Proposed Conceptual Designs to Improve Slope Stability for Old Vienna Road, Tillsonburg, ON

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1. INTRODUCTION

DST Consulting Engineers Inc., a Division of Englobe (“DST”) was retained by the Town of Tillsonburg (“the Town”) to conceptualize three design solutions and one “Do Nothing” option for the existing slope between Big Otter Creek and Old Vienna Road as per RFP 2018-004 near 27 Old Vienna Road, Tillsonburg, ON (“the Site”). Authorization to proceed with this work was received from the Town of Tillsonburg through Mr. Shayne Reitsma, Manager of Engineering, following execution of the Consultant Agreement on January 9, 2019.

DST recently completed a geotechnical borehole investigation and evaluation of the stability of the existing slope at the Site under the same RFP. The findings and conclusions of that work were presented in our draft geotechnical report entitled, “Geotechnical Investigation and Evaluation of Slope Stability for Old Vienna Road, Tillsonburg, ON”, submitted to the Town on May 10, 2019.

This conceptual design report aims to present three design solutions and one “Do Nothing” option in the context of the findings of our recent geotechnical investigation and slope stability evaluation, taken to represent the Site in its present conditions. The purpose of this report is to provide an adequate amount of information to permit the informed selection of a single option to address the observed instabilities at the Site. Each of the conceptual design solutions presented herein have been suggested as viable options to achieve the Town’s specified desired minimum Factor of Safety of 1.3.

This conceptual design report has been prepared for the sole use of the Town of Tillsonburg, and any use or reliance on this report by another other party is the responsibility of such party. This report is also subject to the limitations described in Appendix A.

2. SUMMARY OF GEOTECHNICAL INVESTIGATION FINDINGS

DST recently completed a geotechnical investigation program at the Site which comprised a preliminary slope evaluation, desktop study, a subsurface field investigation, a geotechnical laboratory testing program, and analytical modelling of the three critical cross-sections along the existing slope. The above-noted field activities were carried out between March 13, 2019 and March 27, 2019, followed by the completion of our laboratory testing and analytical modelling, and the release of our draft geotechnical investigation report on May 10, 2019.

2.1 Stratigraphy

Based on the results of our geotechnical investigation, the Site stratigraphy can generally be summarized according to location, as follows:

- Along the paved Old Vienna Road alignment (BH19-03 through BH19-05), the subsurface stratigraphy comprised a thick (150 to 165 mm) layer of asphalt underlain by a compact to dense sand and gravel granular FILL. The granular FILL layer extended to 0.3 m depth in BH19-04 and BH19-05, and to 1.5 m depth in BH19-03. A compact sand to sandy silt FILL was identified below the granular FILL layer, generally extending to 0.8 m depth except in BH19-03, where the sandy FILL was present to 3.4 m depth before transitioning into a possible native silty sand unit that extended to 4.6 m depth. The substantial increase in FILL thickness observed in BH19-03 may have been necessitated by the construction of...
Old Vienna Road through wetland deposits at this borehole location, as evidenced by the existing wetland adjacent to the road. These FILL layers were all underlain by a stiff to very stiff native deposit of silty clay (CL) TILL which extended to the termination depth in each borehole. The TILL generally possessed silt-filled fissures in the upper few metres, and brittle inclined silt seams at intermediate depths. Also, of note, a 1.5 m thick layer of sandy silt to silty clay was identified at 11.0 m depth in BH19-05, sandwiched within the TILL. This interbedded layer may have been derived from Big Otter Creek at an earlier point in (geologic) time.

- Within the grass-covered backyard behind 29 Old Vienna Road and along the slope faces (BH19-01, BH19-02, BH19-06, and AH19-07 through AH19-11), a 100- to 200-mm thick layer of topsoil and sandy loam was encountered at surface. In BH19-01, BH19-02, BH19-06 and AH19-11, this layer was underlain by a loose sand to sandy silt FILL which extended to 1.1 to 1.6 m depth. In each borehole, the above-noted native deposit of silty clay (CL) TILL was encountered below the topsoil and sandy loam or FILL. All boreholes were terminated in this unit.

Groundwater levels were measured in each borehole upon the completion of drilling activities and in three standpipe piezometer monitoring wells installed in BH19-01, BH19-02 and BH19-05 over two follow-up site visits. The depth of groundwater varied between 2.1 and 3.9 mbgs depending on location but was generally found to be consistent over the one-month period during which the follow-up site visits were carried out.

2.2 Analytical Modelling of Existing Slope

The existing slope was modelled using three selected critical cross-sections. The topographic profiles for these three selected cross-sections were constructed using topographic elevation data provided by Kim Husted Surveying Ltd., a local OLS-certified surveying company that was commissioned to survey the Site. An interpretation of the subsurface stratigraphy was constructed using the recent borehole data and reasonable geological interpretation. Base cases were modelled using reasonable soil strength parameters based on our experience and understanding of the materials.

The soil strength parameters summarized in Table 2.1 were selected and used duringmodelling to represent the materials encountered at the Site. Details regarding the methodology that was adopted to determine these parameters are included in our geotechnical investigation report referenced in Section 1. The Silty Clay TILL was considered the controlling material governing stability of the overall slope based on its prevalence and the brittle silt seams that were identified within this unit. As such, the strength parameters for this unit were calibrated such that a calculated factor of safety equal to unity was obtained, to represent the existing slope in a pseudo-stable condition reflecting the observed instabilities in its current condition.
Table 2.1  Summary of the soil strength parameters adopted for modelling

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Bulk Unit Weight ($\gamma$) (kN/m$^3$)</th>
<th>Effective Cohesion ($c'$) (kPa)</th>
<th>Effective Friction Angle ($\phi'$) (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FILL (mostly Loose Sandy Silt)</td>
<td>19.0</td>
<td>0</td>
<td>22</td>
</tr>
<tr>
<td>Compacted Granular FILL (Sand)</td>
<td>21.0</td>
<td>0</td>
<td>32</td>
</tr>
<tr>
<td>Granular FILL (Dense Sand &amp; Gravel)</td>
<td>21.0</td>
<td>0</td>
<td>35</td>
</tr>
<tr>
<td>Stiff to Very Stiff Silty Clay TILL</td>
<td>20.8</td>
<td>8</td>
<td>24</td>
</tr>
</tbody>
</table>

2.3  Summary of Likely Mechanisms of Instability and Estimated Historic Erosion Rates

Based on the results of our analytical modelling of the stability of the three critical cross-sections selected to represent the existing slope, two primary mechanisms were identified as the most likely sources of the observed instabilities. These two mechanisms are active toe erosion and long-term material softening from the infiltration of groundwater.

