6915 or Will Hultman at (206) 695-6816.

Sincerely,

**SHANNON & WILSON, INC.**

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
G. J. BUECHEL
# 151485

June 12, 2013

Gerard J. Buechel, P.Eng.
President

TJS:WAH:GJB/tjs

Enc: References
- Figure 1 – Vicinity Map
- Figure 2 – BNSF Station Map Excerpt (3 sheets)
- Figure 3 – Aerial Photo with BNSF Station Map Overlay (3 sheets)
- Appendix A – Geotechnical Report – Levelton Consultants, Ltd. (Levelton) (2011)
- Appendix B – Geotechnical Report – EBA Engineering Consultants, Ltd. (EBA) (1994)
- Appendix C – Geotechnical Summary of Inclinometer Readings, Levelton Consultants, Ltd. (Levelton) (2013)
- Appendix D – Copy of Indenture No. 40862 Dated August 14, 1950
- Appendix E – Important Information About Your Geotechnical/Environmental Report

# REFERENCES

EBA Engineering Consultants Ltd. (EBA), 1994, Slope stability assessment report, Marine Drive between Foster Street and Cypress Street, White Rock, B.C.: Report prepared by EBA Engineering Consultants Ltd., Vancouver, B.C., file no. 0806-86535, for the City of White Rock, September.

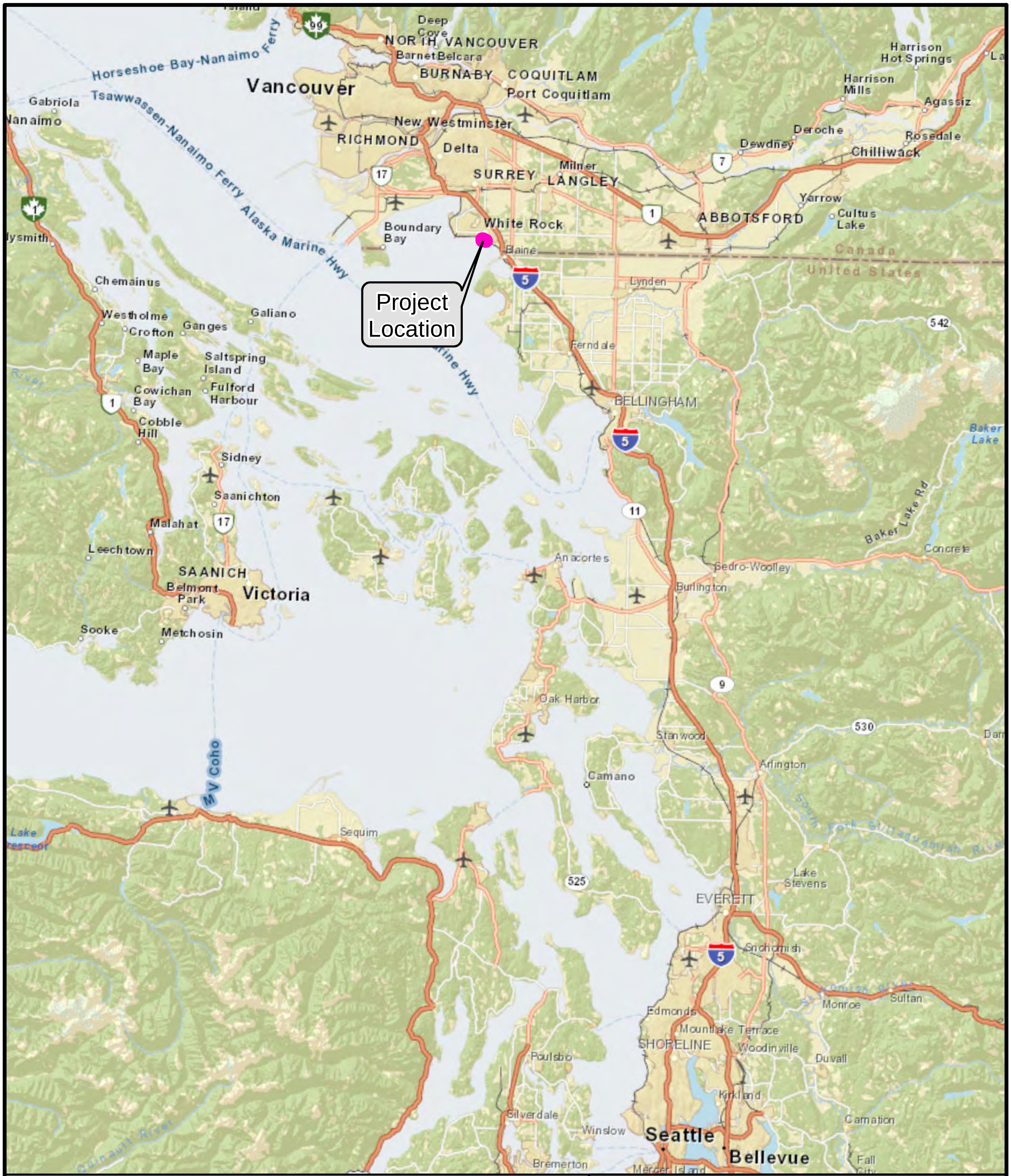
Great Northern Railway, 1964, White Rock no. 1, New Westminster District B.C., sections 10 & 11, T. 1: St. Paul, Minn., Office of Chief Engineer, August.

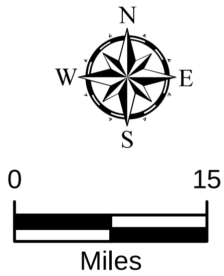
Great Northern Railway Company (GNRC) and The Corporation of the District of Surrey, British Columbia (Surrey), 1950, Construction of timber cribs and pile bulk heads and sloping of lands on Great Northern Railway Company property to provide lateral support to Washington Avenue: Indenture between GNRC and Surrey, 5 p., August 14.

Levelton Consultants, Ltd. (Levelton), 2011, Geotechnical assessment report revision 1, retaining wall & slope stability review, hump hillside, Marine Drive, White Rock, BC: Report prepared by Levelton Consultants, Ltd., Abbotsford, BC, file FV11-0658-00, for The Corporation of the City of White Rock, June.

Levelton Consultants, Ltd. (Levelton), 2013, Summary of inclinometer readings – April 2013, Hump Hillside, Marine Drive, White Rock, British Columbia: Report prepared by Levelton Consultants Ltd., Surrey, B.C., file no. FV11-0658-01, for the Corporation of the City of White Rock, White Rock, B.C., April 25.

White Rock Online, 2001, White Rock history: Available: http://www.whiterockonline.com/communityhistory.htm, accessed April, 2013.





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BNSF Railway Stability Review Services
City of White Rock, White Rock, British Columbia

## VICINITY MAP

May 2013

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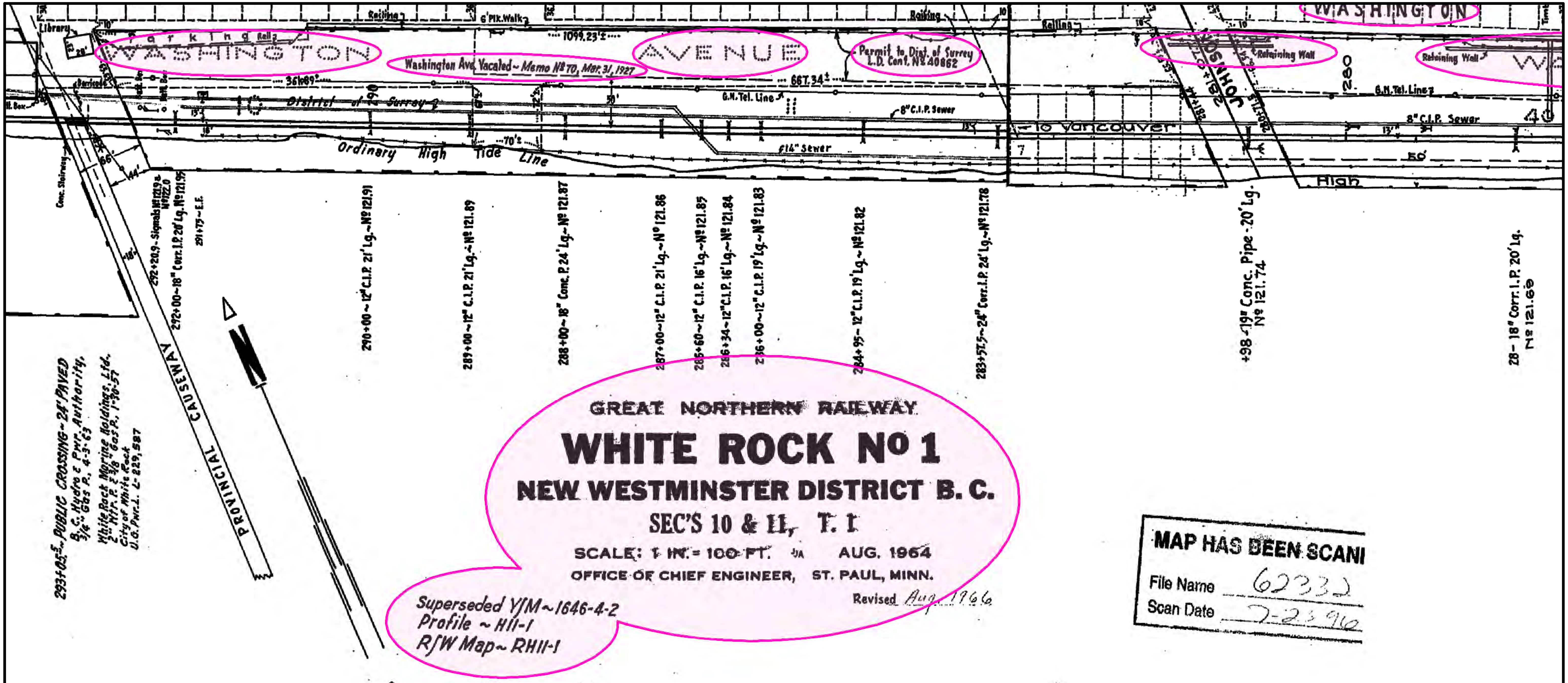
**FIG. 1**



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Approximate Scale in Feet

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N E S W

0 50 100

Feet

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## BNSF STATION MAP EXCERPT

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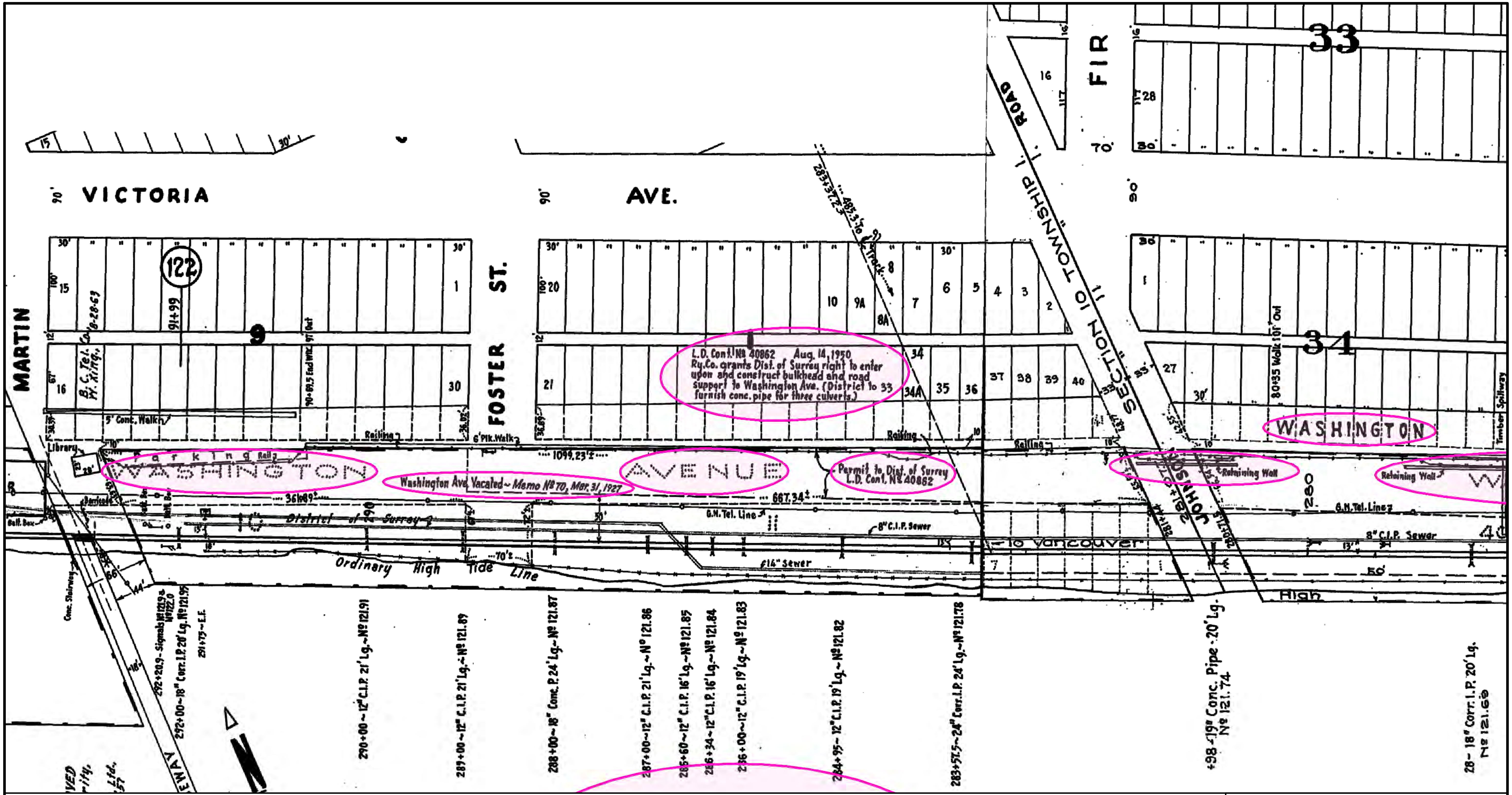
**FIG. 2**
Sheet 1 of 3



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Feet

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## BNSF STATION MAP EXCERPT

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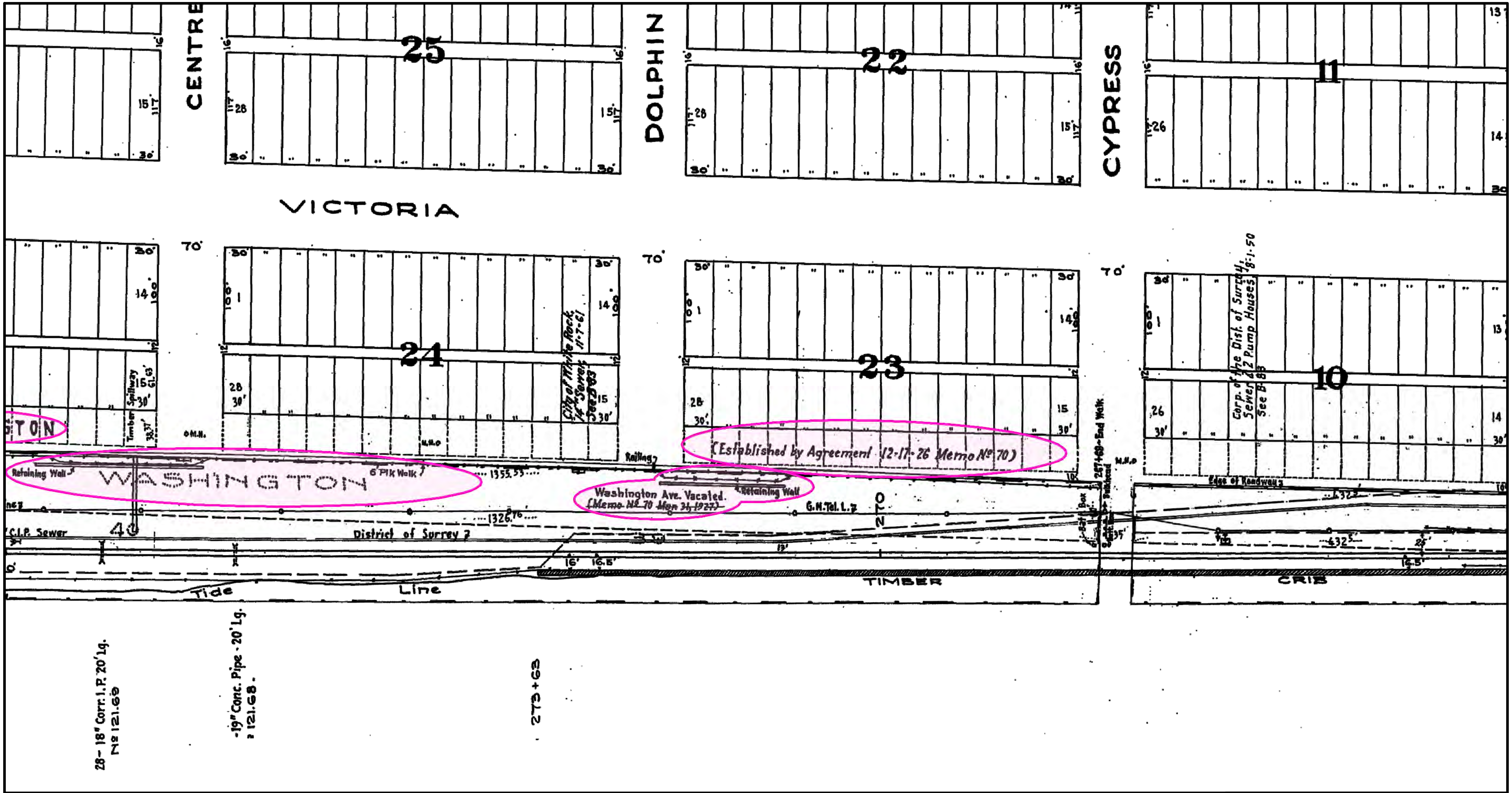
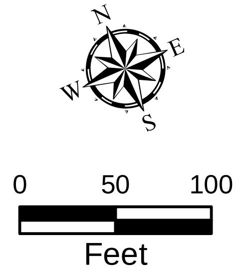
**FIG. 2**
Sheet 2 of 3





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## BNSF STATION MAP EXCERPT

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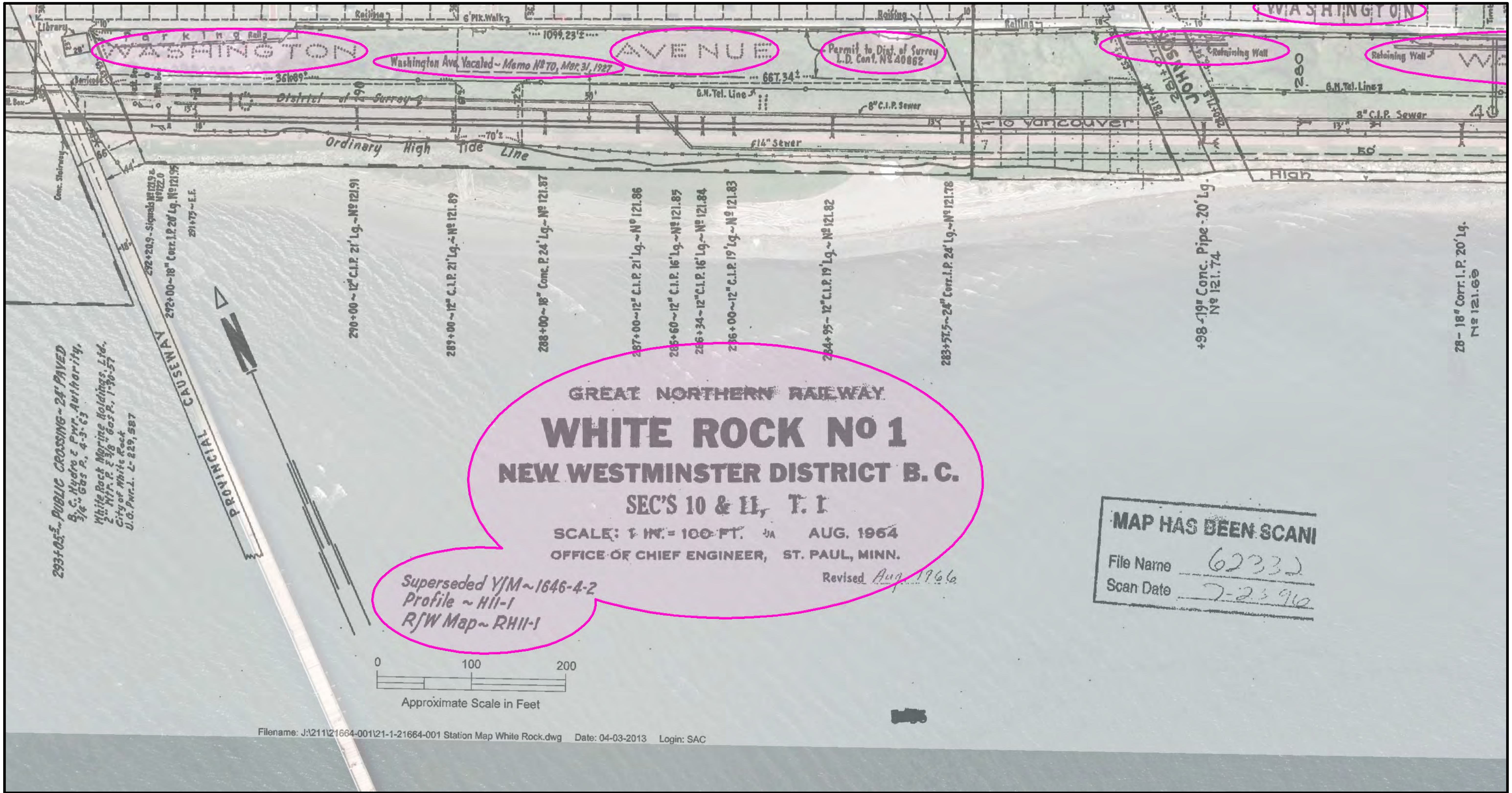
**FIG. 2**
Sheet 3 of 3



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## AERIAL PHOTO WITH BNSF STATION MAP OVERLAY

May 2013

21-1-21664-001

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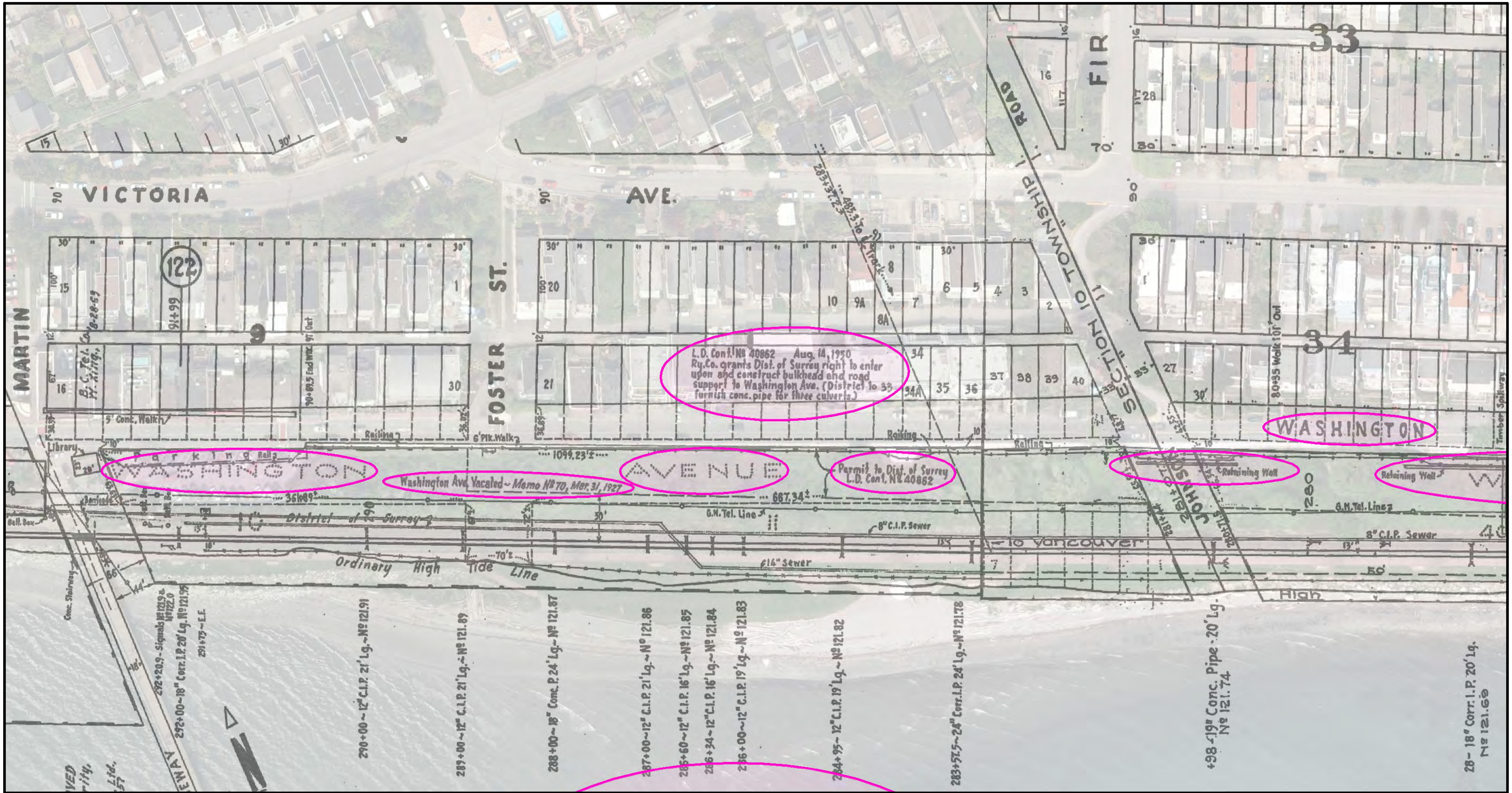
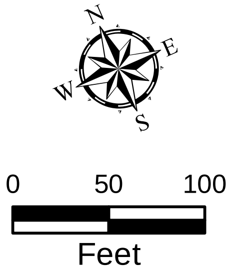
FIG. 3
Sheet 1 of 3





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## AERIAL PHOTO WITH BNSF STATION MAP OVERLAY

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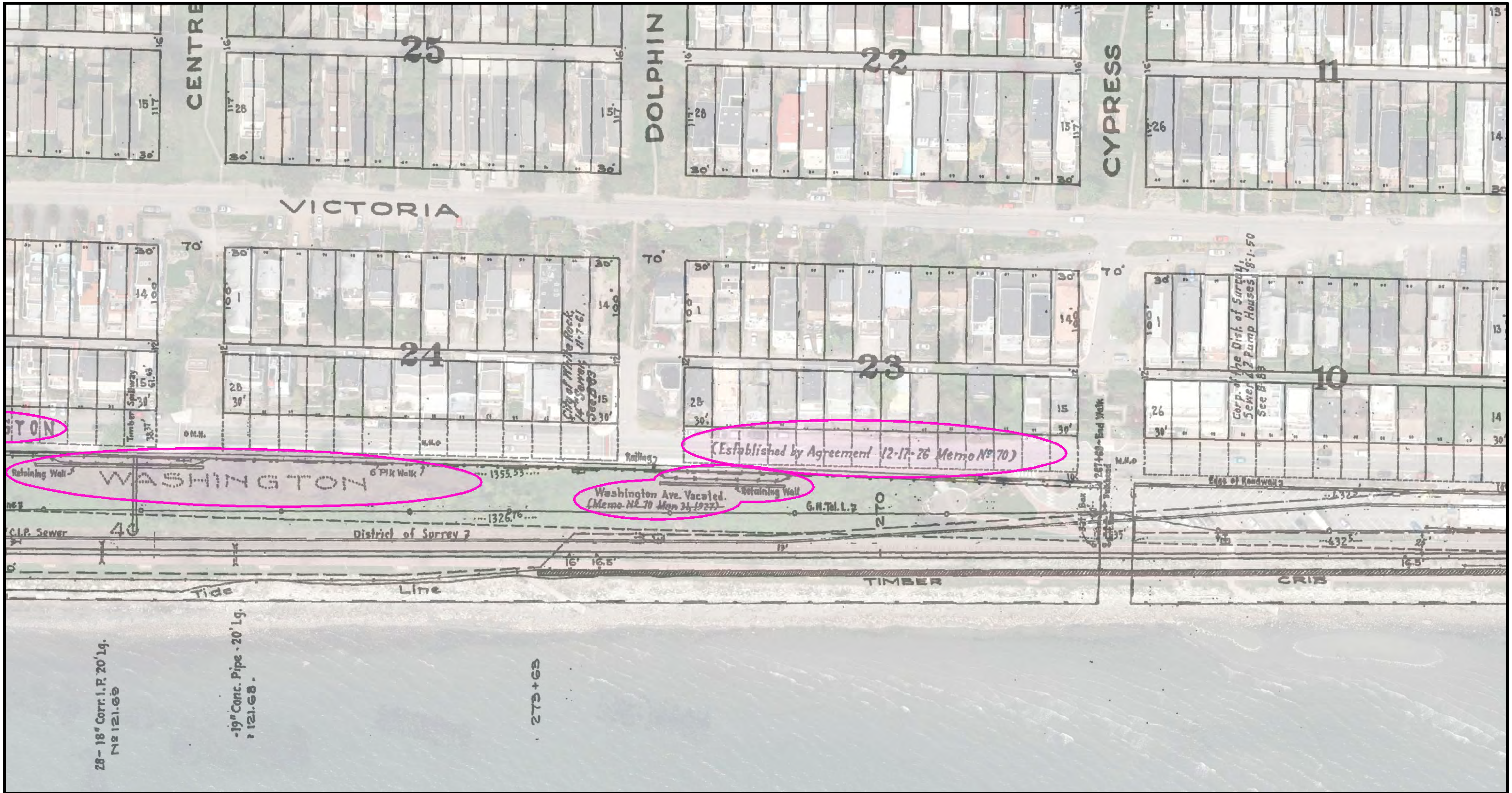
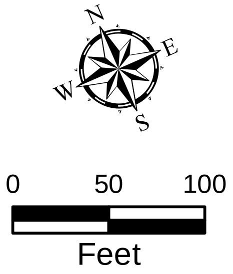
**FIG. 3**
Sheet 2 of 3





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## AERIAL PHOTO WITH BNSF STATION MAP OVERLAY

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**FIG. 3**
Sheet 3 of 3

# APPENDIX A

# GEOTECHNICAL REPORT – LEVELTON CONSULTANTS, LTD. (LEVELTON) (2011)

# APPENDIX A

## GEOTECHNICAL REPORT – LEVELTON CONSULTANTS, LTD. (LEVELTON) (2011)

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# GEOTECHNICAL ASSESSMENT REPORT
## REVISION 1
# RETAINING WALL & SLOPE STABILITY REVIEW
# HUMP HILLSIDE, MARINE DRIVE
# WHITE ROCK, BC

Prepared for:

**The Corporation of the City of White Rock**
877 Keil Street
White Rock, BC
V4B 4V6

Attention: Mr. Robert Thompson, MA, MCIP

Prepared by:

**Levelton Consultants Ltd.**
110 – 34077 Gladys Avenue
Abbotsford, BC
V2S 2E8

20 June 2011

File: FV11-0658-00

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# 1. INTRODUCTION

Levelton Consultants Ltd. (Levelton) presents herein our revised Geotechnical Assessment Report to the Corporation of the City of White Rock (the Client) for the review of an existing metal bin retaining wall and surrounding hillside. The scope of work for this project is provided in the Levelton proposal PR10-1377-00, revised 16 March 2011. This revised report was prepared to address vegetation maintenance considerations for the hump hillside and to provide further explanation pertaining to the seismic slope stability analysis conducted for this report. The report recommendations provided herein supersede the recommendations provided in Levelton's previous report dated 09 June 2011.

