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Mr. David Morel
Assistant Deputy Minister
Environmental Protection Division
Ministry of Environment and Climate Change Strategy

cc. Tessa Graham, AJ Downie

Dear Mr. Morel,

Sperling Hansen Associates is pleased to submit the Updated Final Closure Plan (2019) for the Cobble Hill Holdings Landfill (CHL), located at 460 Stebbings Road, for review by Minister George Heyman.

The updated closure plan introduces a stabilizing soil wedge concept and reduced side slope angles on the PEA cap. These design modifications make the updated closure system design more protective of the environment for the following reason.

- Static slope stability is improved
- Seismic slope stability is improved
- Cover system becomes less vulnerable to root penetration
- Cover system becomes less susceptible to erosion
- Cover system will require less post closure maintenance
- 0.8 Ha of existing smooth LLDPE geomembrane will not be wasted

The revised Updated Final Closure Plan (2019 Closure Plan) has considered and, where necessary, made allowances and justifications for all relevant comments and correspondence provided by the Ministry of Environment (MOE) and the Ministry of Environment and Climate Change Strategy (ENV) including, but not limited to, those posed in the following documents:

- Second Amended SPO MO 01701 - 29 June 2017
- Correspondence from ENV including:
  - Ministry of Environment. Re: Input from Ministry staff to be addressed and responded to as part of Final Closure Plan pursuant to the amended Spill Prevention Order (SPO) issued by Minister Polak on March 15, 2017 - 17 March 2017.

I trust you will find the Updated Final Closure Plan (2019) exceeds the environmental protection measures that were presented in the previous versions of the Closure Plan for the Landfill.

The purpose of this letter is to illustrate to ENV that the significance of the aforementioned correspondence is understood and Sperling Hansen Associates has addressed these matters during the development of the plan. Unless required for value or understanding, the comments and justifications included in this letter are not explicitly stated in the 2019 Closure Plan; instead this letter will serve as the response. A summary of the requirements for the revised 2019 Closure Plan are included and attachment to this letter, below.
Ministry of Environment. Re: Input from Ministry staff to be addressed and responded to as part of Final Closure Plan pursuant to the amended Spill Prevention Order (SPO) issued by Minister Polak on March 15, 2017. (17 March 2017).

The information provided in the aforementioned correspondence has been reviewed and addressed as follows:

<table>
<thead>
<tr>
<th>Request/Comment</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Complete up to date “As-Built” plans and specifications of the existing facility.</td>
<td>Complete up-to-date “As-Built” plans and specifications were submitted to the Ministry by CHH, previously.</td>
</tr>
<tr>
<td>2. Assessment of the adequacy of the existing Facility and, if applicable, recommended revisions.</td>
<td>An assessment of the adequacy of the facility was discussed in Section 3.4 of the Updated Final Closure Plan (2017) and in Section 2.4 of the Updated Final Closure Plan (2019); no substantial issues have been identified at this time. A static and seismic stability assessment was completed previously and has been completed for the new proposed design for the Updated Final Closure Plan (2019) (see Chapter 9 - Geotechnical Considerations - of the attached report). The proposed design meets stability objectives under static and seismic conditions. The proposed design results in final slopes that are 5H:1V which also meets the objectives of the Landfill Criteria (minimum 3H:1V).</td>
</tr>
<tr>
<td>a. include assessment of all works including seepage blanket, landfill base liner, anchor trenches, leachate collection and leak detection works, landfill cells, and leachate storage ponds.</td>
<td></td>
</tr>
<tr>
<td>b. slope stability.</td>
<td></td>
</tr>
<tr>
<td>3. A plan for the management of any contaminated soil stored in the soil management area.</td>
<td>A soil relocation management plan is included in the Updated Final Closure Plan (2019) in Chapter 3 of the report and was completed in the previous version also.</td>
</tr>
<tr>
<td>a. Include tonnage(s) and type(s) of contaminated soil</td>
<td></td>
</tr>
<tr>
<td>4. Proposed landfill final cover, stability assessment and hydrologic modeling.</td>
<td>A stability assessment (see Chapter 9 - Geotech Considerations – of the attached report) and hydrologic model (HELP Modeling included in Chapter 7) has been completed for the new proposed Final Cover. The assessment shows that the existing smooth geomembrane will be stable under SHA’s closure design.</td>
</tr>
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<tr>
<td>5.</td>
<td><strong>A Leachate collection and storage plan including hydrologic modeling.</strong></td>
</tr>
<tr>
<td>6.</td>
<td><strong>The post-closure period.</strong></td>
</tr>
<tr>
<td>7.</td>
<td><strong>An environmental monitoring program, including leachate monitoring, to verify that the escape or spill of Leachate into the environment has not occurred.</strong></td>
</tr>
<tr>
<td>8.</td>
<td><strong>Contingency measures to address any failure of the works or the escape or spill of Leachate into the environment.</strong></td>
</tr>
</tbody>
</table>
Second Amended Spill Prevention Order (SPO) MO 01701 (29 June 2017)

The implications of the SPO have been reviewed and addressed as follows:

<table>
<thead>
<tr>
<th>Request/Comment</th>
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</tr>
</thead>
<tbody>
<tr>
<td>1. The Named Parties must ensure that:</td>
<td>a. Completed in 2017 Minor Construction Works; Final Closure is pending approval.</td>
</tr>
<tr>
<td>a. the landfill is covered completely with weighted and secured impermeable cover,</td>
<td>b. 2017 Minor Construction works included upgrades to the leachate collection and conveyance systems and storage facility; leachate from the PEA is collected, stored on site and trucked offsite to a licensed treatment facility.</td>
</tr>
<tr>
<td>and that sufficient weather protection is provided for the cover in order to ensure</td>
<td></td>
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<tr>
<td>its effectiveness, except as needed for implementation of an approved Updated</td>
<td></td>
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<tr>
<td>Final Closure Plan;</td>
<td></td>
</tr>
<tr>
<td>b. all Leachate generated at the Facility, including from the landfill, soil</td>
<td></td>
</tr>
<tr>
<td>management area and wheel wash area, is collected, stored temporarily pending</td>
<td></td>
</tr>
<tr>
<td>removal from the Facility, and transported from the Facility to an off-site facility</td>
<td></td>
</tr>
<tr>
<td>that is authorized to treat and/or dispose of the Leachate. The collection and</td>
<td></td>
</tr>
<tr>
<td>temporary storage of Leachate at the Facility must be carried out so as to prevent an</td>
<td></td>
</tr>
<tr>
<td>escape or spill of Leachate into the environment;</td>
<td></td>
</tr>
<tr>
<td>c. all works for the collection and temporary storage of Leachate generated at the</td>
<td></td>
</tr>
<tr>
<td>Facility are inspected regularly and maintained in good working order; and</td>
<td></td>
</tr>
<tr>
<td>d. records of the volumes of Leachate collected, stored and transported, including</td>
<td></td>
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<tr>
<td>the name and location of the authorized facility(ies) receiving the Leachate, are</td>
<td></td>
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<tr>
<td>maintained and submitted to the director by the 15th and 30th of each month (or the</td>
<td></td>
</tr>
<tr>
<td>next business day thereafter if the 15th or 30th of the month is not a business day),</td>
<td></td>
</tr>
<tr>
<td>until the Named Parties have complied with section 5 of this order. Submissions must</td>
<td></td>
</tr>
<tr>
<td>be made electronically to the following email inbox:</td>
<td></td>
</tr>
<tr>
<td><a href="mailto:EnvironmentalCompliance@gov.bc.ca">EnvironmentalCompliance@gov.bc.ca</a>.</td>
<td></td>
</tr>
</tbody>
</table>
2. The Ministry acknowledges receipt of a package of “As-Built” plans and specifications for the existing facility, and a report entitled “Cobble Hill Landfill Final Closure Plan Report” prepared by Sperling Hansen Associates Inc., dated May 31, 2017 (the “Final Closure Plan”). By July 21, 2017, the Named Parties must submit an updated version of the Final Closure Plan to the Ministry for review and approval (the “Updated Final Closure Plan”). The Updated Final Closure Plan must be certified by a qualified professional as defined in the Landfill Criteria for Municipal Solid Waste, Second Edition, June 2016 (“Qualified Professional”) and must revise the Final Closure Plan to include the following:

   a. a description of how leachate, and leak detection, collection and storage will be carried out during the transition from the existing leachate, and leak detection, collection and storage works to the new leachate, and leak detection, collection and storage works. If temporary collection and storage works are necessary during the transition, provide a description and details of the temporary works;

   b. procedures for management and monitoring of the leachate storage. The procedures are to ensure there is no risk of overflow from the tanks, including details relating to tank inspections and frequencies of inspections, and consideration of trigger levels, metering, remote monitoring and/or high-level alarms. The discussion and procedures must demonstrate that for worst case (eg. very high volume of inflow) leachate inflows:

      i. the trigger level provides adequate freeboard in the leachate storage tank, and

      ii. the inspection frequencies and monitoring activities are adequate to ensure that the trigger level will not be exceeded.


   b. Complete and previously documented in Updated Final Closure Plan (2017).
c. A full as-built package has previously been submitted to MOE (by CHH) which shows a compacted soil wedge against both the south and west perimeter of the landfill. Additional run-on diversion ditching adjacent to the existing landfill cell along the southern and western perimeters was completed and upgraded as part of the 2017 Minor Construction Works. Additionally, all precipitation falling on the crest of the landfill is collected in lined crest ditches which convey the clean water off the crest and discharge to the north toe.

d. Previously justified and documented in the Updated Closure Plan (2017). Additionally, GHD sampled the Basal Clay Liner in 2017 where the carbon content from the samples ranges from 0.246 to 0.5% which exceed the requirements of the 2016 LCMSW of 0.1%. Also, permeability testing revealed the clay exceeds the requirements of the 1993 and 2016 LCMSW as per GHD.


g. Complete and previously documented in Updated Final Closure Plan (2017). An updated EMP is proposed in Chapter 10 of the new 2019 Closure Plan. Contingency measures are also addressed in Section 7 of the attached report.

c. a description of the works and actions to be carried out to eliminate or minimize non-contact storm and surface water from entering the leak detection system, including related details and cross-sections for the south and west perimeter of the landfill final cover;

d. an evaluation and assessment of the secondary clay liner organic carbon content and stability (structure and permeability) when exposed to leachate, and a description of how any issues identified through the evaluation and assessment will be addressed;

e. revisions to the monitoring program

f. Revisions to Section 9.6 Seepage Blanket Monitoring

g. contingency measures to address any failure of the works, or the escape or spill of Leachate or Contaminated Soil into the environment, including any significant leachate leakage through the landfill base liner(s); and
h. an implementation schedule which provides for commencement of closure activities by August 14, 2017, significant progress and appropriate interim cover/stabilization by October 31, 2017 prior to winter (if final closure is not complete), and completion of all closure activities as early as is practical and no later than September 30, 2018.

Capping was completed in 2016. Minor Closure Works to improve the leachate collection infrastructure were undertaken in 2017. A new schedule has been included in the 2019 Closure Plan, as per recent correspondence with David Morel, November 6th 2018.

3. Following approval of the Updated Final Closure Plan, the Named Parties must carry out all closure activities set out in the approved Updated Final Closure Plan in accordance with any conditions of the approval. A Qualified Professional must be continuously present on-site to supervise all closure activities and must carry out the following: inspect and approve works as they are constructed for conformance with plans and specifications; perform quality assurance and quality control, including for the clay secondary liner and the geomembrane base and cover liners; perform testing, including seam and leak testing; and report to the Ministry in accordance with section 4 below. The Named Parties must carry out the closure activities in accordance with the implementation schedule in the approved Updated Final Closure Plan or such other dates as specified by the Minister.

2017 Minor Construction Works were completed as specified in the August 11th 2017 Letter under oversight by SHA’s Qualified Professional (below).

Pending approval of the 2019 Closure Plan, all works will be completed under QP oversight. Sperling Hansen’s QA/QC staff will be on site for all critical inspections. Local QP may be retained for inspection of bulk soil fill placement. All inspections will be carried out in accordance with the proposed construction schedule, included in the attached report, Section 4 – Landfill Closure Design.

4. Commencing in the month that closure activities commence pursuant to the approved Updated Final Closure Plan, the Named Parties must submit semi-monthly status reports, certified by a Qualified Professional. The reports must include the status of closure activities, inspection results, quality control and testing results, photographs which support/document the quality control and testing results, inspection reports and other supporting documents as needed to fully document all stages and components of the closure activities. Status reports must be submitted by the 15th and 30th of each month (or the next business day thereafter if the 15th or 30th of the month is not a business day) until closure activities have been completed. Submissions must be made electronically.

Ongoing requirement, semi-monthly status reports will continue to be submitted by the QP to the MOE whenever active closure activities are under way.
5. The Named Parties must submit complete detailed final “As-Built” plans and specifications, certified by a Qualified Professional, of any revisions to the Facility including the landfill final cover, resulting from implementing the approved Updated Final Closure Plan in accordance with any conditions of the approval, within 30 days after the works have been constructed.

Detailed as-built record drawings will be submitted by SHA’s QP once the final closure project is completed.

6. Following completion of all closure activities in the approved Updated Final Closure Plan, the Named Parties must submit quarterly implementation reports to the Ministry on or immediately before the last day of March, June, September and December of each year, for the duration specified in the approved Updated Final Closure Plan. Implementation reports must include records of inspections, operations and maintenance of the Facility, records of the volumes of Leachate collected, stored and transported, including the name and location of the authorized facility(ies) receiving the Leachate, and environmental monitoring program records interpreted and certified by a Qualified Professional. Submissions must be made electronically to the following email inbox: EnvironmentalCompliance@gov.bc.ca.

This is currently an ongoing requirement and quarterly implementation reports will be completed based on the approved 2019 Closure Plan and the subsequent closure construction.

The aforementioned Letter from Honourable George Heyman has been reviewed to address comments relevant to the preparation of the Updated Final Closure Plan (2019); responses to these comments are included below:

<table>
<thead>
<tr>
<th>Request/Comment</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Construction activities for the Minor Construction Works must commence by August 28th 2017 and be completed as early as possible and no later than October 31st 2017.</td>
<td>Minor construction works were completed in 2017 and are documented in previously submitted inspection reports. Works completed during this time include: secondary clay liner investigation; installation of new leachate and leak detection storage facility; installation of twin piping for existing Permanent Encapsulation Area (PEA) to new leachate storage facility; installation of high-level alarm system in leachate storage tank; decommissioning of contact water containment pond; stockpile and cover existing soil in Soil Management Area (SMA); installation of Seepage Blanket Monitoring Wells (3); PEA liner repairs and testing; PEA crest ditch ballasting; run-on diversion ditch around the south side of PEA.</td>
</tr>
<tr>
<td>2. A qualified professional as defined in the Landfill Criteria for Municipal Solid Waste, Second Edition, June 2016 (Qualified Professional or QP) must be continuously present on-site to supervise all Minor Construction Works activities and must carry out the following: inspect and approve works as they are constructed for conformance with plans and specifications; perform quality assurance and quality control monitoring and testing; and report to the Ministry in accordance with condition 6 below.</td>
<td>2017 Minor Construction Works were supervised by a Qualified Professional who inspected the works, performed quality assurance and quality control, and provided reporting to the Ministry.</td>
</tr>
</tbody>
</table>
3. By September 15 2017, four test pits must be excavated along the edge of the toe of the northern slope of the landfill cell and the presence and integrity of the basal clay layer must be assessed and documented.

Four test pits were completed during the 2017 Minor Construction Works by CHH with oversight from GHD (Ministry consultant) to assess the presence and integrity of the basal clay liner, and soil samples were collected (by GHD); the results of the investigation are documented in GHD’s December 11th 2017 Clay Basal Liner Evaluation Report. Four (4) test pits were completed with three of the four locations showing a minimum 1.0m thick clay liner was in place. Although GHD found that the clay at one of the test pits was less than 1 metre thick (as per the 1993 Landfill Criteria), GHD indicated that the dual liner system is considered to “exceed the 1993 Landfill Criteria Requirements. Additionally, GHD indicates that existing secondary liner generally meets the requirements of the 2016 LCMSW. It should be noted that the basal clay liner test pits were completed outside (north toe) of the PEA basal liner area and all indications show that the clay basal liner is in fact 1.0m thick under the PEA base.

4. The Environmental Monitoring Program must involve collection of monthly rather than quarterly samples from the surface water, groundwater and seepage blanket as identified in sections 9.4 to 9.6 of the Updated Final Closure Plan. With regard to section 9.3, leachate in the Leak Detection Tank must also be sampled monthly when liquid is present in the tank and analyzed for the same parameters as indicated in the Leachate Tank sampling program.

The Environmental Monitoring Program at the site has been altered to include monthly sampling since the receipt of this letter. As discussed below, based on new recommendations provided by ENV (19 Dec 2018) the 2019 Closure Plan includes recommendations to reverting back to quarterly sampling. This is supported by the results of the monthly sampling to this date, and the estimated speed of groundwater flow.

5. A high-water level alarm system must be installed by October 31 2017 and remain operational in the leachate collection tank to minimize the risk of an unforeseen overflow of leachate.

A high-level alarm has been installed in the leachate collection tank and will continue to remain operational until a QP recommends otherwise. As per the 2019 Closure Plan, environmental monitoring at the site will include checking/inspecting the functionality of the high-level alarm.

6. Additional reporting to the Ministry must be carried out as follows:

The reporting requirements were carried out as part of the QP’s efforts during the 2017 Minor

The aforementioned report by GHD has been reviewed to address comments relevant to the preparation of the Updated Final Closure Plan (2019); responses to these comments are included below:

<table>
<thead>
<tr>
<th>GHD Report (Completion of 2017 Minor Works)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Request/Comment</strong></td>
</tr>
<tr>
<td>1. Inspections of the leachate detection and storage facility should include reporting of the condition of the secondary liner</td>
</tr>
<tr>
<td>Consider whether the installed locations of the seepage monitoring wells are appropriate.</td>
</tr>
<tr>
<td>The QP should consider installing “Confined Space” signage on the door of the leachate detection and storage facility.</td>
</tr>
<tr>
<td>2. The Ministry may want to consider whether obtaining additional technical justification is warranted regarding the adequacy of the basal liner that was installed based on the Waste Discharge Permit and the 1993 Landfill Criteria. Field data, such as the cell 1C basal liner thickness, could be obtained during the next phase of construction.</td>
</tr>
</tbody>
</table>
The Ministry may want to consider whether the existing ballasting of the PEA cover liner being only located within the crest ditch and not the remaining areas of the PEA is adequate to percent potential wind damage to the cover liner.

Based on recent pre-winter inspections and recent photos of the PEA and liner system, SHA is of the opinion that the existing ballast is sufficient until commencement of Closure construction.

<table>
<thead>
<tr>
<th>3.</th>
<th>The methodology could clarify how the new clay layer is to be placed and compacted against the existing clay layer such that a competent seal is established.</th>
<th>No new clay liner extension required or is planned in the new 2019 Closure Plan.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discussion could include technical justification to support leaving the smooth 40 mil LLDPE liner in place on the crest of the PEA in light of it being exposed to the environment from the time of its installation in the fall of 2016 to the implementation of the final closure works.</td>
<td>As above, no new liner extension is planned in the new design. SHA has provided technical justification in the 2019 Closure Plan which supports leaving the existing smooth 40 mil LLDPE liner in place and providing additional stabilizing and environmental protection measures, as outlined in Chapter 4. The black geomembrane liner contains carbon black UV protection which allows the liner to maintain its functionality even when exposed to sunlight for many years. A service life of at least 20 years is expected geomembrane lined ponds.</td>
<td></td>
</tr>
<tr>
<td>Identification of the proposed post-closure land use for the Site could be identified as requirements by the ...2016 Landfill Criteria.</td>
<td>The proposed post-closure land use, as outlined in the Quarry Permit Q-8-094 and specified in the Updated Final Closure Plan is Forestry/Industrial. The proposed post-closure land use is for the landfill area is reclaimed land with sufficient soil cover over the PEA liner system to support native species of grasses and trees.</td>
<td></td>
</tr>
<tr>
<td>A revision to the updated Final Closure Plan could be submitted based on the completion of the 2017 minor works</td>
<td>See attached Updated Final Closure Plan (2019).</td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>As mentioned in Section 5.8 of the Updated Final Closure Plan, GHD supports the recommendation that the facility be also inspected immediately after a high intensity rain event and snowmelt event. The monitoring should include reference to whether any accumulation of liquid within the secondary containment is observed.</td>
<td>SHA has included these recommendations in the 2019 Closure Plan, (Chapter 7, 8 &amp; 10).</td>
</tr>
</tbody>
</table>
The Ministry may want to consider evaluating how precipitation entering the SMA through the open ends of the SMA or through the spaces between the lock blocks will avoid contacting the soil or, if avoidance is not possible, how potentially contaminated storm water accumulated in the SMA will be managed with no available leachate collection system since the containment pond was decommissioned during the 2017 minor works.

<table>
<thead>
<tr>
<th>R</th>
<th>QP could include confirmation of continued operation of the leachate storage tank high-level alarm in the reporting to the Ministry.</th>
<th>Complete and ongoing during bi-monthly reporting by QP to ENV.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>QP could include reference in the reporting to the Ministry as to whether any accumulation of liquid within the secondary containment is and/or has been observed.</td>
<td>SHA has included this recommendation in the updated EMP (Section 10) of the 2019 Closure Plan</td>
</tr>
<tr>
<td></td>
<td>Based on the information reviewed by GHD to date the correlation between rainfall events and leachate production due to cover liner leaks has not been included with the reporting as was identified in the Updated Final Closure Plan.</td>
<td>The observed leachate collection volumes appear to be trending down; SHA continues to recommend that leachate generation be correlated to precipitation data during annual reporting as outlined in 2019 Closure Plan. The reporting included in the bi-weekly inspection reports is a requirement of the SPO. This section was not required as per the SPO although SHA expects it will occur as recommended in the Updated Closure Plan.</td>
</tr>
<tr>
<td></td>
<td>The Ministry may want to consider obtaining an evaluation of whether the installed locations of the seepage monitoring wells are appropriate in consideration of their intended purpose.</td>
<td>SHA has completed a conceptual hydrogeologic model for the site within the 2019 Closure Plan (Chapter 6). The model suggests that the seepage monitoring wells are in the appropriate location.</td>
</tr>
<tr>
<td></td>
<td>The Ministry may want to closely monitor the parameters detected in the seepage layer monitoring wells to determine if the concentrations are persistent, increasing or decreasing</td>
<td>Ongoing monitoring of these wells and a review of trends is recommended in the EMP as outlined in Chapter 10 of the 2019 Closure Plan.</td>
</tr>
</tbody>
</table>

The aforementioned report by GHD has been reviewed to address comments relevant to the preparation of the Updated Final Closure Plan (2019); responses to these comments are included below:

### GHD Report (Clay Basal Liner Evaluation)

<table>
<thead>
<tr>
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</tr>
</thead>
<tbody>
<tr>
<td>The dual basal liner... appears to meet the Permit requirements. Although potions of the clay layer were observed to be less than 1 m thick... the presence of both the geomembrane and clay liners would generally have been considered to exceed the 1993 Landfill Criteria. The Clay liner also generally meets the 2016 Landfill Criteria requirements. The Ministry may want to consider whether obtaining additional technical justification is warranted regarding the basal liner that was installed based on the Permit and 1993 Landfill Criteria. Field data, such as the cell 1C basal liner thickness could be obtained during the next phase of construction.”</td>
<td>GHD confirms that the existing basal liner exceeds the 1993 Landfill Criteria and generally meets the 2016 LCMSW requirements. As such, SHA does not recommend that the Cell 1C base liner be exposed during construction of the final cover system.</td>
</tr>
<tr>
<td>Prior to approving the Final Closure Plan, GHD recommends that the ministry ensure sufficient technical justification is provided for the engineering design and specifications and consider any newly acquired information in order to determine whether the basal liner system will be protective of the environment. The technical justification such as the dual-liner system, leachate generation quantity and natural attenuation capacity of the Site can be incorporated into the overall evaluation of the basal liners effectiveness at protecting the environment.</td>
<td>As outlined above, GHD acknowledges that the basal liner meets the requirements of the 2016 LCMSW. It is SHA’s opinion that – at this point – ensuring the final cover layer provides an effective cap to reduce infiltration and leachate generation, as well as re-confirming the monitoring program through completion of an updated conceptual model, is the most appropriate strategy for closure and for environmental protection. This is further outlined in the 2019 Closure Plan.</td>
</tr>
</tbody>
</table>
The aforementioned letter from David Morel has been reviewed to address comments relevant to the preparation of the Updated Final Closure Plan (2019); responses to these comments are included below:

**ENV Letter 20 Sept 2018**

<table>
<thead>
<tr>
<th>Request/Comment</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Construction activities must be commenced on or before June 1st 2019 and be completed no later than September 30th 2019</td>
<td>2019 Closure Plan proposes to have a proposed construction commencement date of March 1st, 2019 and a completion date of October 31st 2019 to ensure that all environmental control systems including the soil stabilizing wedge (which is part of the proposed design) and revegetation/reclamation works. This is a request to extend the previously suggested timeline of June 1st to September 30th 2019. SHA is of the opinion that extending the construction work does not pose any additional risk to the environment, based on previously completed construction works in October 2017. It is SHA’s expectation that closure construction will wrap-up as soon as reasonably possible, however, to ensure that CHH complies with the conditions of approval, our request is to have the deadline for completion extended until October 31st 2019.</td>
</tr>
<tr>
<td>2. Before November 15th 2018, a comprehensive pre-winter inspection of the landfill must be conducted.</td>
<td>Inspection report was completed and provided copies to ENV of the pre-winter inspection of the landfill.</td>
</tr>
<tr>
<td>3. Before the commencement of construction activities, a report certified by a QP acceptable to the Ministry must be submitted to the Ministry for review and approval. The report must include: <strong>a.</strong> Any revisions to the closure activities to reflect the completed minor construction works and current site conditions.</td>
<td><strong>a.</strong> As requested, attached to this Letter is an Updated Final Closure Plan (2019) that incorporates the completed minor construction works and current site conditions.</td>
</tr>
</tbody>
</table>
b. Clear and detailed descriptions regarding how the comments, conclusions and recommendations in the GHD reports will be addressed during the construction activities including:
   i. For areas of the existing secondary clay base liner that will be exposed during construction activities, a description of the inspections, sampling, testing, assessment and work that will be conducted to demonstrate substantial conformance with the specifications for a secondary compacted clay liner as specified in the LCMSW.
   ii. For the new secondary clay base liner extension, a description of how it is to be placed and compacted against the existing secondary clay base liner such that a competent seal with no leaks is established and a description of the inspections
   iii. description of how GHD comments regarding clay mineralogy that under certain circumstances could affect clay permeability will be addressed.
   iv. If it is still proposed to use the existing smooth geomembrane cover liner on the landfill top plateau, then technical justification that supports this use must be provided including consideration of damage/degradation from exposure to the environment including sunlight since Fall 2016, liner specifications, inspections, and any necessary sampling, testing and repairs.

b. GHD comments are addressed in this report (see above).

   As outlined previously, the new proposed design does not require exposure of the existing clay base liner; as such, no additional inspections, sampling, testing or assessment of the liner will be completed. Further, no extension of the secondary clay base liner will be required.

   iii)SHA’s primary strategy to address environmental concerns is to ensure the landfill is closed with a properly designed final cover system; this will reduce the amount of infiltration into the waste and reduce leachate generation. The suitability and chemical stability of the Victoria clays is discussed in Section 2.4 of the 2019 Closure Plan. In addition, the EMP that is in place is designed to monitor the effectiveness of the basal liner. As outlined Chapter 7, the predicted leachate generation is very low. If the monitoring results indicate the liner is failing, contingency measures have been included, such as: a Leachate cut-off trench at the North toe of the facility which can be implemented under QP recommendation.

   iv) SHA’s proposed design plans to use the existing smooth geomembrane cover. SHA has completed stability analysis on the updated design and the results indicate the new design provides equivalent (and better) slope stability than the design proposed in the July 2017 version of the closure plan. The new design meets the appropriate Factor of Safety for the site.
SHA acknowledges the existing liner has been exposed to sunlight since Fall 2016. CHH provided literature to the MOE which indicates liner exposure, similar to that of the PEA, is not a long-term issue. Service life of exposed HDPE and LLDPE liners is generally considered to be 20 plus years.

SHA, MOE and GHD inspected the existing 40 mil LLDPE cap liner on the PEA during the ‘2017 Minor Construction Works’ where all holes, deficiencies and welds were inspected, mapped, deficiencies remediated and tested. This work is documented in the as-built / summary report for the ‘2017 Minor Construction Works’. Islander Engineering also completes monthly / bi-monthly inspections of the site including the PEA liner system and to date no further deficiencies have been reported.

During the recent meeting with the Community representatives, it was pointed out that bonding of one patch may have failed. Based on routine inspections at the site, which include PEA liner inspections, as well as the ‘Pre-Winter Inspection Report’ provided to MOE, we are not aware of any further liner deficiencies. However, prior to placement of any closure materials over top of the existing PEA liner, a detailed final QA/QC inspection will be completed by liner contractor and QP. The final inspection will be conducted by Dr. Tony Sperling, P.Eng., President of SHA or Scott Garthwaite, A.Sc.T., SHA’s most senior QA/QC professional.
<table>
<thead>
<tr>
<th>c. A complete, consolidated and updated post closure monitoring program that includes:</th>
<th>c. A complete, consolidated and updated post closure monitoring program has been included in Chapter 10 of the 2019 Updated Final Closure Plan, which includes: Post closure inspections, operations, maintenance, monitoring, environmental monitoring and reporting. A tabular summary of the post closure monitoring plan has been included in the report.</th>
</tr>
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<tr>
<td>4. Before the commencement of construction activities or March 31 2019, whichever is earlier, a detailed construction activity work plan and implementation schedule certified by a QP acceptable to the Ministry must be submitted to the Ministry for review and approval</td>
<td>A construction activity workplan and implementation has been completed and is included in the 2019 Closure Plan, Chapter 4.</td>
</tr>
<tr>
<td>5. The semi-monthly status reports submitted pursuant to section 4 of the SPO must also include:</td>
<td>Construction Status / Progress Reporting to ENV is proposed at monthly intervals completed by QAQC Inspector (QP) and will include construction deviation tracking, construction activities, implementation schedule and will outline planned activities for the next reporting cycle.</td>
</tr>
<tr>
<td>a. Identification of any deviations from the construction activities work plan and implementation schedule referenced in condition 3) of this approval,</td>
<td>2019 Environmental Monitoring to be adjusted from Monthly to Quarterly as per correspondence with Luc Lachance (December 19th 2018)</td>
</tr>
<tr>
<td>b. The planned activities and associated timing for the next reporting cycle, and</td>
<td></td>
</tr>
<tr>
<td>c. The environmental monitoring program laboratory reports and tabulated results</td>
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Reference Document: Environmental Monitoring Program Recommendations from Staff:
The aforementioned Reference Document has been reviewed to address comments relevant to the preparation of the Updated Final Closure Plan (2019); responses to these comments are included below:

<table>
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<th>Request/Comment</th>
<th>Response</th>
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<tbody>
<tr>
<td>1. A conceptual groundwater model should be developed for the landfill. The model should describe the groundwater system and guide its monitoring program.</td>
<td>SHA’s resident Hydrogeologist has completed a conceptual model that has been incorporated into the 2019 Closure Plan (Chapter 6 &amp; 10). This model will guide the recommendations for the monitoring plan.</td>
</tr>
<tr>
<td>2. Two shallow bedrock monitoring wells should be installed near and downslope from the landfill. The wells should be installed within the uppermost portion of the water-bearing bedrock. The wells should be advanced using a diamond core drilling method or similar methods that can ensure that the wells are screened within fracture zones.</td>
<td>As per ENV recommendations, SHA has considered the installation of two shallow bedrock monitoring wells near and downslope from the landfill within the uppermost portion of the water-bearing bedrock. In addition, SHA has used the information gathered from the updated conceptual model to guide the EMP at the site. Based on the conceptual model, SHA proposes that that additional groundwater wells be developed based on data and sampling performance in the coming years. If signs of leachate migration are confirmed by a QP, the additional wells should be installed (see Chapter 6 &amp; Chapter 10 of the 2019 Closure Plan).</td>
</tr>
<tr>
<td>3. Water collection from the newly installed background well MW-6 is very hard and reports detectable hydrocarbon concentrations and therefore its use for background comparison may be limited. Instead, background groundwater quality benchmarks should be developed from the historical data set.</td>
<td>SHA has worked to develop background groundwater quality benchmarks from the historical data set; the parameters are included in the 2019 Closure Plan (Chapter 10).</td>
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<td>4.</td>
<td>The groundwater quality monitoring network should be modified to only include the seepage wells, new shallow bedrock wells and surface water monitoring site (SW-1) at the ephemeral creek. SW-1 should be monitored at the same frequency as groundwater wells. If notable changes in groundwater quality are observed after the landfill closure, the other existing monitoring wells should be re-introduced to the monitoring network.</td>
</tr>
<tr>
<td>5.</td>
<td>Groundwater levels should be routinely collected at all existing and new groundwater monitoring sites, including at the seepage blanket monitoring wells. Those well should also be surveyed for elevation to allow for a geodetic water level monitoring.</td>
</tr>
<tr>
<td>6.</td>
<td>Since the interflow is controlled by precipitation which can dilute the water chemistry, the seepage blanket monitoring wells should be sampled either prior to large precipitation events or sometime after such events.</td>
</tr>
<tr>
<td>7.</td>
<td>Piper diagrams should be used to characterize the leachate and all water samples. The diagrams and their interpretations should be routinely reported</td>
</tr>
<tr>
<td>8.</td>
<td>The Environmental Monitoring Plan should identify actions to be implemented (e.g. confirmatory sampling, monitoring programs (s) adjustments, additional studies, migrating actions, etc.) if results from the groundwater and surface water monitoring sampling show elevated values of parameters or if analysis of results (e.g. Piper Diagrams) reveal increasing trends or other shifts in water quality.</td>
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Morel, David. Letter to Named Parties. 6 November 2018.

As per the November 6th 2018 Letter from David Morel, a Revised Updated Final Closure Plan must:

<table>
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<tr>
<td>1. Be submitted to ENV by December 12 2018</td>
<td>See attached Updated Final Closure Plan (2019); the date of submission was extended by ENV.</td>
</tr>
<tr>
<td>2. Be certified by a &quot;Qualified Professional&quot; as defined in the Landfill Criteria for Municipal Solid Waste, Second Edition, June 2016 (2016 LCMSW).</td>
<td>The Updated Final Closure Plan was developed by Sperling Hansen Associates and is certified by a Professional Engineer who meets the QP definitions as per the 2016 LCMSW.</td>
</tr>
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</table>
| 3. Use the 2016 LCMSW for guidance ...As indicated in the 2016 LCMSW, site-specific requests for exceptions from specific requirements of the 2016 LCMSW can be made, provided that the requests include sufficient technical justification to demonstrate that the proposed site-specific alternatives provide an equivalent or better level of environmental protection. | Any site-specific exceptions to the 2016 LCMSW that are proposed are outlined in the Updated Final Closure Plan (2019). These exceptions include:
- Use of smooth geomembrane: as discussed throughout this letter and in the Updated Final Closure Plan (2019)
  SHA has completed the appropriate stability and hydrologic modeling for the existing liner system and the system has been found to meet environmental objectives.                                                                                                                                                                                                                     |
| 4. Address technical matters including:                                           | These matters have been addressed above.                                                                                                                                                                                                                                                                                                                                                                                                                           |
| a. Static and seismic stability analyses, conclusions and recommendations for the landfill and for the final cover layers including geomembrane. | The Updated Final Closure Plan includes a stability analysis of the proposed design.                                                                                                                                                                                                                                                                                                                                                                           |
| b. If it is proposed to use the existing smooth geomembrane cover liner on the landfill plateau or side slopes, technical justification that supports this use including consideration of damage/degradation from exposure to the environment including sunlight since Fall 2016, liner specifications, inspections, and any necessary sampling testing and repairs. | The existing geomembrane is not believed to pose any additional risk to the environment, based on recent inspections and literature regarding exposure to sunlight.                                                                                                                                                                                                                                                                                        |

In addition to the previous ENV documents, ENV provided the following additional comments to be addressed in the revised updated final closure plan; the responses are as follows:

<table>
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<tr>
<td>A cover letter and a complete consolidated revised updated final closure plan will be submitted. The cover letter will, in table format, list the items that the revised updated final closure plan must address including the items from previous ENV documents (including the amended Spill Prevention Orders, ENV correspondence, and GHD reports) and this correspondence, and summarize how and where they are addressed in the revised updated final closure plan.</td>
<td>Complete (this document).</td>
</tr>
<tr>
<td>Technical justification to demonstrate the proposed site-specific alternatives (e.g. use of existing smooth geomembrane, increased final cover thickness at 6H:1V slope, drain tubes in final cover that drain to drain pipe/trench at the landfill toe) provide an equivalent or better level of environmental protection must be provided including, but not limited to, for the following matters:</td>
<td>Technical justification for site specific exemptions has been completed and is included in the 2019 Closure Plan.</td>
</tr>
<tr>
<td>a. Use of the existing smooth geomembrane including on existing slopes up to 2.5H:1V.</td>
<td>a. See Section 4.61 and 4.71.</td>
</tr>
<tr>
<td>b. Increased final cover thickness</td>
<td>b. See Section 4.7.5, as well as 4.6.3, 4.6.4, 4.7.6 and 4.8.</td>
</tr>
<tr>
<td>c. Drainage of water from the landfill top plateau, the drain tubes on the slopes, and the drain pipe at the landfill toe that will be buried under the increased thickness of final cover.</td>
<td>c. SHA has revised design to use Drain Tubes only on landfill crest and to use a 200 mm gravel drainage layer on the side slopes as this solution is considered more reliable in the very long term. See discussion in Sections 4.6.2 and 4.7.3.</td>
</tr>
<tr>
<td>d. Identification of the parts of the final cover (e.g. layers, slopes and thicknesses) that are required in the revised updated final closure plan to achieve environmental protection (e.g. to provide static and seismic stability, and to prevent erosion), and the part(s) of the final cover that are not required</td>
<td>d. See Chapter 4 including supporting drawings for design details and Chapter 9 for discussion on slope stability..</td>
</tr>
</tbody>
</table>
to achieve environmental protection but are being proposed for other purposes including reclamation or aesthetic purposes, and the reasons for identifying the parts of the final cover as required, and not required.

e. The intended post-closure end use(s) and the final cover soil Contaminated Sites Regulation soil quality.

f. Demonstration that the proposed final cover will not pose unacceptable risks to the environment due to the proposed soil quality and quantity through the generation of leachate and potentially impacted surface water runoff.

e. Post closure end use if Forest Lands / Industrial per zoning and soil quality is proposed to be as per CSR Industrial Limits.

f. The generator will be required to submit TCLP testing to establish that run-off from the cover soils will not exceed established CSR thresholds for surface water and groundwater.

**Environmental Monitoring Plan** – The Environmental Monitoring Plan (EMP) must address prior ENV correspondence (e.g. ENV letter dated Sept 20, 2018 and attachments (item 3(c) of the letter requires a complete, consolidated and updated post closure monitoring program that also addresses the attached “Environmental Monitoring Program Recommendations from Staff’)).

Some of the key improvements recommended by staff were discussed at the meeting and are summarized below:

a. Development of a conceptual groundwater flow model, which guides the improvements to the monitoring network and program.

b. Additional shallow monitoring wells should be installed directly downslope from the landfill.

c. A modified groundwater monitoring program should only include the seepage blanket wells, the new shallow wells and the surface water monitoring site SW-1. None of the existing monitoring wells are recommended for the long-term groundwater quality monitoring, including the background well MW-6. That well reports very hard water and elevated concentrations of the indicator parameters (chloride, sulphate, sodium, hardness).

d. Conceptual groundwater model and flow map completed and included in Chapter 6 of the Updated Final Closure Plan (2019)

e. As per the Conceptual Model and the EMP (Chapter 6 and 10 of the 2019 Closure Plan), SHA is recommending a phased approach to monitoring at the site. Recommendations for shallow monitoring wells have been included based on monitoring, sampling and analysis. If seepage is detected in the leak detection system, or if the surface water and seepage blanket wells begin to show signs of a potential leak to the environment, based on QP review, additional wells will be installed, in consultation with MOE re: preferred location and installation details.

c. The EMP incorporates these comments. SHA has developed background groundwater quality benchmarks from historic data sets to be used as background for the proposed EMP, as outlined in Chapter 2 and 10 of the 2019 Closure Plan.
d. All new monitoring wells and the existing seepage blanket wells should be surveyed to allow for a geodetic water level monitoring after the landfill closure.

e. The groundwater monitoring frequency should be relaxed from monthly to quarterly. Further reduction should be considered at a later date if it is supported by the monitoring results.

f. The detectable hydrocarbon concentrations in groundwater require further investigation and explanation.

4. The EMP must also address the leachate monitoring location (LE-1) and confirm that LE-1 is for untreated leachate (as per 2016 Landfill Criteria (section 9.1) & Guidelines for Environmental Monitoring at Municipal Solid Waste Landfills (section 5.3)).

LE-1 is untreated leachate (raw) collected from the PEA and stored in the leachate storage facility. Post LE-1 sampling, leachate is then treated in the storage facility prior to removal offsite as completed during the 2017 Minor Construction Works.

The purpose of this cover letter and the attached Updated Final Closure Plan 2019 is to address all comments from the MOE, GHD, and other third-parties, either through responses, justifications, or changes to the Closure Plan itself. We trust that these documents meet your expectations.

Sincerely,

Dr. Tony Sperling, P.Eng
President & Chief Engineer
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Executive Summary

Sperling Hansen Associates (SHA) was retained by Cobble Hill Holdings Ltd. (CHH) to complete an Updated Final Closure Plan for submission to the BC Ministry of Environment and Climate Change Strategy (MOE).

The Cobble Hill Landfill Updated Final Closure Plan reflects the current state and location of the landfill and outlines a detailed closure design for the PEA. The intent of this Updated Final Closure Plan is to address and satisfy concerns of the Ministry, while designing a Closure system that meets design and performance objectives of the second edition BC Landfill Criteria for Municipal Solid Waste (LCMSW).

In addition to the required Chapters, this closure plan includes:

- Assessment of the adequacy of the existing facility,
- Landfill Stability assessment and Hydrologic modelling that demonstrates the final cover and ditching will be stable and adequate for worst case conditions including 1 in 200-year storm event, plus snowmelt and multi-day precipitation events,
- Leachate collection and storage works assessment – ability to prevent an escape or spill of leachate into the environment,
- Leachate collection and storage plan – including hydrologic modelling that demonstrates the infrastructure is adequate for the worst-case conditions including 1 in 200-year storm events plus snowmelt plus precipitation,
- Leachate removal and transport plan,
- A plan for the relocation of contaminated soil stored in the Soil Management Area,
- An updated Conceptual Hydrogeologic Model for the Site
- Post closure inspection, operation and maintenance and environmental monitoring program including a Trigger Response Plan,
- Proposed Closure Construction Schedule.

Site Description

Chapter 2 provides a description of the site including the physical setting, geology, and climate. A review of the current water quality monitoring results is included in this chapter; as well as a description of background quality benchmarks that were created using the BC Ministry of Water, Land and Air Protection Protocol 9 for Contaminated Sites. Based on the monitoring results, the
environmental protection measures in place appear to be functional and working as intended. An assessment of the existing landfill is provided at the end of this chapter, which reviews the adequacy of the seepage layer, depth to water table, clay liner, acid rock drainage potential, 40 mil LLDPE primary liner, leachate collection, soil filter, and grading of the PEA.

**Soil Relocation and Encapsulation Plan**

Chapter 3 outlines the Soil Relocation and Encapsulation plan for soil located in the Soil Management Area (SMA) at the Cobble Hill Landfill. Soil will be relocated from the SMA to the Permanent Encapsulation Area (PEA) in a way that minimizes the risk of spilling contaminated soil into the surrounding environment. The existing smooth 40 mil LLDPE geomembrane liner will be carefully trimmed along the perimeter of the crest by qualified liner installation contractors; the removed liner will be set-aside for re-use during crest capping works. The soil from the SMA will be relocated and discharged into the PEA for permanent encapsulation. It is estimated that the soil relocation and encapsulation process will be completed within a one-week time period. Following relocation, the PEA crest liner will be re-established and the appropriate QAQC testing will be completed. All work relating to soil relocation will be completed during dry weather.

**Final Closure Design**

The Final Closure Design Chapter outlines a detailed guide for construction and closure of the landfill, as well as a preliminary construction schedule. The side slopes of the PEA will remain at approximately 2.5H:1V, and a 50mm sand friction layer will be placed overtop of side slopes to ensure long term slope stability. Common fill will be used to reduce slopes above the liner to approximately 5H:1V at completion. The common fill will enhance the stability and environmental protection for the landfill (further discussed in Chapter 9). In addition, the closure design includes an eastern soil wedge that will remediate the steep slopes in the south-east corner of the quarry to provide final contours that are similar to produce aesthetically pleasing topography and increase safety measures. A preliminary construction schedule is outlined at the end of this chapter including proposed starting and completion dates.