Toe Erosion: According to a Long Point Region Watershed Characterization Report produced in 2008 by the Lake Erie Source Protection Region Technical Team in association with the Long Point Region Conservation Authority, Big Otter Creek has been identified as Canada’s largest source of sediment contamination to Lake Erie. The presence of multiple scour features, the high levels of the river water (relative to the river banks), and the relatively high peak discharge rates (up to 120 m$^3$/s recorded in 2018) suggests toe erosion is extremely likely to continue at the Site. The continual removal of this toe material has the effect of reducing the resisting mass at the toe of the slope, resulting in a reduction to stability. The slope must therefore continually deform (i.e. landslide) to restore a stable geometry. If toe erosion is permitted to continue unabated, the landslide is expected to continue to impact Old Vienna Road and the properties along Van Street.

At the request of the Town, DST estimated the apparent rates of erosion at various locations across the Site using the available historical maps obtained during DST’s desktop study and the recent topographic survey completed as part of DST’s investigation at the Site. For this work, a historical plan map of the Site obtained from a Cyril J. Demeyere Ltd. (CJDL) report (entitled Report on Soil Investigation for Proposed Otter Creek North Trunk Sanitary Sewer, Simcoe Street Crossing, Tillsonburg, Ontario, dated May 19, 1992) was selected to serve as a baseline reference. The historic plan map and DST’s recent survey data were geo-referenced to each other approximately using catch basins and utility poles, which were assumed to be static benchmarks.

The estimated historic rates of erosion for the Site are summarized in Appendix C. The rate of erosion at the Site was estimated to be greatest in the area of the former house located at 27 Old Vienna Road (near Section C), with apparent soil loss estimated at upwards of 360 mm/year for the past 27-year period. Minimal vegetation in a historical satellite image from 2006, accessed on Google Earth, yielded a lower estimated erosion rate of 176 mm/year for the earlier 14-year period between 1992 and 2006. Special attention should be paid to the higher estimated erosion rate over the past 27 years (more than double the estimated erosion rate for the earlier 14 years), suggesting an apparent acceleration of the ground loss experienced at this part
of the Site. The most important cause/causes of this apparent increase in the rate of erosion cannot be identified with certainty using the limited information available at this stage.

Interestingly, the lowest estimated rate of erosion at the Site, 100 mm/year corresponded to Section A, where the gabion wall was observed to be present at the toe of the slope. In contrast, however, the location of the south end of the gabion wall was compared and based on the available survey data, it appears that more than 24 m of the original gabion wall may have potentially been washed out. This represents roughly a loss of more than 900 mm of wall per year over the 27-year period between the baseline CJDL map and our recent topographic survey at the Site. However, with no as-built records available for the gabion wall structure, this possible loss of wall cannot be confirmed.

It should be noted that these rough estimates are highly susceptible to error aggregated from items which could include inaccurate surveying and geo-referencing during the timeframe considered. Furthermore, these estimates are based on past observations and site conditions which may or may not be representative of future rates of erosion that could be experienced at the Site under different physical and environmental conditions.

Material Softening: The presence of fissures near the top of the Silty Clay Till and silt seams throughout the lower regions of this deposit are expected to provide a relatively permeable conduit for the infiltration of groundwater into the slope. Over time, the water would be expected to have a softening effect on the surrounding material (i.e. the clay), resulting in a progressive reduction to its internal strength. Once the internal strength of the material has reduced such that it is no longer capable of supporting itself, the material will yield. The progressive failure of the Silty Clay Till is likely to blame for the multiple observed slide blocks along the slope, taken as evidence of an on-going retrogressive failure. If not for this softening, the Silty Clay Till would be otherwise expected to be a relatively strong and stable material in its undisturbed condition.

A retrogressive failure is characterized by the successive slippage or movement of slices of a clay-dominated slope, the presence of multiple back-scars and an apron of failure debris at the slope toe. This type of failure can be expected to continue for great distances, left unmitigated.

3. APPLICATION OF GEOTECHNICAL PRINCIPLES TO IMPROVE STABILITY

The objective of this report is to provide three recommended remedial options and one “Do Nothing” option for the existing slopes near 27 Old Vienna Road. The above-noted Base Cases were used to evaluate the possible benefits of the recommended remedial design options and to inform the design such that the minimum required FOS of 1.3 is satisfied by those measures.

At this stage, the recommended remedial options are limited to a conceptual design level of detail. The primary intention of this report is to provide enough information for the Town to make an informed selection of the most practical and effective solution that meets their current requirements. The recommendations contained herein are not to be released for construction or tender and are in no way intended as instructions for a construction contractor.
3.1 **MNR Minimum Slope Geometry Requirements**

The Ministry of Natural Resources (MNR) has produced technical guides applying to the management of natural hazards in Ontario. This study was prepared and implemented in accordance with the latest revision of the MNR’s publication, *Understanding Natural Hazards*. Slopes and landslides are considered an erosion hazard, wherein river and stream systems are classified as either confined or unconfined systems. The slope along Old Vienna Road is considered a **confined system**, requiring that the following parameters be defined:

- **Toe Erosion Allowance**: an allowance for the recession of the toe of a slope in a confined river system (i.e. a river valley).