The subject bin retaining wall (RW1) is located on a steep hillside locally referred to as the "hump" between the Burlington Northern Sante Fe (BNSF) railway tracks and Marine Drive. The location of the hump hillside is presented on Figure 1 in attached to this report.

RW1 begins at the intersection of Centre Street and Marine Drive and extends approximately 50 m to the west. RW1 supports the eastbound lane and south sidewalk of Marine Drive. The stability of RW1 is essential to the BNSF railway, Marine Drive and underground utilities along Marine Drive that service the City of White Rock. The wall location is indicated on Figure 2 attached to this report.

Levelton previously conducted a cursory review along the top of the RW1 and hump hillside as it pertained to the assessment of whether existing trees should be removed. This report has been prepared to address seismic stability of RW1 and the general stability of the surrounding hump hillside.

Our scope of work for this project included the following:

- Gather and review all available documented information that pertains to the hillside and RW1;
- Conduct a site reconnaissance to review present site conditions of RW1 including measurements of slope features and general slope profile, and soil and groundwater conditions at the surface level;
- Drill three 15 m deep test boreholes at select locations, conduct Standard Penetration Testing (SPT) at 0.75 to 1.5 m intervals, and install two slope inclinometers to allow for future slope displacement monitoring;
- Conduct two sets of inclinometer readings after installation of the inclinometers to monitor slope movement;
- Install one standpipe piezometer to establish and monitor the groundwater table position;
- Establish the soil parameters based on the soil conditions encountered in the drilled boreholes and relevant literature review;
- Analyze the present stability of RW1 and slope under static and seismic loading and future stability after implementation of the proposed remedial work plans, if such remedial works are recommended. The analysis was completed using a limit-equilibrium and finite element methods;

- Prepare a geotechnical report summarizing the findings of the geotechnical assessment; and
- Attend a meeting with the Client to present the findings of the assessment.

Our scope of services did not include assessment of the soil or groundwater at the project location with respect to environmental or corrosion considerations.

# 2. SITE INVESTIGATION

## 2.1 LEVELTON INVESTIGATION

The Levelton site investigation was comprised of a desktop study, site reconnaissance and subsurface exploration completed in April 2011. Further details pertaining to the Levelton site investigation is presented in the following sections.

### 2.1.1 Site Reconnaissance and Desktop Study

A site reconnaissance was conducted on April 1, 2011 by two Levelton geotechnical engineers. The reconnaissance included traverse of the slope below RW1, traverse of the base of RW1, review of the existing wood crib retaining walls on the surrounding hump hillside, and review of pavement and sidewalk at the top of the wall. The site reconnaissance also included review of the slope to the east and west of RW1. Measurements of RW1, slope features and general slope profile, and review of soil and groundwater conditions at the surface level was conducted during the site reconnaissance.

The desktop study of the subject site was comprised of a review of the following:

- Aerial photographs from 1940 through 2004;
- As-built drawings for underground utilities in the area of the subject site provided by the Client;
- EBA Engineering Consultants Ltd. (EBA) report "Slope Stability Assessment Report, Marine Drive between Foster Street and Cypress Street, White Rock, B.C." dated September 1994, File 0806-86535 provided to Levelton by the Client;
- Historical information regarding previous instability at the subject site; and
- Geotechnical research papers related to the shear strength of the local soils.

Levelton attempted to locate design or as-built drawings of the existing retaining walls. However, the drawings were not available from BNSF or the City of White Rock for this study.

### 2.1.2 Subsurface Exploration

The subsurface exploration was conducted on April 5, 2011. It included three mud-rotary boreholes (BH11-01, BH11-02 and BH11-03) advanced to a depth of approximately 15 m below existing ground surface. The boreholes were located based on observations made during the site reconnaissance. Boreholes BH11-01 and BH11-02 were advanced along the eastbound lane of Marine Drive, setback approximately 6 m from RW1. BH11-03 was also advanced in the eastbound lane of Marine Drive, approximately 50 m to the west of RW1. BH11-03 was setback from the slope a distance of approximately 6 m. The borehole locations are identified on Figure 2 attached to this report.

To assess the *in-situ* density of the site soils, Standard Penetration Tests (SPTs) were conducted in the boreholes in accordance with ASTM D 1586. The SPTs were conducted at intervals of 0.75 m to a depth of approximately 6 m below the existing grade. Below 6 m depth, SPTs were conducted at intervals of 1.5 m to a depth of 15 m. The SPT N-values are presented on the soil logs.

The soil and groundwater conditions encountered at the boreholes were logged in the field by a member of our geotechnical staff. Disturbed split barrel soil samples were collected from the boreholes for visual classification, sieve analysis and moisture content determination purposes. Detailed descriptions of the soil and groundwater conditions observed at the boreholes and soil moisture contents are provided on the attached soil logs in Appendix A. The laboratory test results are provided in Appendix B.

Inclinometer casings were installed in the boreholes BH11-01 and BH11-03 to monitor slope movement. A piezometer was installed BH11-02 to monitor groundwater level. The piezometer installation details are provided on the soil log for BH11-02. Flush-mounted covers were installed on all three boreholes at the completion of the drilling investigation.

Levelton conducted two sets of inclinometer and piezometer readings after completion of the drilling investigation. The first inclinometer reading (the baseline reading) was taken on April 13, 2011. The second reading was taken on May 3, 2011. Results of the inclinometer readings are presented in graphical form in Appendix C.

Two piezometer groundwater level readings were taken on April 13 and May 3, 2011 respectively. The groundwater level readings are presented on the soil log for BH11-02.

## 2.2 INVESTIGATION BY OTHERS

EBA was retained by the City of White Rock in 1994 to conduct an assessment of potential slope instability at the hump hillside. The subject site was identified as the slope on the south side of Marine Drive between Foster Street and Cypress Street. The EBA report provides the results of the stability assessment.

The EBA assessment included drilling four solid-stem boreholes (BH1, BH2, BH3 and BH4) advanced to depths between 12.2 and 15.2 m. The boreholes were advanced along Marine Drive between Cypress Street and Foster Street. A standpipe piezometer was installed at each borehole location. Dynamic Cone Penetration Tests (DCPTs) were also conducted at each borehole to give a general indication of the *in-situ* density of the site soils.

EBA presented the probable sequence of events that led to potential slope instability at the subject site as follows:

- Prior to 1940, fill was placed on the down-slope side of Marine Drive during road grading construction to a relatively steep gradient (close to 1H:1V). Fill thickness increased at areas across original ravines and was likely not placed in accordance with current standards for fill placement and compaction.

- Surficial slips/erosion subsequently occurred at areas of the deep fill and eventually three bin walls were constructed to contain the fill at the most eroded areas.

- The three bin walls performed as designed (under static loading) as settlement/soil movement behind the walls were minimal at the time of the EBA investigation in 1994.

EBA reported that the handrails at the locations of the bin walls were in much better condition than at areas of the hump hillside outside of the bin walls (i.e. handrails at the bin walls were observed to be straight).

- Areas outside the three bin wall locations continued to suffer deterioration. The surficial fill continued to settle and cause severe cracking of the sidewalks at the top of the subject slopes (as observed by EBA in 1994).

The EBA report concluded that the existing slope is generally marginally stable with a factor of safety slightly over 1.0 under static loading conditions. A factor of safety is defined in the EBA report as the ratio of the average available shear strength of the soil along a critical slip surface to that required to maintain equilibrium. The slope stability assessment conducted by EBA did not take into consideration seismic loading at the site. Additionally, the report indicated that the most critical slip surfaces for slope instability are located in the surficial fill and that deep seated slope failure was not considered likely at the site. The report indicated that failure would likely be confined to the crest of the subject slope (i.e. within the surficial fill layer).

# 3. SITE CONDITIONS

## 3.1 SURFACE CONDITIONS

The hump hillside slope borders the south side of Marine Drive, above the BNSF railway tracks, approximately between Foster Street and Cypress Street, a distance of about 700 m. The hillside reaches a maximum height of approximately 20 m above the BNSF tracks and has slope gradients ranging between approximately 1H:1V and 1.75H:1V (horizontal:vertical). Multi-storey single and multi-family residences front the north side of Marine Drive in the area of the subject site.

Multiple underground services run along the alignment of Marine Drive and BNSF tracks in the area of the subject site and include, but are not limited to, the following:

- A water main that runs from the west to Fir Street. The water main runs parallel to Marine Drive along Marine Lane from Fir Street to Balsam Street. At Balsam Street, the main returns to Marine Drive. Based on the drawings provided by the City of White Rock, no water main runs along Marine Drive at the location of RW1;
- Various BC Hydro conduits that service private residences on the north side of Marine Drive;
- A sanitary gravity main that runs along the westbound lane of Marine Drive.
- Two sanitary force mains run along the bottom of the hump hillside slope, approximately parallel to the BNSF tracks;
- A storm main that runs along the westbound lane of Marine Drive;
- Gutter services; and
- An electrical conduit that runs along the sidewalk located on the south side of Marine Drive.

In addition to the above-mentioned underground utilities, overhead electrical service exists along the north side of Marine Drive in the area of the subject site. No gas service runs along Marine Drive in the general area of the subject site. However, gas service runs parallel to Marine Drive along Marine Lane located behind the residences fronting the north side of Marine Drive.

The hump hillside slope is generally vegetated with scattered deciduous trees and dense brush. Blackberry bushes make up a large part of the brush vegetation. The slope reaches a maximum height of approximately 20 m above the BNSF tracks and has slope inclinations between approximately 30° and 45°, as measured in the field with a hand-held inclinometer. Photograph 1 below illustrates the representative vegetation on the hump hillside.

Photograph 1: Representative Vegetation on the Hump Hillside

Shallow and medium seated landslide scarps were observed at multiple locations along the extent of the slope. Slope inclination is greatest at the scarp locations and increases to a maximum inclination of approximately 45°. The landslide scarps were generally observed to be vegetated with young deciduous trees and brush. Curved and leaning tree trunks were also observed on the subject hillside.

Vegetated scarps indicate that landslide activity had not occurred in the immediate past. However, as landslide scarps curve/bent tree trunks are present on the subject slope, the slope is considered susceptible to localized surficial creep and shallow seated failures. A vegetated landslide scarp observed during the site reconnaissance is presented in Photograph 2 below.

Photograph 2: Example of a Vegetated Landslide Scarp

Various retaining walls exist along the hump hillside slope. The walls were observed to consist of three metal bin retaining walls (including RW1) and at least four wood crib retaining walls located along the top of the slope. The wood bin retaining walls were observed to generally flank the east and west extents of the metal bin retaining walls.

The extent of Marine Drive associated with the hump hillside exhibited asphalt tension cracking and localized ground settlement. In particular, asphalt cracking was observed in both the eastbound and westbound lanes of Marine Drive behind RW1. Based on information provided by the Client, Levelton understands that the section of Marine Drive associated with the hump hillside was paved within approximately the last two years. Additionally, a review of Google Earth Streetview indicated that longitudinal cracking was present in the asphalt prior to re-paving. Photograph 3 below illustrates longitudinal (tension) cracking in the Marine Drive new asphalt located behind RW1.

Photograph 3: Asphalt Cracking Along Marine Drive Behind RW1

The existing condition of the RW1 was reviewed by Levelton during the site reconnaissance. The eastbound extent of RW1 is located approximately at the intersection of Marine Drive and Centre Street. RW1 extends west from this point a length of approximately 50 m. The walls comprised of a system of adjoining closed-face metal bins, each 3.05 m long. The adjoining bins are bolted together. Some rust was observed on the bolts and wall bins.

The exterior of RW1 was comprised of lightweight horizontal steel member facing (stringers) and vertical metal connectors. Vertical gaps up to approximately 20 mm wide were observed between the stringers and vertical connectors at locations along the wall exterior. RW1 backfill observed through the gaps appeared to be comprised of pea gravel. Where visible through gaps in the wall exterior, the pea gravel backfill occupied approximately half the height of one stringer, indicating that some wall backfill has likely been lost through the vertical gaps. Photograph 4 below illustrates a representative gap in the wall facing.

Photograph 4: Representative Gap in RW1 Exterior Facing

RW1 had a maximum height of approximately 6 m and was battered at an angle of about 1H:6V. Information regarding how far RW1 extends towards Marine Drive was not available. Some localized outward bulging of the wall facing was observed near the wall centre.

Wood crib retaining walls less tan 2 m high were located at either end of RW1. The wood crib retaining walls are not considered to provide substantial support to the slope. The wood crib retaining wall located at the west end of RW1 is illustrated in Photograph 5 below.

Photograph 5: Wood Crib Retaining Wall and RW1

Shallow seated landslide scarps were present directly downslope from RW1. These scarps initiated as close as approximately 0.3 m from the toe of RW1. The landslide scarps varied between approximately 3.0 m and 4.5 m wide and were inclined at angles between approximately 30° and 35°. The scarps were generall y vegetated with brush material.

## 3.2 REVIEW OF AERIAL PHOTOGRAPHS

Aerial photographs were obtained from the Geographic Information Centre of the University of British Columbia. The reviewed photographs were taken in the following years: 1940, 1949, 1954, 1963, 1974, 1979, 1984, 1991, 1997 and 2004. The scale of the aerial photographs ranged between approximately 1:10,000 and 1:25,000.

The review of aerial photographs indicated that the hump hillside slope has a history of shallow landslide activity and erosion. Additionally, landslide activity from the hillside was observed to cross the BNSF tracks in 1949.

Based on the review of the aerial photographs, the three bin retaining walls along the hump hillside appear to have been constructed by the year 1963. Cases of shallow landslide activity and slope erosion are still observable on the hump hillside after the construction of these walls. However, the extent of landslide/erosion activity appears to decrease from the year 1984. This could in part be the result of the proliferation of vegetation on the hump hillside subsequent to the year 1974.

Levelton did not observe any indications of large and/or deep seated slide areas on the aerial photographs in the area of the hump hillside.

## 3.3 HISTORY OF INSTABILITY

Levelton completed a review of historical information pertaining to previous instability at the subject site. The review was comprised of the following: available online information, information available from the White Rock Museum and Archives and relevant documents prepared by the Geological Survey of Canada.

The results of the historical review indicate that the steep slopes above Semiahmoo Bay (such as the hump slope hillside) are susceptible to landslide activity. These slopes are particularly susceptible to landslide activity during periods of heavy precipitation or subsequent to the removal of slope vegetation. Landslides from these slopes have also been reported to cause the derailment of trains on the BNSF tracks.

## 3.4 SURFICIAL GEOLOGY

The surficial geology of the area, as shown on Geological Survey of Canada Map 1484A, is generally comprised of Pleistocene age Capilano Sediments including raised marine and glaciomarine stony (including till-like deposits) to stoneless silt loam to clay loam with minor sand and silt normally less than 3 m thick but up to 30 m thick, containing marine shells. Map 1484A also indicates that the west portion of the hump hillside, near the White Rock Pier, may be associated with Vashon Drift deposits comprised of lodgement till (with sandy loam matrix) and minor flow till containing lenses and interbeds of glaciolacustrine laminated stony silt.

The GSC Map 1484A does not indicate the presence of Recent Slides (RS) at the site, but illustrates the presence of RS in other areas to the west of the site along the same hillside in the same geologic profile.

Vashon Drift deposits are known to underlie Capilano Sediments in the general area of the subject site. Additionally, subsurface cross-sections prepared by the Geological Survey of Canada for a site to the east of the hump hillside indicate that Quadra Sand underlies the Vashon Drift deposits (Geological Survey of Canada Paper 83-23, 1984).

With the exception of the encountered fill material, the soil conditions encountered were generally consistent with those reported on the surficial geology map.

### 3.5 SOIL CONDITIONS

Levelton conducted the subsurface exploration as detailed in Section 2.1.2 above. The general soil conditions encountered at the borehole locations are summarized below. A more detailed description of the soil conditions at the borehole locations is provided on the soil logs presented in Appendix A.

- **Asphalt;**

  Existing asphalt was encountered at all borehole locations. Asphalt was observed to be present in multiple layers at BH11-01 and in a single layer at BH11-02 and BH11-03. Asphalt layer thickness was between approximately 75 mm and 200 mm at the borehole locations.

- **Fill** (Silty Sand and Gravel to Sandy Silt, very loose to compact, brown);

  Fill was observed at all three borehole locations. The thickness of the fill varied between approximately 1.4 m and 3.0 m. Samples of the fill material encountered had moisture contents within the range of 5% to 15%. SPT N-values in the fill ranged from 4 to 15 blows per 305 mm (foot).

- **Inter-layered Sand/Sand and Silt/Silty Sand/Silt** (fine to coarse grained sand, trace to some gravel, compact to very dense, grey to brown)

  Deposits comprised of sand and silt were encountered at all three borehole locations. These deposits correspond with the surficial geology of the area, as shown on Geological Survey of Canada Map 1484A, and consist of till-like Capilano glaciomarine sediments overlying Vashon lodgement till. The Vashon lodgment till was encountered to a depths between approximately 12.5 m and 13.4 m below existing grade. Samples of these deposits had moisture contents within the range of 13% to 31%. SPT N-values in this layer ranged from 32 to greater than 100 blows per 305 mm (foot).

- **Sand** (fine to coarse grained, trace to some gravel, very dense, grey to brown)

  Sand deposits classified by the Geological Survey of Canada as Quadra Sands were encountered underlying to the above mentioned Vashon lodgement till. The Quadra Sand was encountered to the terminus of all three boreholes at depths between 15.2 m and 15.8 m below existing grade. Samples of these sand deposits had moisture contents within the range of 18% to 22%. SPT N-values in the lower sand unit were greater than 100 blows per 305 mm (foot).

### 3.6 GROUNDWATER CONDITIONS

Two groundwater level readings were taken in the piezometer installed to a depth of approximately 15.9 m in BH11-02 subsequent to the completion of the subsurface investigation. Table 1 below presents the water levels recorded on the dates indicated.

**Table 1: Water Table Readings in BH11-02**

| Date | Depth Below Existing Ground Surface (m) |
|---|---|
| April 13, 2011 | 11.30 |
| May 3, 2011 | 11.97 |

## 4. INCLINOMETER MONITORING RESULTS

Two sets of inclinometer readings were taken in the boreholes BH11-01 and BH11-03 on April 13, 2011 and May 3, 2011, respectively. The results of the inclinometer readings showed no significant movement at the inclinometer locations between the reading dates. The inclinometer readings are presented in graphical form in Appendix C.

## 5. ASSUMPTIONS FOR SLOPE STABILITY ANALYSIS

Based upon review of the boreholes BH11-01 through BH11-03, Levelton divided the soil stratigraphy to be modeled in the stability analyses into four general units. These units are classified as: loose fill, dense sand and silt (Capilano sediments), very dense sand and silt (Vashon lodgment till), and very dense sand (Quadra sands).

The soil parameters for the slope stability analyses were based on a review of the soil logs, Levelton's site reconnaissance, laboratory testing, review of available technical research papers, and our local experience. The soil shear strength parameters (i.e. friction angle and cohesion) correspond to the most probable best and worst case site soil conditions, as determined from the results of the subsurface investigation. The best and worst case soil strength parameters were employed to give a range of the possible stability conditions that could be associated with RW1 and the slope.

The soil parameters used for the slope stability analyses are provided in Table 2 below.

**Table 2 – Slope Stability Analysis Soil Parameters**

| Unit | Unit Weight (kN/m$^3$) | Cohesion (kPa) | | Friction Angle | | Elastic Modulus (kPa) | Poisson's Ratio |
|---|---|---|---|---|---|---|---|
| | | Worst Case | Best Case | Worst Case | Best Case | | |
| Loose Fill | 16 | 0 | 0 | 30° | 34° | 7800 | 0.25 |
| Dense Sand and Silt | 18 | 10 | 10 | 30° | 36° | 20000 | 0.35 |
| Very Dense Sand and Silt | 20 | 10 | 15 | 32° | 38° | 20000 | 0.35 |
| Very Dense Sand | 23 | 0 | 0 | 38° | 42° | 80000 | 0.4 |

It was assumed that the groundwater table was 11.3 m below the ground surface for the stability analyses, based on the measured depth of the groundwater in BH11-02. As the design drawings for RW1 were not available, it was assumed that RW1 was constructed in accordance with typical drawings for bin-type retaining walls. Levelton reviewed typical bin-type retaining wall drawings provided by Armtec Ltd. (a supplier of bin-type retaining walls).

The modeled slope topography, wall dimensions and soil layering are illustrated on the results of the stability analyses presented in Appendix D and Appendix E.

# 6. CONCLUSIONS AND RECOMMENDATIONS – BIN WALL RW1

## 6.1 GLOBAL STABILITY CONSIDERATIONS

### 6.1.1 General

It is of vital importance to the stability of Marine Drive and existing underground infrastructure that the subject bin retaining wall be globally stable under long term loading conditions. Levelton evaluated the overall stability of RW1 and the slope directly below RW1 by way of limit equilibrium stability analyses using RocScience *Slide 5.0* software employing the Bishop's simplified method of slices with circular arcs. This analysis computes a minimum Factor of Safety (FoS) based on the following:

- Topography;
- Soil layering, wall dimensions, and loading;
- Levelton estimates of the soil shear strength;
- Groundwater table position;
- External loading conditions, such as Live Loads and earthquake loads.

The FoS represents the ratio of moments and forces resisting failure over the moments and forces acting to induce failure. A FoS less than 1.0 represents unstable conditions. Conversely, a FoS 1.0 or greater indicates stable conditions.

Levelton also evaluated the overall stability of the subject bin retaining wall and slope by way of finite-element stability analyses using RocScience *Phase2 7.0*. This analysis computes a minimum Strength Reduction Factor (SRF) based on the same items listed above for the limit equilibrium stability analyses. The SRF is defined as the factor by which the input soil strength parameters are either decreased or increased by to bring the slope to the verge of failure. A SRF less than 1.0 represents unstable conditions. A SRF of 1.0 or greater indicates stable conditions.

Finite-element analysis software has the ability to account for material stress-strain behavior and can provide information on deformations at working stress levels. Furthermore, finite-element analyses have the potential to reveal the progress of failure so that the potential slope failure mechanisms can be identified. Running both the limit equilibrium and finite-element analyses in conjunction gives increased confidence in the results obtained.

Both the SRF and FoS define a “safety factor” for the subject slope and can be treated as equivalent for the purposes of this report. This report will refer to the both SRF and FoS as a “safety factor”.

### 6.1.2 Static Loading

RW1 and the slope at its toe were analyzed under static conditions for both the best and worst case geotechnical soil strength parameters presented in Table 2 above. The static analysis was based on the following conditions:

- Soil layering inferred from the boreholes and site reconnaissance observations;
- Water table at a depth of 11.3 m; and
- Live Load of 12 kPa on Marine Drive (i.e. Typical BC Ministry of Transportation and Infrastructure highway loading).

The results of the slope stability analyses indicate that RW1 and the associated slope are marginally stable under static loading conditions, for both the best and worst case soil strength parameters, with a safety factor between 1.1 and 1.3. The results are provided in Table 3 below.

**Table 3 – Static Slope Stability Analysis Results**

| **Load Case** | **Soil Strength Parameters** | **Safety Factor** | |
|---|---|---|---|
| | | **SRF** ***(Phase$^2$)*** | **FOS** ***(Slide)*** |
| Load Case 1 | Best Case | 1.29 | 1.27 |
| | Worst Case | 1.09 | 1.09 |

The results of the static global stability analyses for Load Case 1 are provided in graphical form in Appendix D. The results are in general agreement with EBA’s findings.

### 6.1.3 Seismic Loading

The stability of RW1 and the associated slope was estimated under seismic loading using both the pseudo-static limit equilibrium and finite-element slope stability analysis techniques. The pseudo-static force was based on the site specific peak ground acceleration (PGA) obtained from the Natural Resources Canada (NRC) seismic hazards web site. These PGA values as follows:

- 40% in 50 years Probability of Exceedance Earthquake (A100): PGA = 0.143g
- 10% in 50 years Probability of Exceedance Earthquake (A475): PGA = 0.306g

- 5% in 50 years Probability of Exceedance Earthquake (A975): PGA = 0.411g
- 2% in 50 years Probability of Exceedance Earthquake (A2475) PGA = 0.566g

In Canada, design seismic hazards are specified in terms of probabilistic ground motions being exceeded (i.e. design is based on the PGA). The PGA is a measure the maximum acceleration experienced on the ground during the course of earthquake motion. The PGA values presented above correspond to probabilistic ground motions having a 2% chance of being exceeded in 50 years (PGA = 0.566g), 5% chance of being exceeded in 50 years (PGA = 0.411g), etc. These peak ground motions are dependent on the following:

- Subsurface soil/rock conditions;
- Magnitude of the earthquake;
- Depth of the earthquake;
- Fault characteristics;
- Distance of the subject site from the earthquake epicentre; and
- Earthquake duration/frequency.

Earthquake magnitude is a measure of the energy released during an earthquake and does not directly correlate to the probabilistic ground motions presented above. However, research pertaining to earthquakes in the vicinity of Vancouver, BC indicates that the above-referenced probability of exceedance earthquakes (A100 through A2475) approximately correspond to an earthquake with a mean magnitude of about 7.0. Consequently, the different PGA values presented above are dependent on more than earthquake magnitude alone. Parameters such as subsurface conditions, distance of the site from the epicentre, etc. are all contributing factors to the site specific PGA.

The seismic coefficient (k) input into the pseudo-static seismic stability analyses was taken as 67% of the site specific PGA, which is consistent with the current geotechnical state of practice for a project of this nature. The following scenarios were analyzed for RW1 and the associated slope under seismic loading:

**Load Case 1**

- A100 Seismic coefficient (k) = 0.10g; and
- Water table at a depth of 11.3 m; and
- Live Load of 5 kPa on Marine Drive (i.e. Estimated normal traffic loading);

**Load Case 2**

Same as conditions as Load Case 1 above, except the following:

- A475 Seismic coefficient (k) = 0.20g.

**Load Case 3**

Same as conditions as Load Case 2 above, except the following:

- A975 Seismic coefficient (k) = 0.27g.

**Load Case 4**

Same as conditions as Load Case 3 above, except the following:

- A2475 Seismic coefficient (k) = 0.38g.

The results of the seismic global stability analyses are presented in Table 4.

**Table 4 – Seismic Slope Stability Analysis Results**

| Load Case | Soil Strength Parameters | Safety Factor | |
|---|---|---|---|
| | | SRF<br>*Phase2* | FOS<br>*Slide* |
| **Load Case 1**<br>(k=0.10g) | Best | 1.09 | 1.12 |
| | Worst | 0.93 | 0.95 |
| **Load Case 2**<br>(k=0.20g) | Best | 0.88 | 0.97 |
| | Worst | 0.75 | 0.82 |
| **Load Case 3**<br>(k=0.27g) | Best | 0.76 | 0.89 |
| | Worst | 0.64 | 0.76 |
| **Load Case 4**<br>(k=0.38g) | Best | 0.53 | 0.77 |
| | Worst | 0.47 | 0.66 |

The results of the seismic stability analyses are summarized below.

**Load Case 1 (A100 Seismic Event)**

- The Load Case 1 stability analysis indicates that RW1 and the associated slope have a safety factor between approximately 0.90 and 1.10 for the A100 design magnitude earthquake, indicating potentially unstable or marginally stable slope conditions; and
- The depth of instability is primarily confined to the surficial fill and dense sand and silt layers. The lateral extent of instability is generally restricted to the eastbound lane of Marine Drive.