**Erosion Control**

The cover system will be progressively hydroseeded during construction as final design grades are met and will be completed as per the preliminary construction schedule to minimize erosion and soil loss. Straw wattle and erosion control matting are recommended to minimize mounding in the topsoil and subsoil layers, to maintain stability at the site. Ditches will be lined with rip rap or an erosion control blanket, depending on the expected velocity.
Conceptual Hydrogeologic Model

An updated conceptual model was prepared for the site to help guide the environmental monitoring program at the landfill and to better understand subsurface conditions at the site. The bedrock beneath the landfill is very hard from surface to approximately 75 metres below ground. Only a few fractures have been observed in the bedrock during drilling, rendering the deep rockmass nearly impervious. As such, the fractures do not have the capacity to convey significant quantities of groundwater flow.

There is a groundwater flow divide located east of the Landfill and separates groundwater flowing into Shawnigan Creek and its tributaries and groundwater flowing to the Landfill.

Groundwater within the region of the encapsulation area will travel in a north easterly direction, in addition to groundwater north of the encapsulation area. Groundwater east of the encapsulation area will travel east to Shawnigan Creek and its tributaries. Groundwater hydraulic gradients were calculated at the Landfill: for a change in distance over a metre approximately a 0.04 m change in hydraulic head was observed, yielding a gradient of 4%.

Leachate Management

A Leachate Management Strategy has been developed for the Cobble Hill Landfill, including minimizing infiltration during closure works; run-on diversion; and leachate collection and removal. HELP modelling was completed for leachate generation, using a safety factor of 1.5. This safety factor is based on review of historical extreme weather from the Lake Cowichan weather station, and represents worst case scenario 200 year wet weather, including snowmelt and multiday precipitation. The post closure leachate generation estimate for the site is 12 cubic meters per year. Leachate will drain via gravity into the existing leachate storage facility which contains two 10,000-gallon leachate tanks and one 10,000-gallon leak detection tank. As a contingency measure, to ensure no leachate is spilled into the environment (through leaking or cracked tanks), all three tanks reside in a covered facility that includes a secondary geomembrane liner and gravel cushion layer. Leachate is currently transported to an offsite treatment/disposal facility; this strategy will continue to be used in the future as opportunities for zero discharge water treatment such as evaporation are explored.

As leachate collection trench has been conceptually designed as a contingency measure if it is determined that the basal liner is failing.
Surface Water Management

The goal of the surface water management plan for the Cobble Hill Landfill is to keep clean water clean and minimize leachate production by diverting onsite clean surface water away from the PEA. All Surface Water Management works were designed to be adequate for Worst-Case conditions, including 200 year storm including snowmelt and multi-day precipitation. The design includes details for Crest ditches, Toe Ditches, Downchutes and erosion control measures.

Geotechnical Considerations and Slope Stability

Stability Analysis for the re-graded landfill was completed using SLIDE. The proposed design was found to be stable for all static and seismic loading conditions. The results indicate the buttressing wedge design is stable for all static and seismic loading conditions and provides more protection than SHA’s previous 2017 design. In both cases, the deep-seated FOS for the Circular Search were found to be more than 1.5 and 1.0 for static and seismic conditions respectively, suggesting that the landfill will be globally stable post-closure.

Environmental Monitoring & Post Closure

The Post-Closure Monitoring plan was designed to meet the regulatory requirements for the landfill site and was developed based on the conceptual model.

Leachate is collected in an in-ground HDPE tank(s) and removed from the site using vac-trucks. The tank is contained in a secondary geomembrane liner and covered by a roof structure; the geomembrane will serve as leak detection for the tank(s). A high-level alarm and trigger levels are in place to ensure that sufficient additional storage capacity is available prior to off-site removal. During leachate removal the leak detection tank will be monitored to ensure the landfill liner is not leaking. Leachate and Leak Detection quantity will be monitored and recorded during each removal event.

Surface water monitoring includes one sampling location downstream of the settling pond discharge quarterly at the same time as groundwater sampling.

Seepage Blanket monitoring will be conducted at three wells down-gradient of the PEA. Because the seepage blanket wells are influenced by precipitation, monitoring should be conducted either prior to or sometime after large precipitation events.
A detailed review of the conceptual hydrogeologic model has revealed that the primary pathways for leachate migration are through the seepage blanket and through surface water flow. Because there is an upward hydraulic gradient and competent bedrock with very few fractures, the chance of detecting any impact in the deep groundwater wells is extremely remote. Therefore, the proposed monitoring plan does not include continued sampling from the existing monitoring wells at this time. These wells can be reactivated if leachate leakage is suspected to be occurring. Additionally, SHA has proposed two new shallow groundwater wells, to be installed under a phased approach, should the monitoring program indicate the landfill is leaking.

As the landfill is not a Municipal Solid Waste Landfill and the gas generation rate for the types of waste being landfilled is extremely slow if at all, Landfill Gas is not deemed to be of great concern at the site. A review of leachate testing has indicated that VOC’s are generally non-detect in the leachate. Therefore; release of volatile organic compounds through the barrier layer is no longer a concern. This should be verified with a one-time monitoring run of VOC’s emissions from the landfill.

An annual geotechnical inspection will be conducted at the landfill site to identify problems arising from slumping, cracking, or erosion.

An annual Post Closure Inspection will also be undertaken by a Qualified Professional, to assess:
- Final Cover
- Ditching
- Topsoil/Vegetation
- Erosion
- Leachate Breakouts
- Leachate Collection, Conveyance and Storage
- Environmental Monitoring Infrastructure

Any observed deficiencies will be recorded and remediated appropriately.

Each year a Qualified Professional will collect the required monitoring data and compile an Annual Report outlining the closure performance of the landfill.

Chapter 10 also outlines Contingency Actions and Trigger Response Plan. The plan includes a five-step approach to any incident at the site, that will include 1. Routine Monitoring, 2. Confirmatory Monitoring, 3. Additional Investigation, 4. Mitigation Strategy, 5. Follow-up monitoring. Contingency measures for failures of the leachate management infrastructure and surface water management works are also identified.
1. INTRODUCTION

1.1 Background

The Cobble Hill Holdings Landfill (CHL) is located at 460 Stebbings Road, in South Shawnigan Lake Area (Electoral Area B) within the Cowichan Valley Regional District as outlined in Figure 1-1.

Previously, the site was operated as a rock quarry under the jurisdiction of the BC Ministry of Mines permit number Q-8-094. In conjunction with mining operations, the site was also permitted for Authorization to Discharge Waste under permit number 105809, allowing for deposition of up to 100,000 tonnes of contaminated soil per year to be treated and permanently encapsulated onsite as part of the mine reclamation plan. The waste discharge permit was issued on **August 21st, 2013**. These permits are provided in Appendix A.

SHA have been involved with Cobble Hill Holdings (CHH) dating back to the early part of 2016 when SHA’s President and Chief Engineer, Dr. Tony Sperling P.Eng., was asked to complete third-party review of the permanent cell closure that was underway at the landfill. In November, 2016 SHA agreed to continue working with CHH to complete an updated Closure Plan for submission to the BC Ministry of Environment (MOE) before the end of 2016. SHA was formally retained by CHH to prepare the Final Closure Plan.

Sperling Hansen Associates (SHA) was retained by CHH to complete a Revised Updated Final Closure Plan for submission to the MOE.

On **December 20th, 2016** SHA completed the first Cobble Hill Landfill Closure Plan. The SHA Closure Plan assumed that rock quarry and landfill operations at CHL would continue decades into the future. It was assumed that the landfill would be operated until 2046. The report was subsequently forwarded to MOE for review.

On **January 27th, 2016** MOE issued a Spill Prevention Order MO1701. The order required CHH to ensure that all leachate generated from the facility is collected, temporarily stored, and then trucked off-site to an authorized leachate treatment facility. Furthermore, the Order required that the leachate works be regularly monitored and inspected, and that data be submitted to MOE on the 1st and 15th day of each month.

In the Order MOE staff requested that SHA update the Closure Plan to address the following:

- Climate Station: WSP and SHA used different climate stations as representation of the landfill site.
- Cost Estimates: SHA did not fully address all outlined Post Closure Costs.
- Post Closure Period: Post Closure Period was not in accordance with the Landfill Criteria for Municipal Solid Waste, 2nd Edition (LCMSW).
- Surface/Stormwater Conveyance: the LCMSW does not allow for water diversion beneath the landfill footprint.
- Geomembrane: SHA report cited a 40-mil geomembrane basal liner was to be used rather than 60 mil.

As well, a comprehensive review report was presented, dated January 19th, 2017, prepared by MOE’s A. Leuschen, Senior Environmental Protection Officer. The review report provided additional details on noted deficiencies in SHA’s first Closure Plan and in WSP’s leachate management strategy.

On January 27th, 2017 Waste Discharge Permit 105809 was concurrently suspended.

On February 18th, 2017 SHA completed an update to the Cobble Hill Landfill Closure Plan that addressed the above review comments on the initial draft of the Closure plan.

On February 23rd, 2017 MOE cancelled the Waste Discharge Permit.

On March 15th, 2017 Minister of Environment, Mary Polak issued an Amended Spill Prevention Order MO1701. The updated order required that:

a) The landfill be covered completely with an impervious cover,

b) All leachate from the facility, including the landfill, soil management areas and wheel wash be collected, stored and periodically removed from the site,

c) That all leachate treatment systems be regularly inspected, and

d) That records be maintained of the volumes of leachate collected and disposed off site.

Furthermore, the order required that the landfill be permanently closed or that all contaminated soil be removed from the facility.

The order required that “As-Built” information be provided to the MOE on or before April 17th, 2017 and that a Final Closure Plan for CHH Landfill be submitted to the Ministry on or before May 31st, 2017. The closure plan was to be developed following the guidance in the LCMSW dated June, 2016.

On March 17th, 2017 MOE provided additional “Input from Ministry Staff to be addressed as part of Final Closure Plan pursuant to the amended Spill Prevention Order (SPO) issued by Minister Polak on March 15th, 2017”. The input requested more up to date details on the “as-built” information previously provided as well as an update of the previously submitted SHA Closure Plan to address the following:
• An assessment of the adequacy of the seepage blanket
• A review of landfill slope stability under static and seismic conditions
• A plan for managing soil remaining in the soil management area (SMA)
• Stability monitoring of the proposed landfill cover
• Hydrologic modelling of the storm water control systems

On April 18th, 2017, CHH submitted As-Built Plans and specifications, as requested by the above order.

A month later, On May 18th, 2017 A.J. Downie, Director Authorizations South, MOE provided “Interim Additional Input from Ministry Staff to be addressed and responded to as part of the Final Closure Plan pursuant to the amended Spill Prevention Order (SPO) dated March 15th, 2017. The letter requested that the Final Closure Plan respond to the requirements identified in the following:

- March 17th, 2017 MOE Input
- April 13th, 2017 MOE Letter regarding soil relocation from the soil management area
- Interim Additional Input in MOE letter dated May 18th, 2017

On May 26th, 2017 MOE forwarded a detailed technical review of SHA’s Closure Plan from Hemmera, a broad-based environmental consultancy with expertise in environmental assessments and contaminated sites. Hemmera was identified as the MOE’s Qualified Professional Contractor on this file in Mr. Downie’s May 18th, Interim Additional Input Letter.

Sperling Hansen Associates reviewed correspondence from Ministry staff regarding the Cobble Hill Landfill Closure and including the amended Spill Prevention Order (SPO) MO1701 and in the three-week window that was available, updated the plan to address the issues that had been flagged. The second Final Closure Plan was submitted on May 31, 2017.

On June 29th 2017, a second amended Spill Prevention Order:MO1701 was issued to the named parties. The SPO outlined revision requirements for submission of an Updated Final Closure Plan.

On July 21, 2017 SHA submitted The Updated Final Closure Plan. The plan was designed to meet the requirements outlined in the aforementioned Ministry Correspondence, the second amended SPO, and the LCMSW. In addition to the required chapters, the closure plan included:

- Assessment of the adequacy of the existing facility,
- Landfill Stability assessment and Hydrologic modelling that demonstrates the final cover and ditching will be stable and adequate for worst case conditions including 1 in 200-year storm event, plus snowmelt and multi-day precipitation events,
- Leachate collection and storage works assessment – ability to prevent an escape or spill of leachate into the environment,
- Leachate collection and storage plan– including hydrologic modelling that demonstrates the infrastructure is adequate for the worst-case conditions including 1 in 200-year storm events plus snowmelt plus precipitation,
- Leachate removal and transport plan,
- A plan for the management of contaminated soil stored in the Soil Management Area,
- Post closure inspection, operation and maintenance and environmental monitoring program,
- Implementation schedule for commencement and completion of closure activities.

On Aug 11, 2017, the MOE provided a letter by Minister George Heyman conditionally approving minor construction works (termed “2017 Minor Construction Works”) outlined in the Updated Final Closure Plan. Further site investigations were also requested in this letter and listed below. Items included:

- Start and end dates for the completion 2017 Minor Construction Works and investigations,
- The requirement for a QEP to be on site for Ministry reporting, QA/QC, conformation with plans and specifications,
- The implementation of a basal clay liner investigation,
- Monthly water monitoring rather than quarterly for groundwater, seepage flow, surface water, leak detection (when liquid is present), analysis of water quality for the same parameters as leachate tank sampling program,
- Installation of a high-water level alarm system in the leachate tank,
- And the inclusion of tabulated water quality results in semi-monthly reports.

Works and investigations were completed under QEP supervision and documented in this Revised Updated Final Closure Plan as well as in Sperling Hansen Associates progress reports from Sep 30, 2017 and Oct 15, 2017.

On Sep 20, 2018, the MOE sent a draft conditional approval letter for closure to the named parties including requirements for a new revised Updated Final Closure Plan.

On Nov 6, 2018, the MOE confirmed consideration for a revision of the Updated Final Closure Plan based on further conditions.
On Dec 19, 2018 the MOE sent an email following a meeting with the named parties and SHA requesting further requirements to be included in the Revised Updated Final Closure Plan.

A summary of the Ministry requirements for the Revised Updated Final Closure Plan which are documented in letters from Sep 20, 2018, Nov 6, 2018, the Dec 19, 2018 email, and first amended SPO date March 15, 2017, the Ministry input letter from the March 17, 2017, and the second amended SPO June 29, 2017 are displayed in the tabulated cover letter.

This Revised Updated Final Closure Plan encompasses requirements from these documents in a consolidated detailed design for closure and reflects the current state and location of the landfill.

### 1.2 Purpose and Scope

SHA have been involved with CHH dating back to the early part of 2016 when SHA’s President and Chief Engineer, Dr. Tony Sperling P.Eng., was asked to complete third-party review of the permanent cell closure that was underway at the landfill. In November, 2016 SHA agreed to continue working with CHH to complete an updated Closure Plan for submission to MOE before the end of 2016. Following the suspension of this permit, CHH engaged SHA once again to develop a Final Closure Plan and detailed design for closure of the permanent encapsulation area (PEA) currently on site. In September 2017, SHA was the onsite QEP for 2017 minor construction works reporting and investigations. In late 2017, CHH engaged SHA to revise the Updated Final Closure Plan, as per recent correspondence (described in the section above). This report presents the culmination of 15 months of effort by SHA and CHH, working with review and comment from MOE and local concerned citizen groups to develop a quality closure strategy that will be protective of the environment.

#### 1.2.1 Project Startup and Field Work

SHA staff have completed numerous site visits since mid 2016. Based on the site visits and review of numerous documents including Technical Assessment Reports (AEE), Environmental Procedures Manual / Operations, Maintenance and Surveillance Manual (CHH), Permits and others, SHA has a good understanding of historic site operations, environmental controls and the current state of the site.

During the development of this Updated Final Closure Plan, SHA has attended meetings with the Ministry of Environment and Climate Change Strategy (MOE), CHH, and other stakeholders to ensure that the final plan for the landfill will meet the expectations of all parties.

#### 1.2.2 Updated Final Closure Plan (2019)

The Updated Final Closure Plan (2019) includes the following sub-tasks: Site Description, Soil Relocation and Encapsulation, Final Closure Design, Filling plan for Soil Stabilizing Wedge and

SHA has considered and addressed previous comments, questions, and stipulations imposed by MOE and other bodies, as outlined in the comprehensive cover letter included in the submission of the report.
Site Location
2. SITE DESCRIPTION

2.1 Physical Setting

The CHL is located at 460 Stebbings Road, in the South Shawnigan Lake Area, approximately 5 km south of Shawnigan Lake, BC.

The existing site property boundary encloses a 21 Ha area of land, of which approximately 1 Ha is occupied by the Permanent Encapsulation area (PEA).

A Soil Management Area (SMA) is located at the site, and currently houses approximately 3,360 tonnes of soil. Prior to final closure, this soil will be transported and enclosed in the PEA.

The site’s contact water pond was decommissioned during the 2017 Minor Construction Works. The existing area is now thriving with natural vegetation. Details of this decommissioning are documented in Chapter 3 of this report.

An additional 2 Ha was previously occupied by front end operations such as a weigh scale and scale house as well as machinery parking areas and wheel wash facilities. Some of these facilities have been decommissioned such as the wheel wash based on the cancellation of the waste discharge permit. The existing topography and infrastructure onsite are presented in Figure 2-1. Figure 2-2 shows the existing PEA located in the southwest portion of the site.

The Shawnigan Creek corridor crosses Lot 23 on the western 1/3 of the site, draining south to north. The land to the south and west, Lot 22, is Crown Land. CHH owns the parcel of land directly to the north, Lot 21 and the site is bound on the east by Stebbings Road.

2.2 Site History

In August 2006, a notice of intention to commence work on a quarry, including a plan of the proposed work system and a program for the protection and reclamation of the surface of the land and watercourses affected by the work was filed. A quarry permit (No. Q-8-094) was issued in October 2006, amended in April 2009 and again in July 2015.

In 2013, a permit (No. 105809) authorizing discharge of treated soil from a contaminated soil treatment facility at 460 Stebbings Road, was granted to CHH. The site was originally operated by South Island Aggregates (a subsidiary of CHH). From June 2015 to March 31st, 2017, the quarry had been operated by...
South Island Resource Management Ltd. for which a termination agreement was accepted by CHH. The site has not operated as a landfill since suspension of the permit in January 2017.

CHH completed full encapsulation of one cell in the western portion of the site during the fall of 2016. The encapsulation included a basal lining system and a cap liner composed of 40mil LLDPE Geomembrane. The double basal liner exceeded the requirements of the 1993 Landfill Criteria. Drainage layers were provided above and below contaminated soil layers. A second LLDPE 40 mil geomembrane was deployed in the fall of 2016 to cap the PEA in order to minimize the generation of leachate (contact water). At the time the geomembrane was not protected by overlying cushion, drainage and erosion control layers as construction took place during inclement weather.

The quarry development plan called for this cell to eventually be relocated to the ultimate pit bottom once rock was quarried from that area. As is commonly done in such circumstances, internal slopes were established at a maximum safe grade (between 2.5H:1V and 3H:1V) and a smooth 40 mil LLDPE membrane was deployed over the site as a capping material. At the time of construction, it was anticipated that the geomembrane cap would eventually be decommissioned and that additional waste would be landfilled onto the side slopes and crest of the completed Phase 1A, B and C cells.

The unexpected cancellation of the CHH Waste Discharge Permit now requires the owner to permanently and securely cap the soil on site or dispose off-site; however, off-site disposal is not a fiscally realistic option. Tipping fees alone would cost at least $5 million, transportation would add another $2 to $3 million and then the quarry would have to be reclaimed to meet requirements of the Mines Act for a total cost estimated in excess of $10 million. This level of expenditure is simply beyond the financial capacity of CHH. Furthermore, relocating nearly 100,000 tonnes of soil to an alternate facility would introduce another level of environmental impacts including traffic risks associated with 8,000 round trip movements, GHG emissions, dust release, etc. It is SHA’s professional opinion that managing the waste in place, with the proper environmental controls in place, is the environmentally preferred option.

Fortunately, the non-hazardous waste soil can be securely contained on site by the complete encapsulation and closure method proposed in this Updated Final Closure Plan - 2019 and documented in following Chapters. The method proposed to cap the landfill far exceeds the minimum design requirements of the 2nd Edition LCMSW and is more protective than the 3H:1V veneer previously proposed by SHA in the July 2017 Updated Closure Plan.

Currently, approximately 3,360 tonnes of soil are stored in the site’s Soil Management Area (SMA). MOE has approved relocating the soil currently in the SMA to the PEA prior to completing the final closure.
2.2.1 Site Location

CHL is located approximately 5km south of Shawnigan Lake in the Cowichan Valley Regional District (CVRD).

The land surrounding the site is used primarily for forestry as well as mineral extraction. Five rock quarries exist within 1 kilometer of the site. Land use zoning for parcels surrounding the site includes Primary Forestry, Secondary Forestry and Community Land Stewardship.

According to the CVRD’s zoning by-law, land zoned for Primary Forestry (F-1) may be used for the following purposes:

- management and harvesting of primary forest products excluding sawmills and manufacturing
- extraction, crushing and milling of aggregate material
- single family residential
- agriculture, horticulture, silviculture
- home based business
- bed and breakfasts
- secondary suite on parcels less than 10.0 hectares
- secondary suite or secondary dwelling on parcels greater than 10.0 hectares

Section 2 of the site’s Quarry Permit Q-8-094 amended October 28, 2015 states that the surface of the land and watercourses shall be reclaimed to the following land use: Forestry/Industrial

Figure 1-1 presented in the previous chapter is a Google Earth image of the Cobble Hill Quarry and Landfill and surrounding land use. Neighbouring quarries and logging blocks are clearly visible.

The Primary Forestry lands adjacent to the site are owned by the CVRD and include the Stebbings Road Community Forest. Two residences exist on land located approximately 320 meters southeast of the site.

Land zoned for Secondary Forestry includes similar land uses as Primary Forestry; however, aggregate mining is not permitted. There are five one-hectare parcels zoned for Secondary Forestry northeast of the site.

Land zoned for Community Land Stewardship exists 200 meters south of the site extending to 2.5 kilometers south. This zone includes a variety of land uses ranging from ecological conservation, single family dwellings, bed and breakfasts, home based business, equestrian centers, daycare, convenience store, schools and more.
An Industrial Park also exists in the vicinity of the site, located off Shawnigan Lake Road.

2.2.2 Legal
The legal description of the land parcel for the Shawnigan Lake Quarry is as follows:

Parcel I.D.: 026-226-502
Legal: Lot 23, Plan VIP78459, Blocks 156,201,323, Malahat Land District.

2.2.3 Permit
On August 21, 2013, the CHL, received Permit 105809 to discharge waste. The permit authorized discharge of contaminated soil. The permit allowed for soil treatment onsite; however, CVRD municipal bylaws, according to BC Supreme Court, indicate that soil treatment is not within the respective land use; therefore, soil treatment was not conducted onsite.

A covered soil management area exists for soils contaminated with hydrocarbons, styrene, methyl tertiary butyl ether, volatile petroleum hydrocarbons, light and heavy extractable petroleum hydrocarbons, polycyclic aromatic hydrocarbons, chlorinated hydrocarbons, phenols, chloride, sodium, and glycols.

The permit allowed discharge of contaminated soils and ash into a lined CHL. Contaminant levels less than Hazardous Waste, as regulated by the Hazardous Waste Regulation, were permitted for landfilling. Contaminants permitted include those listed above as well as soils impacted by metals, dioxins, and furans.

Effluent from the site was permitted at a maximum discharge rate of 274 cubic meters per day, and had to meet British Columbia Approved Water Quality Guidelines for Drinking Water and Freshwater Aquatic Life. The above-mentioned rate was under review by the MOE to allow for additional discharge during high storm events.

The Waste Discharge permit is no longer in place.

2.2.4 Geology and Hydrogeology
The site is underlain by Wark Gneiss bedrock – a formation composed of massive and gneissic metadiorite, metagabbro, and amphibolites (Active Earth Engineering Ltd., 2012). The site also includes a hard, granitic bedrock exposure.

There are no faults located under the site; two faults occur three kilometers to the southwest and six kilometers to the northwest.
The local flow regime is that of fractured flow through bedrock, with most groundwater flow occurring in the near surface where fractures are more prevalent. There is no bedrock aquifer mapped directly underneath the site, however there are two bedrock aquifers located near the site: Spectacle Lake/Malahat Bedrock Aquifer and Shawnigan Lake/Cobble Hill Bedrock Aquifer. The site is serviced by an on-site groundwater well.

Due to previous quarrying at the site, there are no native soils in the immediate area. However, soils surrounding the site originate from glacial till.

2.2.5 Climate

The temperature and precipitation data for 1981 to 2010 were sourced from the Environment Canada website using the Lake Cowichan weather station. The nearest weather station is located at Shawnigan Lake, B.C. about 15 km to the North of the site, however the Lake Cowichan weather station has been chosen as a reference site as it provides a conservative representation of annual rainfall and thus provides a “worst case scenario” baseline for our analysis.

The climate data is summarized in Table 2-1 below. The average annual precipitation is approximately 2047.5 mm with approximately 1975.6 mm falling as rain and 72.0 cm falling as snow. The average annual temperature is approximately 9.8°C with an average peak of 18.1°C occurring in August and the minimum average temperature of 2.5°C occurring in December. The maximum average snowfall of 19.8 cm occurs in January. Table 2-1 presents the average monthly precipitation and temperature for the Lake Cowichan weather station.

Table 2-1 Climate Data for Cowichan Lake station, 1981 to 2010 (Environment Canada, 2017)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
<th>Jul</th>
<th>Aug</th>
<th>Sep</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall (mm)</td>
<td>327.3</td>
<td>206.2</td>
<td>209.2</td>
<td>135.9</td>
<td>85.2</td>
<td>57.2</td>
<td>34.7</td>
<td>40.2</td>
<td>51.7</td>
<td>212.5</td>
<td>334.8</td>
<td>280.9</td>
<td>1975.6</td>
</tr>
<tr>
<td>Snowfall (cm)</td>
<td>19.8</td>
<td>19.8</td>
<td>7.0</td>
<td>1.5</td>
<td>0.3</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.8</td>
<td>8.4</td>
<td>14.4</td>
<td>72.0</td>
</tr>
<tr>
<td>Precipitation (mm)</td>
<td>347.0</td>
<td>226.0</td>
<td>216.2</td>
<td>137.4</td>
<td>85.4</td>
<td>57.2</td>
<td>34.7</td>
<td>40.2</td>
<td>51.7</td>
<td>213.3</td>
<td>343.2</td>
<td>295.3</td>
<td>2047.5</td>
</tr>
<tr>
<td>Daily Average (°C)</td>
<td>3.2</td>
<td>4.0</td>
<td>6.2</td>
<td>8.6</td>
<td>12.1</td>
<td>15.0</td>
<td>17.8</td>
<td>18.1</td>
<td>15.2</td>
<td>9.8</td>
<td>5.4</td>
<td>2.5</td>
<td>9.8</td>
</tr>
</tbody>
</table>

In order to complete hydrologic modelling for a 1 in 200-year storm event, including snowmelt and multi-day precipitation, the IDF curve for the Lake Cowichan Environment Canada Climate station, was utilized, as presented in Figure 2-3. The maximum 1 in 100-year 24-hour storm intensity for the area is estimated at 6.5 mm per hour, or 156 mm/day. Although the 200-year intensity is not listed,
based on the logarithmic progression of the graph, an intensity of about 7.5 mm per hour appears appropriate, resulting in a design rainfall intensity of 180 mm/day. Accounting for 6 mm of snow melt/day increases the design intensity to 186 mm/day. This is similar to the 200-year intensity outlined by WSP in the Addendum: Review of Contact and Non-Contact Water Management Systems as 7 mm/hr or 168.7 mm/day. A further analysis of appropriate hydrologic modelling for the worst-case scenario Surface Water Management is found in Chapter 8.

For design of leachate management systems, a longer-term wet season scenario is more appropriate. For this, monthly climate normals were reviewed. The average precipitation values represent the average total precipitation from the Canadian Climate Normals. The “Extreme Year” precipitation was found by compiling annual climate data and discovering the year with the maximum annual precipitation. The overall percent difference between the max year and the average climate is 132% or a factor of 1.3. The “Max Month” data represents maximum total precipitation for each month recorded over a period of 1961 – 2006. In an extreme situation, where each maximum month is summed for an extreme maximum annual precipitation, the overall percent difference to average is 228% or 2.3. SHA is of the opinion that this results in a severe over-estimate, as it is unlikely to get back to back worst case precipitation month, after month, after month. Therefore, SHA has used a conservative multiplier of 1.5 for hydrologic modelling of infrastructure at the site.

Table 2-2 Total Precipitation (mm) Tabulated from data retrieved from Environment Canada.

<table>
<thead>
<tr>
<th>Month</th>
<th>Average (Climate Normals)</th>
<th>Extreme Year Max Annual (1997)</th>
<th>% of Average</th>
<th>Max Month (1961-2006)</th>
<th>% of Average</th>
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<tbody>
<tr>
<td>Jan</td>
<td>347</td>
<td>374</td>
<td>108%</td>
<td>661</td>
<td>191%</td>
</tr>
<tr>
<td>Feb</td>
<td>226</td>
<td>130</td>
<td>57%</td>
<td>582</td>
<td>258%</td>
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<tr>
<td>Mar</td>
<td>216</td>
<td>491</td>
<td>227%</td>
<td>491</td>
<td>227%</td>
</tr>
<tr>
<td>April</td>
<td>137</td>
<td>193</td>
<td>141%</td>
<td>321</td>
<td>233%</td>
</tr>
<tr>
<td>May</td>
<td>85</td>
<td>156</td>
<td>182%</td>
<td>187</td>
<td>219%</td>
</tr>
<tr>
<td>June</td>
<td>57</td>
<td>116</td>
<td>203%</td>
<td>116</td>
<td>203%</td>
</tr>
<tr>
<td>July</td>
<td>35</td>
<td>75</td>
<td>216%</td>
<td>125</td>
<td>361%</td>
</tr>
<tr>
<td>Aug</td>
<td>40</td>
<td>66</td>
<td>164%</td>
<td>215</td>
<td>536%</td>
</tr>
<tr>
<td>Sept</td>
<td>52</td>
<td>186</td>
<td>360%</td>
<td>192</td>
<td>371%</td>
</tr>
<tr>
<td>Oct</td>
<td>213</td>
<td>368</td>
<td>172%</td>
<td>557</td>
<td>261%</td>
</tr>
<tr>
<td>Nov</td>
<td>343</td>
<td>280</td>
<td>82%</td>
<td>631</td>
<td>184%</td>
</tr>
<tr>
<td>Dec</td>
<td>295</td>
<td>278</td>
<td>94%</td>
<td>593</td>
<td>201%</td>
</tr>
<tr>
<td>TOTAL</td>
<td>2,048</td>
<td>2,711</td>
<td>132%</td>
<td>4,671</td>
<td>228%</td>
</tr>
</tbody>
</table>
2.3 Environmental Conditions

CHH completes water quality monitoring and reporting for the CHL for groundwater monitoring wells, seepage blanket monitoring wells (SB), surface water and leachate. Based on the most recent reporting period available to SHA (September 2017 to April 2018), the results indicate the works are functional and are working as intended in accordance with the SPO. This recent reporting indicates that slight exceedances in water quality standards occur on site due to naturally occurring elevated concentrations of manganese and as documented in BC MOE releases from the CSR DW groundwater standard for manganese under CSR Protocol 9 for a number of sites located within this specific region of Southern Vancouver Island. Other natural occurrences have been detected as elevated dissolved aluminum and pH. Minor exceedances, observed infrequently, in Benzo(a)pyrene are considered false positives as a result of elevated turbidity in samples. The SHA Environmental Monitoring Plan in Chapter 10 takes this into consideration and outlines requirements to move to a low flow sampling method to reduce turbidity levels.

Leachate that is generated from the Landfill is collected and sampled before being transferred to an off-site treatment facility. The leachate is observed to be high in chloride and sodium, and illustrates conductivity of between 5,680 – 13,000 µS/cm. In contrast, samples collected from the seepage blanket wells show conductivities of 41 – 1,310 µS/cm.

Figure 2-4 shows a piper plot of 2017 Water Quality Data. The distribution of the geochemistry indicates that the leachate in the PEA is distinctly different than the water in the seepage blanket wells, with chloride as the dominant anion and sodium as the dominant cation.

The proposed post-closure monitoring program is discussed in detail in Chapter 10 of this report.

2.3.1 Hydrocarbons in Groundwater

Inconsistent hydrocarbon concentrations at low levels have been historically observed at MW-05 and MW-06, within 5 times respective detection limits, as shown in Table 2-3 and Table 2-4. The inconsistent hydrocarbon detections pre-date Landfill operations and as such cannot be attributed to Landfill activities. Silica gel analysis performed in February, June, and September 2016 have indicated that a large component of these detections are due to naturally occurring biogenic organics that may include lipids, plant oils, tannins, lignins, animal fats, proteins, humic acids, fatty acids, and resin acids. As mentioned previously the land surrounding the Landfill is under development, with quarrying and construction activities occurring frequently. The development surrounding the Landfill may be the source of the inconsistent hydrocarbon concentrations. The noted hydrocarbons at the Landfill have been observed on different dates and at different monitoring wells, the movement of the hydrocarbons to different wells suggests the source of the hydrocarbons does not remain in one location and represent a low impact to groundwater chemistry. There are a few possible transport...
pathways for the hydrocarbons to be transported, and run-off transportation including improperly sealed wells or migration through one of the few bedrock fractures that are present would explain the inconsistent nature of the hydrocarbon concentrations observed at the Landfill. Moreover, the detection of a hydrocarbon plume is not evident in the data as the concentrations of VOC’s and PAH’s are below detection prior and post landfilling operations.

Table 2-3: Hydrocarbon Concentrations at MW-5 and MW-6

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>CSR Standards⁽¹⁾</th>
<th>MW-5S</th>
<th>MW-6</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>As-built Well Depths</td>
<td>29 m</td>
<td>47 m</td>
</tr>
<tr>
<td>Hydrocarbons ug/L</td>
<td>EPH10-19</td>
<td>5000</td>
<td>5000</td>
</tr>
<tr>
<td></td>
<td>EPH10-19 (SG)</td>
<td>5000</td>
<td>5000</td>
</tr>
<tr>
<td></td>
<td>EPH19-32</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>EPH19-32 (SG)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>LEPH</td>
<td>500</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>HEPH</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 2-4: Hydrocarbon Concentrations for the Monitoring Well Network

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>CSR Standards⁽¹⁾</th>
<th>MW-2</th>
<th>MW-1D</th>
<th>MW1S</th>
<th>MW-3S</th>
<th>MW-3D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>As-built Well Depths</td>
<td>43 m</td>
<td>84 m</td>
<td>52 m</td>
<td>23 m</td>
<td>46 m</td>
</tr>
<tr>
<td>Hydrocarbons ug/L</td>
<td>EPH10-19</td>
<td>5000</td>
<td>5000</td>
<td>&lt;100</td>
<td>256</td>
<td>&lt;250</td>
</tr>
<tr>
<td></td>
<td>EPH10-19 (SG)</td>
<td>5000</td>
<td>5000</td>
<td>-</td>
<td>-</td>
<td>&lt;250</td>
</tr>
<tr>
<td></td>
<td>EPH19-32</td>
<td>-</td>
<td>-</td>
<td>175</td>
<td>275</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td>EPH19-32 (SG)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>&lt;250</td>
</tr>
<tr>
<td></td>
<td>LEPH</td>
<td>500</td>
<td>-</td>
<td>&lt;100</td>
<td>256</td>
<td>&lt;250</td>
</tr>
<tr>
<td></td>
<td>HEPH</td>
<td>-</td>
<td>-</td>
<td>175</td>
<td>275</td>
<td>290</td>
</tr>
</tbody>
</table>

2.3.2 Hydrocarbons in Leachate

From September 2017 to November of 2018, the site’s leachate has been analyzed monthly to monitor if hydrocarbons including LEPH/HEPH and Polyaromatic hydrocarbons are present in the Landfill’s leachate. It is noted that detections of LEPH/HEPH were observed in June and July of 2018. However, silica gel analysis of the duplicate sample from June 2018 indicates that this LEPH/HEPH fraction
was from non-anthropogenic sources possibly caused during the increased summer temperatures. Subsequently, following July 2018, LEPH/HEPH levels have remained below detection limits.

2.3.3 Groundwater Benchmarks

Groundwater benchmarks were created using the BC Ministry of Water, Land and Air Protection Protocol 9 for Contaminated Sites and Technical Guidance 12 Statistics for Contaminated Sites. A large data set consisting of groundwater monitoring locations MW-1S/D, MW-2, MW-3S/D, MW-4, MW-5S/D, and MW-6 were used for statistical analysis from the period of March 2011 to Dec 2018.

This data set, which predates landfilling on site and continues to the end of 2018, indicates that there have been no deleterious effects to groundwater caused by landfilling which began in July 2015. Of note, MW-01S data was omitted from the benchmark analysis prior to 2015 and reintroduced back into the data set in 2016 due to suspected tampering that occurred in May 2014. Analysis of parameters at MW-1S in 2014 displayed suspicious fluctuations in sodium, chloride, conductivity, total dissolved solids, and hydrocarbons.

The groundwater benchmarks represent the 95th percentile of the data set from 2011 to 2018 and are a conservative estimate of trigger concentrations that would warrant potential initiation of the site’s trigger and response plan outlined in Chapter 10. Benchmarks will be used in conjunction with Piper Plot trend analysis to determine whether responses are warranted. All benchmarks are well below CSR groundwater aquatic life and drinking water standards. The interquartile range was chosen to represent data variability by excluding large and small outliers in the data set. In cases where laboratory results were below the method detection limit (MDL), statistics were carried out using the formula 0.5MDL as per BC Ministry of Environment Technical Guidance 4 Environmental Management Act Authorizations.

Table 2-5 displays groundwater benchmarks for the CHL including data variability as displayed by the interquartile range. Key parameters in leachate are highlighted that represent parameters that are significantly above these groundwater benchmarks. The CHL Environmental Monitoring Plan discussed in Chapter 10 is designed to capture these parameters in laboratory analysis as well as analysis for conductivity, hardness, pH, total dissolved solids, fluoride, nitrate, and dissolved metals.
<table>
<thead>
<tr>
<th>Physical Tests</th>
<th>Groundwater Benchmarks March 2011 to December 2018</th>
<th>Key Parameters in Leachate (Highlighted)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>95th Percentile</td>
<td>Interquartile Range</td>
</tr>
<tr>
<td>Colour, True (TCU)</td>
<td>20.99</td>
<td>4.75</td>
</tr>
<tr>
<td>Conductivity (uS/cm)</td>
<td>1310</td>
<td>318</td>
</tr>
<tr>
<td>Hardness (as CaCO3) (mg/L)</td>
<td>614</td>
<td>155</td>
</tr>
<tr>
<td>pH</td>
<td>8.588</td>
<td>0.52</td>
</tr>
<tr>
<td>Total Suspended Solids (mg/L)</td>
<td>45.63</td>
<td>9.08</td>
</tr>
<tr>
<td>Total Dissolved Solids (mg/L)</td>
<td>732</td>
<td>261</td>
</tr>
<tr>
<td>Turbidity (NTU)</td>
<td>161.50</td>
<td>21.59</td>
</tr>
<tr>
<td>Anions and Nutrients</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alkalinity, Total (as CaCO3) (mg/L)</td>
<td>630.00</td>
<td>124.00</td>
</tr>
<tr>
<td>Chloride (Cl) (mg/L)</td>
<td>39.28</td>
<td>12.90</td>
</tr>
<tr>
<td>Fluoride (F) (mg/L)</td>
<td>0.36</td>
<td>0.11</td>
</tr>
<tr>
<td>Nitrate (as N) (mg/L)</td>
<td>1.13</td>
<td>0.05</td>
</tr>
<tr>
<td>Nitrate (mg/L) Cl&gt;10 mg/L</td>
<td>0.013</td>
<td>0.003</td>
</tr>
<tr>
<td>Sulfate (SO4) (mg/L)</td>
<td>87.13</td>
<td>24.28</td>
</tr>
<tr>
<td>Dissolved Metals</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aluminum (Al)-Dissolved</td>
<td>0.0596</td>
<td>0.0039</td>
</tr>
<tr>
<td>Antimony (Sb)-Dissolved</td>
<td>0.00118</td>
<td>0.0001</td>
</tr>
<tr>
<td>Arsenic (As)-Dissolved</td>
<td>0.0049945</td>
<td>0.0011325</td>
</tr>
<tr>
<td>Barium (Ba)-Dissolved</td>
<td>0.08256</td>
<td>0.0192</td>
</tr>
<tr>
<td>Beryllium (Be)-Dissolved</td>
<td>0.00005</td>
<td>0</td>
</tr>
<tr>
<td>Bismuth (Bi)-Dissolved</td>
<td>0.00005</td>
<td>0</td>
</tr>
<tr>
<td>Boron (B)-Dissolved</td>
<td>0.07704</td>
<td>0.0301</td>
</tr>
<tr>
<td>Cadmium (Cd)-Dissolved</td>
<td>0.001214</td>
<td>0.000015</td>
</tr>
<tr>
<td>Calcium (Ca)-Dissolved</td>
<td>192.40</td>
<td>49.00</td>
</tr>
<tr>
<td>Chromium (Cr)-Dissolved</td>
<td>0.000632</td>
<td>0</td>
</tr>
<tr>
<td>Cobalt (Co)-Dissolved</td>
<td>0.00250</td>
<td>0.001355</td>
</tr>
<tr>
<td>Copper (Cu)-Dissolved</td>
<td>0.00249</td>
<td>0.0003</td>
</tr>
<tr>
<td>Iron (Fe)-Dissolved</td>
<td>2.972</td>
<td>0.235</td>
</tr>
<tr>
<td>Lead (Pb)-Dissolved</td>
<td>0.0001</td>
<td>0.000075</td>
</tr>
<tr>
<td>Lithium (Li)-Dissolved</td>
<td>0.01214</td>
<td>0.00095</td>
</tr>
<tr>
<td>Magnesium (Mg)-Dissolved</td>
<td>37.26</td>
<td>7.82</td>
</tr>
<tr>
<td>Manganese (Mn)-Dissolved</td>
<td>2.134</td>
<td>0.301</td>
</tr>
<tr>
<td>Mercury (Hg)-Dissolved</td>
<td>0.000052</td>
<td>0.000005</td>
</tr>
<tr>
<td>Molybdenum (Mo)-Dissolved</td>
<td>0.00952</td>
<td>0.00524</td>
</tr>
<tr>
<td>Nickel (Ni)-Dissolved</td>
<td>0.004844</td>
<td>0.001165</td>
</tr>
<tr>
<td>Phosphorus(P)-Dissolved</td>
<td>0.1585</td>
<td>0.09675</td>
</tr>
<tr>
<td>Potassium (K)-Dissolved</td>
<td>4.2965</td>
<td>0.725</td>
</tr>
<tr>
<td>Selenium (Se)-Dissolved</td>
<td>0.00076</td>
<td>0</td>
</tr>
<tr>
<td>Silver (Ag)-Dissolved</td>
<td>0.000025</td>
<td>0</td>
</tr>
<tr>
<td>Sodium (Na)-Dissolved</td>
<td>71.52</td>
<td>10.20</td>
</tr>
<tr>
<td>Strontium (Sr)-Dissolved</td>
<td>0.6055</td>
<td>0.2805</td>
</tr>
<tr>
<td>Sulfur (S)-Dissolved</td>
<td>26.78</td>
<td>12.63</td>
</tr>
<tr>
<td>Tellurium (Te)-Dissolved</td>
<td>0.00025</td>
<td>0</td>
</tr>
<tr>
<td>Thallium (Tl)-Dissolved</td>
<td>0.0000276</td>
<td>0</td>
</tr>
<tr>
<td>Thorium (Th)-Dissolved</td>
<td>0.00005</td>
<td>0</td>
</tr>
<tr>
<td>Tin (Sn)-Dissolved</td>
<td>0.00019</td>
<td>0</td>
</tr>
<tr>
<td>Titanium (Ti)-Dissolved</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>Uranium (U)-Dissolved</td>
<td>0.007924</td>
<td>0.000853</td>
</tr>
<tr>
<td>Vanadium (V)-Dissolved</td>
<td>0.00594</td>
<td>0</td>
</tr>
<tr>
<td>Zinc (Zn)-Dissolved</td>
<td>0.025</td>
<td>0.006</td>
</tr>
</tbody>
</table>
2.3.4 Post Closure Monitoring Locations and Key Parameters

The post closure monitoring plan is discussed in detail in Chapter 10. Based on industry standards, hydrogeological modelling (Chapter 6), Guidelines for Environmental Monitoring and Municipal Solid Waste Landfills, and data collected from SPO monthly sampling requirements, quarterly monitoring is recommended at this site. Since the most likely pathways for any contaminant movement are through the seepage blanket downgradient of the site and surface water runoff, SHA recommends that the monitoring program focus on the seepage blanket wells during the dry season (2x events/year) and surface water station SW-1 on a quarterly basis.