- **Stable Slope Allowance**: a straight line commencing at the toe of a slope and projecting upslope at an angle of 3 horizontal to 1 vertical or as determined by a Geotechnical Engineer using accepted geotechnical principles.

- **Top of Stable Slope**: the point at the crest of a slope defined by the stable slope allowance line.

- **Erosion Access Allowance**: a horizontal setback distance beginning at the top of stable slope to provide for: emergency access to erosion prone areas; construction access for regular maintenance and access to the site in the event of an erosion event or failure of a structure, and; protection against unforeseen or predicted external conditions which could have an adverse effect on the natural conditions or processes acting on or within an erosion prone area of provincial interest.

- **Erosion Hazard Limit**: the combined horizontal distance defined as the sum of the toe erosion allowance, the stable slope allowance, and the erosion access allowance (for a confined river / stream system).

The values summarized in Table 3.1 below have been calculated using the minimum values suggested in the MNR Technical Guide for Erosion Hazards for the parameters defined above.

**Table 3.1** Summary of the MNR-suggested hazard allowances for the slopes along Old Vienna Road

<table>
<thead>
<tr>
<th>Cross Section ID</th>
<th>Toe Erosion Allowance (m)</th>
<th>Stable Slope Allowance (H : V)</th>
<th>Approximate Height of Slope (m)</th>
<th>Erosion Access Allowance (m)</th>
<th>Erosion Hazard Allowance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS-1</td>
<td>8.0</td>
<td>3 : 1</td>
<td>10.7</td>
<td>6.0</td>
<td>46.1</td>
</tr>
<tr>
<td>CS-2</td>
<td>8.0</td>
<td>3 : 1</td>
<td>11.5</td>
<td>6.0</td>
<td>48.5</td>
</tr>
<tr>
<td>CS-3</td>
<td>8.0</td>
<td>3 : 1</td>
<td>9.4</td>
<td>6.0</td>
<td>42.2</td>
</tr>
</tbody>
</table>

The MNR Technical Guide states that the suggested minimum hazard allowances may be substituted with allowances determined using accepted geotechnical principles applied by a Professional Geotechnical Engineer. Based on our own evaluation of the existing slope and the results from our modelled cross-sections, the hazard allowances summarized in Table 3.2 below are deemed to be reasonable for the slopes at the Site in their existing observed state.
### Table 3.2 Summary of the estimated hazard allowances using accepted geotechnical principles

<table>
<thead>
<tr>
<th>Cross Section ID</th>
<th>Toe Erosion Allowance (m)</th>
<th>Stable Slope Allowance (H : V)</th>
<th>Approximate Height of Slope (m)</th>
<th>Erosion Access Allowance (m)</th>
<th>Erosion Hazard Allowance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS-1</td>
<td>8.0</td>
<td>2.3 : 1</td>
<td>10.7</td>
<td>6.0</td>
<td>38.6</td>
</tr>
<tr>
<td>CS-2</td>
<td>8.0</td>
<td>2.3 : 1</td>
<td>11.5</td>
<td>6.0</td>
<td>40.5</td>
</tr>
<tr>
<td>CS-3</td>
<td>8.0</td>
<td>2.3 : 1</td>
<td>9.4</td>
<td>6.0</td>
<td>35.6</td>
</tr>
</tbody>
</table>

As can be seen in Table 3.1 above, the erosion hazard allowances determined using the MNR-suggested parameters would require that Old Vienna Road be set back no less than 42.2 m from Big Otter Creek. This compares to a minimum erosion hazard allowance of 35.6 m for CS-3, based on our geotechnical analysis for the slopes at the Site in their existing observed state. Adherence to these MNR-suggested allowances is neither practical nor possible within the spatial constraints of the Site. As such, a remedial solution would have to be adopted at the Site to reinforce the slope and to permit a highly reduced, but safe, erosion hazard allowance for the long-term stability of the subject portion of Old Vienna Road and the safety of the public.

Considering the prevailing physical and current operational constraints of Old Vienna Road, the above-noted hazard allowances can be reduced depending on the solution that is adopted. The updated hazard allowances will be depicted on the plan figures of the Site as part of the detailed design phase for the selected remedial option.

#### 3.2 Other Considerations for Long-Term Stability

Some additional factors were also considered during the slope stability analyses: the seismic hazard, and the effect of rapid drawdown.

**Seismic Hazard:** The possibility of a seismic event at the Site was considered, as it would be anticipated to negatively affect the stability of the slopes. Based on the observed processes and our characterization of the existing slopes, seismicity was not deemed to be a likely trigger for the current instability at the Site. As such, seismicity was not incorporated into the Base Case analyses. However, seismicity is planned to be incorporated into the analyses performed for the detailed design phase for the selected remedial option.

**Rapid Drawdown:** A rapid drawdown event constitutes a relatively large drop in surface water levels over a relatively short period of time. When surface water levels rise, the soil comprising an adjacent slope will become saturated. The water that infiltrates the soil pores takes on some of the stresses from the surrounding materials, resulting in porewater pressure. This porewater pressure reduces the effective stress acting on the soil particles, thereby reducing friction and reducing stability. While the water level is high, the surface water exerts a force normal to the slope face, resulting in a buttressing effect. The buttressing effect typically counteracts the effect of the high porewater pressure, for the most part. If water levels recede, the pore space can remain saturated for some time afterward, particularly if the soil is relatively impermeable. Without the buttressing effect of the surface water, these events are expected to negatively affect the stability of slopes. For the slopes along Old Vienna Road, the extreme maximum water level in Big Otter Creek has been recorded at approximately 6 m based on the water level data obtained.
from Station 02GC010. The presence of sloughing along the toe of the slope suggests the effects of rapid drawdown should be considered at the Site. However, since the slope is already failing, these effects will only be incorporated in the analyses during the detailed design of the selected remedial option.

