REPORT

Geotechnical Assessment
Erie Shore Drive, Municipality of Chatham Kent, Ontario

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Distribution List

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Executive Summary

This report presents the results of the geotechnical assessment carried out to evaluate the current stability of the existing Erie Shore Drive roadway embankment/dyke structure, extending from the western limit of McGeachy Pond to approximately 1.9 kilometres westward (adjacent to municipal numbers 17982 to 18416 Erie Shore Drive), in the Municipality of Chatham-Kent, Ontario. The section of the dyke structure included in the current assessment was designated based on the locations where historical overtopping has been observed and where future overtopping would be anticipated based on topography, ongoing erosion and maintenance requirements at current discharge channel locations and recent lake level projections.

Due to rising lake levels, overtopping events from storm surges/wind-driven lake water have been occurring since March of 2016. The frequency of overtopping events increased and an initial evaluation of the impacts on the integrity and stability of the dyke was carried out in 2018. The results of the initial evaluation indicated that the dyke structure was stable under normal (dry) conditions but would become unstable to marginally stable during sustained flooding events (with floodwaters some 100 millimetres higher than the embankment crest elevation). The short-term mitigation approach recommended in the 2018 report was to use concrete blocks/barriers (each block having dimensions of 600 millimetres by 600 millimetres by 1,200 millimetres) to effectively raise the grade of the dyke to control overtopping and to direct floodwaters to rip-rap lined channels discharging to the municipal drain.

Due to increasingly high lake levels, the magnitude of the overtopping events has increased (floodwaters some 300 millimetres higher than the crest elevation of the embankment). Due to the increased volumes of water diverted to the rip-rap lined channels, the discharge flow velocities have increased and the erosion control measures (geotextile and rip-rap) in the channels are no longer effective. This has resulted in undercutting, scour and erosion of the channels and a requirement for extensive reconstruction/maintenance work following flood events. The saturation of the dyke materials, and effects of ongoing erosion and scour, have negatively impacted the stability of the dyke structure. Accordingly, the current mitigation approach is considered unsustainable, particularly given the current forecasts for record high lake levels in 2020.

A static lake level elevation of 174.92 metres was recorded on February 10, 2020. This exceeds the previous record high lake levels measured in January, February and March of 1985/1986. There is a 50 per cent probability of above-normal precipitation rates in February, March and April 2020. The previous highest annual static lake level elevation in the vicinity of Erie Shore Drive was 175.19 metres, measured in June of 2019. There is reportedly a 40 per cent probability of this level being exceeded in April 2020 and a 5 per cent probability of a static lake level elevation of 175.25 metres.

The lowest top of pavement elevation on Erieau Road is 173.78 metres, at a point located north of McGeachy Pond approximately 170 metres northwest of the intersection of Erieau Road and Erie Shore Drive. That point is approximately 1.1 metres lower than the current static lake level, 1.4 metres below the 40 per cent probable level for April 2020 and about 1.5 metres below the 5 per cent probable level.

A recent visual assessment of the embankment indicated a recent slope failure, the development of new longitudinal tension cracks and the widening of existing cracks indicating that approximately the north half of the embankment is moving northward toward the adjacent municipal drain.
Based on the visual assessment, the results of new slope stability analyses, predicted lake levels and the historical storm frequency in the late winter/early spring months, it is considered that there is approximately a 5 to 40 per cent chance for overtopping conditions to develop that would render the dyke unstable with progressive failures potentially leading to a significant breach. Although the probability of a major breach is less than 50 per cent, the consequences of such a breach would be severe given the low ground surface elevations to the north and the low elevation of Erieau Road. Erieau Road provides the only access to the Village of Erieau and flooding due to a major breach of the dyke would effectively isolate the community. In addition, the overtopping events and any resulting failure of the embankment would preclude direct emergency vehicle access to the subject section of Erie Shore Drive and the residences to the south.

In the near term, work is required to raise the embankment grade to prevent uncontrolled overtopping. Work is also required to provide more robust erosion control measures to reduce erosion/scour at the locations of discharge channels. Concrete blocks/earth fill used to raise the grade should be placed as close to south edge of roadway as possible (so as not to destabilize the north slope) without imposing loads on existing underground utilities that would cause compression of the underlying organic soils and resulting damage to the utilities. Given the lake level forecasts, the historical late winter/early spring storm frequency, and the severity of the consequences of a breach, there is some urgency in completing these efforts as soon as reasonably possible. As noted previously, the embankment serves as both a transportation corridor and a flood control structure. Given the current spatial and geometric constraints (width of right of way, proximity of Lakeshore Drain, slope inclinations), the grade raise required to prevent overtopping and the flatter slopes required for the discharge spillways preclude the continued use of the dyke structure as a transportation corridor for the short- to medium-term.

In the longer term, work is required to also protect the embankment from saturation during flood events, to reduce the flow velocities in the discharge spillways, and to reinforce and/or flatten the north slope to enhance stability and further reduce spillway discharge velocities. Flattening the slope would require the relocation of the municipal drain further to the north.

For both the short-term and long-term mitigative measures, the timing and duration of construction activities would be highly weather dependent. If uncompleted works are exposed to severe weather/flooding events, the stability of the dyke would be further compromised potentially leading to severe erosion and failure. Accordingly, construction activities should be carried out when weather permits and the work should be carried out in relatively short sections that can be completed during the anticipated windows of suitable weather.

Alternatively, the flood control function could be performed by a new seawall and armouring system installed along the Lake Erie shoreline. However, such an alternative would be technically complex and could prove to be economically infeasible.

Conceptual designs are presented and discussed for short-term and longer-term mitigative measures.
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1.0 INTRODUCTION

This report presents the results of the geotechnical assessment carried out to evaluate the current stability of the existing Erie Shore Drive roadway embankment/flood control structure, extending from the western limit of McGeachy Pond to approximately 1.9 kilometres westward (adjacent to municipal numbers 17982 to 18416 Erie Shore Drive), in the Municipality of Chatham-Kent, Ontario. The location of the site is shown on the Key Plan, Figure 1. The assessment consisted of a review of the available information for the site, a site reconnaissance, surveying of cross sections of the north embankment slope, and slope stability analyses for both dry and inundated/flooded conditions.

The section of the dyke structure included in the current assessment was designated based on the locations where historical overtopping has been observed and where future overtopping would be anticipated based on topography, ongoing erosion and maintenance requirements at current discharge channel locations and recent lake level projections.

The purpose of the exploration was to assess the existing pavement and near surface soil and groundwater conditions at the site and to provide geotechnical recommendations and comments pertaining to protection for the existing roadway embankment/dyke structure on recent flooding (late August 2019 to February 2020). Additional site visits have been carried out during and after flooding events.

This report should be read in conjunction with the attached document "Important Information and Limitations of This Report", which comprises an integral component hereof. The reader’s attention is specifically drawn to this material, as it is essential for proper use and interpretation of the information presented and discussed herein.

2.0 BACKGROUND

Golder Associates Ltd. (Golder) previously carried out two geotechnical exploration programs along Erie Shore Drive and the results were provided in the following:

- Golder Report No. 1792661 titled “Geotechnical Exploration, Erie Shore Drive, Municipality of Chatham Kent, Ontario” dated June 6, 2018; and

For this report, BH-201 to BH-203 will be referenced due to the locations and drilled depths of the Boreholes. The approximate locations of boreholes are shown on Figure 1.

The south edge of the Erie Shore Drive embankment is located some 26 to 48 metres north of the Lake Erie shoreline. The embankment height ranges from about 3 to 5 metres above the Lake Shore municipal drain which is located along the toe of the north embankment slope. The embankment has a crest width of about 7 metres and the inclination of the north embankment slope ranges from about 1.1 to 2.7 horizontal (H) to 1.0 vertical (V) or between about 20 to 42 degrees from the horizontal. The embankment serves as both a transportation corridor (two-lane roadway) and a flood control structure, protecting low-lying lands and infrastructure to the north.

Due to rising lake levels, overtopping events from storm surges/wind-driven lake water have been occurring since March of 2016. The frequency of overtopping events increased and an initial evaluation of the impacts on the integrity and stability of the dyke was carried out in 2018. Golder’s 2018 report, referenced above, provides the
results of the June 2018 geotechnical exploration and evaluation of the stability of the embankment. The results of the 2018 evaluation indicated that the dyke structure was stable under normal (dry) conditions but would become unstable to marginally stable during sustained flooding events (with floodwaters some 100 millimetres higher than the embankment crest elevation). The short-term mitigation approach recommended in the report was to use concrete blocks/barriers (each block having dimensions of 600 millimetres by 600 millimetres by 1,200 millimetres) to effectively raise the grade of the dyke to control overtopping and to direct floodwaters to rip-rap lined channels discharging to the municipal drain.

Due to increasingly high lake levels, the magnitude of the overtopping events has increased (floodwaters recently some 300 millimetres higher than the crest elevation of the embankment). Due to the increased volumes of water diverted to the rip-rap lined channels, the discharge flow velocities have increased and the erosion control measures (geotextile and rip-rap) in the channels are no longer effective. This has resulted in undercutting, scour and erosion of the channels and a requirement for extensive reconstruction/maintenance work following flood events. The saturation of the dyke materials, and effects of ongoing erosion and scour, have negatively impacted the stability of the dyke structure. Accordingly, the current mitigation approach is considered unsustainable, particularly given the current forecasts for record high lake levels in 2020.

3.0 PROCEDURE

On June 26, 2019, a site visit was carried out by Golder to visually assess the pavement structure and dyke slopes in the subject section of Erie Shore Drive that had experienced flooding and overtopping events. Based on the results of visual inspection it was recommended that three boreholes be advanced through the westbound lane adjacent to the following locations:

- Municipal Address 18290;
- Municipal Address 18190; and
- Municipal Address 18100.

The boreholes were drilled using a truck mounted power auger supplied and operated by a specialist drilling contractor under the direction of a geotechnical engineer from our staff. Standard penetration testing and sampling was carried out in the boreholes at suitable intervals of depth using 38-millimetre inside diameter split spoon sampling equipment and an automatic hammer in accordance with ASTM D1586 “Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils”. All of the samples obtained during the exploration were transported to our laboratory for further examination and representative classification testing.

4.0 EXISTING CONDITIONS

4.1 Meteorological Data, Lake Levels and Ground Surface Elevations

Data on predicted 2020 rainfall rates, the frequency of historical storm events and Lake Erie water levels was obtained from the Municipality of Chatham Kent and the Lower Thames Conservation Authority.
Based on data recorded from 2017 to 2020, on average, about 5 critical storm events per year (defined as south to southwest winds with speeds in excess of 25 kilometres per hour for a duration of at least 3 hours) occur in each of the late winter/early spring months.