**Load Case 2 (A475 Seismic Event)**

- The Load Case 2 stability analysis indicates that RW1 and the associated slope have a safety factor between approximately 0.75 and 1.00 for the A475 design magnitude earthquake; and
- The depth of instability is primarily confined to the surficial fill and dense sand and silt layers. The very dense Vashon lodgement till deposits (very dense sand and silt) prevent failure surfaces from penetrating a significant depth into the till deposits. Failure surfaces are observed to affect both the eastbound and westbound lanes of Marine Drive.

**Load Case 3 (A975 Seismic Event)**

- The Load Case 3 stability analysis indicates that RW1 and the associated slope have a safety factor between approximately 0.65 and 0.90 for the A975 design magnitude earthquake; and

- The depth of instability is primarily confined to the surficial fill and dense sand and silt layers. The very dense Vashon lodgement till deposits (very dense sand and silt) prevent failure surfaces from penetrating a significant depth into the till deposits. Failure surfaces are observed to affect both lanes of Marine Drive and extend north of Marine Drive.

**Load Case 4 (A2475 Seismic Event)**

- The Load Case 4 stability analysis indicates that RW1 and the associated slope have a safety factor between approximately 0.50 and 0.80 for the A2475 design magnitude earthquake; and
- The depth of instability extends through the surficial fill and dense sand and silt layers and begins to penetrate into the Vashon lodgement till deposits (very dense sand and silt). Failure surfaces are observed to affect both lanes of Marine Drive and extend north of Marine Drive.

Typically, a minimum safety factor of 1.0 to 1.2 is desired under deign basis seismic loading conditions. The results indicate that RW1 and the associated slope are unstable under seismic loading corresponding to earthquakes with a magnitude greater than A100 for a factor of safety greater than 1.0. Additionally, RW1 and the associated slope are only considered marginally stable under the A100 design ground motion earthquake.

The safety factors obtained using the limit equilibrium (*Slide)* and finite-element analyses (*Phase*$^2$*)* were generally comparable. Similar results are considered to provide increased confidence in the stability analyses.

The stability analysis results for Load Case 2 (A475 Seismic Event) are provided in Appendix E.

## 6.2 SLOPE INSTABILITY IMPACTS TO MARINE DRIVE

As summarized in Section 6.1.3, failure of the slope associated with the metal bin retaining wall under seismic loading would likely impact Marine Drive. However, the variability and inherent heterogeneity in the subsurface conditions makes it difficult to accurately predict the extent of impact that failure of the slope and RW1 would have on Marine Drive. Additionally, the extent of failure is largely dependent on the seismic load experienced at site.

Generally, the slope stability analyses indicate that the extent of failure could range between less than 10 cm movement at the surface of Marine Drive and the complete failure of Marine Drive, depending on earthquake intensity and soil strength parameters. Additionally, the stability of the slope is expected to vary with the depth of surficial fill.

The A475 design magnitude earthquake is the earthquake loading applied in slope stability analyses by the British Columbia Ministry of Transportation and Infrastructure highway design. The slope stability analyses illustrate that seismic loading corresponding to the A475 design magnitude earthquake would cause the eastbound lane of Marine Drive in the area of the hump hillside to require servicing or complete repair. There is also significant potential for the westbound lane of Marine Drive to be impacted as a result of slope/retaining wall failure under the A475 earthquake.

Displacement of Marine Drive behind RW1 would be capable of impacting underground utilities located in Marine Drive. As reported in Section 3.1, electrical conduits, a sanitary gravity main, a storm main and gutter services are known to be contained within the Marine Drive right-of-way, in the area of RW1. All of these utilities would potentially be subject to significant damage, should failure occur at RW1.

### 6.3 SLOPE INSTABILITY IMPACTS BELOW RW1

The Levelton slope stability analysis findings indicate that the design ground motion earthquakes considered could result in slope failures. The failures would impact the BNSF Railway tracks and the adjacent White Rock Promenade in the form of the rapid deposition of mobilized soil.

As-built drawings provided by the Client (Reid Crowther Drawing No. C04, Rev. 4, March 13, 2000) indicate that the existing 350 mm and 450 mm sanitary sewer lines run adjacent to the BNSF tracks near the toe of the slope associated with RW1. The results of the stability analyses indicate that failure surfaces associated with RW1 instability would likely not intersect these sanitary lines under the A475 seismic event.

### 6.4 BIN WALL LONGEVITY CONSIDERATIONS

To provide longevity to RW1, Levelton recommends the following:

1. The wood crib retaining walls at the east and west extents of RW1 should be replaced. These retaining walls were observed to have deteriorated and are not considered to provide significant support in their current condition.

2. The condition of the metal facing and structural connections should be further examined for RW1 and the two additional bin walls. Consideration should be given to including a corrosion assessment as part of this review, to estimate the remaining design life of the structure.

### 6.5 POSSIBLE UPGRADE OPTIONS

While the subject bin wall appears to be performing as designed under static loading conditions, upgrades to RW1 are considered necessary to provide seismic stability. The methods considered feasible to provide permanent stability for Marine Drive at the location of RW1 includes the following:

1. Installation of piles along Marine Drive adjacent to RW1. Piles would be installed to a design depth to reinforce the site soils to reduce the potential for failure of Marine Drive in response seismic loading. However, such a system would likely only marginally improve the stability of RW1;

2. Installation of a soil anchor retention system along the exterior of RW1. The feasibility of this type of upgrade option is uncertain due to constraints on equipment access to the exterior of the bin retaining walls; and/or

3. Buttressing the toe of RW1 and/or slope below through combination of retaining structures, piles and soil anchors. However, access to the base of RW1 and toe of slope may be difficult and could result in this option being financially unrealistic.

## 7. ADDITIONAL RETAINING WALLS

Levelton observed two additional metal bin retaining walls on the hump hillside surrounding RW1. One bin wall was located at approximately 15191 Marine Drive (west of RW1) and the other at approximately 15349 Marine Drive (east of RW1). These walls were observed to reach a maximum height of approximately 4.0 m.

Levelton considers it reasonable to assume that similiar soil conditions exist at each bin wall location (i.e. loose fill underlain by dense to very dense native soils). Consequently, seismic instability in general accordance with Section 6.1.3 is possible at each bin wall location. Consideration should be given to the future upgrade options presented in Section 6.5 for each of the two additional metal bin retaining walls.

The bin wall to the west of RW1 was observed to be flanked by two wood crib retaining walls at either end of the bin wall. The bin wall to the east of RW1 was observed to have a wood crib retaining wall located at the west extent of this wall. These wood crib walls are considered marginally unstable and require immediate replacement with an engineered retaining wall system. Photograph 6 below illustrates the wood crib retaining wall associated with the bin wall located near 15191 Marine Drive.

Photograph 6: Wood Crib Wall near 15191 Marine Drive

## 8. SURROUNDING HILLSIDE – GENERAL STABILITY

In general, the hump hillside surrounding RW1 and the additional retaining walls is considered marginally stable under static loading conditions. Levelton observed numerous shallow landslide scarps and curve/bent tree trunks on the subject hillside. These observations indicate that localized surficial creep and failures occur on the hillside slope on an ongoing basis. Due to the steepness of the subject slope, these localized shallow slides could occur periodically in response to periods of heavy precipitation and runoff. Levelton recommends the development and implementation of a sustainable vegetation management plan so that vegetation on the hillside helps to mitigate against potential shallow slide activity. Shallow landslide activity on the slope could potentially impact the BNSF tracks and White Rock Promenade. Further comments pertaining to the vegetation management plan are presented in Section 9.0.

Landslide activity on the hump slope could also be triggered by seismic loading. However, the slope stability analyses under static and seismic loading indicate that the extent of slope instability is primarily influenced by the presence and thickness of loose surficial fill overlying the more stable dense to very dense native soils.

It is probable that the bin retaining walls were originally constructed at locations where fill placed on the down-slope side of Marine Drive was thickest during the construction of Marine Drive (e.g. infilled steep gully locations). This conclusion was also presented in the EBA Slope Stability Assessment Report dated September 1994. As a result, slope instability under seismic loading at locations outside the bin retaining walls may not be as extensive as presented in Section 6.1.3 due to reduced fill thickness. Consequently, it is expected that such landslides would be shallow seated, not resulting in the failure of Marine Drive. However, there is a potential for such slope failures to impact the stability of the south sidewalk on Marine Drive.

# 9. MONITORING & MAINTENANCE

The stability analyses and site observations indicate that the metal bin retaining walls are generally performing as designed and considered stable under static conditions. Nevertheless, due to cracking observed in the asphalt behind RW1, the overall slope condition, and the presence of underground infrastructure, it is considered appropriate to conduct monitoring over the next three to five years to confirm RW1 and the other bin walls are performing as anticipated. The actual duration of the monitoring should be determined by Levelton in conjunction with the Client.

Levelton recommends the following monitoring program:

1. The Levelton subsurface exploration included the installation of the two inclinometers at BH11-01 and BH11-03 and one piezometer at BH11-02. Levelton recommends that inclinometer and groundwater level readings be conducted at these locations once every six months for a period not less than three years. The readings should be completed by Levelton.

2. The locations of BH11-01, BH11-02 and BH11-03 should be surveyed and survey monuments should be established along RW1 and the top of the two other metal bin walls to supplement inclinometer readings. The survey monuments should be established where they can readily be accessed by a surveyor without interruption of traffic, such as along the sidewalk adjacent to the bin retaining wall. The surveyor should prepare a plan showing the location of the monuments, and their location. Each survey should include the position and elevation of each monument. General practice guidelines indicate that review of retaining wall structures should be completed on an annual basis, as a minimum. Levelton recommends that survey monument readings be conducted at each location once every six months for a period not less than three years. The data obtained from the survey should be provided to Levelton for review.

3. A sustainable vegetation management plan should be developed in order that vegetation on the hillside contributes to its ongoing stability, The plan should include the following:

    - Annual assessment of all trees on the hillside to determine their structural integrity should be conducted by an experienced arborist. This is especially important for trees growing at the base of the retaining walls, but is applicable to all trees on the hillside. Immediate removal of unstable trees is recommended. Failure of mature unstable trees could locally destabilize the slope, possibly resulting in displacement of the retaining walls and/or a debris avalanche on the BNSF track.

    - Maintaining ground cover and low vegetation that will contribute to slope stability and minimize the potential for surface erosions.

- o Periodic cutting back of vegetation, provided that vegetation removal does not contribute to potential slope instability. A qualified professional should review the proposed vegetation cutting prior to occurrence. Removal of slope stabilizing vegetation should not be considered unless an alternative method of stabilizing the slope is proposed.

4. The current condition of the sanitary and storm mains along Marine Drive in the vicinity of RW1 should be reviewed. Each main should be investigated for the presence water leakage which may be a contributing factor to asphalt cracking, fill settlement or potential unstable slope conditions. Levelton should be notified of the results of the investigation to supplement our present understanding of the site conditions.

The actual details of the monitoring program may be modified with time. If recorded movements are negligible after three years of monitoring, the monitoring program can be discontinued.

## 10. LIMITATIONS & CLOSURE

This geotechnical assessment report has been prepared by Levelton Consultants Ltd. exclusively for the Corporation of the City of White Rock and their appointed agents. The information contained in this report reflects our judgment in light of the information provided to us at the time that it was prepared.

Any use of this report by third parties, or any reliance on or decisions made based on it, are the responsibility of such third parties. Levelton does not accept responsibility for damages suffered, if any, by a third party as a result of their use of this report.

The soil logs appended to this report provide description of the soil and groundwater conditions encountered at discrete borehole locations. Soil conditions are expected to vary between borehole locations.

Contractors should make their own interpretation of the soil logs and the site conditions for the purposes of bidding and performing work at the site.

The attached Terms of Reference should be read in conjunction with and form an integral part of this report.

We trust this information meets your immediate requirements. If you have any questions or require further information, please contact the undersigned.

**LEVELTON CONSULTANTS LTD.**

***Original Signed By***

Per: Graeme McAllister, EIT
Staff Geotechnical Engineer

Per: Calum Buchan, P.E., P.Eng.
Senior Geotechnical Engineer
Technical Services Manager, FV Region

Reviewed By:

***Original Signed By***

Per: Dejan Jovanovic, P.Eng.
Geotechnical Engineer

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## 5. INTERPRETATION OF THE REPORT

a. **Nature and Exactness of Descriptions**: The classification and identification of soils, rocks and geological units, as well as engineering assessments and estimates have been based on investigations performed in accordance with the standards set out in Paragraph 1 above. The classification and identification of these items are judgmental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations or assessments utilizing the standards of Paragraph 1 involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to changes over time and the parties making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or when the Client has special considerations or requirements, the Client must disclose them to Levelton so that additional or special investigations may be undertaken, which would not otherwise be within the scope of investigations made by Levelton or the purposes of the Report.

b. **Reliance on information**: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site investigation and field review and on the basis of information provided to Levelton. Levelton has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Levelton cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the report as a result of misstatements, omissions, misrepresentations or fraudulent acts of persons providing information.

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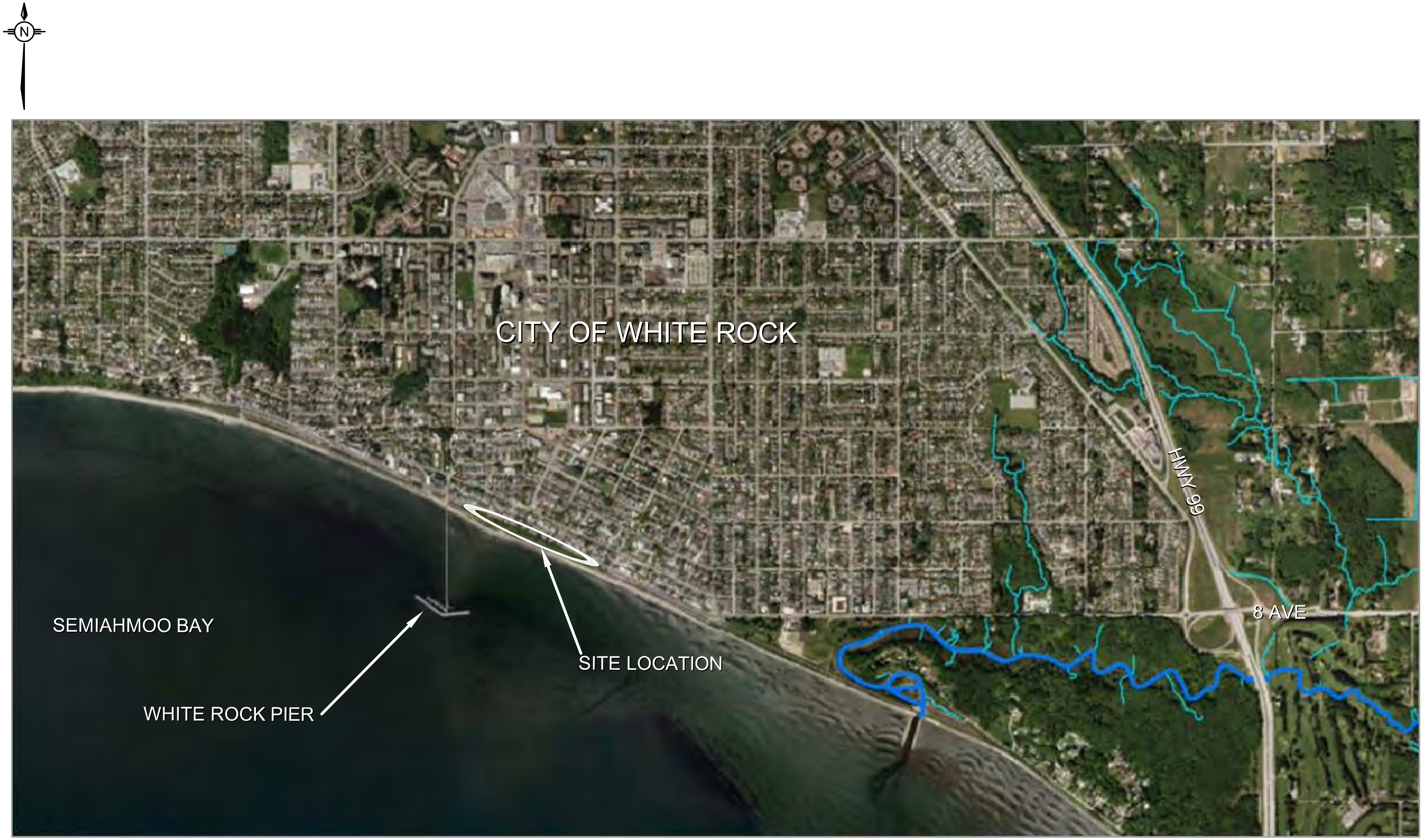
## 6. ALTERNATE REPORT FORMAT

When Levelton submits both electronic and hard copy versions of the Instruments of Professional Services, the Client agrees that only the signed and sealed hard copy versions shall be considered final and legally binding upon Levelton. The hard copy versions submitted by Levelton shall be the original documents for record and working purposes, and, in the event of a dispute or discrepancy, the hard copy versions shall govern over the electronic versions; furthermore, the Client agrees and waives all future right of dispute that the original hard copy signed and sealed versions of the Instruments of Professional Services maintained or retained, or both, by Levelton shall be deemed to be the overall originals for the Project.

The Client agrees that the electronic file and hard copy versions of Instruments of Professional Services shall not, under any circumstances, no matter who owns or uses them, be altered by any party except Levelton. The Client warrants that the Instruments of Professional Services will be used only and exactly as submitted by Levelton.

The Client recognizes and agrees that Levelton prepared and submitted electronic files using specific software or hardware systems, or both. Levelton makes no representation about the compatibility of these files with the current or future software and hardware systems of the Client, the Approved Users or any other party. The Client further agrees that Levelton is under no obligation, unless otherwise expressly specified, to provide the Client, the Approved Users and any other party, or any or all of them, with specific software and hardware systems that are compatible with any electronic submitted by Levelton. The Client further agrees that should the Client, an Approved User or a third party require Levelton to provide specific software or hardware systems, or both, compatible with the electronic files prepared and submitted by Levelton, for any reason whatsoever included but not restricted to an order from a court, then the Client will pay Levelton for all reasonable costs related to the provision of the specific software or hardware systems, or both. The Client further agrees to indemnify and hold harmless Levelton, its officers, directors, employees, agents, representative or sub-consultant, or any or all of them, against any claim or any nature whatsoever brought against Levelton, whether in contract or in tort, arising or related to the provision or use or any specific software or hardware provided by Levelton.



| REVISIONS | | | | |
|---|---|---|---|---|
| REV | Date | Rev. Description | Dwn | Check |

ADAPTED FROM
City of Surrey Mapping Online System

DATE
May 28, 2011

PROJECT NO.
N/A

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Abbotsford, BC, V2S 2E8
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f:604-853-1186
www.levelton.com

TITLE
OVERVIEW SITE LOCATION PLAN

PROJECT
RETAINING WALL & SLOPE STABILITY REVIEW - HUMP HILLSIDE

ADDRESS
MARINE DRIVE, WHITE ROCK, BC

CLIENT
CORPORATION OF THE CITY OF WHITE ROCK

DSN

CHK
CB

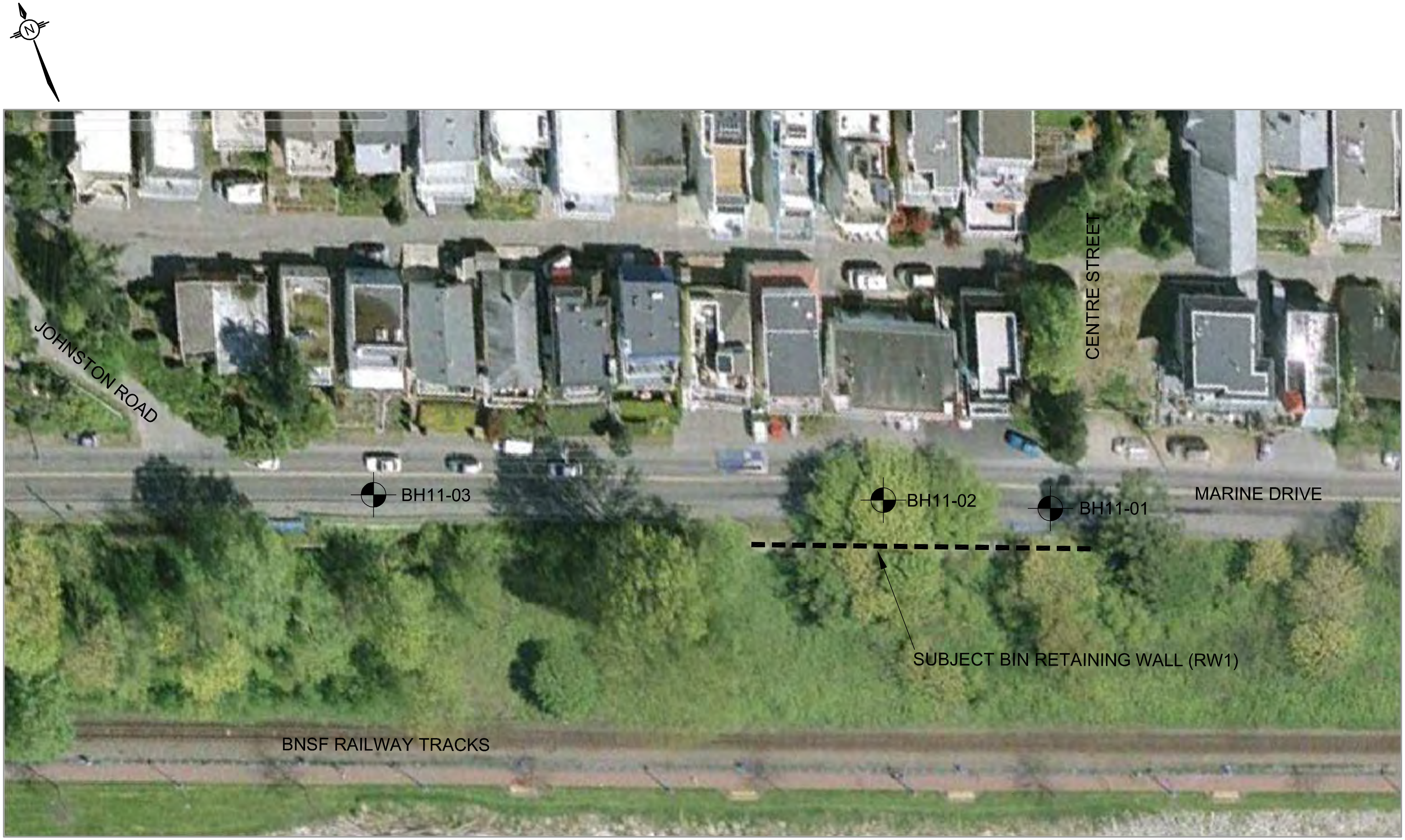
DWN
GM

SCALE
NTS

DATE
JUNE 2011

PROJECT NO.
FV11-0658-00

FIGURE NO.
1



Note: All borehole locations are approximate

LEGEND

⊕ BH11-03 APPROXIMATE BOREHOLE LOCATION

ADAPTED FROM
City of Surrey Mapping Online System

DATE
May 28, 2011

PROJECT NO.
N/A

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TITLE
BOREHOLE LOCATION PLAN

PROJECT
RETAINING WALL & SLOPE STABILITY REVIEW - HUMP HILLSIDE

ADDRESS
MARINE DRIVE, WHITE ROCK, BC

CLIENT
CORPORATION OF THE CITY OF WHITE ROCK

DSN

CHK
CB

DWN
GM

SCALE
NTS

DATE
JUNE 2011

PROJECT NO.
FV11-0658-00

FIGURE NO.
2

# Appendix A

# Soil Logs

**Levelton Consultants Ltd.**

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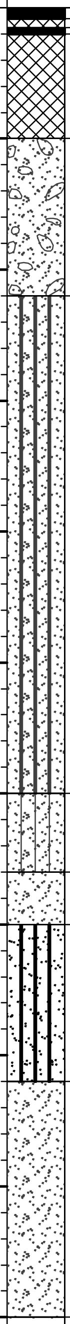
# Retaining Wall & Slope Stability Review
Hump Hillside
Marine Drive, White Rock, BC
SOIL BORING LOG

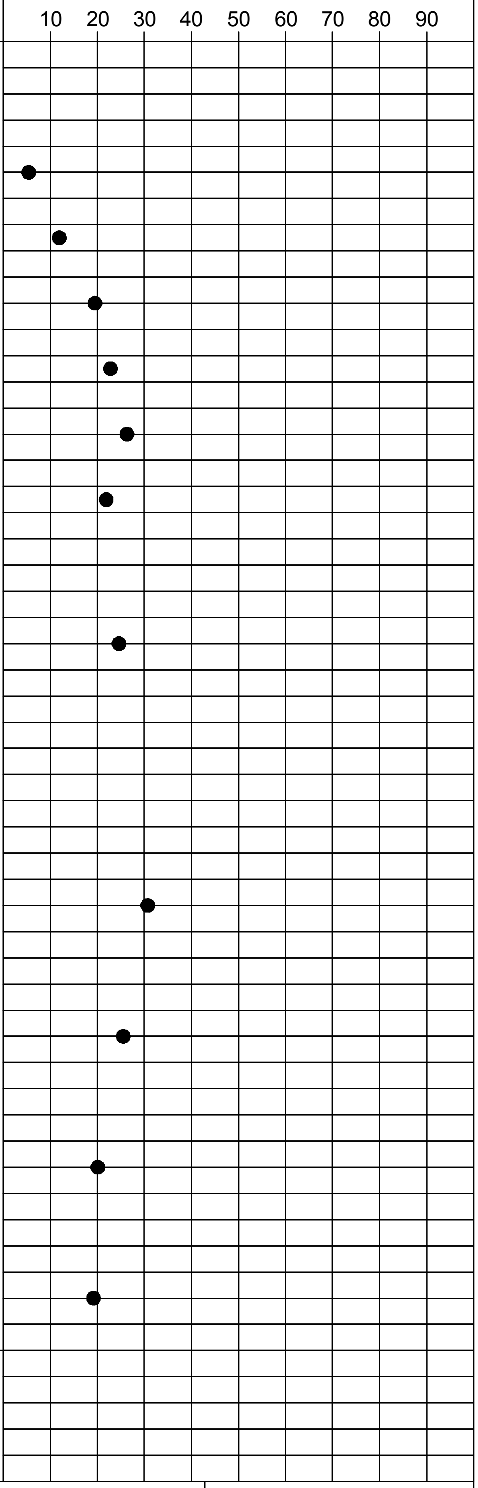
# BH11-01

Pg 1 of 1

Project No: FV11-0658-00

| Depth (m) | Depth (ft) | Description | C | N | Type | Water Level |
|---|---|---|---|---|---|---|
| | | **ASPHALT** (125mm thick) | | | | |
| | | Brown sand, **FILL**, some gravel (100mm thick) | | | | |
| | | **ASPHALT** (75mm thick) | | 14 | SPT | |
| | 5 | Compact, brown, **GRAVELLY SAND FILL**, coarse grained | | 5 | SPT | |
| 2 | | Very loose to loose, brown, **GRAVELLY SAND**, coarse grained (possibly fill) | | 4 | SPT | |
| | 10 | compact | | 11 | SPT | |
| 4 | | Compact to dense, brown, **SAND AND SILT**, medium to coarse grained, trace gravel | | 32 | SPT | |
| | 15 | fine grained sand | | 34 | SPT | |
| | | | | 40 | SPT | |
| 6 | 20 | | | | | |
| | | very dense | | 79 | SPT | |
| 8 | 25 | No SPT sample recovery | | >100 | SPT | |
| | 30 | Very dense, brown, **SILTY SAND**, fine grained, trace gravel | | | | |
| 10 | | Very dense, brown, **SAND**, fine grained, trace gravel | | 85 | SPT | |
| | 35 | Hard, grey brown, **SAND AND SILT**, fine grained, trace gravel | | 77 | SPT | |
| 12 | 40 | Very dense, brown to grey, **SAND**, medium grained, trace gravel | | >100 | SPT | |
| 14 | 45 | | | >100 | SPT | |
| | 50 | Inclinometer casing installed to 15.24 m below existing grade.<br>Bottom of hole at 15.24 meters | | | | |
| 16 | 55 | | | | | |





**C: Condition of Sample**
Good ▇
Disturbed ▥
No Recovery ☐

**Type: Type of Sampler**
SPT : 2 in. standard
ST : Shelby
FP : Fixed Piston
G : Grab
CORE

**N: Number of Blows**
WH : Weight of Hammer
WR : Weight of Rod
Standard Penetration Test : ASTM D1586
Hammer Type:

- ● Moisture Content %
- ► Plastic Limit %
- ◄ Liquid Limit %
- ⊻ Ground Water Level
- ⊗ Shear strength in kPa (Torvane or Penetrometer)
- X Shear strength in kPa (Unconfined)
- ⊗ Shear strength in kPa (field vane)
- ⊠ Remolded strength in kPa
- ■ Percent Passing # 200 sieve

Drill Method: Mud Rotary
Date Drilled: 4/5/2011
Logged By: AA

*THIS LOG IS FOR GEOTECHNICAL PURPOSES ONLY*

**Levelton Consultants Ltd.**

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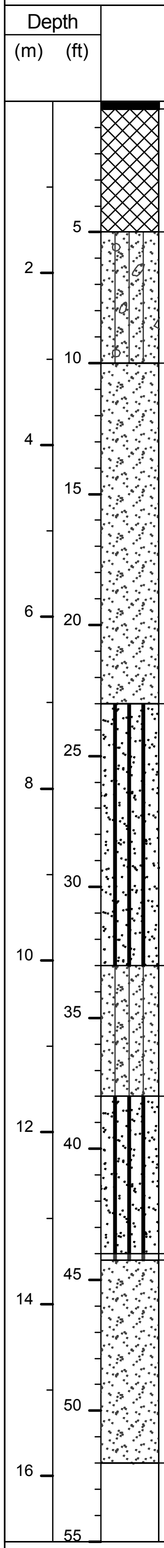
# Retaining Wall & Slope Stability Review
Hump Hillside
Marine Drive, White Rock, BC
SOIL BORING LOG