4. PROPOSED CONCEPTUAL SOLUTIONS

Three conceptual design solutions and one “Do Nothing” option are described below. The conceptual design solutions have been prepared by selecting combinations of stabilization techniques that are typically effective for addressing the likely mechanisms of instability identified during our geotechnical investigation and slope stability evaluation. The specific stabilization techniques recommended below are each sufficiently unique to provide the Town with a variety of options to choose from depending on possible constraints such as right-of-way limitations, the presence of underground utilities, environmental constraints, cost and ease of implementation. The general strengths and concessions associated with each option are summarized in Table 4.4 at the end of this section.

It should be noted that analytical modelling of Cross Section 2 suggests that portion of the slope is adequately stable in its existing state. As such, the remedial solutions described below were not modelled for Cross Section 2. However, it is anticipated that the continued erosion of the toe area due to surface water runoff and the flow of Big Otter Creek will result in the modification of the observed geometry. The presence of the observed failed material at the time of the preliminary slope assessment is also considered unacceptable as it poses a potential risk of localized failure. Therefore, general regrading, revegetation, and toe armouring is still recommended across this part of the slope.

4.1 Contemplated Slope Remediation Techniques

4.1.1 Toe Armouring

In general, one of the two primary factors contributing to the observed instability of the existing slope is toe erosion / undercutting along the outside meander bend of Big Otter Creek within the Site limits. Until this mechanism is adequately addressed, it is anticipated that the loss of material and movement of the slope will continue. As such, each of the below conceptual design options include the provision of toe armouring in the form of heavy-duty, large-diameter riprap (i.e., rockfill) material along the affected portions of the slope toe area. This shall include the additional placement of riprap along the discharge zone below the existing stormwater outflow drain at the north end of the Site. All riprap armouring should be designed and sourced such that it is able to adequately withstand a practical range of river flow scenarios based on historical flow data for Big Otter Creek, as well as overtopping scenarios constructed using the available historical river level data.

A suitable alternative to riprap toe armouring might include bio-armouring. This alternative typically involves the use of natural features such as the placement of logs or planting of riparian vegetation. The objective of this approach is to sufficiently slow the flow of water to mitigate or eliminate the energy conditions required for erosion to take place. This technique also has the
added benefit of possibly enhancing the natural ecosystem by expanding the habitat for the existing flora and fauna.

4.1.2 Drainage Improvements

Enhancing drainage is typically a cost-effective solution for improving the stability of slopes affected by water conditions. However, this approach is not expected to be effective for the slopes along Old Vienna Road. The silt seams identified within the Silty Clay TILL unit are believed to be acting as a conduit for groundwater infiltration into the slope, facilitating material softening and reducing the strength of the material. Attempting to enhance drainage in this clay-dominated material would require the targeting of these silt seams with drains; this is not practical and the risk of the drains being ineffective is considered high. The reliance on drainage improvements is therefore not advised, and any possible enhancement to the existing drainage conditions resulting from the implementation of the selected remedial measures should be strictly considered as a “bonus”.

4.1.3 Re-alignment of Old Vienna Road or Big Otter Creek

Another approach to potentially eliminate the interaction between Big Otter Creek and Old Vienna Road would be to realign the road or river such that the two are separated. The available alignment for Old Vienna Road is understood to be highly constrained, but the existence of historic dam raceway channels could potentially be exploited for the rerouting of Big Otter Creek. However, such a drastic measure would be anticipated to require extensive consultation with the applicable regulatory bodies and the likely exacerbation or creation of unstable slopes downstream. The possible impacts of this approach are therefore considered to be prohibitive and this approach is not advised.

4.1.4 Slope Reinforcement and/or Soil Improvement

One of the mechanisms believed to be behind the observed instability of the existing slope is material softening resulting from the infiltration of groundwater via the silt seams identified in the Silty Clay TILL. While the native material has been sufficiently softened to result in failure, the introduction of appropriately designed reinforcing members (such as soil nails or shear piles) or the replacement of the softened native material with high-strength backfill materials (such as through the construction of a granular shear key) could adequately increase the composite soil strength such that stable conditions are restored.

4.1.5 Slope Reconstruction, Replacement, and/or Regrading

The last approaches considered for this Site are partial reconstruction, full replacement, and/or regrading of the existing slope. For partial reconstruction, we considered sub-excavation of the failed materials and the introduction of a benched geometry utilizing retaining walls. Full replacement of the slope would involve sub-excavation of the existing slope materials to a minimum depth determined based on analytical modelling and reconstructing it with a suitable, stronger material than the in-situ native material. Finally, fully regrading would involve cutting the slope back at an angle determined to be adequately stable. Each of these approaches would
require the removal of all existing vegetation on the slope. Considering the spatial constraints associated with the Site, only benched geometries were considered.

4.2 “Do Nothing” Option

The viability of a “Do Nothing” option was evaluated based on the observed conditions at the existing slope in its current conditions. Left unchanged, the slope would be expected to continue deforming and sloughing such that a stable geometry is established naturally. However, the possibility of a drastic slope failure under adverse conditions could not be ruled out and continued removal of resisting material at the toe of the slope due to the erosive undercutting action of Big Otter Creek will not permit the slope to establish a permanent (long-term) stable geometry unless the river changes course. As such, we must conclude that a “Do Nothing” approach is not a viable long-term option for the slope in question in consideration of the identified mechanisms of instability.

In the short- to medium-term, a “Do Nothing” approach could be considered if a suitable monitoring program and warning system is implemented for the most critical sections of the slope to ensure anticipated future ground movements are documented, assessed, and compared to allowable thresholds determined for the on-site infrastructure. The monitoring program would also serve as an early-warning system to inform the Town as to when remedial action must be urgently taken.