A static lake level elevation of 174.92 metres was recorded on February 10, 2020. This exceeds the previous record high lake levels measured in January, February and March of 1985/1986. There is a 50 per cent probability of above-normal precipitation rates in February, March and April 2020. The previous highest annual static lake level elevation in the vicinity of Erie Shore Drive was 175.19 metres, measured in June of 2019. There is reportedly a 40 per cent probability of this level being exceeded in April 2020 and a 5 per cent probability of a static lake level elevation of 175.25 metres.

The crest elevation of the flood control dyke (top of pavement elevation) ranges from about 175.3 to 177.4 metres with a typical elevation of about 175.45 metres in the subject section of Erie Shore Drive. Wind-driven storm surges and waves have previously resulted in reported overtopping of the dyke with sustained dynamic water elevations of 175.75 metres (i.e., approximately 0.3 metres higher than the current dyke crest).

The lowest top of pavement elevation on Erieau Road is 173.78 metres, at a point located north of McGeachy Pond approximately 170 metres northwest of the intersection of Erieau Road and Erie Shore Drive. That point is approximately 1.1 metres lower than the current static lake level, 1.4 metres below the 40 per cent probable level for April 2020 and about 1.5 metres below the 5 per cent probable level.

4.2 Geological Mapping

The site lies within the physiographic region of southern Ontario known as the St. Clair Clay Pains as identified in “The Physiography of Southern Ontario” by Chapman and Putnam (1984). The predominant soils in the area of the site consist of extensive clay plains. Based on the Ontario Geological Survey Map P.1973, “Quaternary Geology, Bothwell-Ridgetown Area”, the soils adjacent to Lake Erie consist of organic deposits of peat and muck. The soil changes to lacustrine deep water deposits of sand, silt and clay further north from Lake Erie.

Based on the Geological Survey of Canada, Department of Energy, Mines and Resources Map 1262A, “Geology, Toronto-Windsor Area”, Scale 1:250,000, dated 1969, the subcropping bedrock in the area of the site is reported to consist of black bituminous shale with silty shale interbeds belonging to the Kettle Point formation of Upper Devonian Age. Based on geological mapping, the bedrock surface is reported to be at approximately elevation 136 metres, or at a depth of approximately 40 metres below to the top of slope surface.

4.3 Subsurface Conditions

Asphalt was encountered at the pavement surface at all three locations and was about 80 to 150 millimetres thick. Beneath the asphalt, granular base and subbase were encountered. The granular fill was about 500 to 800 millimetres thick.

Beneath the pavement structure, the subgrade conditions consisted of cohesive fill materials with varying amounts of organics. Where fully penetrated in the current and previous boreholes, the cohesive fill materials ranged from about 1.5 to 1.7 metres in thickness. Samples of the subgrade materials had SPT N values of 4 to 8 blows per 0.3 metres and water contents ranging from about 17 to 29 per cent.
Underlying the cohesive fill, layers of peat ranging from about 0.3 to 1.0 metres in thickness were encountered in the three boreholes at depths of about 2.1 to 2.5 metres. The peat had SPT N values ranging from 2 to 7 blows per 0.3 metres with water contents of about 240 to 340 per cent.

The subsurface soil conditions are established at the borehole locations only and should be expected to vary between and beyond these locations.

Groundwater was encountered in BH-201 at a depth of about 2.1 metres below ground surface. BH-202 and BH-203 remained dry during drilling on July 22, 2019. Groundwater and seepage conditions should be expected to fluctuate seasonally and in response to significant precipitation events, lake levels and during periods of flooding.

Based on the nature of the encountered materials and historical local practice, the embankment was likely constructed using the materials excavated from the municipal drain during its construction. That would account for the mixed cohesive fill with varying amounts of topsoil placed over peat and native organic silt. The materials and the likely method of construction are not appropriate for a critical flood control structure. The fill was apparently not uniformly and adequately compacted during placement and consists of clods/blocks of cohesive material that were not adequately reworked and consolidated. As a result, joints and voids are present between the various clods and blocks of cohesive material and the overall permeability of the soil mass is not as low as would be the case for properly placed and compacted cohesive fill. In addition, the peat and organic silt materials should have been removed from beneath the footprint of the structure as these materials are highly compressible, inherently weak and unsuitable as an embankment foundation.

5.0 VISUAL INSPECTIONS

On June 26, 2019, a site visit was carried out following an overtopping event to visually assess the pavement structure and dyke slopes. The following observations were noted during the site visit:

- No visual signs of rutting of asphalt;
- Longitudinal cracking, both older and relatively new, was noted along the outside edge (0.8 to 0.9 metres from edge) of the eastbound lane;
- The outside edge of the westbound lane appeared to have settled about 5 to 10 millimetres in some areas; and
- Areas of slope erosion, likely caused by the recent flooding, have been restored using filter cloth and rip-rap.

A subsequent visual assessment was carried out following an overtopping event on November 4, 2019, and it was noted that the Municipality had carried out the short-term mitigative recommendations provided in our letter dated August 9, 2019. The westbound lane had been closed. Concrete blocks with clay fill placed behind them had been placed within the entire length of Erie Shore Drive affected by overtopping. The concrete blocks had been configured to direct surface water to the areas of the embankment slope that had been repaired with rip-rap and geotextile. However, during our site visit, it was noted that the rip-rap and geotextile should be extended further down to the toe of the slope extending into the municipal drain and fanned out along the base of the drain to reduce erosion and scour.

It was also noted during our site visit that there were no visual signs of distress along the westbound lane where the additional blocks and clay fill had been placed.
On February 13, 2020, a site visit was carried out by Golder to visually inspect the roadway and the north slope after a recent flooding event. Surveying was also carried out at three cross section locations. The embankment cross section locations are shown on Figure 1 and the cross sections are shown on Figures 2 to Figure 4. The following observations were noted during the site visit:

- No visual signs of rutting of asphalt; however, some minor depressions were noted;
- Longitudinal tension cracking, both older and relatively new, was noted along the outside edge (0.1 to 1.6 metres from north edge) of the westbound lane;
- The previously existing longitudinal tension cracks were noticeably wider than observed during the previous visual assessments with crack widths up to 40 millimetres;
- Longitudinal cracking was noted in the eastbound lane in area of MN 18100;
- An approximately 4-metre wide slope failure was noted in the north embankment slope north of MN 18100.

6.0 VISUAL SLOPE ASSESSMENT

Slope stability assessments of the three surveyed cross sections were carried out using the Ontario Ministry of Natural Resources (MNR) Slope Stability Rating Chart. Soil classifications at the site were based on visual observations of soils exposed on the slope face and the results of the boreholes drilled as part of the geotechnical exploration. The slope sections for the north slope along the subject section of Erie Shore Drive received ratings ranging between 42 and 48, indicating that, based on the MNR rating system, the slope sections have a “moderate” potential for instability. It should be noted that this is the highest potential for instability addressed by the MNR rating system.

7.0 SLOPE STABILITY ANALYSES

The factor of safety for slope stability is defined as the total forces or moments acting to resist failure divided by the total forces or moments acting to destabilize the slope. A factor of safety of unity indicates incipient failure since the destabilizing and stabilizing forces are equal. Instabilities can be manifested as deep rotational (global) failures or more shallow translational failures and/or sloughing. The analyses were conducted to assess the stability of the existing slopes based on the geological soil data, the existing topographic information and the field data. The table below provides a range of factors of safety for the terms used in this report.

<table>
<thead>
<tr>
<th>Term</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stable</td>
<td>Equal to or greater than 1.3</td>
</tr>
<tr>
<td>Marginally Stable</td>
<td>Between 1.0 and 1.3</td>
</tr>
<tr>
<td>Unstable</td>
<td>Equal to or less than 1.0</td>
</tr>
</tbody>
</table>
The subsurface conditions and soil properties used in the analyses were based on the results of the boreholes drilled in close proximity to the subject slopes as part of the geotechnical exploration and our knowledge of the range of mechanical properties of these soil types.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Unit Weight (kN/m³)</th>
<th>Effective Cohesion Intercept, c' (kPa)</th>
<th>Effective Angle of Internal Friction, $\phi'$ (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compact Granular Fill</td>
<td>18</td>
<td>0</td>
<td>32</td>
</tr>
<tr>
<td>Firm to Stiff Cohesive Fill</td>
<td>20</td>
<td>0</td>
<td>26</td>
</tr>
<tr>
<td>Organics</td>
<td>16</td>
<td>0</td>
<td>24</td>
</tr>
<tr>
<td>Loose to Compact Silty Sand</td>
<td>19</td>
<td>0</td>
<td>30</td>
</tr>
<tr>
<td>Firm to Very Stiff Silty Clay</td>
<td>21</td>
<td>0</td>
<td>28</td>
</tr>
</tbody>
</table>

No groundwater seepage was observed on the slope during our assessment on February 13, 2020. The boreholes contained no free water during drilling.

8.0 DISCUSSION

The slope failure noted on February 13, 2020 indicates that under certain conditions the slope is unstable. The new longitudinal tension cracks and widening of the previously observed tension cracks indicates that the embankment soil mass within approximately the northern half of the dyke is moving northward. There are generally two different mechanisms for slope failure: 1) deep-seated rotational, or global, failure during which essentially an entire longitudinal section of the embankment would fail in one overall movement; and 2) shallow translational failure during which portions of the slope (typically 0.5 to 1 metre in thickness) slide down the face of the slope - if these materials are washed away due to storm/flood activity, progressive oversteepening and subsequent shallow failures would occur, ultimately resulting in the failure of the full embankment.

Based on the results of our slope stability analyses, under normal (dry) conditions, the slopes in their existing configurations have an estimated factor of safety against a deep global rotational failure of greater than 1.3. However, under normal (dry) conditions the slopes are only marginally stable with respect to more shallow translational failures and/or sloughing with a factor of safety of approximately 1.1 to 1.2.

Analyses were also performed at selected cross-sections for fully saturated soil conditions to emulate the periodic flooding conditions that occur at the roadway. Given the moderately permeable cohesive fill and organic soils comprising the majority of the dyke embankment, while also considering the ingress of surface water through the existing asphalt cracks, the models used for the analyses assumed the embankment would take approximately 1 to 3 days to fully saturate with approximately 300 millimetres of sustained flood water on top of the roadway. Under the fully-saturated conditions, the factor of safety against a deep global rotational failure was between 0.9
and 1.1 which indicates an unstable (in the steeper sections) to marginally stable slope under sustained flooding with respect to deep-seated rotational failures.

Under similar conditions (i.e., overtopping by 300 millimetres of water), only about 4 to 8 hours of sustained inundation would be required to fully saturate the upper metre of the embankment and slope (this is exacerbated by the asphalt cracks which allow water ingress and more rapid saturation of the underlying materials). The resulting saturation and erosive conditions of the upper portions of the embankment and slope reduce the factor of safety for shallow translational failures and sloughing to less than 1.0, indicating unstable conditions and the onset of failure. Without the presence of the municipal drain, the materials from these shallow failures would generally accumulate at the toe of the slope, effectively reducing the slope inclination and eventually resulting in a marginally stable slope. However, due to the presence of the drain and the high flow velocities that occur under storm/flood conditions, the failed materials are eroded and transported away with the flowing water resulting in progressive over-steepening and regression of the slope toward the south – into the roadway. The oversteepening would also potentially cause larger deeper movements under submerged conditions resulting in a breach in the dyke structure and flooding of the lands to the north. A slope inclination of 2.5H to 1.0V or flatter would be required to achieve a factor of safety of 1.3 or greater for translational failures under inundated conditions.