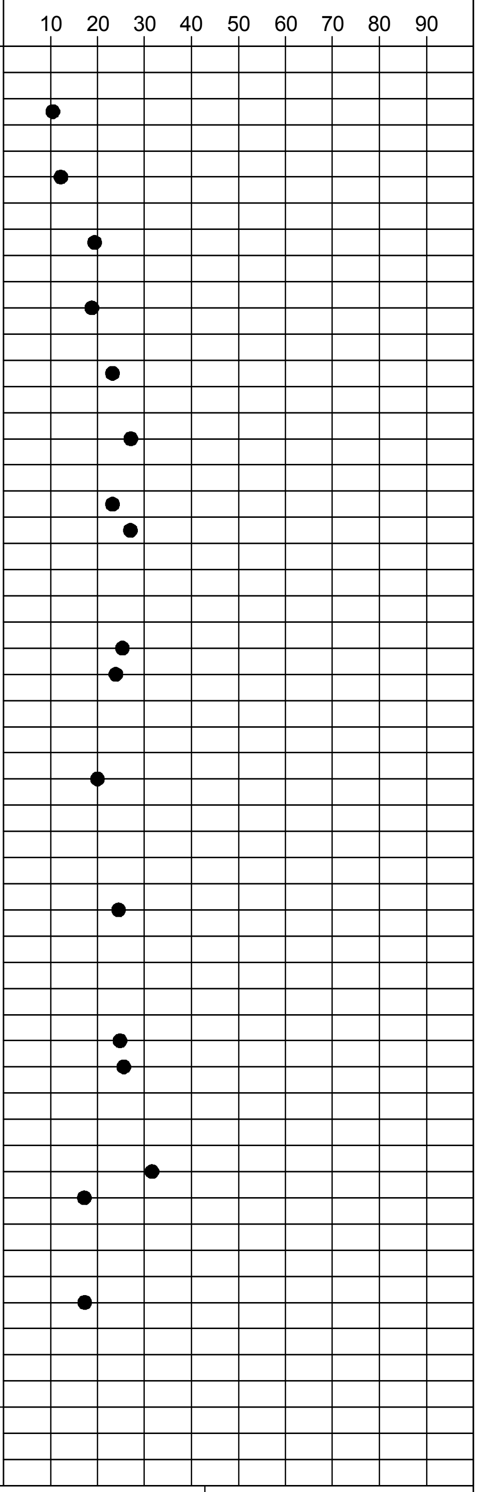
# BH11-02

Pg 1 of 1

Project No: FV11-0658-00



| Description | N | Type | Water Level |
|---|---|---|---|
| Asphalt (100mm thick) | | | |
| Loose, brown, **SILTY SAND/SANDY SILT FILL**, some gravel to 20mm diameter | 5 | SPT | |
| Loose, brown, **SILTY SAND**, well graded, trace to some gravel to 40mm diameter, subrounded to subangular, trace coal | 10 | SPT | |
| very dense | 52 | SPT | |
| Dense, orange brown to brown grey, **SAND**, fine to medium grained, trace silt to silty, oxidation staining | 42 | SPT | |
| | 36 | SPT | |
| very dense | 51 | SPT | |
| | 59 | SPT<br>G | |
| Very dense, grey, **SAND AND SILT**, well graded sand, oxidation staining | 53 | SPT<br>G | |
| | >100 | SPT | |
| Very dense, grey, **SILTY SAND**, fine grained, oxidation staining | 74 | SPT | Apr 7 2011<br>Apr 13 2011 |
| Hard, grey, **SILT**, some sand, fine grained, trace coarse sand | 72 | SPT<br>G | |
| | >100 | SPT<br>G | |
| Hard, brown, **SILT**, some gravel to 40 mm diameter, subangular to subrounded, some coarse grained sand (75mm thick) | | | |
| Very dense, brown grey, **SAND**, medium to coarse grained, trace gravel to 50mm diameter, subangular | >100 | SPT | |
| Piezometer installed to 15.85 m below existing grade.<br>Bottom of hole at 15.85 meters | | | |



**C: Condition of Sample**
Good ▬
Disturbed ▥
No Recovery ☐

**Type: Type of Sampler**
SPT : 2 in. standard
ST : Shelby
FP : Fixed Piston
G : Grab
CORE

**N: Number of Blows**
WH : Weight of Hammer
WR : Weight of Rod
Standard Penetration Test : ASTM D1586
Hammer Type:

● Moisture Content %
► Plastic Limit %
◄ Liquid Limit %
▼ Ground Water Level
⊗ Shear strength in kPa (Torvane or Penetrometer)
X Shear strength in kPa (Unconfined)
⊗ Shear strength in kPa (field vane)
☒ Remolded strength in kPa
■ Percent Passing # 200 sieve

Bentonite/Grout Plug
Solid Pipe
Cuttings
Slotted Pipe
Sand/Pea-Gravel

Drill Method: Mud Rotary
Date Drilled: 4/6/2011
Logged By: GM

***THIS LOG IS FOR GEOTECHNICAL PURPOSES ONLY***

LEVELTON
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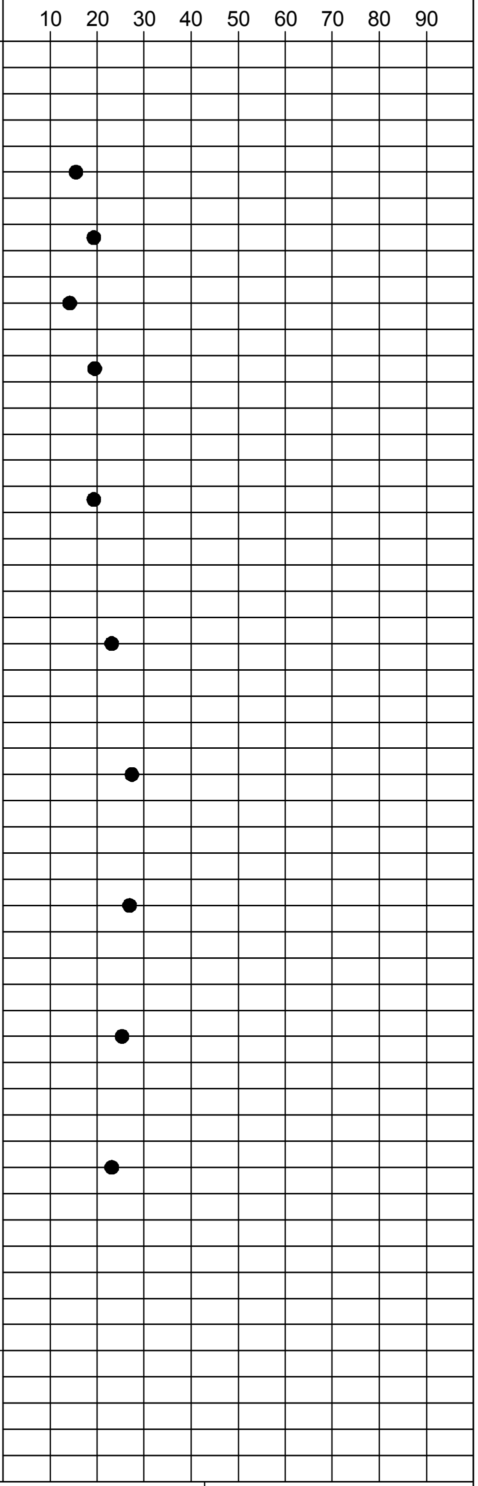
# Retaining Wall & Slope Stability Review
Hump Hillside
Marine Drive, White Rock, BC
SOIL BORING LOG

# BH11-03
Pg 1 of 1
Project No: FV11-0658-00

| Description | C | N | Type |
|---|---|---|---|
| Asphalt (200mm thick) | | | |
| Loose to compact, brown, **SAND AND GRAVEL FILL** | | 15 | SPT |
| | | 10 | SPT |
| Very dense to dense, brown, **SAND**, fine to medium grained, trace to some gravel, trace silt, gravel fine grained below 17' | | >100 | SPT |
| | | >100 | SPT |
| | | 70 | SPT |
| | | >100 | SPT |
| | | 49 | SPT |
| Hard, brown, **SILT**, some sand, fine grained | | | |
| | | 40 | SPT |
| | | 37 | SPT |
| | | 48 | SPT |
| | | 62 | SPT |
| Very dense, grey, **SAND**, fine to medium grained, some gravel, trace silt | | >100 | SPT |
| | | >100 | SPT |
| Inclinometer casing installed to 15.24m below existing grade.<br>Bottom of hole at 15.24 meters | | | |

Depth (m): 2, 4, 6, 8, 10, 12, 14, 16
Depth (ft): 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55

Water Level



**C: Condition of Sample**
- Good ▆
- Disturbed ▥
- No Recovery ☐

**Type: Type of Sampler**
- SPT : 2 in. standard
- ST : Shelby
- FP : Fixed Piston
- G : Grab
- CORE

**N: Number of Blows**
- WH : Weight of Hammer
- WR : Weight of Rod
- Standard Penetration Test : ASTM D1586
- Hammer Type:

- ● Moisture Content %
- ► Plastic Limit %
- ◄ Liquid Limit %
- ▼ Ground Water Level
- (X) Shear strength in kPa (Torvane or Penetrometer)
- X Shear strength in kPa (Unconfined)
- ⊗ Shear strength in kPa (field vane)
- ⊠ Remolded strength in kPa
- ■ Percent Passing # 200 sieve

Drill Method: Mud Rotary
Date Drilled: 4/7/2011
Logged By: TBH

# Appendix B

# Laboratory Test Results

# Levelton Consultants Ltd.

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---

**Client:** City of White Rock
**Project:** Hump Slope - Retaining Wall Assessment
**Site Address:** Marine Drive White Rock, B.C.

**File No.:** FV11-0658-00
**Task:** N.A.

---

## Report of Grain Size Analysis

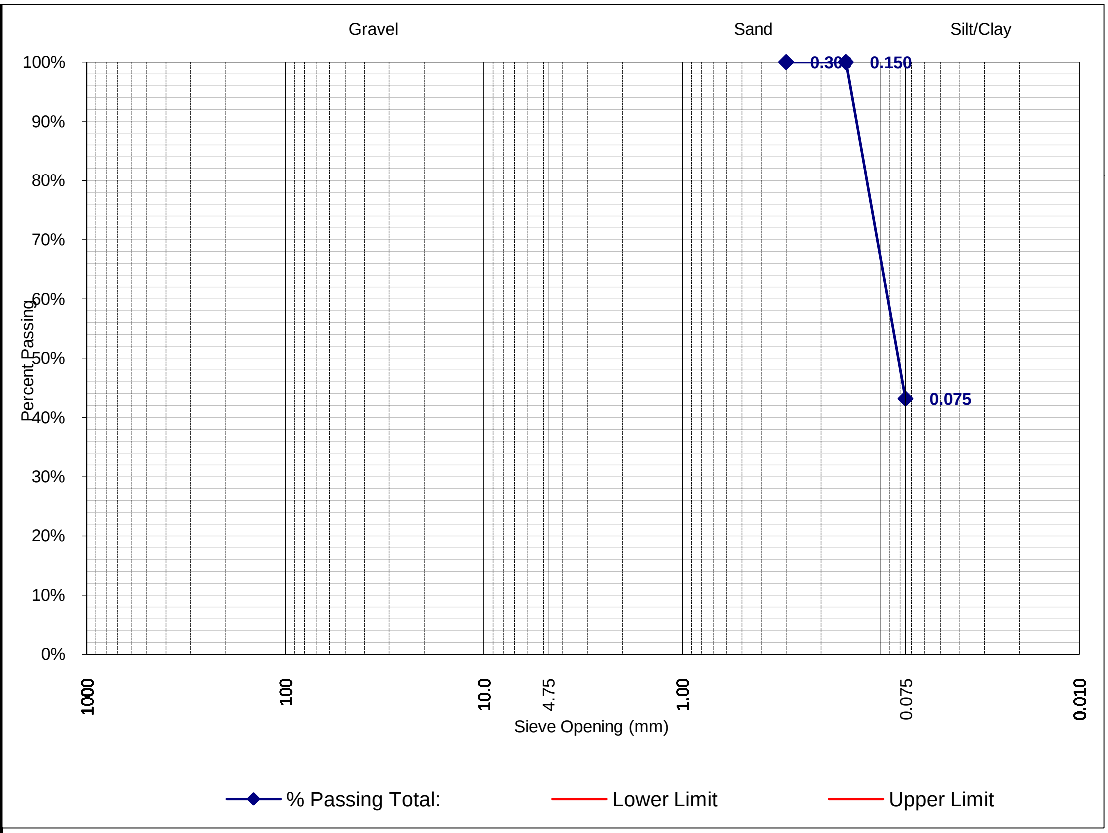
**Sample Location:** BH11 - 01 @ 23ft. - 25ft.
**Supplier:** N.A.
**Material Type:** Gray sand and silt
**Usage:** N.A.
**Specification:** N.A.

**Sampled By:** GM
**Tested By:** SM/ELD
**Date Sampled:** 04-April-2011
**Date Tested:** 19-April-2011
**Sieve No.** 200

**Moisture Content (as received):** 24%

**Washed Sieve**

| Screen Opening (mm): | % Passing Total: | Specification | |
|---|---|---|---|
| | | Upper Limit | Lower Limit |
| *150.0* | | | |
| *100.0* | | | |
| *75.0* | | | |
| *50.0* | | | |
| *37.5* | | | |
| *25.0* | | | |
| *19.0* | | | |
| *12.5* | | | |
| *9.51* | | | |
| *4.75* | | | |
| *2.36* | | | |
| *1.18* | | | |
| *0.600* | 100.0% | | |
| *0.425* | | | |
| *0.300* | 100.0% | | |
| *0.150* | 100.0% | | |
| *0.075* | 43.1% | | |



---

Remarks: ____________________

**Levelton Consultants Ltd.**

Reporting of these results constitutes a testing service only.
No engineering interpretation of the results is expressed or implied.
Engineering review and interpretation of these results can be provided upon written request.

Per: ____________________

# Levelton Consultants Ltd.

**Fraser Valley Group and Southern Interior**

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Kelowna, BC V1X 5C3
Tel: (250) 491-9778
Fax: (250) 491-9729
Email: kelowna@levelton.com

**Client:** City of White Rock
**Project:** Hump Slope - Retaining Wall Assessment
**Site Address:** Marine Drive White Rock, B.C.

**File No.:** FV11-0658-00
**Task:** N.A.

## Report of Grain Size Analysis

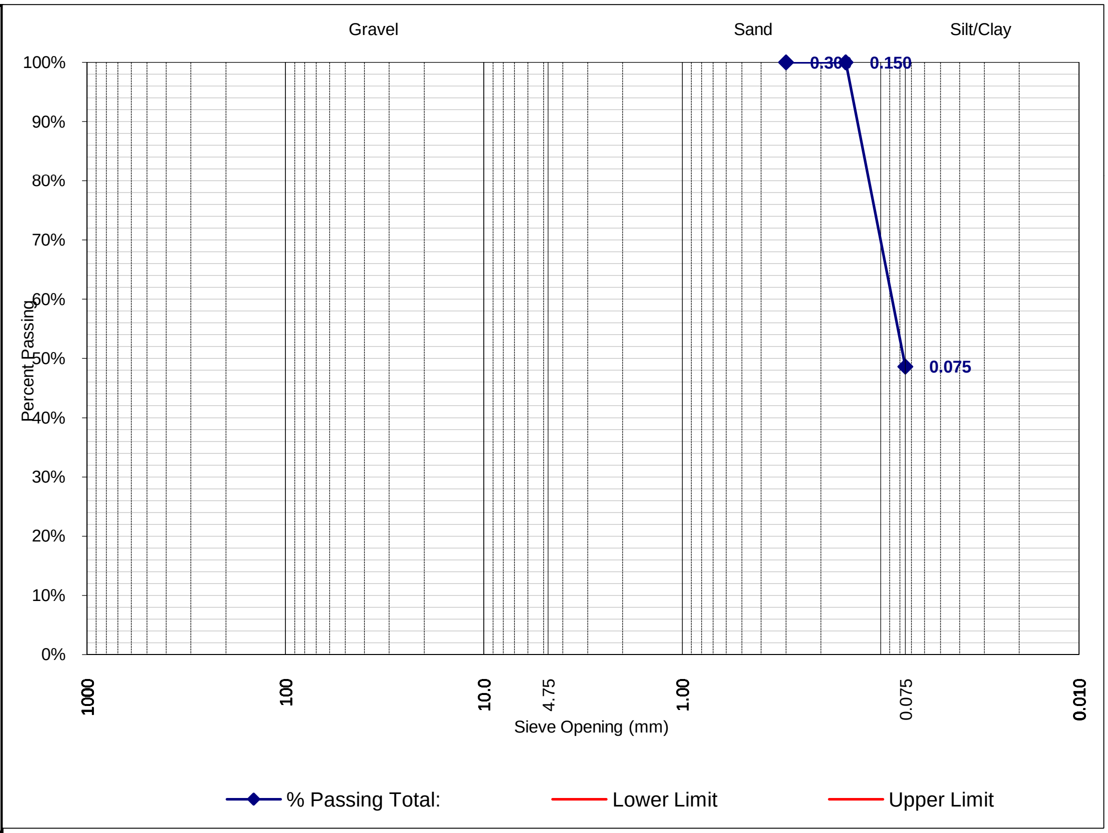
**Sample Location:** BH11 - 01 @ 38ft. - 40ft.
**Supplier:** N.A.
**Material Type:** Gray sand and silt
**Usage:** N.A.
**Specification:** N.A.

**Sampled By:** GM
**Tested By:** SM/ELD
**Date Sampled:** 04-April-2011
**Date Tested:** 19-April-2011
**Sieve No.** 200

**Moisture Content (as received):** 23%

**Washed Sieve**

| Screen Opening (mm): | % Passing Total: | Specification | |
|---|---|---|---|
| | | Upper Limit | Lower Limit |
| 150.0 | | | |
| 100.0 | | | |
| 75.0 | | | |
| 50.0 | | | |
| 37.5 | | | |
| 25.0 | | | |
| 19.0 | | | |
| 12.5 | | | |
| 9.51 | | | |
| 4.75 | | | |
| 2.36 | | | |
| 1.18 | | | |
| 0.600 | 100.0% | | |
| 0.425 | | | |
| 0.300 | 100.0% | | |
| 0.150 | 100.0% | | |
| 0.075 | 48.6% | | |



Remarks: ______

**Levelton Consultants Ltd.**

Reporting of these results constitutes a testing service only.
No engineering interpretation of the results is expressed or implied.
Engineering review and interpretation of these results can be provided upon written request.

Per: ______

# Levelton Consultants Ltd.

**Fraser Valley Group and Southern Interior**

LEVELTON

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#108, 3677 Hwy 97N
Kelowna, BC V1X 5C3
Tel: (250) 491-9778
Fax: (250) 491-9729
Email: kelowna@levelton.com

**Client:** City of White Rock
**Project:** Hump Slope - Retaining Wall Assessment
**Site Address:** Marine Drive White Rock, B.C.

**File No.:** FV11-0658-00
**Task:** N.A.

## Report of Grain Size Analysis

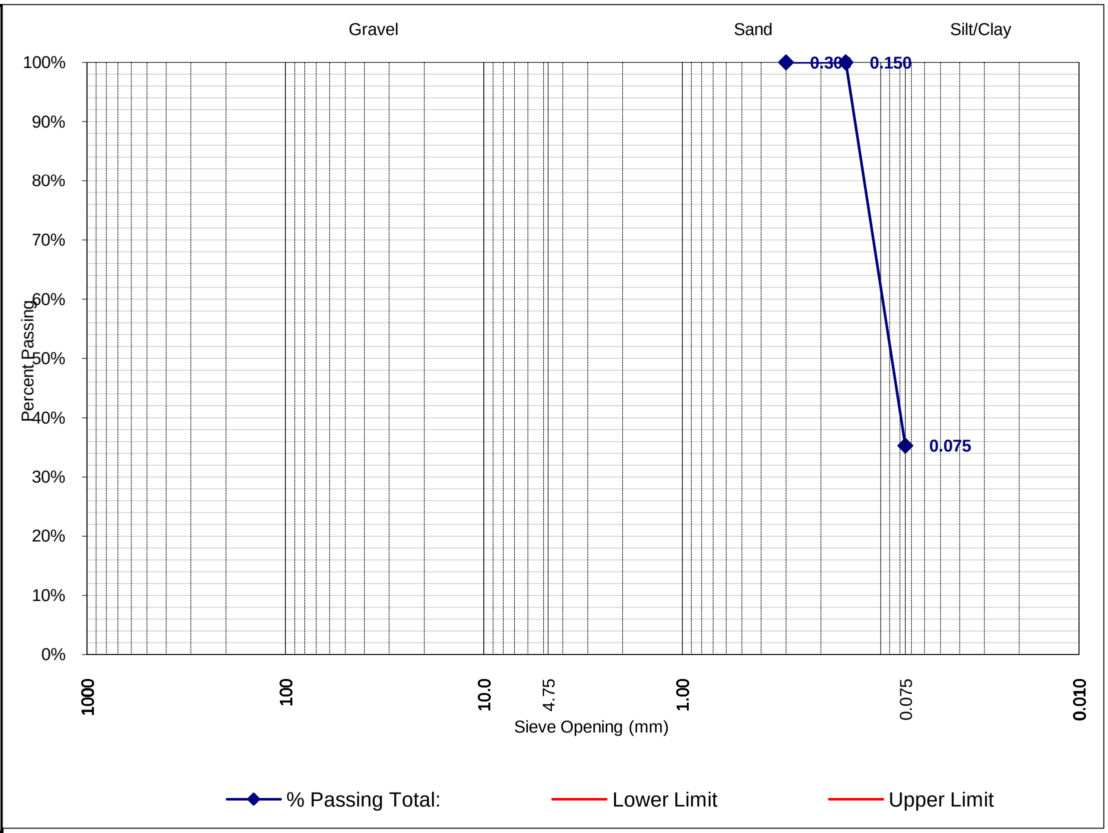
**Sample Location:** BH11 - 02 @ 28ft.
**Supplier:** N.A.
**Material Type:** Brown sand and silt
**Usage:** N.A.
**Specification:** N.A.

**Sampled By:** GM
**Tested By:** SM/ELD
**Date Sampled:** 04-April-2011
**Date Tested:** 19-April-2011
**Sieve No.** 200

**Moisture Content (as received):** 20%

**Washed Sieve**

| **Screen Opening (mm):** | **% Passing Total:** | **Specification** | |
|---|---|---|---|
| | | **Upper Limit** | **Lower Limit** |
| *150.0* | | | |
| *100.0* | | | |
| *75.0* | | | |
| *50.0* | | | |
| *37.5* | | | |
| *25.0* | | | |
| *19.0* | | | |
| *12.5* | | | |
| *9.51* | | | |
| *4.75* | | | |
| *2.36* | | | |
| *1.18* | | | |
| *0.600* | 100.0% | | |
| *0.425* | | | |
| *0.300* | 100.0% | | |
| *0.150* | 100.0% | | |
| *0.075* | 35.3% | | |



Remarks: ______________________________

**Levelton Consultants Ltd.**

Reporting of these results constitutes a testing service only.
No engineering interpretation of the results is expressed or implied.
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Per: ______________________

# Levelton Consultants Ltd.

**Fraser Valley Group and Southern Interior**

| | | |
|---|---|---|
| #110, 34077 Gladys Avenue | #301, 19292-60 Avenue | #108, 3677 Hwy 97N |
| Abbotsford, BC V2S 2E8 | Surrey, BC V3S 3M2 | Kelowna, BC V1X 5C3 |
| Tel: (604) 855-0206 | Tel: (604) 533-2992 | Tel: (250) 491-9778 |
| Fax: (604) 853-1186 | Fax: (604) 533-0768 | Fax: (250) 491-9729 |
| Email: abbotsford@levelton.com | Email: surrey@levelton.com | Email: kelowna@levelton.com |

**Client:** City of White Rock
**Project:** Hump Slope - Retaining Wall Assessment
**Site Address:** Marine Drive White Rock, B.C.

**File No.:** FV11-0658-00
**Task:** N.A.

## Report of Grain Size Analysis

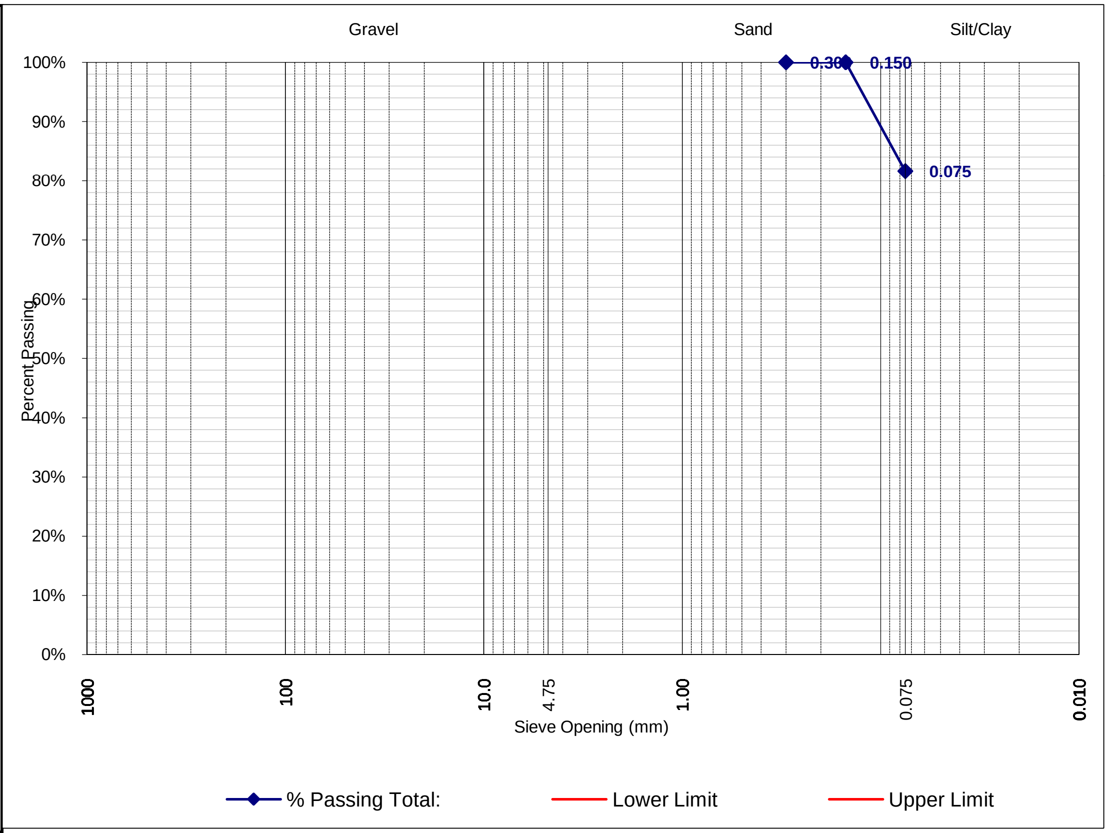
**Sample Location:** BH11 - 02 @ 38ft.
**Supplier:** N.A.
**Material Type:** Gray silt, some sand
**Usage:** N.A.
**Specification:** N.A.

**Sampled By:** GM
**Tested By:** SM/ELD
**Date Sampled:** 04-April-2011
**Date Tested:** 19-April-2011
**Sieve No.** 200

**Moisture Content (as received):** 24%

**Washed Sieve**

| Screen Opening (mm): | % Passing Total: | Specification | |
|---|---|---|---|
| | | Upper Limit | Lower Limit |
| *150.0* | | | |
| *100.0* | | | |
| *75.0* | | | |
| *50.0* | | | |
| *37.5* | | | |
| *25.0* | | | |
| *19.0* | | | |
| *12.5* | | | |
| *9.51* | | | |
| *4.75* | | | |
| *2.36* | | | |
| *1.18* | | | |
| *0.600* | 100.0% | | |
| *0.425* | | | |
| *0.300* | 100.0% | | |
| *0.150* | 100.0% | | |
| *0.075* | 81.6% | | |



Remarks: ____________________

**Levelton Consultants Ltd.**

Per: ____________________

# Levelton Consultants Ltd.

**Fraser Valley Group and Southern Interior**

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#108, 3677 Hwy 97N
Kelowna, BC V1X 5C3
Tel: (250) 491-9778
Fax: (250) 491-9729
Email: kelowna@levelton.com

---

**Client:** City of White Rock
**Project:** Hump Slope - Retaining Wall Assessment
**Site Address:** Marine Drive White Rock, B.C.