4.3 Conceptual Design Solution 1: Partial Slope Reconstruction with Benched Retaining Walls and Toe Armouring

The Town may achieve a slope with the required minimum slope stability improvement of 30% (i.e., a factor of safety of 1.3 assuming current stability is equal to unity) by partially reconstructing the slopes behind 27 Old Vienna Road and below Old Vienna Road in a benched geometry utilizing retaining walls and toe-armouring.

For the slope behind 27 Old Vienna Road, a conceptual design featuring one mid-level bench (i.e., three-tiered slope: upper, middle, and lower levels) was found to be viable. The middle and lower levels would each be supported locally by a gravity retaining wall with free-draining granular backfill and subdrains to prevent the build-up of water behind the walls. Based on the conceptual modelling, the upper retaining wall would need to be in the order of 5.1 m high, while the lower retaining wall would need to be roughly 3.2 m high. The upper wall would be located approximately 6.2 m back from the crest of the existing slope, with the wall and backfill extending another 3.7 m back for a total setback distance of 9.9 m. The middle terrace would be 12.6 m long including the top of the lower wall, graded at 2% toward the river to prevent ponding. The lower level of the slope would be 9.6 m long, also graded at 2% toward the river. A sloped upper bank would be established next to the river, supported by riprap toe armouring. Each terrace would then be revegetated to mitigate surface erosion from flood and rain events. The described conceptual solution is depicted in the attached Figure 1, included in Appendix B.

For the slope below Old Vienna Road, a similar design involving one mid-level bench was contemplated. However, conceptual modelling indicates retaining wall tie-back anchors would be necessary to ensure an adequate degree of stability. The tie-back anchors for the middle and
lower level retaining walls would need to be 13 m and 15 m in length, respectively, installed at a 15° inclination and with a 6-m long grouted section and a 1-m lateral spacing. The anchors were modelled with a 1-ft bond diameter, pull-out resistance of 60 kPa, and tensile capacity of 400 kN. The upper and lower walls for this section would need to be 5.3 m and 3.0 m high, inclined at 30° from vertical, with a 3.0-m setback from the existing sidewalk at the top of the slope. The middle terrace would be 4.0 m long, and the lower terrace would be 4.6 m long, extending to the riprap proposed along the edge of Big Otter Creek. A 2% grade is recommended at the bench level to ensure adequate surface drainage. This conceptual solution is depicted in the attached Figure 5, included in Appendix B.

The toe of the slope for each of the above-described cases would need to be armoured using heavy-duty riprap designed by a qualified person (such as a hydraulic engineer) to withstand the flow and water levels of Big Otter Creek. The minimum erosion hazard allowances suggested for the slopes at the Site if they were to be remediated using this approach are summarized below in Table 4.1.

<table>
<thead>
<tr>
<th>Cross Section ID</th>
<th>Toe Erosion Allowance (m)</th>
<th>Stable Slope Allowance (H : V)</th>
<th>Approximate Height of Slope (m)</th>
<th>Erosion Access Allowance (m)</th>
<th>Erosion Hazard Allowance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS-1</td>
<td>2.0</td>
<td>2.6 : 1</td>
<td>10.7</td>
<td>6.0</td>
<td>35.8</td>
</tr>
<tr>
<td>CS-3</td>
<td>2.0</td>
<td>1.5 : 1</td>
<td>9.4</td>
<td>6.0</td>
<td>29.6</td>
</tr>
</tbody>
</table>

### 4.4 Conceptual Design Solution 2: Soil Nails with Toe Armouring

Preliminary modelling suggests soil nails are a viable and practical soil reinforcement technique that may be used to achieve the minimum prescribed stability increase of 30% for the existing slopes at the Site. The soil nails would be required to be installed in at least three rows with a 2-m lateral spacing.

For the slope behind 27 Old Vienna Road, conceptual modelling suggests the upper two rows of soil nails would need to be 17.0 m in length, installed at a 35° inclination from the horizontal. The third, lower row could be reduced to 12.0 m in length. The conceptual model featured the bottom row at an offset of 10 m upslope of the toe, with the next row 5 m further upslope, and the final row another 5 m upslope of the second. The soil nails were modelled as being fully-bonded with a 0.32-m bonded diameter and a design pullout resistance of 60 kPa. The grouted-in-place tendons were modelled with a tensile capacity of 400 kN with face plates. A sketch of the above-described conceptual solution is shown in Figure 2 of Appendix B, although alternative designs could be contemplated during the detailed design process.

For the slope between Old Vienna Road and Big Otter Creek, conceptual modelling indicated four rows of soil nails would be required, distributed equally across the existing slope. The nails were modelled with a 30° inclination from the horizontal, ranging from 14 m long for the top row, 15 m long for the two middle rows, and 16 m long for the bottom row. A 2m lateral spacing with a 0.32-
m bonded diameter and design pullout resistance of 60 kPa were used again for the soil nails in this section. This conceptual solution is shown in Figure 6 of Appendix B for reference.

Soil nailing can be completed with minimal impact to an existing slope face. However, the failed blocks observed during our preliminary slope assessment would need to be removed and regraded to ensure the possibility of localized failures is mitigated. As such, the existing vegetation would likely have to be removed and replaced, so the installation of a surface facing material (such as MacMat R, by Maccaferri) is suggested. The facing would cover the exposed face of the reinforced slope, protecting it from surficial erosion until the slope is fully revegetated.

While this method is expected to adequately address the stability of the slopes in their existing geometries, the application of toe armouring would also be required to address the on-going erosion of the toe of the slopes. As stated previously, the toe armouring is expected to involve the placement of heavy-duty riprap capable of withstanding the flow and water levels of Big Otter Creek. The minimum erosion hazard allowances suggested for the slopes at the Site if they were to be remediated using this approach are summarized below in Table 4.2.