Sampling of deep groundwater wells is not considered necessary since there is an upward gradient present beneath the site which causes groundwater to upwell and the deep bedrock is not conducive to conducting large quantities of groundwater (very few fractures with low fracture conductivity).

To provide insight on changes in groundwater chemistry, SHA recommends that a Piper Plot analysis be undertaken to interpret the quarterly sampling data. The key parameters developed from the historical data set addressed in this Chapter should be analyzed, along with the parameters needed to prepare and analyse the piper plots, calcium, magnesium, sodium, bicarbonate, sulfate, and chloride. Previous requirements for analysis have also included LEPH/HEPH and PAH, however, monthly site water quality data has demonstrated that these parameters are not contaminants of concern in the Landfill’s leachate since they occur below detection levels.

2.4 Landfill Design Assessment

As outlined in the Spill Prevention Order MO1701 and CHH As-builts Documents – MOE Comments, the final closure plan should include an assessment of the adequacy of the existing facility, including: seepage blanket, landfill base liner, anchor trenches, leachate collection and leak detection works, landfill cells and leachate storage upgrades.

**Basal Seepage Layer:** SHA was not involved in the detailed design nor construction QA/QC of those systems. However, the base of the landfill is being developed in a rock quarry. We have been assured by representatives of CHH that a continuous layer of shot rock was achieved by overblasting the rock quarry a minimum depth of 2.0 m below design grade prior to placement of the clay barrier. During SHA’s presence onsite during the ‘2017 Minor Construction Works’ several test pits were completed during leachate conveyance piping construction as well as the installation of the Seepage Blanket Wells where it was confirmed that the Seepage Blanket Layer seemed to be a minimum of 2.0m in depth. Overblasting of the competent Wark Gneiss bedrock has opened up fractures that now allow groundwater to permeate more freely than was the case in the past. This shallow layer is referenced as the seepage blanket.
**Depth to Water Table:** The PEA was designed to be in conformance with the applicable guidelines and regulations of the day. Waste Discharge Permit 105809 does not provide any requirements on the nature of the containment structures. As the containment facility for the PEA was constructed in 2014 and 2015, the applicable guidelines of the day were the 1993 Landfill Criteria. The 1993 Criteria recommended that natural control landfills be sited in areas where the water table is at least 1.2 m below ground. There is no comparable guidance for engineered landfills in the 1993 Landfill Criteria.

The 2016 LCMSW stipulates that all new landfills and lateral expansions be sited in areas where the water table is at least 1.5 m below ground. Although this new guidance does not apply, ground water monitoring of the water table undertaken by CHH has indicated that the water table is consistently several meters below the ground surface.

Previous analysis of deep monitoring wells has shown that the potentiometric surface in groundwater wells developed in the deep competent bedrock is near the quarry floor (see Active Earth drawing Figure 6, Detailed Cross Section B at Quarry) 2012-02-21. It is important not to confuse this piezometric surface in the competent bedrock with the water table. The water table has consistently been observed several metres below the pit floor when drilling blast holes, according to CHH representatives, providing clear evidence that an upward gradient exists beneath the landfill site.

**Clay Secondary Liner:** The PEA is lined with a 1 m thick impervious brown marine clay barrier sourced from the Victoria area. The existence and properties of this clay were verified during 2017 Minor Construction Works Secondary Clay Liner Investigation and documented in GHD report entitled Clay Basal Liner Investigation dated Dec 2017. Four test pits were excavated along the northern toe / edge of the Permanent Encapsulation Area (PEA) to assess the integrity of the secondary liner and visually confirm its depth. MOE and GHD personnel visually confirmed the presence and depth of the secondary clay liner, sent samples for analysis, and performed a Ground Penetrating Radar survey around the north, west, and south sides of the PEA.

Clays with an illite, chlorite and smectite mineralogy are generally considered relatively stable and in case of reactions typically convert to minerals that have higher cation exchange capacity. Swelling clays such as montmorillonite are susceptible to the collapse of the double layer which dramatically increases the permeability of the clay barrier when exposed to leachate with elevated salinity and/or increased acidity. X-ray diffraction testing was completed on the clay samples taken in 2017. The results indicate that the major mineral assemblages include smectite (at >30% Wt), with moderate amounts of chlorite and kaolinite (at 10-30% Wt), and minor amounts of illite and vermiculite. As per the testing notes, the smectite clay group also includes montmorillonite and nontronite.
The Victoria area marine clay used at the Hartland Landfill has not historically caused any problems with respect to leachate interaction. As outlined in this closure plan, the primary strategy to address environmental concerns at the site will be to install an effective final cover system that will minimize percolation and consequently reduce leachate generation. As discussed in Chapter 7, the HELP modeling completed for the site indicates that long term, post consolidation, the leachate generation will be only 0.12m³ per year.

**Acid Rock Drainage Potential:**

The MOE has requested the Closure Plan evaluate if the soils discharged in the PEA are potentially acid generating soils. Concerns have been expressed to the MOE that the high concentrations of sulfates could indicate potentially acid generating soils were discharged. SHA reviewed the soil quality of incoming soil available, and although the high concentrations of sulfates are present, the soil has low iron concentrations. Generally, the most common potentially acid generating rock is pyrite (FeS₂) which would result in high concentrations of iron and sulfate in the soil.

During a recent meeting with community representatives it was noted that the CHL received a shipment of contaminated soil from a storm water pond redevelopment project at Pacific Coast Terminals (PCT) in Port Moody. In the Teranis Executive Summary that was provided to us by the community representatives it was noted that the soil contained elemental sulphur and although sulphur is not a parameter that that is regulated under Schedule 7 of the CSR, it is recognized that this material has the potential to generate acidic leachate as the sulphur oxidizes.

To limit the potential for acid rock drainage (ARD) and to stabilize the soil Portland cement was added at a rate of 200 Kg/m³ at the PCT site prior to shipment. An additional 100 Kg/m³ of cement were added prior to compaction in the landfill. Addition of such a large amount of cement provides a very high level of neutralization potential. As a result, SHA is confident that the soil has been effectively neutralized.

As an additional safeguard to prevent the potential for ARD, Teranis proposed an encapsulation plan with a geomembrane “to minimize or eliminate leachate production and migration and to mitigate potential risks associated with acid production and metal leaching.” This is essentially the same strategy that has been adopted at the CHL site. Encapsulation is a recognized form of ARD mitigation as identified by B.C.’s top ARD experts (Price and Errington, 1998). Based on the leachate generated at the site, which is circum-neutral, it does not appear that the leachate is currently producing acid.

In the worst case scenario that the capping system allowed some infiltration and the cement neutralization potential was not sufficient to neutralize acid produced, ARD would potentially be generated, leading to dissolution of some metals such as iron and manganese. The resulting leachate
would still be effectively contained by the liner system and would then be neutralized on site and hauled off site for disposal.

**40 mil LLDPE Primary Liner**
The CHL PEA is lined with a 40-mil geomembrane which serves as the primary liner. The double liner approach adopted by CHL makes the PEA much more secure than most MSW landfills in B.C. that contain far more hazardous wastes. Furthermore, in SHA’s opinion, the double liner is equivalent to the liner requirements of the 2016 LCMSW which call for a 60 mil HDPE primary liner and a 750 mm thick compacted clay liner. This is supported by GHD’s report on the Clay Basal Liner Integrity where GHD indicated that the dual liner system is considered to exceed the 1993 Landfill Criteria Requirements and meet the requirements of the 2016 LCMSW.

The 2016 LCMSW recommends that a 60 mil HDPE liner be used as the primary geomembrane. This recommendation was made by SHA when originally developing the 2016 LCMSW for the Ministry in recognition of research undertaken by Dr. Kerry Rowe which revealed that geomembrane liners tend to deteriorate rapidly when subject to elevated temperatures. This is a particular concern in biologically active landfills that receive typical municipal solid waste, but it is not a concern at CHL because all of the soils placed into PEA are biologically inert soils that will not generate elevated temperatures. Therefore, the primary geomembrane will be subject to far less thermal stress and a 40-mil thickness will be adequate to provide the desired long term performance.

Given that the geomembrane will not be subject to elevated temperatures and that it is well cushioned top and bottom by 200 mm thick sand cushion layers, a service life in excess of 100 years is anticipated. However, as SHA has not had any involvement in the construction of this liner, we cannot warrant the liner integrity or service life, but only offer a professional opinion that a long service life is expected.

**Leachate Collection Layer:** The leachate collection layer was constructed of a 300 mm thick sand layer built at 2% grade and with perforated leachate collection pipes. As the PEA has been fully encapsulated and will remain fully encapsulated once the new final cover system is constructed, there will be no new precipitation entering the liner, other than minimum quantities of water through any undetected liner defects. Typically, such infiltration is minimal.

**Soil Filter:** A geotextile filter was not installed in the PEA above the sand drainage layer. Hemmera has raised concerns about this alleged omission. There is no requirement for a filter layer in the 1993 Landfill Criteria. The 2016 LCMSW guidance is for installation of a geotextile filter layer above the drainage blanket, or installation of an engineered graded soil filter.

Based on practical experience with geotextile clogging, SHA’s preferred approach to the design of leachate collection systems is to avoid geotextile filter layers and to instead utilize graded soil filters.
which are less prone to clogging. In this case, the 300 mm sand layer is expected to provide a high degree of filtration capacity and should be effective in preventing migration of fines toward the leachate collector. Furthermore, given that there is no new water entering the PEA there is no opportunity for water to carry the fines into the drainage layer and ultimately into the leachate collection system.

**Leachate Collection Piping:** The current condition of leachate collection piping represents the extension carried out during 2017 Minor Construction Works to accommodate the construction of the new leachate/leak detection facility. Further extensions will not be required as the Revised Updated Final Closure Plan does not warrant basal layer extension.

Leachate collection piping has not been installed in a herringbone fashion within the drainage layer in this case because it was not required in the 1993 Landfill Criteria. The minimum drainage layer requirement in the 1993 Criteria is a 300 mm thick sand drainage layer. Furthermore, the 300 mm sand drainage layer has more than enough hydraulic capacity to convey any collected leachate to the landfill toe as the PEA is fully encapsulated with negligible leachate flow.

**Grading of Permanent Encapsulation Area:** The design objectives of the PEA have changed as a result of the recently issued Spill Prevention Order and the final reclamation requirements issued in the site’s quarry permit. Since a permanent cover system is now to be constructed and long-term stability of that cover system must be assured, a site-specific system will be used to cap the existing side slopes and crest of the PEA. A 5H:1V grade will be developed to meet and exceed 2016 LCMSW stability and seismic requirements, productivity levels required by the site’s quarry permit, and further erosional and vegetative objectives. The final contours of the PEA have been adjusted to achieve the following objectives:

- Maximum final grades of 5H: 1V
- Minimum final grades of 25H: 1V (4%)
- Surface water crest and toe ditch
- Promote surface water run off
- Keep clean water clean

With the addition of soil from the SMA to the PEA and regrading of the existing PEA geometry on the crest, the final contours for CHL will be established at the desired grade. Elevations will range from 330 m at the landfill toe to 345 m at the landfill crest along the eastern edge of the PEA.
The existing settling pond located to the north of the site will manage onsite clean run-off from closed areas and clean quarry floor to knock out any suspended sediments prior to discharge.

The current state of the PEA on site includes slopes at approximately 2.5H:1V, specifically along the eastern slope. SHA has developed a strategy to mitigate contaminated soil exposure during construction by eliminating removal of existing 40 mil LLDPE Smooth Geomembrane from the sloped areas. The Revised Updated Final Closure Plan will require only a crest liner cut to accommodate soil transported from the sites SMA.
PERMANENT ENCAPSULATION AREA AND LEACHATE STORAGE FACILITY

CLIENT: COBBLE HILL HOLDINGS LTD.

PROJECT: COBBLE HILL LANDFILL
UPDATED FINAL CLOSURE PLAN 2019

TITLE: FIGURE 2-2
Short Duration Rainfall Intensity-Duration-Frequency Data

Donnée sur l'intensité, la durée et la fréquence des chutes de pluie de courte durée

Caution/Sujet à caution:
* 95% Confidence Interval > ±25%
  Intervalle de confiance de 95% > ±25%

LAKE COWICHAN
BC
1012055

1983 - 2002
14 years / ans

Latitude
48° 50’N

Longitude
121° 3’W

Elevation / Altitude
171 m

Return Periods/
Périodes de retour

Years / ans
100 90 70 50 25 10 5 2

Environment
Canada

Canada

Environment
Canada

CLIENT:
COBBLE HILL
HOLDINGS LTD.

PROJECT:
COBBLE HILL LANDFILL
UPDATED FINAL CLOSURE
PLAN 2019

TITLE:
INTENSITY DURATION FREQUENCY
(IDF) DATA FOR
LAKE COWICHAN BC

SCALE:
NTS

DATE:
2019/01/17

PROJECT NO:
PRJ18074

DESIGNED:
N/A

DRAWN:
MG

CHECKED:
TS

FIGURE 2-3
FIGURE 2-4

2017 WATER QUALITY DATA

COBBLE HILL LANDFILL
UPDATED FINAL CLOSURE
PLAN 2019

CLIENT: COBBLE HILL HOLDINGS LTD.
PROJECT: COBBLE HILL LANDFILL
TITLE: UPDATED FINAL CLOSURE
PLAN 2019

Legend

- SB-1
- SB-2
- SB-3
- SB-1
- SB-2
- SB-3
- Leachate
- MW-08
- MW-03S
3. SOIL RELOCATION AND ENCAPSULATION

Current CHL conditions include one completed and capped waste soil cell, referred to as the Permanent Encapsulation Cell (PEA), a Soil Management Area (SMA), a leachate/leak detection collection system and storage facility, a settling pond, and various stormwater ditch and diversion works.

Recently completed ‘2017 Minor Construction Works’ included but not limited too:

- Detailed inspection and completion of existing PEA liner system repairs and testing – QP and Liner Contractor inspected the entire PEA and identified approximately 21 minor defects (mainly ‘pin holes’ with a few patches needed) requiring repairs / patches and testing. Prior to liner repair work, all liner welding equipment was qualified and approved by QP in the field including parameters for; elongation, tensile strength and tear / shear resistance. QP inspected and approved the 40 mil LLDPE geomembrane liner material (used for patches) for thickness, density, carbon black content, puncture stability resistance and dimension. Post liner repair completion, QP inspected and approved all repair testing with ‘vac-box’ testing equipment;

- PEA crest ditch protection and ballasting – All crest ditch alignments were lined with heavy-weight non-woven geotextile and ballast was provided by placing sand bags and rubber tires (no rims) along the invert of all crest ditch alignments. During the above mentioned work, ballast material was changed from gravel to sand bags and tires based on a recommendation from the QP to minimize traffic (both foot traffic and lightweight equipment) on the liner system. It is the opinion of SHA that rubber tires and sand bags provide adequate ballast to eliminate the potential ‘trampoline effect’ in the lined crest ditches as well as provide sufficient energy dissipation to clean surface water run-off from the PEA to the quarry floor.

- Decommissioning of the Contact Water Pond – All contact water in the pond was removed and hauled offsite to a licensed disposal facility, existing pond liner was cut, folded and disposed of offsite at a licensed facility, a small amount of sludge was deposited in the SMA and finally the Contact Water Pond was backfilled with granular material, track packed in even lifts, and final grades hand seeded to promote rapid re-vegetation. All works were completed under QP oversight.

- SMA soil consolidation, tarping, ballasting and clean up – All existing soil within the SMA (and small quantity of sludge from the decommissioned Contact Water Pond) was consolidated into one centralized windrow under the existing SMA cover system. The lock block walls and asphalt floor were thoroughly swept and cleaned using hand tools and brooms. Windrowed soils were bucket packed with an excavator and covered with approved 6 mil poly sheets and secured with
a continuous row of sand bags along the base and with wooden pallets along the crest. The QP approved not using water to wash-down the lock blocks walls and floor as the re-introduction of water into the existing soils within the SMA is not desirable. Currently, the consolidated soil and cover system within the SMA is providing adequate protection to existing soils and shows no signs of further Contact Water generation or soil migration.

Further details regarding the above mentioned works have been submitted to MOE as part of the 2017 Minor Construction Works – As-Builts and Records Document.

Currently the PEA contains approximately 94,235 tonnes of soil; of which, 44,722 tonnes were received in 2015 and 49,513 tonnes were received in 2016. CHH has been granted permission to relocate the approximately 3,360 tonnes of soil from the SMA to the PEA to enable final landfill closure activities. The total amount of soil that will be contained in the PEA for final closure will be 97,595 tonnes.

The approximate 3,360 tonnes of soil proposed for relocation from the SMA to the PEA, represented approximately 1,867 m$^3$ based on an estimated soil density of 1.8 tonnes/m$^3$.

Described below is SHA’s recommended approach on how to relocate the remaining contaminated soil from the SMA to the PEA and complete the PEA liner encapsulation, repair and testing prior to completion of the remaining proposed PEA cover system works, to make certain that it will be protective of the environment and that the closure works are be compliant with the new LCMSW.

### 3.1 Contaminated Soil Relocation and Encapsulation Plan

Based on the most recent inspection reports for the Landfill, it is SHA’s understanding that the entire PEA liner system is completely sealed and operating as designed, fully containing previously landfilled soils, and collecting / shedding all surface water in a controlled manner. To minimize potential impacts to the PEA liner system, it is proposed that all contaminated soil from the SMA area will be relocated to the crest area of the PEA only as equipment access exists from the south side at the approximate same elevation as the PEA crest.

Prior to relocating SMA soils, the PEA crest area will need to prepared for soil acceptance. This involves detailed work completed by qualified liner installation contractors who will cut the existing liner around the perimeter of the crest area as outlined in Figure 3-1. Once the crest area PEA liner has been isolated, it should be carefully rolled up or folded and set aside for re-use during crest capping works.
Figure 3-1 outlines approximately 2,130 m² of operational area on the PEA crest for soil deposition. Based on the additional 1,867 m³ of contaminated soils proposed to be capped within the PEA, the average soil depth on the crest of the PEA will increase by approximately 900 mm. Figure 3-2 provides a conceptual cross section of the SMA soil post relocation and capping within the PEA.

The existing lined surface water ditch alignments on the PEA crest will remain unchanged as all relocated soil from the SMA will be deposited at least 3m offset (inward) from the existing ditch alignments. The proposed 3m offset will provide adequate space for the completion of the remaining portions of the cover system including a deep soil horizon layer for potential re-growth of large tree species overtop of the PEA post closure construction. The final grades on the PEA crest (liner system) will be maintained at a minimum 4%, towards the outside slopes.

Once all SMA soils have been relocated to the proposed PEA crest, the 40mil LLDPE geomembrane liner system, previously removed, will be re-installed over the top and re-welded to the existing PEA liner on all sides.

Once soil relocation is completed, the entire PEA liner system will undergo one final inspection with SHA’s QP and the Liner Contractor to identify an further repairs and testing before the remaining final cover system construction is commenced. It will also be imperative that the liner system is inspected daily during final cover system construction to make sure the 40 mil LLDPE geomembrane is covered without further damage.

Detailed Liner QAQC including daily inspection, equipment qualification, testing, documentation and reporting must be undertaken during all aspects of liner work.

Finally, the proposed temporary removal of the PEA crest liner, soil relocation from the SMA and re-establishment of the liner and liner welding should be scheduled during a time when weather forecasts predict a period of minimal to no precipitation. Additionally, this work should be completed as quickly as possible to minimize the amount of time the PEA crest is exposed to the environment and SMA soils are being transported and consolidated. A preliminary construction schedule has been conceptualized based on the proposed closure design and is presented in Chapter 4. As can be seen, the SMA Soil Relocation program is estimated to be completed within a 1-week period.

Further details on the remaining proposed cover system design are presented in Chapter 4.
3.2  Relocation of Soil from the Soil Management Area (SMA)

Soil from the SMA (approximately 1,867 m³ as outlined above) will be consolidated on the PEA crest surface and graded to approximately 4% within the existing crest perimeter ditches, as outlined above.

Soil from the SMA will be cleared from the concrete pad using a front end loader and placed into trucks for transport to the PEA. Care will be taken to ensure the soil is not discharged into the environment. Transport of the soil will take place during only on dry days to make sure no additional run-off or contact water is created due to precipitation falling on the open PEA. Trucks will carefully be loaded with soil within the SMA to ensure no overfilling takes place and no soil is spilled into the surrounding environment during transport. The loading area will be kept clean to minimize the amount of soil coming in contact with vehicle wheels.

The soil will be trucked to the PEA, approximately 250m from the SMA, using the existing crest and toe access roads. Truck access to the PEA crest will also require short metal ramps spanning over the existing south ditch which is approximately 2m wide. Soil will be carefully placed into the PEA to ensure no soil is discharged into the environment. Once the remainder of the soil has been relocated, placed and compacted in minimum 600mm lifts (2 lifts envisioned), the previously removed 40 mil LLDPE geomembrane will be re-positioned and securely welded. Further details to the final cover system design are outlined in Chapter 4.

Following soil relocation, the SMA (including the existing cover-all structure and asphalt pad) will be cleaned and decontaminated using a dry method for cleaning, i.e. the use of a skid steer, shovels, brooms and small hand tools. Contaminated contents from all clean up will be placed within the PEA for encapsulation. Poly tarps, sand bags and any other temporary cover and ballast material / infrastructure used in containing contaminated soils in the SMA will be disposed of offsite at a licensed disposal facility. Once decontaminated, the SMA may be de-constructed and removed at the owners discretion.

3.3  Contact Water Management

During the soil relocation period, any contact water / leachate from the PEA will continue to be managed using the existing leachate collection system. As discussed in Chapter 5, and as per 2017 completion of Minor Construction Works, a new double lined leachate storage facility currently exists on site, directly north of the PEA, for storage of leachate generated and collected from the PEA. The new infrastructure has been constructed inline with the existing system. The location of these tanks and associated works is outlined further in Chapter 7.
As the contact water storage pond was decommissioned during 2017 Minor Construction Works, there is potential for contaminated soil in the SMA to create contact water if work were to proceed during inclement weather conditions. Due to this, relocation will only be carried out during dry weather. Contaminated soil will remain partially covered during relocation to mitigate exposure to precipitation. Contaminated soil will also remain fully covered in the SMA during non working hours.

Figure 2-1 outlines the current locations of the relocated leachate and leak detection storage facilities.
EXISTING SMOOTH 40mil LLDPE GEOMEMBRANE LINER TO BE CUT ALONG THE SOUTH AND NORTH AND CAREFULLY REMOVED FROM THE CREST IN PREPARATION OF SOIL INSTALLATION FROM THE SMA.

EXISTING WASTE SOIL

EXISTING SAND CUSHION LAYER

EXISTING CROSS SECTION 1-1'

EXISTING 40mil SMOOTH LLDPE GEOMEMBRANE LINER REMOVED FROM CREST TO BE RE-INSTALLED OVER SOIL FROM THE RELOCATED SMA SOIL AND SAND FRICTION LAYER.

EXISTING WASTE SOIL

EXISTING SAND CUSHION LAYER

PROPOSED CROSS SECTION 1-1'

900 mm LAYER OF WASTE SOIL FROM THE SMA.
4. LANDFILL CLOSURE DESIGN

4.1 Introduction

In this chapter, the final closure design for the PEA Area is outlined. The objective of this chapter is to provide a detailed guide for the construction of an effective final closure system at the CHL. Additionally, requirements and concerns from the MOE are outlined and addressed. Finally, a preliminary schedule for closure construction is outlined based on the proposed commencement date of March 1st, 2019 and a completion date of October 31st, 2019.

The new PEA final closure design addresses comments and requirements made by the MOE in their Dec 19th, 2018 email, November 6th, 2018 letter, September 20th, 2018 letter and previous draft approval, as well as the second amended Spill Prevention Order (SPO) dated June 29th, 2017. Also, considerations have been included with regards to the Quarry Permit Q-8-094 approving work system and reclamation program, attached in Appendix A.

In addition, commentary responding to previous concerns outlined by MOE related to the above documents that has been provided in the July 21st, 2017 Updated Final Closure Plan has been expanded in this report:

- Contingency measures to address any failure of the works or escape or spill of leachate or contaminated soil into the environment (Chapter 5 and Section 10.12).
- Provide QP opinion and sign off on the difference in base liner thickness, material, and expected lifespan relative to the use of 40mil LLDPE vs. 60mil HDPE in LCMSW (discussed in Chapter 2).
- Provide QP opinion and sign off on the adequacy of the existing cover liner thickness and type (40mil LLDPE smooth non-textured) relative to the use of textured geomembrane or geocomposite equivalent to a 600 mm barrier layer with hydraulic conductivity equal to or less than 1x10^-7 cm/sec as specified in the LCMSW. As well, the use of smooth non-textured geomembrane should be re-evaluated before the final cover started (Chapter 2).
- QP inspection and documentation of all closure construction works in PEA Area (Chapter 4).
4.2 Final Cover Objectives

The purpose of final closure of any landfill is to put in place the necessary environmental control systems to effectively manage leachate, surface water and landfill gas (not present in this case). A well-designed closure system should provide the following benefits:

- Isolation of refuse preventing direct contact with humans and vectors.
- Minimization of infiltration and leachate production through diversion and run-off.
- Protection of the cover system from erosion by providing adequate cover soil depths to allow re-vegetation to a self-sustaining state using appropriate plant species that allow for natural succession, minimizing post closure maintenance.

In developing the final cover design for CHL to meet the above objectives, the local site conditions had to be considered. The types and thickness of soils and other materials selected for the cover were based both on regulatory guidelines as well as site-specific objectives and best management practices. Key elements considered in the final cover design were:

- Increased static and seismic stability will be achieved at the site by a slope grade of 5H:1V on the outer slopes versus the recommended slope of 3H:1V from the 2016 LCMSW. It has been determined by SHA through slope stability analysis (Chapter 8) that utilization of this slope will provide better environmental stability versus the 2016 LCMSW objective, sustain environmental protection during construction by eliminating the need to extend the clay basal liner and geomembrane on slopes, and meet the site’s quarry permit requirements of site productivity.

- The existing smooth geomembrane on the landfill slopes will be used. The Revised Updated Final Closure Plan will include a gentle slope of 5H:1V on north and east slopes. A 50mm sand friction layer will be applied on top of the liner on the slopes, followed by a gravel drainage layer over top to maintain appropriate forces of friction and promote drainage of infiltrated surface water above the existing liner. The gentle slope will also minimize erosion and soil loss inline with requirements from the MOE and Ministry of Energy and Mines. SHA has completed stability analysis utilizing the existing geomembrane with results that exceed recommendations supplied in the 2016 LCMSW. SHA has also determined through literature, industry examples, and observations from the CHH 2018 pre-winter liner inspection produced by Islander Engineering Ltd., that there have been no adverse effects of UV radiation or other forces to the existing PEA liner.

- SHA has determined there will be no adverse effects of loading caused by the increase in final cover thickness on existing infrastructure including PVC SDR 28 leachate/leak detection piping, HDPE DR17 leachate conveyance piping within the PEA, and proposed clean outs.
given the maximum thickness of soil surcharge which will be 8 m. The selected pipe can withstand such loads with minimal deflection, well within specified tolerances.

- According to the LCMSW leachate minimization objectives are to be achieved with a low permeability soil barrier layer with K (hydraulic conductivity) less than 1x10^{-7} cm/s on the PEA crest or a a 40 mil LLDPE geomembrane barrier. In this case, both will be present to provide maximum environmental protection and to minimize future leachate disposal costs. Hauling of leachate off-site for disposal is very expensive. Therefore, it is in CHH’s best interests to minimize future leachate production as much as practically achievable.

- Gravel drainage layers are proposed on the existing PEA slopes and a geocomposite drainage layer (Drain Tubes) on the crest area, above the existing 40mil LLDPE barrier layer in the cover system to prevent head build up and saturation of the subsoil and topsoil layers to further enhance cover system performance. This approach provides the desired performance resulting in minimal head build up and achieving the highest possible level of environmental protection post closure.

- A subsoil stabilizing layer comprised of till and/or residential soil at a minimum thickness of 1,500mm above the 40 mil LLDPE geomembrane and drainage layers and 500 mm thick topsoil / erosion control layer, will be adopted to protect the underlying barrier layer and to provide a medium that will support root structure and vibrant vegetation growth for the areas where a natural vegetated end use is planned with allowance for natural succession, as per the site’s quarry permit. Where an industrial end use is planned, the erosion control layer can be substituted by a pavement or gravel layer.

- End use will conform to the requirements by the Ministry of Energy and Mines. Post closure, the site will be naturally revegetated with native species, including quick / fast germinating grass species such as Fall Rye (hydroseed) to manage soil loss and minimize erosion during first winter/wet season. As per the site’s quarry permit, the site must be reclaimed to forestry/industrial quality. For this reason, imported soil quality for common fill will be in accordance to CSR guidelines for industrial land use, with specific factors in the CSR Omnibus that will be used on site to determine soil quality with consideration to site specific factors including protection to human health, to soil invertebrates and plants, and groundwater flow to fresh water surface water used by aquatic life.
4.3 Regulatory Requirements

Regulatory requirements for landfill closure have been stipulated in the 2016 LCMSW. The key requirements that dictate design of the final cover system are summarized below:

- The final cover barrier layer shall consist of a minimum of 600 mm of low permeability (<1x10^-7 cm/s) compacted soil (or equivalent) cap.
- The barrier layer shall be protected with a minimum 150 mm thick topsoil layer with approved vegetation established.
- Final cover shall be sloped at a minimum of 4%, to promote surface water runoff, to a maximum slope of 33%.
- Surface water runoff shall be directed outside and away from the leachate collection system.

In order to provide a high degree of security, the current cover system described in this plan exceeds the minimum requirements of the LCMSW, as described further in Section 4.5 through 4.7, below. Additionally, slope reclamation works will also include a maximum 5H:1V soil wedge constructed against the existing steep rock slopes adjacent to the PEA, on the east side. This work is being proposed to increase safety at the site, including allowances for long term road access to the toe of the landfill for inspection and monitoring events, in areas around the existing PEA post closure in addition to providing reclamation requirements as outlined in the Quarry Permit. The soil wedge will allow for land to be re-vegetated to a self-sustaining state with appropriate plant species and natural succession. This additional construction measure will provide for aesthetically pleasing site topography that will emulate pre-development slopes and will further aid in controlling surface water in the PEA Area post closure and provide further erosion controls measures, otherwise not in place, as outlined below in Section 4.8.

In particular, the engineered cover system being proposed to fully encapsulate the PEA and surrounding area will have performance levels that will result in several orders of magnitude less leakage than the minimum cover system design stipulated in the 2016 LCMSW.

By controlling pore pressure through internal drainage, the cover system will be at far less risk of slope failure on the upper geomembrane interface.

By adopting a thicker soil horizon above the barrier layers, the cover system will support a broader diversity of vegetation which will reduce the risk of long-term erosion damage and root penetration.
4.4 Earthworks and Environmental Controls to be Completed prior to Final Cover System Construction

As documented in Chapter 3, SHA has recommended that the side slopes of the PEA remain approximately 2.5H:1V to limit exposure of contaminated soil during final cover system construction. A 50mm sand friction layer will be placed overtop of side slopes to ensure long term slope stability, as discussed further in Chapter 9. Common fill will be used to reduce slopes above the liner to approximately 5H:1V at completion. The updated grading plan for the PEA is shown in Figure 4-1.

As mentioned previously in this report, a number of work tasks need to be completed prior to the final cover system installation to ensure stability of the final cover and environmental protection. SHA has mitigated exposure of contaminated soil by eliminating trimming of liner on side slopes as well as eliminating the need for base extensions of the landfill on the north toe. SHA envisions sequencing the following work tasks to complete the final closure of the PEA: (reference to Figure 4-3 will aid in understanding the tasks outlined below).

1. Trim existing liner along the PEA crest alignment adjacent to existing surface ditch works, roll back existing crest smooth LLDPE liner to allow placement of contaminated soil from the SMA.

2. The 1,867 m³ of waste soil present in the SMA will be transported, placed and compacted on the PEA crest to a grade of approximately 4%, during a period of dry weather.

3. A 12 oz. Non-woven Geotextile cushion layer will be placed overtop the relocated soils from the SMA followed by a re-installation of the 40 mil LLDPE geomembrane crest liner. Liner will be extended as necessary to tie in with existing barrier liners on the slopes.

4. The crest liner will be welded, by a certified liner contractor to the remaining existing 40 mil LLDPE geomembrane liner system and followed by leak testing and QA/QC inspection.

5. Prior to any further closure construction works, SHA’s QP and liner installation contractor will complete a detailed inspection of the entire PEA 40 mil LLDPE geomembrane liner, with repairs and testing as required.

6. Two leachate collection pipe cleanouts and two leak detection pipe clean outs will be installed at both the east and west ends of the existing collection systems within the north toe of the PEA to allow for flushing / maintenance of the collection systems if necessary, as outlined in Figure 7-1.

7. The final cover system for the both the crest and slope area will then be completed as per design outlined in Figure 4-2 and 4-3.
It should be noted that all construction works associated with re-opening the PEA to complete approved filling of remaining soil from the SMA area need to be completed during dry weather conditions in order to ensure minimal contact between waste soil planned for relocation to the PEA crest from and precipitation. This includes potential temporary tarping of exposure areas when inclement weather is forecast.

Once all soil from the SMA is transported and placed within the PEA, the SMA should undergo a final decontamination, using a dry method i.e. utilizing a skid steer, brooms, shovels and small hand tools to remove end contaminants for disposal to the PEA crest.

4.5 Final Contour Design

The final contour design at the CHL is presented in Figure 4-1. The design involves a gentle sloping dome shaped crest at approximately 345m ASL and sloping to the south, west and north to surface water ditches before grading at approximately 5H:1V down the north and east slopes to the quarry floor at an elevation of 330m ASL, approximately 15m below the crest elevation.

Additionally, based on quarry permit reclamation requirements, adjacent to the PEA area, a 5H:1V slope will be completed against the southeastern vertical shear cliffs of the quarry. This will be completed during subsoil importation works as part of the stabilizing soil wedge proposed off the north and east slopes of the PEA, as outlined in Figure 4-1. This reclamation work is further discussed below in Section 4.8. Figure 4-4 outlines the overall material volume requirement for proposed cover system and reclamation aspects of the closure works. As shown, the total aggregate and soil requirements include approximately 44,772 m$^3$, of which approximately 1,867 m$^3$ includes the contaminated soils to be relocated from the SMA, 850 m$^3$ for gravel drainage layer materials proposed on the outer slopes of the PEA, 32,710 m$^3$ for subsoil and 9,345 m$^3$ of topsoil / erosion control layer.

4.6 Elements of the Final Cover Design - on Landfill Crest

To achieve the objectives outlined previously, a minimum cover system consisting of a topsoil horizon and a barrier layer is required by the MOE. Additional layers that are usually introduced by SHA in our cover designs include a drainage layer on top of the barrier system. Depending on the particle size gradation of the various layers, it may also be necessary to introduce geotextile separation / cushion layers at key interfaces to prevent migration of topsoil or clay into the various drainage layers. Healthy vegetation is also a key element of final closure. In the discussion below, layers are presented in a bottom to top order.
Figure 4-2 provides a detailed illustration of the recommended final cover veneer designed for the CHL. Note that the crest and side slope areas will utilize the existing 40 mil LLDPE smooth geomembrane, but possess slightly different closure systems to ensure required stability.

The smooth 40 mil LLDPE geomembrane liner should function well with no stability concerns on the landfill crest. This layer will be sandwiched between a 12 oz. Non-woven Geotextile on top of the relocated contaminated soils from the SMA and a geocomposite cushion / drainage layer above. Overtop of the geocomposite cushion / drainage, a low permeability soil layer is proposed at a minimum 1,500mm thickness. Finally, a 500mm topsoil / re-vegetative layer will be installed to provide a healthy and natural plant and tree community.

The additional proposed soil depths within the cover system, above the 40mil LLDPE geomembrane liner, of minimum 2,000mm (subsoil & topsoil) will aid in facilitating natural succession and regrowth of potential deep rooting trees and shrub species. The minimum 2,000mm soil depth was ascertained from the Forestry Chronicles, provided for reference in Appendix B.

Below is a description of each proposed closure design layers for the Crest Area, as presented in Figure 4-2.

4.6.1 Primary Barrier Layer

The existing 40mil LLDPE smooth geomembrane liner will be used as the primary barrier to minimize infiltration and to efficiently shed surface water from the above layers to ensure stability of the overall cover system and minimize further leachate generation. As discussed previously, this layer will be carefully removed to allow for relocation of soils from the SMA area, prior to being re-installed. SHA recommends the use of a 12 oz. Non-woven Geotextile be placed overtop of the graded soils from the SMA prior to re-installation of the barrier layer. This task should be completed under full time QP supervision by SHA’s highly experienced inspector and will require Liner QA/QC testing overseen by SHA.

4.6.2 Drainage / Cushion Layer

To protect the primary barrier layer geomembrane liner from above, a geocomposite ‘drain tube’ system is specified in SHA’s design. Material specification for the above mentioned ‘drain tube’ layer are provided in Appendix C. The product features a drainage layer and filter layers comprised of short synthetic staple fibers of 100% polypropylene or polyester needle punched together with perforated corrugated polypropylene pipes regularly spaced at approximately 4 pipes /meter between two geotextile filter layers. This geocomposite layer will be fitted to drain the crest into the existing crest.
ditch system and will overlap the gravel drainage layer proposed on the slope areas by a minimum 1,000mm.

The purpose of a drainage layer on top of the primary barrier is to quickly convey water that infiltrates the topsoil and secondary barrier layer to the landfill crest ditches. Without an effective drainage layer, the topsoil could become saturated during heavy rainfall events. Given the gradual existing slopes on the crest area and the presence of two crest ditches, SHA is confident that this geocomposite installation will be adequate in draining any infiltrating clean water that accumulates due to precipitation. Additionally, the upslope run-on ditching will ensure the crest drainage layer will only need to manage precipitation falling on the PEA itself.

It should be noted that prior to installing the geocomposite drainage layer on the crest, the existing geomembrane barrier layer should be inspected in detail for deficiencies such as punctures, tears, holes and corrected in the presence of SHA’s qualified professional for sign off.

4.6.3 Subsoil Layer
A minimum 1,500mm thick compacted subsoil layer, with a hydraulic conductivity of 1x10^-6 cm/s or lower, will serve as a secondary containment / protective layer. To achieve this low level of permeability, soils must contain a significant percentage of clay-sized particles. As mentioned previously, the additional soil depth of this layer will provide protection to the underlying barrier layer in future years when large trees and shrubs begin to establish as per the end use objectives of reclaiming the landfill area to a nature state.

As per the site’s quarry permit, the site must be reclaimed to forestry/industrial quality. For this reason, imported soil quality for common fill will be in accordance to CSR Omnibus regulations.

4.6.4 Top Soil Layer
A layer of organic topsoil is essential to allow a healthy and sustainable vegetative community on top of the final cover system to develop as well as aid in erosion control and prevention. The minimum requirement is for a 150 mm thick layer of topsoil. In most final cover designs SHA recommends a thicker layer of topsoil, in this case a 500 mm thick layer is planned to provide for a vibrant natural plant / forest community developed through natural succession. The thick topsoil layer will provide sufficient moisture retention in the soil during periods of drought, thereby preventing plant mortality in the early stages, and will reduce the risk of root penetration into the underlying barrier layer. Finally, the topsoil / vegetative layer should receive hydroseed application directly after installation to promote vegetative growth prior to Fall / Winter rain events.
Based on the preliminary construction schedule outlined below in Section 4.10, completion of the closure construction on the crest area of the PEA will be expedited early on in the schedule to make certain surface water infrastructure and erosion control measures can be completed before the winter rains set in.

4.7 Elements of the Final Cover Design - on Side Slopes

In this plan the cover system for the side slopes has been modified to achieve a stabilizing wedge which will allow the existing smooth LLDPE liner to continue to function as the permanent primary barrier layer. The following provides a description of each final cover system layer for the north and east outer slopes of the PEA where grades range from 3H:1V to 5H:1V.

The 2016 LCMSW guidelines recommend the final cover should be graded from a minimum 4% to a maximum of 33% (3H:1V). In this updated closure plan the outer slopes have been designed at approximately 5H:1V (20%) to provide additional reinforcement for the proposed cover system by constructing a buttressing wedge at the landfill’s toe and against the existing 2.5H:1V geomembrane covered slope of the PEA. Further, this solution makes the final cover more protective of the environment. Reduced side slopes will result in less potential for erosion and the increased thickness of the subsoil and topsoil layers will protect the underlying barrier layers from root penetration damage. This will allow the site to establish a natural and sustainable vegetation community through plant succession by providing adequate rooting depth that will support shrub and tree species to mature without having to periodically cut down the trees to protect the liner.

Below is a description of each proposed closure design layers for the PEA side slopes, as outlined in Figure 4-2. Also, SHA has provided details for the proposed cover system at the transition from the crest area to the sloped areas of the PEA, as presented in Figure 4-3.

4.7.1 Primary Barrier Layer

The existing 40mil LLDPE smooth geomembrane liner will remain in place on the side slopes of the PEA as its SHA’s opinion that removing and replacing this layer with an equivalent liner with added textured stability would increase the risk to the potential receiving environment during replacement works. As well, it would significantly increase construction costs and waste approximately one Ha of perfectly function geomembrane material.

Instead, with the stabilizing wedge approach the existing smooth membrane can be left in place while realizing an improved level of global slope stability. Further details on slope stability are outlined in Chapter 8. The existing 40 mil LLDPE smooth geomembrane will be sandwiched between the existing
200mm sand from below, and the placement of a 50mm coarse sand friction layer and 12 oz. non-woven geotextile above. Based on test results of smooth LLDPE and concrete sand from other projects, we are anticipating a friction angle of 23.6° at the upper and lower geomembrane interface. The friction angle will need to be confirmed by laboratory testing once the sand for the interface is selected.

The existing geomembrane barrier layer will continue to prevent infiltration into the PEA and will direct all infiltrating precipitation into the overlying drainage layer, as described below. Prevention of pore pressure buildup and the very gentle slope angles will ensure long term stability of the overall cover system.

4.7.2 Sand Friction Layer and Cushion Geotextile

Due to the nature of smooth geomembranes, stability on slopes greater than 10% can be of concern based on the friction angle between the smooth liner and a cushion geotextile layer, which can be as low as 8°. In order to make certain that the required level of stability is met, SHA’s updated design includes a 50mm coarse sand friction layer to be applied directly to the existing 40 mil smooth sheet. This can be completed by manual labor or by way of a ‘stone slinger’. Both installation techniques have been successfully used to deploy the friction layer under SHA’s supervision on past projects and we are confident that the same approaches can be adopted at this site. The sand friction layer is expected to increase the interface friction angle to about 23°.

In order to make certain the sand friction layer remains intact and does not migrate due to continued drainage, SHA has specified that a 12 oz. non-woven geotextile be placed over top of the sand friction layer. The non-woven geotextile will also provide additional puncture protection, in conjunction with the 200mm thick coarse aggregate drainage layer that will be placed on top of the geotextile filter.

4.7.3 Drainage Layer

The purpose of a drainage layer on top of the barrier layer is to quickly convey water passing through the soil horizon down slope to the landfill toe or interceptor ditch. Without an effective drainage layer, the above soils could become saturated during heavy rainfall events. This condition could lead to excessive head build-up on the barrier layer which in turn could lead to erosion and slumping problems on side slopes and increased infiltration. Use of a high permeability topsoil medium could be considered; however, in our opinion a high permeability topsoil layer would not achieve the same performance as a gravel drainage layer and would likely become saturated and unstable during extreme precipitation events. To meet functionality requirements, the drainage layer should have a hydraulic conductivity of 1x10⁻³ cm/s or better.
SHA is recommending a 200mm thick gravel drainage layer on the PEA side slopes. Specifications include a clear crush 5-25mm gravel with at minimum 2 flat sides and minimum round particles. This drainage layer will effectively drain any infiltrating surface water down the outer slopes of the PEA to the landfill toe without allowing any significant head build up on top of the liner. As shown on the attached Figures 4-5 and 4-6, a linear PEA toe drainage soak away trench will be constructed with pervious shot rock and separation geotextile, and will drain all infiltrated surface water away from the foundation of the landfill and proposed cover system, into existing porous Seepage Blanket layer, underneath and around the PEA.

4.7.4 Filter Layer
To prevent migrating fines from the subsoil and topsoil layer from clogging the underlying drainage layer, we recommend that a non-woven geotextile blanket be placed over top of the drainage layer. The most important characteristic of the geotextile in this application is the opening size. This should be small enough to ensure that overlying topsoil or subsoil material cannot pass into the drainage layer, a light to medium weight non-woven geotextile should be utilized. Geotextile panels should be sewn/heat fusion welded in the field to ensure that they do not become separated during placement. Standard geotextile filter criteria should be referenced to ensure that the material will prevent migration of the fine cover soils.