**File No.:** FV11-0658-00
**Task:** N.A.

---

## Report of Grain Size Analysis

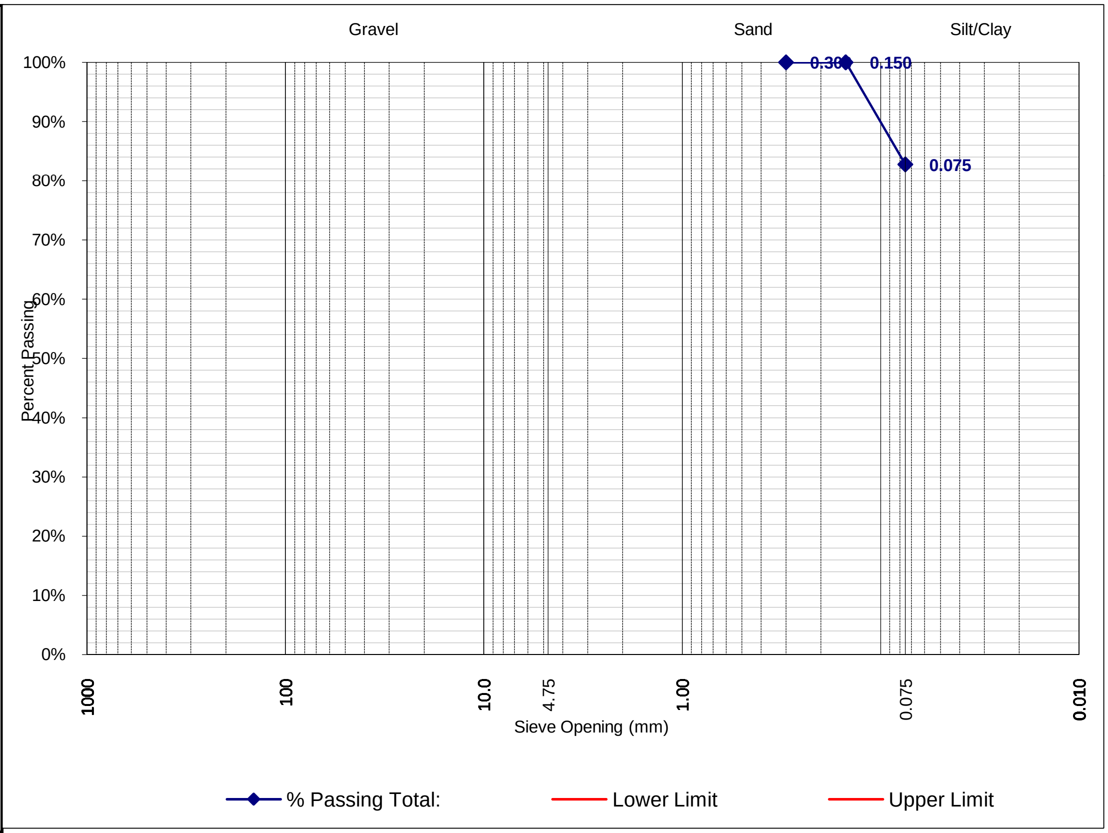
**Sample Location:** BH11 - 03 @ 28ft - 30ft.
**Supplier:** N.A.
**Material Type:** Gray silt, some sand
**Usage:** N.A.
**Specification:** N.A.

**Sampled By:** GM
**Tested By:** SM/ELD
**Date Sampled:** 04-April-2011
**Date Tested:** 19-April-2011
**Sieve No.** 200

**Moisture Content (as received):** 26%

**Washed Sieve**

| Screen Opening (mm): | % Passing Total: | Specification | |
|---|---|---|---|
| | | Upper Limit | Lower Limit |
| *150.0* | | | |
| *100.0* | | | |
| *75.0* | | | |
| *50.0* | | | |
| *37.5* | | | |
| *25.0* | | | |
| *19.0* | | | |
| *12.5* | | | |
| *9.51* | | | |
| *4.75* | | | |
| *2.36* | | | |
| *1.18* | | | |
| *0.600* | 100.0% | | |
| *0.425* | | | |
| *0.300* | 100.0% | | |
| *0.150* | 100.0% | | |
| *0.075* | 82.8% | | |



---

Remarks: ____________________

**Levelton Consultants Ltd.**

Reporting of these results constitutes a testing service only.
No engineering interpretation of the results is expressed or implied.
Engineering review and interpretation of these results can be provided upon written request.

Per: ____________________

# Appendix C

# Inclinometer Reading Results

RST Instruments Ltd.

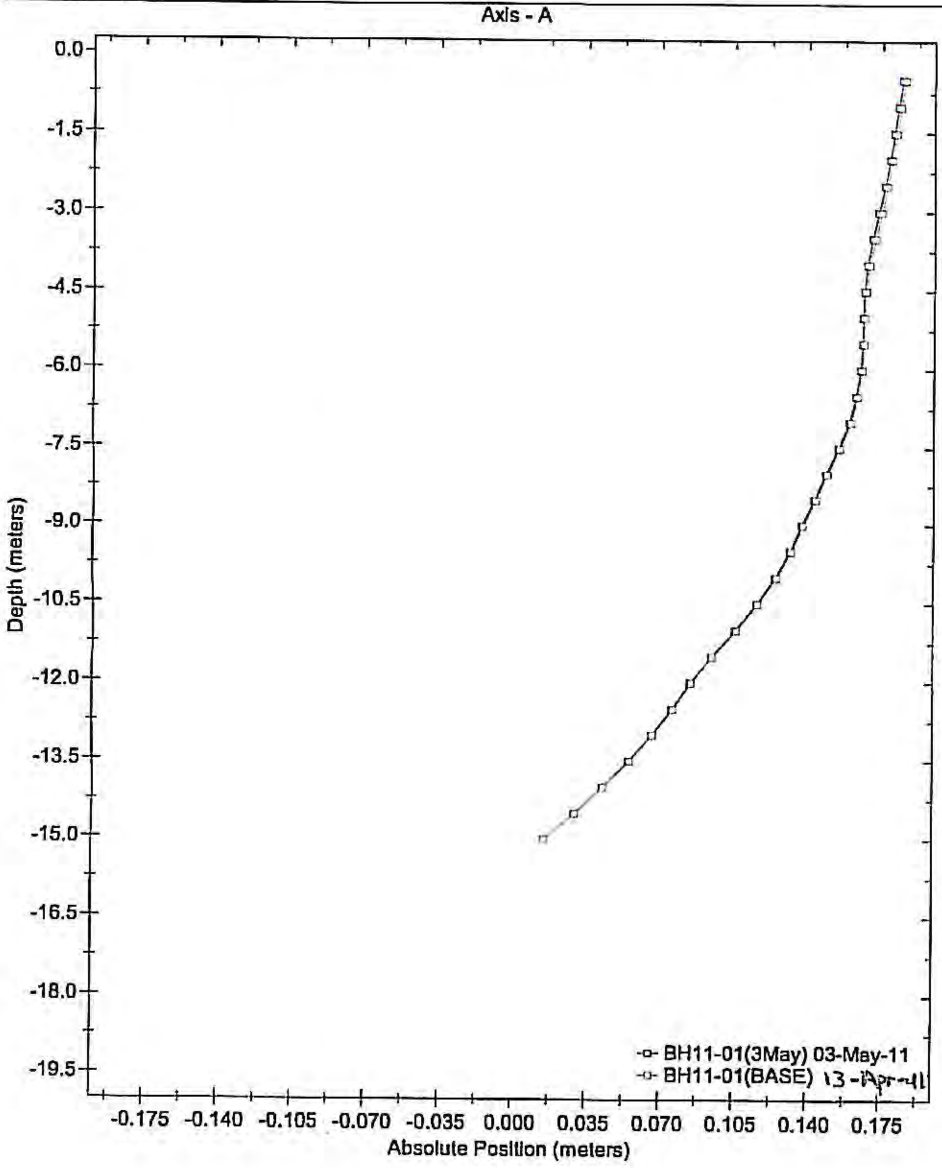
# ABSOLUTE POSITION

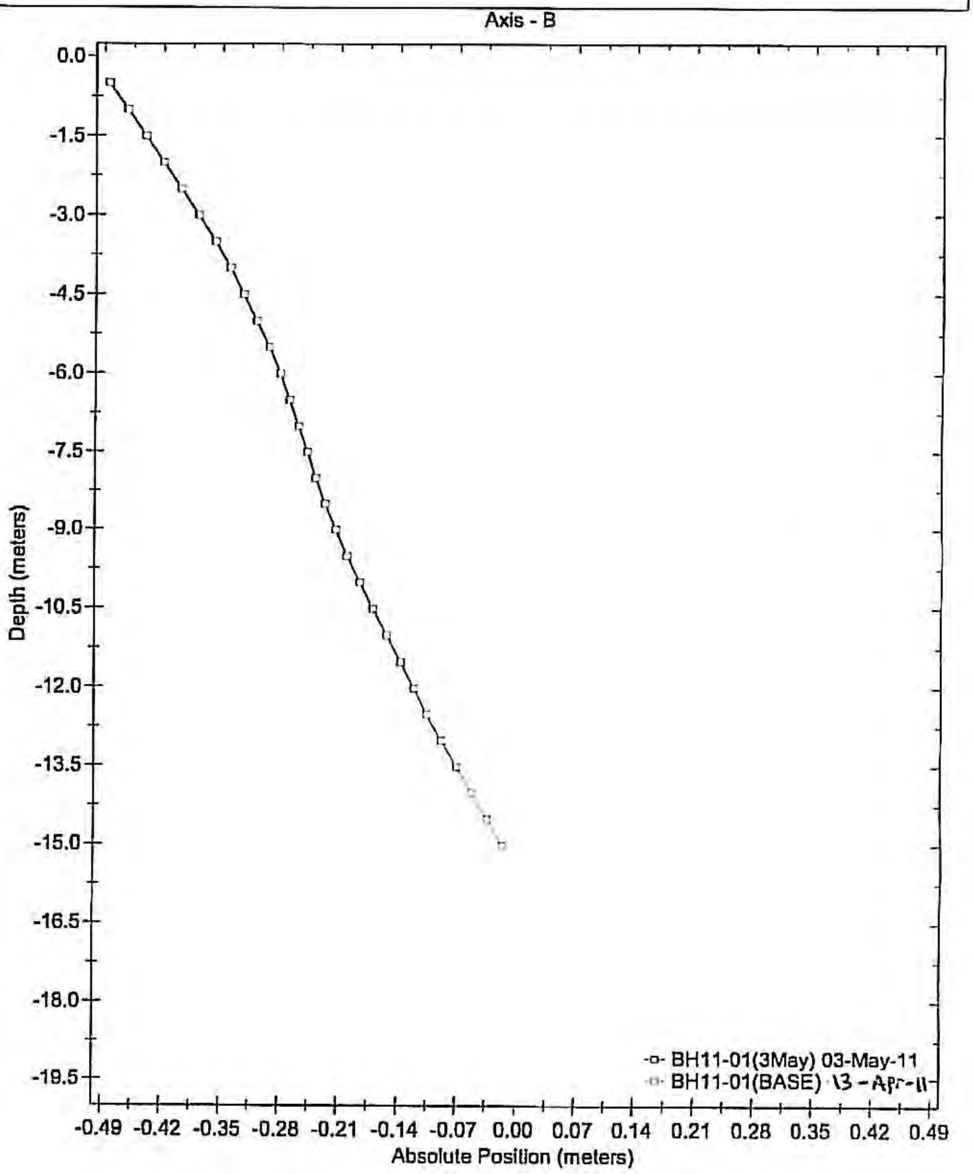
Inclinalysis v.2.30

Borehole : BH11-01
Project : FV11-0658-00
Location :
Northing :
Easting :
Collar :

Spiral Correction : N/A
Collar Elevation : 0.0 meters
Borehole Total Depth : 15.0 meters
North Groove Azimuth :
Base Reading : 13 - APR - 2011
Axis A Azimuth : 0.0 degrees

LEVELTON CONSULTANTS LTD.
Hump" Hillside Slope Stability Analysis
White Rock, BC


Axis - A


Axis - B

RST Instruments Ltd.

# ABSOLUTE POSITION

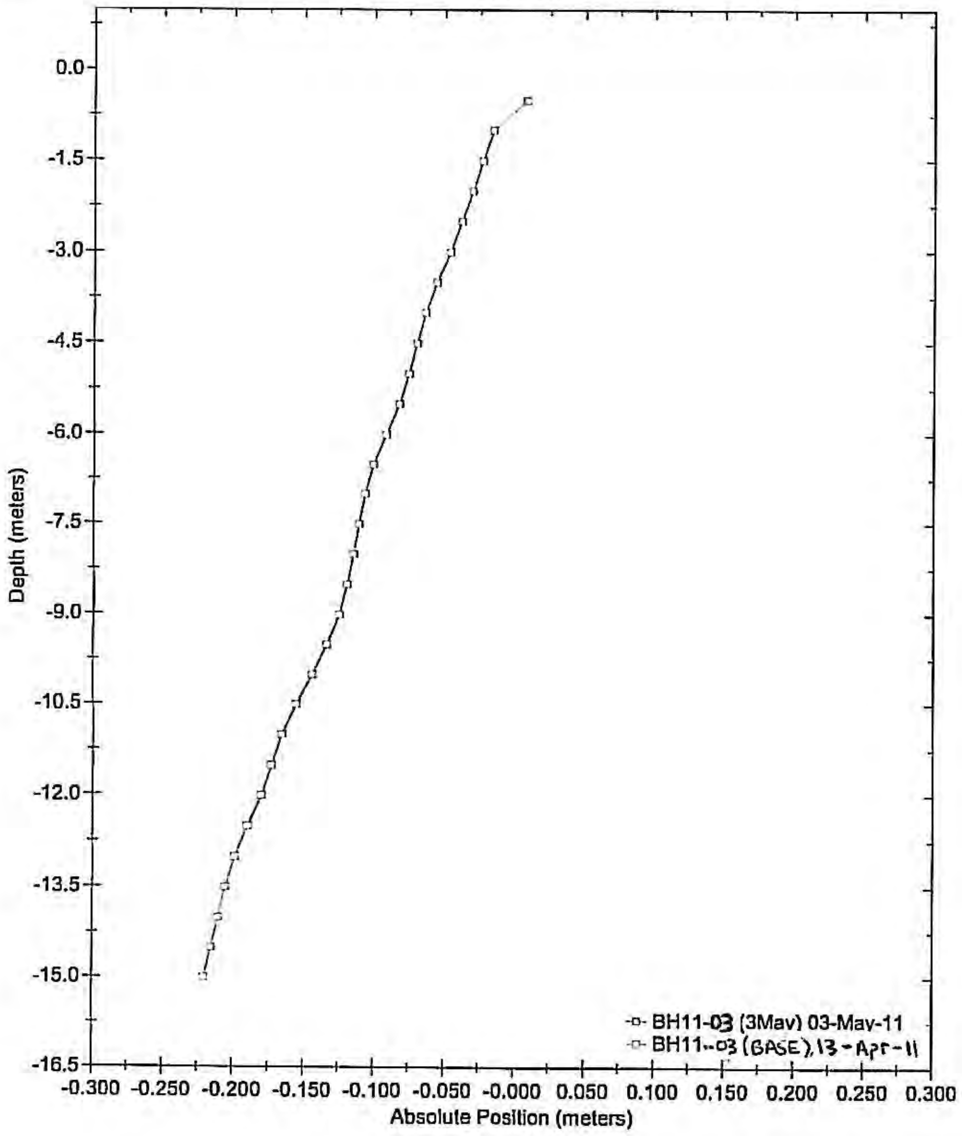
Inclinalysis v.2.30

Borehole : BH11-03
Project : FV11-0658-00
Location :
Northing :
Easting :
Collar :

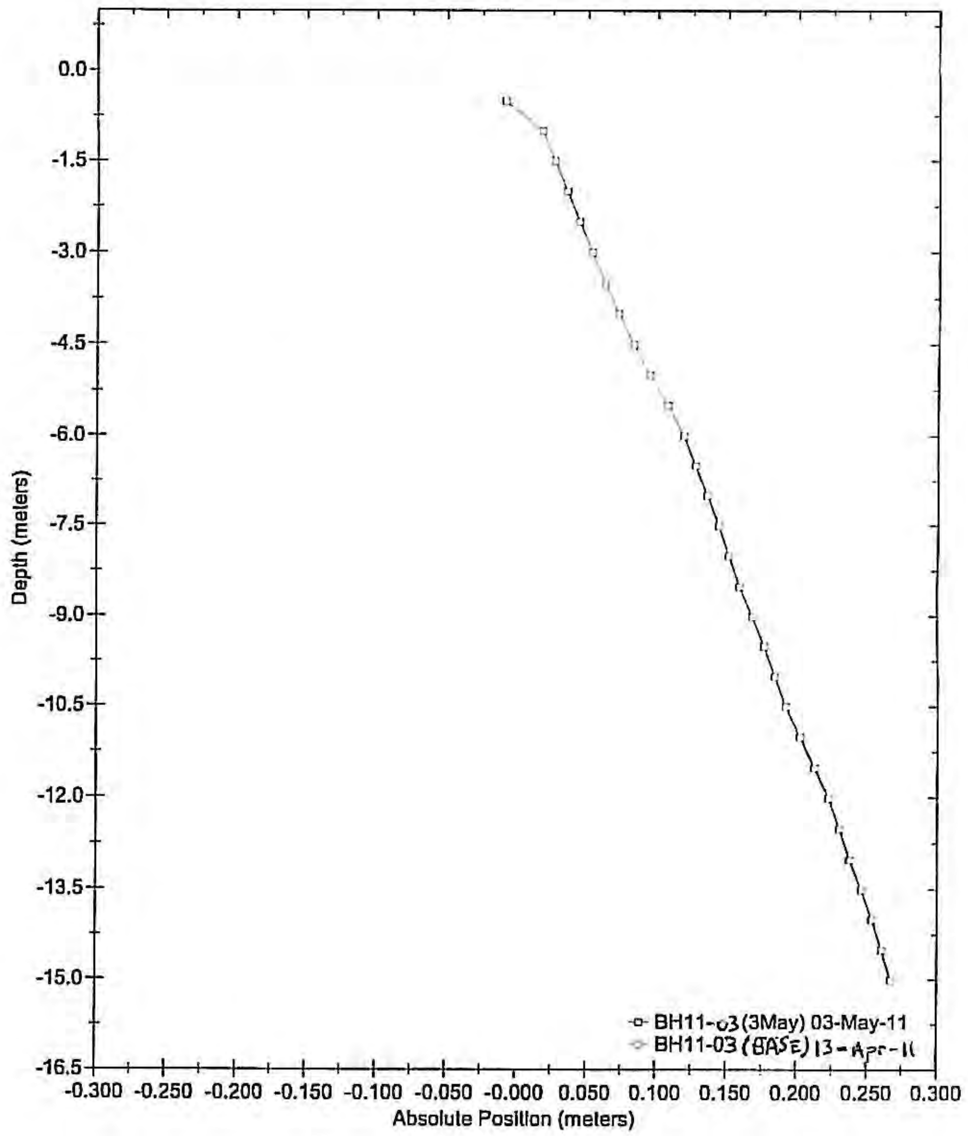
Spiral Correction : N/A
Collar Elevation : 0.0 meters
Borehole Total Depth : 15.0 meters
North Groove Azimuth :
Base Reading : 13-APR-2011
Axis A Azimuth : 0.0 degrees

LEVELTON CONSULTANTS LTD.
"Hump" Hillside Slope Stability Analysis
White Rock, BC

Axis - A



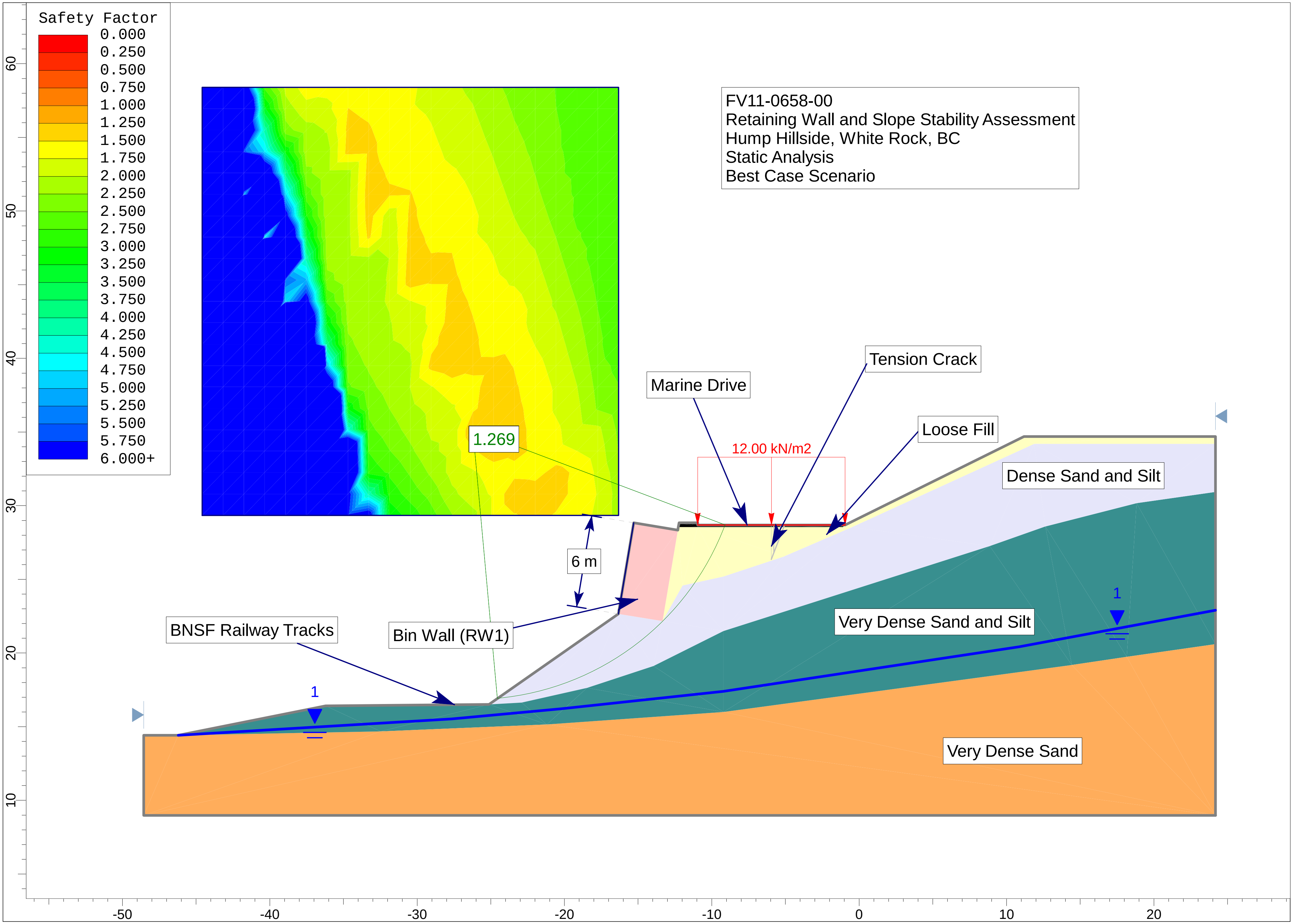
Axis - B



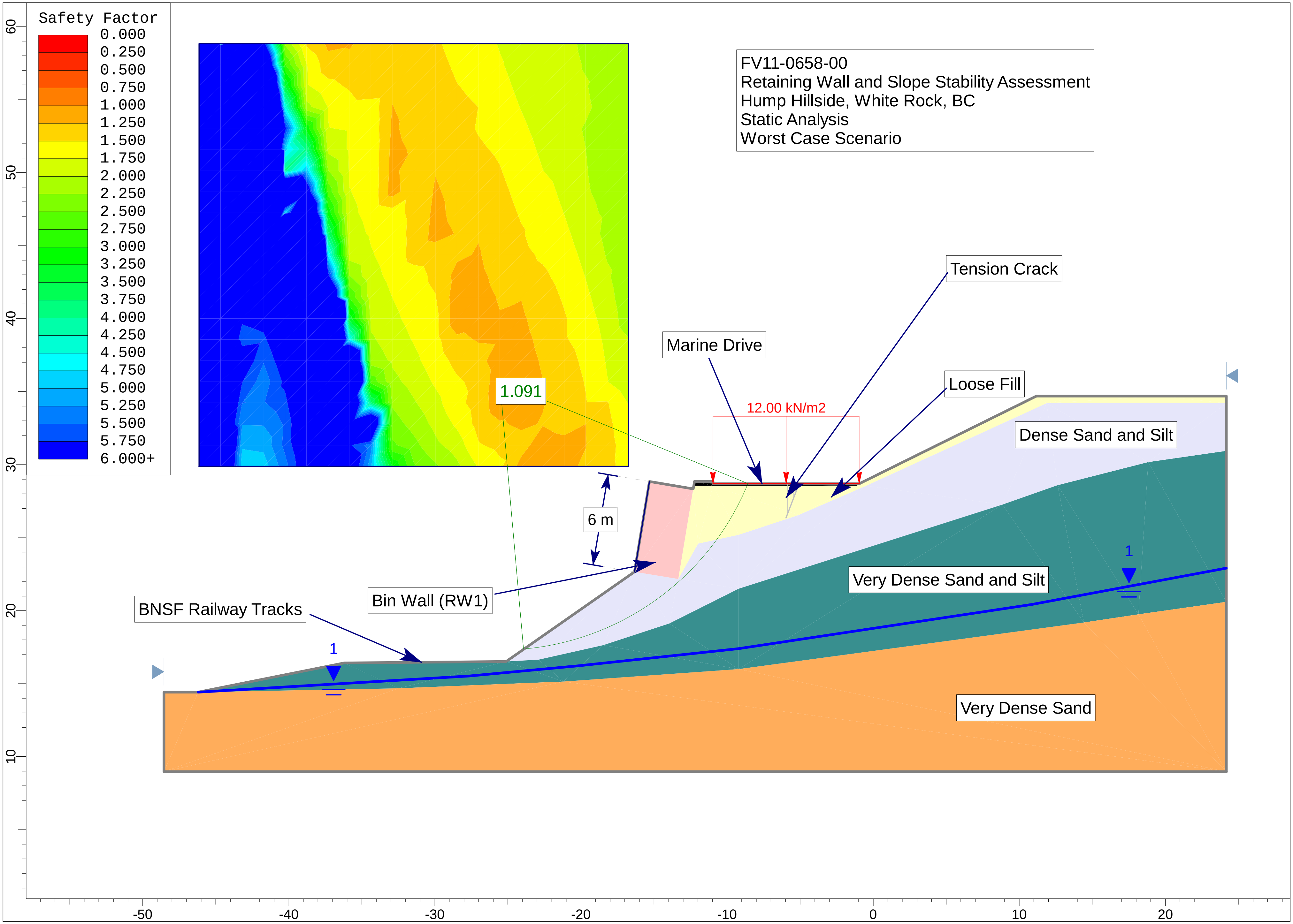
# Appendix D

## Static Stability Analysis Results

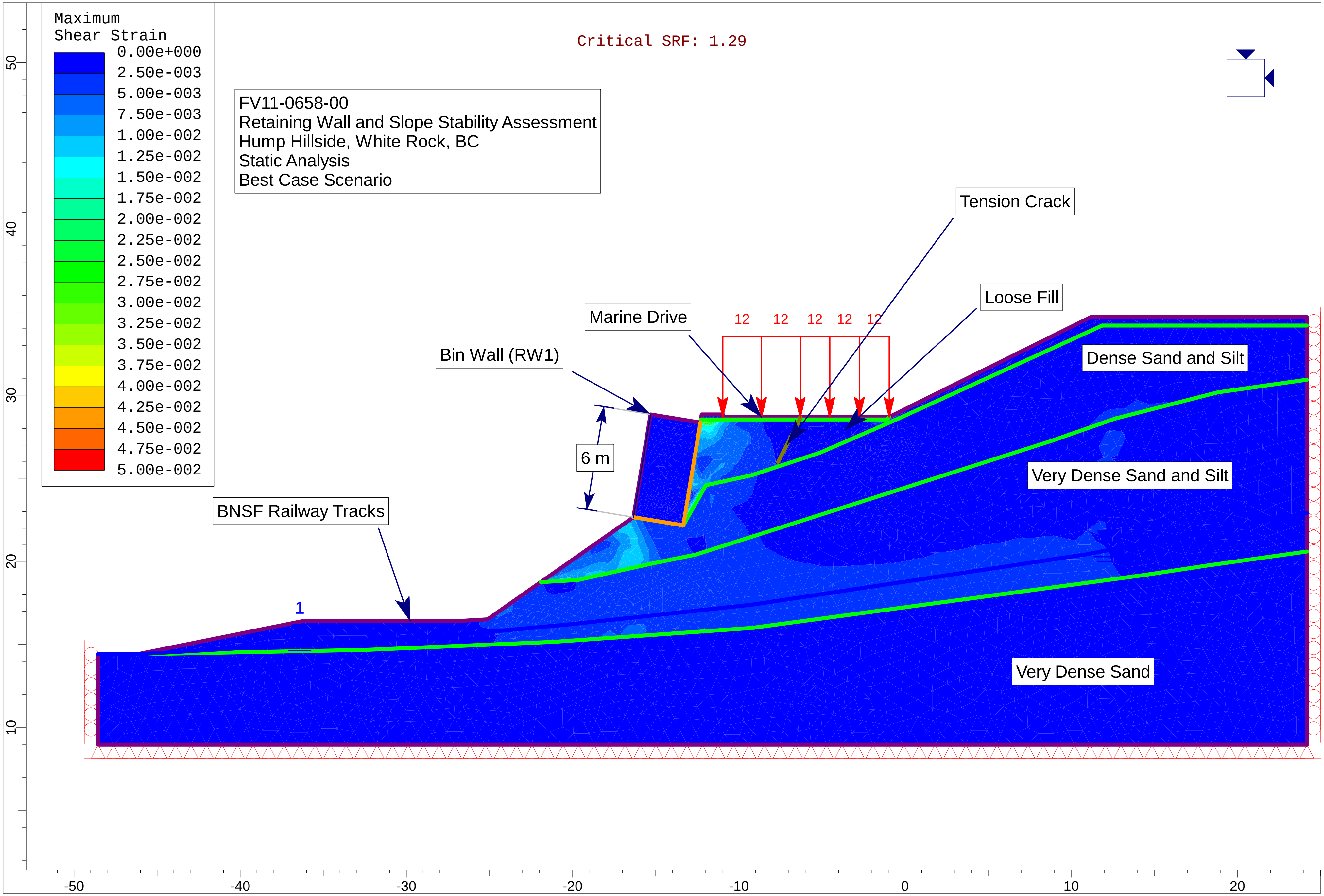
# Slide Limit Equilibrium Analysis



FV11-0658-00
Retaining Wall and Slope Stability Assessment
Hump Hillside, White Rock, BC
Static Analysis
Best Case Scenario