Based on the predicted lake levels and the historical storm frequency in the late winter/early spring months, it is considered that there is approximately a 5 to 40 per cent (global rotational or progressive shallow translational failures, respectively) chance for conditions to develop that would render the dyke unstable with progressive failures potentially leading to a significant breach. Although the probability of a major breach is less than 50 per cent, the consequences of such a breach would be severe given the low ground surface elevations to the north and the low elevation of Erieau Road. Erieau Road provides the only access to the Village of Erieau and flooding due to a major breach of the dyke would effectively isolate the community. In addition, the overtopping events and any resulting failure of the embankment would preclude direct emergency vehicle access to the subject section of Erie Shore Drive and the residences to the south.

9.0 RECOMMENDATIONS

In the near term, work is required to raise the embankment grade to prevent uncontrolled overtopping. Work is also required to provide more robust erosion control measures to reduce erosion/scour at the locations of discharge channels. Additional concrete blocks/earth fill used to raise the grade on the order of 1.0 to 1.5 metres should be placed as close to south edge of roadway as possible (so as not to destabilize the north slope) without imposing loads on existing underground utilities that would cause compression of the underlying organic soils and resulting damage to the utilities. Given the lake level forecasts, the historical late winter/early spring storm frequency, and the severity of the consequences of a breach, there is some urgency in completing these efforts as soon as reasonably possible. As noted previously, the embankment serves as both a transportation corridor and a flood control structure. Given the current spatial and geometric constraints (width of right of way, proximity of Lakeshore Drain, slope inclinations), the grade raise required to prevent overtopping and the flatter slopes required for the discharge spillways preclude the continued use of the dyke structure as a transportation corridor for the short- to medium-term.

In the longer term, work is required to also protect the embankment from saturation during flood events, to reduce the flow velocities in the discharge spillways, and to reinforce and/or flatten the north slope to enhance stability.
and further reduce spillway discharge velocities. Flattening the slope would require the relocation of the municipal drain further to the north. Alternatively, the flood control function could be performed by a new seawall and armouring system installed along the Lake Erie shoreline. However, such an alternative would be technically complex and could prove to be economically infeasible.

For both the short-term and long-term mitigative measures, the timing and duration of construction activities would be highly weather dependent. If uncompleted works are exposed to severe weather/flooding events, the stability of the dyke would be further compromised potentially leading to severe erosion and failure. To address this risk, construction activities should be carried out when weather permits and the work should be phased and carried out in relatively short sections that can be completed during the anticipated windows of suitable weather.

The following recommendations have been separated into three categories: short term, intermediate term and long term.

9.1.1 Short Term Protection

Our recommended approach for short term mitigation measures includes using the materials/methods already in use with additional alterations to help mitigate further damage to the dyke’s slope and to allow time for the design for a more permanent solution. Given the current spatial and geometric constraints (width of right of way, proximity of Lakeshore Drain, existing slope inclinations), the measures required to prevent overtopping and incorporate flatter slopes required for the emergency spillways preclude the continued use of the dyke structure as a transportation corridor for the short-term. The recommended works consist of:

- Use concrete barriers and cohesive fill material placed in the eastbound lane to direct water to emergency spillways;
- Repair areas of existing slope failure by excavating the failed materials and backfilling with rip-rap underlain by a robust non-woven geotextile (terrafix 800R, or equivalent);
- Reduce/optimize the number of emergency spillways;
- The spillways should be relatively shallow and broad and installed with as flat a slope as possible – potentially resulting in some excavation below the road surface in the northern portion of the embankment crest.
- The erosion protection should be augmented to include concrete cable mats underlain by a heavy, robust non-woven geotextile (terrafix 1200R, or equivalent); the geotextile and armor protection should extend across the drain and extended up the north bank of the drain at the discharge point and downstream (conceptual spillway details are shown on Figure 5);
- The spillway inlets should be raised relative to the crest of the block wall and earth fill such that discharges are less frequent and flow velocities are reduced;
- The concrete blocks should be augmented by constructing a clay berm along the eastbound lane and diverting the water to the flatter, better-protected spillway(s);
- The slopes of the new berm would need to be protected by with a non-woven geotextile (Terrafix 800R) covered with rip rap;
- Continue monitoring of roadway/dyke during and after flooding events; and
- Routine maintenance of berm and channels.
9.1.2 Longer-Term Mitigation

Our recommended approach for longer-term mitigation measures includes a subsurface flow barrier along the south edge of the existing roadway, relocating the municipal drain further north and reconstructing/raising the embankment crest while flattening the north slope as shown on Figure 6. An alternative approach to provide long-term protection of the dyke would be the installation of a coastal seawall along the Lake Erie shoreline. We understand that a shoreline seawall is being considered as part of the ongoing Chatham-Kent Lake Erie Shoreline Study. Accordingly, for this report only the realignment of the drain/dyke is discussed.

The following general sequence of events would be anticipated:

- Decommission and/or reroute utilities as necessary;
- Excavate a trench some 1.2 to 1.5 metres deep along the south edge of the pavement and use well-compacted approved cohesive fill material keyed into existing soils along the south roadway edge to protect the pavement structure and embankment materials from saturation during storm/flood events;
- Provide a construction/maintenance access corridor consisting of 25 to 50-millimetre crushed stone placed over a non-woven geotextile to the south of the berm, essentially extending from the center of the eastbound lane to the edge of the right-of-way; the access corridor would be intended for infrequent periodic use during periods of favourable weather with maximum speeds of less than 10 kilometers per hour;
- Relocate the municipal drain 25 to 30 metres further north;
- Construct new embankment further north while keying into the existing drain invert and benching into the existing dyke materials; backfill should consist of properly compacted approved cohesive material, potentially with geogrid reinforcement with slopes constructed at an inclination of 3H:1V or flatter;
- Lake side slope lined with non-woven geotextile and rip rap;
- North slope covered with an erosion control blanket that promotes rapid vegetation growth or protected with rip-rap placed over a non-woven geotextile;
- Regrade and/or construct swales at selected locations between the embankment and Lake Erie to allow for drainage of flood waters back to the lake following storm events; the outlets to Lake Erie should be armoured with large rip rap to protect the shoreline and swale and prevent wave run-up during storms while allowing for drainage back to the lake post storm event;
- If an asphalt pavement structure is to be included in the design, the subgrade should be reinforced with a biaxial geogrid and a pavement fabric should be considered within the asphalt since the additional fill materials will induce settlements in the underlying organic soils that would result in differential movements and pavement cracking; the geogrid and pavement fabric will reduce cracking and maintain a waterproofing element to prevent the ingress of water into the embankment structure;
- Continue monitoring of dyke following flooding events; and
- Provide routine maintenance of berm.

The height of the berm should extend to an elevation of 1.5 metres higher than the forecast peak water elevation for Lake Erie. It would be beneficial for the design to provide the ability to further increase the height of the dyke if it appears that Lake Erie water elevations could surpass the currently projected levels.
It would be prudent that a detailed topographic survey be carried out and the results be provided to this office for review. Additional geotechnical exploration would be required to support the design of the new embankment structure.

We trust that this report provides sufficient information for your present requirements. Should any point require further clarification, or when we can be of additional assistance, please contact this office.
Signature Page

Golder Associates Ltd.

Randy Axford
Senior Geotechnical Technician

Mark A. Swallow, P. E., P.Eng.
Principal and Senior Practice Leader

RA/MAS/cr

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**Soil, Rock and Ground Water Conditions:** Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.
Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

**Sample Disposal:** Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client’s expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

**Follow-Up and Construction Services:** All details of the design were not known at the time of submission of Golder’s report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder’s report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder’s report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder’s report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder’s responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.
**Changed Conditions and Drainage:** Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.
**METHOD OF SOIL CLASSIFICATION**

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

| Organic or Inorganic | Soil Group | Type of Soil | Laboratory Tests | Field Indicators | Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML. For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between “clean” and “dirty” sand or gravel. For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left). | Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML. A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum. |
|----------------------|------------|--------------|------------------|------------------|-----------------------------|-------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|
| Organic Content ≤30% by mass | INORGANIC COARSE-GRAINED SOILS (>50% by mass of coarse fraction is larger than 0.075 mm) | Gravels with ≤12% fines (by mass) | Poorly Graded | $<4$ | $\leq 1$ or $\geq 3$ | ≤30% | GP - GRAVEL |
|                      |            | Gravels with >12% fines (by mass) | Below A Line | n/a |                      |                      | GW - GRAVEL |
|                      |            | Above A Line | n/a |                      |                      |                      | GM - SILTY GRAVEL |
|                      |            | Sands with ≤12% fines (by mass) | Poorly Graded | $<6$ | $\leq 1$ or $\geq 3$ |                      | GC - CLAYEY GRAVEL |
|                      |            | Well Graded | $\geq 6$ | $1$ to $3$ |                      |                      | SP - SAND |
|                      |            | Sands with >12% fines (by mass) | Below A Line | n/a |                      |                      | SW - SAND |
|                      |            | Above A Line | n/a |                      |                      |                      | SM - SILTY SAND |
|                      |            | Above A Line | n/a |                      |                      |                      | SC - CLAYEY SAND |

| Organic Content ≤30% by mass | INORGANIC FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm) | Silts (Non-Plastic or PL and LL plot below) | Liquid Limit | Rapid | None | None | >6 mm | N/A (can’t roll 3 mm thread) | ≤5% | ML - SILT |
|                            |            | Clays (Pl and LL plot above) | Liquid Limit | Slow to very slow | Low to medium | Dull to slight | 3 mm to 6 mm | Low | 5% to 30% | OL - ORGANIC SILT |
|                            |            | (see Note 2) | Liquid Limit | None | Medium to high | Dull to slight | 1 mm to 3 mm | Medium to high | 5% to 30% | OH - ORGANIC SILT |
|                            |            | (see Note 2) | Liquid Limit | None | Low to medium | Slight to shiny | ~3 mm | Low to medium | 0% to 30% | CL - SILTY CLAY |
|                            |            | Predominantly peat, may contain some mineral soil, fibrous or amorphous peat | Liquid Limit | None | Medium to high | Slight to shiny | 1 mm to 3 mm | Medium | 75% to 100% | PT - PEAT, SANDY PEAT |

**Note 1** — Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.

**Note 2** — For soils with <5% organic content, include the descriptor “trace organics” for soils with between 5% and 30% organic content include the prefix “organic” before the Primary name.
PARTICLE SIZES OF CONSTITUENTS