4.7.5 Subsoil Layer (Stabilizing Soil Wedge)
A minimum 1,500mm thick compacted common fill soil barrier layer with a hydraulic conductivity of $1 \times 10^{-5}$ cm/s or lower will be utilized to enhance environmental controls as well as provide the stabilizing soil wedge on the north and east slopes of the landfill. This material will be placed in compacted lifts not exceeding 300mm in depth. As shown in section on Figure 4-7, the stabilizing soil wedge will have a maximum thickness of approximately 8m near the north and east toe of the existing PEA and will daylight to the north where the proposed 5H:1V slope meets the quarry floor and toe surface water ditch.

4.7.6 Top Soil Layer
A layer of organic topsoil is essential to ensure a healthy and sustainable vegetative community on top of the final cover system as well as aid in erosion control and prevention. The minimum requirement is for a 150 mm thick layer of topsoil. In most final cover designs SHA recommends a thicker layer of topsoil, in this case a 500 mm thick layer is planned to provide for a vibrant natural plant / forest community developed through natural succession. The thick topsoil layer will provide sufficient moisture retention in the soil during periods of drought, thereby preventing plant mortality in the early stages, and will reduce the risk of root penetration into the underlying barrier layer. Finally, the topsoil / vegetative layer should receive hydroseed application directly after installation to promote vegetative growth prior to Fall / Winter rain events.
4.8 Common Fill Importation

The site is zoned for Primary Forestry (F-1) according to the CVRD’s zoning by-law. Land zoned for Primary Forestry (F-1) may be used for the following industrial purposes: extraction, crushing and milling of aggregate material.

As per the MOE’s Fact Sheet 41 on Contaminated Sites, numerical environmental quality standards for the site are set according to this industrial land use. The design of the updated final closure plan will follow F-1 zoning and adhere to Section 2 of the site’s Quarry Permit Q-8-094 amended October 28th, 2015 which states that the surface of the land and watercourses shall be reclaimed to the following land use: Industrial.

Common fill soil quality will consist of parameters specified in Schedule 3.1 of the Environmental Management Act Contaminated Sites Regulation Consolidation Current to July 24th, 2018 with concentrations up to the permitted Column 9 Industrial Land Use Standards. Stringent site-specific factors for Human Health Protection and Environmental Protection will be taken into account to provide safety for the site’s groundwater as listed in Table 4-1 below. The most stringent concentration between the four site specific factors listed in the table will be utilized to determine soil quality which ultimately will be below CSR Schedule 3.1 Column 9 Industrial Land Use standards.

<table>
<thead>
<tr>
<th>CSR Schedule 3.1 Column 1</th>
<th>Four Site-specific Factor will be used to calculate environmental standards</th>
</tr>
</thead>
<tbody>
<tr>
<td>Human Health Protection</td>
<td>1. Intake of contaminated Soil</td>
</tr>
<tr>
<td></td>
<td>2. Groundwater used for drinking water</td>
</tr>
<tr>
<td>Environmental Protection</td>
<td>3. Toxicity to soil invertebrates and plants</td>
</tr>
<tr>
<td></td>
<td>4. Groundwater flow to surface water used by aquatic life</td>
</tr>
</tbody>
</table>

Soil quality of imported soil material will be assessed by a QP in the form of a Waste Approval Application form (WAA). The general WAA form template is displayed in Appendix E and includes soil description, quantity, generator site information, and contaminants of concern. Changes in soil quality observed on site will be accompanied by generator QP certification. It is expected that generator sites will follow Technical guidance 1 in soil classification and delineation of contamination.
for soil removal and the assessments will be conducted under the oversight of a QP who will be responsible for ensuring that only materials that meet the acceptance criteria are shipped to the site. The project management team will undertake independent confirmatory sampling if there is evidence or indication that imported soil exceeds the acceptance criteria. SHA does not believe that further confirmatory sampling and testing will be necessary unless the soil quality is observed to change drastically during importation events.

During importation and placement, common fill soil will be spread and compacted in maximum 300mm lifts and will be contained to the footprint defined by the 5H:1V slope off the PEA crest. The outer slopes of the soil stabilizing wedge will receive erosion protection measures as the wedge develops vertically to promote rapid germination and minimize soil loss.

To make certain that all existing and proposed infrastructure such as PEA liner system, leachate and leak detection pipes, clean outs and seepage blanket wells remain intact and are not compromised during proposed earthworks, SHA recommends that all equipment maintain a minimum buffer of 1m away from all above-mentioned works. Soil must be placed by hand or with small dedicated equipment to ensure integrity of all environmental and monitoring systems within the exclusion zone.

SHA has reviewed all existing and proposed subsurface collection, conveyance and monitoring pipe infrastructure based on the proposed final design concept and confirmed that the proposed loads will not deflect the existing PVC SDR28 and proposed HDPE DR17 pipe beyond acceptable limits. This is further discussed in Section 7.6 – Leachate Collection and Management.

SHA proposes that common soil fill be placed and compacted to 95% of the maximum Standard Proctor Density.

A full-time construction superintendent will be onsite at all times during soil import events with a dedicated traffic foreman managing all haul trucks and earthmoving equipment to make certain all operators are working in a safe manner. Daily safety meetings are to be conducted during all active construction days onsite prior to work commencement to discuss daily activities, outline safety concerns and will be documented in daily site reports.
4.9 Eastern Soil Wedge

Similar to reasons mentioned above, in order to meet the requirements of the MOE and the Mines / Quarry permit, we are proposing that the vertical shear cliffs in the south eastern corner of the quarry are remediated to similar grades proposed on the PEA outer slopes, at approximately 5H:1V. Justification for this included safety during post closure inspection and monitoring works as well as a provision to return the PEA Area closer to pre-development grading and to provide suitable habitat for native species of grasses and trees. Additionally, this concept will aid in overall erosion prevention measures at the site while creating room to construct a stable and long-term access road from the southern crest of the site to the new leachate collection infrastructure, monitoring well network, south surface water settlement pond, etc. This concept will provide an aesthetically pleasing topography and green space post closure construction. The eastern soil wedge is shown in plan view in Figure 4-1.

During importation and placement, common fill soil will be spread and compacted in maximum 300mm lifts and will be contained to the footprint defined by the final contour plan outlined in Figure 4-1. The outer slopes of the eastern soil wedge will receive erosion protection measures as the wedge develops to promote rapid germination and minimize soil loss.

SHA proposes that common soil fill be placed and compacted to 95% of the maximum Standard Proctor Density.

Finally, monitoring well MW-4 will need to be decommissioned prior to the construction of the Eastern Soil Wedge. It is expected that decommissioning work will follow best management practices and industry standards to ensure protection of groundwater.

4.10 Design Contingency

It is important that reviewers of this plan recognize the commitment of CHH to ensure that the site will be closed in a way that will minimize future environmental impacts. In particular, the minimum closure requirement for landfill sites in non-arid regions is a 600 mm low permeability soil cap with a hydraulic conductivity of less than $1 \times 10^{-7}$ cm/s, covered with 150 mm of top soil. SHA has conducted numerous comparative analyses that demonstrate that clay liners allow 40 to 50% of precipitation to permeate into the waste whereas geomembrane caps divert close to 100% of the precipitation falling on the landfill surface. To ensure that there will be minimal leachate generated in the long term, CHH is committed to installing a geomembrane cap rather than a soil cap. In this application a geomembrane cap will be more costly than a clay cap because it requires a drainage layer to prevent head build up on the cover barrier layer and a robust stabilizing subsoil wedge layer that will have the
functionality of a secondary cap liner. Additionally, as outlined above, to ensure stability to the overall system, the outside slopes of the final cover have been readjusted to 5H:1V.

Furthermore, a 1,500 mm secondary low permeability soil barrier is being installed on the crest areas, above the 40mil LLDPE geomembrane primary barrier layer, as a design contingency to provide very long-term diversion of precipitation far into the future., beyond the expected service life of the geomembrane.

SHA’s design adds robust drainage and filter layers on both the crest and slope areas to provide long term protection of the geomembrane and to ensure that the top soil horizon will not become saturated, which could result in instability of that horizon.

Finally, the final cover design increases the top soil horizon thickness from 150 mm as required in the landfill criteria to 500 mm. This will allow the final cover to support vegetative growth including shrubs and trees such that the final cover will not need to be maintained in perpetuity.

In addition, as discussed in Chapter 5, in the event that seepage is detected in the leak detection system, and groundwater and seepage blanket monitoring locations start to indicate leachate is leaking from the landfill, a contingency leachate collection trench and sump can be excavated on top of solid bedrock. These works could be quickly installed, if needed, to collect leachate from the downgradient toe of the landfill and then pump it into the leachate storage tank for removal offsite.

These enhancements have been incorporated into the design to ensure longevity of the cover, to reduce risk and to minimize future leakage and associated treatment costs. By adopting a conservative design approach our intent has been to avoid costs down the road post closure that would initiate the need for injection of funds from a contingency.

It is important to recognize the many additional safety features of the proposed cover system design that go far beyond the minimum landfill criteria requirements to protect the environment. The review comments from the Ministry’s technical contractor should also capture these important measures being taken to protect the environment and to give concerned citizens in the area that will likely be reviewing these reports a comfort level that the CHL is being properly closed to a standard that is higher than most MSW landfills in B.C.
4.11 Preliminary Closure Construction Scheduling

CHH is proposing slight modifications to the construction schedule, based on MOE’s September 20, 2018 letter that states:

- Construction activities for implementation of the Updated Final Closure Plan must be commenced on or before June 1, 2019;
- Before the commencement of construction activities or March 31, 2019, whichever is earlier, a detailed construction activity work plan and implementation schedule certified by a Qualified Professional acceptable to the Minister must be submitted to the MOE for review and approval; and
- Construction must be completed by September 30th, 2019.

Alternatives, as per the construction schedule outlined in Table 4-2 below, incoming soil fill for placement on the PEA base and slopes is proposed to commence in early March, 2019 and will continue through till mid October, 2019. The earlier proposed start date is requested to accommodate a secured soil source for the closure works that will need to be imported earlier than March, 31st, 2019 to minimize double handling of materials and prevent soil loss through erosion.

Additionally, PEA slope cover system construction, including extending wells and cleanouts will need to be completed prior or during common fill importation, followed by drainage layer works, topsoil, hydromeed and erosion protection works.

Soil relocation from the SMA to the PEA will begin in early June, 2019 during a period of dry weather and is scheduled to last approximately 1 week.

The PEA cover system will follow the above soil relocation works and should be completed within 1 month and will also include ditching and armoring works.

Finally, site clean up, ditching, roadways and erosion protection works will be completed before the end of October, 2019.

SHA recommends that continuous QA/QC be undertaken during all geomembrane work and during placement of the first 300 mm of materials on top of the liner. All soil import and stabilizing soil wedge construction can be completed under biweekly inspection from SHA’s QP. Daily inspection should be reactivated during final shaping of side slopes and hydromeed.
It should be noted that closure construction works will not commence prior to receipt of Closure Plan Approval from MOE.

Tabulated preliminary construction schedule is presented below in Table 4-2:

### Table 4-2 Proposed Construction Schedule - Preliminary (CHH)

<table>
<thead>
<tr>
<th>Task</th>
<th>Start Date</th>
<th>End Date</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Preliminary Construction of Soil Stabilization Wedge</strong></td>
<td>01-Mar</td>
<td>15-Oct</td>
</tr>
<tr>
<td>- Incoming Fill Placement for PEA Slopes</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>PEA Slope Cover System</strong></td>
<td>01-Mar</td>
<td>15-Oct</td>
</tr>
<tr>
<td>- Extend Seepage Blanket Wells</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Extend Cleanouts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Place Common Fill (Clean Fill on Soil Wedge)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Place Growing Medium</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- HydroSeed and Erosion Protection</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Drainage Layer Install on Slopes</strong></td>
<td>01-Mar</td>
<td>01-Sep</td>
</tr>
<tr>
<td>- Coarse Sand Friction Layer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Drainage Layer (Gravel)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Install Toe Drainage Soak Away Trench</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>PEA Leachate /Leak Detection Piping Cleanout and Extensions</strong></td>
<td>20-May</td>
<td>01-Jun</td>
</tr>
<tr>
<td>- Cut liner and install/weld piping for cleanouts</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Re-Seal Liner</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Waste Soil Relocation and Cover / Seal</strong></td>
<td>01-Jun</td>
<td>07-Jun</td>
</tr>
<tr>
<td>- Open Crest Liner</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Relocate SMA soil and compact on PEA crest</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Weld Liner System on Crest of PEA</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>PEA Crest Cover System</strong></td>
<td>07-Jun</td>
<td>31-Jul</td>
</tr>
<tr>
<td>- Subsoil Layer (Possibly Clay or low permeability soil)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Place thick Growing Medium Layer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Hydroseed</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>PEA Crest Ditching &amp; Armoring</strong></td>
<td>15-Jun</td>
<td>20-Jun</td>
</tr>
<tr>
<td>- Ditch liner aggregates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- RipRap</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Site Clean Up, Ditching, Roadways, Erosion Protection, etc.</strong></td>
<td>15-Sep</td>
<td>31-Oct</td>
</tr>
<tr>
<td><strong>Construction QAQC</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Inspections schedule based on Complexity of Work Being Complete</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Project Wrap Up and Reporting</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
TYPICAL COVER SYSTEM - CREST AREA

HYDRO-SEED AND NATURAL RE-GROWTH

TOPSOIL

SUBSOIL

EXISTING SAND CUSHION LAYER

EXISTING WASTE SOIL

5 to 25 mm CLEAR CRUSH DRAINAGE GRAVEL

EXISTING 40mil SMOOTH LLDPE LINER

12 oz NON-WOVEN GEOTEXTILE

EXISTING 40mil SMOOTH LLDPE LINER

PROPOSED 8 oz NON-WOVEN GEOTEXTILE

DRAINTUBE 606 ST4 50% OR APPROVED EQUIVALENT

WASTE SOIL TO BE RE-LOCATED FROM THE SMA TO THE PEA CREST

TYPICAL COVER SYSTEM - SLOPED AREA

HYDRO-SEED AND NATURAL RE-GROWTH

TOPSOIL

COMMON FILL COMPACTED IN 600mm LIFTS TO 95% MODIFIED PROCTOR

EXISTING SAND CUSHION LAYER

EXISTING WASTE SOIL

EXISTING 40mil SMOOTH LLDPE LINER

12 oz NON-WOVEN GEOTEXTILE

PROPOSED DRAINTUBE

PROPOSED 12oz NON-WOVEN GEOTEXTILE

PROPOSED 8oz NON-WOVEN GEOTEXTILE

EXISTING 40mil SMOOTH LLDPE LINER

50mm COARSE SAND FRICTION LAYER

8 oz NON-WOVEN GEOTEXTILE
**TYPICAL COVER SYSTEM DETAIL - CREST TO SLOPE TRANSITION**

**SCALE:** 1:50

---

**EXISTING 40mil LLDPE SMOOTH LINER**

**PROPOSED 12oz NON-WOVEN GEOTEXTILE**

**PROPOSED DRAINTUBE**

**PROPOSED 8oz NON-WOVEN GEOTEXTILE**

---

**EXISTING 40mil SMOOTH LLDPE LINER TO BE REPLACED ON CREST AREA AFTER SOIL RE-LOCATION FROM SMA**

**DRAINTUBE 606 ST4 D25 OR APPROVED EQUIVALENT**

**EXEMPLARY RE-VEGETATION (GRASS, TREE AND SHRUBS SPECIES)**

**THICK SOIL HORIZON - MIN 2,000 mm THICK OVER PEA LINER TO SUPPORT NATURAL RE-VEGETATION OF GRASSES AND TREE SPECIES**

**COMMON FILL COMPACTED IN 600mm LIFTS (PROPOSED)**

**500mm TOP SOIL LAYER**

**2m 50mm COARSE SAND FRICTION LAYER**

**500mm TOP SOIL LAYER**

**EXISTING 40mil SMOOTH LLDPE LINER TO BE CAREFULLY REMOVED FROM PEA CREST FOR SMA SOIL APPLICATION**

**EXISTING 40mil SMOOTH LLDPE LINER TO BE REPLACED ON CREST AREA AFTER SOIL RE-LOCATION FROM SMA**

**SMA RE-LAID TO SUPPORT NATURAL RE-VEGETATION OF GRASSES AND TREE SPECIES**

**NATIVE RE-VEGETATION**

**500mm TOP SOIL LAYER**

**200mm 5 to 25 mm CLEAR CRUSH DRAINAGE GRAVEL**

**50mm COARSE SAND FRICTION LAYER**

**12 oz NON-WOVEN GEOTEXTILE**

**8 oz NON-WOVEN GEOTEXTILE**

**PROPOSED 8oz NON-WOVEN GEOTEXTILE**

**ENCAPSULATED EXISTING WASTE SOIL**

**500mm TOP SOIL LAYER**

**EXISTING 40mil SMOOTH LLDPE LINER TO BE REPLACED ON CREST AREA AFTER SOIL RE-LOCATION FROM SMA**

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**COBBLE HILL LANDFILL**

**UPDATED FINAL CLOSURE PLAN 2019**

---

**COBBLE HILL HOLDINGS LTD.**

**LANDFILL SERVICES GROUP**

**ADDRESS:**

**PHONE:**

**FAX:**

---

**DATE:**

**PROJECT NO.:**

**DESIGNED:**

**DRAWN:**

**CHECKED:**

---

**SPECIFIED & ASSOCIATES**

**PERLING S HANSEN**

**CLIENT:**

**PROJECT:**

**FIGURE 4-3**
TOE DRAINAGE SOAK AWAY TRENCH DETAIL. SEE FIGURE 4-6.
PEA TOE DRAINAGE SOAK AWAY TRENCH

SCALE 1:20

EXISTING 40mil SMOOTH LLDPE LINER

COMMON FILL COMPACTED IN 600mm LIFTS (PROPOSED)

ENCAPSULATED EXISTING WASTE SOIL

50mm COARSE SAND FRICTION LAYER

12 oz NON-WOVEN GEOTEXTILE

8 oz NON-WOVEN GEOTEXTILE

200mm 5 to 25 mm CLEAR CRUSH DRAINAGE GRAVEL

EXISTING GROUND

EXISTING PRIMARY BASAL LINER

EXISTING SECONDARY CLAY BASAL LINER

EXISTING LEACHATE/LEAK DETECTION COLLECTION PIPING

EXISTING SAND CUSHION LAYER

EXISTING SAND CUSHION LAYER

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

EXISTING PIPING

EXISTING GROUND

EXISTING 40mil SMOOTH LLDPE LINER

EXISTING 40mil SMOOTH LLDPE LINER

EXISTING PRIMARY BASAL LINER

EXISTING PRIMARY BASAL LINER

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)
SOIL STABILIZATION WEDGE FILL (PROPOSED)

- Encapsulated existing waste soil
- Approximate bedrock surface
- Existing leachate collection layer
- Existing primary basal liner
- Existing leak detection layer
- Existing secondary clay basal liner

**PROPOSED**

- 12oz non-woven geotextile
- 8oz non-woven geotextile
- Drainage tube
- Seepage blanket well to be extended
- Common fill compacted in 600mm lifts (proposed)
- Existing seepage blanket layer (fractured bedrock)
- Surface water toe ditch (typ.)
- Existing leachate collection layer
- Existing primary basal liner
- Existing leak detection layer
- Existing secondary clay basal liner

**EXISTING**

- Existing piping
- Existing ground
- Existing 40mil LLDPE smooth liner
- Approximate bedrock surface
- Proposed piping
- Proposed 12oz non-woven geotextile
- Proposed 8oz non-woven geotextile
- Proposed drainage tube
- Surface water toe ditch (typ.)
- Existing leachate collection layer
- Existing primary basal liner
- Existing leak detection layer
- Existing secondary clay basal liner

**TYPICAL SLOPE CLOSURE SYSTEM (PROPOSED). SEE FIGURE 4-2.**

**FIGURE 4-7**

**COBBLE HILL LANDFILL**

**UPDATED FINAL CLOSURE PLAN**

**2019**
5. EROSION CONTROL

This Chapter discusses the Erosion Control Plan for the CHL the erosion control measures that are proposed are hydroseeding immediately after placing the final cover layer, straw wattles, erosion control blanket on the slopes, and rip rap. The main feature of the storm water management system will be the storm water retention / settling pond. By attenuating peak flows from the closed PEA, downstream environments will be protected from scouring. Furthermore, retention of clean storm water in the settlement pond will allow for suspended solids to settle out. The main objective with the erosion control plan is to prevent soil migration in the first place.

5.1 Hydroseeding Plan

In order to protect the cover system from erosion during the first winter after construction, a strict monitoring and maintenance program to control erosion should be implemented. The most important element when safeguarding against erosion is to establish vegetation as early in the year as possible, giving the design seed mixture time to develop and establish itself. Hydraulic seeding can be applied to closure areas excluding armored ditches and roadways directly after the topsoil layer has been installed. A system comprised of both permanent and temporary measures such as erosion control blankets, straw bales and straw wattles is envisioned, with some elements being removed after the first winter and others remaining in place.
5.2 **Straw Wattle Ditch Protection**

Straw wattles are an effective and economical alternative to silt fence for sediment control and storm water runoff. Compared to the silt fence the straw wattles allow the water to slowly percolate through the wattles, slowing down the velocity and trapping sediment. They work by slowing down the velocity of rain runoff and help to prevent rill and gully slope erosion by holding bare soil in place and trapping sediment. Comprised of cylinders of recycled, compressed, 100% agricultural straw, straw wattles are wrapped in U.V. stabilized black polyethylene netting and can last up to two years allowing the design seed mixture time to develop and establish itself. Straw wattles are installed by staking in place, and can be used individually or tied together to achieve any desired length. Photo 8-2 below, illustrates the usage of straw wattles and erosion control matting at the Vancouver Landfill. SHA has also begun using straw bales, temporarily, on the inlet and outlet structures of culverts and downchutes which does a very effective job of collecting the fine particles suspended in the collected surface water. The use of straw bales and straw wattles are examples of temporary erosion control measures which can be removed once the vegetation layer is well established.

![Photo 5-2 Erosion Control Measures on Phase 2 at Vancouver Landfill](image)

5.3 **Straw Slope Protection**

Loose straw application on slopes is an effective temporary solution to prevent rilling and other erosion while vegetation is being established. An example of loose straw applied on a slope is shown in Photo 5-3 at the DSS Landfill in Delta. The slopes are now well vegetated during the second winter post-
construction and the straw has degraded. It is not recommended to use composted yard waste for this application as it is too dense and would prevent germination of the grass seeds. Straw application for slope protection.

![Photo 5-3: Loose Straw Application on DSS Landfill in Delta.](image)

### 5.4 Erosion Control in Ditches

A well-established grass cover will be effective in preventing uncontrolled erosion. In order to keep the ditch clean and free of sediments during construction and before grass is established, straw wattles should be installed on the upslope side of all ditches.

For the surface water conveyance system, there is a need to allow the drainage layer to efficiently drain into the ditch without pore pressure build up, therefore a free-draining rip rap armour is recommended.

For this reason, the bottom and side slopes of the ditches and downchutes should be armoured with a 300 mm thick layer of rip rap or similar material. Photo 5-4 presents our vision of how the site ditches should appear once integrated into the final cover system. Rip rap sizing is critical to ensure stability. Typically, 150 to 300 mm rip rap is sufficient for low gradient ditches while 300 to 600 mm rip rap is generally required for down-chutes. Rip rap sizing should be confirmed during detailed design.
In some ditches, an erosion control blanket can be used instead of the rip rap where the velocity of the surface water flow will be minimal. An example of erosion control matting in place of rip rap is shown in Photo 5-5.
6. CONCEPTUAL MODEL

SHA was requested to prepare a hydrogeological conceptual model of the CHL at the request of the MOE, to present and serve as a guide for formulating the future environmental monitoring program at the Landfill. This chapter includes a summary of the Conceptual Hydrogeologic Model included in Appendix D.

6.1 Regional and Local Surficial and Bedrock Geology

The Landfill is underlain by Wark Colquitz gneiss bedrock from the lower Paleozoic Period (295-384 million years) and comprises of deep crustal sequence of interfingering dioritic intrusions that formed at the root of the Bonaza arc, collectively comprising the West Coast Crystalline complex (Canil et al., 2010).

As part of the environmental monitoring program requirements outlined by the MOE several monitoring wells and seepage blanket wells were drilled and completed at the Landfill, as shown in Figure 6-1. A list of the monitoring wells and seepage wells are provided below along with a geologic description. Three rock types have been identified at the Landfill and include a medium to coarse grained dark green gabbro, a medium to fine grained/medium to dark green diorite and a pale green, fine grained diorite.

- MW-01 [S 51 mbg/D 84 mbg]: Interbedded green gabbro and dark green diorite approximately 25 to 30 metres thick. Fracture was observed at 81 mbg, and had a 3 gallon per minute yield.
- MW-02 [S 43 mbg]: Interbedded green gabbro and dark green diorite approximately 15 to 30 metres thick. Fracture was observed from 36 to 42 mbg, but generated little water.
- MW-03 [S 22 mbg/46 mbg]: Interbedded green gabbro and dark green diorite approximately 5 to 20 metres thick. Fracture was observed from 17 to 18 mbg, but generated little water.
- MW-13-04 [41 mbg]: Interbedded green gabbro and dark green diorite approximately 5 to 30 metres thick. Fracture was observed from 7 to 7.3 mbg, but generated little water.
- MW-13-05 [41 mbg]: Interbedded green gabbro and dark green diorite approximately 3 to 25 metres thick. No fractures observed.
- SB-01 [4.0 mbg]: Green gabbro.
- SB-02 [3.3 mbg]: Green gabbro.
- SB-03 [3.5 mbg]: Green gabbro.
- MW-04: Wark, gneiss-granodiorite.
- MW-05: Wark, gneiss-granodiorite.
- MW-06: Green diorite, fracture at 37.8 mbg.
The Wark Colquitz gneiss bedrock was studied by Hancock (2012) where Hancock examined the
ductures in the exposed rock at the quarry site noted that there were three types, tight, filled, and veins.

Hancock completed a geologic study of the South Island Aggregates Quarry in 2012 and Hancock did
not observe any geologic faults underlying the Landfill. Two regional faults exist and are located 3
km southwest and 6 km northwest of Landfill (Hancock 2012, and Pye and Kneale 2012).

6.2 Hydrogeological Conceptual Model

As discussed in the previous section the Landfill is underlain by a thin stratigraphic unit of quaternary
cover and is less than 2 metres thick which underlain by thick bedrock comprised of a gabbro/dioritic
gneiss. The bedrock is very competent and hard from surface to approximately 75 mbg. As such only
a few fractures have been observed during drilling, rendering the deep rockmass nearly impervious.
The fractures within the bedrock do not have the frequency of open interconnected fractures needed
to convey significant quantities of groundwater flow. As such, the intact bedrock down to a depth of
75 m is extremely unlikely to provide sufficient groundwater yield for drinking water supply wells
because of the limited ability to transmit groundwater flow. The upper bedrock at the Landfill provides
a 65 to 75 m confining layer of lower permeability rock that will act to help protect the deep bedrock
aquiifer. The bedrock aquitard is bound in the east by faults and the Finlayson Arm of the Saanich
Inlet.

A packer test was performed on two monitoring wells drilled and completed within the rock quarry in
proximity to the Landfill. A fracture was observed at 19 mbg in MW13-05 and 34 mbg in MW13-04,
and had a measured hydraulic conductivity of 1.1 x 10^-7 m/s, and 8.1 x 10^-8 m/s, respectively. The
relative scale of hydraulic conductivities and permeabilities ranges from 10^5 m/s for gravels to 10^-8
m/s for unfractured metamorphic and igneous rocks respectively. The hydraulic conductivity of the
intact bedrock underlying the Landfill when compared to the relative scale is extremely low.
Groundwater flow in the bedrock is dependent exclusively upon the availability, connectivity, and size
of the aperture of the fractures in the rock mass. It has been concluded by Active Earth that the geology
and hydrogeology of the Landfill indicate that the shallow bedrock does not readily transmit the flow
of water. Recharge from precipitation is predominantly conveyed downslope as surface water runoff
and as shallow groundwater flow in the overburden horizon, where present, and in the fractured
rockmass within the blasted Seepage Layer.

Below 75 m depth, the bedrock is more fractured and provides higher groundwater yields. The onsite
well which is developed to a depth of 94 m produces up to 20 GPM, with the first water bearing zone
encountered at 78.6 m. Given there are very few fractures noted in the 75 m of competent bedrock
between the water bearing formation at depth, and that an upward gradient is present, SHA is of the
opinion that there is very little risk of contamination migrating to the deep water bearing zone from which drinking water is sourced even in the event of a liner failure.

An average Darcy linear velocity for the deep bedrock formation at the Landfill was reported by Active Earth at a rate of 0.001 m/year. This rate would suggest an effective travel time of 3 million years for recharge water from the Landfill to reach Shawnigan Lake; and the vertical travel time was reported to travel from the bedrock to the deeper bedrock is greater than 100,000 years. However, the actual advective velocity of water flowing within the few fractures that are present is likely to be much faster.

The vertical hydraulic gradient was calculated for the Landfill at nested monitoring wells MW-01 shallow and deep. The vertical hydraulic gradient at MW-01S/D is -0.02 suggesting the deep aquifer has a strong upward vertical gradient. This condition is very protective of the deep water bearing formation provided the recharge zone further upgradient is free industrial activity and contamination.

Groundwater monitoring wells were drilled and completed within the competent bedrock, a summary of the completion details including the most recent groundwater elevations is provided in Table 6-1. The groundwater elevations range from 319.1 masl to 342.5 masl in the shallow bedrock and range from 319.1 masl to 331.2 masl in the deep bedrock. Figure 6-3 illustrates a time series plot of the groundwater elevations from November 2010 to January 2019. The potentiometric elevations of the monitoring wells follow a similar trend suggesting that the network of fractures of both the shallow and the deep system responds to seasonal recharge fluctuations, and that some interconnectivity of the fracture networks may exist. The potentiometric elevations within the bedrock wells are at their highest in winter months and lowest in the summer months suggesting recharge of the aquitard occurs in the wetter winter months.

Figure 6-4 illustrates the groundwater flow conceptual flow map for the Landfill and Figure 6-2 illustrates a hydrogeologic cross section showing the groundwater elevations and predominant flow directions. There is a groundwater flow divide located east of the Landfill and separates groundwater flowing into Shawnigan Creek and its tributaries and groundwater flowing to the Landfill.

Groundwater within the region of the encapsulation area will travel in a north easterly direction, in addition to groundwater north of the encapsulation area. Groundwater east of the encapsulation area will travel east to Shawnigan Creek and its tributaries. Groundwater hydraulic gradients were calculated at the Landfill: for a change in distance over a metre approximately a 0.04 m change in hydraulic head was observed, yielding a gradient of 4%.

There is limited hydrogeologic data available regarding the deep regional bedrock water bearing formation which is believed to be present from greater than 300 mbg in proximity to the Landfill. There are several groundwater wells drilled and completed in proximity to the Landfill and have yields
of approximately 2 to 20 US GPM. The highest water well yields were observed in wells that were screened within the limestone bedrock at depth.
Groundwater Elevations Over Time

PROJECT: COBBLE HILL LANDFILL
UPDATED FINAL CLOSURE PLAN 2019

TITLE: GROUNDWATER ELEVATIONS OVER TIME

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**Notes:**
Ground elevations measured from topographic survey.
7. LEACHATE MANAGEMENT

7.1 Introduction

In this chapter, the leachate management strategy will be outlined. The objective of this chapter is to provide a detailed guide for construction of an effective collection and storage system to ensure minimal risk to the receiving environment upon closure of the CHL.

As requested by MOE, the following concerns have been reviewed and addressed:

- Leachate generation potential including projected quantities and quality
- Landfill Liner Strategy – outlined in Chapter 3
- Leachate Collection System – outlined in Chapter 3
- Leachate Storage and Transportation Planning
- Tracking of Leachate Generation and Quality Post Closure

7.2 Leachate Management Strategy

The current definition of contact water at the Cobble Hill Site, as per WSP’s Review of Contact and Non-Contact Water Management Systems, includes: leachate from the PEA; run-off from the PEA; drainage from the SMA; drainage from the vehicle wheel wash area and backwash from the contact water treatment system.

The wheel wash facility has been decommissioned since cancellation of the waste discharge permit at the site. Clean water runoff from the PEA is handled in the settling pond, as outlined in Chapter 6. As discussed in Chapter 3 - Soil Relocation and Encapsulation; following relocation of soil from the SMA to the PEA, the SMA will be decontaminated and decommissioned, thus eliminating the area as a source of contact water. Therefore, the main source of contact water (leachate) will be the PEA.

The primary strategy for minimizing generation of contact water at the site is to close the PEA with a final cover system. Currently, the PEA has been capped with 40mil smooth LLDPE geomembrane since the Fall of 2016. This has already minimized leachate generation as a result of effective diversion of precipitation and complete elimination of rainwater infiltration. In addition, SHA recommends the following strategies for managing leachate:

- During soil relocation and closure operations, the use of poly tarps and temporary covers will be used as necessary to aid in minimizing clean water contact with contaminated soil. All
closure construction works should be completed during dry weather to ensure no additional precipitation is introduced into the PEA.

- Clean run-on from above the PEA should continue to be managed by way of the existing run-on diversion ditching around the perimeter of the PEA and directed offsite or to the existing settling pond in the northwest corner of the site.
- Clean run-on originating from areas outside the quarry footprint should be diverted into natural water courses around the quarry perimeter.

The current leachate management system in which generated leachate collected from within the PEA, as well as the leak detection system, is collected, conveyed and stored onsite was constructed in the Fall of 2017 as designed per the 2017 Minor Construction Works, approved by MOE. As outlined in Figure 7-1, all generated leachate from within the PEA is collected and conveyed in 100mm HDPE DR17 pipe (at a minimum design grade of 2%) to the storage facility consisting of two 10,000-gallon tanks for leachate and one 10,000-gal tank for leak detection. All three tanks currently reside in a covered facility which includes a secondary containment liner, as presented in Figure 7-2. Generated leachate stored in the aforementioned storage facility is removed from the storage tanks using a vac-truck, and transported offsite to an approved liquid waste acceptance facility (discussed in Section 7.8.1 below).

Of note, since the construction of this collection and storage system, no leachate has been generated by way of the leak detection system, proving the existing leachate collection system below the primary basal liner of the PEA is operating as designed.

Since capping the PEA in Fall of 2016, contact water (leachate) generation at the site has significantly decreased from approximately 3.8 – 5.7 m$^3$/day (prior to capping) to approximately 0.8 m$^3$/day in February 2017 to approximately 0.3 m$^3$/day in December 2018. With time, leachate generation will continue to trend downward. Additionally, HELP modelling of leachate generation estimates the average annual leachate production will be reduced to approximately 0.00033 m$^3$/day (0.12 m$^3$/year) after the final cover system is constructed. The profiles used for the crest and slope final cover system are outlined in Table 7-1 and the HELP Leachate Generation Results are outlined in Table 7-2.

Figure 7-3 illustrates the results of leachate tank monitoring data collected by CHH as part of routine monitoring at the site; as shown, the volume of leachate collected has trended downward over the past 14 months. Please note the total values shown for January 2018, May 2018, and September 2018 have been estimated, due to the leachate being removed from the tank between recording periods. It should be noted that there may be a slight increase in leachate captured during and directly after the proposed closure due to the further consolidation of the contaminated soils in the PEA due to additional surcharge from final cover system materials being applied. However, once the final cover system is in place the volume of leachate generated is expected to decrease.
As can be seen in Figure 7-3, the rate of leachate discharge is trending downward, indicating that the final capping system is effective in preventing entry of precipitation into the PEA.

![Monthly Volume of Leachate Collected at Cobble Hill Landfill](chart.png)

**Note:** Leachate levels are generally monitored on the 15th and 30th of each month; or the next available business day.

*January, May, September 2018 Values Estimated to Reflect Leachate Removal Events During that Period. (Leachate removed from tank).

**Figure 7-3: Leachate Generation and Capture Observed Monthly Volumes**

### 7.3 Leachate Management Design Contingency

If the seepage blanket monitoring wells and/or leak detection system indicate the basal liner is failing and leachate is being released into the environment, SHA has conceptualized a Leachate Contingency Ditch / Trench at the downgradient toe of the landfill as shown in Figure 7-4. This ditch would be blasted / excavated into competent bedrock. The ditch/trench would drain, on grade, to a sump located...
at the low point of the collection system and all collected leachate would be pumped to the existing onsite leachate storage facility.

The Leachate Contingency Ditch / Trench is proposed in an east-west direction, approximately 100m in length, draining from east to west at a minimum 2% grade along the north toe of the proposed 5H:1V stabilizing soil wedge of the PEA closure system to a centralized sump. The trench would be excavated through the approximate 2m deep seepage blanket layer to competent bedrock and further blasted or rock hammered to make certain the trench invert is at least 500mm below the upper extent of the bedrock layer, similar to the previously developed Seepage Blanket Wells. The sidewalls of the trench should be grades at approximately 1.5H:1V with small safety berms remaining on either side of the trench, as outlined in Figure 7-5.

If fractures in the bedrock are encountered along the invert of the Leachate Contingency Ditch / Trench alignment or near the centralized sump, they should be sealed with fibrous shotcrete. The centralized sump should be further blasted / hammered to create a sump capable of housing a sump pump operated on a level indicator, discharging all collected leachate to the onsite storage facility.

If a trench crossing is required for vehicular access, a large diameter culvert (min 500mm HDPE) should be placed in the trench/ditch invert and backfilled with clean shot rock. A geotextile should then be placed over the shot rock prior to road mulch application.

Contingency measures are further discussed in Chapter 10 of this report.

7.4 Leachate Generation

The leachate generation estimation was performed using the Hydrologic Evaluation of Landfill Performance (HELP) model. HELP is a quasi-two-dimensional hydrologic model of water movement across, into, through and out of landfills. The model accepts weather, soil and design data and uses solution techniques that account for the effect of surface storage, snowmelt, runoff, infiltration, evapotranspiration, vegetation growth, soil moisture storage, lateral subsurface drainage, leachate recirculation, unsaturated vertical drainage, and leakage through liners.

HELP modelling of leachate production with the recommended cover system, documented in Chapter 4, forecasts an average annual leachate production rate of 0 mm/year on the crest area and 0.01 mm/year on the side slope area, translating to leachate production rates of 0 m³/year and 0.12 m³/year for the approximate 1.2 Ha site, respectively. Hence, very little leachate is expected from the PEA once the soils in the cell fully consolidate.
Based on a 200-year wet winter weather forecast with monthly rainfall intensity of 1.5 times the average and including snow melt and multi-day precipitation, an annual leachate production rate on the crest area of 0 mm/year and the slope area of 0.01 mm/year or 0 m³/year and 0.12 m³/year for the 1.2 Ha site, respectively, is predicted as a worst-case leachate generation forecast after closure. SHA expects that over time, leachate production will decline as the soil consolidation process is completed. Also, a thorough review and inspection program will be undertaken of the entire existing 40mil LLDPE geomembrane cap covering the PEA prior to final closure construction to make certain that all seams, boots and minor holes identified during the last inspection have been properly sealed up and that any new deficiencies are repaired.

Overall, we anticipate that leachate currently being generated at approximately 0.3 m³/day should reduce to approximately 0.00033 m³/day or 0.12 m³/year, post closure.

7.5 Leachate Quality

A review of contact water quality data representing the site’s containment pond and landfill leachate collected over the past several years, dating back to 2016, was conducted by SHA. Parameters that exceeded the B.C. aquatic life standards (BCAWWQG) from time to time included colour, total suspended solids, turbidity, sulfate, arsenic, copper, iron, pyroxene and benzo (a) pyroxene. The same parameters are often elevated in municipal leachate and in run-off from disturbed areas.

Since 2017, leachate quality samples have been collected from LE-1; which represent raw leachate sourced from the PEA. The leachate collected demonstrates elevated levels of Chloride, Sulfate, Chronic Cobalt, Chronic Manganese, and on occasion, pH and Turbidity.

The amount of leachate generated is very small. Leachate is treated onsite, post sampling, to reduce certain constituents, for offsite licensed discharge on Vancouver Island. The leachate is collected passively through the PEA collection system and conveyance network to the storage facility prior to transportation off-site to a licensed waste water treatment facility. Collected and stored leachate volumes are being tracked and recorded as part of the ongoing monitoring and inspection program.

7.6 Leachate Collection and Management

The existing PEA has been constructed to effectively collect leachate via the sand drainage layers on top of 40mil LLDPE geomembrane basal liner graded at approximately 2% from south to north with 100mm perforated PVC SDR 28 collection piping along the downgradient toe (north side) of the cell. The PEA is also equipped with a leachate leak detection system below the basal liner geomembrane. This system is composed of a sand drainage layer overtop of a compacted clay liner graded at
approximately 2% from south to north with 100mm perforated PVC SDR 28 collection piping along the downgradient toe (north side) of the cell.

As per the updated closure system design presented in this report, approximately 8m of additional soil will be placed overtop of the toe of the PEA and against the north slope; this will result in additional loading on top of the infrastructure. To make certain that all existing infrastructure within the PEA, including the above-mentioned collection systems, will maintain functional integrity once incorporated into the proposed final cover system, SHA completed a review and calculation check on the existing 100mm PVC SDR28 and proposed 100mm HDPE DR17 collection pipes using applicable versions of the Modified Iowa Equation. Assuming the pipes are constructed using coarse-grained soils with moderate compaction (85-95% Proctor Density.), deflection estimates of 3-5% were obtained. This suggests that the additional proposed loads will not deflect the existing PVC SDR28 and proposed HDPE DR17 beyond acceptable limits (7.5% is typically the recommended maximum allowable deflection for non-pressure applications).

7.6.1 Summary of 2017 Minor Construction Works for the Leachate / Leak Detection Collection, Conveyance Systems and Storage Facility

2017 Minor construction works included installation of new twin pipe conveyance system from existing PEA collection pipes to a new storage facility located approximately 60m north of the PEA. The following tasks were completed and inspected by the QP’s throughout construction:

7.6.2 Leachate/Leak Detection Conveyance System:
- Excavation for the twin pipe trench was completed with adequate grade, depth and width to accommodate the new conveyance piping;
- A combination of 5mm-12.5mm clear round drainage gravel and well graded sand was used for pipe bedding, pipe surround and as backfill material. Samples of both materials were sent to the laboratory for gradation analysis with acceptable results. Onsite QP inspected both materials onsite prior to use and approved both products;
- 2x conveyance piping was installed on approximate 2% grade (south to north) and was verified in the field with a laser level under QP supervision. As-built survey records will confirm all inverts and pipe grades once data has been processed;
- QP confirmed specifications for conveyance piping, warning tape, pipe bedding material and onsite fill;
- Completed pipe trench cross section includes 300mm of pipe bedding material, 2x100mm solid HDPE DR17 pipes, overlain by 250-300mm of pipe bedding material (mechanically compacted), warning / tracer tape, overlain by a minimum of 300mm of well graded shot rock;
- A valve was included at mid-span between the PEA and storage facility on the leachate conveyance pipe;
• New HDPE conveyance piping was tied in to PEA leachate collection pipe with a ‘Fernco coupling’ and to the leak detection collection pipe with a PVC to HDPE threaded connection. QP inspected and approved all connections; and
• Cleanouts were not installed on the conveyance lines as adequate access to the twin pipes is available at the tank connection location. However, two leachate collection pipe cleanouts and two leak detection pipe cleanouts are proposed at the east and west ends of both systems. This is discussed further below in Section 7.7.

7.6.3 Leachate/Leak Detection Storage Facility:
• Leak Detection Storage Tank – one 10,000-gallon HDPE Plastic Tank excavated into existing ground to ensure gravity drainage from the PEA and to provide full use / storage capabilities of the tank volume.
• Leachate Storage Tank – two 10,000-gallon HDPE Plastic Tanks excavated into existing ground to ensure gravity drainage from the PEA and to provide full use / storage capabilities of the tank volume.

To ensure secondary containment in the event of a leak from the aforementioned tanks, the subsurface tanks were constructed within a lined lock block basin constructed during the 2017 minor construction works program. The new leachate / leak detection storage facility is approximately 60m north of the PEA. The following tasks were inspected by the QP (SHA & Islander Engineering) throughout construction:
• All excavations were completed in open cuts;
• Allowance for surface water drainage around the lock block basin was provided by way of clear drainage backfill around the base and side slopes overlain with a geotextile prior to backfill at grade. QP inspected granular subgrade prior to lock block placement;
• QP inspected granular subgrade prior to installation of heavy weight non-woven geotextile cushion layer
• QP inspected 40 mil LLDPE geomembrane secondary containment liner prior to installation and once installed. Secondary liner was secured to the lock block storage facility along the top of the 4th row of blocks, allowing adequate containment and storage if tanks were to breach;
• QP inspected the installation of the non-woven geotextile layer underneath the storage tanks;
• QP inspected tank installation work and tie-in piping connections;
• QP inspected roof structure which includes wooden joists overlain by a poly liner. Poly roof liner extends over the roof onto each exterior wall approximately 1m down and is secured to a 2x4” with ‘hilti-bolts’ into the concrete lock blocks;
• High level alarm has been installed in the leachate storage facility and is operational. Works included installation of a solar panel, control panel, transducer, floats and fully automated high
and low-level alarm system for leachate tanks. Smartphone application was downloaded by the Electrical Contractor during install and functionality of the remote system was explained, tested and verified by the QP. With the smartphone system, levels in the tanks can be monitored and floats and alarms adjusted. The smartphone application has been downloaded by CHH and will be used to manage leachate levels with the storage facility; and,

- Interconnecting hose (4” flex hose) between the two leachate storage tanks was installed. Connections were inspected and verified by QP.