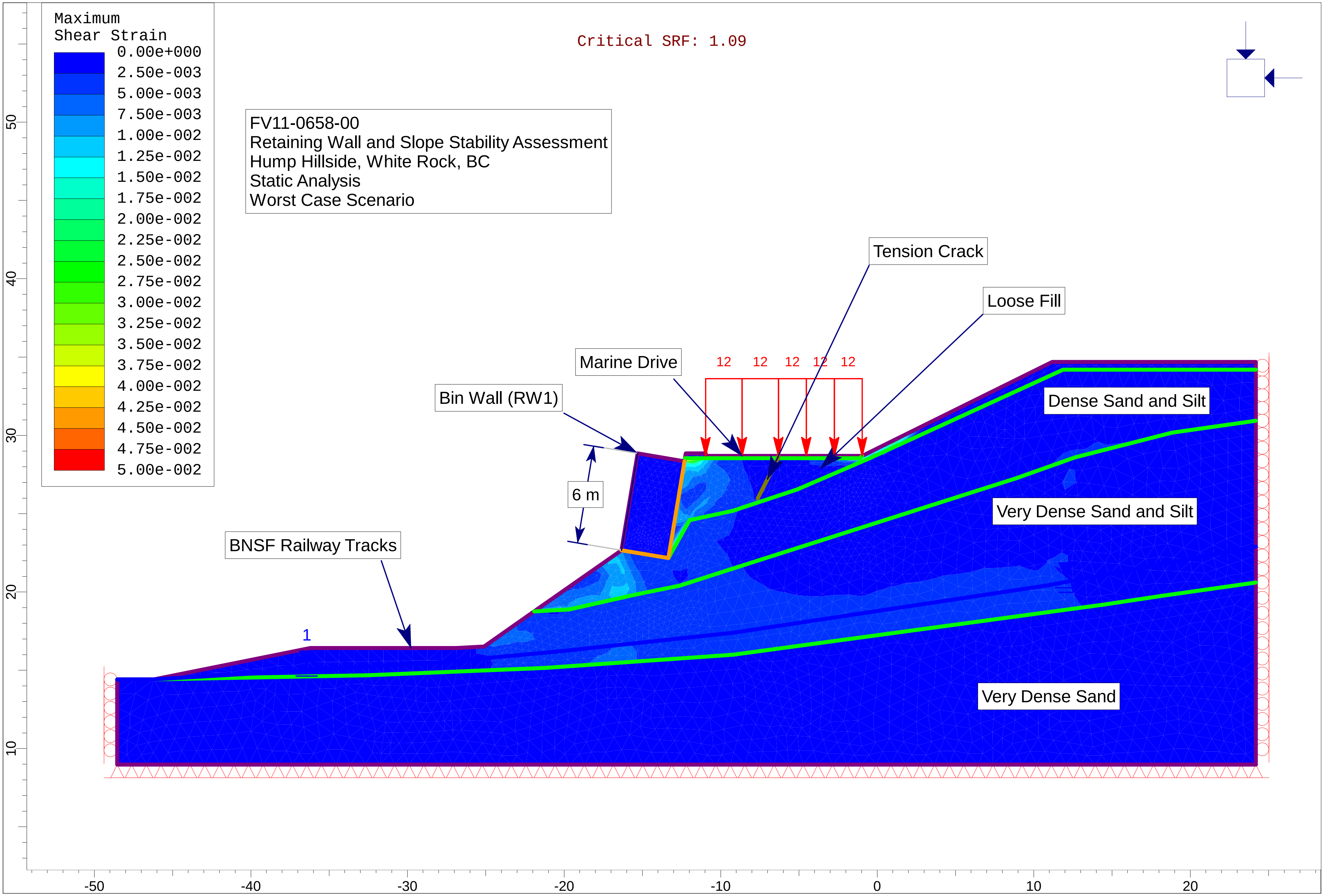

Safety Factor
0.000
0.250
0.500
0.750
1.000
1.250
1.500
1.750
2.000
2.250
2.500
2.750
3.000
3.250
3.500
3.750
4.000
4.250
4.500
4.750
5.000
5.250
5.500
5.750
6.000+
FV11-0658-00
Retaining Wall and Slope Stability Assessment
Hump Hillside, White Rock, BC
Static Analysis
Worst Case Scenario
1.091
Tension Crack
Marine Drive
Loose Fill
12.00 kN/m2
Dense Sand and Silt
6 m
Very Dense Sand and Silt
1
BNSF Railway Tracks
Bin Wall (RW1)
1
Very Dense Sand
60
50
40
30
20
10
-50
-40
-30
-20
-10
0
10
20

# Phase$^2$ Finite-Element Analysis



FV11-0658-00 Static Best Case.fez

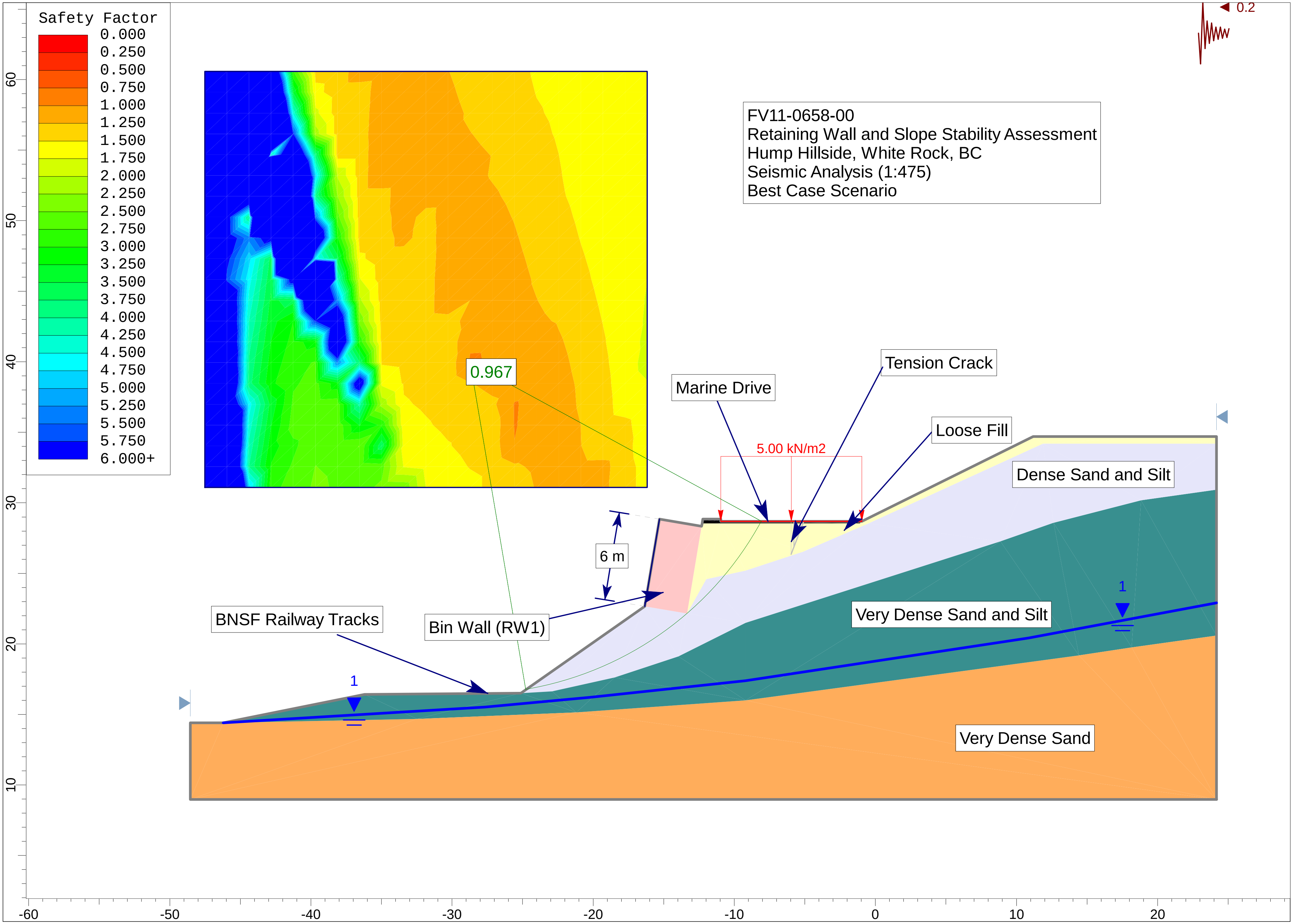
4/19/2011, 11:27:16 AM


Maximum
Shear Strain
0.00e+000
2.50e-003
5.00e-003
7.50e-003
1.00e-002
1.25e-002
1.50e-002
1.75e-002
2.00e-002
2.25e-002
2.50e-002
2.75e-002
3.00e-002
3.25e-002
3.50e-002
3.75e-002
4.00e-002
4.25e-002
4.50e-002
4.75e-002
5.00e-002
Critical SRF: 1.09
FV11-0658-00
Retaining Wall and Slope Stability Assessment
Hump Hillside, White Rock, BC
Static Analysis
Worst Case Scenario
Tension Crack
Loose Fill
Marine Drive
12
12
12
12
12
Bin Wall (RW1)
Dense Sand and Silt
6 m
Very Dense Sand and Silt
BNSF Railway Tracks
1
Very Dense Sand
50
40
30
20
10
-50
-40
-30
-20
-10
0
10
20

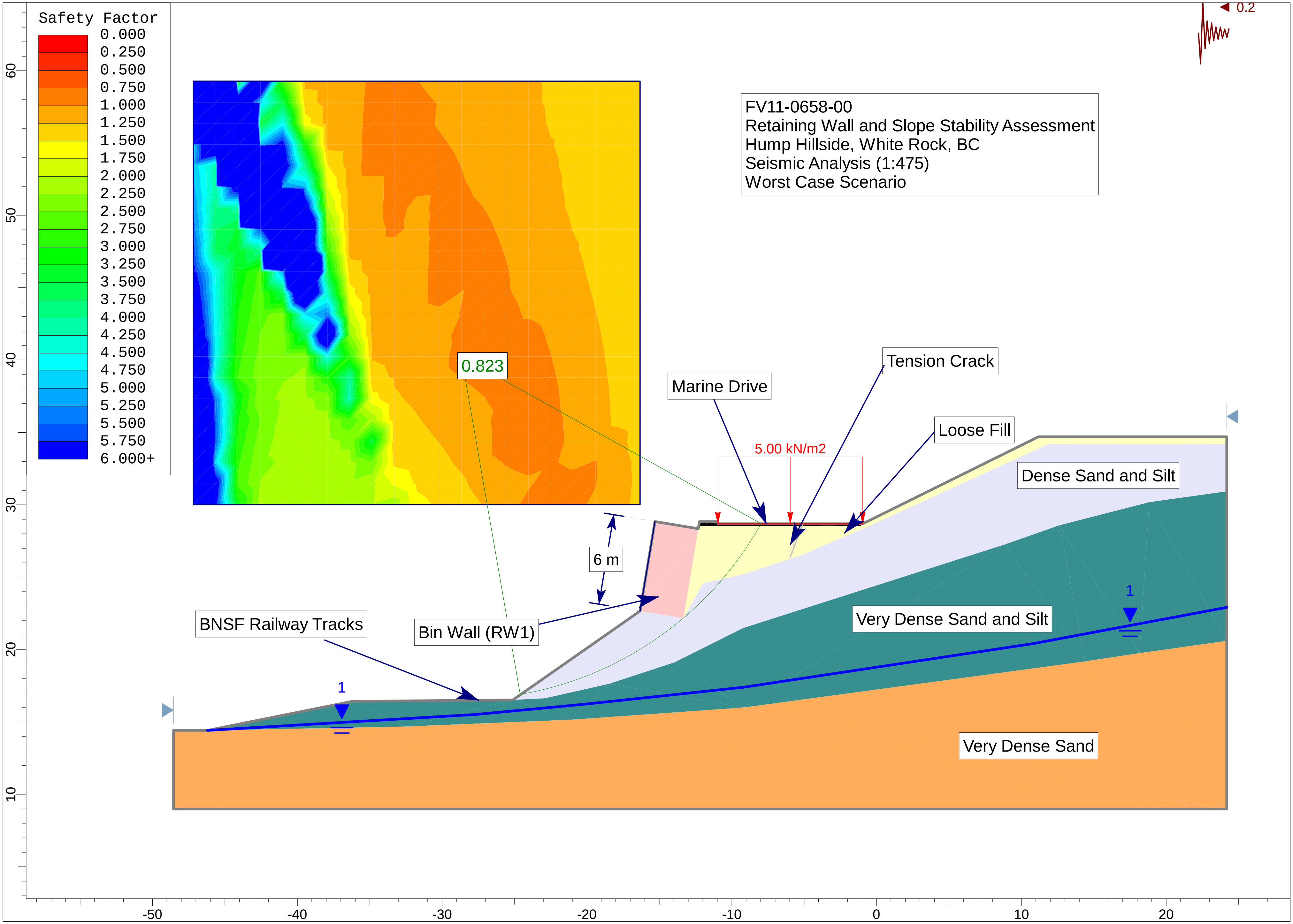
# Appendix E

# Seismic Stability Analysis Results

# Slide Limit Equilibrium Analysis

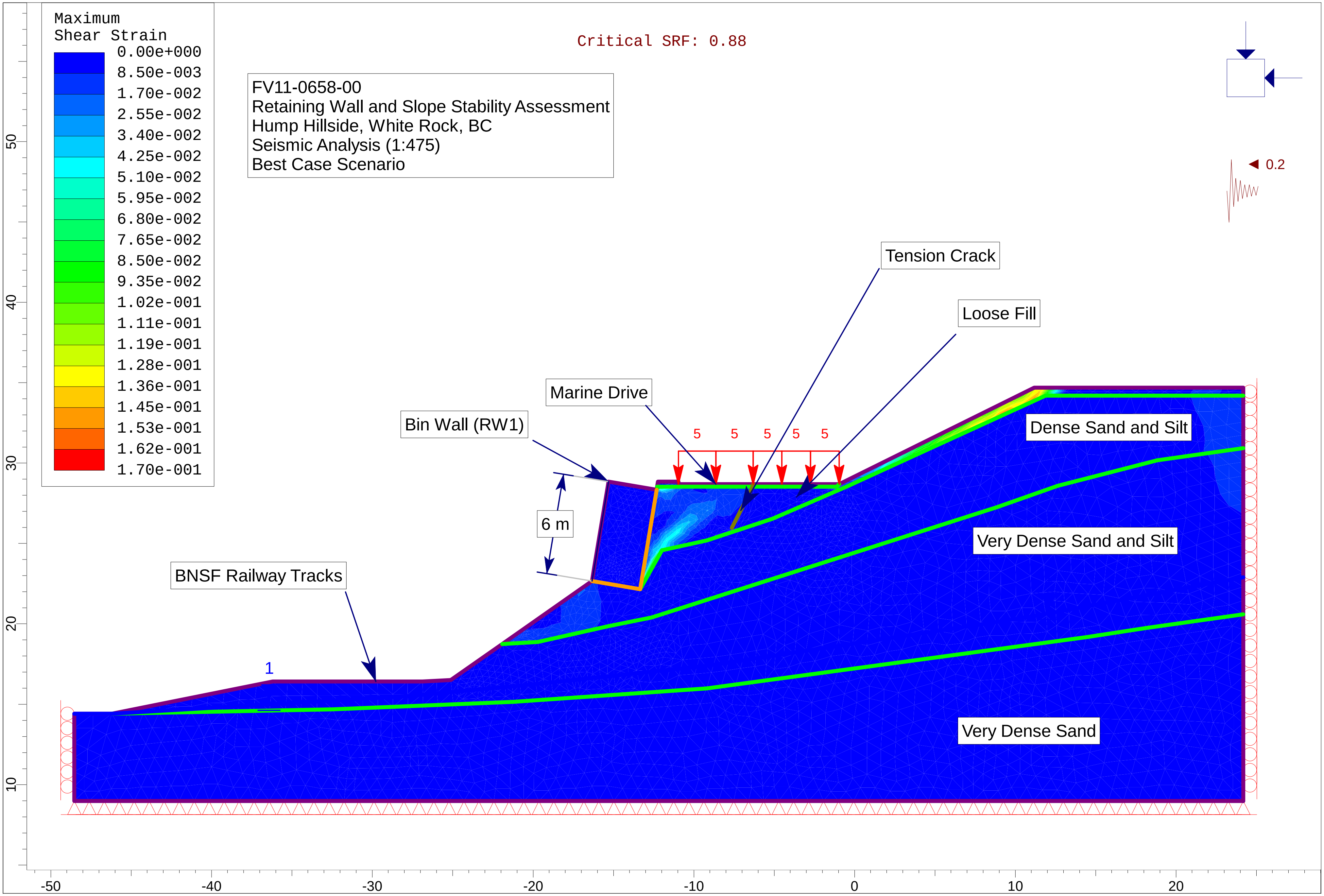


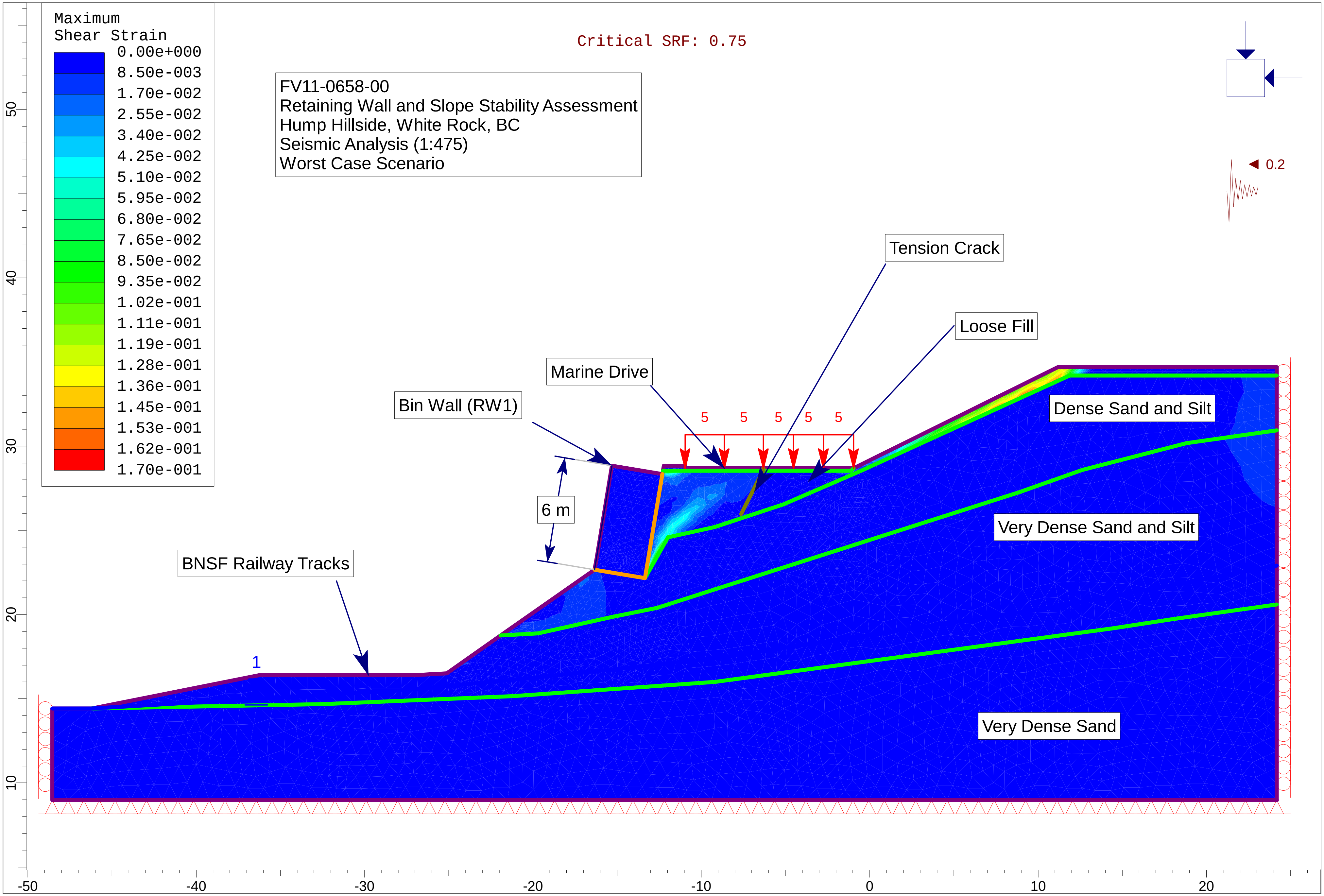
FV11-0658-00
Retaining Wall and Slope Stability Assessment
Hump Hillside, White Rock, BC
Seismic Analysis (1:475)
Best Case Scenario



FV11-0658-00
Retaining Wall and Slope Stability Assessment
Hump Hillside, White Rock, BC
Seismic Analysis (1:475)
Worst Case Scenario

# Phase$^2$ Finite-Element Analysis


Maximum Shear Strain
0.00e+000
8.50e-003
1.70e-002
2.55e-002
3.40e-002
4.25e-002
5.10e-002
5.95e-002
6.80e-002
7.65e-002
8.50e-002
9.35e-002
1.02e-001
1.11e-001
1.19e-001
1.28e-001
1.36e-001
1.45e-001
1.53e-001
1.62e-001
1.70e-001
Critical SRF: 0.88
FV11-0658-00
Retaining Wall and Slope Stability Assessment
Hump Hillside, White Rock, BC
Seismic Analysis (1:475)
Best Case Scenario
0.2
Tension Crack
Loose Fill
Marine Drive
Bin Wall (RW1)
5
5
5
5
5
6 m
Dense Sand and Silt
Very Dense Sand and Silt
BNSF Railway Tracks
1
Very Dense Sand
50
40
30
20
10
-50
-40
-30
-20
-10
0
10
20



FV11-0658-00 1_475 Worse Case.fez

4/19/2011, 11:27:16 AM

# APPENDIX B

# GEOTECHNICAL REPORT – EBA ENGINEERING CONSULTANTS, LTD. (EBA) (1994)

# APPENDIX B

## GEOTECHNICAL REPORT – EBA ENGINEERING CONSULTANTS, LTD. (EBA) (1994)

## TABLE OF CONTENTS

# EBA Engineering Consultants Ltd.

SLOPE STABILITY ASSESSMENT REPORT
Marine Drive between Foster Street
and Cypress Street
White Rock, B.C.

Prepared for:

The City of White Rock

**September 1994**

**File: 0806-86535**

Suite 550, 1100 Melville Street, Vancouver, B.C. V6E 4A6
• Telephone (604) 685-0275 • FAX (604) 684-6241 •

## Table of Contents

### LIST OF APPENDICES

## 1.0 INTRODUCTION

EBA Engineering Consultants Ltd. (EBA) has carried out a stability assessment for the slope on the south side of Marine Drive, between Foster Street and Cypress Street in White Rock, B.C. This report summarizes the results of field and laboratory works, and presents recommendations for remedial measures.

## 2.0 SCOPE OF WORK

EBA's workscope for this project was documented in the City of White Rock's Request for Proposal dated June 27, 1994 and EBA's proposal letter of July 14, 1994. It is summarized below:

- Determine soil and groundwater conditions through field investigation
- Conduct a topographic survey (by others)
- Carry out a slope stability analysis
- Make recommendations on remedial measures for the slope

## 3.0 SITE DESCRIPTION

The slope on the south side of Marine Drive, between Foster Street and Cypress Street is approximately 700 m in length and up to 20 m in height. The slope has gradients ranging from 1H:1V to 1.5H:1V.

There are signs of slope instability including ground settlement and development of cracks in the sidewalk at the crest of the slope. The handrail along the sidewalk is crooked due to the movement of soils which it is based on. It is our understanding that soil sloughing onto the Burlington Northern Railway which runs along the bottom of the slope has occurred.

The sidewalk has been resurfaced recently with asphaltic concrete. The slope is covered by dense scrub vegetation and scattered trees. There are three existing bin walls with length of up to 45 m and height of about 6 m located along the top of the slope. Some localized wood cribbings were also noted near the top of slope between the bin walls for soil retaining purpose.

Aerial photographs of the site obtained in 1940, 1954, 1963, 1976, 1984 and 1989 have been reviewed and are summarized as follows:

| | |
|---|---|
| 1940 | The subject slope existed but not much could be seen on the slope due to the poor quality of the photographs. Developments above Marine Drive extended to just beyond 20th Avenue. |
| 1954 | A few erosion/slip scars were noted on the slope surface. Some of them extended from the top to bottom of the slope. Other areas were covered with dense shrubs. |
| 1963 | The three bin walls have been constructed and trees were planted in the vicinity of the wall locations. Some of the previous slip scars were no longer visible. |
| 1976 | Trees near the bin wall locations grew bigger. |
| 1984, 1989 | Conditions similar to that in 1976. |

## 4.0 FIELD AND LABORATORY WORK

Field work was carried out on August 05, 1994 utilizing a truck mounted auger rig contracted from Uniwide Drilling Ltd. of Vancouver, B.C. The rig was equipped with 150 mm diameter solid stem augers as well as dynamic cone penetration equipment.

Four boreholes (BH1 to BH4) were drilled to depths varying from 12.2 m to 15.2 m at the top of slope at approximate locations shown in Figure 1. During the course of drilling, Dinamac Holdings Ltd. was retained to control the traffic along Marine Drive.

Soils attached to the augers were visually logged and representative samples retained for laboratory testing. A slotted PVC standpipe was installed in each of the boreholes for future groundwater level monitoring. In addition, a dynamic cone penetration test was conducted in each of BH1 and BH4. The number of blows per unit penetration of the test cone provides an indication of the subsoil consistency/relative density. The test results and the soil logs for the boreholes are presented in Appendix B.

Six typical cross-sections (A-A to F-F) of the existing slope were surveyed by Olsen & Associates Land Surveyors Ltd. The cross sections are included in Drawing 1.

Laboratory tests were subsequently performed on selected soil samples recovered from the boreholes. These included Atterberg Limits and natural moisture content. The test results are presented in the borehole logs in Appendix B.

## 5.0 SUBSURFACE CONDITIONS

The subsoil conditions encountered at the borehole locations generally consist of fill overlying silt at most areas of the slope (BH1, BH2 and BH3). At the eastern end of slope (BH4), the subsoils consist of fill overlying sand.

Fill was encountered on the surface of all four boreholes to a depth of approximately 4 to 5 m. The fill can be described as a loose to very loose sand with some gravel and trace silt. Moderate amount of organics were noted near the bottom of the fill layer.

The silt encountered beneath the fill at BH1, BH2 and BH3 is sandy with trace to some gravel. Occasional medium to coarse grained sand lenses were logged within the silt. The consistency of the silt is generally firm to stiff near the top and becomes stiff to very stiff near the bottom of the boreholes. A layer of soft to very soft silt with a thickness of approximately 3 m was encountered at depths varying from about 5.5 m to 8 m.

The sand beneath the fill at BH4 is gravelly with trace of silt. Its relative density is generally compact near the top and becomes dense as depth increases.

## 6.0 GROUNDWATER CONDITIONS

At the time of drilling, seepage was encountered in BH1, BH2 and BH3 within the soft to very soft silt at depths varying from about 5.5 m to 8 m. BH4 was dry.

Upon completion of drilling, groundwater was measured at a depth of 10.5 m in BH3. The other three boreholes were dry.

On September 1, 1994 another set of groundwater reading was taken and the results are as follows:

| Borehole No. | Groundwater Depth below Existing Grade (m) |
|---|---|
| BH1 | 5.8 |
| BH2 | Dry to bottom of standpipe at 13.7 m |
| BH3 | 10.5 |
| BH4 | Dry to bottom of standpipe at 4.9 m |

## 7.0 SLOPE STABILITY

Several computer runs were carried out using the Simplified Bishop stability analysis procedure. The analyses were performed on two cross sections A-A and D-D. Section A-A is a typical slope profile and Section D-D contains the highest bin wall at the top of slope. The shear strength parameters of the soils were chosen on the basis of the data obtained from the field and laboratory testing. The groundwater condition used in the analyses was that as measured in the field. It should be noted that earthquake forces have not been considered in the analyses in determining the existing slope safetv factor.

The results of the stability analyses indicate that the existing slope generally has a marginal safety factor (i.e. slightly over 1.0). A safety factor is defined as the ratio of average available shear strength of the soil along the critical slip surface to that required to maintain equilibrium. The most critical slip surfaces at this site are found to be those which pass through the surficial fill. The lower portion of the slope is relatively more stable than the upper (fill) portion. This is consistent with the conditions as observed on site where the sidewalk settlement and handrail leaning have occurred. This is also consistent with the fact that there are no cracks in the pavement along Marine Drive or instability of the houses on the uphill side of Marine Drive.