<table>
<thead>
<tr>
<th>Soil Constituent</th>
<th>Particle Size Description</th>
<th>Millimetres</th>
<th>Inches (US Std. Sieve Size)</th>
</tr>
</thead>
<tbody>
<tr>
<td>BOULDERS</td>
<td>Not Applicable</td>
<td>&gt;300</td>
<td>&gt;12</td>
</tr>
<tr>
<td>COBBLES</td>
<td>Not Applicable</td>
<td>75 to 300</td>
<td>3 to 12</td>
</tr>
<tr>
<td>GRAVEL</td>
<td>Coarse</td>
<td>19 to 75</td>
<td>0.75 to 3 (4 to 0.75)</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>4.75 to 19</td>
<td></td>
</tr>
<tr>
<td>SAND</td>
<td>Coarse</td>
<td>2.00 to 4.75</td>
<td>(10) to (4) (20) to (40)</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>0.425 to 0.075</td>
<td>(40) to (10) (200) to (40)</td>
</tr>
<tr>
<td>SILT/CLAY</td>
<td>Classified by plasticity</td>
<td>&lt;0.075</td>
<td>&lt; (200)</td>
</tr>
</tbody>
</table>

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

<table>
<thead>
<tr>
<th>Percentage by Mass</th>
<th>Modifier</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;35</td>
<td>Use 'and' to combine major constituents (i.e., SAND and GRAVEL)</td>
</tr>
<tr>
<td>&gt; 12 to 35</td>
<td>Primary soil name prefixed with &quot;gravelly, sandy, SILTY, CLAYEY&quot; as applicable</td>
</tr>
<tr>
<td>&gt; 5 to 12</td>
<td>some</td>
</tr>
<tr>
<td>≤ 5</td>
<td>trace</td>
</tr>
</tbody>
</table>

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT), N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

NON-COHESIVE (COHESIONLESS) SOILS

<table>
<thead>
<tr>
<th>Term</th>
<th>SPT 'N' (blows/0.3m)</th>
<th>Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>0 to 4</td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>4 to 10</td>
<td></td>
</tr>
<tr>
<td>Compact</td>
<td>10 to 30</td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>30 to 50</td>
<td></td>
</tr>
</tbody>
</table>

1. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>Soil flows freely through fingers.</td>
</tr>
<tr>
<td>Moist</td>
<td>Soils are darker than in the dry condition and may feel cool.</td>
</tr>
<tr>
<td>Wet</td>
<td>As moist, but with free water forming on hands when handled.</td>
</tr>
</tbody>
</table>

COHESIVE SOILS

<table>
<thead>
<tr>
<th>Term</th>
<th>Undrained Shear Strength (kPa)</th>
<th>SPT 'N' (blows/0.3m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>&lt;12</td>
<td>0 to 2</td>
</tr>
<tr>
<td>Soft</td>
<td>12 to 25</td>
<td>2 to 4</td>
</tr>
<tr>
<td>Firm</td>
<td>25 to 50</td>
<td>4 to 8</td>
</tr>
<tr>
<td>Stiff</td>
<td>50 to 100</td>
<td>8 to 15</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>100 to 200</td>
<td>15 to 30</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt;200</td>
<td>&gt;30</td>
</tr>
</tbody>
</table>

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>w &lt; PL</td>
<td>Material is estimated to be drier than the Plastic Limit.</td>
</tr>
<tr>
<td>w ~ PL</td>
<td>Material is estimated to be close to the Plastic Limit.</td>
</tr>
<tr>
<td>w &gt; PL</td>
<td>Material is estimated to be wetter than the Plastic Limit.</td>
</tr>
</tbody>
</table>

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS</td>
<td>Auger sample</td>
</tr>
<tr>
<td>BS</td>
<td>Block sample</td>
</tr>
<tr>
<td>CS</td>
<td>Chunk sample</td>
</tr>
<tr>
<td>DD</td>
<td>Diamond Drilling</td>
</tr>
<tr>
<td>DO or DP</td>
<td>Seamless open ended, driven or pushed tube sampler – note size</td>
</tr>
<tr>
<td>DS</td>
<td>Denison type sample</td>
</tr>
<tr>
<td>GS</td>
<td>Grab Sample</td>
</tr>
<tr>
<td>MC</td>
<td>Modified California Samples</td>
</tr>
<tr>
<td>MS</td>
<td>Modified Shelby (for frozen soil)</td>
</tr>
<tr>
<td>RC</td>
<td>Rock core</td>
</tr>
<tr>
<td>SC</td>
<td>Soil core</td>
</tr>
<tr>
<td>SS</td>
<td>Split spoon sampler – note size</td>
</tr>
<tr>
<td>ST</td>
<td>Slotted tube</td>
</tr>
<tr>
<td>TO</td>
<td>Thin-walled, open – note size (Shelby tube)</td>
</tr>
<tr>
<td>TP</td>
<td>Thin-walled, piston – note size (Shelby tube)</td>
</tr>
<tr>
<td>WS</td>
<td>Wash sample</td>
</tr>
</tbody>
</table>

SOIL TESTS

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>w</td>
<td>water content</td>
</tr>
<tr>
<td>PL</td>
<td>plastic limit</td>
</tr>
<tr>
<td>LL</td>
<td>liquid limit</td>
</tr>
<tr>
<td>C</td>
<td>consolidation (oedometer) test</td>
</tr>
<tr>
<td>CHEM</td>
<td>chemical analysis (refer to text)</td>
</tr>
<tr>
<td>CID</td>
<td>consolidated isotropically drained triaxial test¹</td>
</tr>
<tr>
<td>CIU</td>
<td>consolidated isotropically undrained triaxial test with porewater pressure measurement¹</td>
</tr>
<tr>
<td>D_k</td>
<td>relative density (specific gravity, G_s)</td>
</tr>
<tr>
<td>DS</td>
<td>direct shear test</td>
</tr>
<tr>
<td>GS</td>
<td>specific gravity</td>
</tr>
<tr>
<td>M</td>
<td>sieve analysis for particle size</td>
</tr>
<tr>
<td>MH</td>
<td>combined sieve and hydrometer (H) analysis</td>
</tr>
<tr>
<td>MPC</td>
<td>Modified Proctor compaction test</td>
</tr>
<tr>
<td>SPC</td>
<td>Standard Proctor compaction test</td>
</tr>
<tr>
<td>OC</td>
<td>organic content test</td>
</tr>
<tr>
<td>SO</td>
<td>concentration of water-soluble sulphates</td>
</tr>
<tr>
<td>UC</td>
<td>unconfined compression test</td>
</tr>
<tr>
<td>UU</td>
<td>unconsolidated undrained triaxial test</td>
</tr>
<tr>
<td>V (FV)</td>
<td>field vane (LV-laboratory vane test)</td>
</tr>
<tr>
<td>Y</td>
<td>unit weight</td>
</tr>
</tbody>
</table>

¹ Tests anisotropically consolidated prior to shear are shown as CAD, CAU.
**LIST OF SYMBOLS**

Unless otherwise stated, the symbols employed in the report are as follows:

<table>
<thead>
<tr>
<th>I. GENERAL</th>
<th>(a) Index Properties (continued)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \pi )</td>
<td>3.1416</td>
</tr>
<tr>
<td>( \ln x )</td>
<td>natural logarithm of ( x )</td>
</tr>
<tr>
<td>( \log_{10} x )</td>
<td>x or ( \log x ), logarithm of ( x ) to base 10</td>
</tr>
<tr>
<td>( g )</td>
<td>acceleration due to gravity</td>
</tr>
<tr>
<td>( t )</td>
<td>time</td>
</tr>
<tr>
<td>( w )</td>
<td>water content</td>
</tr>
<tr>
<td>( w_l )</td>
<td>liquid limit</td>
</tr>
<tr>
<td>( w_p )</td>
<td>plastic limit</td>
</tr>
<tr>
<td>( I_p )</td>
<td>plasticity index ( = (w_l - w_p) )</td>
</tr>
<tr>
<td>( NP )</td>
<td>non-plastic</td>
</tr>
<tr>
<td>( w_s )</td>
<td>shrinkage limit</td>
</tr>
<tr>
<td>( I_L )</td>
<td>liquidity index ( = (w - w_p) / I_p )</td>
</tr>
<tr>
<td>( I_C )</td>
<td>consistency index ( = (w_w - w) / I_p )</td>
</tr>
<tr>
<td>( e_{\text{max}} )</td>
<td>void ratio in loosest state</td>
</tr>
<tr>
<td>( e_{\text{min}} )</td>
<td>void ratio in densest state</td>
</tr>
<tr>
<td>( I_D )</td>
<td>density index ( = (e_{\text{max}} - e) / (e_{\text{max}} - e_{\text{min}}) ) (formerly relative density)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>II. STRESS AND STRAIN</th>
<th>(b) Hydraulic Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma )</td>
<td>shear strain</td>
</tr>
<tr>
<td>( \Delta )</td>
<td>change in, e.g. in stress: ( \Delta \sigma )</td>
</tr>
<tr>
<td>( \varepsilon )</td>
<td>linear strain</td>
</tr>
<tr>
<td>( \varepsilon_v )</td>
<td>volumetric strain</td>
</tr>
<tr>
<td>( \eta )</td>
<td>coefficient of viscosity</td>
</tr>
<tr>
<td>( u )</td>
<td>Poisson's ratio</td>
</tr>
<tr>
<td>( \sigma )</td>
<td>total stress</td>
</tr>
<tr>
<td>( \sigma' )</td>
<td>effective stress ( \sigma' = \sigma - u )</td>
</tr>
<tr>
<td>( \sigma'_{vo} )</td>
<td>initial effective overburden stress</td>
</tr>
<tr>
<td>( \sigma_{\text{oct}} )</td>
<td>mean stress or octahedral stress ( = (\sigma_1 + \sigma_2 + \sigma_3)/3 )</td>
</tr>
<tr>
<td>( \tau )</td>
<td>shear stress</td>
</tr>
<tr>
<td>( u )</td>
<td>porewater pressure</td>
</tr>
<tr>
<td>( E )</td>
<td>modulus of deformation</td>
</tr>
<tr>
<td>( G )</td>
<td>shear modulus of deformation</td>
</tr>
<tr>
<td>( K )</td>
<td>bulk modulus of compressibility</td>
</tr>
<tr>
<td>( h )</td>
<td>hydraulic head or potential</td>
</tr>
<tr>
<td>( q )</td>
<td>rate of flow</td>
</tr>
<tr>
<td>( v )</td>
<td>velocity of flow</td>
</tr>
<tr>
<td>( i )</td>
<td>hydraulic gradient</td>
</tr>
<tr>
<td>( k )</td>
<td>hydraulic conductivity</td>
</tr>
<tr>
<td>( j )</td>
<td>(coefficient of permeability)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>III. SOIL PROPERTIES</th>
<th>(c) Consolidation (one-dimensional)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \rho(\gamma) )</td>
<td>bulk density (bulk unit weight)*</td>
</tr>
<tr>
<td>( \rho_d(\gamma_d) )</td>
<td>dry density (dry unit weight)</td>
</tr>
<tr>
<td>( \rho_w(\gamma_w) )</td>
<td>density (unit weight) of water</td>
</tr>
<tr>
<td>( \rho_s(\gamma_s) )</td>
<td>density (unit weight) of solid particles ( (\gamma' = \gamma - \gamma_w) )</td>
</tr>
<tr>
<td>( \gamma' )</td>
<td>unit weight of submerged soil</td>
</tr>
<tr>
<td>( D_R )</td>
<td>relative density (specific gravity) of solid particles ( (D_R = \rho_s / \rho_w) )  (formerly ( G_s ))</td>
</tr>
<tr>
<td>( e )</td>
<td>void ratio</td>
</tr>
<tr>
<td>( n )</td>
<td>porosity</td>
</tr>
<tr>
<td>( S )</td>
<td>degree of saturation</td>
</tr>
<tr>
<td>( C_c )</td>
<td>compression index</td>
</tr>
<tr>
<td>( C_r )</td>
<td>reconsolidation index</td>
</tr>
<tr>
<td>( C_s )</td>
<td>swelling index</td>
</tr>
<tr>
<td>( C_a )</td>
<td>secondary compression index</td>
</tr>
<tr>
<td>( m_v )</td>
<td>coefficient of volume change</td>
</tr>
<tr>
<td>( c_v )</td>
<td>coefficient of consolidation (vertical direction)</td>
</tr>
<tr>
<td>( c_h )</td>
<td>coefficient of consolidation (horizontal direction)</td>
</tr>
<tr>
<td>( T_v )</td>
<td>time factor (vertical direction)</td>
</tr>
<tr>
<td>( U )</td>
<td>degree of consolidation</td>
</tr>
<tr>
<td>( \sigma'_{p} )</td>
<td>pre-consolidation stress</td>
</tr>
<tr>
<td>( OCR )</td>
<td>over-consolidation ratio ( = \sigma'<em>{p} / \sigma'</em>{vo} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(d) Shear Strength</th>
<th>( \tau ) = ( \sigma' + \sigma' \tan \phi' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \tau_p, \tau_r )</td>
<td>peak and residual shear strength</td>
</tr>
<tr>
<td>( \phi' )</td>
<td>effective angle of internal friction</td>
</tr>
<tr>
<td>( \delta )</td>
<td>angle of interface friction</td>
</tr>
<tr>
<td>( \mu )</td>
<td>coefficient of friction ( = \tan \delta )</td>
</tr>
<tr>
<td>( c' )</td>
<td>effective cohesion</td>
</tr>
<tr>
<td>( c_u, s_u )</td>
<td>undrained shear strength ( (\phi = 0 ) analysis)</td>
</tr>
<tr>
<td>( p )</td>
<td>mean total stress ( (\sigma_1 + \sigma_3)/2 )</td>
</tr>
<tr>
<td>( p' )</td>
<td>mean effective stress ( (\sigma_1' + \sigma_3')/2 )</td>
</tr>
<tr>
<td>( q )</td>
<td>( (\sigma_1 - \sigma_3)/2 ) or ( (\sigma_1' - \sigma_3')/2 )</td>
</tr>
<tr>
<td>( q_u )</td>
<td>compressive strength ( (\sigma_1 - \sigma_3) )</td>
</tr>
<tr>
<td>( S_t )</td>
<td>sensitivity</td>
</tr>
</tbody>
</table>