### 7.7 Leachate Management System – Proposed Works for Final Closure

Based on the proposed closure system design presented in Chapter 4, additional leachate and leak detection infrastructure will be required during closure construction. These new works include clean-out access to the existing leachate and leak detection systems currently existing within the PEA, at the north toe. The proposed soil stabilizing soil wedge will be constructed at approximately 5H:1V grade over the existing north and east 2.5H:1V side slopes. To ensure access to the existing system is maintained, SHA’s design calls for four (4) new cleanouts; two for both the leachate and leak detection collection system at both the west and east terminus. The recommended locations of these cleanouts are presented in plan view in Figure 7-1.

Two typical sections illustrate the proposed works. Figure 7-6 shows a cross section of the leachate clean-out extension to be installed along the existing 2.5H:1V slope and Figure 7-7 shows the blind flange infrastructure to be installed at the end of the clean-out. SHA recommends that the cleanout pipe extensions be installed, including all pipe and liner boot welding, prior to placement of stabilizing soil wedge to allow for full QP supervision and liner system QAQC. Figure 7-8 provides details to tie the clean-out into the leachate collector/leak detection system. SHA recommends all HDPE pipe fitting joints and bends are completed with either 22 or 45 deg elbows, as required, to allow for downhole camera scoping in the future if a situation arises. Downhole camera scoping may become warranted if subsurface piping and infrastructure are compromised during construction, based on the opinion of the QP onsite.

Similarly, the two existing Seepage Blanket wells (SB-2 and SB-3) will need to be extended vertically to allow for safe access for future monitoring once the stabilizing soil wedge has been constructed as proposed in Chapter 4. SB-1 will not need to be extended as its current location is north of the proposed closure works. Figure 7-6 and 7-7 also provide details of the riser that will be attached to the existing well infrastructure.

During common fill placement around all existing and proposed leachate and monitoring infrastructure, construction equipment should maintain a minimum 1.0 m distance at all times to
ensure protection. Soil should be placed and compacted by hand or by dedicated small equipment under QP supervision when backfilling near piping infrastructure.

### 7.8 Leachate Generation Monitoring

In order to track the on-site water balance, both the leachate collection system and the leak detection system will be monitored for actual production at the leachate storage facility. The monitoring will involve measuring and recording the leachate and leak detection tank volumes prior to leachate removal and transfer off-site.

Inspection of the leachate tank will occur quarterly and/or at a greater frequency as required. It is recommended that the leachate storage facility is inspected quarterly, with leachate levels documented during each inspection.

It is also recommended that the leachate storage facility is inspected following a high intensity rain event, including potential snow melt (1:10 year event) to ensure the final cover system of the PEA is operating as designed and all precipitation is being shed off the final surface and not infiltrating the PEA, generating further leachate.

During specific closure construction activities, SHA proposes that leachate generation be monitored on a daily basis at the leachate storage facility (within the leachate and leak detection tanks) to make certain that leachate collection / leak detection piping is not compromised. Daily volumes of collected leachate should be measured and recorded to allow for tracking and verification that daily volumes match the existing generation trends. If daily leachate collection volumes cease or are reduced drastically during construction, this may be a sign that subsurface collection / conveyance infrastructure has been compromised and further investigation measures may be warranted. Additional investigative measures could include down hole camera / scoping works or other measures as required by QP.

Daily leachate volume monitoring should commence only when common soil fill placement activities are directly adjacent to or overtop of collection and conveyance piping and should continue as directed by the onsite QP.

#### 7.8.1 Leachate Transportation Off Site

Currently all leachate generated within the PEA and collection via the leachate collection system and leak detection system and stored onsite in the leachate storage facility is being transported offsite to a licensed disposal facility.
SHA’s understanding is leachate is currently being generated at a rate of approximately 0.5 m³/day (approx. 80 gal/day), based on the previous year of monitoring. At the current generation rate of 80 gallons per day, it is estimated the tandem leachate collection tank holds enough capacity for approximately 250 days (at 20,000 gallons combined). The existing trigger for the alarm at the site is set for 3 metres of depth in the tank which is approximately 7,925 gallons (or 30m³) per tank (15,850 gallons combined). Once leachate in the tank reaches this trigger level, the tank is emptied via tanker truck. SHA believes this trigger level allows for sufficient time to arrange for removal and transport off-site; at this level, the tanks allow for an additional 4,000 gallons of capacity, which works out to approximately 50 days assuming leachate is generated at 80 gallons per day (at the current rate).

Post closure, leachate generation will be further reduced, as outlined above. Although not right away, leachate generation is expected to reduce to approximately 0.01 m³/month. Based on post closure generation projections and the available capacity, the tanks will need to be pumped, transported and disposed of less than once a year. In the short term, an increase in leachate production rates is expected due to consolidation induced by the stabilizing soil wedge.

SHA estimates that once closure is completed and consolidation runs its course, the primary and secondary liner system, along with evapotranspiration from established vegetation will reduce the frequency of servicing the leachate storage tanks.

### 7.9 Clean Run-off Treatment Strategy

Clean run-off from the closed PEA as well as run-off from the rock quarry will be directed to the clean water settlement pond. There, any suspended sediments will be reduced through retention.

The rate of seepage into the blanket from the rock below is not sufficient to cause any appreciable groundwater mounding within the drainage blanket, such that it may contact the encapsulated soil cells above. The clay base liner also immediately overlies the Seepage Blanket. This limits the potential for impacts to the Seepage Blanket water quality from overlying permanent encapsulation cells. This is further discussed in Chapter 6 – Hydrogeology Review.

Groundwater seepage from beneath the basal liner system will be allowed to flow naturally beneath the base of the quarry and make its way to the existing onsite surface water settling pond.

The PEA will be fully closed off to the environment rendering all precipitation, run-on, run-off and any potential groundwater seepage below the PEA as clean water. These clean flows will all report to the settling pond for retention and discharge. Sampling and monitoring of the site’s surface water discharge point is outlined further on in Chapter 10.
LEACHATE/LEAK DETECTION CONTINGENCY TRENCH/DITCH

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

EXISTING GROUND

APPROXIMATE BEDROCK SURFACE

CURRENT EXISTING GROUND

ROCK HAMMER TRENCH
MINIMUM 500mm INTO
COMPETENT BEDROCK

NOTE: MINIMUM 2% GRADE ON DITCH
INVERT TO CENTRAL SUMP

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

COMPETENT BEDROCK

1:50

NOTE: MINIMUM 2% GRADE ON DITCH
INVERT TO CENTRAL SUMP

EXISTING GROUND

APPROXIMATE BEDROCK SURFACE

CURRENT EXISTING GROUND

ROCK HAMMER TRENCH
MINIMUM 500mm INTO
COMPETENT BEDROCK

NOTE: MINIMUM 2% GRADE ON DITCH
INVERT TO CENTRAL SUMP

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

COMPETENT BEDROCK

1:50

NOTE: MINIMUM 2% GRADE ON DITCH
INVERT TO CENTRAL SUMP

EXISTING GROUND

APPROXIMATE BEDROCK SURFACE

CURRENT EXISTING GROUND

ROCK HAMMER TRENCH
MINIMUM 500mm INTO
COMPETENT BEDROCK

NOTE: MINIMUM 2% GRADE ON DITCH
INVERT TO CENTRAL SUMP

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

EXISTING SEEPAGE BLANKET LAYER (FRACTURED BEDROCK)

COMPETENT BEDROCK

1:50

NOTE: MINIMUM 2% GRADE ON DITCH
INVERT TO CENTRAL SUMP
LEACHATE/LEAK DETECTION AND SEEPAGE BLANKET CLEAN-OUT EXTENSION ALONG NORTH SLOPE OF PEA (PROPOSED)

EXISTING LEACHATE COLLECTION LAYER
EXISTING PRIMARY BASAL LINER
EXISTING LEAK DETECTION LAYER
EXISTING SECONDARY CLAY BASAL LINER

APPROXIMATE BEDROCK SURFACE

EXISTING SEEPAGE BLANKET LAYER
(FRACTURED BEDROCK)

EXISTING PIPING
EXISTING GROUND
EXISTING 40mil LLDPE SMOOTH LINER
APPROXIMATE BEDROCK SURFACE
PROPOSED 12oz NON-WOVEN GEOTEXTILE
PROPOSED 8oz NON-WOVEN GEOTEXTILE
PROPOSED DRAINTUBE

COMMON FILL COMPACTED IN 600mm LIFTS (PROPOSED)

LEACHATE CLEAN-OUT TIE-IN. SEE FIGURE 7-8.

LEACHATE/LEAK DETECTION CLEAN-OUTS. SEE DETAIL ON FIGURE 7-7.

TYPICAL SLOPE CLOSURE SYSTEM (PROPOSED). SEE FIGURE 4-2.

SOLID PIPE EXTENSION ALONG EXISTING PEA SLOPE FOR CLEAN-OUT ACCESS TO LEACHATE/LEAK DETECTION SYSTEM (PROPOSED)

SEEPAGE BLANKET WELL TO BE EXTENDED. SEE DETAIL ON FIGURE 7-7.

100mm HDPE DR17

ENCAPSULATED EXISTING WASTE SOIL

LARGE BOULDERS (TYP.)

EXISTING PRIMARY BASAL LINER
EXISTING SECONDARY CLAY BASAL LINER

FIGURE 7-6
TYPICAL LEAK DETECTION/LEACHATE CLEAN-OUT DETAIL AT SURFACE

TYPICAL SEEPAGE BLANKET WELL CLEAN-OUT DETAIL AT SURFACE
LEACHATE COLLECTION AND LEAK DETECTION
CLEAN-OUT TO BE SPACED MINIMUM 1m APART.

100mm Ø HDPE DR17 SOLID PIPE TO BE WELDED TO EXISTING PVC SDR28 PIPE

12 oz NON-WOVEN GEOTEXTILE

8 oz NON-WOVEN GEOTEXTILE

200mm 5 to 25 mm CLEAR CRUSH DRAINAGE GRAVEL

100mm Ø SOLID HDPE DR17 PIPE

LINER BOOTS DETAIL

SCALE 1:10

LINER BOOTS

SEE "LINER BOOT" DETAIL ON FIGURE 7-8.

SMAKLEX CAULKING

12mm STAINLESS STEEL BAND

HDPE BOOT LINER

LINER BOOT DETAIL

SCALE 1:10

SCALE 1:10

NEOPRENE GASKET

8 oz NON-WOVEN GEOTEXTILE

12 oz NON-WOVEN GEOTEXTILE

COMMON FILL COMPACTED IN 600mm Lifts (PROPOSED)

EXISTING 40mil SMOOTH LLDPE LINER

EXISTING SAND CUSHION LAYER

EXISTING GROUND

EXISTING 40mil LLDPE SMOOTH LINER

EXISTING COMPACTED SECONDARY CLAY BASAL LINER

ENCAPSULATED EXISTING WASTE SOIL

LEACHATE COLLECTION AND LEAK DETECTION
CLEAN-OUT TIE-IN DETAIL

LEACHATE COLLECTION AND LEAK DETECTION
CLEAN-OUT TO BE SPACED MINIMUM 1m APART.

EXISTING 40mil SMOOTH LLDPE LINER

EXISTING SAND CUSHION LAYER

EXISTING GROUND

EXISTING 40mil LLDPE SMOOTH LINER

EXISTING COMPACTED SECONDARY CLAY BASAL LINER

ENCAPSULATED EXISTING WASTE SOIL

LEACHATE COLLECTION AND LEAK DETECTION
CLEAN-OUT TO BE SPACED MINIMUM 1m APART.

EXISTING 40mil SMOOTH LLDPE LINER

EXISTING SAND CUSHION LAYER

EXISTING GROUND

EXISTING 40mil LLDPE SMOOTH LINER

EXISTING COMPACTED SECONDARY CLAY BASAL LINER

ENCAPSULATED EXISTING WASTE SOIL

LEACHATE COLLECTION AND LEAK DETECTION
CLEAN-OUT TO BE SPACED MINIMUM 1m APART.

EXISTING 40mil SMOOTH LLDPE LINER

EXISTING SAND CUSHION LAYER

EXISTING GROUND

EXISTING 40mil LLDPE SMOOTH LINER

EXISTING COMPACTED SECONDARY CLAY BASAL LINER

ENCAPSULATED EXISTING WASTE SOIL

LEACHATE COLLECTION AND LEAK DETECTION
CLEAN-OUT TO BE SPACED MINIMUM 1m APART.

EXISTING 40mil SMOOTH LLDPE LINER

EXISTING SAND CUSHION LAYER

EXISTING GROUND

EXISTING 40mil LLDPE SMOOTH LINER

EXISTING COMPACTED SECONDARY CLAY BASAL LINER

ENCAPSULATED EXISTING WASTE SOIL

LEACHATE COLLECTION AND LEAK DETECTION
CLEAN-OUT TO BE SPACED MINIMUM 1m APART.
<table>
<thead>
<tr>
<th>Layer</th>
<th>Material</th>
<th>Function</th>
<th>Thickness (mm)</th>
<th>K Value (cm/s)</th>
<th>Layer</th>
<th>Material</th>
<th>Function</th>
<th>Thickness (mm)</th>
<th>K Value (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Topsoil</td>
<td>Vertical Percolation Layer</td>
<td>500</td>
<td>1x10⁻⁴</td>
<td>1</td>
<td>Topsoil</td>
<td>Vertical Percolation Layer</td>
<td>500</td>
<td>1x10⁻⁴</td>
</tr>
<tr>
<td></td>
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<td></td>
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<td></td>
<td>2</td>
<td>Subsoil</td>
<td>Vertical Percolation Layer</td>
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<td>1x10⁻⁷</td>
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<td>3</td>
<td>Drainpipes</td>
<td>Drainage Layer</td>
<td>1.32</td>
<td>1x10⁶</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td>4</td>
<td>Geomembrane</td>
<td>Barrier Layer</td>
<td>40 mL</td>
<td>2.0x10⁻¹²</td>
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<tr>
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<td>5</td>
<td>Waste Soil</td>
<td>Vertical Percolation Layer</td>
<td>13,000</td>
<td>1x10⁻¹</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6</td>
<td>Sand Cushion</td>
<td>Vertical Percolation Layer</td>
<td>50</td>
<td>1x10⁻²</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7</td>
<td>Waste Soil</td>
<td>Vertical Percolation Layer</td>
<td>13,000</td>
<td>1x10⁻¹</td>
</tr>
<tr>
<td>Scenarios</td>
<td>Evapotranspiration (mm/yr)</td>
<td>Evapotranspiration as % of P</td>
<td>Runoff (mm/yr)</td>
<td>Runoff as % of P</td>
<td>Percolation to leachate (mm/yr)</td>
<td>Percolation to Leachate % of P</td>
<td>Lateral Drainage (mm/yr)</td>
<td>Lateral Drainage % of P</td>
<td>Change in water storage (mm/yr)</td>
</tr>
<tr>
<td>-----------</td>
<td>---------------------------</td>
<td>-----------------------------</td>
<td>---------------</td>
<td>-----------------</td>
<td>-------------------------------</td>
<td>-------------------------------</td>
<td>-----------------------------</td>
<td>--------------------------</td>
<td>--------------------------------</td>
</tr>
<tr>
<td>Scenario 1 Average</td>
<td>468.8</td>
<td>25.0%</td>
<td>474.8</td>
<td>25.3%</td>
<td>0.01</td>
<td>0.0%</td>
<td>933.57</td>
<td>49.7%</td>
<td>-0.485</td>
</tr>
<tr>
<td>Scenario 2 200 yr</td>
<td>456.1</td>
<td>16.2%</td>
<td>1186</td>
<td>42.1%</td>
<td>0.01</td>
<td>0.0%</td>
<td>1173.6</td>
<td>41.7%</td>
<td>-1.27</td>
</tr>
<tr>
<td>Scenario 1 Average</td>
<td>551.8</td>
<td>29.4%</td>
<td>1295</td>
<td>69.0%</td>
<td>0.00</td>
<td>0.0%</td>
<td>29.94</td>
<td>1.6%</td>
<td>-0.081</td>
</tr>
<tr>
<td>Scenario 2 200 yr</td>
<td>513.8</td>
<td>18.3%</td>
<td>2261</td>
<td>80.3%</td>
<td>0.00</td>
<td>0.0%</td>
<td>40.33</td>
<td>1.4%</td>
<td>-0.11</td>
</tr>
</tbody>
</table>
8. SURFACE WATER MANAGEMENT

The primary objectives of the surface water management plan for the CHL are to:

- Keep clean water clean;
- Prevent erosion of the final cover system;
- Minimize percolation through the top surface of the landfill cover system;
- Prevent ponding of surface water on the cover system;
- Manage suspended sediments;
- Minimize leachate production; and
- Control surface water in a manner compatible with the proposed end-uses.

This chapter reviews the background information on existing conditions and provides recommendations on run-on diversion, ditch sizing, storm water control and runoff controls during landfill closure development and post closure period.

In order to determine the sizing of the toe/road swales/ditches, peak flows were determined using the Rational Method, which is commonly used to determine the peak flow runoff rates in small watersheds. The rationale for the method is that steady uniform rainfall intensity will cause runoff to reach its maximum rate when all parts of a watershed are contributing to the point of outflow. This is dependent on the time of concentration, which is taken as the time for water to flow to the outflow from the most remote point of the watershed. Along with the rainfall intensity and drainage area, which are relatively straightforward to determine, the peak flow is dependent on the runoff coefficient. The runoff coefficient is dependent on the final cover design. It is primarily influenced by topography, vegetation, the seasons and the subsurface material type. The method and coefficients for the analysis were obtained from the BC Agricultural Drainage Manual (1997). This method allows for variations of the material types, the vegetation types and the topography (slope) conditions.

In order to satisfy the requirements outlined in the amended Spill Prevention Order MO:1701, stormwater works were modelled using a 1 in 200-year design storm event with allowances for snowmelt and multi-day precipitation. To do so, four scenarios were calculated, as seen in Table 8-1 below.
### Table 8-1: Rainfall Intensity Calculations.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Description</th>
<th>Intensity</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>mm/day</td>
<td>mm/hr</td>
</tr>
<tr>
<td>Scenario 1</td>
<td>200 years Rainfall</td>
<td>168.7</td>
<td>7.0</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>Multi Day Event</td>
<td>55.08</td>
<td>2.29</td>
</tr>
<tr>
<td>Scenario 3</td>
<td>Peak Daily Plus Snow Melt</td>
<td>223.0</td>
<td>9.3</td>
</tr>
<tr>
<td>Scenario 4</td>
<td>5 minute Tc</td>
<td>1056.0</td>
<td>44.0</td>
</tr>
</tbody>
</table>

Scenario 1: The 200 year 24-hour rainfall amount estimate outlined in the WSP Addendum Review of Contact and Non-Contact Water Management Systems is 168.7mm, based on an extrapolation from the IDF curve.

Scenario 2: SHA retrieved Daily climate data for the Lake Cowichan weather station for the year with the peak amount of precipitation recorded (1997). Multi-Day rainfall events were tabulated through processing the data and summing the total precipitation received over consecutive days. A rain event was considered to start at the first day with precipitation recorded and end at the last consecutive day with precipitation recorded. The maximum multi-day rain event occurred between March 16th to March 20th 1997 where 275.4 mm of total precipitation was recovered over 5 days, resulting in a daily intensity of 55.08 mm/day.

Scenario 3: The maximum total daily precipitation observed in 1997 was 136.8mm on March 17th 1997. The peak snowfall recorded in 1997 was 17.4cm on January 1st. To account for snowmelt, SHA calculated the amount of snowmelt that would occur from 5 days of peak snowfall (17.4cm). A scenario was created in which snowmelt from 5 days of peak snowfall occurred at the same day as peak rainfall (136.8mm). This provided a daily maximum of 223 mm of precipitation/snowmelt on this day. This also correlates well with a 1 in 200-year 24-hour event and 5 cm of snow melt, which is a reasonable worst-case scenario as it is very unlikely that a major snow melt event would occur concurrently with a 1 in 200-year rainfall event.

Scenario 4: SHA extrapolated a rainfall event from the IDF curve, for a time of concentration of 5 minutes. This event provided a rainfall intensity of 44mm per hour, translating into 1,056 mm per day.

As Scenario 4 resulted in the maximum daily precipitation and maximum rainfall intensity, this value was used to size the ditches. This rainfall intensity is greater than 600% that of the 200-year storm intensity.

The calculated flow rate for the above-mentioned extreme event, using the Rational Method, is 0.09 m$^3$/s. SHA’s surface water ditch design has capacity for 4.92 m$^3$/s and the downchute has 13.13 m$^3$/s
capacity. Therefore, all stormwater ditches and downchutes are designed to accommodate for a 1 in 200-year rainfall event, with allowances for snowmelt and multi-day precipitation.

8.1 Drainage Plan for the Cobble Hill Holdings Landfill

The key goal of the surface water management plan for the CHL is to keep clean water clean and minimize leachate production by diverting onsite clean surface water away from potential contact with the landfill cell. The goal is to direct clean water through ditching and overland flow to the Western Settlement Pond prior to discharging offsite to an ephemeral tributary of Shawnigan Creek. The run-on diversion ditches, stormwater routing ditches, and landfill toe ditches will direct water to appropriate areas to minimize - if not eliminate - storm and surface water from potential overland flow onto the lined PEA and ultimately into the armoured ditch network or from entering the leak detection system.

8.1.1 Run-On Diversion

Based on the topography surrounding the site, all surface water which accumulates due to precipitation and that does not fall on the landfill portion of the site will be directed to Shawnigan Creek to the east and south and to the ephemeral tributary of Shawnigan Creek to the north, as shown in Figure 8-1. 2017 Minor Construction Works also included a run-on ditch extension around the south side of the PEA under QP oversight and inspection to divert stormwater to the western run-on ditch. Run-on ditching was completed in existing low permeability soil around the south side of the PEA at positive grades and tied into the existing western run-on ditch.

8.1.2 Storm Water Routing

Upon final closure of the CHL, diversion of clean run-off from the PEA will be required. Figure 8-1 shows the location and orientation of the ditches and downchutes for the closure of the CHL. It is proposed that the crest of the landfill be graded at a minimum 4% grade as per the Criteria.

Surface water run-off from the crest will drain at a minimum grade of 4% through ditches and downchutes. Run-off draining to the north and west will be collected at the toe of the landfill and directed towards the Western Settlement Pond through the clean surface water conveyance ditch shown in Figure 8-1.

8.1.3 Toe/ Road Ditches

Control of erosion in the ditch is a key consideration. A surface water toe ditch has been designed to run along the entire perimeter of the closed landfill which will direct run-off to the settling pond.
The top layer of the ditch will consist of a 300 mm thick layer of rip rap or an erosion control blanket. Figure 8-2 shows the toe ditch with the rip rap material overlying light weight geotextile. It may be possible to replace the rip rap with a less expensive erosion control mat, as shown in Photo 8-1, when ditch grades are less than 5% grade.

SHA recommends ditches with a triangular cross section, 0.75m depth and 2.5H:1V side slopes lined with a 300mm layer of 150mm to 300mm Rip Rap as shown in Figure 8-2. The sizing of this ditch exceeds the requirements for a 1 in 200-year storm event including snowmelt and multi-day precipitation.

8.1.4 Crest Ditch
The crest ditch will collect runoff falling on the closed crest area, and direct it to the toe of the western landfill slope. Based on the run-off analysis using the Rational Method, the crest ditch will need to be triangular in shape and have a total depth of 0.75 m, with side slopes at 2.5H:1V, as seen in Figure 8-2. The crest ditch will be lined with 300 mm of 150-300 mm Rip Rap for erosion control. The sizing of this ditch exceeds the requirements for a 1 in 200-year storm event including snowmelt and multi-day precipitation.

8.1.5 Down Chute
Downchutes will be constructed to convey clean run-off collected from the upper crest area of the landfill and conveyed to the toe of the landfill. Similarly, a downchute is also envisioned at the inlet.
of the settlement pond at the western boundary of the site. Photo 8-2 shows a downchute at the recently completed Delta Shake and Shingle Landfill implemented by SHA.

The downchutes will be trapezoidal in shape and have a total depth of 0.50 m, bottom width of 1.0 m and side slopes at 2.5H:1V. The downchutes will be lined with a heavy weight geotextile, with a 600mm thick 300 to 600 mm Rip Rap layer on top for the full depth of the downchutes as outlined in Figure 8-2.

The Rip Rap layer will ensure that there is no erosion due to the high velocities that will be experienced within the downchutes. Also, it will be important to provide some form of energy dissipation at the bottom of each chute to protect the lower ditch works from scour.
EXISTING WASTE SOIL

MINIMUM 1.7m of subsoil to be placed along ditch alignments to ensure 2.0m of cover over PLA liner

EXISTING 40mil SMOOTH LLDPE GEOMEMBRANE LINER

12 oz NON-WOVEN GEOTEXTILE

500 mm TOPSOIL LAYER

HYDROSEED/VEGETATION

PROPOSED DRAINTUBE

8 oz NON-WOVEN GEOTEXTILE

EXISTING 40mil LLDPE SMOOTH LINER

NOTE: WHERE DITCHES/DOWNCHUTES ARE >5% IN GRADE 300 mm TO 600 mm RIP RAP WILL BE USED IN PLACE OF 150 mm TO 300 RIP RAP.

TYPICAL CREST DITCH SECTION

SCALE 1:50

TYPICAL TOE DITCH SECTION

SCALE 1:50

TYPICAL DOWNCHUTE SECTION

SCALE 1:50

EXISTING ROCK QUARRY SURFACE

500 mm TOPSOIL LAYER

600 mm LAYER OF 300 to 600 mm RIP RAP

EXISTING QUARRY SURFACE

6 oz NON-WOVEN GEOTEXTILE

EXISTING GROUND

600 mm 150 to 300 mm RIP RAP

1.0m

1.0m

1.0m

1000

500

500
9. GEOTECHNICAL CONSIDERATIONS & SLOPE STABILITY

As previously outlined in Chapters 3, 4 and 5, SHA is proposing to leave the existing 40mil LLDPE Smooth Geomembrane Liner on the sloped areas of the PEA, and provide the necessary closure stability by way of a stabilizing soil wedge on the east and north slope of the PEA. A gravel drainage layer has been included at the base of the stabilizing wedge to increase transmissivity of any infiltration surface water away from the primary barrier layer.

This section includes an assessment of stability of the above proposed closure scenario for both static and seismic conditions. The global stability was assessed for Cross Section 3-3’ of Figure 9-1 and considers the weak interface created by the existing smooth geomembrane and coarse sand to be buried within compacted till at a proposed 5H:1V slope.

9.1 Underlying Stratigraphy

The ground surface at the site is an expression of an igneous intrusion of very hard granite bedrock through underlying bedrock known as Wark Gneiss. This hard granite rock, as well as the Wark Gneiss, are the source of materials for the quarry. The groundwater flow regime in the vicinity of the site is predominantly via fractured flow within a deep bedrock aquifer. Overburden soils are not thick enough in the region to develop an overburden aquifer.

9.2 Slope Stability Analysis

9.2.1 Slope Stability Model

Stability of the cross section was modeled using Rocscience’s SLIDE 4.0 © designed for 2-D limit equilibrium methods for soil and rock slopes. The modeled materials, lithology, and section geometry are presented in Figure 9-2 and 9-3. The limit equilibrium analysis was performed using the Bishop Simplified method. Materials strength was modeled the Mohr-Coulomb equation.

Failure scenarios were modeled for both static and seismic (earthquake) loading conditions for the proposed and existing profiles. The following factors of safety (FOS) for slope failure have been adopted as minimum standards:

- Static Conditions adjacent to Developed Land and Infrastructure 1.5
- Seismic (Earthquake) Loading 1.0
A pseudo-static analysis was performed for the seismic loading scenario to assess possible ground movement during an earthquake. The National Building Code of Canada 2012, Volume 2 provides seismic values for a number of locations across Canada. The peak horizontal ground acceleration (PGA) of 0.61 g for the Victoria area was found. This PGA has a probability of exceedance of 2% in 50 years. It is noted that a more accurate peak ground acceleration may be obtained by performing site specific response analysis. This is typically done when a site is very high risk, or foundations are known to be problematic. It is our opinion that neither is the case at CHL.

As recommended by the United States Environmental Protection Agency document “RCRA Subtitle D (258) Seismic Design Guidance for Municipal Waste Landfill Facilities (1995)”, a 50% reduction of the PGA was applied for the analysis based on findings of Hynes and Franklin (1984), who indicated that deformations in embankments should remain below 0.3m when the yield acceleration is less than or equal to 50% of the PGA (thus, no significant movement is anticipated if the Factor of Safety is 1 or higher when the PGA is reduced by 50% for the pseudostatic analysis). It is noted that this reduction factor is typically applied in embankments for two reasons:

- To account for the fact that soils usually amplify bedrock accelerations during an Earthquake. Therefore, peak acceleration at ground level is typically higher than what it would be within the fill where the slip occurs.
- As the horizontal force from the PGA is applied to each slice of mass calculated using the Simplified Bishop Method, an average acceleration across the slipping mass should be used instead of the PGA. Using the acceleration at ground level would be too conservative.

A number of assumptions were made in the process of simplifying complex situations in the field to a computer model:

- Unsaturated strengths from matric suction were not considered
- Stabilizing effects of vegetative cover on the side slopes were not included;

9.2.2 Soil Strength Parameters

Table 9-1 outlines the geotechnical parameters used for the modeled materials. Five types of materials were chosen to represent the site conditions: waste / remediated soil, sand, till, low permeability fill, topsoil, interface, gravel and bedrock. Unit weights are also outlined for both saturated and unsaturated states.

Parameters of the underlying foundation material used in this analysis were taken from AE (1985) and Alta Tech (2015) reports and SHA’s data base. It is suspected that the addition of large volumes of cement has increased the shear strength of the waste materials above normal values, but as the material was not tested the strength contribution of the cement was not factored in our analysis.
Table 9-1  Geotechnical Parameters for SLIDE

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight, γ (kN/m³)</th>
<th>Cohesion, c’ (kN/m²)</th>
<th>Internal Friction Angle, φ’ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unsaturated</td>
<td>Saturated</td>
<td></td>
</tr>
<tr>
<td>Waste/Remediated Soil</td>
<td>16</td>
<td>16</td>
<td>2</td>
</tr>
<tr>
<td>Sand</td>
<td>16</td>
<td>18</td>
<td>0</td>
</tr>
<tr>
<td>Till</td>
<td>18</td>
<td>19</td>
<td>5</td>
</tr>
<tr>
<td>Low Permeability Fill</td>
<td>18</td>
<td>19</td>
<td>1</td>
</tr>
<tr>
<td>Topsoil</td>
<td>14</td>
<td>15</td>
<td>2</td>
</tr>
<tr>
<td>Interface</td>
<td>18</td>
<td>19</td>
<td>5</td>
</tr>
<tr>
<td>Gravel</td>
<td>18</td>
<td>20</td>
<td>1</td>
</tr>
<tr>
<td>Bedrock</td>
<td>24</td>
<td>25</td>
<td>2</td>
</tr>
</tbody>
</table>

9.2.3  Ground Water Conditions

Monitoring wells were installed to determine the groundwater conditions beneath the site. A stratification of fracture density/permeability beneath the site was observed that can be represented into two distinct layers as follows:

- 0 to 75 m- Upper Bedrock: Negligible groundwater flow
- Below 75 m- Deep Bedrock: Minor groundwater flow

The data indicate that there is no water level mounding acting on the base of the liner, nor is there any perched water table acting within the PEA. Therefore, it is anticipated that the waste mass will not have any destabilizing pore pressure acting on the soil mass within the PEA beneath the cover. Precipitation will introduce water on the surface which could lead to the development of a perched water table above the geomembrane cap. As a highly transmissive gravel drainage layer will be introduced above the geomembrane, mounding of water within this layer is not expected; however, as a worst case condition we assumed a conservative worst-case long-term water table condition, having the water level in the embankment immediately above the drainage layer overlying the smooth geomembrane. This condition was modeled for both static and seismic scenarios.
9.2.4 Global Slope Stability Results

Results of the SLIDE analysis for the proposed grading design and existing conditions can be found in Figure 9-2 and 9-3. Each figure shows the soil profile, the resultant failure circle, the minimum FOS and the deep-seated FOS. The following Table 9-2 summarizes the FOS obtained for the cross section both for static and seismic conditions.

<table>
<thead>
<tr>
<th>Slope Cross Section</th>
<th>Condition</th>
<th>Maximum Slope Height (m)</th>
<th>Slope Angle (H:V)</th>
<th>Static</th>
<th>Seismic</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-3’</td>
<td>Proposed</td>
<td>13</td>
<td>5:1</td>
<td>2.6</td>
<td>1.12</td>
</tr>
</tbody>
</table>

The model results suggest that the proposed design will be stable for all static loading conditions with FOS values exceeding 2.0. For the seismic loading conditions, FOS values greater than 1.0 are obtained at the cross section. The seismic results suggest a high degree of slope stability and low risk of failure.

The above results, including the stabilizing wedge have increased the factor of safety and make the design more protective than SHA’s earlier cover veneer design that yielded a static FOS of 2.05 to 2.16 and a seismic FOS of 1.06 to 1.16.

9.2.5 Conclusion

The results indicate that the revised buttressing wedge design is stable for all static and seismic loading conditions and is more protective than SHA’s July 21st, 2017 design. In both cases, the deep seated FOS for the Circular Search were found to be more than 1.5 and 1.0 for static and seismic conditions respectively, suggesting that the landfill will be globally stable post-closure.

SHA recommends that erosion control mats/straw wattles be used on the slopes to control erosion and that a highly transmissive gravel drainage layer be used above the geomembrane rather than Drain Tubes so that the mounding can be minimized in the topsoil and subsoil layer on the slope.

Based on our closure construction experience, it is recommended that backfill materials be placed with a very light LGP Dozer with a total machine weight of less than 8 tonnes (e.g., John Deere 450J) when working within 1.0 m of the liner. Above that, larger equipment shall be used to achieve a high degree of density in the stabilizing fill.
SLOPE STABILITY FOR CROSS SECTION 3-3' (STATIC CONDITIONS)

CLIENT: COBBLE HILL HOLDINGS LTD.

PROJECT: COBBLE HILL LANDFILL UPDATED FINAL CLOSURE PLAN 2019

TITLE: FIGURE 9-2
SLOPE STABILITY FOR CROSS SECTION 3-3' (SEISMIC CONDITIONS)
10. ENVIRONMENTAL MONITORING & POST-CLOSURE REQUIREMENTS

The monitoring plan proposed for the CHL is based on standard landfill monitoring protocols implemented at other landfill sites in B.C. and adjusted to capture the monitoring requirements specified in MOE’s correspondence on this file. The key goals of the program are outlined below, and include:

- Inspection, operation and maintenance of the landfill final cover, including stormwater management works, ditching, topsoil, vegetation and the repair of any damage due to erosion, leachate breakouts, slope failures, settlement and burrowing animals.
- Inspection, operation and maintenance of environmental monitoring and leachate collection and storage works
- Environmental monitoring program, including leachate monitoring, to verify the escape or spill of leachate into the environment has not occurred.

Produced volumes of leachate to be tracked at storage facility before transport offsite

10.1 Post Closure Period

The CHL accepted contaminated soils, with contaminant levels below Hazardous Waste Regulations. During the operating life of the landfill, and including the soil currently contained in the SMA, approximately 97,595 tonnes of soil will have been landfilled. No further landfilling of soils is planned at CHL, with the exception of soils already contained in the Soil Management Area.

Recent correspondence with MOE has made it clear that the CHL is expected to follow the guidance outlined in the 2016 LCMSW in preparing this Closure Plan and the associated monitoring program. As per Section 7.4 of the 2016 LCMSW, the contaminating lifespan of a landfill shall not be assumed to be less than 30 years when determining the requirements for post closure care and financial security. Since CHL stopped receiving waste in 2015 and the site was capped with a geomembrane in 2016, the monitoring program should extend to 2046.

SHA is of the opinion that a 30 year post closure period is indeed appropriate because the nature of wastes received (non-leachable contaminated soils) and the nature of the double encapsulation system adopted at the CHL site surpass normal landfilling practices in British Columbia. Due to the impervious final cover system, the relatively low volumes of leachate released from the PEA, and the significant dilution that will be provided by clean run-off from the final cover system and shallow groundwater flow beneath the cell, SHA is of the opinion that CHL is a protective landfill and that the
characteristics of soil disposed at CHL will not pose a risk to human health or the environment beyond the 30-year post closure period and that aquatic life water quality objectives will be achieved at the property boundary after the post closure maintenance period has ended.

SHA notes that the CHL was designed and permitted under the 1993 Landfill Criteria which required only a 25 year post closure period; however, for the purpose of this Closure Plan a 30 year post-closure period is assumed at this time to be consistent with the minimum requirements of Section 7.4 of the LCMSW. As such, SHA proposes that the post-closure maintenance and monitoring that is outlined in this Chapter be implemented for a total period of 30 years; unless long term monitoring results or QP guidance reveals that a duration of less or more time is required and supported by monitoring data. Considering that the geomembrane liner has been in place on the PEA since 2016 and environmental monitoring has been regularly conducted since that time, SHA proposes the post-closure period finish in 2046.

10.2 Environmental Monitoring Plan

A detailed Environmental Monitoring Plan (EMP) for leachate, groundwater, surface water, and landfill gas is outlined in the following section and will be implemented during landfill closure and post closure. The EMP has been developed in accordance with the "Guidelines for Environmental Monitoring at Municipal Solid Waste Landfills" for groundwater, surface water, leachate, soils and vegetation. Best management practices should be followed as outlined in the British Columbia Field Sampling Manual complete with QA/QC sampling. The EMP has been designed with guidance from the Conceptual Model for the site.

The EMP’s objectives are to:

- Demonstrate compliance with the performance criteria.
- Demonstrate that monitoring results are consistent with the applicable plans and reports, including the groundwater and surface water impact assessment.
- Address the need for monitoring within 1 km of the landfill footprint.
- Given favorable results over time, the monitoring regime may be reduced upon review by a qualified professional.

Table 10-1 Summarizes the EMP in table form.
As described in Chapter 2, the site’s leachate contains parameters above groundwater benchmarks for conductivity, hardness, chloride, sulfate, calcium, magnesium, and sodium. In the letter dated September 20, 2018, MOE staff recommended that the reporting of the site’s water quality include Piper plot analysis. To achieve this, the following table outlines the key parameters that will be required to run the analysis. Parameters will include physical parameters, nutrients, and dissolved metals in the seepage blanket, and dissolved metals in surface water and leachate stations.

Table 10-1 – Summary of the Proposed Environmental Monitoring Plan

<table>
<thead>
<tr>
<th>Monitoring Location</th>
<th>Monitoring Frequency</th>
<th>Reporting Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Groundwater: SHA MW19-01, SHA MW19-02 (contingency wells, if and when installed)</td>
<td>Quarterly, dependent on precipitation and rainfall events; minimum two events annually at beginning and end of dry season.</td>
<td>Annually</td>
</tr>
<tr>
<td>Seepage Blanket SB-1, SB-2, SB-3</td>
<td>Dependant on precipitation and rainfall events; minimum two events annually at beginning and end of dry season.</td>
<td>Annually</td>
</tr>
<tr>
<td>Surface water SW-1</td>
<td>Quarterly</td>
<td>Annually</td>
</tr>
<tr>
<td>Leachate LE-1</td>
<td>Prior to preparation for off-site transportation and or prior to treatment.</td>
<td>Annually</td>
</tr>
</tbody>
</table>
Table 10-2 Key Leachate Parameters

<table>
<thead>
<tr>
<th>Physical Tests</th>
<th>Anions and Nutrients</th>
<th>Dissolved Metals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conductivity  (uS/cm)</td>
<td>Alkalinity, Total (as CaCO3)</td>
<td>Calcium (Ca)</td>
</tr>
<tr>
<td>Hardness (as CaCO3)</td>
<td>Chloride (Cl)</td>
<td>Magnesium (Mg)</td>
</tr>
<tr>
<td>pH</td>
<td>Fluoride (F)</td>
<td>Manganese (Mn)</td>
</tr>
<tr>
<td>Total Dissolved Solids</td>
<td>Nitrate (as N)</td>
<td>Potassium (K)</td>
</tr>
<tr>
<td>Turbidity (NTU)</td>
<td>Sulfate (SO4)</td>
<td>Sodium (Na)</td>
</tr>
</tbody>
</table>

10.3 Leachate Monitoring

As described in Chapter 7, it is expected that the leachate generation rate will continue to diminish as the site is capped with a geomembrane and overlaying soil layers and the natural consolidation processes are completed. A brief uptick in leachate production is expected due to the extra surcharge and resulting consolidation that will occur during soil wedge placement.

Leachate that is collected at the toe of the landfill is stored in the Leachate Storage Facility until it reaches a volume such that it is trucked off site to a regulated disposal facility. Prior to leachate removal, the leachate is treated to reduce metals concentrations to meet the requirements of the off-site discharge facility. Currently, the leachate is polished onsite utilizing bag filtration and the addition of Potassium Permanganate prior to removal to reduce manganese levels to below 5 ppm, as required by the receiving leachate disposal facility. All leachate samples for leachate characterization testing should be taken prior to treatment. Samples for WWTP acceptance compliance will be taken post treatment.
It is anticipated the leachate tanks will continue to be emptied by vac truck. During leachate removal, leak detection monitoring for the leachate storage tank will also be conducted. During this time, the leak detection tank will also be monitored and a visual inspection of the condition of the secondary liner will be completed. In the event a leak is observed on the base of the secondary liner system (outlined in Chapter 7), the entire contents of the tank will be removed and the tank will be inspected and repaired or replaced. Interim tanks will be installed to collect all leachate during this time to ensure no spill is made to the environment.

As per Hemmera’s recommendations, leachate and leak detection tank volumes will be recorded during each leachate removal and/or leachate tank monitoring event. This will be correlated to precipitation data to assess and confirm cover integrity and demonstrate there is no correlation between rainfall events and leachate production due to cover liner leaks. In the event potential contact water / leachate is observed in the leak detection storage tank, field testing for conductivity will be completed in the seepage blanket monitoring wells, and samples will be analyzed for leachate parameters as required. Further monitoring and testing of the leak detection system will be undertaken. Remedial measures will then be determined based laboratory analysis outlined above and recommendations by a QP.