Table 4.2  Suggested Erosion Hazard Allowances for the Site slopes with soil nail reinforcement

<table>
<thead>
<tr>
<th>Cross Section ID</th>
<th>Toe Erosion Allowance (m)</th>
<th>Stable Slope Allowance (H : V)</th>
<th>Approximate Height of Slope (m)</th>
<th>Erosion Access Allowance (m)</th>
<th>Erosion Hazard Allowance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CS-1</td>
<td>2.0</td>
<td>2.6 : 1</td>
<td>10.7</td>
<td>6.0</td>
<td>35.8</td>
</tr>
<tr>
<td>CS-3</td>
<td>2.0</td>
<td>1.7 : 1</td>
<td>9.4</td>
<td>6.0</td>
<td>24.0</td>
</tr>
</tbody>
</table>

4.5 Conceptual Design Solution 3: Granular Shear Key and/or Shear Piles with Toe Armouring

A third viable and practical slope stabilization option, as indicated by preliminary modelling, is a combined-method approach involving the construction of a granular shear key in the toe region of the existing slope near 27 Old Vienna Road (Cross Section 1) and the installation of shear piles along the toe region of the slope bordering Old Vienna Road (Cross Section 3). Alternatively, a shear pile wall was also found to be viable for stabilizing the slope near Cross Section 1 and could be adopted instead of a granular shear key if borrow material and hauling costs were found to be prohibitive.

Granular Shear Keys

Granular shear keys are a soil replacement technique wherein the native soils are removed and replaced with a granular backfill material with greater shear strength, along a large trench constructed perpendicular to the direction of sliding (i.e., along the width of a slope). This type of structure is generally quick to construct and possesses a relatively small footprint. The structure can easily be lengthened or modified depending on observed conditions during construction.

The backfill material is typically angular, high-strength gravel or rockfill sourced from a material that is resistive to physical and chemical degradation. This material is also typically free-draining, which can potentially result in the shear key acting as a cut-off trench and lowering the in-situ
water table (if the shear key is provided with adequate subdrains). While the possible drainage improvement that could be provided by the introduction of a granular shear key in the toe of the slope would be anticipated to result in an improvement to stability, this effect cannot be considered as reliable in the long-term due to the possibility for the clogging of subdrains or fouling of the granular material. As such, any possible drainage benefits resulting from the introduction of a granular shear key have not been considered in the preliminary modelling of this technique. Nevertheless, the free-draining nature of the backfill material makes this method highly suitable to construction next to Big Otter Creek, as its performance is expected to be materially unaffected by the anticipated changes in the river level.

The effectiveness of a granular shear key can be enhanced by constructing a toe berm on top of the granular material. A toe berm typically imparts two primary benefits: an increase to the applied effective stress acting on the granular material, thereby resulting in an increase to shear resistance, and; a direct buttressing (resisting) effect on the upslope materials derived from the addition of mass to the toe of the slope. Conceptual modelling suggested a 1.75-m high toe berm would be required above the contemplated granular shear key. The toe berm in this case could double as toe armouring, to mitigate or eliminate toe scour. Alternative designs could include or consider the use of stone-filled counterforts constructed parallel to the direction of sliding (like French drains), rockfill columns, or steel sheet pile ribs.

For the conceptual design solution, it was found that a granular shear key would need to be cut into the riverbank and extended 7.4 m into the slope to adequately stabilize the overall slope. The shear key trench walls would be sloped at an angle of 1H:1.75V, with a bottom width of 1 m. The estimated cross-sectional area of the contemplated shear key would be 43 m², inclusive of the proposed toe berm. A sketch of the conceptual granular shear key design is shown in Figure 3 of Appendix B.

**Shear Piles**

Shear piles are a type of structural member typically comprising concrete or steel in the form of pre-cast driven micropiles or cast-in-place drilled piles. Construction of drilled piles may also involve temporary or permanent steel casings depending on constructability and the specific design requirements. Shear piles may be customized to different lengths, diameters, battering angles and concrete specifications depending on any project-specific and location-specific requirements or constraints.

Analysis of the existing slopes at the Site indicated the installation of a shear pile wall was a viable approach to achieve the minimum prescribed stability improvement of 30%. The conceptual models suggest such a wall would need to consist of high-strength concrete or steel micropiles with a unit shear capacity of 200 kN near Cross Section 1 and 400 kN near Cross Section 3, installed along the toe of the slope. The piles would need to be keyed at least 0.5 m into the limestone bedrock, anticipated at a depth of 6 to 9 mbgs. A sketch of this conceptual design solution is shown in Figure 7 of Appendix B.

The minimum erosion hazard allowances suggested for the slopes at the Site if they were to be remediated using this combined-method approach are summarized below in Table 4.3.
Table 4.3  Suggested Erosion Hazard Allowances for the Site slopes with a granular shear key

<table>
<thead>
<tr>
<th>Cross Section ID</th>
<th>Toe Erosion Allowance (m)</th>
<th>Stable Slope Allowance (H : V)</th>
<th>Approximate Height of Slope (m)</th>
<th>Erosion Access Allowance (m)</th>
<th>Erosion Hazard Allowance (m)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>2.6 : 1</td>
<td>10.7</td>
<td>6.0</td>
<td>35.8</td>
</tr>
<tr>
<td>CS-3</td>
<td>2.0</td>
<td>1.7 : 1</td>
<td>9.4</td>
<td>6.0</td>
<td>24.0</td>
</tr>
</tbody>
</table>

4.6  Summary of Remedial Options

Except for the “Do Nothing” option, the above conceptual design solutions for the slopes along Old Vienna Road are each considered to be viable and practical options to address the observed instabilities and increase stability by a factor of 30%. The relative benefits, shortfalls, costs and suitability of these options are summarized and compared in Table 4.4 below. More detailed cost estimate breakdowns are presented in Appendix D. It should be noted that the considerable differences between the north and south parts of the Site, specifically the existing slope angle and space availability at the top, will likely necessitate the adaptation of each proposed method to the location-specific geometry of the slope. Therefore, the costs provided below should be taken only as high-level estimates at this time and are no substitute for a full, detailed quote prepared by a qualified contractor experienced in performing the described works. Furthermore, seismic design requirements have not been considered at this phase of design and could result in increased costs.
Table 4.4  Summary of Conceptual Design Options recommended for Old Vienna Road slopes

<table>
<thead>
<tr>
<th>Conceptual Design Option</th>
<th>Brief Description of Design Option</th>
<th>Anticipated Benefits</th>
<th>Possible Shortfalls / Challenges</th>
<th>High-Level Cost Estimate Summary</th>
<th>Most Suitable Location(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Do Nothing</td>
<td>Monitoring program implemented. Slope and/or road maintenance/repairs could be required over time. Possibility for damage to/loss of property and risk to public safety.</td>
<td>Defer cost of full remediation to a later time. Use monitoring data to further refine the design of a remedial solution.</td>
<td>Risk to public safety is unacceptable if slope is left unmonitored. This option is not viable in the long-term.</td>
<td>$38K to $52K* + Variable Cost to Monitor / Report + Unknown cost of possible damages</td>
<td>No suitable locations (i.e., not recommend-ed)</td>
</tr>
<tr>
<td>CONCEPTUAL DESIGN OPTION 1: Partial Slope Reconstruction with Benched Retaining Walls and Toe Armouring</td>
<td>Slope is benched using retaining walls and locally regraded; toe of slope is armoured to mitigate continued erosion / degradation of geometry.</td>
<td>Common construction equipment / machinery required.</td>
<td>Would require removal of all existing vegetation on slope faces. Requires removal (hauling) of large volumes of material. Requires relatively large footprint / setback distance. Reduction of property size at crest of slope.</td>
<td>$2.5M to $3.5M or $12.2K to $16.1K / m</td>
<td>27 Old Vienna Road (CS-1, CS-2)</td>
</tr>
<tr>
<td>CONCEPTUAL DESIGN OPTION 2: Soil Nails with Toe Armouring</td>
<td>Slope is reinforced using soil nails in a grid pattern; toe of slope is armoured to mitigate continued erosion / degradation of geometry.</td>
<td>Limited removal of material required. Relatively quick installation.</td>
<td>Specialized equipment required for installation. Possibly constrained/obstructed by underground utilities below Old Vienna Road.</td>
<td>$2M to $3M or $10.6K to $13.9K / m</td>
<td>All areas**</td>
</tr>
<tr>
<td>CONCEPTUAL DESIGN OPTION 3: Granular Shear Key and/or Shear Piles with Toe Armouring</td>
<td>27 Old Vienna Road slope is reinforced using a trenched granular shear key and toe berm; Old Vienna Road slope is reinforced using shear piles; toe of slope is armoured to mitigate continued erosion / degradation of geometry.</td>
<td>Limited impact to vegetation further upslope. Creation of usable riverside land (top of berm) that could serve as a recreational trail / nature lookout. Possible localized improvement to slope drainage.</td>
<td>Difficult to construct next to river if water levels are high. If granular shear key is adopted, sequenced excavation must be coordinated to maintain safety during construction, and large quantity of fill material to be hauled in and excavated material to be removed.</td>
<td>$1M to $2M*** or $4.2K to $8.3K / m</td>
<td>All areas</td>
</tr>
</tbody>
</table>

* Cost will depend on length and complexity of monitoring program and possible extent of future damages.

** Mid-slope area behind 27 Old Vienna Road may be difficult to reach and presence of underground utilities below Old Vienna Road could potentially obstruct soil nail installation depending on installation depth and angle.

*** Cost of granular backfill and riprap highly dependent on nearby availability of granular fill borrow pits.
4.6.1 Recommended Option: Conceptual Design Solution #3

The selection of a suitable option from the above-noted solutions will come down to the relative cost and environmental impact of the considered options.

In the interest of selecting the option with the lowest anticipated costs and the least environmental impact, we recommend stabilizing the existing slopes by adopting Conceptual Design Solution #3, by constructing a shear pile wall comprised of micropiles keyed into bedrock. The next preferred option is Conceptual Design Solution #2, the use of soil nails. Each option is anticipated to involve minimal volumes of hauled material and staging / logistical requirements.

In addition to the recommended remedial work, the slope faces should be redressed such that any failed material is removed and/or adequately scaled back to mitigate the potential for any local failures and continued surface erosion. The regrading of the slope is also anticipated to facilitate access for shear pile or soil nail installation. The application of a protective mat to mitigate surface erosion and expedite revegetation is recommended following completion of the redressing and installation work.

If the existing gabion wall is left in place, the placement of riprap toe armouring along that portion of the slope may not be necessary, possibly creating an additional source of cost savings. However, riprap toe armouring will still be necessary along the exposed portions of the slope toe. Riprap should be keyed at least 0.5 m into the existing ground surface such that it will not dislodge or slide into Big Otter Creek. The installation of riprap should be performed after the global slope stabilization measures are completed, to mitigate the likelihood of an upslope failure during keying of the riprap.

All remedial work should be undertaken between late July and early October, when groundwater levels are anticipated to be at a seasonal low. This is expected to result in a temporarily increased state of stability for the existing slopes, which can be exploited to mitigate the risk of failures during work on the slopes.

Finally, considering the potential impacts of some of the proposed remedial measures, additional slope stability evaluations beyond our original scope of work may also be required to ensure long-term global stability of the up- or downstream slopes and/or upslope main roadways.
5. CLOSURE

The detailed design of one conceptual solution selected by the Town can be commenced immediately after DST receives details of the Town's decision and authorization to proceed. The detailed design process will involve the preparation of a 60% design, 90% design, and a 100% design through a phased approach to ensure the Town's continued satisfaction and transparency throughout the entire process.

We thank you for this opportunity to provide our services to the Town of Tillsonburg. If you have any questions regarding the content of this document, please do not hesitate to contact the undersigned.

Sincerely,

For DST Consulting Engineers Inc., A Division of Englobe Corp.:

Geotechnical Engineer

Brennan Bailey, M.A.Sc., P.Eng.
Geotechnical Engineer, Associate

Farbod Saadat, Ph.D., P.Eng.
Chief Geotechnical Engineer, Associate
APPENDIX A
LIMITATIONS OF REPORT
LIMITATIONS OF REPORT
CONCEPTUAL DESIGN OF SLOPE REMEDIATION OPTIONS

The data, conclusions and recommendations presented in this report, and the quality thereof, are based on a scope of work authorized by the Client. Note that no scope of work, no matter how exhaustive, can identify all conditions below ground. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the specific locations tested, and conditions may become apparent during construction which were not detected and could not be anticipated at the time of the Site investigation. The number of boreholes may not be sufficient to determine all the factors that may affect construction methods, remediation option performance, and costs, e.g. the thickness of surficial topsoil, fill layers, bedrock depth, and presence of boulders/cobbles may vary markedly and unpredictably. The contractors bidding on this project or undertaking the future construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work. Conditions can also change with time. It is recommended practice that DST Consulting Engineers Inc. be retained during construction to confirm that the subsurface conditions throughout the Site do not deviate materially from those encountered in the boreholes.

The factual content, conceptual design options, broad budgetary estimates, and recommendations given in this report are intended only for the guidance of the Client. The conceptual design options presented herein are intended to provide the Client with high-level options for improving the stability of the existing slopes at the Site. While a great deal of effort was taken to select the most critical cross sections for analysis, the ground conditions and slope geometries between the selected cross sections may vary considerably and the stability of the slope between the considered cross sections may not be the same as those presented in this report. Furthermore, the number of critical cross sections considered in this report may not be sufficient to fully constrain the observed instabilities and current conditions of the slopes at the Site, nor those which extend beyond the current scope of work. Lastly, the conceptual design options presented in this report are not intended to be comprehensive and exclusive, with other practical and effective remedial solutions likely to exist.

The conceptual designs contained herein are not intended for construction and should not be construed as final design drawings. Any construction work performed based on the conceptual designs shall be performed at the sole risk of the Contractor.
APPENDIX B
CONCEPTUAL DESIGN SOLUTION SKETCHES
FIGURE 1: Cross Section 1 - Conceptual Design Solution #1: Benched Retaining Walls with Toe Armouring
FIGURE 2: Cross Section 1 - Conceptual Design Solution #2: Soil Nails with Toe Amouring
FIGURE 3: Cross Section 1 - Conceptual Design Solution #3: Granular Shear Key
FIGURE 4: Cross Section 2 - Native Slope with Toe Armouring
FIGURE 5: Cross Section 3 - Conceptual Design Solution #1: Tied-Back Retaining Walls with Toe Armouring
BH19-04
Old Vienna Road
Sidewalk
BH19-04
Old Vienna Road
Sidewalk
Existing slope to be maintained. Remove and regrade any slumped material.
Soil Nails:
4 rows, installed at 30 Degree inclination
1-ft bond diameter, fully grouted
14 to 16 m length, 2-m lateral spacing
Big Otter Creek
Riprap Toe Armouring

FIGURE 6: Cross Section 3 - Conceptual Design Solution #2: Soil Nails with Toe Armouring
FIGURE 7: Cross Section 3 - Conceptual Design Solution #3: Shear Piles with Toe Armouring
APPENDIX C
ESTIMATED RATES OF EROSION
Table C.1  Estimated rates of erosion experienced at the Site based on the limited available historic imagery and topography data

<table>
<thead>
<tr>
<th>Location</th>
<th>Feature</th>
<th>Estimated Lateral Magnitudes of Erosion (m)</th>
<th>Calculated Average Lateral Rates of Erosion (mm/yr)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 m north of Outfall</td>
<td>Top of Bank</td>
<td>0.00</td>
<td>3.29</td>
<td>nd</td>
</tr>
<tr>
<td>Outfall</td>
<td>Top of Bank</td>
<td>0.00</td>
<td>7.17</td>
<td>nd</td>
</tr>
<tr>
<td>Section A</td>
<td>End of Gabion</td>
<td>0.00</td>
<td>24.32</td>
<td>nd</td>
</tr>
<tr>
<td></td>
<td>Top of Slope</td>
<td>0.00</td>
<td>2.71</td>
<td>nd</td>
</tr>
<tr>
<td>Section B</td>
<td>Top of Slope</td>
<td>0.00</td>
<td>7.83</td>
<td>nd</td>
</tr>
<tr>
<td>27 Old Vienna Road (Former Site of House)</td>
<td>Top of Slope</td>
<td>0.00</td>
<td>2.47</td>
<td>9.76</td>
</tr>
<tr>
<td></td>
<td>Top of Slope</td>
<td>0.00</td>
<td>5.88</td>
<td>nd</td>
</tr>
<tr>
<td>Section C</td>
<td>Top of Slope</td>
<td>0.00</td>
<td>5.23</td>
<td>nd</td>
</tr>
<tr>
<td>Averages</td>
<td>Top of Bank</td>
<td>n/a</td>
<td>n/a</td>
<td>5.23</td>
</tr>
<tr>
<td></td>
<td>Top of Slope</td>
<td>n/a</td>
<td>2.47</td>
<td>6.55</td>
</tr>
</tbody>
</table>

nd: No data
n/a: Not applicable

Estimated loss of gabion wall, as measured from south end of wall in 1992 vs. 2019.