* Density symbol is \( \rho \). Unit weight symbol is \( \gamma \) where \( \gamma = \rho g \) (i.e. mass density multiplied by acceleration due to gravity)  

**Notes:**  
1. \( \tau = \sigma' + \sigma' \tan \phi' \)  
2. shear strength = (compressive strength)/2
### SOIL PROFILE

<table>
<thead>
<tr>
<th>ELEV. (M)</th>
<th>DESCRIPTION</th>
<th>PENETRATION TEST HAMMER, 63.5 kg; DROP, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>ASPHALT, gravelly sand, trace to silt; brown</td>
<td>0.15</td>
</tr>
<tr>
<td>0.04</td>
<td>FILL, silty clay, some sand, with topsoil; brown to grey; firm</td>
<td>1 SS 6</td>
</tr>
<tr>
<td>2.15</td>
<td>FILL, gravelly sand, trace silt; brown</td>
<td>2 SS 5</td>
</tr>
<tr>
<td>2.45</td>
<td>(PT) PEAT, non-fibrous; black; soft</td>
<td>3 SS 2</td>
</tr>
<tr>
<td>3.00</td>
<td>(CL) SILTY CLAY, some sand, with topsoil and rootlets; grey; very soft</td>
<td>4 SS 2</td>
</tr>
<tr>
<td>3.66</td>
<td>(CL) SILTY CLAY, some sand, trace gravel; brown turning grey; firm to very stiff</td>
<td>5 SS 5</td>
</tr>
<tr>
<td>6.55</td>
<td>END OF BOREHOLE</td>
<td>8 SS 11</td>
</tr>
</tbody>
</table>

### HYDRAULIC CONDUCTIVITY, k, cm/s

<table>
<thead>
<tr>
<th>ELEV. (M)</th>
<th>HYDRAULIC CONDUCTIVITY, k, cm/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>10^0</td>
</tr>
<tr>
<td>2.00</td>
<td>10^-5</td>
</tr>
<tr>
<td>4.00</td>
<td>10^-5</td>
</tr>
<tr>
<td>6.00</td>
<td>10^-5</td>
</tr>
</tbody>
</table>

### INSTALLATION AND GROUNDWATER OBSERVATIONS

- Water level measured at about 2.0m bgs on July 30, 2019.
- Granular bentonite
- Filter sand
- 50mm Diam. Slot
- 10 Schedule 40 PVC Screen
- Water level measured at about 2.0m bgs on July 30, 2019.
**RECORD OF BOREHOLE**  BH-202

**PROJECT:** 1792661

**LOCATION:** REFER TO LOCATION PLAN

**BORING DATE:** July 30, 2019

**DRILLING CONTRACTOR:** Henderson Drilling Inc.

**DATUM:** NOT SURVEYED

**INSTALLATION AND GROUNDWATER OBSERVATIONS**

**ELEVATION**

**DESCRIPTION**

<table>
<thead>
<tr>
<th>PENETRATION TEST HAMMER, 63.5 kg; DROP, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>PENETRATION RESISTANCE, BLOWS/0.3m</td>
</tr>
<tr>
<td>HYDRAULIC CONDUCTIVITY, k, cm/s</td>
</tr>
<tr>
<td>WATER CONTENT PERCENT</td>
</tr>
</tbody>
</table>

**SOIL PROFILE**

<table>
<thead>
<tr>
<th>Type</th>
<th>Number</th>
<th>ELEV. (m)</th>
<th>Depth Scale</th>
<th>Type</th>
<th>Number</th>
<th>ELEV. (m)</th>
<th>Depth Scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASPHALT</td>
<td>1</td>
<td>0.00</td>
<td>0.00</td>
<td>FILL</td>
<td>2</td>
<td>0.76</td>
<td>0.76</td>
</tr>
<tr>
<td>FILL</td>
<td>1</td>
<td>0.10</td>
<td>0.10</td>
<td>FILL</td>
<td>2</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>FILL</td>
<td>3</td>
<td>2.13</td>
<td>2.13</td>
<td>(PT) PEAT</td>
<td>3</td>
<td>3.03</td>
<td>3.03</td>
</tr>
<tr>
<td>(SM) SILTY SAND</td>
<td>4</td>
<td>3.25</td>
<td>3.25</td>
<td>(CL) SILTY CLAY</td>
<td>5</td>
<td>5.03</td>
<td>5.03</td>
</tr>
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</table>

**END OF BOREHOLE**

**SHEAR STRENGTH**

<table>
<thead>
<tr>
<th>Cu, kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
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</table>

**WATER CONTENT PERCENT**

<table>
<thead>
<tr>
<th>Wp</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
</tr>
</tbody>
</table>

**DEPTH SCALE**

1 : 50
## Soil Profile

<table>
<thead>
<tr>
<th>ELEV. (m)</th>
<th>DESCRIPTION</th>
<th>PENETRATION</th>
<th>HYDRAULIC CONDUCTIVITY, ( k, \text{ cm/s} )</th>
<th>PENETRATION TEST HAMMER, 63.5 kg DROP, ( \text{mm} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>ASPHALT</td>
<td>0.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.76</td>
<td>FILL, gravelly sand, trace to some silt; brown</td>
<td>1 ss 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.24</td>
<td>FILL, silty clay, some sand, with topsoil; brown to grey; soft to firm</td>
<td>3 ss 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.00</td>
<td>(PT) PEAT; dark brown to black; soft</td>
<td>4 ss 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.65</td>
<td>(SM) SILTY SAND; trace to some clay; grey to brown; very loose</td>
<td>5 ss 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.79</td>
<td>(CL) SILTY CLAY; some sand, trace gravel; grey; soft to stiff</td>
<td>7 ss 13</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Additional Observations

Water level measured in open hole at about 4.3m bgs on July 30, 2019.
LEGEND
- BOREHOLE/MONITORING WELL

Simplified Stratigraphy
- ASPHALT
- FILL
- PEAT
- SILTY CLAY

Installation Details
- WELL INSTALLATION
- STRATA PLOT
- BLOW/S 0.3m
- MEASURED WATER LEVEL

Notes
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.
REFER TO FIGURE 1 FOR CROSS-SECTION LOCATION.
ALL LOCATIONS ARE APPROXIMATE ONLY.
NOTES

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

REFER TO FIGURE 1 FOR CROSS SECTION LOCATION.

ALL LOCATIONS ARE APPROXIMATE ONLY.
LEGEND
- BOREHOLE

Simplified Stratigraphy
- ASPHALT
- FILL
- PEAT
- SILTY SAND
- SILTY CLAY

INSTALLATION DETAILS
- STRATA PLOT
- BLOWS 0.3m
- MEASURED WATER LEVEL
- FOR DETAILS REFER TO RECORD OF BOREHOLES

NOTES
- THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.
- REFER TO FIGURE 1 FOR CROSS SECTION LOCATION.
- ALL LOCATIONS ARE APPROXIMATE ONLY.
APPENDIX A

Curricula Vitae and Related Technical Services
Mark A. Swallow
Principal and Senior Practice Leader

PROFESSIONAL SUMMARY
GOLDER ASSOCIATES LTD. - LONDON

Mark A. Swallow, P.E., P.Eng. is a Senior Geotechnical Practice Leader and Principal with over 34 years of experience on a broad variety of geotechnical, geo-environmental, and materials engineering projects. He is the Project Director and senior peer reviewer for multiple geotechnical and materials engineering projects carried out by the London and Windsor offices. Mr. Swallow has been responsible for the geotechnical aspects of numerous high profile infrastructure and development projects in southwestern, Ontario and the southeastern United States. His experience includes deep foundations, raft foundations, retaining structures, slope stabilization, embankments, settlement analysis, engineered fill, light-weight fill, construction vibration monitoring and assessment, deep excavation analysis and shoring system design, subsurface risk assessments, preparation of construction specifications, design-build projects, environmental assessment and remediation, structure condition surveys, pavement design and life-cycle analyses, stormwater drainage and management systems, dams and impoundments, ground improvement, slurry walls and soil-mixed cut-off walls, trenchless technology including microtunneling, geotechnical baseline reports, construction dewatering, and geotechnical monitoring and instrumentation.