## 8.0 CONCLUSIONS

Based on the results of investigation, the probable sequence of events that lead to the current conditions may be summarized as follows:

- Prior to 1940, fill was placed on the downslope side of Marine Drive during road grading construction to a relatively steep gradient (close to 1H:1V). Thickness of the fill increased at areas across original ravines. The fill placement was probably not up to the current standard.

- Surficial slips/erosions subsequently occurred at areas of deep fill and eventually three bin walls were constructed to contain the fill at the most eroded areas.

- The three bin walls apparently have been performed as designed and settlement/soil movement behind the walls were minimal. The handrails at the locations of the bin walls are in much better shape than the rest of the areas.

- Areas outside these three wall locations, however, suffered further deterioration. The surficial fill continued to settle and caused severe cracking of the sidewalk at the top of slope. Some temporary remedial works such as the use of wood cribbing to retain the soil have been adopted. However, these temporary measures could only achieve short term success as the wood cribbing began to deteriorate.

The most critical slip surfaces are confined to within the surficial fill which comprises mostly of loose sand. A deep seated slope failure is considered unlikely to occur at this site. The groundwater within the soil is well below grade and is not considered as a major contributing factor to the problem.

## 9.0 RECOMMENDATIONS

### 9.1 Temporary Remedial Measures

It is our opinion that failure of the slope is/will be confined to the crest of the slope. However, a monitoring program is recommended prior to the implementation of the permanent stabilization work to confirm that our conclusion is correct. The monitoring program should comprise the following:

- groundwater level reading on the installed standpipes to monitor fluctuation of groundwater levels

- installation of four inclinometers/slope indicators to monitor the rate of lateral earth movement

- survey monitoring along the sidewalk and existing bin walls to supplement inclinometer readings

- periodic visual inspection, especially during rainy season

A minimum 2 year monitoring record is required to obtain some meaningful data which will help to optimize the design of permanent remedial works and to confirm the failure mechanism as discussed in the preceding sections. The total cost of inclinometer installation and all monitoring are in the order of $25,000 to $30,000.

It should be noted that periodic maintenance similar to that currently undertaken will still be required during the monitoring period prior to the implementation of permanent remedial measure. These may include sidewalk paving, handrail repair and replacement/addition of wood cribbing for retaining soil.

### 9.2 Permanent Solution

The methods available for permanently stabilizing the slope may be divided into the following three broad categories:

a) Decrease the driving forces which are causing downhill movements

b) Increase the resistance of the existing slope to downhill movements

c) Retaining wall construction

Category A involves unloading the soil at the crest of the slope which will eliminate the sidewalk or portion of the pavement. This is probably considered not feasible.

Category B can usually be achieved by placement of additional fill at the toe of slope to create a flatter slope. The space limitation between the toe of the slope and the railway ruled out this option.

The construction of a retaining wall system (Category C) is considered to be the most feasible and warrant further consideration as a permanent solution to the problem along this section of Marine Drive. Two types of retaining wall may be considered:

- Gravity type including reinforced earth wall and crib/bin wall
- Cantilever type including sheet piling wall and solider pile and lagging wall

Construction of the gravity type of retaining walls will involve overexcavation of the existing fill and backfilling. Temporary shoring may be required to support the sides of excavation. For cantilever type of retaining walls excavation is not required as the construction involves driving sheet piles or H-piles at the interface of fill and native soil. Limited backfilling behind these walls will be required.

Construction of the retaining wall may be staged between a period of 3 to 5 years after the two year monitoring period.

A rough cost estimate for the construction of retaining wall is in the order of $450 to $600 per m$^2$ of surface wall area. Allow a total wall length of approximately 600m and an average height of about 3 m, the total construction cost will be approximately $0.8 to 1.1 million.

## 10.0 LIMITATIONS

Recommendations presented herein are based on a geotechnical evaluation of the findings in four boreholes. The conditions encountered during the fieldwork are considered to be reasonably representative of the site. If, however, conditions other than those reported be noted during subsequent phases of the project, EBA should be notified and given the opportunity to review our current recommendations in light of new findings.

This report has been prepared for the exclusive use of The City of White Rock. It has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty is made, either expressed or implied.

For further limitations, reference should be made to the General Conditions in Appendix A of this report.

## 11.0 CLOSURE

We trust this report meets your present requirements. We would be pleased to provide any further details on our recommended monitoring program and the geotechnical input on the permanent stabilizing measure. Should you have questions on this report or require any additional information, please do not hesitate to contact our office.

Yours truly,

**EBA ENGINEERING CONSULTANTS LTD.**

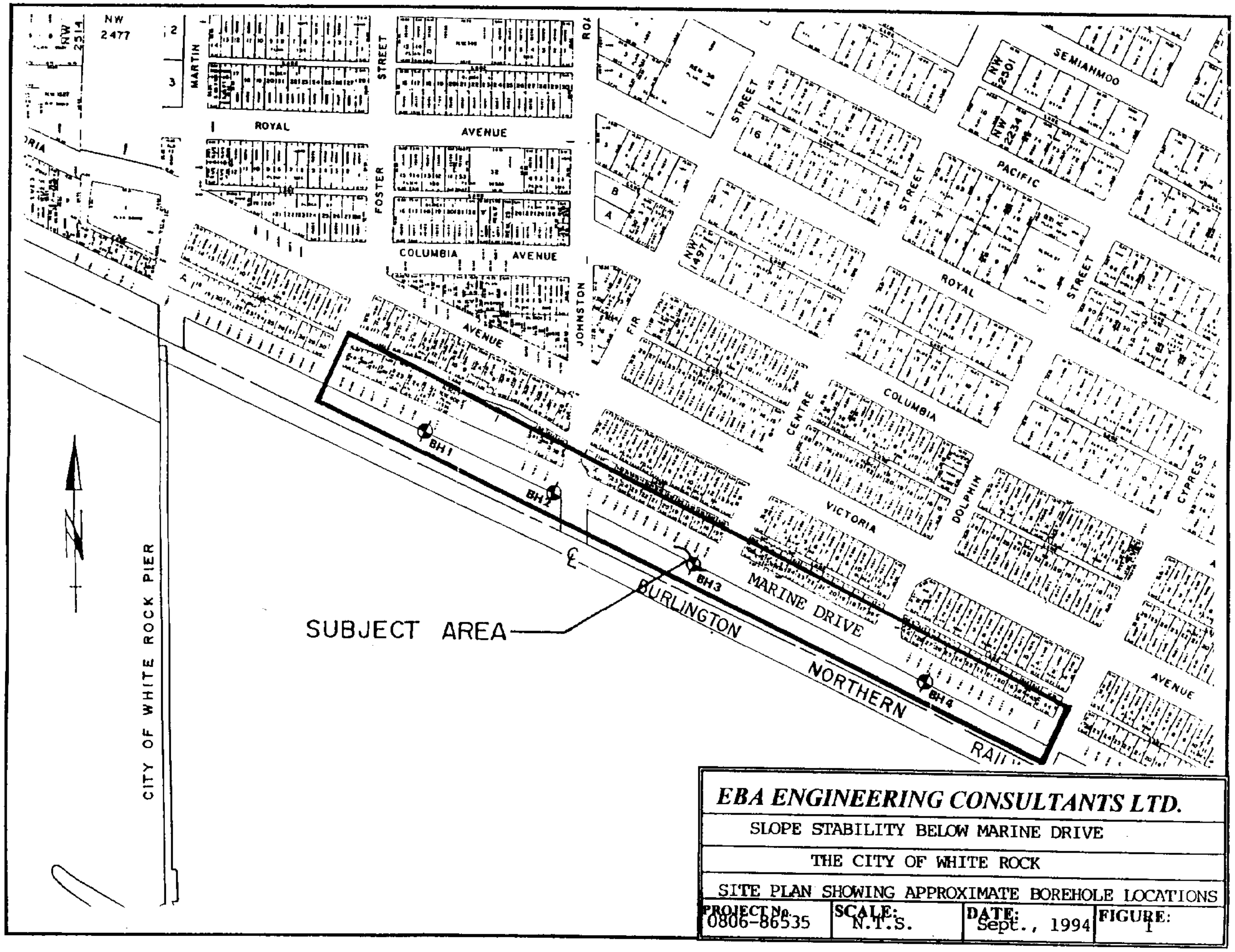
James Lau
Sep 15, 1994

James W. Lau, P.Eng
Project Engineer

Patrick Y. Chiu, P.Eng.
Project Director

JWL\PYC\cl.

**FIGURE 1**



**EBA ENGINEERING CONSULTANTS LTD.**

SLOPE STABILITY BELOW MARINE DRIVE

THE CITY OF WHITE ROCK

SITE PLAN SHOWING APPROXIMATE BOREHOLE LOCATIONS

| PROJECT No. | SCALE: | DATE: | FIGURE: |
|---|---|---|---|
| 0806-86535 | N.T.S. | Sept., 1994 | 1 |

DRAWING 1

# Appendix A

# General Conditions

EBA ENGINEERING CONSULTANTS LTD.

# GEOTECHNICAL REPORT
# GENERAL CONDITIONS

## A.1 USE OF REPORT AND OWNERSHIP

This geotechnical report pertains to a specific site and development. It is not applicable to adjacent sites nor is it valid for types of development other than that to which it refers. Any variation from the site, or development, necessitates a geotechnical review in order to determine the validity of the design concepts evolved herein.

This report is not to be reproduced in part or in whole without consent in writing from EBA Engineering Consultants Ltd. (EBA). Additional copies of the report, if required, may be obtained upon request. Isolated information, logs of borings, or profiles are not to be reproduced, copied or transferred.

## A.2 NATURE AND EXACTNESS OF SOIL DESCRIPTION

Classification and identification of soils are based upon commonly accepted methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system prevail, they are specifically mentioned.

Classification and identification of soil and geologic units are judgmental in nature as to both type and condition. EBA does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

## A.3 LOGS OF BORINGS

The boring logs are a compilation of conditions and classification of soils as obtained from field observations and laboratory testing of selected samples. Soil zones have been interpreted. Change from one geologic zone to the other, indicated on the logs as a distinct line, is in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil zone transition elevations may require special evaluation.

## A.4 STRATIGRAPHIC AND GEOLOGIC SECTIONS

The stratigraphic and geologic sections indicated on drawings contained in this report are evolved from logs of borings. Stratigraphy is known precisely only at the locations of the borings. Actual geology and stratigraphy between borings may vary from that shown on these drawings. Natural variations in geologic conditions are inherent and a function of historic environment. EBA does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of exact locations of geologic units is necessary, it is cautioned that such determination requires special attention.

## A.5 GROUNDWATER CONDITIONS

Groundwater conditions represented in this report refer only to those observed at the times recorded on logs of borings, and/or within the text of this report. These conditions vary with geologic detail between borings; annual, seasonal and special meteorologic conditions; and with construction activity. Where instruments have been established to record groundwater variations on an ongoing basis, the records will be specifically referred to. Interpretation of groundwater conditions from observations and records is judgmental and constitutes an evaluation of circumstances as influenced by geology, meteorology and construction activity. Deviations from these observations, may occur. No other warranty, express, or implied, is made by EBA.

## A.6 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geologic materials to meteorological elements. Many geologic materials deteriorate rapidly upon exposure to climatic elements. Severe deterioration of materials may be caused by precipitation and/or the action of frost on exposures. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from elements, particularly all forms of moisture, desiccation from arid conditions and frost action.

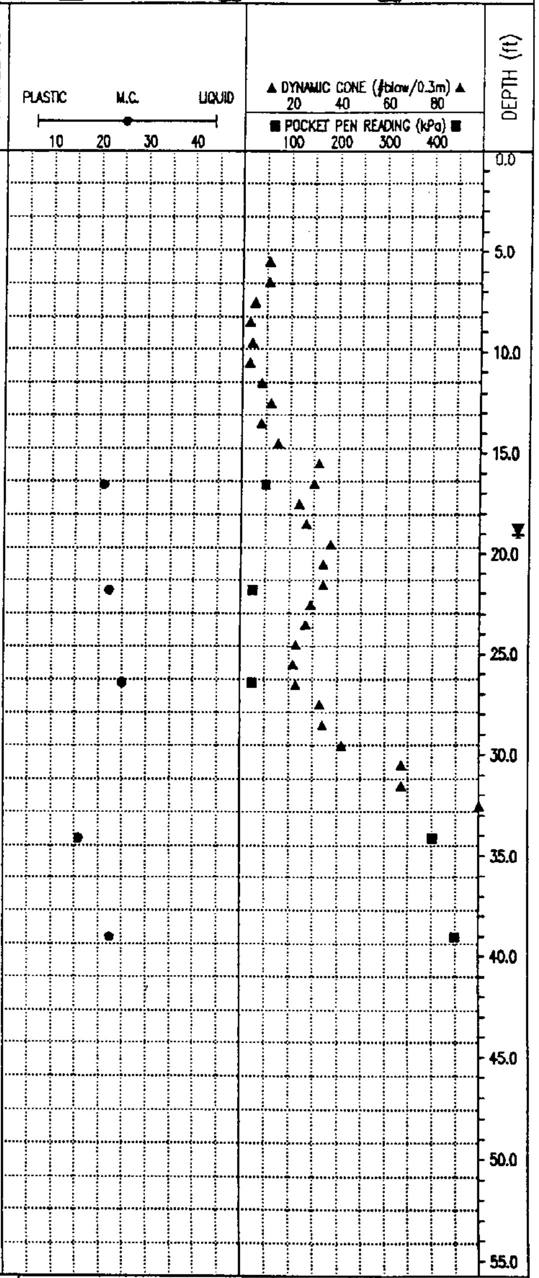
# Appendix B

## Borehole Logs

| | | |
|---|---|---|
| PROJECT: SLOPE STABILITY BELOW MARINE DRIVE | BOREHOLE LOCATION: SEE FIGURE 1 | BOREHOLE No: 86535-BH1 |
| FROM FOSTER TO CYPRESS, WHITE ROCK, B.C. | DRILL RIG: 150mm SOLID STEM AUGERS | Project No: 0806-86535 |
| CLIENT: THE CITY OF WHITE ROCK | RIG SUPPLIED BY: UNIWIDE DRILLING LTD. | ELEVATION: 18.200 (m) |

SAMPLE TYPE ■ DISTURBED ▨ NO RECOVERY ☒ SPT ☰ A-CASING ▯ SHELBY TUBE ▮ CORE

| DEPTH (m) | SOIL DESCRIPTION | SAMPLE NO |
|---|---|---|
| 0.0 | ASPHALT | |
| | SAND (FILL) - loose, some gravel, trace silt, fine to medium grained, dry to damp, brown to light brown | B1 |
| 2.0 | - becomes loose to very loose, some silt, trace gravel | B2 |
| 4.0 | - trace wood debris, silty, damp to moist | B3 |
| 5.0 | SILT - soft to firm, some sand, trace gravel, non-plastic, moist, brown<br>- becomes soft to very soft, wet, grey | B4 |
| 6.0 | interbedded medium grained sand, wet | |
| | | B5 |
| 8.0 | | B6 |
| 9.0 | - becomes stiff to very stiff, damp to moist | |
| 10.0 | - some gravel, very stiff to hard | B7 |
| 11.0 | interbedded fine to medium grained sand | |
| 12.0 | | B8 |
| | End of borehole at 12.2m<br>Slotted PVC standpipe installed to 6.0m<br>Dry upon completion<br>Groundwater at 5.8m depth on Sep. 1, 1994 | |



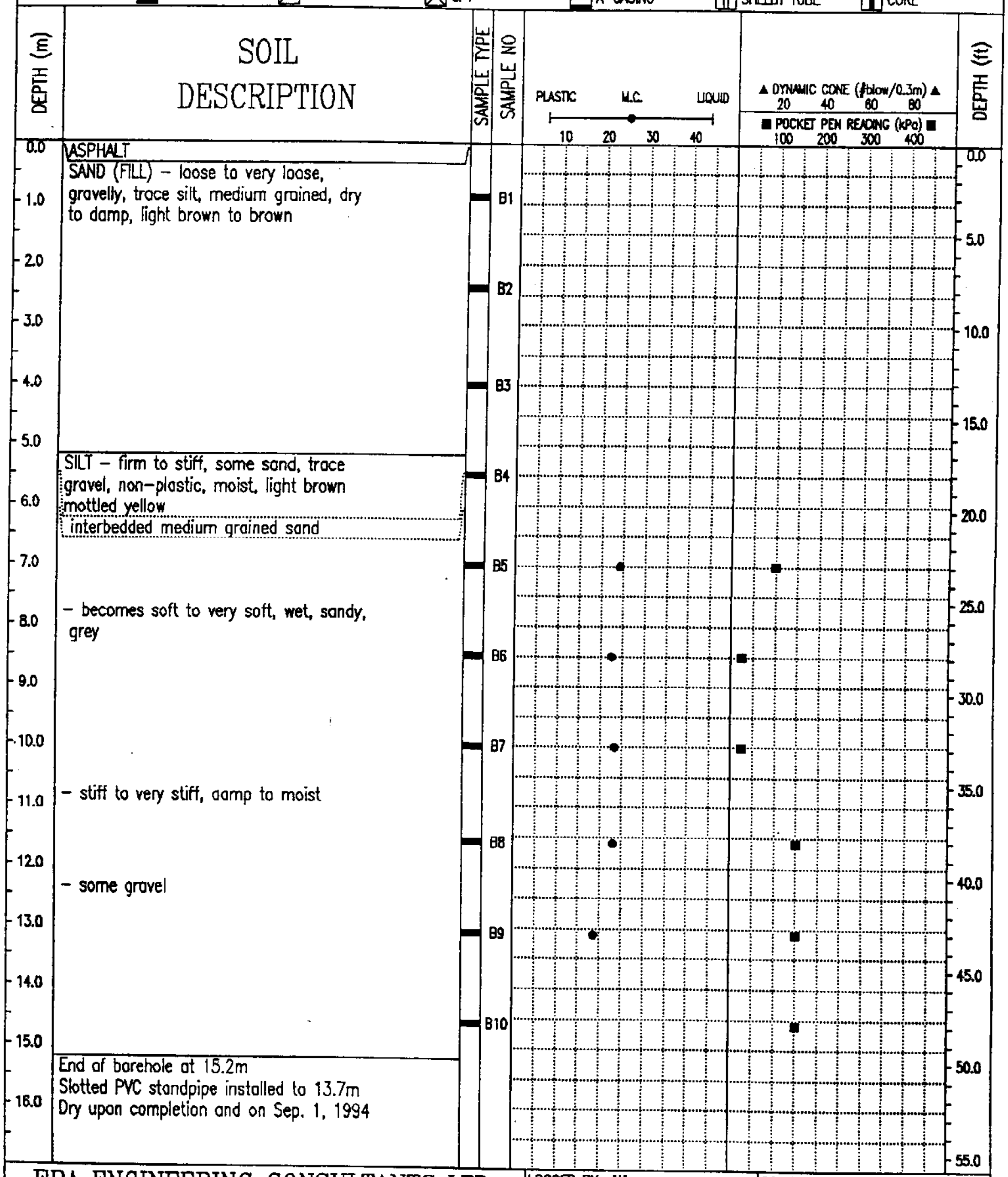
DEPTH (m): 0.0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0 11.0 12.0 13.0 14.0 15.0 16.0

# EBA ENGINEERING CONSULTANTS LTD.
Vancouver, B.C.

| | |
|---|---|
| LOGGED BY: JWL | COMPLETION DEPTH: 12.2 m |
| REVIEWED BY: PC | COMPLETE: 94/08/05 |
| Fig. No: 1 | Page 1 of 1 |

| | | |
|---|---|---|
| PROJECT: SLOPE STABILITY BELOW MARINE DRIVE | BOREHOLE LOCATION: SEE FIGURE 1 | BOREHOLE No: 86535-BH2 |
| FROM FOSTER TO CYPRESS, WHITE ROCK, B.C. | DRILL RIG: 150mm SOLID STEM AUGERS | Project No: 0806-86535 |
| CLIENT: THE CITY OF WHITE ROCK | RIG SUPPLIED BY: UNIWIDE DRILLING LTD. | ELEVATION: 22.900 (m) |

SAMPLE TYPE ■ DISTURBED ▨ NO RECOVERY ☒ SPT ☰ A-CASING ▯ SHELBY TUBE ▮ CORE



ASPHALT

SAND (FILL) – loose to very loose, gravelly, trace silt, medium grained, dry to damp, light brown to brown

SILT – firm to stiff, some sand, trace gravel, non-plastic, moist, light brown mottled yellow

interbedded medium grained sand

– becomes soft to very soft, wet, sandy, grey

– stiff to very stiff, damp to moist

– some gravel

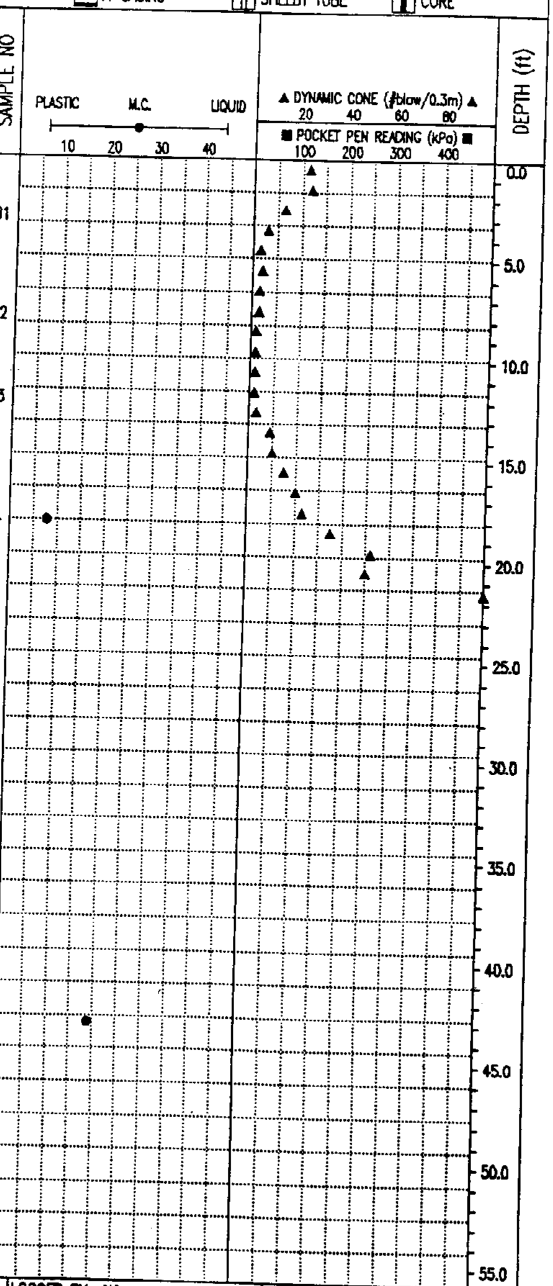
End of borehole at 15.2m
Slotted PVC standpipe installed to 13.7m
Dry upon completion and on Sep. 1, 1994

| | | |
|---|---|---|
| EBA ENGINEERING CONSULTANTS LTD.<br>Vancouver, B.C. | LOGGED BY: JWL | COMPLETION DEPTH: 15.2 m |
| | REVIEWED BY: PC | COMPLETE: 94/08/05 |
| | Fig. No: 1 | Page 1 of 1 |

| | | |
|---|---|---|
| PROJECT: SLOPE STABILITY BELOW MARINE DRIVE | BOREHOLE LOCATION: SEE FIGURE 1 | BOREHOLE No: 86535-BH4 |
| FROM FOSTER TO CYPRESS, WHITE ROCK, B.C. | DRILL RIG: 150mm SOLID STEM AUGERS | Project No: 0806-86535 |
| CLIENT: THE CITY OF WHITE ROCK | RIG SUPPLIED BY: UNIWIDE DRILLING LTD. | ELEVATION: 18.500 (m) |

SAMPLE TYPE ■ DISTURBED ▨ NO RECOVERY ☒ SPT ☰ A-CASING ▯ SHELBY TUBE ▯ CORE

| DEPTH (m) | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NO |
|---|---|---|---|
| 0.0 | ASPHALT | | |
| | SAND (FILL) – loose to very loose, gravelly to some gravel, trace silt, medium grained, dry to damp, brown | | |
| 1.0 | | ■ | B1 |
| 2.0 | | | |
| | | ■ | B2 |
| 3.0 | | | |
| | | ■ | B3 |
| | – abundant wood debris | | |
| 4.0 | SAND – compact, gravelly, trace silt, medium grained, damp, brown to light brown | | |
| 5.0 | – becomes silty<br>– compact to dense | | |
| | | ■ | B4 |
| 6.0 | | | |
| 7.0 | | ■ | B5 |
| 8.0 | | | |
| 9.0 | | ■ | B6 |
| 10.0 | | ■ | B7 |
| 11.0 | | | |
| 12.0 | | ■ | B8 |
| 13.0 | – silty | ■ | B9 |
| 14.0 | End of borehole at 13.7m<br>Slotted PVC standpipe installed to 4.9m<br>Dry upon completion and on Sep. 1, 1994 | | |
| 15.0 | | | |
| 16.0 | | | |



| | | |
|---|---|---|
| EBA ENGINEERING CONSULTANTS LTD.<br>Vancouver, B.C. | LOGGED BY: JWL | COMPLETION DEPTH: 13.7 m |
| | REVIEWED BY: PC | COMPLETE: 94/08/05 |
| | Fig. No: 1 | Page 1 of 1 |

# APPENDIX C

## GEOTECHNICAL SUMMARY OF INCLINOMETER READINGS LEVELTON CONSULTANTS, LTD. (LEVELTON) (2013)

# APPENDIX C

## GEOTECHNICAL SUMMARY OF INCLINOMETER READINGS LEVELTON CONSULTANTS, LTD. (LEVELTON) (2013)

## TABLE OF CONTENTS

**Levelton Consultants Ltd.**

**Fraser Valley Group**

301 – 19292 60th Avenue
Surrey, B.C.
Canada V3S 3M2

Tel: 604 533-2992

Fax: 604 533-0768

E-Mail: surrey@levelton.com

#110, 34077 Gladys Ave
Abbotsford, B.C.
Canada V2S 2E8

Tel: 604 855-0206

Fax: 604 853-1186

E-Mail: abbotsford@levelton.com

**Southern Interior**

108 – 3677 Highway 97N
Kelowna, B.C.
Canada, V1X 5C3

Tel: 250-491-9778

Fax: 250-491-9729

Email: kelowna@levelton.com

Web Site: www.levelton.com

*Construction Materials*

*Building Science*

*Geotechnical*

*Corrosion Prevention*

*Metallurgy*

*Environmental*

*Physical Testing*

April 25, 2013
File: FV11-0658-01

The Corporation of the City of White Rock
877 Keil Street
White Rock, BC
V4B 4V6

Attention: Mr. Paul Slack

Dear Sir:

**Re: Summary of Inclinometer Readings – April 2013**
**Hump Hillside, Marine Drive**
**White Rock, British Columbia**

As requested, Levelton Consultant Ltd. (Levelton) has prepared this memorandum to provide commentary with respect to the recent inclinometer readings taken April 9, 2013 at the above named site. Levelton previously installed inclinometer casings at geotechnical test holes BH11-01 and BH11-03 in April 2011 as part of a geotechnical stability assessment of the Hump Hillside and retaining walls along the subject portion of Marine Drive (*Geotechnical Assessment Report – Revision 1, Retaining Wall & Slope Stability Review,* dated June 20, 2011, Levelton File No. FV11-0658-00). The approximate locations of the inclinometers are indicated on the attached Figure 2, which was originally issued as part of the above-referenced report. Inclinometer readings were previously recorded in April and May 2011.

Review of the data collected to date indicates that the “checksum” data for the initial inclinometer readings collected in April 2011 are generally an order of magnitude higher than the “checksum” data for the subsequent May 2011 and April 2013 readings. Checksum data provides an indication of data error and should remain reasonably constant and of small magnitude. As a result, we recommend that data collected in May 2011 be selected as the base reading, and that April 2011 data be disregarded. Selecting May 2011 as the base reading should have minimal impact on the actual long-term displacements recorded at the inclinometer casings.

The A+ direction of the inclinometer is oriented south, toward the Hump Hillside slope. The total cumulative displacements were calculated at both BH11-01 and BH11-03 and are illustrated on the attached plots.

Generally, the inclinometer readings at BH11-01 and BH11-03 between May 2011 and April 2013 show more significant movements above the depth of 6.5 m below grade, particularly at and in the vicinity of the loose granular fill. Notable inclinometer displacements above 6.5 m below grade are summarized as follows:

**Inclinometer at BH11-01**

- 2.45 mm displacement in the A+ direction between 0.5 and 2.5 mm below grade.
- 0.63 mm displacement in the A- direction between 2.5 and 4.0 m below grade.
- 0.43 mm displacement in the A+ direction between 5.0 and 6.5 mm below grade.

- 3.50 mm displacement in the B- direction between 0.5 and 3.0 m below grade.
- 0.85 mm displacement in the B+ direction between 3.0 and 4.0 m below grade.
- 0.37 mm displacement in the B- direction between 5.0 and 6.5 m below grade.

**Inclinometer at BH11-03**

- 2.77 mm displacement in the A- direction between 0.5 and 1.5 m below grade.
- 2.23 mm displacement in the A+ direction between 1.5 and 4.0 m below grade.
- 0.55 mm displacement in the A- direction between 4.0 and 6.5 m below grade.

- 0.65 mm displacement in the B+ direction between 0.5 and 1.0 m below grade.
- 0.85 mm displacement in the B- direction between 1.0 and 3.5 m below grade.
- 0.93 mm displacement in the B+ direction between 3.5 and 6.0 m below grade.