As discussed above, SHA recommends leachate monitoring and sampling be conducted from the leachate tank prior to the tank contents being prepared for removal and transportation to the treatment plant (for the parameters outlined below). Leachate monitoring will be conducted until a Qualified Professional and the Chief Inspector deem otherwise. Table 10-3 illustrates the proposed leachate monitoring locations, parameters and frequency. In addition, as discussed above, the Piper plot analysis will be conducted annually and will include calcium, magnesium, sodium, potassium, bicarbonate, sulfate, chloride, dissolved solids, and conductivity.
### Table 10-3 – Leachate Monitoring Summary

<table>
<thead>
<tr>
<th>Monitoring Location</th>
<th>Physical Tests</th>
<th>Anions and Nutrients</th>
<th>Total Metals Suite Including</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Leachate Tank (SHA-LE-1)</td>
<td>Conductivity</td>
<td>Alkalinity, Total (as CaCO3)</td>
<td>Calcium (Ca)</td>
<td>Inspected Quarterly/ Sampled During Leachate Removal Prior to Treatment</td>
</tr>
<tr>
<td></td>
<td>Hardness (as CaCO3)</td>
<td>Chloride (Cl)</td>
<td>Magnesium (Mg)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pH</td>
<td>Fluoride (F)</td>
<td>Manganese (Mn)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total Dissolved Solids</td>
<td>Nitrate (as N)</td>
<td>Potassium (K)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Turbidity (NTU)</td>
<td>Sulfate (SO4)</td>
<td>Sodium (Na)</td>
<td></td>
</tr>
<tr>
<td>Leak Detection Tank (SHA LD-1)</td>
<td>Conductivity</td>
<td>Alkalinity, Total (as CaCO3)</td>
<td>Calcium (Ca)</td>
<td>Inspected Quarterly/ Sampled if Liquid is Present</td>
</tr>
<tr>
<td></td>
<td>Hardness (as CaCO3)</td>
<td>Chloride (Cl)</td>
<td>Magnesium (Mg)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pH</td>
<td>Fluoride (F)</td>
<td>Manganese (Mn)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total Dissolved Solids</td>
<td>Nitrate (as N)</td>
<td>Potassium (K)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Turbidity (NTU)</td>
<td>Sulfate (SO4)</td>
<td>Sodium (Na)</td>
<td></td>
</tr>
<tr>
<td>Leachate Storage Facility</td>
<td></td>
<td></td>
<td></td>
<td>Visual Inspection of Secondary Liner Following Leachate Removal</td>
</tr>
</tbody>
</table>

### 10.4 Surface Water Monitoring

Surface water monitoring is completed at SW-1; which is the ephemeral creek immediately downstream of the settling pond outlet. Flow in this creek is intermittent and only present during saturated conditions. The proposed monitoring is quarterly, at the same time as groundwater monitoring. Table 10-4 illustrates the proposed surface water monitoring locations, parameters and frequency. In addition, as discussed above, the Piper plot analysis will include calcium, magnesium, sodium, potassium, bicarbonate, sulfate, chloride, dissolved solids, and conductivity.
Table 10-4 – Surface Water Monitoring Summary

<table>
<thead>
<tr>
<th>Monitoring Location</th>
<th>Physical Tests</th>
<th>Anions and Nutrients</th>
<th>Total Metals Suite Including</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ephemeral Creek (SHA-SW-1)</td>
<td>Conductivity</td>
<td>Alkalinity, Total (as CaCO₃)</td>
<td>Calcium (Ca)</td>
<td>Inspected Quarterly/ Sampled Quarterly</td>
</tr>
<tr>
<td></td>
<td>Hardness (as CaCO₃)</td>
<td>Chloride (Cl)</td>
<td>Magnesium (Mg)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pH</td>
<td>Fluoride (F)</td>
<td>Manganese (Mn)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total Dissolved Solids</td>
<td>Nitrate (as N)</td>
<td>Potassium (K)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Turbidity (NTU)</td>
<td>Sulfate (SO₄)</td>
<td>Sodium (Na)</td>
<td></td>
</tr>
</tbody>
</table>

10.5 Seepage Blanket Monitoring

Three seepage blanket monitoring wells were installed in 2017. Following closure construction, these wells should be surveyed to allow for geodetic water level monitoring, including depth, ground elevation and stick-up. Since these wells were installed downgradient of the PEA it is anticipated that they will serve as a good detection system for any leachate escaping from the PEA. Flow through the seepage blanket is considered the primary pathway for leachate should a failure of the double liner system ever occur. The results from the seepage blanket wells should be compared to leachate and leak detection system water to assess liner integrity. The primary means of impact detection should be a Piper plot, augmented by trend line graphs for key leachate indicator parameters.

Conductivity readings from the wells should be collected during on site quarterly inspections. During closure activities and for at least two years following the completion of closure activities, quarterly detailed water quality analysis will also be completed at the same time as groundwater sampling. A QP will review the findings of the analysis to determine if the seepage layer monitoring program is sufficient and based on the results determine if additional monitoring locations are required.

Since the seepage blanket interflow is controlled by precipitation which can dilute the water chemistry, the seepage blanket monitoring wells should be sampled either prior to large precipitation events or sometime after such events. Table 10-5 illustrates the proposed seepage blanket monitoring locations,
parameters and frequency. In addition, as discussed above, the Piper plot analysis will include calcium, magnesium, sodium, potassium, bicarbonate, sulfate, chloride, dissolved solids, and conductivity. During each sampling event, the total well depth, depth to water, and geodetic water level should be recorded. Because the landfill closure design involves extending two of the three seepage blanket wells to facilitate the placement of final cover; once closure is completed, all wells should be surveyed and the elevations should be updated.

Common groundwater sampling methods that involve purging three to five well volumes from a monitoring wells can cause high levels of turbidity in collected samples (Puls and Barcelona 1996). High levels of turbidity can result “in the inclusion of otherwise immobile artifactual particles which produce an overestimation of certain analytes of interest (e.g. metals or hydrophobic organic compounds).” To ensure the collected water samples do not have high levels of turbidity, low stress (low flow) sampling procedures should be used at the site. Samples must contain turbidity levels below 50 NTU.
<table>
<thead>
<tr>
<th>Monitoring Location</th>
<th>Physical Tests</th>
<th>Anions and Nutrients</th>
<th>Dissolved Metals Suite Including</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seepage Blanket (SHA-SB-1)</td>
<td>Conductivity Hardness (as CaCO3) pH Total Dissolved Solids Turbidity (NTU)</td>
<td>Alkalinity, Total (as CaCO3) Chloride (Cl) Fluoride (F) Nitrate (as N) Sulfate (SO4)</td>
<td>Calcium (Ca) Magnesium (Mg) Manganese (Mn) Potassium (K) Sodium (Na)</td>
<td>Inspected Quarterly/ Sampled During Summer Months Between Spring Freshet and First Fall Flush Water Levels &amp; Total depth</td>
</tr>
<tr>
<td>Seepage Blanket (SHA-SB-2)</td>
<td>Conductivity Hardness (as CaCO3) pH Total Dissolved Solids Turbidity (NTU)</td>
<td>Alkalinity, Total (as CaCO3) Chloride (Cl) Fluoride (F) Nitrate (as N) Sulfate (SO4)</td>
<td>Calcium (Ca) Magnesium (Mg) Manganese (Mn) Potassium (K) Sodium (Na)</td>
<td>Inspected Quarterly/ Sampled During Summer Months Between Spring Freshet and First Fall Flush Water Levels &amp; Total depth</td>
</tr>
<tr>
<td>Seepage Blanket (SHA-SB-3)</td>
<td>Conductivity Hardness (as CaCO3) pH Total Dissolved Solids Turbidity (NTU)</td>
<td>Alkalinity, Total (as CaCO3) Chloride (Cl) Fluoride (F) Nitrate (as N) Sulfate (SO4)</td>
<td>Calcium (Ca) Magnesium (Mg) Manganese (Mn) Potassium (K) Sodium (Na)</td>
<td>Inspected Quarterly/ Sampled During Summer Months Between Spring Freshet and First Fall Flush Water Levels &amp; Total depth</td>
</tr>
</tbody>
</table>
10.6 Groundwater Monitoring

The original waste discharge permit stated that a minimum of seven groundwater wells would be installed and maintained. A detailed review of the conceptual hydrogeologic model has revealed that the primary pathways for leachate migration are through the seepage blanket and through surface water flow. Because there is an upward hydraulic gradient and competent bedrock with very few fractures, the chance of detecting any impact in the deep groundwater wells is extremely remote. Therefore, SHA recommends in this plan to stop monitoring the deep wells until such a time that a breach of the liner system is suspected, either by detecting water in the leak detection system, or detecting leachate impacts in the seepage wells.

As an additional contingency, two new shallow bedrock wells MW19-01 and MW19-02, downgradient of the PEA, will be installed at the site under a phased approach if any of the monitoring data indicates that leakage of leachate may be occurring. Drilling of the additional wells will be triggered if leachate is detected in the leak detection system, if Piper Plot analysis from any of the existing shall bedrock wells is indicating leachate impact, or if there is a marked increase in the volume of leachate collected in the leachate collection system (which would indicate a partial failure of the encapsulation membrane). The contingency shallow bedrock wells will be installed to further develop the dataset for the site and confirm if contamination is leaching from the landfill.

Should significant changes be observed in the groundwater, additional wells existing on the site can be reactivated to the monitoring program, under QP guidance.

During environmental monitoring, the wells should be inspected and any maintenance required on the well caps and standpipes should be performed. Groundwater level should also be recorded while environmental sampling is occurring. Any new wells should be surveyed, including depth of well, ground elevation and stick-up, to allow for geodetic water level monitoring.

Table 10-6 illustrates the contemplated future groundwater monitoring locations (if required), parameters and frequency. In addition, as discussed above, the Piper plot analysis would include calcium, magnesium, sodium, potassium, bicarbonate, sulfate, chloride, dissolved solids, and conductivity.
Table 10-6 – Groundwater Monitoring Summary

<table>
<thead>
<tr>
<th>Monitoring Location</th>
<th>Parameters</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHA MW19-01 (New Shallow)</td>
<td>Total/Dissolved Metals</td>
<td>Quarterly</td>
</tr>
<tr>
<td>SHA MW-19-02 (New Shallow)</td>
<td>Anions and Nutrients</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Physical Tests</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Groundwater level</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Total Depth</td>
<td></td>
</tr>
<tr>
<td>MW-06</td>
<td>Total Depth</td>
<td></td>
</tr>
</tbody>
</table>

Locations for monitoring wells, water wells, surface water stations and air sampling stations are presented on Figure 10-1.

### 10.7 Landfill Gas Monitoring and Ambient Air Monitoring

As the landfill is not a Municipal Waste Landfill and the gas generation rate for the types of waste being landfilled is extremely slow if at all, Landfill Gas is not deemed to be of great concern at the site.

The current soil cell (PEA) has been capped with 40 mil LLDPE liner since the Fall of 2016. Other than ballasting that was completed in 2017, the landfill cell is not currently being held down with significant weight. Since installation, no uplifting of the liner due to landfill gas has been observed. SHA does not foresee volatile gases being an issue at this site after closure.

A review of leachate testing has indicated that VOC’s are generally non-detect in the leachate. Therefore; release of volatile organic compounds through the barrier layer is no longer a concern. This should be verified with a one-time monitoring run of VOC’s emissions from the landfill. A VOC instrument will be used to sample from a minimum of 10 locations (1 sample per 200 m²) or as directed by the Chief Inspector. Should the annual testing confirm that the landfill cap is preventing the release of any VOC’s then the number of sampling locations could be reduced.

Even though the risk is deemed to be low, no permanent structures should be built on top of the landfill unless they are constructed with properly ventilated foundations. Also, standard confined space entry procedures must be followed when entering any manholes or other structures on or near the landfill site as landfill gas can accumulate in such structures over time.
Additionally, ambient air monitoring was required due to potential dust concerns from soil mixing, blasting, mining and landfilling activities. Given there is no activity currently in operation at the site, SHA does not foresee the need for continued monitoring for air quality post closure.

10.8 Geotechnical Inspection

A geotechnical inspection should be undertaken by a QP on an annual basis to inspect the landfill footprint. This inspection should also include checking the cover for potential problems arising from slumping, cracking or erosion, and determining the state of other infrastructure that does not receive regular inspection. If significant issues with infrastructure are identified, a Qualified Professional should be retained to resolve them.

10.9 Post Closure Inspection

An annual inspection of all infrastructure will be completed by a QP prior to compilation of the annual report. This inspection will include the following:

- Final Cover System
- Ditching
- Topsoil / Vegetation
- Erosion
- Potential Leachate Breakouts – not expected with geosynthetic basal and final cover geomembrane system
- Leachate Collection, Conveyance and Storage Facility
- Environmental Monitoring Infrastructure.

Any observed deficiencies will be recorded and remedied as required.

10.10 Annual Report

Each year a Qualified Professional will collect the required monitoring data and compile an Annual Report outlining the closure performance of the landfill. Each year the Qualified Professional will assess the groundwater and surface water quality for potential impacts and evaluate the leachate generation rate and quality. Piper diagrams will be used to characterize the leachate and all water samples; and these diagrams will be included in annual reporting as will trend diagrams for key leachate indicator parameters.
All infrastructure maintenance and improvements identified in the post closure inspection are to be outlined in the annual report. In addition, the report should indicate if any accumulation of liquid was observed in the secondary containment of the leachate storage facility. In each report the Qualified Professional will make recommendations on the existing monitoring program and identify any changes that would improve the post closure monitoring.

Surface water structures will be investigated during monitoring events at the site. Ditches and downchutes will be maintained to ensure proper drainage at the site, and to minimize the potential of erosion of surrounding structures.

As part of the on-going leachate collection and on-site treatment, the system will be investigated for operational effectiveness. The double lined leachate storage facility will allow for leak detection monitoring, as outlined previously. Maintenance will be completed as required.

**10.11 Adjustments to Monitoring Programs Over Time**

For many years SHA has recognized there is a strong correlation between electrical conductivity and leachate impacts in groundwater at landfill sites in B.C. It is recommended that at the CHL landfill electrical conductivity be recorded during each sample run and that the quarterly environmental monitoring samples be correlated to conductivity for the first 10 years of sampling. Provided that a good level of correlation exists, SHA recommends that laboratory sampling of groundwater and leachate be scaled back from quarterly to semi-annual after 5 years of sampling, and from semi annual to annual after 10 years of sampling provided that trend lines continue to show steady state or improving water quality conditions. These recommendations should be reviewed and confirmed by a QP based on observed results prior to any changes in the monitoring program.

**10.12 Contingency Actions for EMP – Trigger Response Plan**

This contingency plan outlines planned responses that would be undertaken in the event that problems are identified or natural events transpire that potentially compromise the integrity of the PEA. In responding to any incident, a systematic five step approach will be adopted that will include the following:

1. Routine Monitoring
2. Confirmatory Monitoring
3. Additional Investigation to establish magnitude of problem
4. Mitigation Strategy to fix problem
5. Follow up monitoring to confirm mitigation strategy was successful
1. **Routine Monitoring:** In the event results from the EMP indicate that there is a negative trend in water quality detected, or there are other events that could cause a breach of the containment system like an earthquake, a massive erosion event or slope failure, CHL should engage with a QP to develop appropriate remedial actions (if required).

2. **Confirmatory Monitoring:** If elevated water quality is detected in the monitoring network, then the sample should be retested to validate the exceedance. If validation confirms elevated levels, then the confirmatory sampling should be expanded to adjacent wells as well and additional investigation should be initiated. Sampling of deep wells would also be initiated.

3. **Additional Investigation:** Additional investigation will depend on the problem encountered, but will likely involve development of new sampling points downgradient, including additional seepage blanket wells, new surface water sampling sites, and frequent monitoring of leachate being captured by the leachate collection and leak detection systems. The integrity of the PEA would also be inspected by a QP for any signs of visible damage.

   A Risk Assessment may be warranted if the water quality data suggests the landfill is impacting the water quality beneath the landfill and/or off-site. The assessment should look at the specific parameters of concern and determine if there is a risk to the receiving environment and drinking water supplies; and if remedial measures are required.

4. **Mitigation Strategy:** The mitigation strategy will depend on the nature of the problem. An early line of defense may include installation of a leachate / leak detection contingency trench to capture all shallow groundwater moving through the seepage blanket. Details on this trench are presented in Section 7.3 of this plan. Examples of additional measures could involve excavation, exposure and repair of the liner in visibly distressed areas, development of an on-site treatment process to remediate impacted groundwater, lining of the up-slope diversion ditch to further limit infiltration of water below the PEA. Other mitigation measures may be identified, depending on the characterization of the problem in Additional Investigation task.

5. **Follow Up Monitoring:** will involve resampling the impacted sites to verify that the water quality at the impacted sites is improving, that the trend is one of reduced concentrations. Follow up monitoring should continue at an increased frequency (e.g. monthly) until there is a very high degree of confidence that the remedial measures were effective.
10.12.1 Leachate Management Contingency Plan

Contemplated circumstances that may cause failure of the leachate works include exhaustion of leachate storage capacity, leachate leaking through the liner and a breach of the final cover geomembrane.

As discussed in Chapter 7, the leachate storage tanks are equipped with a high-level alarm system which sends a notification when the leachate has reached a certain level in the tanks. The high-level alarm is set at a level which provides at least one month of additional capacity once the alarm has been triggered. This alarm will continue to remain operational at the site.

As outlined in Section 7.3 this report, SHA has designed a conceptual leachate cut-off trench which could be implemented in the event that the basal liner is confirmed to be failing. This trench is envisioned to be excavated into the competent bedrock downgradient of the PEA. Leachate flowing through the subsurface would be captured in this trench, diverted to the leachate collection and storage tank, and removed for off-site treatment.

In the event that leachate storage capacity is exceeded, leachate would start to collect on the secondary containment liner. In that event vac trucks would be organized on an emergency response basis to transport the leachate to an approved treatment facility. If pre-treatment was required, leachate would be temporarily transferred to the leak detection tank, and if additional capacity was required, additional storage tanks would be mobilized to the site.

In the event that an earthquake or catastrophic weather event causes a failure of the cover system, steps will be taken to implement a temporary cover to prevent further ingress of precipitation into the PEA. A plan will then be developed to identify a cause and remediate the failed liner.

All of the above contingency actions are considered highly unlikely, but have been developed as a worst-case scenario response.

10.12.2 Surface Water Management Contingency Plan

Contemplated circumstances that may cause failure and/or non-compliance of the surface water works include: extreme weather events, increased snowmelt, clogging or blockage of infrastructure, erosion and water quality issues during first flush run-off.

As a precaution, SHA has designed surface water management works (ditches and downchutes) with flow capacity and ditch geometry that is able to handle flows in excess of a 1 in 200 year storm event.
Post-closure monitoring should include inspection of the surface water management works and any required maintenance of the infrastructure should be completed in a timely matter.

In the event that a rain event causes a wash out to the stormwater ditches then the washout should be assessed quickly and temporary erosion control measures implemented to prevent continued erosion and sediment transport. The washout should then be repaired as quickly as possible.

In the event that run-off from the capped area exceeds some water quality parameters, run-off shall be stored in the settling pond until water quality is acceptable for discharge. Additional filtration and erosion control measures may need to be added, including straw bales in ditches, additional loose straw on slopes, additional straw wattles at toe of slope and possible geotextile filtration.

In the event that run-off from the PEA does not meet discharge objectives or background levels (whichever are higher), then additional on-site treatment of water in the settling pond may need to be introduced. This treatment process would be developed by a qualified QP.

In the event that environmental monitoring indicates the surface water downstream of the landfill is being impacted, the CHL shall retain a QP to determine the appropriate remedial or mitigative measures required.
11. LIMITATIONS

This report has been prepared by Sperling Hansen Associates (SHA) on behalf of Cobble Hill Holdings Ltd. in accordance with generally accepted engineering practices to a level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions in British Columbia, subject to the time limits and financial and physical constraints applicable to the services.

The report, which specifically includes all tables and figures, is based on engineering analysis by SHA staff of data compiled during the course of the project. Except where specifically stated to the contrary, the information on which this study is based has been obtained from external sources. This external information has not been independently verified or otherwise examined by SHA to determine its accuracy and completeness. SHA has relied in good faith on this information and does not accept responsibility of any deficiency, misstatements or inaccuracies contained in the reports as a result of omissions, misinterpretation and/or fraudulent acts of the persons interviewed or contacted, or errors or omissions in the reviewed documentation.

The report is intended solely for the use of Cobble Hill Holdings Ltd. Any use which a third party makes of this report, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. SHA does not accept any responsibility for other uses of the material contained herein nor for damages, if any, suffered by any third party because of decisions made or actions based on this report. Copying of this intellectual property for other purposes is not permitted.

The findings and conclusions of this report are valid only as of the date of this report. The interpretations presented in this report and the conclusions and recommendations that are drawn are based on information that was made available to SHA during the course of this project. Should additional new data become available in the future, SHA should be requested to re-evaluate the findings of this report and modify the conclusions and recommendations drawn, as required.

Report prepared by:

Scott Garthwaite, AScT  
Office Manager & Civil Technologist

Dr. Iqbal Bhuiyan, P.Eng  
Senior Environmental Engineer
12. REFERENCES


APPENDIX A
Permits and Certificates
August 21, 2013

Tracking Number: 225272
Authorization Number: 105809

REGISTERED MAIL

Cobble Hill Holdings Ltd. (BC0754588)
Herald Street Law
101-536 Herald Street
Victoria BC V8W 1S6

Dear Permittee:

Enclosed is Permit 105809 issued under the provisions of the Environmental Management Act. Your attention is respectfully directed to the terms and conditions outlined in the permit. An annual fee will be determined according to the Permit Fees Regulation.

This permit does not authorize entry upon, crossing over, or use for any purpose of private or Crown lands or works, unless and except as authorized by the owner of such lands or works. The responsibility for obtaining such authority rests with the Permittee. This permit is issued pursuant to the provisions of the Environmental Management Act to ensure compliance with Section 120(3) of that statute, which makes it an offence to discharge waste, from a prescribed industry or activity, without proper authorization. It is also the responsibility of the Permittee to ensure that all activities conducted under this authorization are carried out with regard to the rights of third parties, and comply with other applicable legislation that may be in force.

This decision may be appealed to the Environmental Appeal Board in accordance with Part 8 of the Environmental Management Act. An appeal must be delivered within 30 days from the date that notice of this decision is given. For further information, please contact the Environmental Appeal Board at (250) 387-3464.
Administration of this permit will be carried out by staff from the West Coast Region. Plans, data and reports pertinent to the permit are to be submitted to the Regional Manager, Environmental Protection, at Ministry of Environment, Regional Operations, West Coast Region, 2080A Labieux Road, Nanaimo, BC V9T 6J9.

Yours truly,

[Signature]

Hubert Bunce
for Director, Environmental Management Act
West Coast Region

Enclosure

cc: Environment Canada
PERMIT
PR-105809

Under the Provisions of the Environmental Management Act

Cobble Hill Holdings Ltd. (BC0754588)

Herald Street Law
101-536 Herald Street
Victoria BC V8W 1S6

is authorized to discharge refuse to ground and effluent to an ephemeral stream from a contaminated soil treatment facility and a landfill facility located at 640 Stebbings Road, Shawnigan Lake, British Columbia, subject to the terms and conditions listed below. Contravention of any of these conditions is a violation of the Environmental Management Act and may lead to prosecution.

1. AUTHORIZED DISCHARGES

1.1 Authorized Discharges – General Conditions

This section applies to the discharge of refuse from a contaminated soil treatment and to the landfill facility.

1.1.1 The combined maximum rate of discharge from the treatment and to the landfill facility is 100000 tonnes per year. The estimated density of soil accepted at the site ranges from 1.5 to 1.8 t/m³ for the purpose of sampling incoming soil or treated soil for characterization. The above density estimate may be modified at any time with a scientific sampling method approved by the Director.

1.1.2 The authorized discharge period is between 7am and 5pm Monday to Friday.

1.1.3 The characteristics of the discharges must be as described under Subsections 1.2 and 1.3.

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Soil relocation requirements of the Contaminated Sites Regulation (CSR) apply to all other parameters than those specified in this permit and in the Soil Acceptance Plan referred to under Section 2.2.

Soils meeting facility location background quality in accordance with CSR Protocol 4 may also be discharged.

If land use or site specific factors specified in Column I of Schedule 5 of the CSR change at the permitted site, the Permittee must promptly notify the Director and immediately apply them for the purpose of Subsections 1.2 and 1.3.

1.1.4 The authorized works as defined under Subsections 1.2.1, 1.3.1, 1.4.5 and 1.5.4 must be complete and in operation while discharging.

1.1.5 The location of the facilities and the points of discharge is Lot 23, Plan VIP78459, Blocks 156, 201 and 323, Malahat Land District.

1.2 **Authorized Discharge – Treatment Facility**

This section applies to the discharge of refuse from a soil treatment facility. The site reference number for this discharge is E292169.

1.2.1 The authorized works are a lined asphalt paved soil management and bio-remediation treatment area of approximately 1800 m², temporary soil holding area (as described under Subsection 2.3), biocell, berm, primary and secondary containment detection and inspection sumps and associated cleanout ports, catch basins, groundwater monitoring wells (as described under Subsection 3.3), management works and related appurtenances approximately located as shown on Figure A.

1.2.2 The characteristics of the discharge must be equivalent to or better than:

soil suitable for industrial land use, as described by the Generic and Matrix Numerical Soil Standards in Schedule 4, 5, 7 and 10 (Column IV “Commercial, Industrial Soil Standard”) of the CSR, including the most stringent applicable site specific factors as defined in the Environmental Procedures Manual (EPM) referred to in Subsection 2.13, considering intake of contaminated soil, toxicity to soil invertebrates and plants and...

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groundwater flow to surface water used by freshwater aquatic life for the authorized soil treatment and discharge parameters as specified in Subsection 1.2.3.

1.2.3 The types of soil that can be bio-remediated at the treatment facility are soils contaminated with hydrocarbons, specifically soils contaminated with Benzene, Toluene, Ethylbenzene, Xylene (BTEX), Styrene, Methyl Tertiary Butyl Ether (MTBE), Volatile Petroleum Hydrocarbons (VPHs), Light and Heavy Extractable Petroleum Hydrocarbons (LEPHs/HEPHs), Polycyclic Aromatic Hydrocarbons (PAHs), Chlorinated Hydrocarbons, Phenolic Substances, Chloride, Sodium and Glycols as defined in Schedules 4 and 5 of the CSR.

Soils co-contaminated with hydrocarbons as described in this section and metals or other contaminants not suitable for bioremediation meeting industrial land use standards as defined in Schedules 4 and 5 of the CSR may also be accepted for treatment at the biocell.

1.3 **Authorized Discharge – Landfill Facility**

This section applies to the discharge of refuse from a soil treatment facility and from relocated contaminated soil and associated ash. The site reference number for this discharge is E292889.

1.3.1 The authorized works are a landfill, engineered lined landfill cells, perimeter ditches, erosion and sedimentation control infrastructure, primary and secondary containment detection and inspection sumps and associated cleanout ports, catch basins, groundwater monitoring wells, management works and related appurtenances approximately located as shown on Figure A.

1.3.2 The characteristics of the discharge must be better than:

Hazardous waste, as described in the Schedule 1, 1.1, 3 and 4 (Part 3, table 1 – Leachate Quality Standards) of the Hazardous Waste Regulation (HWR) and must be limited to contaminated soils and associated ash. Hazardous waste (as defined in the *Environmental Management Act* and the HWR), liquids, putrescible and other wastes must not be discharged.

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The Director may specify different standards and other substances in writing for the protection of human health or the environment.

1.3.3 The types of soil that can be discharged at the landfill facility are soils and associated ash contaminated with metals, Dioxins, Furans, BTEX, MTBE, VPHs, LEPHs/HEPHs, PAHs, Styrene, Chlorinated Hydrocarbons, Phenolic Substances, Chloride, Sodium and Glycols as defined in Schedules 4 and 5 of the CSR.

1.4 Ancillary Discharge – Water Treatment System

This section applies to the discharge of effluent from the water treatment system (WTS). The site reference number for the WTS discharge is E292170.

1.4.1 The annual average rate of the WTS discharge is 12.1 cubic metres per day.

1.4.2 The maximum rate of the WTS discharge is 274 cubic metres per day.

1.4.3 The authorized discharge period is continuous.

1.4.4 The characteristics of the discharged treated effluent must be equivalent to or better than the most stringent of those British Columbia Approved Water Quality Guidelines (BCAQWG) and A Compendium of Working Water Quality Guidelines for British Columbia (BCWWQG) for Freshwater Aquatic Life (AL) protection and Drinking Water (DW) uses for the parameters of concern: Inorganic Substances including metals, VPHw, LEPHw, VHW6-10, EPHw10-19, PAHs, BTEX, Styrene, Chlorinated Hydrocarbons, Phenolic Substances, Chloride, Sodium, Glycols, pH and Oil & Grease.

Dioxins and Furans analysis must be conducted at a laboratory and using an analytical method agreed to by the Director and results must be below detection limit at all times.

The source of the discharge must be limited to site stormwater runoff and water from the primary and secondary containment systems authorized under Subsections 1.2.1, 1.3.1 and 1.4.5.

The Director may specify different standards and other substances in
writing for the protection of human health or the environment.

1.4.5 The authorized works are surface runoff collection and diversion ditches associated with the WTS, WTS (including pH control and flocculent injection system, settling tank, bag and activated carbon filters), leachate and leak detection reservoirs, flow measurement device, monitoring and sampling equipment, reservoirs and related appurtenances approximately located as shown on Figure A.

1.4.6 The authorized works must be complete and in operation while discharging.

1.4.7 The location of the facilities from which the discharge originates and the point of discharge is Lot 23, Plan VIP78459, Blocks 156, 201 and 323, Malahat Land District.

1.5 Ancillary Discharge – Settling Pond

This section applies to the discharge of stormwater from the settling pond. The site reference number for the settling pond outlet is E292898.

1.5.1 The rate of the settling pond discharge is 42,500 cubic metres per day for up to 1 in 10 year return period flood event of 24 hour duration.

1.5.2 The authorized discharge period is continuous.

1.5.3 The characteristics of the settling pond discharge effluent (SW-1) must be equivalent to or better than the most stringent of those BCAWQG and BCWWQG for Freshwater Aquatic Life uses and Total Suspended Solids (TSS) must not exceed 25 mg/L for up to 1 in 10 year return period flood event of 24 hour duration.

For flood events greater than 1 in 10 year return period flood event of 24 hour duration, the characteristics of the settling pond discharge must not exceed background concentrations (SW-4).

The source of the discharge must be limited to non contact site stormwater runoff and treated effluent released from the WTS described in Subsection 1.4.

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The Director may specify different standards and other substances in writing for the protection of human health or the environment.

1.5.4 The authorized works are surface runoff collection and diversion ditches, leachate, surface runoff and leak detection control reservoirs, one surface settling pond, flow measurement device, monitoring and sampling equipment, emergency overflow and related appurtenances approximately located as shown on Figure A.

1.5.5 The authorized works must be complete and in operation while discharging.

1.5.6 Settled solids which have accumulated in the settling pond must be removed as required to maintain a minimum water depth below the pond decant of 0.5 metre. The removed solids must be disposed of in a manner approved by the Director.

1.5.7 The location of the facilities from which the discharge originates and the point of discharge is Lot 23, Plan VIP78459, Blocks 156, 201 and 323, Malahat Land District.

2. GENERAL REQUIREMENTS

2.1 Soils and Associated Ash Unacceptable for Treatment

The following types of waste must not be accepted for treatment at the site:
1) Hazardous waste as defined in the HWR;
2) Soils contaminated with any substances not included in Subsection 1.2 above with concentrations exceeding relevant standards specified in Schedule 4 and 5 of the CSR;
3) Soils and associated ash that cannot be treated or landfilled successfully in the opinion of the Director; and
4) Liquid waste or soil and associated ash with a water content exceeding those described in the Soil Acceptance Plan.
5) Restricted wastes listed in the Soil Acceptance Plan described in Subsection 2.2 of this permit.

2.2 Screening and Acceptance of Soil

The Permittee must submit a Soil Acceptance Plan prepared by a Qualified

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Professional to the satisfaction of the Director for screening soil and associated ash for all potential contaminants of concern prior to receiving any material at the facility. No changes must be made to the plan without prior approval by the Director. The Director may amend the plan for the protection of human health or the environment.

Those soils suspected to be unacceptable must be either rejected immediately or placed in a holding area (as defined in Subsection 2.3) within the soil management area waiting further re-characterization by a Qualified Professional in accordance with Technical Guidance Document #1 (Site Characterization and Confirmation Testing). If further characterization confirms soils as unacceptable for treatment or landfilling (as defined in Subsections 1.2 and 1.3) the soil must not be mixed with any other soil and must be removed from the facility in accordance with the requirements of the Environmental Management Act and of the CSR.

2.3 Holding Area for Soil and Associated Ash Suspected/Determined to be Unacceptable

The Permittee must designate a holding area within the soil management area for short term storage of soil waiting for re-characterization or shipment to an appropriate management site as determined by a Qualified Professional. Short term storage must not exceed 30 days from the day of the delivery or as agreed by the Director. The soil must be kept separate from the soil treatment area and be protected from the weather at all times.

2.4 Bedrock Integrity Inspection and Risk Assessment

A bedrock integrity inspection and risk assessment report must be submitted to the Director prior to the construction of any landfill cells. For any abnormalities (open fractures, presence of water, percolation, etc) identified during the inspection, the Permittee must notify the Director immediately and issue a structural report within 30 days following the inspection. The report must be submitted to the satisfaction of the Director and prepared by a suitably Qualified Professional and must include, but is not limited to:

a) all relevant information collected during the inspection and detailing the abnormality;

b) an explanation and/or interpretation of the abnormality;

c) a risk assessment in regards to the risk to human health and the receiving

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environment; and
d) remedial action planned and/or taken to control the risks.

2.5 **Soil Aeration**

a) Where the thickness of contaminated soil within the soil treatment facility is greater than 30 cm, the Permittee must periodically conduct mechanical soil aeration. Soil aeration must only be done under the following conditions to prevent nuisance to potential receptors:

i. Ventilation index for Southern Vancouver Island for the day of soil turning is forecast as “good”;

ii. No sooner than three hours after sunrise and no later than two hours before sunset but within the authorized discharge period defined under Subsection 1.1.2;

iii. Favorable weather conditions (considering temperature and wind direction, etc.)

b) Prior to every soil aeration event the Permittee must record the ventilation index forecast, time of sunrise and sunset, time and duration of aeration, and ambient temperature. Records must be tabulated along with soil volumes aerated and chemical characteristics in the biocell at the time of aeration.

2.6 **Soil Amendment and Prohibition of Blending**

Bioremediation must be undertaken without blending/mixing of contaminated soil with cleaner soils for the purpose of dilution to meet the required standards.

Soil amendments which will enhance remediation potential, including bulking materials such as sawdust or straw, may be added prior to or during treatment. Should water be required to enhance soil treatment, contact water generated at the facility must be used in priority.

2.7 **Weather Protection**

The Permittee must cover the soil treatment piles, soil holding area and active landfill areas completely from November to April when not actively worked on and provide sufficient weather protection and containment for nutrients stored at the site for the protection of human health and the environment.
The Permittee must cover any soil stored within the holding area at all times.

2.8 **Erosion and Sedimentation Control**

The Permittee must ensure erosion and sedimentation control measures are implemented with the soil management and treatment area and the landfill area, to limit sediment releases to the settling pond, the water treatment system and to the receiving waters. Storm water runoff must be diverted away from the soil management and treatment area and all active landfill areas at all times. Erosion and sedimentation controls must be developed and implemented according to industry best management practices and consider the *Aggregate Operators Best Management Practices Handbook* prepared by the Ministry of Energy and Mines.

2.9 **Odour Control**

There must be no objectionable hydrocarbon odour evident outside the property boundaries. The Permittee must, at a minimum, implement contingency measures if the ambient air quality sampling results exceed the air quality standards defined under Subsection 3.5. The contingency measures must be defined in the EPM as documented in Subsection 2.13 and include, but are not limited to, reduced soil aeration times and the covering of soil piles.

The Director may amend the permit to require the implementation of additional control measures to limit odour generation.

2.10 **Dust Control**

Fugitive dust created within the operation area must be suppressed. Measured dustfall must not exceed the B.C. Ambient Air Quality Residential Objective of 1.7 mg/(dm²-day) over a two week averaging period at the property boundary. The contingency measures must be documented in the EPM as defined in Subsection 2.13 and include, but not limited to, reduced activities, covering or application of dust suppressant on soil piles and exposed areas.

The Director may amend the permit to require the implementation of additional control measures on fugitive dust sources.
2.11 Spill Reporting

All spills to the environment (as defined in the Spill Reporting Regulation) must be reported immediately in accordance with the Spill Reporting Regulation. Notification must be via the Provincial Emergency Program at 1-800-663-3456.

2.12 Maintenance of Works and Emergency Procedures

The Permittee must inspect the authorized works regularly and maintain them in good working order. In the event of an emergency or condition beyond the control of the Permittee which prevents effective operation of the authorized works or leads to unauthorized discharge, the Permittee must comply with all applicable statutory requirements, immediately notify the Director, and take appropriate remedial action for the prevention or mitigation of pollution. The Director may reduce or suspend operations to protect human health or the environment until the authorized works have been restored and/or corrective steps have been taken to prevent unauthorized discharges.

The Permittee must prepare and maintain an Emergency Response Plan (ERP) to the satisfaction of the Director that describes the procedures to be taken to prevent or mitigate any discharge in contravention of the EPM. The ERP must be immediately implemented if there is a discharge, or any risk of a discharge in contravention of the EPM. In addition, an up-dated ERP, including a report on any emergency responses, taken in the previous year, must be kept available, on site for inspection, as defined under Subsection 5.1.

The Permittee must review the ERP at least on an annual basis to determine if any changes are required and submit any revisions to the Director for acceptance.

2.13 Environmental Procedures Manual

An Environmental Procedures Manual (EPM) must be prepared and submitted by the Permittee to the Director. No soil may be received prior to acceptance of the EPM by the Director. The EPM must be kept current and available for use as a guide at all times at the facility. The manual must cover all typical aspects of an Environmental Management Systems (EMS) relevant to the management of the soil treatment, water treatment and landfill facilities including but not limited to, the following items:

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a) Risk identification and prioritization;
b) Administrative and engineering controls;
c) Roles and responsibilities;
d) Training requirements;
e) A Soil Acceptance Plan;
f) A Water Management Plan;
g) An Environmental Monitoring Plan, including on and off site monitoring locations and the sampling procedures for soil, water, groundwater and air quality, as required;
h) An Emergency Response Plan, including contingency measures.
i) Details on the site preparation and the construction of landfill cells;
j) Operation, inspection and maintenance of the soil management and treatment facility, the landfill facility, the water treatment system, erosion and sediment controls measures, the settling pond and associated appurtenances;
k) Internal and external EMS audits, and;
l) Notification, reporting, investigation and corrective and preventive measures.

The Permittee must review the EPM at least on an annual basis to determine if any changes are required and submit any revisions to the Director for acceptance. Annual reviews and submission of revisions are due on March 31 of each year.

2.14 Advisory Committee

The Permittee must establish an Advisory Committee and develop terms of references to the satisfaction of the Director. The Committee must be composed of one representative of each relevant regulatory agency and one representative from the local government. The Committee must meet annually within 3 months of the submission of the annual report as required under Subsection 5.3 and provide advice to the Director within 30 days of the meeting. Based on advice of the Committee, the Director may revise the monitoring, sampling and reporting requirements in Sections 3 and 5.

2.15 Qualified Professionals

All facilities and information, including works, plans, bedrock integrity and risk assessment, assessments, sampling, monitoring, investigations, surveys, programs and reports, must be conducted and certified by Qualified

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Professionals.

"Qualified Professional" means a person who
a) is registered to practice in British Columbia with his or her appropriate
professional association, acts under that professional association’s code of
ethics, and is subject to disciplinary action by that professional association,
and;

b) through suitable education, experience, accreditation and knowledge may be
reasonably relied on to provide advice within his or her area of expertise as
it relates to this permit.

2.16 Bypasses

The discharge of contaminants which have bypassed the authorized treatment
works is prohibited unless the prior approval of the Director is obtained and
confirmed in writing, except those authorized under Subsection 1.2 of this
permit.

Temporary storage or accidental deposit of contaminated soil at areas other than
the soil management area is considered a bypass.

2.17 Process Modifications

The Director must be notified in writing prior to implementing changes to any
process that may adversely affect the quality and/or quantity of the discharge.

2.18 Plans - New Works

Plans and specifications of the works must be certified by a Qualified
Professional registered to practice in the Province of British Columbia, and
submitted to the Director. A Qualified Professional must certify that the works
have been constructed in accordance with the plans before discharge
commences.

2.19 Notification

The Director must be notified of a change in ownership of the works a
minimum of 10 days prior to an ownership change.
2.20 **Amended or Additional Requirements**

Based on the results of the monitoring programs, the Director may:

a) Amend the monitoring and reporting requirements;
b) Amend the requirements of any of the information required by this permit; including plans, program and studies;
c) Require additional investigations, tests, surveys or studies; or
d) Require additional treatment facilities.

3. **MONITORING AND SAMPLING REQUIREMENTS**

3.1 **Incoming Soil and Associated Ash Sampling and Analysis**

The Permittee must follow sampling procedures and frequency specified in the approved Soil Acceptance Plan described under Subsection 2.2 to verify soil and associated ash quality. The contaminants must include, but not be limited to, the parameters of concern listed in Subsection 1.3.3, as determined by a Qualified Professional. The Director may require testing of soil and associated ash for additional parameters.

3.2 **Treated Soil Sampling and Analysis**

The Permittee must sample and characterize each batch of treated soil in accordance with Technical Guidance #1 Site Characterization and Confirmation Testing or an equivalent sampling protocol approved by the Director. Each batch must be considered to be of suspect waste soil quality. Soil must be analysed prior to disposal as authorised in Subsection 1.2 and 1.3 of this permit. The samples must be analysed for the parameters relevant to the type of contamination for which the soil is undergoing treatment as determined by a Qualified Professional. The appropriate parameters must include, but must not be limited to, the parameters of concern listed in Subsection 1.3.3 as determined by a Qualified Professional.

Confirmation of completion of soil treatment must be obtained in writing from a Qualified Professional prior to discharge, for each stockpile of treated soil.

3.3 **Groundwater Sampling and Analysis**

The Permittee must install and maintain a minimum of seven groundwater...
sampling facilities (MW-1(S/D), MW-2, MW-3(S/D), MW-4 and MW-5) as shown on Figure B and obtain groundwater samples once each quarter in a manner satisfactory to the Director. MW-4 and MW-5 must be drilled using a non-destructive method and cores must be logged by a Qualified Professional. The design and location of the wells must be to the satisfaction of the Director. Proper care must be taken in sampling, storing and transporting the samples to adequately control temperature and avoid contamination, breakage, etc.

Groundwater samples must be analysed for all potential contaminants of concern. The contaminants may include, but not be limited to, the parameters of concern listed in Subsection 1.3.3, as determined by a Qualified Professional. The groundwater quality must be compared to the standards described in Schedules 6 and 10 of the CSR or any additional standards specified by the Director in writing.

The Permittee may be required to install additional groundwater sampling facilities upon request. The location and structural details of these sampling facilities are subject to the approval of the Director.

3.4 Surface Water Sampling and Analysis

The Permittee must sample the water treatment system effluent (WTS) and the settling pond discharge point (SW-1) monthly and every 2000 m³ for the water treatment system discharge effluent in a manner suitable to the Director. Proper care must be taken in sampling, storing and transporting the samples to adequately control temperature and avoid contamination, breakage, etc.

Turbidity of the settling pond discharge effluent (SW-1) must be monitored bi-weekly between November to April and after every event greater than 1 in 10 year return period flood event of 24 hour duration.

Surface water samples must be analysed for all potential contaminants of concern. The contaminants may include, but not be limited to, the parameters of concern listed in Subsection 1.3.3, as determined by a Qualified Professional. The surface water quality results must be compared to the standards set out in Subsection 1.4.4 and 1.4.5.

3.5 Air Quality Monitoring

The Permittee must collect monthly ambient air samples during the active
season (i.e. between April and November, inclusive) at the down-wind property line using a Summa® Canister. Ambient air samples must also be collected using a Summa® Canister if and when soils with measurable volatile contaminant concentrations exceeding the established thresholds are being managed or treated at the soil treatment facility at the location and as documented in the EPM.

The ambient air sample must be analysed for the all potential contaminants of concern, as determined by a Qualified Professional, and results must be compared to the CSR Schedule 11 RL standards. In the event that results exceed the standards, the Permittee must follow the requirements stated under Subsection 2.9.

3.6 **Receiving Environment Sampling**

The Permittee must implement a receiving environment monitoring program for the receiving groundwater and surface water summarized in the table below and as defined under the EPM:

<table>
<thead>
<tr>
<th>Receiving Waters</th>
<th>Monitoring Locations</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Groundwater</td>
<td>Up Gradient (MW-4) Southeast corner of the site</td>
<td>Quarterly</td>
</tr>
<tr>
<td></td>
<td>Down Gradient (MW-1(S/D)) On site</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(MW-2) Property boundary</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(MW-3(S/D)) Property</td>
</tr>
<tr>
<td></td>
<td></td>
<td>boundary</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(MW-5) North of the site</td>
</tr>
<tr>
<td>Surface Water</td>
<td>Up Gradient (SW-4) Shawnigan Creek</td>
<td>5 in 30** (2 times/year,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>conducted during fall</td>
</tr>
<tr>
<td></td>
<td></td>
<td>first flush event and in the</td>
</tr>
<tr>
<td></td>
<td></td>
<td>spring freshet)</td>
</tr>
<tr>
<td></td>
<td>Down Gradient (SW-2) Ephemeral Creek 1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>(SW-5) Shawnigan Creek</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(SW-3) Ephemeral Creek 2</td>
</tr>
</tbody>
</table>

* 5 in 30 refers to at least 5 weekly samples taken in a period of 30 days. Due to the ephemeral nature of some of the creeks, the first 5 in 30 sample should be collected when the ground has first been saturated.

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Flow measurements must be collected from all surface water monitoring locations at the time of sampling. Based on the results from the receiving environment monitoring program, the monitoring requirements may be extended or altered by the Director.

3.7 Sampling Procedures

Sampling is to be carried out in accordance with the procedures described in the "British Columbia Field Sampling Manual for Continuous Monitoring and the Collection of Air, Air-Emission, Water, Wastewater, Soil, Sediment, and Biological Samples, 2003 Edition (Permittee)", or most recent edition, or by suitable alternative procedures as authorized by the Director.

A copy of the above manual is available on the Ministry web page at www.env.gov.bc.ca/epd/wamr/labsys/lab_meth_manual.html

3.8 Analytical Procedures

Analyses are to be carried out in accordance with procedures described in the "British Columbia Laboratory Manual (2009 Permittee Edition)", or the most recent edition, or by suitable alternative procedures as authorized by the Director.

A copy of the above manual is available on the Ministry web page at www.env.gov.bc.ca/epd/wamr/labsys/lab_meth_manual.html

3.9 Quality Assurance

a) The Permittee must obtain from the analytical laboratory (ies) their precision, accuracy and blank data for each sample set submitted as well as an evaluation of the data acceptability, based on the criteria set by the laboratory.

b) A duplicate sample must be prepared and submitted for analysis for each parameter sampled for each monitoring period.

c) The analytical laboratory (ies) must be registered in accordance with the Canadian Association of Laboratory Accreditation (CALA) unless otherwise instructed by the Director.
4. **SECURITY REQUIREMENTS**

4.1 **Closure Plan**

The Permittee must submit a closure plan to the satisfaction of the Director in 6 months after the issuance of this permit. Based on monitoring results or changes in the operation, the Director may require amendment of the plan for environmental protection.

The closure plan must include, but may not be limited to investigations of soil, sediments, surface water and groundwater quality and treatment, identification and assessment of any residual contamination. If any residual contamination is identified, the Permittee will be required to remediate the site to meet the applicable soil, surface water and groundwater standards and objectives, as determined by the Director.

The closure plan must be reviewed at least every five (5) years to inform the security adjustment defined in Subsection 4.2.

4.2 **Posting of Security and Costs**

The Permittee must submit a cost estimate for maintenance, monitoring, remediation and closure of the landfill for the active life of the site and a minimum twenty-five year post-closure period based on the current updated Closure Plan referred to in Subsection 4.1. The cost estimate must be prepared or reviewed by a suitably qualified, independent third party. The cost estimate is subject to the Director's approval.

An updated cost estimate must be reassessed and submitted to the Director for approval at least once every five (5) years and the security adjusted accordingly. The Director has the discretion to require reassessment on a more frequent basis.

The Permittee must provide and maintain security in a form and amount specified by the Director. At the discretion of the Director security may be applied, to any of the following:

- To correct any inadequacy of the works relating to their construction,
operation and maintenance;

- To correct any non-compliance with this permit or the Environmental Management Act; and remediation.

Any money spent from the posted security must be replenished within sixty (60) days or as otherwise specified by the Director.

The operation of the facility without valid security is not authorized.

The Permittee may request the return of security where the title of the works has been transferred to a municipal authority or where the posted amount exceeds the estimated closure and post-closure costs, including remediation. Granting the request is at the discretion of the Director.

5. REPORTING REQUIREMENTS

5.1 Records

Maintain for inspection by Environmental Protection Division staff, a record of the following logs, suitably tabulated:

1) Landfill cells construction QA/QC results;
2) Maintenance records of pollution control equipments listed as authorized works;
3) Facility inspection log with a record of observations of the soil management and treatment and landfill areas (including but not limited to bedrock integrity, liner, cover, stormwater and effluent collection and treatment works inspections), and preventative and corrective actions identified and implemented;
4) Current soil and associated ash inventory, including volumes and characteristics of soils and associated ash in the soil management and treatment area and landfill area;
5) Tracking ID number linked to soil and associated ash analysis results and the signature of a Qualified Professional who certifies completion of remediation in accordance with the requirements of the CSR and compliance with this permit;
6) Location of each batch of soil and associated ash in the soil management and treatment and landfill area on a map;
7) Analyses of screening of incoming soils and associated ash, and

Date issued: August 21, 2013

Hubert Bunce
for Director, Environmental Management Act
West Coast Region
associated QA/QC results, as described in Subsection 2.1 and 2.2 of this permit;
8) Soil treatment activities including turning records and quantities of nutrients, bacteria seed or amendments added by date;
9) Weather conditions during turning events as described in Subsection 2.5 of this permit;
10) Results of the vapour and dust monitoring activities as required;
11) Analyses of treated soil, and associated QA/QC results, as described in Subsection 1.2 of this permit;
12) Quarterly volumes of soil stored in the holding area, awaiting final disposal as described in Subsection 2.3 of this permit;
13) A summary of Emergency Response Plan exercises, and incidents, including effluent/soil spills, requiring the Emergency Response Plan implementation.

The above records of analyses for the re-characterization or characterization of incoming soil or treated soil, respectively, must include batch sizes, number of samples collected and analysed per volume.

Records must be kept on site or at another location acceptable to the Director for at least three years and made available upon request.

5.2 **Environmental Quarterly Reports**

The Permittee must submit environmental quarterly reports prepared by a Qualified Professional with all monitoring data and associated QA/QC results, interpretations, conclusions and recommendations in a format acceptable to the Director and post the results online and provide a hard copy to the Director no later than 30 days after the end of each quarter.

5.3 **Environmental Annual Reports**

The Permittee must submit an environmental annual report prepared by a Qualified Professional with monitoring data and associated QA/QC results, interpretations, conclusions and recommendations in a format acceptable to the Director no later than March 31 of each year.

The environmental annual report must include, but is not limited to, the following:

\[\text{Date issued: August 21, 2013} \]

[Signature]

Hubert Bunce
for Director, Environmental Management Act
West Coast Region

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Permit Number: 105809
1) An executive summary;
2) Quality and quantity (in tonnes and m$^3$) of soil and associated ash received for treatment, direct landfilling and as direct landfill cover;
3) Quality and quantity (in tonnes and m$^3$) of soil and associated ash that could not be treated in the soil treatment facility and soil and associated ash rejected and diverted to other facilities for treatment and/or disposal;
4) Updated maps showing the active landfill area, the areas reclaimed and the location of each landfill cells (completed and in progress);
5) Landfill operational plan and remaining landfill life and capacity;
6) Review of the preceding year of operation, plans for the next year and a summary of any new information or changes to the facilities and plans, assessments, programs and reports;
7) Review of any non-compliances with the conditions of this permit, including an action plan and schedule to achieve compliance (as per Subsection 6.1); and
8) Results from the Environmental Monitoring Plan with interpretations, conclusions and recommendations.

The Permittee must post the environmental annual report online and provide a hard copy to the local library by March 31 of each year. The Permittee may omit proprietary information from the publically available environmental annual report in accordance with the Freedom of Information and Protection of Privacy Act, as agreed to by the Director.

6. NON-COMPLIANCE REPORTING

6.1 Non-compliance Reporting

For any non-compliance with the requirements of this permit, the Permittee must submit to the Director, Environmental Protection, a written report within 30 days of the non-compliance occurrence. The report must include, but is not necessarily limited to, the following:

a) all relevant test results related to the non-compliance;
b) an explanation of the most probable cause(s) of the non-compliance; and
c) remedial action planned and/or taken to prevent similar non-compliance(s) in the future.

Date issued: August 21, 2013

Hubert Bunce
for Director, Environmental Management Act
West Coast Region
June 4, 2015

Cobble Hill Holdings Ltd. (BC0754588)
Herald Street Law
101 - 536 Herald Street
Victoria BC  V8W 1S6

Dear Cobble Hill Holdings Ltd. (BC0754588),

Re: Environmental Appeal Board Directions - Amendments to the Permit under the Environmental Management Act

On March 20, 2015, the Environmental Appeal Board confirmed the permit subject to directions. A copy of the decision, including directions, is available at the Environmental Appeal Board’s website http://www.eab.gov.bc.ca/index.htm.

Pursuant to the Environmental Management Act, Permit 105809 is hereby amended:

1. To amend the subject sentence of section 2.14 Advisory Committee from:

   The Committee must be composed of one representative of each relevant regulatory agency and one representative from the local government.

   to:

   The Committee must be composed of one representative of each relevant regulatory agency, one representative from the local government, one representative from the Shawnigan Residents Association and/or other interested community members as chosen by the Director.

2. To add section:

   **2.4.1 Reuse of Landfill Cell Liners Prohibited**

   Reuse of geomembrane landfill cell liners is prohibited. This prohibition must be included in the Environmental Procedures Manual.
3. **Effective March 20, 2016**, to amend section **2.7 Weather Protection** from:

2.7 *Weather Protection*

The Permittee must cover the soil treatment piles, soil holding area and active landfill areas completely from November to April when not actively worked on and provide sufficient weather protection and containment for nutrients stored at the site for the protection of human health and the environment. The Permittee must cover any soil stored within the holding area at all times.

to:

2.7 *Weather Protection*

A permanent roof must be placed over, cover, and prevent precipitation from entering the soil management and bio-remediation treatment area including the temporary soil holding area (as described under subsection 2.3), referred to in subsection 1.2.1.

The Permittee must cover the active landfill areas completely from November to April when not actively worked on and provide sufficient weather protection and containment for nutrients stored at the site for the protection of human health and the environment.

4. To add section:

2.7.1 *Wheel Rinsing*

Before soil transport vehicles leave the site, their wheels must be rinsed to remove all soil and waste. Soil and waste must be managed in accordance with the permit. Rinse water must be directed to the leachate and leak detection reservoir(s). These requirements must be included in the Environmental Procedures Manual.

5. To add to section **3.6 Receiving Environment Sampling**, Table, Row 3 Surface Water, Column 3 Frequency:

Immediately after a 1-in-200 year, 24-hour storm event, at Monitoring Locations (SW-2) Ephemeral Creek 1 and (SW-3) Ephemeral Creek 2.

Please note that although a revised permit has not been produced at this time, a copy of this letter is being placed on the permit file, as an addendum to the permit. Your attention is respectfully directed to the conditions of the permit. An annual fee for the permit will be determined in accordance with the Permit Fees Regulation.
This permit does not authorize entry upon, crossing over, or use for any purpose of private or Crown lands or works, unless and except as authorized by the owner of such lands or works. The responsibility for obtaining such authority rests with the permittee. This permit is issued pursuant to the provisions of the *Environmental Management Act* to ensure compliance with Section 120(3) of that statute, which makes it an offence to discharge waste, from a prescribed industry or activity, without proper authorization. It is also the responsibility of the permittee to ensure that all activities conducted under this authorization are carried out with regard to the rights of third parties, and comply with other applicable legislation that may be in force.

This decision may be appealed to the Environmental Appeal Board in accordance with Part 8 of the *Environmental Management Act*. An appeal must be delivered within 30 days from the date that notice of this decision is given. For further information, please contact the Environmental Appeal Board at (250) 387-3464.

Administration of this permit will be carried out by staff from the regional office. Plans, data and reports pertinent to the permit are to be submitted to the Regional Director, at Ministry of Environment, Environmental Protection Division, Authorizations - South, 2080A Labieux Rd, Nanaimo BC V9T 6J9.

Yours truly,

A.J. Downie, M.Sc., P.Ag.
for Director, Environmental Management Act

CC: Environment Canada
    Ministry of Energy and Mines

ENCL: None
PROVINCE OF BRITISH COLUMBIA
MINISTRY OF ENERGY, MINES AND PETROLEUM RESOURCES

QUARRY PERMIT
Amendment: April 20 2009

APPROVING WORK SYSTEM AND RECLAMATION PROGRAM
(Issued pursuant to Section 10 of the Mines Act R.S.B.C. 1996, C.293)

Permit: Q-8-094                                      Mine No.: 1610355

Issued to: South Island Aggregates Ltd
           497 A Garbally Road
           Victoria BC V8T 2J9

For work located at the following property: South Island Aggregates Quarry

Lot 23, Blocks 156, 201 and 323, Malahat District, Plan VIP78459

This approval and permit is subject to the appended conditions.

Issued this 4th day October in the year 2006
Amended this 20th day of April, in the year 2009
Amended this 17th day of July in the year 2015

__________________________________________
Al. Hoffman. P. Eng
Chief Inspector
INTRODUCTION

This amendment issued July 17, 2015, replaces all previous permits and subsequent amendments. It incorporates conditions established through previous amendments and, as a result of the meeting with the Chief Inspector of Mines following discussions related to hours of work. In addition, it includes conditions established by the Senior Inspector of Mines to address concerns associated with the operation of this quarry.

This amendment issued July 17, 2015 includes the change of end land use and includes the conditions necessary to construct and operate the Waste Cells in accordance with, and in addition to, the Ministry of Environment Permit “PR-105809”. This amendment includes conditions as required by the ruling of the Environmental Appeal Board Decision Nos. 2013-EMA-15(b) and 2013-EMA-019(c).

PREAMBLE

Notice of intention to commence work on a quarry, including a plan of the proposed work system and a program for the protection and reclamation of the surface of the land and watercourses affected by the work dated August 23, 2006, was filed with the Inspector on August 23, 2006. Notice of such filing was published in The Pictorial on September 3, 2006, and in the BC Gazette on September 7, 2006.

This permit contains the requirements of the Ministry of Energy and Mines for reclamation. It is also compatible, to the extent possible, with the requirements of other provincial ministries for reclamation issues. The amount of security required by this permit, and the manner in which this security may be applied, will also reflect the requirements of those ministries. Nothing in this permit, however, limits the authority of other provincial ministries to set other conditions, or to act independently, under their respective permits and legislation.

This amendment references and includes terms of the following Reports:


Unless modified by Permit Q-8-094, or the Ministry of Environment Permit PR-105809, all terms of the referenced report form a part of this permit. Should there be a conflict between this permit and the Ministry of Environment (MOE) permit related to requirements under the terms of the MOE permit related to environmental protection, the terms of the MOE permit shall take precedence.

Decisions made by staff of the Ministry of Energy and Mines will be made in consultation with other ministries.

CONDITIONS

The Chief Inspector of Mines (Chief Inspector) hereby approves the work plan and the program for protection and reclamation of the land surface and watercourses subject to compliance with the following conditions: Unless modified by this amended permit all conditions within the original Notice of Work, dated August 23, 2006, and the subsequent amendment form an integral part of this permit.

1. Reclamation Security

(a) The owner, agent or manager (herein called the Permittee) shall maintain with the Minister of Finance securities in the amount of five thousand dollars ($55,000). The security will be held by the Minister of Finance for the proper performance of the approved program and all the conditions of this permit in a manner satisfactory to the Chief Inspector.

(b) The Permittee shall conform to all forest tenure requirements of the Ministry of Forests. Should the Permittee not conform to these requirements then all or part of the security may be used to cover the costs of these requirements.

(c) The Permittee shall conform to all Ministry of Environment approval, licence and permit conditions, as well as requirements under the Wildlife Act. Should the Permittee not conform to these conditions, then all or part of the security may be used to fulfill these requirements.
2. **Land Use**

The surface of the land and watercourses shall be reclaimed to the following land use: **Industrial Encapsulated Contaminated Soil containment cells**

3. **Productivity**

The level of land productivity to be achieved on reclaimed areas shall not be less than existed prior to mining on an average property basis unless the Permittee can provide evidence which demonstrates, to the satisfaction of the Chief Inspector, the impracticality of doing so.

4. **Revegetation**

Land shall be re-vegetated to a self-sustaining state using appropriate plant species.

5. **Use of Suitable Growth Medium**

(a) On all lands to be revegetated, the growth medium shall satisfy land use, productivity, and water quality objectives. Topsoil and overburden (to rooting depth) shall be removed from operational areas prior to any disturbance of the land and stockpiled separately on the property for use in reclamation programs, unless the Permittee can provide evidence which demonstrates, to the satisfaction of the Chief Inspector, that reclamation objectives can otherwise be achieved.

(b) No topsoil shall be removed from the property without the specific written permission of the Inspector.

6. **Buffer Zones and Berms**

Buffer zones and/or berms shall be established between the mine and the property boundary unless exempted in writing by the Inspector.

7. **Treatment of Structures and Equipment**

Prior to abandonment, and unless the Chief Inspector has made a ruling otherwise, such as heritage project consideration or industrial use:
(a) all machinery, equipment and building superstructures shall be removed;

(b) concrete foundations shall be covered and revegetated unless, because of demonstrated impracticality, they have been exempted by the Inspector; and,

(c) all scrap material shall be disposed of in a manner acceptable to the Inspector.

8. Watercourses

(a) Watercourses shall be reclaimed to a condition that ensures:

(1) long-term water quality is maintained to a standard acceptable to the Chief Inspector;

(2) drainage is restored either to original watercourses or to new watercourses which will sustain themselves without maintenance: and,

(3) use and productivity objectives are achieved and the level of productivity shall not be less than existed prior to mining unless the Permittee can provide evidence which demonstrates to the satisfaction of the Chief Inspector the impracticality of doing so.

(b) Water which flows from disturbed areas shall be collected and diverted into settling ponds.

9. Roads

(a) All roads shall be reclaimed in accordance with land use objectives unless permanent access is required to be maintained.

(b) Individual roads will be exempted from the requirement for total reclamation under condition 9(a) if either:

(1) the Permittee can demonstrate that an agency of the Crown has explicitly accepted responsibility for the operation, maintenance and ultimate deactivation and abandonment of the road, or
(2) the Permittee can demonstrate that another private party has explicitly agreed to accept responsibility for the operation, maintenance and ultimate deactivation and abandonment of the road and has, in this regard, agreed to comply with all the terms and conditions, including bonding provisions, of this reclamation permit, and to comply with all other relevant provincial government (and federal government) regulatory requirements.

10. Disposal of Fuels and Toxic Chemicals

Fuels, chemicals or reagents which cannot be returned to the manufacturer/supplier are to be disposed of as directed by the Chief Inspector in compliance with municipal, regional, provincial and federal statutes.

11. Temporary Shutdown

If this quarry ceases operation for a period longer than one year the Permittee shall either continue to carry out the conditions of the permit or apply for an amendment setting out a revised program for approval by the Chief Inspector.


All safety and other provisions of the Mines Act shall be complied with to the satisfaction of the Chief Inspector.

13. Monitoring

The Permittee shall undertake monitoring programs, as required by the Inspector, to demonstrate that reclamation objectives are being achieved.

14. Alterations to the Program

Substantial changes to the program must be submitted to the Inspector for approval.

15. Notice of Closure

Pursuant to Part 10.6.1 of the Health, Safety and Reclamation Code for Mines in British Columbia, a Notice of Completion of Work shall be filed with the Inspector not less than seven days prior to cessation of work.
16. **Annual Report**

Annual reports shall be submitted in a form and containing the information as and if required by the Inspector.

17. **Site Stability**

a) The inspector shall be advised in writing at the earliest opportunity of any unforeseen conditions that could adversely affect the extraction of materials, site stability, erosion control or the reclamation of the site.

b) The stability of the slopes shall be maintained at all times and erosion shall be controlled at all times.

c) The discovery of any significant subsurface flows of water, seeps, substantial amounts of fine textured, soils, silts and clays, as well as significant adverse geological conditions shall be reported to the inspector as soon as possible and work shall cease until the inspector advises otherwise.

**SITE SPECIFIC CONDITIONS:**

1. The importation of soil is permitted subject to the following conditions:

   a) Soil imported must meet Ministry of Environment Soil Guidelines for the intended end land use, as identified in the Ministry of Environment Permit PR-105809.

   b) Importation of material other than defined in 18(a) is prohibited unless approved by the Inspector.

   c) The approval as required in 18(b) shall be processed as an amendment to this permit.

   d) Documentation identifying the soil condition and suitability for the intended end land use must be maintained at the mine site office and made available to the Inspector on demand.

2. Property boundaries shall be permanently marked and maintained, and pit boundaries (mine footprint) shall be permanently marked and maintained. All
persons working on the property will be instructed as to the meaning of the markings; and,

a) The Permittee shall install a substantial fence along the property boundary.

b) This fence can be installed in stages with completion by September 1, 2016.

c) The portion of the property abutting the lands owned by the Cowichan Valley Regional District (CVRD) shall be fenced by September 2015. This includes lands abutting the restrictive covenant along Shawinigan Creek.

3. An 8-metre wide vegetation buffer shall be maintained on the northeast property boundary. The exiting trees shall not be removed.

4. All blasts shall be electronically monitored.

5. Blast limits are established at 50 millimeters per second peak particle velocity and 120 decibels on the L scale, at the property boundary, and:

a) The electronic monitor unit shall be located such that the air pressure (microphone) sensor has a clear unobstructed line of sight to the centre of the blast. The Inspector may allow or require monitoring at specific locations on a case by case basis as may be required.

b) The Manager shall maintain at the Mine Site Office, a signed copy of the Blast Log for each blast and a copy of the Electronic Monitor Record. Such records shall be made available to the Inspector on request.

c) Residents within 1km of the centre of the Quarry, and the Inspector, shall be given 24 hours notice of each scheduled blast. This 24 hours notice will establish a window of 1.5 hours within which the blast can be fired.

i. If, due to circumstances beyond the control of the Manager, a blast has been loaded and cannot be detonated within the time frame as described above, the Manager shall secure the site, post a watchman, and fire the blast the next day following the issuing of the required 24 hours notice. The Inspector may, at his discretion, allow the blast to be fired outside of the 24 hour notice window or, outside
of normal hours of work. In such cases the Inspector shall establish the conditions necessary for firing the blast.

6. For purposes of establishing the 1 km radius, the centre of the quarry is defined as: **W 48° 33.103, N 123° 36.390**

**Standard Quarry Blasting Conditions:**

7. To the extent practical, all blasts initiated on the quarry shall be videoed, and:

a) A copy of the video shall be kept at the mine office, and made available to the Inspector on request.

b) The video file shall include the following identification information as a word document:

1. the pit name, and mine number
2. the bench/location identification, including a map showing the location on the mine footprint.
3. the name of the blaster
4. the date of the blast
5. the time of the blast

c) Other information and records as may be required as conditions of the permit, or directives of the Inspector.

d) The video shall clearly show the conduct of the blast in sequence of events including.

e) The free faces prior to the blast, with emphasis placed on the face profile and the rock structure.

f) The layout of the blast pattern including the tie ins.

g) The overall site layout of the area within the “danger zone.”

8. Within 1 month of the date of this amendment to Permit Q-8-094, the Manager shall file with the Inspector an approved plan for ensuring compliance with Part 8, sections 8.7.1 to 8.7.4 of the Health Safety and Reclamation Code for Mines in British Columbia.

9. Hours of work shall be between 7am and 5pm Monday to Friday. No work, except as defined below, shall occur on weekends or Statutory Holidays:

a) Light maintenance is permitted on Saturdays between 9am and 4pm. *Light Maintenance is defined as:* work requiring the only the use of hand
tools. It does not include air impact tools, air arcing, or any heavy equipment to perform a task.

b) Drilling operations shall be limited to the hours of 8am to 4pm Monday to Friday.

c) Notwithstanding the above, nothing in this condition prevents the Manager from working outside the permitted hours of work should:

i) a safety concern on site is such that a failure to complete necessary work can result in harm or risk to workers, members of the public, or the environment or,

ii) an agency having jurisdiction declares an emergency and product from this operation is required to mitigate or assist in the mitigation of the emergency.

d) Should the provisions of condition 23(c) be implemented the Manager shall advise the Inspector without delay.

e) A sign shall be posted at the entrance to the Quarry clearly indicating the permitted hours of work.

10. The Manager shall forward to the Inspector a copy of the updated mine plan required by the code. This code section refers to updates every three months.

11. The Manager shall schedule truck traffic entering or leaving the Quarry such that the trucks do not conflict with elementary school bus pick-up or drop off times.

12. Occupational Health and Safety Committee:

a) The Manager shall establish and maintain an Occupation Health and Safety Committee (HSRC) in accordance with the Health, Safety, and Reclamation Code for Mines in British Columbia 1.6.1(b).

b) HSRC 1.6.8 which requires Occupational Health and Safety Committee members to receive training shall apply to this site.

13. Within six months of the date of issue of this amendment, the Manager shall ensure one supervisor, as defined in the HSRC, is the holder of an Open Pit Shiftboss Certificate.
Permit Conditions related to the Construction, operation, and Maintenance of the Waste Cells as referenced in this Permit.

14. Blasting:
   a) No blasts shall be initiated during the installation of the liner, (geo-tech liner) including the upper liner as required by the approved plan.
   b) Installation includes the completion of any soil cover to a compactness of 0.66 meters thick.

15. Blasting of final walls in the quarry and for the waste cells:
   a) All final walls within the quarry shall be blasted using controlled blasting techniques, commonly referred to as “smooth blasting”.
   b) Following the blast all walls shall be scaled as may be required.
   c) Any row of holes to be blasted within 10 meters of the common boundary between the Quarry and property owned by the CVRD shall be surveyed in by a Licensed Land Surveyor. A copy of the survey shall be forwarded to the Inspector within one week of the blast.

16. Clay placed above the bedrock shall be placed in 250mm lifts, and compacted to 90% standard proctor until the Clay is 1 meter compacted thickness.

17. At the completion of each 1 meter (compacted) lift the Manager shall provide the Inspector an as built of the lift signed by a suitable registered professional, registered in the Province of British Columbia.
   a) For soil imported into the cell, not including clay or sand, the Engineer of record shall identify soils where 95 Proctor could not be obtained, and shall identify the type of soil, the maximum compactness the soil can sustain, and the maximum moisture content to attain the compaction.
   b) For purposes of clarity, the engineer of record is not required to provide the above information on soil for every square foot of surface area but can provide the report in accordance with good engineering practice and standards.
18. All surface water shall be drained and controlled such that surface water does not have free access to the waste cell.
   a) Following rainfall, snow melt, or inadvertent flow of water into the waste cell, the Permittee shall take such measures as may be necessary to drain any accumulations of surface water from the cell.
   b) This may require suitable time frames to allow the drying of the soil to the point that the engineer of record is satisfied the moisture content does not compromise the achievement and maintenance of the required compaction as defined in this permit.

19. Geotechnical

1. Design and Construction
   a) The construction of the waste storage facility, as described in the application, is approved.
   b) The sediment control pond shall be designed with a minimum 1 metre freeboard during the 200-year flood event.
   c) The Permittee shall ensure the facility is constructed under the supervision of a qualified professional engineer.
   d) Rock cuts and slope design shall be reviewed by a professional geotechnical engineer following blasting and excavation. The requirement for scaling and/or stabilization measures shall be evaluated to ensure the safety of workers working below these slopes.
   e) The facility shall be constructed in accordance with the design and construction specifications outlined in the application and approved by the Engineer of Record. The Engineer of Record shall review the construction drawings and specifications to verify that recommendations are properly incorporated as per design. Any changes to the proposed method of development will require previous approval of the Inspector.
   f) During construction, appropriate Quality Assurance/Quality Control (QAQC) shall be carried out. Within 30 days of completing construction, a construction QAQC report shall be submitted to the Inspector. This report shall include a summary of the liner installation, materials testing and
compaction information and the QAQC measures employed during construction.

g) The Permittee shall submit an as-built report with drawings to the Inspector prior to operation of the facility. As-built reports shall be sealed by a professional engineer and shall include a statement indicating that the facility was constructed in “general conformance with the design and specifications.” A complete set of As-built drawings shall be kept at the mine site at all times and be provided to any Mines Inspector upon request.

2. Operation and Monitoring

a) Prior to operation of the facility, the Permittee shall submit an updated Operation, Maintenance, and Surveillance (OMS) manual and a Mine Emergency Response Plan (MERP) to the Inspector that outlines procedures for the successful operation, maintenance, and surveillance of the facility and emergency preparedness and response procedures. These documents shall be kept current and updated over time as procedures are modified.

b) All waste materials entering the facility shall meet the specifications as specified by the geotechnical engineer in the stability analyses and design of the facility. No waste materials that are subject to liquefaction (regardless of triggering mechanism) shall be disposed in the facility. Materials not meeting design specifications or operational requirements must be spoiled off-site at an alternate approved location.

c) Instrumentation shall be installed as recommended by the professional geotechnical engineer to monitor conditions related to the stability of the facility. Monitoring frequency, thresholds, and response procedures shall be determined by the geotechnical engineer and be clearly described in the OMS manual.

d) During operations, appropriate Quality Assurance/Quality Control (QA/QC) shall be carried out on the waste materials to ensure material properties meet geotechnical design and compaction requirements. Results of this testing shall be provided to the Inspector upon request. An up-to-date copy of QA/QC procedures, testing results, and inspection logs shall be maintained at site and made available for any Inspector upon request.
3. **Reporting**

   a) Annual inspections of the waste storage facility shall be undertaken by a qualified Professional Geotechnical Engineer with a report submitted to the Inspector by March 31 of the year following the inspection. The report shall include a summary of observations, review of monitoring data including instrumentation, QA/QC procedures, testing results, and recommendations with respect to any necessary changes to operating procedures. Any recommendations relating to health and safety or geotechnical stability shall be followed unless a suitable alternative course of action is approved in writing by the professional undertaking the review, or by a third party qualified Professional Engineer, as may be determined by the Inspector.

20. **Completion of the cell:**

   a) The final cover of each cell shall consist of two meters of till or residential classification soil, compacted to the degree necessary to prevent/limit erosion and sustain growth of appropriate vegetation.

   b) The permitted shall prior to applying any vegetation cover to the completed cell provide the inspector a plan designed by an appropriate Qualified Person which demonstrates the vegetation cover is suitable for the area, and as cover for the waste cell.

   c) Filling of the cells shall be conducted on a one cell at a time basis. Filling of the next cell can only commence upon completion of the cell the previous cell.

   d) The previous condition does not prevent the Permittee form doing cell preparation, up to the point of being ready to receive fill material.

   e) Prior to receiving fill in any cell the Permittee must provide a signed as built of the construction of the cell to date. This as built, signed by the engineer of record shall state that this construction meets the standards required by this permit and Ministry of Environment Permit PR-105809.

   f) Each completed cell shall remain in and be subject ongoing monitoring under the terms of this permit for the life of the mine.
g) Once completed a cell shall not be disturbed unless work is necessary for maintenance or repair, and then only with the written approval of the Inspector.

h) The Manager shall, by March 31 of each year, provide the Inspector a report identifying the volume of water treated through the treatment plant, and shall include all operating costs associated with the operation and maintenance of the treatment plant.

21. The Manager shall forward to the Inspector a copy of the report submitted to the Minister of Finance in relation to the annual Health and Safety Assessment. This report provides a report stating the annual production.

22. Surface water not subject to treatment in the water treatment plant shall be monitored at the discharge point to the receiving environment and suspended solids shall not exceed 25mg/litre. In addition this monitoring shall include analysis for nitrates, and nitrate content shall not exceed the limits specified for drinking water.

23. Production from this quarry is limited to 240,000 tonnes annually.
APPENDIX B
Forestry Chronicle Soil Rooting Depth
ROOT SPREAD CAN BE ESTIMATED FROM CROWN WIDTH
OF DOUGLAS FIR, LODGEPOLE PINE, AND
OTHER BRITISH COLUMBIA TREE SPECIES

By J. HARRY G. SMITH

ABSTRACT

Knowledge of the relationships between lateral root spread and crown width is needed to guide plans for providing optimum space per tree during silvicultural operations. Crown width is a valuable indicator of root spread of open-grown Douglas fir, lodgepole and ponderosa pines, and white, Engelmann and Sitka spruces. The association between root spread and crown width is less reliable in forest-grown trees but still useful.

Detailed studies of root systems were made by excavation of roots of trees blown down in the U.B.C. Campus Forest by a typhoon in October, 1962. Roots of 89 Douglas fir, 81 western hemlock, 61 western red cedar, and 33 red alder trees were mapped and analyzed in relation to 18 tree and stand variables. In addition, average and maximum root depth, number of main roots, the ratio of maximum to average root spread, and portion of rooting zone occupied by roots were studied.

Average and maximum radius of roots of Douglas fir and lodgepole pine were analyzed in relation to crown width, d.b.h., height, age and other tree and stand characteristics. Appropriate regression and correlation analyses were completed for all species and the possible advantages of ratio estimates were noted.

Ratios of root spread to crown width were influenced by species, stand density, and kind of soil. Ratios averaged 1.1 for open- and 0.9 for forest-grown Douglas fir but were 2.4 for both open- and forest-grown lodgepole pine on peat or poorly drained soils.

The Need to Study Root Spread

With few exceptions the lateral extent of roots in relation to crown width has been studied only incidentally during research which had as its main goal the description of root systems. Yet the relative spread of roots in relation to crown is of vital importance in plans for controlling space among trees for optimum stand development. For example, if root spread were more limiting than crown spread it would be necessary to plan for a suitable amount of surplus space for roots to spread beyond the edge of crowns. On
the other hand if roots do not usually spread and occupy the soil beyond the edge of tree crowns there is evidence for following a triangular rather than square pattern of spacing, thereby permitting increase in number of trees grown per acre by a factor of \(4/\pi\) (Smith, 1963).

Improved knowledge of root and crown spread also is needed to justify use of the individual tree as the basic unit in studies such as those involving response to fertilizers. In addition, the extent to which lateral spread of roots represents effective occupation of the volume of soil available to the tree should be determined.

**Methods Used**

Two methods were employed to measure root spread. One was based on shovel sampling of the soil surface from a randomly chosen point under the crown edge until root radius was determined. The other method took advantage of the opportunity to study roots and crowns of trees blown down in October, 1962 by Typhoon Frieda. Because of the severity of the storm the windfalls were quite representative and not restricted by soil or site condition. Soil and litter were scraped off the roots of windthrown trees to expose all lateral roots from the upper half of the windfall to the ends of its root system. Broken primary roots were traced back by digging until ends about 2 mm thick were found in the undisturbed soil. Each root system was drawn in plan and in two cross-section views at a scale of 1" equals 10'. Soil and tree variables were also measured.

Most attention has been given to determination of average lengths of lateral roots and of branches in the widest complete whorl of branches. These lengths have been doubled to define average lateral root spread and average crown width. Both of these values have a substantial sampling error for any one tree. Average rather than maximum values of root spread are used here because averages are more closely related to the soil area actually occupied by the tree. The conditions that stimulate maximum extension of lateral roots within a given soil type seem to be randomly distributed and therefore unpredictable.

**Trees Measured**

Windfalls were studied in both the U.B.C. Campus and Research Forests. In the Campus Forest, 89 Douglas fir *Pseudotsuga menziesii* (Mirb.) Franco, 81 western hemlock *Tsuga heterophylla* (Raf.) Sarg., 61 western red cedar *Thuja plicata* Donn. and 33 red alder *Alnus rubra* Bong. windfalls were thoroughly analyzed. In the Research Forest near Haney 104 windfalls of Douglas fir, 32 of hemlock, and 13 of cedar were measured to determine relationships among d.b.h., root spread, and crown width. Field identification of roots was facilitated by the work of Gilbertson, Leaphart and Johnson (1961).

Open-grown Douglas fir trees were studied most completely in the Interior of B.C. at Paul Lake north of Kamloops, where 84 trees were measured. In addition, 12 were measured at Haney and 18 Douglas fir were measured on the Wind River Experimental Forest in Washington State. Both open- and forest-grown lodgepole pine *Pinus contorta* Dougl. were measured on the Coast and in the Interior of B.C. Roots of 62 open- and 22 forest-grown lodgepole pine were studied. Forty-two open-grown ponderosa pine
Pinus ponderosa Laws., 7 open-grown Engelmann spruce Picea engelmannii Parry, 23 open-grown white spruce Picea glauca (Moench) Voss, and 7 forest-grown Sitka spruce Picea sitchensis (Bong.) Carr. were measured. Seedlings of each of these species also were measured to determine the relationship between crown width and root spread in very young trees.

LITERATURE ON ROOT SPREAD

Root systems of Douglas fir have been studied in most detail by Hengst (1958) and McMinn (1963). Regarding the influence of the crown on root systems, Hengst concluded: “By some decimeters farther than the crown extends, the soil beneath the individual tree is penetrated by lateral roots. Irregularities of the soil may cause greater deviations.” McMinn found that total length of root systems increased with increasing age, in proportion to the increase in size of crowns. He concluded that the area actually required by a tree is difficult to assess, although the area traversed by root systems seems proportional to the size of crowns. McMinn noted that radial symmetry of root systems can be distorted readily by such factors as slope, proximity of other trees, the occurrence of old roots, and soil disturbance. He found that, in dense stands, considerable intermingling of Douglas fir roots from adjacent trees occurred even in very densely rooted central portions of root systems. McMinn noted that long, sparsely branched, “pioneer” roots were common only in his 10-year-old stand.

Lodgepole pine rooting habits have been described for Alberta by Horton (1958) and for northeastern Oregon by Bishop (1962). Regarding lateral rooting Horton concluded:

“Most of the lateral roots are in the top few inches of soil. They will seek out areas free of root competition, but on a fully-stocked site the root systems of adjoining trees become interwoven, and natural grafts may occur. In wet and very dry conditions both rooting extent and amount of branching are much less than on the more productive sites. For the first 20 to 25 years, maximum lateral root length was found to be about the same as stem height in the open-grown conditions sampled.”

“...In general, rooting is extensive at first, attaining maximum areal coverage by 30 years, well before maximum stem height growth is reached; then, nearing maturity, as tree growth requirements increase, rooting becomes much more intensive and complex.”

Bright (1914) concluded that root competition among ponderosa pine was not severe but suggested that for determination of required growing space per tree an additional allowance varying with d.b.h. should be made to actual crown width. However, Pearson (1950) reported that fastest growing ponderosa pine trees are nearly always isolated, open, or on the margin of clumps of trees and therefore can extend their roots freely in at least one direction. He reasoned from his measurements of d.b.h. growth that demand for root space invariably exceeds the demand for crown space. Pearson suggested that the large crown followed rather than preceded root development since large-crowned trees grow rapidly because they started more or less in the open where abundant soil space permitted extensive root spread. Brent and Gibbons (1958) included ponderosa pine in their studies of root distribution of lodgepole pine, Douglas fir, and aspen, but did not measure crown development.
After study of rooting habits of white and red pine, Brown and Lacate (1961) concluded: "If unhindered by competition from neighboring trees, the longest laterals may extend beyond the limits of the crown and attain a length roughly comparable to the height of the tree."

Simms (1964) observed that during their first three years jack pine seedlings developed a deep tap-root system. Then from 4 to 8 years average length of laterals increased directly with age. Presumably, lodgepole and ponderosa pines also have initially rapid tap-root growth.

Van Slyke (1964) suggested that good use can be made of the general relationships that exist between crown dimensions and tree or stand values. Young (1964) has extended the unifying concepts in analysis of crown development to study of the complete tree. Kramer and Kozlowski (1960) and Romberger (1963) have described physiological aspects of root growth. Kramer and Kozlowski suggested: "It is common for roots of trees to extend out two or three times the radius of the branches, although the region of maximum concentration of absorbing roots is, as a rule, probably under the periphery of the crown."

Kozlowski (1962) included in his text on tree growth a chapter by Bormann which described root grafting and non-competitive relationships between trees. Bormann concluded that, with white pine, competition plays an important role during the first five or ten years of the existence of a stand. He suggested that, thereafter, unions begin to form as a result of root grafting and "forces operating through root grafts will determine which trees will remain as dominant entities". Cerezke (1964) found 59 references dealing with natural root grafting in conifers and concluded that root grafts are common. He suggested, however, that where moisture and nutrients supplies are adequate root grafts are probably of little consequence. Although they are of interest, it is reasonable to conclude here that root grafts can be ignored in planning for optimum spacing.

Aaltonen (1926), following his investigations of regeneration conditions of pine forests on the dry heaths of Finnish Lapland, concluded: "... space arrangement of those parts of trees which are above the soil are mainly decided by their roots and the competition existing between them for the water and food in the ground." However, in his textbook on silviculture, D. M. Smith (1962) suggested that crown competition usually is more decisive than root competition and added "competition for growing space in the soil is also important, but more intricate and exceedingly difficult to observe". The observations which follow suggest that crown competition is indeed most important and that root spread can be estimated adequately from crown width in most cases.

**Observations**

1. **Windfalls**

The data on Campus Forest trees are given in Tables 1-5 to describe basic information and illustrate the degree of association between average root length and 18 tree and site variables. Soil texture was either gravelly (0) or sandy (1). No heavy textured soils were studied. Crown class was coded with dominant equal to 1, codominant 2, intermediate 3, and suppressed 4. Soil moisture varied from dry 0, to very wet 3. Since age was not
TABLE 1
SUMMARY OF ROOT SPREAD STATISTICS FOR 89 DOUGLAS FIR

<table>
<thead>
<tr>
<th>No.</th>
<th>Variable</th>
<th>Units</th>
<th>Mean Values</th>
<th>Standard Deviation</th>
<th>Minimum</th>
<th>Maximum</th>
<th>R on Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>X1</td>
<td>D.b.h.</td>
<td>Inch</td>
<td>13.1</td>
<td>4.26</td>
<td>5.00</td>
<td>23.00</td>
<td>0.593***</td>
</tr>
<tr>
<td>X2</td>
<td>Height</td>
<td>Foot</td>
<td>88.9</td>
<td>17.7</td>
<td>47.0</td>
<td>121</td>
<td>0.356**</td>
</tr>
<tr>
<td>X3</td>
<td>Ht. to the base of last comp. live whorl of branches</td>
<td>Foot</td>
<td>60.3</td>
<td>13.9</td>
<td>31.0</td>
<td>90.0</td>
<td>0.157ns</td>
</tr>
<tr>
<td>X4</td>
<td>Ht. to the base of avge. live crown</td>
<td>Foot</td>
<td>53.5</td>
<td>13.0</td>
<td>25.0</td>
<td>81.0</td>
<td>0.091ns</td>
</tr>
<tr>
<td>X5</td>
<td>Soil Texture</td>
<td>Inch</td>
<td>0.764</td>
<td>0.427</td>
<td>0.00</td>
<td>1.0</td>
<td>0.119ns</td>
</tr>
<tr>
<td>X6</td>
<td>Crown Class</td>
<td>Foot</td>
<td>1.58</td>
<td>0.795</td>
<td>1.00</td>
<td>4.00</td>
<td>-0.418**</td>
</tr>
<tr>
<td>X7</td>
<td>Soil Moisture Regime</td>
<td>Inch</td>
<td>1.22</td>
<td>0.794</td>
<td>0.00</td>
<td>3.00</td>
<td>0.273**</td>
</tr>
<tr>
<td>X8</td>
<td>Vegetative Site (S.I.)</td>
<td>Foot</td>
<td>132</td>
<td>16.4</td>
<td>100</td>
<td>160</td>
<td>-0.036ns</td>
</tr>
<tr>
<td>X9</td>
<td>Butt Diameter</td>
<td>Inch</td>
<td>16.1</td>
<td>5.15</td>
<td>6.80</td>
<td>31.5</td>
<td>0.589**</td>
</tr>
<tr>
<td>X10</td>
<td>Bark Thickness at Butt</td>
<td>Inch</td>
<td>0.961</td>
<td>0.387</td>
<td>0.30</td>
<td>1.80</td>
<td>0.509**</td>
</tr>
<tr>
<td>X11</td>
<td>Longest Branch</td>
<td>Foot</td>
<td>13.1</td>
<td>3.2</td>
<td>5.00</td>
<td>22.0</td>
<td>0.534**</td>
</tr>
<tr>
<td>X12</td>
<td>Average Branch</td>
<td>Foot</td>
<td>10.4</td>
<td>2.88</td>
<td>4.00</td>
<td>17.0</td>
<td>0.567**</td>
</tr>
<tr>
<td>X13</td>
<td>Short Side of Crown</td>
<td>Foot</td>
<td>7.27</td>
<td>2.24</td>
<td>2.00</td>
<td>13.0</td>
<td>0.499**</td>
</tr>
<tr>
<td>X14</td>
<td>Long Side of Crown</td>
<td>Foot</td>
<td>10.1</td>
<td>2.36</td>
<td>4.00</td>
<td>16.0</td>
<td>0.546**</td>
</tr>
<tr>
<td>X15</td>
<td>Ht. / Avge. Branch = X9 / X12</td>
<td></td>
<td>8.96</td>
<td>2.21</td>
<td>4.67</td>
<td>16.3</td>
<td>-0.317**</td>
</tr>
<tr>
<td>X16</td>
<td>X2 - X4</td>
<td></td>
<td>0.398</td>
<td>0.076</td>
<td>0.231</td>
<td>0.606</td>
<td>0.410**</td>
</tr>
</tbody>
</table>

In Tables 1-4, NS indicates non significant correlation, * is significant at p.05, ** is significant at p.01.
<table>
<thead>
<tr>
<th>No.</th>
<th>Variable</th>
<th>Units</th>
<th>Mean Values</th>
<th>Standard Deviation</th>
<th>Minimum</th>
<th>Maximum</th>
<th>R $^2$ on Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>X1</td>
<td>D.b.h.</td>
<td>Inch</td>
<td>13.4</td>
<td>4.79</td>
<td>4.40</td>
<td>26.00</td>
<td>0.714**</td>
</tr>
<tr>
<td>X2</td>
<td>Height</td>
<td>Foot</td>
<td>96.5</td>
<td>19.3</td>
<td>49.0</td>
<td>141</td>
<td>0.498**</td>
</tr>
<tr>
<td>X3</td>
<td>Ht. to the base of last comp. live whorl of branches</td>
<td>Foot</td>
<td>69.0</td>
<td>16.7</td>
<td>28.0</td>
<td>100</td>
<td>0.295**</td>
</tr>
<tr>
<td>X4</td>
<td>Ht. to the base of avge. live crown</td>
<td>Foot</td>
<td>61.2</td>
<td>16.5</td>
<td>22.0</td>
<td>95.0</td>
<td>0.254*</td>
</tr>
<tr>
<td>X5</td>
<td>Soil Texture</td>
<td></td>
<td>0.975</td>
<td>0.156</td>
<td>0.00</td>
<td>1.00</td>
<td>0.021ns</td>
</tr>
<tr>
<td>X6</td>
<td>Crown Class</td>
<td></td>
<td>2.20</td>
<td>1.22</td>
<td>1.00</td>
<td>4.00</td>
<td>-0.591**</td>
</tr>
<tr>
<td>X7</td>
<td>Soil Moisture Regime</td>
<td></td>
<td>0.543</td>
<td>0.962</td>
<td>0.00</td>
<td>3.00</td>
<td>0.260*</td>
</tr>
<tr>
<td>X8</td>
<td>Vegetative Site (S.I.)</td>
<td>Foot</td>
<td>123</td>
<td>14.2</td>
<td>100</td>
<td>150</td>
<td>-0.103ns</td>
</tr>
<tr>
<td>X9</td>
<td>Butt Diameter</td>
<td>Inch</td>
<td>16.1</td>
<td>5.54</td>
<td>5.00</td>
<td>30.0</td>
<td>0.739**</td>
</tr>
<tr>
<td>X10</td>
<td>Bark Thickness at Butt</td>
<td>Inch</td>
<td>0.374</td>
<td>0.119</td>
<td>0.20</td>
<td>0.70</td>
<td>0.447**</td>
</tr>
<tr>
<td>X11</td>
<td>Longest Branch</td>
<td>Foot</td>
<td>14.3</td>
<td>3.57</td>
<td>7.00</td>
<td>25.0</td>
<td>0.652**</td>
</tr>
<tr>
<td>X12</td>
<td>Average Branch</td>
<td>Foot</td>
<td>10.8</td>
<td>2.93</td>
<td>5.00</td>
<td>18.0</td>
<td>0.661**</td>
</tr>
<tr>
<td>X13</td>
<td>Short Side of Crown</td>
<td>Foot</td>
<td>7.28</td>
<td>2.79</td>
<td>3.00</td>
<td>15.0</td>
<td>0.695**</td>
</tr>
<tr>
<td>X14</td>
<td>Long Side of Crown</td>
<td>Foot</td>
<td>11.0</td>
<td>3.11</td>
<td>6.00</td>
<td>20.0</td>
<td>0.659**</td>
</tr>
<tr>
<td>X15</td>
<td>Ht. / Avge. Branch = X2 / X12</td>
<td></td>
<td>9.35</td>
<td>2.17</td>
<td>5.06</td>
<td>14.7</td>
<td>-0.358**</td>
</tr>
<tr>
<td>X16</td>
<td>$X_2 - X_4$</td>
<td></td>
<td>0.367</td>
<td>0.106</td>
<td>0.198</td>
<td>0.726</td>
<td>0.184ns</td>
</tr>
<tr>
<td>X17</td>
<td>Avg. Branch / D.b.h. = X16 / X4</td>
<td></td>
<td>242</td>
<td>125</td>
<td>5.65</td>
<td>1137</td>
<td>-0.151ns</td>
</tr>
<tr>
<td>X18</td>
<td>Avg. Root Diam.</td>
<td>Inch</td>
<td>1.87</td>
<td>0.752</td>
<td>0.30</td>
<td>4.20</td>
<td>0.208ns</td>
</tr>
<tr>
<td>X19</td>
<td>Avg. Root Length</td>
<td>Foot</td>
<td>9.47</td>
<td>3.51</td>
<td>3.00</td>
<td>18.0</td>
<td>1.000</td>
</tr>
</tbody>
</table>
### TABLE 3

**SUMMARY OF ROOT SPREAD STATISTICS FOR 61 CEDAR**

<table>
<thead>
<tr>
<th>No.</th>
<th>Variable</th>
<th>Units</th>
<th>Mean Values</th>
<th>Standard Deviation</th>
<th>Minimum</th>
<th>Maximum</th>
<th>( R ) on ( Y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( X_1 )</td>
<td>D.b.h.</td>
<td>Inch</td>
<td>10.7</td>
<td>5.06</td>
<td>3.20</td>
<td>27.0</td>
<td>0.600**</td>
</tr>
<tr>
<td>( X_2 )</td>
<td>Height</td>
<td>Foot</td>
<td>69.9</td>
<td>19.8</td>
<td>28.0</td>
<td>115</td>
<td>0.375**</td>
</tr>
<tr>
<td>( X_3 )</td>
<td>Ht. to the base of last comp. live whorl of branches</td>
<td>Foot</td>
<td>45.5</td>
<td>16.2</td>
<td>18.0</td>
<td>105</td>
<td>0.189**</td>
</tr>
<tr>
<td>( X_4 )</td>
<td>Ht. to the base of avge. live crown</td>
<td>Foot</td>
<td>38.0</td>
<td>14.6</td>
<td>15.0</td>
<td>89.0</td>
<td>0.152**</td>
</tr>
<tr>
<td>( X_5 )</td>
<td>Soil Texture</td>
<td></td>
<td>0.902</td>
<td>0.300</td>
<td>0.00</td>
<td>1.00</td>
<td>0.113**</td>
</tr>
<tr>
<td>( X_6 )</td>
<td>Crown Class</td>
<td></td>
<td>2.87</td>
<td>1.01</td>
<td>1.00</td>
<td>4.00</td>
<td>-0.544**</td>
</tr>
<tr>
<td>( X_7 )</td>
<td>Soil Moisture Regime</td>
<td></td>
<td>1.43</td>
<td>0.903</td>
<td>0.00</td>
<td>2.00</td>
<td>0.117**</td>
</tr>
<tr>
<td>( X_8 )</td>
<td>Vegetative Site (S.I.)</td>
<td>Foot</td>
<td>119</td>
<td>15.1</td>
<td>1.00</td>
<td>150</td>
<td>0.156**</td>
</tr>
<tr>
<td>( X_9 )</td>
<td>Butt Diameter</td>
<td>Inch</td>
<td>14.6</td>
<td>7.25</td>
<td>4.50</td>
<td>34.0</td>
<td>0.574**</td>
</tr>
<tr>
<td>( X_{10} )</td>
<td>Bark Thickness at Butt</td>
<td>Inch</td>
<td>0.215</td>
<td>0.063</td>
<td>0.10</td>
<td>0.40</td>
<td>0.339**</td>
</tr>
<tr>
<td>( X_{11} )</td>
<td>Longest Branch</td>
<td>Foot</td>
<td>11.8</td>
<td>3.13</td>
<td>6.00</td>
<td>20.0</td>
<td>0.544**</td>
</tr>
<tr>
<td>( X_{12} )</td>
<td>Average Branch</td>
<td>Foot</td>
<td>8.75</td>
<td>2.38</td>
<td>4.00</td>
<td>16.0</td>
<td>0.594**</td>
</tr>
<tr>
<td>( X_{13} )</td>
<td>Short Side of Crown</td>
<td>Foot</td>
<td>5.77</td>
<td>1.99</td>
<td>2.00</td>
<td>13.0</td>
<td>0.518**</td>
</tr>
<tr>
<td>( X_{14} )</td>
<td>Long Side of Crown</td>
<td>Foot</td>
<td>9.16</td>
<td>2.73</td>
<td>5.00</td>
<td>17.0</td>
<td>0.545**</td>
</tr>
<tr>
<td>( X_{15} )</td>
<td>Ht. / Avge. Branch = ( X_9 / X_{13} )</td>
<td></td>
<td>8.18</td>
<td>1.96</td>
<td>3.50</td>
<td>12.8</td>
<td>-0.191**</td>
</tr>
<tr>
<td>( X_{16} )</td>
<td>( X_2 - X_4 )</td>
<td></td>
<td>0.459</td>
<td>0.119</td>
<td>0.211</td>
<td>0.762</td>
<td>-0.232**</td>
</tr>
<tr>
<td>( X_{17} )</td>
<td>Avge. Branch / D.b.h. = ( X_4 / X_1 )</td>
<td></td>
<td>11.0</td>
<td>3.52</td>
<td>6.64</td>
<td>21.3</td>
<td>-0.417**</td>
</tr>
<tr>
<td>( X_{18} )</td>
<td>Avge. Root Diam.</td>
<td>Inch</td>
<td>1.18</td>
<td>0.642</td>
<td>0.40</td>
<td>3.50</td>
<td>0.182**</td>
</tr>
<tr>
<td>( Y )</td>
<td>Avge. Root Length</td>
<td>Foot</td>
<td>7.10</td>
<td>2.74</td>
<td>3.00</td>
<td>17.0</td>
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<td>Mean Values</td>
<td>Standard Deviation</td>
<td>Minimum</td>
<td>Maximum</td>
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</tr>
<tr>
<td>X₁</td>
<td>D.b.h.</td>
<td>Inch</td>
<td>10.9</td>
<td>2.90</td>
<td>4.50</td>
<td>16.4</td>
<td>0.610**</td>
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<tr>
<td>X₂</td>
<td>Height</td>
<td>Foot</td>
<td>72.8</td>
<td>10.3</td>
<td>44.0</td>
<td>93.0</td>
<td>0.331**</td>
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<tr>
<td>X₃</td>
<td>Ht. to the base of last comp. live whorl of branches</td>
<td>Foot</td>
<td>46.6</td>
<td>9.46</td>
<td>30.0</td>
<td>69.0</td>
<td>0.021**</td>
</tr>
<tr>
<td>X₄</td>
<td>Ht. to the base of avge. live crown</td>
<td>Foot</td>
<td>40.8</td>
<td>9.80</td>
<td>22.0</td>
<td>64.0</td>
<td>-0.081**</td>
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<td>Soil Texture</td>
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<td>1.00</td>
<td>-0.356*</td>
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<td>Crown Class</td>
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<td>0.712</td>
<td>1.00</td>
<td>3.00</td>
<td>-0.236**</td>
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<td>Soil Moisture Regime</td>
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<td>0.00</td>
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<tr>
<td>X₈</td>
<td>Vegetative Site (S.I.)</td>
<td>Foot</td>
<td>124</td>
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<td>100</td>
<td>150</td>
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<td>Butt Diameter</td>
<td>Inch</td>
<td>13.2</td>
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<td>5.40</td>
<td>17.7</td>
<td>0.525**</td>
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<tr>
<td>X₁₀</td>
<td>Bark Thickness at Butt</td>
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<td>0.093</td>
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<td>0.50</td>
<td>0.127**</td>
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<td>X₁₁</td>
<td>Longest Branch</td>
<td>Foot</td>
<td>23.9</td>
<td>7.69</td>
<td>10.0</td>
<td>48.0</td>
<td>0.377*</td>
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<tr>
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<td>Average Branch</td>
<td>Foot</td>
<td>16.5</td>
<td>4.62</td>
<td>9.00</td>
<td>28.0</td>
<td>0.518**</td>
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<td>Short Side of Crown</td>
<td>Foot</td>
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<td>4.00</td>
<td>12.0</td>
<td>0.546**</td>
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<td>Long Side of Crown</td>
<td>Foot</td>
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<td>6.00</td>
<td>20.0</td>
<td>0.407**</td>
</tr>
<tr>
<td>X₁₅</td>
<td>Ht. / Avge. Branch = X₉ / X₁₂</td>
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<td>4.65</td>
<td>1.10</td>
<td>2.71</td>
<td>7.36</td>
<td>-0.464**</td>
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<tr>
<td>X₁₆</td>
<td>( X₂ - X₄ )</td>
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<td>0.438</td>
<td>0.108</td>
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<td>0.676</td>
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<td>Avge. Branch / D.b.h. = X₁₆ / X₁</td>
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<td>4.71</td>
<td>10.9</td>
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<td>-0.096**</td>
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<td>Avge. Root Diam.</td>
<td>Inch</td>
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<td>0.40</td>
<td>1.80</td>
<td>0.260**</td>
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<td>Y</td>
<td>Avge. Root Length</td>
<td>Foot</td>
<td>9.73</td>
<td>2.67</td>
<td>3.00</td>
<td>16.0</td>
<td>1.00</td>
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### TABLE 5

**ASSOCIATION BETWEEN AVERAGE ROOT LENGTH AND 18 TREE AND SITE VARIABLES OF WINDFALLEN TREES**

<table>
<thead>
<tr>
<th>No.</th>
<th>Variable</th>
<th>Units</th>
<th>Species</th>
<th>D. fir</th>
<th>Hemlock</th>
<th>Cedar</th>
<th>Alder</th>
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<td></td>
<td></td>
<td></td>
<td>89</td>
<td>81</td>
<td>61</td>
<td>83</td>
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<td>Simple correlation coefficients</td>
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<td>Y on X 1-18</td>
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<tr>
<td>X1</td>
<td>D.b.h.</td>
<td>In.</td>
<td>.593</td>
<td>.714</td>
<td>.600</td>
<td>.610</td>
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<td>2</td>
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<td>Ft.</td>
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<td>.498</td>
<td>.375</td>
<td>.331</td>
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<td>3</td>
<td>Ht. to Full Whorl</td>
<td>Ft.</td>
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<td>.295</td>
<td>.189</td>
<td>.021</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Ht. to Live Crown</td>
<td>Ft.</td>
<td>.091</td>
<td>.254</td>
<td>.152</td>
<td>-.081</td>
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<tr>
<td>5</td>
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<td>.021</td>
<td>.113</td>
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<td>Crown Class</td>
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<td>-.591</td>
<td>-.544</td>
<td>-.236</td>
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<td>7</td>
<td>Soil Moisture Regime</td>
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<td>.260</td>
<td>.117</td>
<td>-.225</td>
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<tr>
<td>8</td>
<td>Site Index</td>
<td>Ft. @ 100</td>
<td>-.036</td>
<td>.103</td>
<td>.156</td>
<td>-.365</td>
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</tr>
<tr>
<td>9</td>
<td>Butt Diam.</td>
<td>In.</td>
<td>.589</td>
<td>.739</td>
<td>.574</td>
<td>.525</td>
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<tr>
<td>10</td>
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<td>In.</td>
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<td>.447</td>
<td>.339</td>
<td>.127</td>
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<td>11</td>
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<td>Ft.</td>
<td>.534</td>
<td>.652</td>
<td>.544</td>
<td>.377</td>
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</tr>
<tr>
<td>12</td>
<td>Length of Average Branch</td>
<td>Ft.</td>
<td>.567</td>
<td>.661</td>
<td>.594</td>
<td>.518</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Length of Short Side</td>
<td>Ft.</td>
<td>.499</td>
<td>.695</td>
<td>.518</td>
<td>.546</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>Length of Long Side</td>
<td>Ft.</td>
<td>.546</td>
<td>.659</td>
<td>.545</td>
<td>.407</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>X2/X12</td>
<td></td>
<td>-.317</td>
<td>-.358</td>
<td>-.191</td>
<td>-.464</td>
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<tr>
<td>16</td>
<td>X2 - X4/X2</td>
<td></td>
<td>.410</td>
<td>.184</td>
<td>.232</td>
<td>.330</td>
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</tr>
<tr>
<td>17</td>
<td>X12/X1</td>
<td></td>
<td>-.286</td>
<td>-.151</td>
<td>-.417</td>
<td>-.096</td>
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</tr>
<tr>
<td>18</td>
<td>Average root diam.</td>
<td>In.</td>
<td>.282</td>
<td>.208</td>
<td>.182</td>
<td>.260</td>
<td></td>
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<tr>
<td>Y</td>
<td>Average root length</td>
<td>Ft.</td>
<td>9.48</td>
<td>9.47</td>
<td>7.10</td>
<td>9.73</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Minimum root length</td>
<td>Ft.</td>
<td>3.00</td>
<td>3.00</td>
<td>3.00</td>
<td>3.00</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Maximum root length</td>
<td>Ft.</td>
<td>18.0</td>
<td>18.0</td>
<td>17.0</td>
<td>16.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Standard deviation</td>
<td>Ft.</td>
<td>3.39</td>
<td>3.51</td>
<td>2.74</td>
<td>2.67</td>
<td></td>
</tr>
</tbody>
</table>

*Not significant correlations are underlined.*

measured, vegetative indicators of site were used to find site index expressed in feet at 100 years. The butt diameter was measured outside bark (o.b.) at ground level. Longest and average branches were measured in the largest complete whorl near the base of the live crown. Short and long sides of the crown were also measured to define irregularity of crown shape within the stands. Ratios of tree height to crown width, of live crown, and of crown width to d.b.h. were approximated in variables 15, 16, and 17, respectively. Average root diameter was measured o.b. at the mid point of the average
root which was defined as the total length of main roots divided by the number of main roots.

D.b.h., butt diameter, and average length of branches in the largest complete whorl are most consistently associated with average root diameter. No variable is highly correlated, but X12, average branch length, provides a useful basis for estimation of root spread.

2. Extreme Lateral Root Spread

Graphs of root spread over crown width for each species were examined to determine the trees which had relatively wide spreading root systems. In Douglas fir these grew on poorly drained sand and sandy gravel soils which had much below average site index, about 100 feet at 100 years. On the other hand the relatively widest spreading roots of hemlock were found on well drained soils of site index 130-140. Cedar with the widest spreading roots were found on poorly drained soils with a site index of about 100. The spruce and alder studied grew only on poorly drained soil so comparisons were not possible for these species.

3. Root Characteristics of Windfalls

Measurements were made on the sketches of root systems drawn to scale in the field for each tree and the results were tabulated to determine distribution of some important characteristics of rooting. As shown in Table 6, the number of main lateral roots increased directly with d.b.h. and averaged about 8. Table 7 gives the number of trees by d.b.h. and ratio of length of longest to length of average roots for each species. For most trees this ratio averages 1.4. In Table 8 the distribution of trees by d.b.h. and depth of rooting is given. Although root depth averages about 1.5 feet for all root systems studied, the maximum depth of rooting averages about three feet. Table 9 shows average percentages of the zone marked by lateral root spread that is actually occupied by roots. These percentages decrease moderately with increased d.b.h. Percentages of the lateral root zone area actually occupied by roots average 20.9 for Douglas fir, 21.7 for hemlock, 24.2 for cedar, 19.5 for alder, and 17.5 for Sitka spruce. The variation in percentage of the root zone occupied by roots was high; for Douglas fir percentage of roots ranged from 9.8 to 42.3.

Figure 1 illustrates the increase in volume of soil in the rooting zone as average root length of Douglas fir increases. The greatest total volume occupied by roots occurs in dry moisture regimes. The greatest root spread in relation to volume of soil occupied by roots occurs in wet to very wet soil moisture regimes where rooting depth is limited. The sample numbers of the extreme trees are given in Figure 1. Volume of soil in the rooting zone varies roughly as the square of average root length.
TABLE 6

NUMBER OF PRIMARY ROOTS PER TREE BY SPECIES AND D.B.H.

<table>
<thead>
<tr>
<th>Number of Primary Roots</th>
<th>Douglas Fir — D.b.h.</th>
<th>Hemlock — D.b.h.</th>
<th>Cedar — D.b.h.</th>
<th>Alder — D.b.h.</th>
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</thead>
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<td></td>
<td>1-4  5-8  9-12  13-16  17-20  20+</td>
<td>1-4  5-8  9-12  13-16  17-20  20+</td>
<td>1-4  5-8  9-12  13-16  17-20  20+</td>
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<tr>
<td>2</td>
<td>4 1 1 1 1 1 2 2 6 2 1 1 2 4 1</td>
<td>1 5 4 3 1 8 6 1 3 1 4 2 3 10 1</td>
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<td></td>
</tr>
<tr>
<td>4</td>
<td>16 5 6 1 1 5 4 3 1 1 8 6 1 1 2 4</td>
<td></td>
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</tr>
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<td>6</td>
<td>1 3 15 8 4 1 3 1 1 6 2 3 11 4 2</td>
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<td></td>
</tr>
<tr>
<td>8</td>
<td>2 11 3 3 6 6 2</td>
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</tr>
<tr>
<td>10</td>
<td>1 4 3 3 5 2 3</td>
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</tr>
<tr>
<td>12</td>
<td>2 2 2</td>
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</tr>
<tr>
<td>14+</td>
<td>2 2 2</td>
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</tr>
</tbody>
</table>

Number of trees in class (No. of primary roots)

1 1 2 2
2 1 1
3 4 2 1
4 1
5 2
6 1
7 1
8 0
9 0
10 0
11 0
12 0
13 0
14+ 0

TABLE 7

RATIO OF THE LONGEST ROOT TO THE AVERAGE ROOT BY SPECIES AND D.B.H.

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<td>1-4  5-8  9-12  13-16  17-20  20+</td>
<td>1-4  5-8  9-12  13-16  17-20  20+</td>
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<td>1-4 5-8 9-12 13-16 17-20 20+</td>
<td>1-4 5-8 9-12 13-16 17-20 20+</td>
<td>1-4 5-8 9-12 13-16 17-20 20+</td>
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<tr>
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<td>1 2 2 2 2 1 2 10 15 6 1 3 8 5 3 2 8 6</td>
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</tr>
<tr>
<td>1 2 3 4 5</td>
<td>1 8 12 6 6 2 2 7 9 12 5 1 9 8 3 4 2 11 1</td>
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TABLE 9

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<th>Cedar</th>
<th>Alder</th>
<th>Spruce</th>
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<td>22.4</td>
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<tr>
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<td>18.3</td>
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<td>20.9</td>
<td>20.5</td>
<td>26.9</td>
<td>20.0</td>
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<tr>
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<td>21.9</td>
<td>20.9</td>
<td>23.8</td>
<td>17.3</td>
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<tr>
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<td>23.7</td>
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<tr>
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<td>18.2</td>
<td>22.5</td>
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<td>17.3</td>
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<tr>
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<td>21.5</td>
<td>18.3</td>
<td>21.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>15.8</td>
<td>26.7</td>
<td></td>
<td></td>
<td>17.6</td>
</tr>
<tr>
<td>24</td>
<td>22.0</td>
<td>25.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>20.0</td>
<td>20.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Avge.</td>
<td>20.9</td>
<td>21.7</td>
<td>24.2</td>
<td>19.5</td>
<td>17.5</td>
</tr>
</tbody>
</table>

4. Open-Grown Trees

Association between average root length and crown width of open-grown trees is shown by regression equations, coefficients of determination ($r^2$) and standard errors of estimate (SEE) in Table 10. Regression equations for open-grown ponderosa pine, lodgepole pine (dry site), Engelmann and white spruces, and Douglas fir are similar. They have an intercept of about 3, a slope of about 1, and a standard error of estimate of 3 to 7 feet. All equations using crown width to estimate root spread account for highly significant proportions of the variation in root spread. Although intercepts of about 3 feet are indicated by the data, the real origin must be zero. Therefore, ratio estimates are appropriate.

The relationship between root spread and crown width of lodgepole pine growing on wet sites was distinctly different from that on dry sites. Thirty-three of the lodgepole pines in the wet site group were measured on peat bogs on the Lower Mainland of B.C.

The ratio of root spread to crown width of 9 ponderosa pines growing on clay loam was 1.25 but not significantly different from the ratio of 1.18 for the 33 trees growing on sand that are included in Table 10.

Figure 2 shows the relationship between root spread and crown width for the 84 interior of B.C. Douglas fir of Table 10. In addition, data on 18 open-grown coastal Douglas fir from Wind River, 12 open-grown Douglas fir from Haney, and 26 seedlings from the U.B.C. Campus are plotted. The relationship between root spread and crown width is similar regardless of soil type or locality sampled. Root spread of 84 Douglas fir from Paul Lake was most strongly associated with crown width ($r=0.730$); it
also was significantly correlated with d.b.h. (r=0.680), total height (r=0.610), bark thickness at breast height (r=0.512), and age (r=0.500). Root spread was not associated with growth rate at breast height (Rg1-5, r=0.032; Rg6-10 years ago, r=0.022). Use of any of these variables with crown width made no significant improvement in ability to estimate root spread.

5. Lodgepole Pine

This species deserves special mention because of its extremely wide spread of roots on wet and peaty soils. The correlation coefficients given in Table 11 for open, forest, and combined classes of stand density illustrate the relative importance of d.b.h., crown width, total age, and tree height as indicators of root spread. The relationship between root spread and the squares of these tree variables is also shown. Crown width is the best indicator of root spread in all cases. Its utility was not improved significantly by addition of any of the other tree variables or their powers in multiple regression analyses.
The ratios of average root spread to average crown width were similar, 2.48 and 2.37, for open- and forest-grown trees, respectively. Large as these ratios are in comparison with the other species and sites studied, they must be increased by an average of 1.52 times to indicate the ratio of maximum lateral extension of roots to crown width.

6. Forest-Grown Trees

The variation in root spread of forest-grown trees usually is much greater than that of open-grown trees, which ordinarily have relatively symmetrical root systems. Open- and forest-grown lodgepole pines are compared in Table 11. Ratios of root spread to crown width of several groups of forest-grown trees are shown in Table 12. The first entry for each species is for windfalls.
TABLE 11
ROOT SPREAD OF OPEN- AND FOREST-GROWN LODGEPOLE PINE

<table>
<thead>
<tr>
<th>No.</th>
<th>Variable</th>
<th>Stand Density</th>
<th>Open</th>
<th>Forest</th>
<th>O and F</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>No. Trees</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>42</td>
<td>22</td>
<td>64</td>
</tr>
<tr>
<td>X 1</td>
<td>D.b.h. (D)</td>
<td>.694</td>
<td>.458</td>
<td>.490</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Crown width (CW)</td>
<td>.707</td>
<td>.505</td>
<td>.711</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Total Age (A)</td>
<td>.659</td>
<td>.427</td>
<td>.256</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Height (H)</td>
<td>.701</td>
<td>.450</td>
<td>.058</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>D^2</td>
<td>.633</td>
<td>.446</td>
<td>.458</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>CW^2</td>
<td>.650</td>
<td>.516</td>
<td>.664</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>A^2</td>
<td>.642</td>
<td>.390</td>
<td>.175</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>H^2</td>
<td>.651</td>
<td>.441</td>
<td>-.043</td>
<td></td>
</tr>
<tr>
<td>Y</td>
<td>Root spread Ave, in feet</td>
<td>14.2</td>
<td>9.31</td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>S.D.</td>
<td>10.4</td>
<td>4.00</td>
<td>8.98</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Min.</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Max.</td>
<td>46.0</td>
<td>19.0</td>
<td>46.0</td>
<td></td>
</tr>
</tbody>
</table>

Simple correlation coefficients for Y on X1-X8

<table>
<thead>
<tr>
<th>No.</th>
<th>Variable</th>
<th>Stand Density</th>
<th>Open</th>
<th>Forest</th>
<th>O and F</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>No. Trees</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>42</td>
<td>22</td>
<td>64</td>
</tr>
</tbody>
</table>

*Not significant values underlined.

TABLE 12
RATIOS OF ROOT SPREAD TO CROWN WIDTH OF FOREST GROWN TREES

<table>
<thead>
<tr>
<th>Species</th>
<th>No. Trees</th>
<th>RS/CW</th>
</tr>
</thead>
<tbody>
<tr>
<td>Douglas fir</td>
<td>89</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>39</td>
<td>0.74</td>
</tr>
<tr>
<td>Western hemlock</td>
<td>81</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>32</td>
<td>0.68</td>
</tr>
<tr>
<td>Western red cedar</td>
<td>61</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>0.48</td>
</tr>
<tr>
<td>Red alder</td>
<td>33</td>
<td>0.59</td>
</tr>
</tbody>
</table>

on the U.B.C. Campus Forest; the second is for windfalls on the U.B.C. Research Forest. The ratios for the U.B.C. Research Forest trees indicate the lengths of exposed roots rather than that of the total root system which was measured in the Campus Forest.

Regression equations for estimation of root spread from d.b.h., height, and crown width of forest-grown trees measured as windfalls in the U.B.C. Campus Forest are summarized in Table 13. Addition of height and crown width to d.b.h. is of little value in the estimation of root spread. In all cases the use of d.b.h. by itself to estimate root spread was only slightly better than use of crown width to estimate root spread of the Campus trees. However, crown width was significantly poorer than d.b.h. for estimation of root spread of the forest-grown U.B.C. Research Forest trees.
7. Seedling Roots and Crowns

Variation in crown and root characteristics of hemlock seedlings was illustrated photographically in Walters’ (1964) paper on juvenile growth in plantations. Data on roots and crowns of Douglas fir seedlings were reported by Smith and Allen (1962). The ratio of root spread to crown width is extremely variable for lodgepole pine, hemlock, Douglas fir, and white spruce seedlings about a general average of 1.5. In one sample the average ratio for cedar and alder was about 0.9. In another (Table 14) the ratio for cedar was 1.8. Table 14 shows that either height or crown width can be used to estimate root spread of Douglas fir, hemlock, and cedar seedlings.

### Table 13

**Regression Equations for Estimation of Root Spread from D.b.h., Height, and Crown Width of Forest-Grown Trees**

<table>
<thead>
<tr>
<th>Species</th>
<th>No. Trees</th>
<th>Intercept</th>
<th>Regression coefficient for:</th>
<th>R² or r²</th>
<th>SE ( \text{in.} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>D.b.h.</td>
<td>Ht.</td>
<td>CW</td>
</tr>
<tr>
<td>Douglas fir</td>
<td>89</td>
<td>10.24</td>
<td>1.078</td>
<td>-.120</td>
<td>.249</td>
</tr>
<tr>
<td></td>
<td>89</td>
<td>12.70</td>
<td>1.394</td>
<td>-.134</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>89</td>
<td>6.62</td>
<td>0.942</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>W. Hemlock</td>
<td>81</td>
<td>4.94</td>
<td>1.014</td>
<td>-.062</td>
<td>.297</td>
</tr>
<tr>
<td>W. Hemlock</td>
<td>81</td>
<td>1.470</td>
<td>0.816</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>W. Hemlock</td>
<td>81</td>
<td>4.38</td>
<td>1.088</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>W. R. cedar</td>
<td>61</td>
<td>7.68</td>
<td>0.700</td>
<td>-.096</td>
<td>.328</td>
</tr>
<tr>
<td>W. R. cedar</td>
<td>61</td>
<td>10.78</td>
<td>0.972</td>
<td>-.100</td>
<td>---</td>
</tr>
<tr>
<td>W. R. cedar</td>
<td>61</td>
<td>7.24</td>
<td>0.652</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

### Table 14

**Regression Equations for Estimation of Root Spread from Height and Crown Width of Young Seedlings**

<table>
<thead>
<tr>
<th>Species</th>
<th>No. Trees</th>
<th>Intercept</th>
<th>Regression coefficient for RS on:</th>
<th>R² or r²</th>
<th>SE ( \text{in.} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ht.</td>
<td>CW</td>
<td></td>
</tr>
<tr>
<td>Douglas fir</td>
<td>35</td>
<td>6.8</td>
<td>0.45</td>
<td>-.041</td>
<td>.381</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>6.1</td>
<td>0.28</td>
<td>---</td>
<td>.338</td>
</tr>
<tr>
<td>Hemlock</td>
<td>40</td>
<td>4.9</td>
<td>0.13</td>
<td>0.43</td>
<td>.289</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>5.6</td>
<td>---</td>
<td>0.68</td>
<td>.276</td>
</tr>
<tr>
<td>Cedar</td>
<td>45</td>
<td>1.8</td>
<td>0.60</td>
<td>-.24</td>
<td>.309</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>1.9</td>
<td>0.48</td>
<td>---</td>
<td>.297</td>
</tr>
</tbody>
</table>
CONCLUSIONS

Complete knowledge about the relative importance of crown and root competition will be expensive to secure and full understanding of the complex problems involved will not come for many years. In the meantime the results of studies such as this can be used in estimating space requirements of individual trees. Since only about 20 percent of the zone marked by lateral spread of roots is actually occupied by them it appears as if competition for rooting space is not likely to be as important as that for crown space for the species studied.

Although the ratio of root spread to crown width is highly variable, an average ratio of 1.0 can be used for planning. This will decrease with increasing age and stand density, and is greatest in species such as spruce and lodgepole pine which have very wide-spreading lateral roots when they are growing on poorly drained sites.

LITERATURE CITED


APPENDIX C
Draintube Specification
The product consists of two 6 oz/sy geotextile layers comprised of short synthetic fibers of 100% polypropylene which are needledpunched together. Corrugated PP pipes with two perforations per valley at 180 degree spacing and rotated 90 degrees per valley are inserted between the geotextile layers at uniform intervals.

The geotextiles are UV resistant at 70% after 500 hrs (AASHTO M288-05 requirement).

<table>
<thead>
<tr>
<th>Geotextiles properties</th>
<th>Standard</th>
<th>Reference</th>
<th>MARV</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass per unit area ASTM D5261</td>
<td></td>
<td></td>
<td>6.0</td>
<td>oz/sy (g/m²)</td>
</tr>
<tr>
<td>Thickness ASTM D5199</td>
<td>Under 2 kPa</td>
<td></td>
<td>67</td>
<td>mil (mm)</td>
</tr>
<tr>
<td>Grab Tensile ASTM D4632</td>
<td>MD &amp; XMD direction</td>
<td></td>
<td>160</td>
<td>lbs (N)</td>
</tr>
<tr>
<td>Grab Elongation</td>
<td>Min-Max value</td>
<td></td>
<td>50-105</td>
<td>%</td>
</tr>
<tr>
<td>Puncture (CBR) ASTM D6241</td>
<td></td>
<td></td>
<td>450</td>
<td>lbs (N)</td>
</tr>
<tr>
<td>UV Resistance (after 500 hrs) ASTMD4355</td>
<td></td>
<td></td>
<td>70</td>
<td>%</td>
</tr>
<tr>
<td>Apparent Opening Size (1) ASTMD4751</td>
<td>70 (212)</td>
<td>US sieve (µm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flow rate (2) ASTMD4491</td>
<td>125</td>
<td>gal/min/ft²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permittivity (2)</td>
<td>1.65</td>
<td>sec⁻¹</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Mini-pipe properties**

| Outside Diameter ASTM D2122-98              | Nominal value     | 0.97 (25) | inch (mm) |
| Pipe stiffness at 5% deflection ASTMD2412-02| 435 (3000)        | psi (kPa) |
| Spacing between pipes 10 to 80 inches (up to 4 pipes per meter of width) |

**Geocomposite properties**

<table>
<thead>
<tr>
<th>Transmissivity (3)</th>
<th>Compression load = 10,000 psf</th>
<th>Hydraulic gradient = 0.1</th>
<th>Seating Time = 100 hours</th>
<th>Product</th>
<th>m²/s</th>
<th>gpm/ft (l/min/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTMD4716</td>
<td>DRAINtube 606 ST0.5 D25</td>
<td>5.00E-04</td>
<td>2.42 (30.0)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DRAINtube 606 ST1 D25</td>
<td>1.00E-03</td>
<td>4.83 (60.0)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DRAINtube 606 ST2 D25</td>
<td>2.00E-03</td>
<td>9.66 (120.0)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DRAINtube 606 ST4 D25</td>
<td>4.00E-03</td>
<td>19.32 (240.0)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Puncture (CBR) ASTMD6241</td>
<td>675 (3000)</td>
<td>lbs (N)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Packaging**

<table>
<thead>
<tr>
<th>Standard roll dimensions</th>
<th>Length</th>
<th>Width</th>
<th>Roll weight</th>
<th>Inside diameter of roll</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>246.06 (75)</td>
<td>13.06 (3.98)</td>
<td>299 - 497</td>
<td>95</td>
</tr>
</tbody>
</table>

Unless noted, the printed values are “Minimum Average Roll Values” (MARV) in the weakest direction as the typical value minus two standard deviations according to a normal distribution and a confidence interval of 95 %.

(1) Maximum Average Roll Values (MaxARV) | (2) Tested on the filter in the area of the pipe after assembly | (3) Typical Value - Transmissivity measured on a 250 mm wide specimen installed as follows: sealed sand / geocomposite / geomembrane / sealed sand, with one pipe (confined in sand) installed in the middle of the test chamber in the longitudinal direction. The given transmissivity is obtained from a linear relationship between the number of pipes and the measured transmissivity.

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Rev. 04-2015
APPENDIX D
Conceptual Hydrogeologic Model Memorandum
January 31, 2019 PRJ18-074

Attention: Cobble Hill Holdings Ltd.

RE: Cobble Hill Landfill – Hydrogeological Conceptual Model

1. Introduction

Sperling Hansen Associates (SHA) was retained by Cobble Hill Holdings Ltd. (CHH) to prepare a hydrogeological conceptual model of the Cobble Hill Holdings Landfill (Landfill) at the request of the Ministry of the Environment (MoE). This report details a review of the surficial and bedrock geology, the groundwater flow regime and a summary of the available hydrogeological data for the Landfill.

The Landfill is located at 460 Stebbings Road, in the South Shawnigan Lake Area within the Cowichan Valley Regional District as outlined in Figure 1. The Landfill is approximately 8.92 Hectares and historically was used as a rock quarry under the jurisdiction of the BC Ministry of Mines.

2. Site Description

2.1 Physiography and Topography

The Landfill is located within the Vancouver Island Ranges physiographic region on Victoria Island located in the upper reach of the Shawnigan Lake Catchment. Surface water bodies residing within the catchment include Shawnigan Creek and its tributaries. Shawnigan Creek is located just west of Stebbings Road and runs north south at an approximate elevation of 331 metres above sea level (masl).

The surrounding Landfill area is moderate to dense forest with numerous logging cuts in various stages of regrowth. The topography is rugged and steep in areas. The average annual precipitation is approximately 2,047.5 mm with approximately 1,975.6 mm falling as rain and 72.0 cm falling as snow (Gov. Environment Canada 2019). The average annual temperature is approximately 9.8 °C with an average peak of 18.1 °C occurring in August and the minimum average temperature of 2.5 °C occurring in December. The maximum average snowfall of 19.8 cm occurs in January.
### 2.2 Regional and Local Surficial and Bedrock Geology

The Landfill is underlain by Wark Colquitz gneiss bedrock from the lower Paleozoic Period (295-384 million years) and comprises of deep crustal sequence of interfingering dioritic intrusions that formed at the root of the Bonaza arc, collectively comprising the West Coast Crystalline complex (Canil et al., 2010).

As part of the environmental monitoring program requirements outlined by the MOE several monitoring wells and seepage blanket wells were drilled and completed at the Landfill, as shown in Figure 6-1. A list of the monitoring wells and seepage wells are provided below along with a geologic description. Three rock types have been identified at the Landfill and include a medium to coarse grained dark green gabbro, a medium to fine grained/medium to dark green diorite and a pale green, fine grained diorite.

- **MW-01 [S 51 mbg/D 84 mbg]:** Interbedded green gabbro and dark green diorite approximately 25 to 30 metres thick. Fracture was observed at 81 mbg, and had a 3 gallon per minute yield.
- **MW-02 [S 43 mbg]:** Interbedded green gabbro and dark green diorite approximately 15 to 30 metres thick. Fracture was observed from 36 to 42 mbg, but generated little water.
- **MW-03 [S 22 mbg/46 mbg]:** Interbedded green gabbro and dark green diorite approximately 5 to 20 metres thick. Fracture was observed from 17 to 18 mbg, but generated little water.
- **MW-13-04 [41 mbg]:** Interbedded green gabbro and dark green diorite approximately 5 to 30 metres thick. Fracture was observed from 7 to 7.3 mbg, but generated little water.
- **MW-13-05 [41 mbg]:** Interbedded green gabbro and dark green diorite approximately 3 to 25 metres thick. No fractures observed.
- **SB-01 [4.0 mbg]:** Green gabbro.
- **SB-02 [3.3 mbg]:** Green gabbro.
- **SB-03 [3.5 mbg]:** Green gabbro.
- **MW-04: Wark, gneiss-granodiorite.**
- **MW-05: Wark, gneiss-granodiorite.**
- **MW-06: Green diorite, fracture at 37.8 mbg.**

The Wark Colquitz gneiss bedrock was studied by Hancock (2012) where Hancock examined the fractures in the exposed rock at the quarry site noted that there were three types, tight, filled, and veins.

Hancock completed a geologic study of the South Island Aggregates Quarry in 2012 and Hancock did not observe any geologic faults underlying the Landfill. Two regional faults exist and are located 3 km southwest and 6 km northwest of Landfill (Hancock 2012, and Pye and Kneale 2012).

2.3 Hydrogeological Conceptual Model

As discussed in the previous section the Landfill is underlain by a thin stratigraphic unit of quaternary cover and is less than 2 metres thick which underlain by thick bedrock comprised of a gabbro/dioritic gneiss. The bedrock is very competent and hard from surface to approximately 75 mbg. As such only a few fractures have been observed during drilling, rendering the deep rockmass nearly impervious. The fractures within the bedrock do not have the frequency of open interconnected fractures needed to convey significant quantities of groundwater flow. As such, the intact bedrock down to a depth of 75 m is extremely unlikely to provide sufficient groundwater yield for drinking water supply wells because of the limited ability to transmit groundwater flow. The upper bedrock at the Landfill provides a 65 to 75 m confining layer of lower permeability rock that will act to help protect the deep bedrock aquifer. The bedrock aquitard is bound in the east by faults and the Finlayson Arm of the Saanich Inlet.

A packer test was performed on two monitoring wells drilled and completed within the rock quarry in proximity to the Landfill. A fracture was observed at 19 mbg in MW13-05 and 34 mbg in MW13-04, and had a measured hydraulic conductivity of 1.1 x 10^-7 m/s, and 8.1 x 10^-8 m/s, respectively. The relative scale of hydraulic conductivities and permeabilities ranges from 10^5 m/s for gravels to 10^-8 m/s for unfractured metamorphic and igneous rocks respectively. The hydraulic conductivity of the intact bedrock underlying the Landfill when compared to the relative scale is extremely low. Groundwater flow in the bedrock is dependent exclusively upon the availability, connectivity, and size of the aperture of the fractures in the rock mass. It has been concluded by Active Earth that the geology and hydrogeology of the Landfill indicate that the shallow bedrock does not readily transmit the flow of water. Recharge from precipitation is predominantly conveyed downslope as surface water runoff and as shallow groundwater flow in the overburden horizon, where present, and in the fractured rockmass within the blasted Seepage Layer.

Below 75 m depth, the bedrock is more fractured and provides higher groundwater yields. The onsite well which is developed to a depth of 94 m produces up to 20 GPM, with the first water bearing zone encountered at 78.6 m. Given there are very few fractures noted in the 75 m of competent bedrock between the water bearing formation at depth, and that an upward gradient is present, SHA is of the opinion that there is very little risk of contamination migrating to the deep water bearing zone from which drinking water is sourced even in the event of a liner failure.

An average Darcy linear velocity for the deep bedrock formation at the Landfill was reported by Active Earth at a rate of 0.001 m/year. This rate would suggest an effective travel time of 3 million years for recharge water from the Landfill to reach Shawnigan Lake; and the vertical travel time

was reported to travel from the bedrock to the deeper bedrock is greater than 100,000 years. However, the actual advective velocity of water flowing within the few fractures that are present is likely to be much faster.

The vertical hydraulic gradient was calculated for the Landfill at nested monitoring wells MW-01 shallow and deep. The vertical hydraulic gradient at MW-01S/D is -0.02 suggesting the deep aquifer has a strong upward vertical gradient. This condition is very protective of the deep water bearing formation provided the recharge zone further upgradient is free industrial activity and contamination.

Groundwater monitoring wells were drilled and completed within the competent bedrock, a summary of the completion details including the most recent groundwater elevations is provided in Table 6-1. The groundwater elevations range from 319.1 masl to 342.5 masl in the shallow bedrock and range from 319.1 masl to 331.2 masl in the deep bedrock. Figure 6-3 illustrates a time series plot of the groundwater elevations from November 2010 to January 2019. The potentiometric elevations of the monitoring wells follow a similar trend suggesting that the network of fractures of both the shallow and the deep system responds to seasonal recharge fluctuations, and that some interconnectivity of the fracture networks may exist. The potentiometric elevations within the bedrock wells are at their highest in winter months and lowest in the summer months suggesting recharge of the aquitard occurs in the wetter winter months.

Figure 6-4 illustrates the groundwater flow conceptual flow map for the Landfill and Figure 6-2 illustrates a hydrogeologic cross section showing the groundwater elevations and predominant flow directions. There is a groundwater flow divide located east of the Landfill and separates groundwater flowing into Shawnigan Creek and its tributaries and groundwater flowing to the Landfill.

Groundwater within the region of the encapsulation area will travel in a north easterly direction, in addition to groundwater north of the encapsulation area. Groundwater east of the encapsulation area will travel east to Shawnigan Creek and its tributaries. Groundwater hydraulic gradients were calculated at the Landfill: for a change in distance over a metre approximately a 0.04 m change in hydraulic head was observed, yielding a gradient of 4%.

There is limited hydrogeologic data available regarding the deep regional bedrock water bearing formation which is believed to be present from greater than 300 mbg in proximity to the Landfill. There are several groundwater wells drilled and completed in proximity to the Landfill and have yields of approximately 2 to 20 US GPM. The highest water well yields were observed in wells that were screened within the limestone bedrock at depth.

GROUNDWATER ELEVATIONS OVER TIME

COBBLE HILL LANDFILL
UPDATED FINAL CLOSURE PLAN 2019

FIGURE 4
APPENDIX B
Tables
### Table 1: Monitoring Well Completion Details

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<th>Ground Elevation (Est)</th>
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APPENDIX E
Waste Approval Application Form
**Waste Approval Application**

**CHH 460 Stebbings Road**

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