EMPLOYMENT HISTORY

Golder Associates Ltd. – London, Ontario
Principal and Senior Practice Leader (2017 to Present)
Senior Practice Leader with over 34 years of experience currently focusing on a broad variety of geotechnical engineering projects in infrastructure/transportation, land development, and manufacturing sectors. Involved in: developing protocols for geotechnical explorations and evaluations; geotechnical and civil design of foundations, slopes, and retaining structures; instrumentation and monitoring; subsurface construction risk assessment and mitigation; construction vibration monitoring and assessment; and design-build projects.

Golder Associates Inc. – Jacksonville, Florida
Principal and Senior Practice Leader (2015 to 2017)
Senior Practice Leader with over 30 years of experience focusing on geotechnical and geo-environmental services primarily for the mining, infrastructure/transportation, land development, and manufacturing sectors. Involved in developing protocols for geotechnical explorations and evaluations, geotechnical and civil design, instrumentation and monitoring, subsurface construction risk assessment and mitigation, design-build projects, construction management, and claims evaluation.
**Golder Associates Inc. – Jacksonville, Florida**  
*President and Principal (2008 to 2015)*

Responsible for strategy, all aspects of operations, and the overall performance of the U.S. operating region. Working with the U.S. Management Team and Board of Directors, Mr. Swallow develops strategy and tactics for profitable growth, client development, employee career development, risk management, and focused investments.

In addition to his role as President, Mr. Swallow provides direction and advice on a variety of geotechnical and environmental remediation projects.

**Golder Construction Services, Inc. – Jacksonville, Florida**  
*President and Principal (2005 to 2008)*

**Golder Associates Inc. – Jacksonville, Florida**  
*Office Manager/Principal (2002 to 2005)*

**Golder Associates Inc. – Jacksonville, Florida**  
*Senior Engineer to Engineering Group Leader/Associate (1993 to 2002)*

**Golder Associates Ltd. – London, Canada**  
*Geotechnical Engineer to Senior Engineer (1986 to 1993)*

**RELEVANT EXPERIENCE – DAMS, IMPOUNDMENTS AND SLOPES**

**Earthen Dyke Stability Review Program, Leamington, Ontario**
Project Director for geotechnical engineering, condition inspections and evaluation of earthen flood control dyke systems along East Beach Drain, East Marsh Drain, Marentette Marsh and Lloyd Drain, in Leamington, Ontario. Sections with marginal stability and ongoing progressive movements were identified. Toe buttressing with rip rap was recommended to enhance stability.

**West London Dyke, London, Ontario**
Project Director for geotechnical engineering services in support multiple phases of the West London Dyke Reconstruction Program for the Upper Thames Conservation Authority. The project consisted of various stages of borehole drilling, stability analyses, and design for transitional sections for Phases 1 to 7, materials testing services, construction monitoring, specifications for the dyke structure, sheet pile design and review.

**Riverfront Protection System and Dam, St. Marys, Ontario**
Geotechnical assessment, condition inspections, stability analyses and remedial grouting design for limestone flood protection works and dam at the Upper Thames River in St. Marys Ontario. Developed recommendations for stabilization of protection works while maintaining permeability to control potentially destabilizing groundwater levels. Developed remedial grouting recommendations for excessively permeable areas.
Flood Protection Dykes, Walkerton, Ontario  
Geotechnical investigation, dyke condition inspections, stability analyses and design of remedial measures for flood protection dykes along the Saugeen River in Walkerton, Ontario. Developed recommendations for regrading and erosion control measures for dyke system in an urban setting.

Brine Storage Facility, Windsor, Ontario  
Project Director for the investigation and design of a brine storage facility for a natural gas storage facility. Geotechnical investigation, stability analyses, evaluation and remedial design for ring dyke and interior dams. Detailed stability analyses indicated unstable conditions in conjunction with high operational water levels. Recommendations for regrading, replacement of berm materials with lightweight fill and construction of a toe buttress.

Brine Recirculation Facility, Windsor, Ontario  
Project Manager for the investigation and design of a brine recirculation facility for a solution salt mining operation. Geotechnical investigation, stability analyses and design for ring dyke system. Numerous boreholes indicated the presence of up to 30 feet of variable soft fill materials that had been present at the site since the late 1800s. Excavation and replacement of the fill proved to be economically unfeasible and, consequently, the facility was designed as a flexible structure capable of accommodating large differential settlements. Design and construction recommendations were provided for containment berms, erosion control systems, lining systems, and access roads.

Lime Sludge Settling Facility, Bruce, Ontario  
Project Manager for a geotechnical investigation and, subsequently, analyses and design for a lime sludge recirculation/settling facility for a nuclear power facility. Investigation included the review of borehole and test pit data, groundwater and lake water level monitoring, and borrow source evaluations. Design recommendations were provided for a low permeability earthen liner system, erosion protection systems, underdrains, side slope stability, concrete access ways for maintenance vehicles, and dewatering requirements to prevent hydrostatic basal uplift during maintenance pump down events.

Stability of Riverfront Retaining Systems, Goderich, Ontario  
Project Engineer for a geotechnical investigation to assess the stability of an existing sheet pile riverbank wharf retaining system for the proposed deepening of the adjacent river bottom to accommodate larger shipping vessels. Boreholes were drilled along the land side of the retaining system and along the water side at the toe of the sheeting using a barge mounted drill rig. Detailed geotechnical analyses were carried out to assess the stability of the existing retaining system and for the design of sheet pile toe stabilization consisting of driven H-piles and sloping rock fill.

Industrial Wastewater Treatment System Design and Permitting, Jacksonville, Florida  
Project Manager and Engineer of Record for design and permitting of a 1,000,000 gallon per day industrial wastewater treatment system for a chemical manufacturing facility. Geotechnical investigation, stability assessment, and design for ring dyke system and interior partition. Permitting included application preparation and an Anti-Degradation Study. The design included cooling/settling ponds, included precipitation through pH adjustment, UV sterilization, and filtration/contact media treatment.

Dyke Stability Assessment, Rapid Infiltration Basin, Palm Coast, Florida  
Project Manager for litigation support and Expert Witness report on the performance of a Rapid Infiltration Basin (RIB) for the subsurface disposal 1,000,000 gallons per day of treated municipal wastewater effluent. Geotechnical investigation, stability analyses, and design recommendations. Extensive hydraulic modelling was conducted to evaluate the cause of significant under-performance. A full-scale performance test was also conducted to evaluate groundwater mounding under various loading rates. Modelling was subsequently conducted to evaluate alternative
measures to improve performance. An upgradient drainage channel was subsequently recommended as the most economical method of improving performance.

**Wastewater Exfiltration Pond, Ring Dyke Design, Jacksonville, Florida**
Task Manager and Engineer of Record for the design of a wastewater exfiltration pond at the site of a former tank manufacturing facility. Geotechnical investigation, ring dyke stability analyses and design. The pond design was based on closed form analyses of groundwater mounding effects and the results of computer seepage modeling. Dyke berm stability was evaluated, and an economical configuration designed to balance cut and fill volumes.

**Phosphate Tailings Basin, Tampa, Florida**
Senior reviewer and technical support for design of earth dams for phosphate mining facility. Detailed analyses were conducted for slope stability and seepage volumes to optimize the dam configuration. Economy was maximized while satisfactory performance and safety were maintained.

**Slope Stability Assessments, Establishment of Setback Limits, Multiple Locations, Ontario**
Project Director and senior peer reviewer for dozens of slope stability evaluations throughout Southern Ontario. Geotechnical explorations, stability analyses, design of remedial measures and/or the establishment of geotechnical setback limits for land development and infrastructure projects in:

- London
- Bayfield
- Port Elgin
- Caledon
- Port Stanley
- Windsor
- Saugeen Shores
- Kingsville
- Toronto

**RELEVANT EXPERIENCE – GEOTECHNICAL ENGINEERING**

**West London Dyke, London, Ontario**
Project Director for geotechnical engineering services in support of the West London Dyke Reconstruction Program. The project consisted of various stages of borehole drilling through Phases 1 to 6, materials testing services, sample analysis, review design notes and specifications for the park walls and the sheet pile design review.

**Thames Valley Corridor, London, Ontario**
Project Director for geotechnical engineering services in support of the design of the Thames Valley Corridor within the Old Victoria Hospital Lands (OVHL) in the City of London, Ontario. The proposed corridor development included the linked pedestrian systems composed of multi-use paths, lighting, park-type structures, pedestrian bridges, roadways, open spaces and off-road paths within the proposed rehabilitation space of the OVHL area.

**Old Victoria Stormwater Management Facility, London, Ontario**
Hydrogeological assessment to support the functional design of the proposed Old Victoria Stormwater Management Facility (SWMF) No. 1, located in London, Ontario. The hydrogeological assessment was undertaken to identify potential groundwater receptors in the vicinity of the Site and to evaluate the potential hydrogeological impacts associated with the proposed construction.
**Tributary ‘C’ Study Area, RiverBend Planning Area, London, Ontario**  
Project Director for an integrated geotechnical investigation and hydrogeological assessment. The work was undertaken as a component of a Municipal Class Environmental Assessment for the City of London. The scope of work consisted of a review of available information and a site reconnaissance to provide a preliminary assessment of: anticipated subsurface soil and groundwater conditions; potential groundwater recharge and discharge areas; pre- and post-development water balance; and slope conditions and constraints on potential development.

**Wilton Grove Road (Commerce Road to City Limits) London, Ontario**  
Project Director for geotechnical engineering, archaeological and cultural heritage services for the detailed design of the Wilton Grove Road improvements. The project consisted of the widening of Wilton Grove Road from two to three lanes from Commerce Road easterly to the City limits (approximately 4.5 kilometres). New bike lanes and sidewalks are also to be constructed. This section of Wilton Grove Road crosses three water courses and a wetland. This project is still in progress.

**Winnipeg Boulevard and Churchill Avenue Reconstruction, London, Ontario**  
Project Director for the geotechnical exploration carried out for the detailed design and construction of new storm, sanitary and water services, as well as full reconstruction of the roadway along Winnipeg Boulevard from Churchill Avenue to Wavell Street and along Churchill Avenue from Winnipeg Boulevard to Edmonton Street adjacent to East Lions Park in London, Ontario.

**Nightingale Avenue, Sanitary Sewer Replacement, London, Ontario**  
Project Director for the geotechnical exploration and testing program carried out for design of the proposed sanitary sewer replacement located on Nightingale Avenue between the manhole structures designated as BB402 and BB248 between Princess Avenue and Dundas Street in London, Ontario. The proposed work consisted of sanitary sewer replacement and full roadway reconstruction.

**Dundas Street Infrastructure Upgrades, London, Ontario**  
Project Director for the geotechnical exploration and testing program carried out for design of the proposed infrastructure upgrades located on Dundas Street from Adelaide Street to Ontario Street in London, Ontario. The proposed work consists of storm and sanitary sewer replacements, a watermain installation from Elizabeth Street to the east project limit and full roadway reconstruction.

**Ponds Mills Road at Highway 401, London, Ontario**  
Project Director for the preliminary geotechnical assessment carried out for the design of the proposed watermain replacement and associated pavement restoration along Pond Mills Road extending from approximately 80 metres north to approximately 80 metres south of the Highway 401 right-of-way. This project consisted of the replacement of the existing watermain along the project area and the reconstruction of the affected portions of roadway.

**McIntosh Drain Reconstruction Southwold Township, Ontario, Canada**  
Senior direction and peer reviewer for a geotechnical investigation carried out for the design of the proposed McIntosh Drain No.2 undercrossing of Highway 401, located approximately 1.8 kilometres west of the Union Road (Elgin Road 20) in Southwold Township, Ontario. The proposed crossing replacement consisted of 63 metres of 610-millimetre diameter pipe, installed using trenchless construction techniques at a depth of about 2.5 metres below existing pavement elevation. Recommendations were provided for entrance and exit pit excavations and temporary support, trenchless crossing techniques, groundwater control, instrumentation, monitoring and inspections.
Bridge Replacement, Embankment Reconstruction, and Railroad Undercrossing, St. Clair Township, Ontario, Canada
Senior direction and peer reviewer for geotechnical exploration and testing program for the replacement of Kerr Line Bridge No. 45, embankment slope reconstruction on Baby Road, and the watermain undercrossing of the CSX Railway tracks on Pointe Line in St. Clair Township, Ontario. Recommendations were provided for foundations, backfill placement and compaction, excavations and groundwater control, embankment reconstruction and stabilization, erosion and scour protection, trenchless crossing techniques, monitoring and inspections.

Water Pressure Booster Station, Township of Warwick, Ontario, Canada
Senior direction and peer reviewer for geotechnical exploration and testing program for a water pressure booster station in the Township of Warwick, Ontario. Recommendations were provided for foundations, lateral earth pressures, uplift, excavations, and pavement structure design.

Underwater Bridge Condition Surveys, Ontario, Canada
Developed the approach and scope for a series of underwater bridge inspections conducted by MTO for multiple structures in southwestern and central Ontario. Developed methodology, forms, safety program, and investigation and documentation programs for the evaluation of the condition of submerged structural elements and surrounding sediments and soils. Evaluated soundness of concrete and steel elements, as well as erosion,

Sewage Treatment and Disposal Facilities, Tobermory, Ontario, Canada
Project Manager for a comprehensive geotechnical and hydrogeological investigation for sewage treatment and on-site subsurface effluent disposal facilities with a design capacity of 150,000 gallons per day. The investigation involved an extensive review of existing geological data, the drilling of 13 boreholes and the excavation of 15 test pits to evaluate subsurface soil and groundwater conditions. In situ and laboratory permeability testing was conducted to evaluate the suitability of native soils for the exfiltration of treated effluent and for use in a low permeability lining system. Extensive, sophisticated hydraulic analyses were conducted for the design of a system of exfiltration ponds. Comprehensive geotechnical recommendations were provided for large-scale earth works, containment dikes, lining systems, revetment systems, structure foundations, access road pavements, and site drainage.

Brine Recirculation Facility, Windsor, Ontario, Canada
Project Manager for the investigation and design of a brine recirculation facility for a solution salt mining operation. Numerous boreholes indicated the presence of up to 30 feet of variable soft fill materials that had been present at the site since the late 1800s. Excavation and replacement of the fill proved to be economically unfeasible and, consequently, the facility was designed as a flexible structure capable of accommodating large differential settlements. Design and construction recommendations were provided for containment berms, erosion control systems, lining systems, and access roads.

Lime Sludge Settling Facility, Bruce, Ontario, Canada
Project Manager for a geotechnical investigation and, subsequently, analyses and design for a lime sludge recirculation/settling facility for a nuclear power facility. Investigation included the review of borehole and test pit data, groundwater and lake water level monitoring, and borrow source evaluations. Design recommendations were provided for a low permeability earthen liner system, erosion protection systems, underdrains, side slope stability, concrete access ways for maintenance vehicles, and dewatering requirements to prevent hydrostatic basal uplift during maintenance pump-down events.
Water Treatment Plant and Water Tower, Port Elgin, Ontario, Canada
Project Engineer for geotechnical investigation and design for a municipal water treatment plant and an associated standpipe water tower. Geotechnical recommendations were provided for construction dewatering, excavation support, and a partially compensated raft foundation. An engineered structural fill was constructed for support of the water tower foundation due to the presence of a surficial stratum of loose sand. Settlement predictions were provided for the new facilities as well as an adjacent existing treatment plant. Monitoring of the existing plant during construction indicated settlements consistent with the predictions. Geotechnical construction supervision and inspection services were provided throughout construction.

Stability of Riverfront Retaining Systems, Goderich, Ontario, Canada
Project Engineer for a geotechnical investigation to assess the stability of an existing sheetpile riverbank wharf retaining system for the proposed deepening of the adjacent river bottom to accommodate larger shipping vessels. Boreholes were drilled along the land side of the retaining system and along the water side at the toe of the sheeting using a barge mounted drill rig. Detailed geotechnical analyses were carried out to assess the stability of the existing retaining system and for the design of sheetpile toe stabilization consisting of driven H-piles and sloping rock fill.

Wonderwood Expressway, Jacksonville, Florida, USA
Project Manager and technical lead for design evaluation for access roads, retaining structures, and approach areas to be built over soft saltwater marsh deposits associated with the construction of a high-volume bridge spanning the Intracoastal Waterway. A comprehensive review of geotechnical data was conducted. The results indicated that extensive ground improvement involving geosynthetics, wick drains, and preloading was necessary.

Trednick Parkway, Jacksonville, Florida, USA
Project Manager and technical lead for a geotechnical investigation for Trednick Parkway, a major arterial collector to be constructed over 30 feet of very loose sands associated with historical hydraulic titanium mining operations. Evaluated alternatives for ground improvement, subgrade preparation, and stabilization. Conducted design analyses and provided recommendations for a flexible pavement structure incorporating geosynthetic reinforcement.

Puckett Creek Bridge, Jacksonville, Florida, USA
Provided senior peer review for the design of an embankment over organic salt marsh soils at a creek crossing located on the alignment of the Wonderwood Expressway. Existing data from extensive subsurface investigations were reviewed and analyses were conducted to evaluate preconsolidation and ground improvement options, wick spacing, surcharge loads, pre-loading sequencing and duration, and pavement design.

Power Plant, Brevard County, Florida, USA
Task Manager for a geotechnical investigation for a proposed power plant. The investigation included conventional soil borings and standard penetration testing. Recommendations were provided for deep foundations for structures and power generating facilities, reinforced earth foundations for large aboveground storage tanks, construction drainage and dewatering, roadway construction, and site preparation.

Development Feasibility Study, Jacksonville, Florida, USA
Project Manager for a site evaluation to assess the feasibility of development of a 5,000-acre parcel of land located in the western part of Duval County, Florida. The geotechnical component of the evaluation involved reviewing existing geological and geotechnical data together with the results of 108 boreholes drilled at the site. Based on the results of the review and various analyses, the site was subdivided into a number of areas based on
geotechnical development potential and conceptual design recommendations were provided for foundations, flexible and rigid roadway pavements, underground utilities, and drainage works.

**Sheeptile Bulkheads, Jacksonville, Florida, USA**

Project Manager for geotechnical evaluation and preliminary design for a sheeptile bulkhead system to be constructed along the north bank of the St. Johns River in the eastern portion of downtown Jacksonville. The bulkhead system was to be installed to support the proposed reclamation of submerged lands within the St. Johns River. Analyses were carried out based on preliminary subsurface data using both fixed earth and free earth methods. Embedment depths, sheeting sections, and anchor geometries were recommended.

**Condition Survey of Riverfront Facilities, Jacksonville, Florida, USA**

Project Manager for a condition survey of the piers and retaining systems present along a section of the north bank of the St. Johns River located immediately east of the downtown core of the City of Jacksonville, Florida. The survey consisted of water depth soundings and an evaluation of the nature and condition of concrete and timber piers, pre-cast concrete pile foundations, concrete retaining systems, steel sheeptile retaining systems and cellular sheeptile retaining systems.
Randy L. Axford  
Senior Geotechnical Technician

PROFESSIONAL SUMMARY
GOLDER ASSOCIATES LTD. - LONDON

Randy Axford is a Senior Geotechnical Technician with over 30 years of experience in Golder’s London, Ontario office. Mr. Axford’s responsibilities include liaising with Project Managers and clients, assessing project requirements and ensuring that suitably trained and qualified staff are assigned, field health and safety and carrying out quality control inspections. Mr. Axford is also available to assist on complex projects and to provide support to address difficult and/or challenging conditions. Mr. Axford has significant experience carrying out geotechnical investigations for pavements, site servicing, foundations and heavy civil projects as well as environmental investigations for contaminated sites. Mr. Axford has also provided field inspections and testing during construction of these types of projects.

EMPLOYMENT HISTORY

Golder Associates Ltd. – London, ON  
Construction Services Team Leader (2013 to 2017)

Randy Axford is a Senior Technician and the Construction Services Team Leader in Golder’s London, Ontario office. Mr. Axford’s responsibilities include the day to day management of the Construction Services Team, liaising with Project Managers and clients, assessing project requirements and ensuring that suitably trained and qualified staff are assigned, field health and safety and carrying out quality control inspections. Mr. Axford is also available to assist on complex projects and to provide support to address difficult and/or challenging conditions. Mr. Axford has significant experience carrying out geotechnical investigations for pavements, site servicing, foundations and heavy civil projects as well as environmental investigations for contaminated sites. Mr. Axford has also provided field inspections and testing during construction of these types of projects.

Golder Associates Ltd. – London, Ontario  
Senior Geotechnical Technician (2010 to 2012)

Golder Associates Innovative Applications (GAIA) Inc. – London/Mississauga, Ontario  
Site Supervisor / Project Manager (2008 to 2009)

Golder Associates Ltd. – London, Ontario  
Senior Environmental Technician (2006 to 2008)

Golder Associates Innovative Applications (GAIA) Inc. – London/Mississauga, Ontario  
Senior Environmental Technician (2003 to 2006)
**GRIDERS ASSOCIATES LTD. – LONDON, ONTARIO**  
Senior Geotechnical/Environmental Technician, (1994 to 2003)  
Geotechnical Technician (1989 to 1994)

**PROJECT EXPERIENCE – GEOTECHNICAL ENGINEERING**

**West London Dyke, London, Ontario**  
Project Manager for geotechnical engineering services in support of the West London Dyke Reconstruction Program. The project consisted of various stages of borehole drilling through Phases 1 to 6, materials testing services, sample analysis, review design notes and specifications for the park walls and the sheet pile design review.

**QA-QC VMP 2015-3002, Phase I and II, London, Ontario**  
Project Manager for the Quality Verification Engineering and materials testing services carried out during the replacement of the Veterans Memorial Parkway underpass at Highway 401 and the corresponding extension of Phase I, Veterans Memorial Parkway to Wilton Grove Road in London, Ontario. Phase II was carried out from Gore to Oxford Street.

**TVDSB 2018 Projects, Southwestern, Ontario**  
Project Manager for the geotechnical investigations carried out for the design of the pavement rehabilitation and/or reconstruction at various Thames Valley District School Board (TVDSB) schools in London, Strathroy, Woodstock and Sparta.

**Tecumseh Public School, Chatham, Ontario**  
Project Manager for the geotechnical exploration and testing program carried out for the purposed reconstruction of the existing pavement structure, new access routes, asphalt playgrounds/basketball courts and related drainage system at Tecumseh Public School in Chatham, Ontario.

**RiverBend Phase 9, London, Ontario**  
Project Manager for the geotechnical inspections and testing carried out for the RiverBend Phase 9 subdivision in London, Ontario. The project also included mix design review.

**Dorchester Pumping Station, Thames Centre, Ontario**  
Project Manager for the geotechnical exploration and testing carried out for the design of the proposed Dorchester DA3 pumping station, forcemain, valve chamber and associated buildings to be constructed in Dorchester, Ontario.

**Gurney Pit Subdivision, Paris, Ontario**  
Project Manager for the geotechnical exploration and testing program carried out for the design of the proposed commercial employment and residential development of a 57 hectare property located northeast of the interchange of Highway 403 and Rest Acres Road in Brant County, Ontario. The project also included fill delineation, slope review and a preliminary hydrogeological assessment.

**2017 Medway Secondary School, Arva, Ontario**  
Project Manager for the geotechnical explorations and testing carried out for the reconstruction of the west parking area and existing running track, construction of long jump pits, relocation of the hammer/discuss throw area and new storm sewers at Medway Secondary School in Arva, Ontario. This project also included geotechnical inspections and testing during construction.
626 – 666 Indian Road, Sarnia, Ontario
Project Manager for the geotechnical exploration carried out for the design of the pavement rehabilitation and/or reconstruction at 646 and 666 Indian Road, Sarnia, Ontario.

West 5 Medical Building, London, Ontario
Project Manager for the geotechnical inspections and materials testing for the Medical Building located in West 5, 1305 RiverBend Road, London, Ontario.

Contract 5, Chippendale Crescent, London, Ontario
Project Manager for the geotechnical and construction inspection and materials testing services for Contract 5, 2017 Infrastructure Renewal Program on Chippendale Crescent, London, Ontario. The project comprised of the installation of sanitary and storm sewers, watermain replacement, and construction of curb and gutter, sidewalk and pavement reconstruction.

Veterans Memorial Parkway Resurfacing (Hwy 401 to Gore Rd), London, Ontario
Geotechnical inspections and materials testing for Veterans Memorial Parkway between Highway 401 and Gore Road. Services included:

- Review of asphalt mix designs;
- Geotechnical inspections of the existing granular base prior to placing the new asphalt as well as testing of any new granular materials required;
- In situ density testing and sampling of the new asphalt;
- Sampling and testing of plastic concrete;
- Laboratory testing;
- Reporting of the above; and
- Geotechnical engineering support during construction, if and as required.

Thames Valley District School Board, London and Area, ON 2017
Projects - geotechnical services carried out for the design and construction phases of the 2017 Thames Valley District School Board (TVDSB) parking lot and roadway upgrades for 9 schools in London and area. Services includes replacement of sidewalk entrances, reconstruction and addition of parking lots and storm sewers and catchbasins associated with parking lots.

Thames Valley District School Board, London and Area, ON 2016
Projects - geotechnical services for the design and construction phases for the 2016 Thames Valley District School Board (TVDSB) parking lot and roadway upgrades for 4 schools in London and area.

Chrisvale Boulevard, Phases 2 and 3, Sarnia, ON
A geotechnical investigation carried out for the proposed site servicing and road construction for Chrisvale Boulevard, Phase 2 and 3, in Sarnia, Ontario. Materials testing services were then carried out during construction of the boulevard streets.

Audi Terminal Retaining Wall, London, ON
Geotechnical inspections carried out at the New Audi Terminal at 481 Wharncliffe Road South in London, Ontario to confirm that suitable founding soils were present beneath the facing of the previously constructed retaining wall located within the southeast corner of the site.

2013 Arterial Roads, London, ON
This project consisted of the 2013 Arterial Road Rehabilitation Program for the City of London involving a
complete geotechnical inspection and testing program on various arterial roads within the City. These inspections ensured that the requirements to achieve proper performing pavement and the design and material specifications were consistently achieved. Services included full reconstruction of parking areas and access roads, storm sewers associated with new parking areas and additions to parking areas.

**Fox Hollow No. 3 SWM Facility and Outlet Channel, London, Ontario**
Project Manager for the Consulting Services for the Detailed Design and Construction Administration for the Fox Hollow No. 3 Stormwater Management Facility and Outlet Channel. Services included peer review, public dig, materials testing and hydrogeological services.

**Contract 2 - Bond Street, London, Ontario**
Materials testing for Contract 2 – Bond Street, Raywood Avenue, Alexandra Street and Lincoln Place, London, Ontario.

**Contract 1 – Waterloo Street, South Street and Hill Street, London, Ontario**
Materials testing for Contract 1 – Waterloo Street, South Street and Hill Street.

**Contract 5 – Landor Street, London, Ontario**
Materials testing for Contract 5 Landor Street from Wethered Street to Highbury Avenue North.

**Veterans Memorial Parkway (Highway 401 to Oxford Street East), London, Ontario**
Materials Testing carried out for the pavement reconstruction on Veterans Memorial Parkway (VMP) from Highway 401 to Oxford Street East.

**2013 Arterial Roads, London, ON**
This project consisted of the 2013 Arterial Road Rehabilitation Program for the City of London involving a complete geotechnical inspection and testing program on various arterial roads within the City. These inspections ensured that the requirements to achieve proper performing pavement and the design and material specifications were consistently achieved. Services included full reconstruction of parking areas and access roads, storm sewers associated with new parking areas and additions to parking areas.

**Innovation Park, London, ON**
Conducted generally full-time geotechnical inspections and testing in conjunction with the construction of the engineered fill and site grading. The purpose of the inspections and testing was to confirm that competent soils were exposed in the base of the fill areas prior to placing the engineered fill materials and that the fill materials were compacted to the specified degree.

**W12A Landfill, London, ON**
Materials testing services were carried out for Cell 7 Landfill Base and Leachate Collection System construction and a new generator pad at the W12A Landfill Site, Project Sw6021.

**Greenway Pollution Control Plant, London, ON**
Materials testing services were carried out for the Greenway Pollution Control Plant. The purpose of the inspections was to confirm the suitability of the soils exposed in the base of the excavation for the support of the wet well.

**Oxford Street Pollution Control Plant, London, ON**
This project consisted of concrete testing of the floor slab of the Chemical Storage Building at the Oxford Street Pollution Control Plants.
Sparta Line, Central Elgin, ON
A site reconnaissance and geotechnical slope stability assessment was carried out for the culvert crossing on Sparta Line.

Bruce Nuclear Power Plant, Tiverton, ON
Geotechnical inspections and materials testing services were carried out for the new fire training facility at the Bruce Power Nuclear Plant.

Village of Inwood, Township of Brook-Alvinston, ON
Materials testing services were carried out on various roads within the Village of Inwood.

Brigden Road, St. Clair, ON
Asphalt testing was carried on Brigden Road between Holt Line and Kerr Line.

Various Roads, Belmont, ON
Materials testing services were carried out on various roads within Belmont, Ontario.

Symes Street, Glencoe, ON
Materials testing services were carried out on Symes Street during the installation of storm sewers and watermains.

Yorke Line, Malahide Township, ON
A geotechnical investigation was carried out on Yorke Line from Putnam Road to Wittaker Road. The purpose of this investigation was to assess the existing pavement component thickness and subgrade conditions at the site and to provide geotechnical engineering recommendations for the design of the pavement reconstruction/rehabilitation.

Bergin Subdivision, Sarnia, ON
A geotechnical investigation and materials testing were carried out for the Chrisvale Boulevard Development, Phase I. The purpose of the investigation was to assess the existing pavement component thickness and subgrade conditions at the site and to provide geotechnical engineering recommendations for the design of the pavement reconstruction/rehabilitation.
Dam and Reservoir Services

Golder is one of the world’s leading companies in design and construction management of dams and reservoirs for water supplies, tailings management, flood control, and hydroelectric power, as well as environmental enhancement, recreation, and fish breeding.

Our experience includes dam expansions and rehabilitation to upgrade dams deemed deficient because of changing seismic or hydrologic criteria. Golder provides performance assessments of existing dams and designs solutions tailored to clients' needs for hydraulic structures, water supply dams and mine tailings storage facilities. We have successfully designed non-standard, customized hydraulic structures using model studies that saved substantial money over more traditional approaches. Our expertise extends to design of concrete and roller-compacted concrete (RCC) dams as well as managing the construction of all types of dams and their ancillary structures.

We understand that our clients' needs extend beyond technical issues to environmental and cultural issues. With our expertise in biological assessments, endangered species evaluations, and social assessments we execute sustainable projects that can be successfully implemented.
DAM AND RESERVOIR SERVICES

Planning and Pre-Design
- Feasibility studies
- Hydrology and hydraulics
- Storage, flood control, irrigation, and hydroelectric evaluations
- Geophysical surveys – crosshole seismic and electrical resistivity
- Geologic and geotechnical investigations
- Environmental, conceptual, and feasibility planning

Permitting
- Environmental impact assessments and statements
- Biological evaluations and endangered species assessments
- Cultural/social resource evaluations and sustainability assessments
- Hazard evaluation, risk assessment, emergency action planning, and potential failure mode analysis (PFMA)
- Public involvement and stakeholder meetings

Design
- Embankment and RCC dam design
- Spillway and outlet works design and evaluation
- Foundation improvement
- Seismic design
- Diversion tunnels, outlet tunnels, and pressure tunnels
- Cost estimating and preparing bid documents
- Pipeline systems

Construction
- Dam and pipeline construction management
- Quality control and assurance
- Design build
- Turnkey construction

Existing Structures – Rehabilitation & Repair
- Dam O&M, inspection, instrumentation, and monitoring
- Stability assessments
- Dam raising, upgrades, and rehabilitation
- Seepage control
- Seismic upgrades
- Emergency action planning