The inclinometers show cumulative movements in variable and opposite directions in both the A and B orientations. As such, it is difficult to surmise the cause of recorded movements based on the data collected to date. The observed inclinometer deformations may be the result of relatively shallow slope / wall related deformations, or could possibly be the result of on-going loose fill settlement. In particular, loose fill settlement may explain the observed movement in the B direction, parallel to the face of the Hump Hillside slope.

Collection of additional inclinometer readings is recommended to better analyze the cause of recorded inclinometer deformations. We recommend that the next inclinometer reading be taken in April 2014.

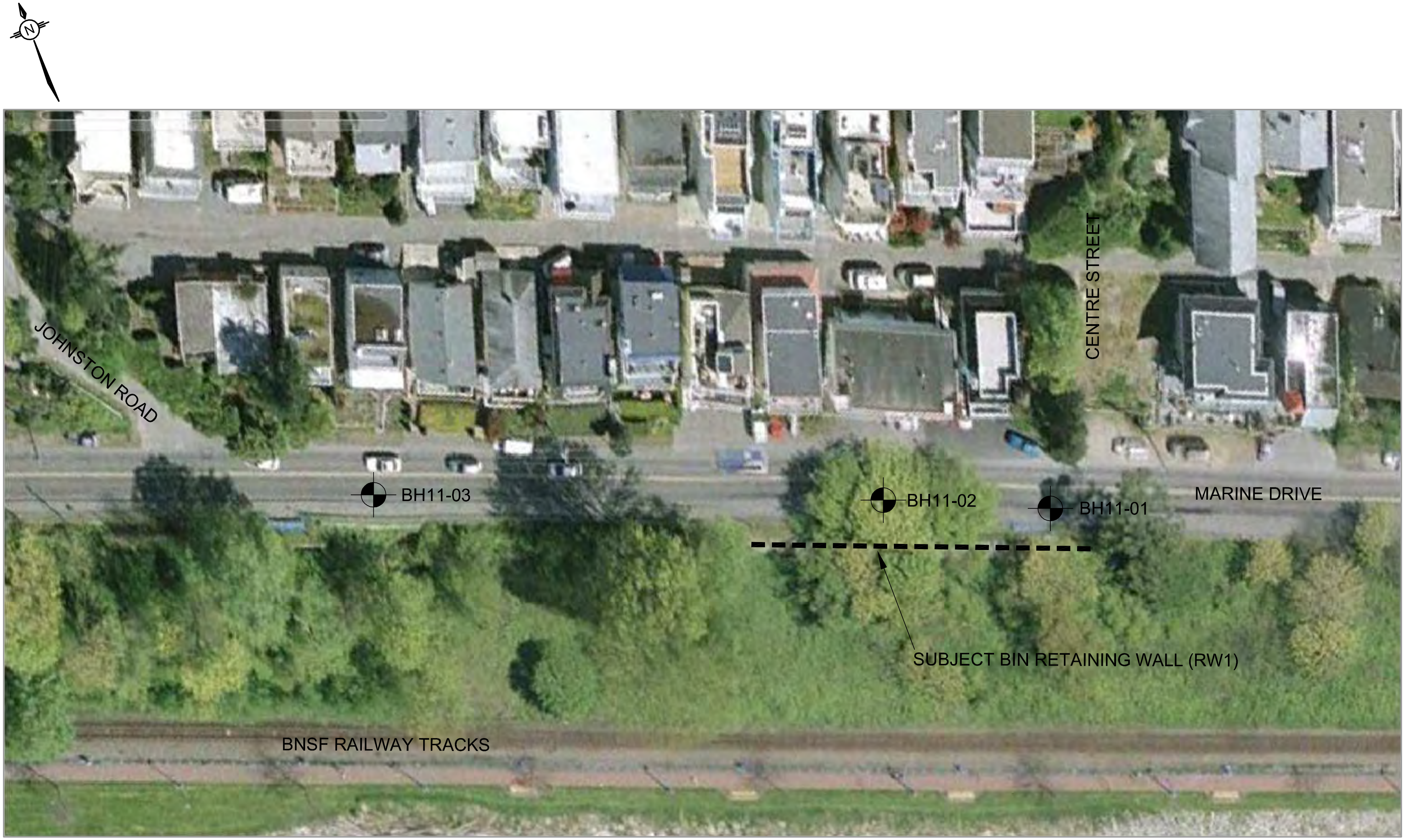
We trust this information meets your immediate requirements. If you have any questions or require further information, please contact the undersigned.

**Levelton Consultants Ltd.**

[Original Signed By: Calum Buchan, P.Eng.]    [Original Signed By: Graeme McAllister, EIT]

Per: Calum Buchan, P.Eng., P.E.
Principal, Senior Geotechnical Engineer

Per: Graeme McAllister, EIT
Junior Geotechnical Engineer



Note: All borehole locations are approximate

LEGEND

⊕ BH11-03 APPROXIMATE BOREHOLE LOCATION

ADAPTED FROM
City of Surrey Mapping Online System

DATE
May 28, 2011

PROJECT NO.
N/A

This drawing is the sole property of Levelton Consultants Ltd. and cannot be used or duplicated in any way without the expressed written consent of Levelton Consultants. The general contractor shall verify all dimensions and report any discrepancies to Levelton Consultants Ltd.

**LEVELTON**
110-34077 Gladys Avenue,
Abbotsford, BC, V2S 2E8
p:604-855-0206
f:604-853-1186
www.levelton.com

TITLE
BOREHOLE LOCATION PLAN

PROJECT
RETAINING WALL & SLOPE STABILITY REVIEW - HUMP HILLSIDE

ADDRESS
MARINE DRIVE, WHITE ROCK, BC

CLIENT
CORPORATION OF THE CITY OF WHITE ROCK

DSN

CHK
CB

DWN
GM

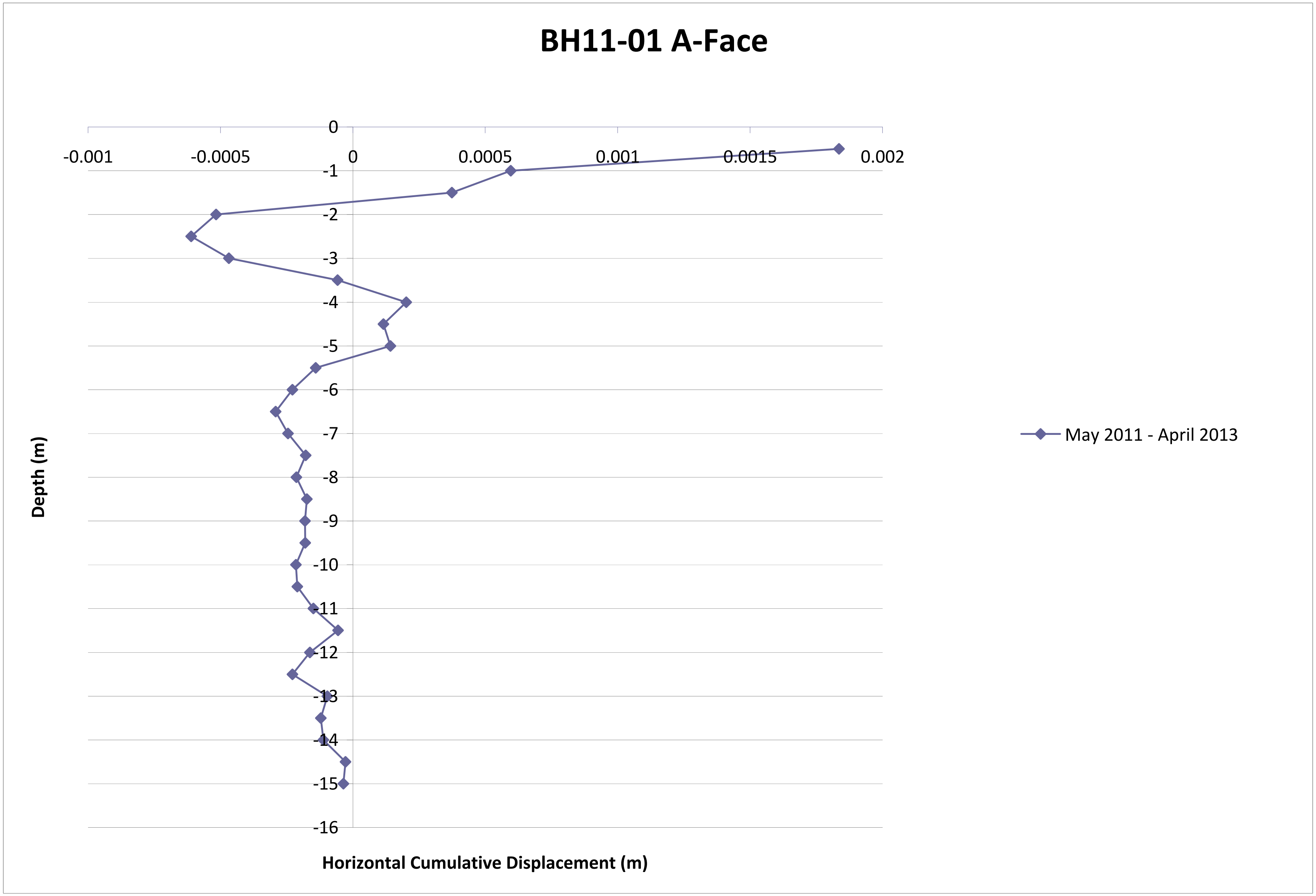
SCALE
NTS

DATE
JUNE 2011

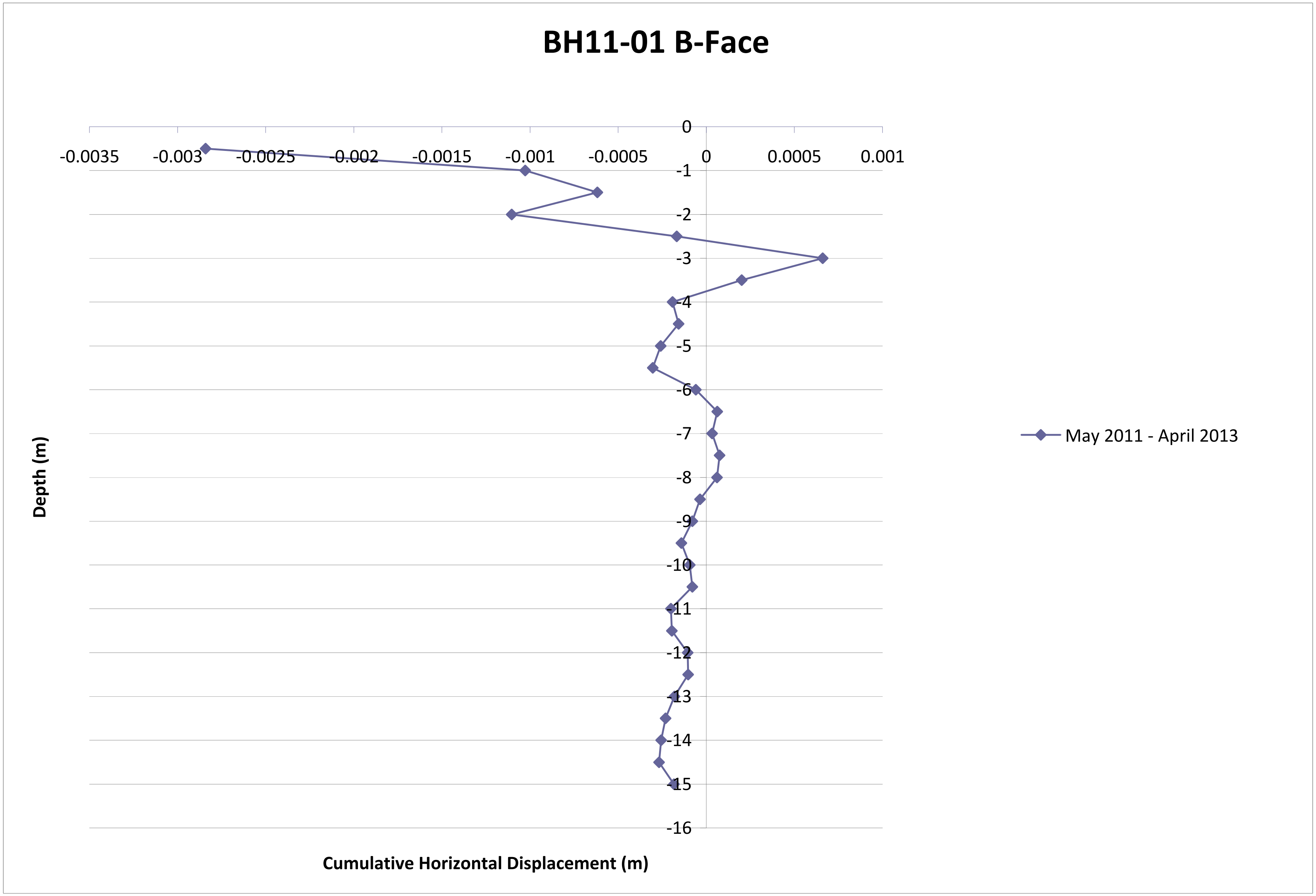
PROJECT NO.
FV11-0658-00

FIGURE NO.
2

Inclinometer Readings
Hump Hillside Marine Drive



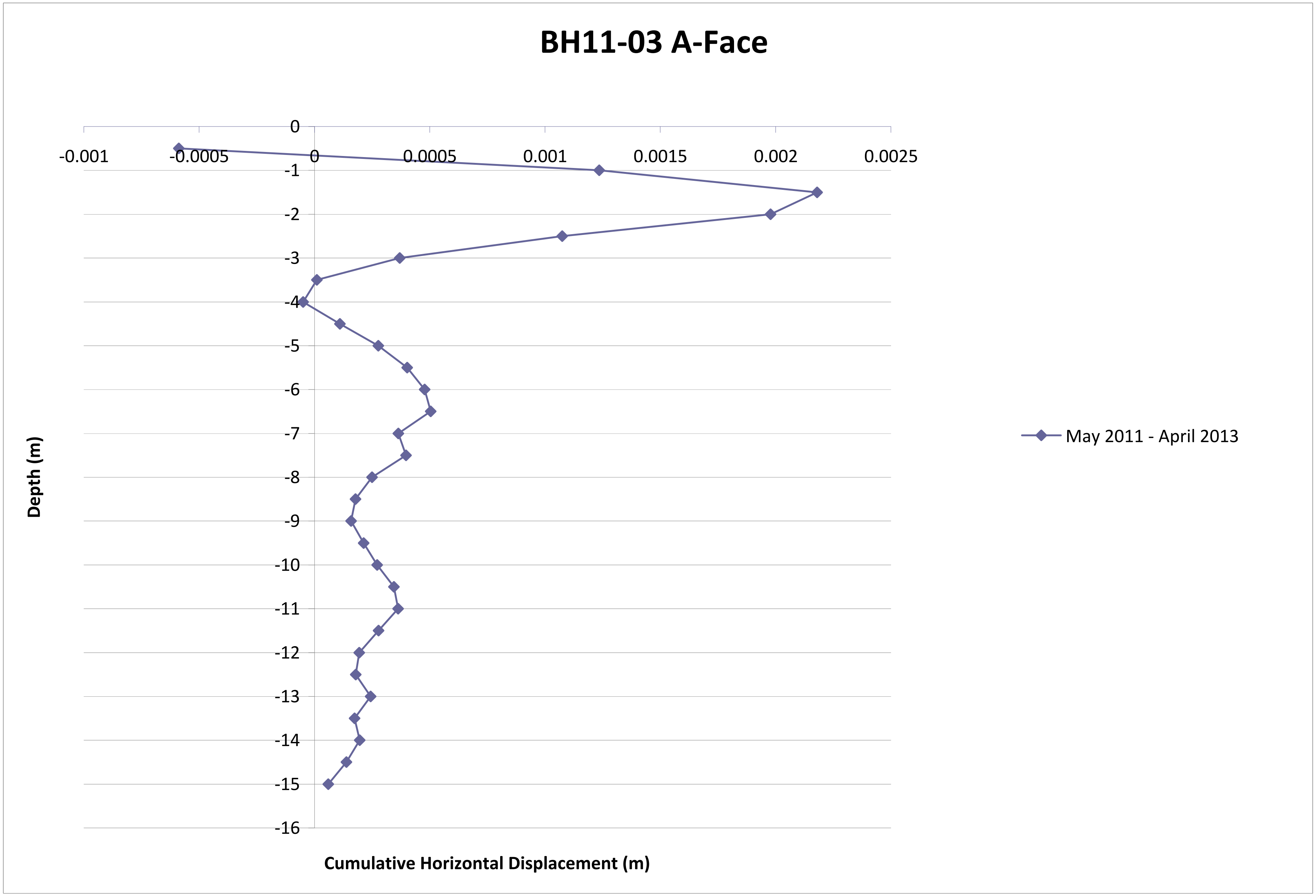
Inclinometer Readings
Hump Hillside Marine Drive

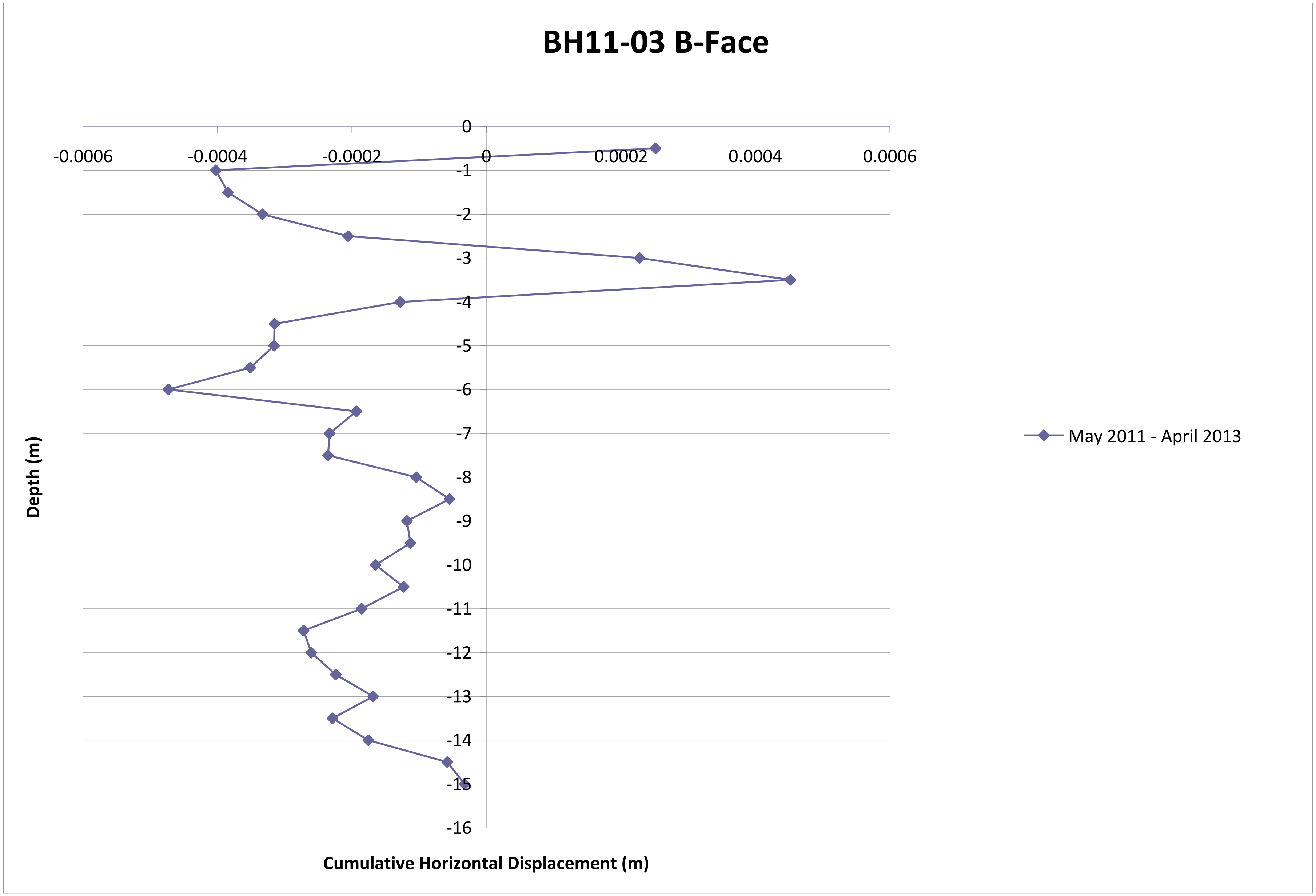


April 25, 2013

Inclinometer Readings
Hump Hillside Marine Drive

Levelton File No. FV11-0658-01




BH11-03 B-Face
0
-0.0006
-0.0004
-0.0002
0
0.0002
0.0004
0.0006
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
-14
-15
-16
Depth (m)
Cumulative Horizontal Displacement (m)
May 2011 - April 2013

# APPENDIX D

## COPY OF INDENTURE NO. 40862 DATED AUGUST 14, 1950

# APPENDIX D

# COPY OF INDENTURE NO. 40862 DATED AUGUST 14, 1950

## TABLE OF CONTENTS

OCT 13 1950
G. N. RY. CO., SEATTLE, WASH.

GN-41207
L-40862

THIS INDENTURE made the 14th. day of

August, A.D. 1950.

**BETWEEN:**

GREAT NORTHERN RAILWAY COMPANY, a Company incorporated under the laws of the State of Minnesota, United States of America, and authorized to do business as a Railway Company in the Dominion of Canada under the provisions of the "Railway Act" with an office at Room 315 Number 602 West Hastings Street, in the City of Vancouver, in the Province of British Columbia,

(hereinafter called the "Railway")

OF THE ONE PART.

THE CORPORATION OF THE DISTRICT OF SURREY, a municipal corporation existing under the laws of the Province of British Columbia,

(hereinafter called the "District")

OF THE OTHER PART.

6

Noted on Sta. Map 9705 ~ M.B. - 10-19-50
" " R/W " 11-1 B.C. ~ M.B. - "

WHEREAS the District as the owner of and responsible for the maintenance of that Highway known as Washington Avenue, in the Townsite of White Rock, has applied to the Railway Company for an easement to construct on the lands of the Railway Company certain timber cribs, pile bulk heads and sloping of lands for the purpose of giving lateral support to Washington Avenue and for the further right and privilege of constructing four concrete culverts across the right of way of the Railway Company at those points hereinafter more particularly described within the Townsite of White Rock for the purpose of giving better drainage.

NOW THEREFORE THIS INDENTURE WITNESSETH that in consideration of the covenants, conditions and agreements hereinafter contained it is agreed by and between the Parties hereto as follows:-

1. The Railway Company grants to the District the right together with its agents, servants and workmen, to enter upon those lands of the Railway Company at or within the vicinity of "C" Street in the Townsite of White Rock, which area is more particularly outlined in red on a blueprint attached hereto and marked as Exhibit "A" and made a part hereof for the purpose of constructing timber cribs, driving pile bulk heads and sloping of lands required for the giving of proper lateral support to Washington Avenue in the immediate vicinity thereof.

2. For the grant aforesaid the District covenants and agrees with the Railway Company to carry out the said work in a workmanlike manner and so as not to interfere in any manner with the operations of the Railway Company.

3. And the District further covenants and agrees to maintain the said works at all times and in such manner so as not to create any possible hazard, detriment or interference to the lands and operations of the Railway Company contiguous thereto.

4. The Railway Company covenants with the District to place at its own expense four culverts across the right of way of the Railway Company within the Townsite of White Rock aforesaid, at points indicated on the said blueprint marked Exhibit "A" and attached hereto, particulars of the said culverts and locations thereof may

be more particularly described as follows:

(1) 30" concrete pipe culvert 42' long at Station 250 + 60.
(1) 24" concrete pipe culvert 66' long at Station 301 + 20.
(1) 24" concrete pipe culvert 60' long at Station 310 + 90.
(1) 24" concrete pipe culvert 60' long at Station 322 + 00.

5. All pipe required for the said works shall be furnished by the Railway Company at the expense of the District and all labour required in the placing of same shall be furnished at the expense of the Railway Company.

6. On completion of the placing of the said culverts the Railway Company shall render to the District an account of the cost of the said pipe and the District covenants and agrees to pay the said account within thirty (30) days after receipt thereof.

THIS INDENTURE shall enure to the benefit of and be binding upon the parties hereto, their respective successors and assigns.

IN WITNESS WHEREOF the said parties hereto have caused their Corporate Seals to be hereunto affixed witnessed by the hands of their proper officers in that behalf, the day and year first above written.

SIGNED, SEALED AND DELIVERED
in the presence of
P. B. HAUBER

GREAT NORTHERN RAILWAY COMPANY,

BY J M BUDD (SEAL)
Vice President.

BY F. L. PAETZOLD
Secretary.

THE CORPORATION OF THE DISTRICT OF SURREY,

BY CHAS. SCHULPE
Reeve.

BY PERCY LIVINGSTON
Clerk.

(SEAL)

## FOR THE SECRETARY (OR OTHER OFFICER) OF A CORPORATION.

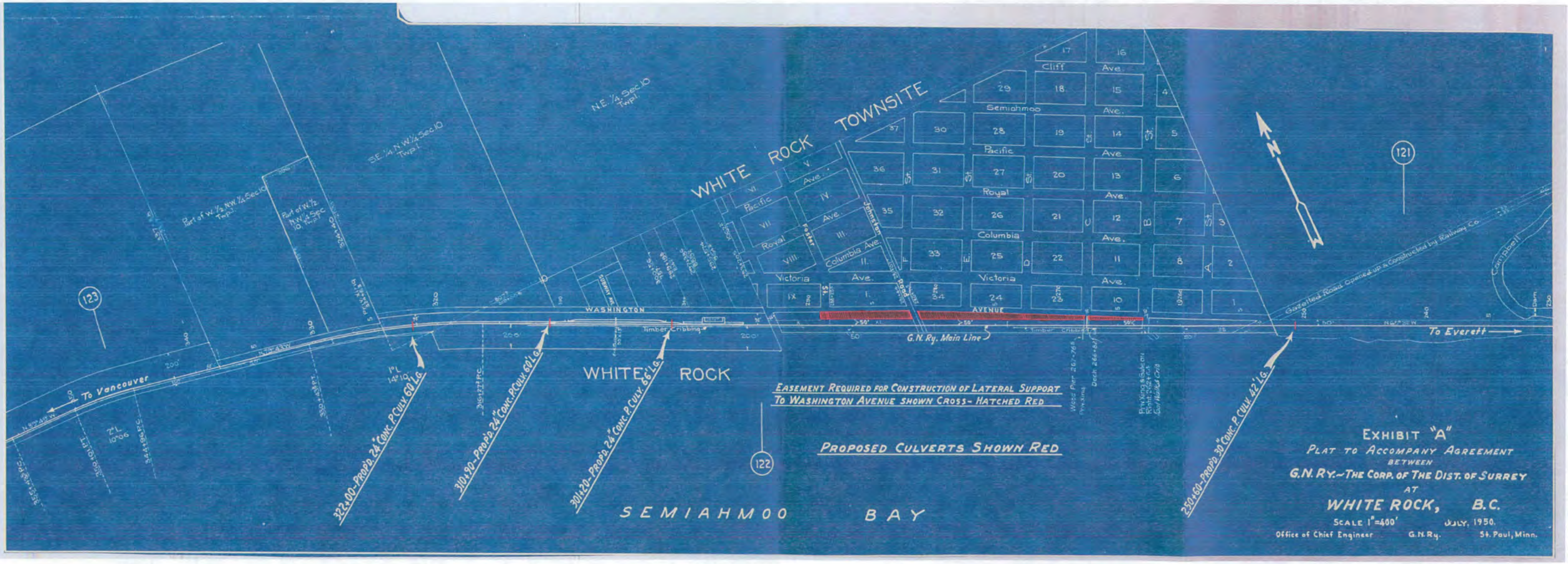
---

I HEREBY CERTIFY that on the 15th day of September, 1950, at the City of St. Paul, in the State of Minnesota, one of the United States of America, F. L. PAETZOLD, who is personally known to me, appeared before me and acknowledged to me that he is the Secretary of GREAT NORTHERN RAILWAY COMPANY and that he is the person who subscribed his name to the annexed instrument as Secretary of the said GREAT NORTHERN RAILWAY COMPANY, and affixed the Seal of the GREAT NORTHERN RAILWAY COMPANY to the said Instrument, that he was first duly authorized to subscribe his name as aforesaid, and affix the said Seal to the said Instrument, and that such corporation is legally entitled to hold and dispose of land in the Province of British Columbia.

IN TESTIMONY WHEREOF I have hereunto set my Hand and Seal of Office at the City of St. Paul in the State of Minnesota, United States of America, this 15th day of September, in the year of Our Lord One Thousand Nine Hundred and Fifty.

A B MORAN
A NOTARY PUBLIC IN AND FOR THE STATE
OF MINNESOTA, U. S. A.

A. B. MORAN,
Notary Public, Ramsey County, Minn.
My Commission Expires July 19, 1955
(SEAL)



Easement Required for Construction of Lateral Support To Washington Avenue Shown Cross-Hatched Red

Proposed Culverts Shown Red

EXHIBIT "A"
PLAT TO ACCOMPANY AGREEMENT
BETWEEN
G.N. RY.—THE CORP. OF THE DIST. OF SURREY
AT
WHITE ROCK, B.C.
SCALE 1"=400' JULY, 1950.
Office of Chief Engineer G.N.Ry. St. Paul, Minn.

# APPENDIX E

# IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

**SHANNON & WILSON, INC.**
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-21664-001

Date: June 12, 2013
To: Mr. Glen Gaz
BNSF Railway Company

# IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

## CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

## THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

## SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

## MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

## A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

## THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

## BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

## READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland