July 2010

CELEBRATING 50 YEARS in 2010

REPORT ON

Slope Stability Assessment Proposed Development Site 101 Wurtemburg Street Ottawa, Ontario

Submitted to:
Claridge Homes Corporation
2001-210 Gladstone Avenue
Ottawa, Ontario
K2P 0Y6

REPORT

Report Number: 10-1121-0003

Distribution:

4 copies - Claridge Homes Corporation

2 copies - Golder Associates Ltd.

A world of capabilities delivered locally

Golder Associates

Table of Contents

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by Claridge Homes Corporation (Claridge) to carry out a slope stability assessment for a proposed development site at 101 Wurtemburg Street in Ottawa, Ontario.

The purpose of this assessment was to evaluate the stability of the existing slope and to provide slope stabilization guidelines for developing the site.

The reader is referred to the "Important Information and Limitations of This Report" which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

The site is located on the east side of Wurtemburg Street, immediately across from Clarence Street, and backs onto the Rideau River (see Key Plan, Figure 1).

The front part of the property is currently occupied by a house. The rear part of the property contains the rear-yard area of the house as well as a significant slope down to the Rideau River. Buildings exist to the north and south of the property.

The site is proposed for development with a high-rise residential building. The building will be 13 storeys high and will have two basement levels. The building footprint will occupy most of the site, and will extend onto the existing slope. It is currently planned to provide a 'walk out' condition from the lower basement level (similar to the building to the north). The slope crest would be lowered by about 6 metres to accommodate that grading, and a narrow terrace area (about 4 metres wide) would be provided between the building face and the new slope crest.

Golder Associates previously carried out a geotechnical investigation on the property in 1989. That investigation included one borehole which was advanced to a depth of about 28 metres (and an adjacent shallow second borehole to collect a Shelby tube sample). The results of that investigation, along with geotechnical guidelines on the development that was proposed at that time, were provided in a report to Claridge Homes Corporation titled "Subsurface Investigation, Proposed Apartment Building, 101 Wurtemburg Street, Ottawa, Ontario," dated May 1989 (report number 891-2060).

Geotechnical investigations were also carried out on adjacent properties, to the north and south of the site, by McRostie and Associates (McRostie) in the 1960's and 1970's. The results of those previous investigations are available in our files from the following reports:

1) Report to Kelton Architect and Adjeleian & Associates by McRostie, Seto, Genest & Associates Ltd. titled "Design Subsurface Investigation for Proposed Diplomatic Premises – U.S.S.R., Wurtemburg Street, Ottawa, Ontario" dated September 17, 1973 (Report No. SF-1625A).

2) Report to Adjeleian & Associates by McRostie & Associates Ltd. titled "Foundation Investigation, East Wurtemburg Street Opposite Heney Street No.2" dated May 2, 1963 (Report No. SF-664).

The approximate locations of the relevant boreholes from these previous subsurface investigations are shown on Figure 2.

The results of the previous investigations indicate that the subsurface conditions on this site consist of a thick deposit of sensitive marine clay, underlain by glacial till. A layer of sandy soil was encountered between the clay and the glacial till. Published geologic mapping indicates that the underlying bedrock consists of limestone of the Lindsay formation, however one of the previous boreholes advanced by McRostie and Associates encountered shale bedrock.

The previous geotechnical assessment of this property also indicated that the slope is potentially unstable.

3.0 SITE RECONNAISSANCE

A reconnaissance of the site was carried out on January 8, 2010 to view the site condition and to measure the slope geometry.

The building to the north (apparently an embassy building) is three storeys high. The grade behind that building, adjacent to the river bank slope, appears to have been excavated to create a ‘walk out’ condition for the basement level. The ground level is therefore about 3 metres lower than in the rear yard of the 101 Wurtemburg site, and the river bank slope is accordingly shorter. The building to the south of the site is an approximately 12 storey high residential building, and is located within about 5 metres of the slope crest. The ground level behind that building is just slightly lower than the current rear-yard level of the property at 101 Wurtemburg Street.

The rear yard of the 101 Wurtemburg site is essentially unvegetated and the ground level is about 1 metre lower than the front part of the property. The slope itself is quite densely vegetated but with relatively juvenile tree cover.

The ground surface at various points along the slope was surveyed (both for horizontal and vertical positions) using a Trimble R8 Global Positioning System (GPS) survey instrument. The slope inclination was also determined using a hand clinometer.

The slope down to the Rideau River is approximately 12 metres high and inclined at just slightly flatter than 1H:1V (horizontal:vertical). The river was frozen (i.e., ice covered) at the time of the reconnaissance, however it appears that the slope toe forms the river bank; i.e., there was no apparent flood plain separating the slope toe from the river bank. The state of erosion at the slope toe could not be assessed (due to the snow cover), however the steepness of the slope toe indicates that there is likely active erosion.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the borehole put down for the previous Golder investigation and in the relevant boreholes put down for the previous McRostie investigations are shown on the borehole records in Appendix A.

In general, the subsurface conditions on this site consist of (in sequence):

- Up to about 3 metres of fill material (but likely less than 1 metre in the table land area closest to the slope).
- About 7 to 10 metres of silty clay, of which the upper portion has been weathered to a stiff crust. About the bottom 6 metres are unweathered and have a firm to stiff consistency.
- About 2 to 5 metres of compact to dense sand (fine sand and silty sand).
- Glacial till extending to about 28 metres depth (but likely thickening from south to north).
- Shale bedrock.

The groundwater level in the sand layer has been measured at about river level, such that the sand layer appears to be under-draining the overlying clay layer which forms most of the slope.

The following sections present a more detailed overview of the subsurface conditions on this site. For this discussion, emphasis is placed on the previous borehole 1 (and the accompanying borehole 1A) previously advanced on the site by Golder (report no. 891-2060). However, reference is also made to the results of the previous McRostie boreholes on the adjacent sites (i.e., borehole 2 of report no. SF1625 and borehole 4 of report no. SF664), particularly regarding the subsurface conditions at depth.

4.2 Fill Material

Borehole 1 appears to have been advanced through the driveway of the existing house and encountered about 3.1 metres of fill material consisting of the pavement structure overlying a mixture of sand as well as sand and gravel. Standard penetration tests carried out within the fill gave ‘N’ values ranging from 3 to 6 blows per 0.3 metres of penetration, indicating a very loose to loose state of packing.

It is inferred that there is less fill in the rear yard of the site (versus the front driveway), based on the ground levels.

About 1 metre of fill was also encountered at ground surface in the previous McRostie borehole 2 to the north of the site.

4.3 Sensitive Silty Clay

The surficial fill materials are underlain by a thick deposit of sensitive silty clay, which extends to about 11 metres depth (about elevation 56 metres).

The upper 2.0 metres of the silty clay at borehole 1 have been weathered to form a grey brown crust. Standard penetration tests carried out within the weathered crust gave ‘N’ values ranging from ‘weight of hammer’ to 2 blows per 0.3 metres of penetration. The results of in situ vane testing in the weathered crust gave undrained shear strengths of 57 and 65 kPa. The results of this in situ testing indicate a stiff consistency. Atterberg limit testing performed on one sample of the weathered crust gave a liquid limit of 57 percent and a plasticity index of 31 percent, reflecting intermediate plasticity. The measured water contents of two samples of the weathered crust were approximately 47 and 52 percent.

The silty clay below the depth of weathering is grey in color. The unweathered grey silty clay deposit is about 6.3 metres thick at borehole 1 (i.e., extending down to elevation 56.1 metres). The results of in situ vane testing in the grey silty clay gave undrained shear strength values ranging from about 34 to greater than 95 kilopascals (increasing with depth). The results of this in situ testing indicate the unweathered portions of the deposit to have a firm to very stiff consistency.

The results of Atterberg limit testing carried out on two samples of the grey silty clay gave liquid limits of 34 and 39 percent and plasticity index values of 14 and 18, reflecting low plasticity.

The measured water contents of the grey silty clay ranged from about 35 to 57 percent, which are at or in excess of the measured liquid limit.

Oedometer consolidation testing was carried out on one sample of the grey silty clay from borehole 1A. The results of that testing are summarized below.

Borehole/ Sample No.	**Sample Depth/Elev. (m)**	**Unit Wt. (kN/m^3)**	**σ_P' (kPa)**	**σ_{VO}' (kPa)**	**Cc**	**Cr**	**e_o**	**OCR**
1A / 2	6.3 / 61.1	18.9	350	87	0.4	0.01	0.91	4.0

Notes:

σ_P' - Apparent preconsolidation pressure

Cc - Compression index

e_o - Initial void ratio

σ_{VO}' - Computed existing vertical effective stress

Cr - Recompression index

OCR - Overconsolidation ratio

A similar silty clay deposit was encountered in the nearby McRostie boreholes, and ranged from 7.6 to 9.4 metres in thickness. The clay extended down to elevations 56.7 and 56.5 metres in these boreholes, which is quite consistent with the bottom elevation of 56.1 metres in borehole 1.

4.4 Sand

A layer of silty fine sand was encountered beneath the silty clay in borehole 1, and has a thickness of about 2.0 metres (i.e., extending down to about elevation 54.1 metres). The result of one standard penetration test yielded an ‘N’ value of about 24 blows per 0.3 metres of penetration, indicating a compact state of packing.

Similar sand layers were encountered below the silty clay in the McRostie boreholes, however the deposits were thicker. The sand layers in boreholes 2 and 4 were about 5.0 and 4.7 metres thick, respectively. The materials encountered in these boreholes were also somewhat more variable in gradation, and included silty layers as well as bouldery intervals.

4.5 Glacial Till

The sand layer is underlain by glacial till. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand and sandy silt, with a trace of clay. The glacial till was proven to a depth of 27.5 metres (i.e., elevation 39.9 metres) in borehole 1 before refusal to augering was encountered. Standard penetration test ‘N’ values in the glacial till ranged from about 5 to 15 blows per 0.3 metres of penetration, indicating a loose to compact state of packing. The measured water contents of two samples of the glacial till were approximately 8 percent.

The glacial till at McRostie boreholes 2 and 4 was proven/penetrated to depths of 20.7 and 24.9 metres, respectively (i.e., elevations of 44.7 and 43.6 metres, respectively). The till appears to have been quite bouldery in the deeper portions of those boreholes.

4.6 Refusal and Bedrock

Refusal to augering was encountered in borehole 1 at 27.5 metres depth (i.e., elevation 39.9 metres). Refusal may reflect the presence of cobbles and boulders in the glacial till deposit or could indicate the bedrock surface.

McRostie borehole 4 (south of the site) was advanced into the underlying bedrock, below about 20.7 metres depth (elevation 44.7 metres) using rotary diamond drilling (i.e., rock coring) techniques. Shale bedrock was encountered and was cored to about 23.9 metres depth.

4.7 Groundwater

The groundwater level in a standpipe sealed into the silty clay in borehole 1A was measured at elevation 61.4 metres on March 7, 1989 (i.e., at about 6 metres depth). This water level was just slightly above the bottom of the standpipe.

The groundwater level was also previously measured in McRostie borehole 2 on May 23 and on June 22, 1973 at elevations of 54.8 and 53.5 metres, respectively. These groundwater levels are located in the sand layer and correspond to about river level.

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year.

5.0 DISCUSSION

5.1 General

This section of the report provides an assessment of the stability of the existing and proposed slope geometries. Guidelines and recommendations on remedial works are also provided, based on our interpretation of the available information and project requirements.

The reader is referred to the "Important Information and Limitations of This Report" which follows the text but forms an integral part of this document.

5.2 Slope Stability Assessment

The slope down to the Rideau River at 101 Wurtemburg Street is approximately 12 metres high and is inclined at just slightly flatter than 1H:1V (horizontal:vertical).

Limit equilibrium slope stability analyses were carried out to assess the stability of the existing slope.

In general, slope failures occur when the forces (or rotational moments) generated by the weight of the soil in a slope, and external loads, exceed the shear strength of the soil. The five main parameters involved in the engineering analysis of the stability of a slope are:

1) The geometry of the slope;
2) The geology of the slope (i.e., the composition of the various soil layers within the slope and their depth, thickness, and orientation);
3) The groundwater conditions (the groundwater levels and the hydraulic gradient/flow conditions);
4) The strength parameters for the soils; and,
5) The unit weights (i.e., densities) of the soils within the slope.

The slope geometry used in the analyses was based on the GPS survey and inclinometer measurements. The geometry is relatively consistent across the length of the slope, with the slope being about 12 metres high and inclined at about 40 to 45 degrees from horizontal.

Bathymetry data for the near-shore area along this section of the Rideau River was also provided by the Rideau Valley Conservation Authority (RVCA). That data shows that the river bed slopes fairly gently down from the bank (much less steeply that the above-water portion of the slope).

The slope geology used in the analyses was based on the stratigraphy encountered at Golder borehole 1 and at McRostie boreholes 2 and 4. There is some variation between the three boreholes and therefore some interpretation of the conditions was required to develop a model stratigraphy that was appropriate to the slope area and represented a conservative evaluation. The resulting stratigraphy used in the analysis consisted of (in sequence from top to bottom, below the table land level of elevation 66.5 metres):

- About 0.5 metres of fill;
- About 3.6 metres of stiff weathered silty clay crust;

- 6 metres of firm to stiff silty clay;
- About 4.5 metres of compact to dense sand (fine sand and silty sand); and,
- Glacial till, extending to about 28 metres depth.

Due to the natural trend for greater drainage and weathering near the slope face, the interface between the weathered and unweathered silty clay was considered to dip down towards the toe (i.e., the weathered crust thickens in the immediate area of the slope face).

Based on this stratigraphy and the slope geometry, it is considered that the toe of slope (i.e., river bank) is within the sand deposit and the river bed is on the surface of the glacial till.

The soil parameters used in the analyses were based on experience with similar soils in eastern Ontario as well as published correlations with the results of the in situ and laboratory testing from the previous investigations. The soil parameters used in the analyses are:

Material	Drained Parameters		Undrained Shear Strength (kPa)	Unit Weight (kN/m^3)
	Effective Angle of Internal Friction (degrees)	Effective Cohesion (kPa)		
Fill	28	0	Note 1	20
Weathered Silty Clay Crust	35	5	75	17.5
Grey Silty Clay	34.7	7.7	45	16.5
Sand	32	0	Note 1	19.0
Glacial Till	Impenetrable			

Note: 1 - Same parameters apply as for drained loading conditions.

Based on the soil stratigraphy and measured groundwater levels, it was considered in the model that the clay deposit was fully saturated but that the groundwater flow pattern in the silty clay would involve predominantly downward vertical flow, due to 'under-drainage' of the clay by the sand layer (in contrast to the more typical condition of the flow being horizontal or parallel to the slope, as typically experienced in homogeneous clay slopes, which leads to reduced stability).

The water level in the underlying sand layer was assumed to be about 1 metre above the river level.

The stability of the slope was evaluated using limit equilibrium methods and the SLOPE/W software. The Morgenstern-Price method was used to compute the factor of safety. The factor of safety is defined as the ratio of the magnitude of the forces/moments tending to resist failure to the magnitude of the forces/moments tending to cause failure. Theoretically, a slope with a factor of safety of less than 1.0 will fail and one with a factor of safety of 1.0 or greater will stand. However, because the modeling is not exact and natural variations exist for all of the parameters affecting slope stability, a factor of safety of 1.5 is used to define a stable slope (for static loading conditions), and/or to define the 'safe' set-back distance from an unstable slope.

For seismic loading conditions, a factor of safety of 1.1 is typically used.

5.2.1 Assessment of Current Slope Geometry

The stability of the existing slope was evaluated for:

- Drained (i.e., long-term, static) conditions, for which effective stress soil parameters were used; and,
- Seismic conditions (i.e., the dynamic loading conditions during an earthquake), for which undrained shear strength parameters were used.

For static loading conditions, the results of the stability analyses indicate that the existing slope generally has a factor of safety against instability of about 1.0 and is therefore unstable. It is considered that the slope has maintained its current steep geometry as a product of the advantageous effects of the under-drainage from the sand layer and the reinforcing effects of the vegetation.

The analysis results are shown graphically on Figure 3, where the white arc shows the 'critical' slip surface (i.e., the slip surface with lowest factor of safety) and the red shading shows the composite limit of all analyzed slip surfaces with factors of safety less than 1.5.

These results indicate that the slope itself and about 15 metres of the rear-yard area could be at-risk of being affected by a slope movement.

Additional analyses were also carried out to check the stability of the slope for the condition of the river being at its 100 year flood level of 56.3 metres (provided by the RVCA). However the factor of safety is actually slightly higher for that condition.

For seismic (earthquake) loading, the potential for instability was evaluated using a simple "pseudo-static" model where a horizontal force is applied to the failure mass. This horizontal force is proportional to the weight of the failure mass and is determined using a "seismic coefficient", which is typically taken as half the design peak horizontal ground acceleration for Ottawa as specified in the National Building Code of Canada, of 0.42. A seismic coefficient of 0.21 was therefore used.

These analyses indicated a factor of safety of about 0.8, which is less than desired value of 1.1. However the potential failure surfaces with factors of safety less than 1.1 are all confined to the lower portion of the slope face and would not jeopardize the table land area.

'Static' loading is therefore considered to be the critical condition, in terms of defining the Limit of Hazard Lands for this site.

5.2.2 Limit of Hazard Lands – Current Slope Geometry

Hazard Lands associated with unstable slopes, as defined by Ministry of Natural Resources (MNR) guidelines and provincial planning policies, are unsuitable for development with either publicly owned infrastructure or private development. In accordance with the MNR guidelines, the setback distance from the crest of an unstable slope to the Limit of Hazard Lands should include three components, as appropriate, namely:

1) A "Stable Slope Allowance", which is determined as the limit beyond which there is an acceptable factor of safety (i.e., greater than about 1.5 for static loading or 1.1 for seismic loading) against the table land being impacted by a slope failure.

2) An “Erosion Allowance”, to account for future movement of the slope toe, in the table land direction, as a result of erosion along the slope toe/river bank. The magnitude of the Erosion Allowance depends upon the type of soil being eroded at the slope toe, the severity of the erosion, and the water course characteristics.

3) An “Access Allowance” of 6 metres, to allow a corridor by which equipment could travel to access and repair a future slope failure. This Access Allowance is included in the determination of the Limit of Hazard Lands wherever the development could restrict future slope access.

The resulting Limit of Hazard Lands for this site is shown on the Site Plan, Figure 2.

The ‘Stable Slope Allowance’ for a factor of safety of 1.5 at this site, as defined by MNR and City of Ottawa guidelines, is considered to extend about 15 metres from the slope crest (as shown on Figure 2).

Additional set-back distance (to define the Limit of Hazard Lands) could also be required to allow for an ‘Erosion Allowance’. An Erosion Allowance needs to be applied wherever there is active erosion, or the potential for active erosion based on the flow velocities. The Rideau Valley Conservation Authority provided flow velocities for floods with return periods ranging between 5 and 100 years. Those flow velocities range from 1.2 to 1.3 metres per second. These velocities are considered to be at about the threshold value above which erosion could be expected. It is therefore considered that the magnitude of the *Erosion Allowance* for this site, based on the MNR guidelines, would be 8 metres. The corresponding Limit of Hazard Lands is also shown on Figure 2.

5.2.3 Assessment of Proposed Slope Geometry

Further stability analyses were carried out for the proposed development condition. The geometry of the proposed slope used in the analyses is based on the understanding that the current slope crest would be lowered by about 6 metres to accommodate a ‘walk out’ condition from the lower basement level, and a narrow terrace area (about 4 metres wide) would be provided between the building face and the new slope crest. The slope would therefore be reduced to about half of its current height. A terrace elevation of 60.5 metres was used for the analyses (versus the current table land level of about elevation 66.5 metres).

The building loads were not considered in this assessment since it is assumed that the building will need to be supported on deep foundations (and therefore the soils which form the slope will not also support the weight of the building).

The stability analysis results for the proposed slope geometry are shown on Figure 4. These results indicate that the proposed slope would have a factor of safety against instability of about 1.1 and is therefore unstable. The Stable Slope Allowance for this case would be about 10 metres (for a factor of safety of 1.5), as measured from the slope crest. The potential slope failures could therefore extend beneath the building area (although the building itself would be supported on piles).

The following options could be considered to stabilize the slope:

- Regrading the slope (by filling at the toe and/or cutting at the crest);
- Excavating and re-constructing the slope with an MSE system;
- Reinforcing the slope, such as with soil anchors; or,
- Constructing an engineered retaining wall system.

The option of regrading (i.e., flattening) the slope by cutting it back from the current slope toe (river bank) to reach the back face of the building (eliminating the terrace area) has been evaluated. The resulting slope would be inclined at slightly steeper than 2H:1V, and would have a factor of safety of only 1.3 (which is less than 1.5 and is therefore still too low).

It is assumed that filling of the slope toe to further flatten the slope is not feasible, due to damage to aquatic habitat and regulations regarding filling of the flood plain.

Therefore, regardless of whether or not the slope is flattened, the factor of safety against instability of the slope would be too low. Although the potential instability would not directly jeopardize the integrity of the building (because it would be supported on piles), the potential slope failures could expose the foundations, possibly undermine the basement floor slab, and destroy any terrace areas.

Therefore, based on the current understanding of the risk/hazard, the objectives of the project, and the costs involved, the option of excavating and re-constructing the slope with a geogrid reinforced Mechanically Stabilized Earth (MSE) system is the preferred method to achieve an adequate factor of safety (greater than 1.5 for static loading and greater than 1.1 for seismic loading) for the proposed development condition.

A suitable MSE system would likely be the TerraSlope 45 system, which involves rebuilding the slope with embedded geogrid, wrapped in lifts around the slope face. The slope could then be reconstructed to the current inclination (about 1H:1V), but would have an adequate factor of safety. The slope could also be re-vegetated so that it ultimately regains its natural appearance.

Reinforced MSE slope systems are generally proprietary, with the design of the system being undertaken by the supplier. The design and supply of the TerraSlope 45 system is licensed in Ontario by Terrafix of Toronto, Ontario. The TerraSlope 45 system would typically include 0.5 metre thick wrapped soil sections along the slope with intermediate primary reinforcing geogrid layers. The primary geogrids typically extend back from the face a distance equal to 80% to 100% of the slope height. The face would also be wrapped in an erosion blanket, the objective of which is to retain the reinforced soil. The outer-most layer of retained soil (within about 0.15 metres of the slope face) would consist of topsoil, to promote vegetation growth, while the remainder of the soil would typically consist of compacted granular backfill (such as OPSS Granular B Type I or II). Several options exist for vegetating this slope, ranging from hydroseeding, to live staking, or to the planting of shrubbery and more mature vegetation. The landscape architect and Terrafix would need to coordinate their designs.

The third option, of reinforcing the slope with drilled anchors, might also be technically feasible, but is unlikely to be cost effective in comparison to an MSE system, particularly considering the difficult access conditions for a drill rig.

The fourth option, of replacing the slope with a retaining wall, would serve the same purpose, but would have a much less natural appearance and would be less likely to be accepted by the RVCA.

The recommended option is therefore to re-construct the slope with an MSE system.

5.2.4 Additional Guidelines on MSE System

The construction of an MSE system, if properly designed, will result in a stable slope geometry (i.e., factor of safety greater than 1.5 for static loading and 1.1 for seismic loading). The arrangement will therefore be an improvement over the current unstable slope condition.

Although the construction of an MSE system is conceptually feasible for this site, the detailed design of the slope will require the input of the supplier/designer and will need to address and consider the following issues:

- As a preliminary guideline, it is recommended that the MSE system start at about 0.5 metres above the normal operating river level (or above the likely flood level that might be experienced over the construction period). Deeper excavations in the sandy soils that exist at the slope toe level could experience significant groundwater inflow. A cofferdam would be required along the river bank and an active dewatering system could be required.
- Given that the lower (and submerged) portions of the slope (which are a relatively small portion of the overall slope height) cannot be reinforced, further assessment of the global stability of the slope will need to be carried out before the length of the geogrid sheets can be confirmed by the supplier i.e., the design of the MSE system will require interaction between the supplier/designer, who designs for internal stability of the slope, and the geotechnical consultant, who checks for global stability.
- The length of the reinforcing geogrid sheets will need to be defined so that the size of the excavation can be determined. It is possible that shoring could be required between this site and the adjacent properties, to avoid undermining the foundations of the neighbouring structures.
- Since the reinforcing geogrid sheets will extend beneath the proposed building, the design will need to address the constructability of the building foundations, which will likely consist of steel piles driven to bedrock. It will need to be confirmed that the piles can be driven through the geogrid sheets without damaging the overall MSE system.
- The design of the system will need to consider interaction with the building foundations under seismic loading conditions. Based on the Ontario Building Code (OBC) seismic design procedures, the ground conditions on this site (with a likely Site Class value of D), and discussions with the structural engineer, it is understood that a base shear value of about 4,000 kilonewtons may need to be supported by the foundations. Unless the foundations are designed to transfer those lateral forces directly to the underlying bedrock at depth (such as by battered piles or inclined rock anchors, or by supporting the building on large diameter caissons which don't require lateral soil support) then the slope will need to resist some or all of that base shear. More detailed assessment of the interaction between the slope, the foundations, and the structure will need to be carried out, in conjunction with the design of the MSE slope.
- Some level of erosion protection will likely need to be provided at the slope toe. The most cost-effective erosion protection system would likely be rip-rap, which would consist of rock fragments placed over the river bank. Either rounded natural cobbles and boulders or quarried rock could be used. The latter option is typically less expensive, but less natural in appearance. The size of the rock fragments to be used, the thickness of rip-rap to be placed, and the slope of the face would depend in part upon the type of material being used and the flow velocities. However, as a preliminary guideline, it is expected that rock fragments ranging up to about 500 millimetres in size would be suitable, and with the front slope placed at an angle of

2 horizontal to 1 vertical. The rip-rap should be underlain by a non-woven geotextile having a Filtration Opening Size not exceeding 75 microns, and that is physically strong and ductile enough to survive the rip-rap placement. The rip-rap should extend up to the 100 year flood level, of elevation 56.3 metres. The rip-rap would also need to extend somewhat out along the river bed, to avoid the rip-rap being undermined by scour.

- Should the option of supporting the building on a raft foundation, rather than piled foundations, need to be considered, then the impact on the MSE system design will need to be evaluated; the guidelines in this report have been based on the expectation that the building will be supported on piled foundations.

6.0 ADDITIONAL CONSIDERATIONS

Because the grade on this site will be lowered to about elevation 60.5 metres, retaining walls may be required along the boundaries with the adjacent properties. The details of those walls will need to be determined at the design stage.

This report has been prepared for the exclusive use of Claridge and their agents, for specific application to the slope stability assessment for the proposed development at 101 Wurtemburg Street in Ottawa, Ontario. The findings and guidelines presented in this report were prepared in accordance with generally accepted geotechnical engineering practice at the time of this study. It is stressed that the information in this portion of the report is intended for this project only.

It is expected that the design and construction of the slope re-construction and erosion protection works will require evaluation of the impacts to aquatic habitat and the obtaining of permits for that work from the RVCA.

GOLDER ASSOCIATES LTD.

Kim Lesage, EIT
Geotechnical Group

Mike Cunningham, P.Eng.
Associate

LICENSED PROFESSIONAL ENGINEER
M. I. CUNNINGHAM
PROVINCE OF ONTARIO

KSL/MIC/cg

n:\active\2010\1121 - geotechnical\10-1121-0003 wurtemburg st\10-1121-0003 rpt-001.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, Claridge Homes Corporation. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater 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.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

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.

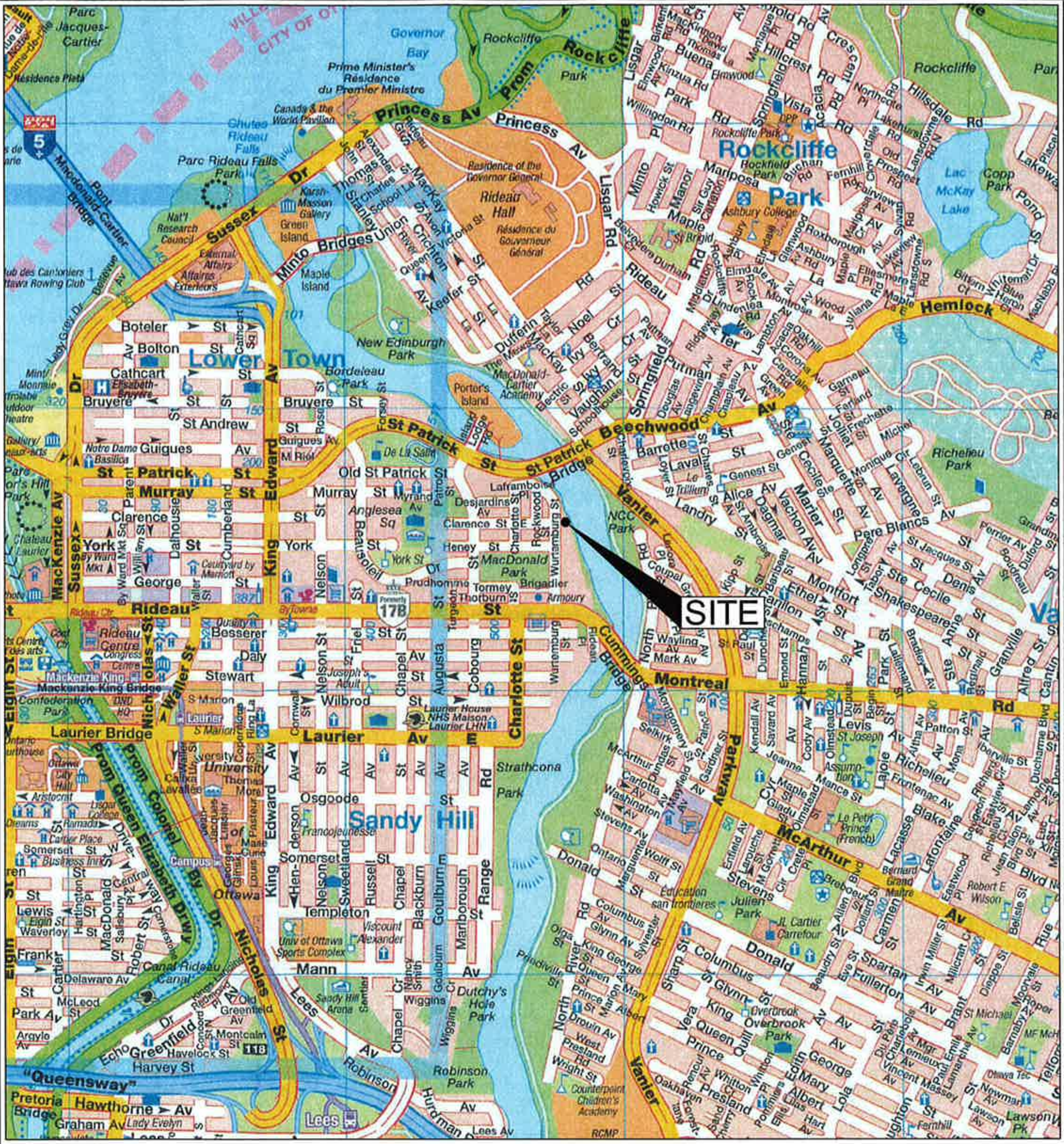
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater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

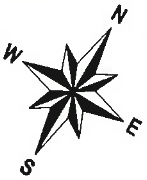
Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

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.



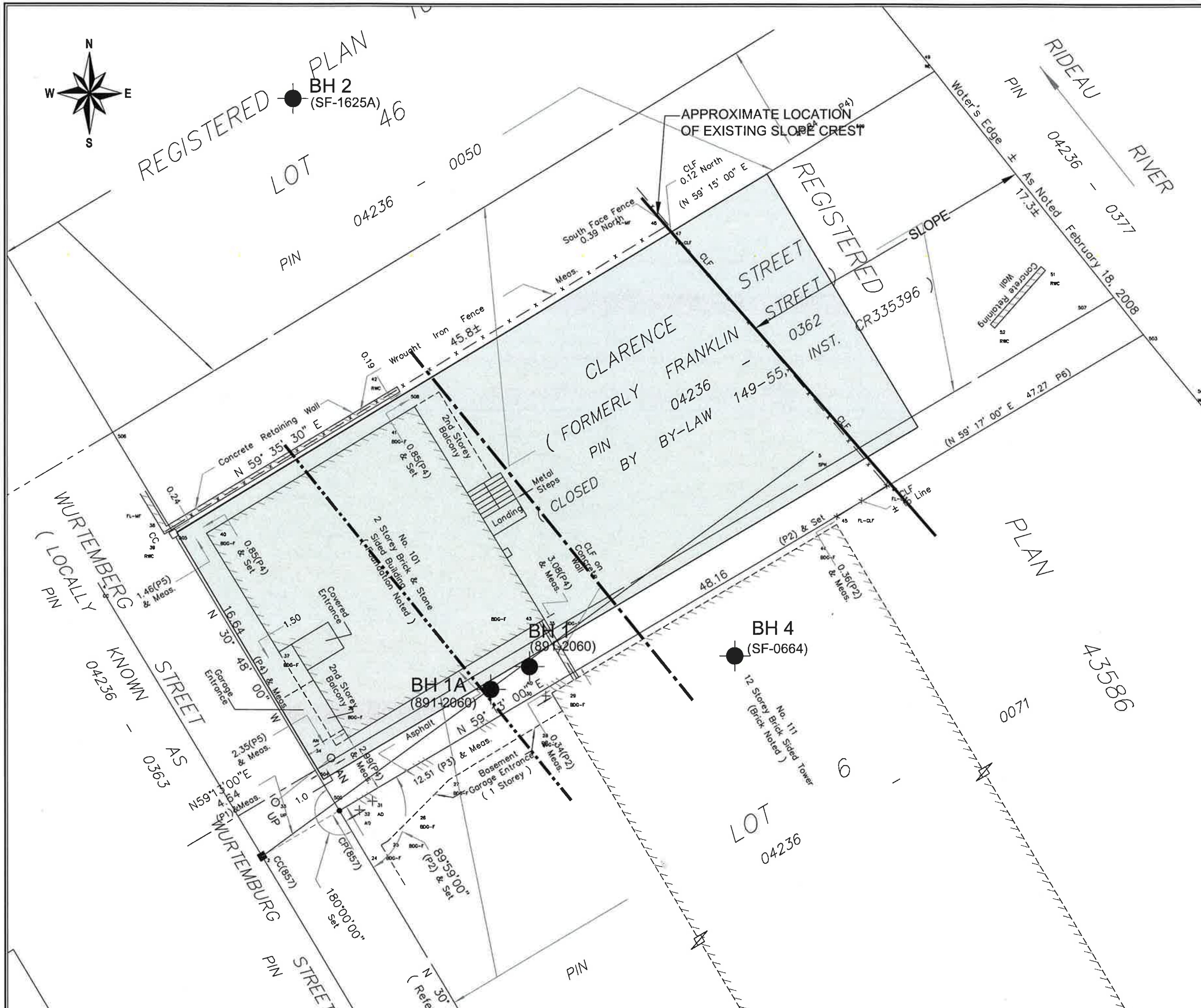


NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 10-1121-0003

Golder Associates Ottawa, Ontario, Canada	SCALE 1:20,000	TITLE	
	DATE 15 JAN. 10	**KEY PLAN**	
	DESIGN		
	CAD J.M.		
FILE No. 1011210003-01.dwg	CHECK JUL. 27/10	SLOPE STABILITY ASSESSMENT WURTEMBURG STREET - OTTAWA, ONTARIO	FIGURE 1
PROJECT No. 10-1121-0003 REV.	REVIEW		

PLOT DATE: July 26, 2010
FILENAME: N:\Active\2010\1121 - Geotechnical\10-1121-0003 Wurtemburg St\ACAD\1011210003-01.dwg



LEGEND

- APPROXIMATE BOREHOLE LOCATION IN PLAN, PREVIOUS INVESTIGATIONS
- APPROXIMATE PROPOSED BUILDING LOCATION
- LIMIT OF HAZARD LANDS INCLUDING STABLE SLOPE ALLOWANCE AND EROSION ALLOWANCE FOR EXISTING SLOPE
- LIMIT OF POTENTIAL SLOPE FAILURE SURFACE HAVING 1.5 FACTOR OF SAFETY FOR EXISTING SLOPE (i.e., STABLE SLOPE ALLOWANCE)

REFERENCE

BASE PLAN SUPPLIED IN ELECTRONIC FORMAT BY CLARIDGE HOMES CORPORATION

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 10-1121-0003

4 0 4
SCALE 1:200 METRES

REV	DATE	DES	REVISION DESCRIPTION	CAD	CHK	RVW

PROJECT
SLOPE STABILITY ASSESSMENT
WURTEMBERG STREET
OTTAWA, ONTARIO

TITLE
SITE PLAN

Golder Associates
Ottawa, Ontario, Canada

PROJECT No.	10-1121-0003		FILE No.	1011210003-02.dwg
DESIGN			SCALE 1:200	REV.
CAD	J.M.	14 JAN. 10	DRAWING No.	
CHECK	FL	JUL 27/10		2
REVIEW	MLC	JUL 27/10		

PLOT DATE: July 26, 2010
FILENAME: N:\Active\2010\1121 - Geotechnical\10-1121-0003 Wurtemburg St\ACAD\1011210003-02.dwg

Drawing file: Static Stability Analysis Output Figures.xlsx 26/07/2010 - 4:56 PM

File Name: 10-1121-0003 Sht 3-1 Drained Analysis_Pz line.gsz
Created By: Cormier, Jacqueline
Date: 27/01/2010
Time: 4:08:31 PM
Method: Morgenstern-Price
PWP Conditions Source: Piezometric Line
Direction of movement: Left to Right

Drained Analysis

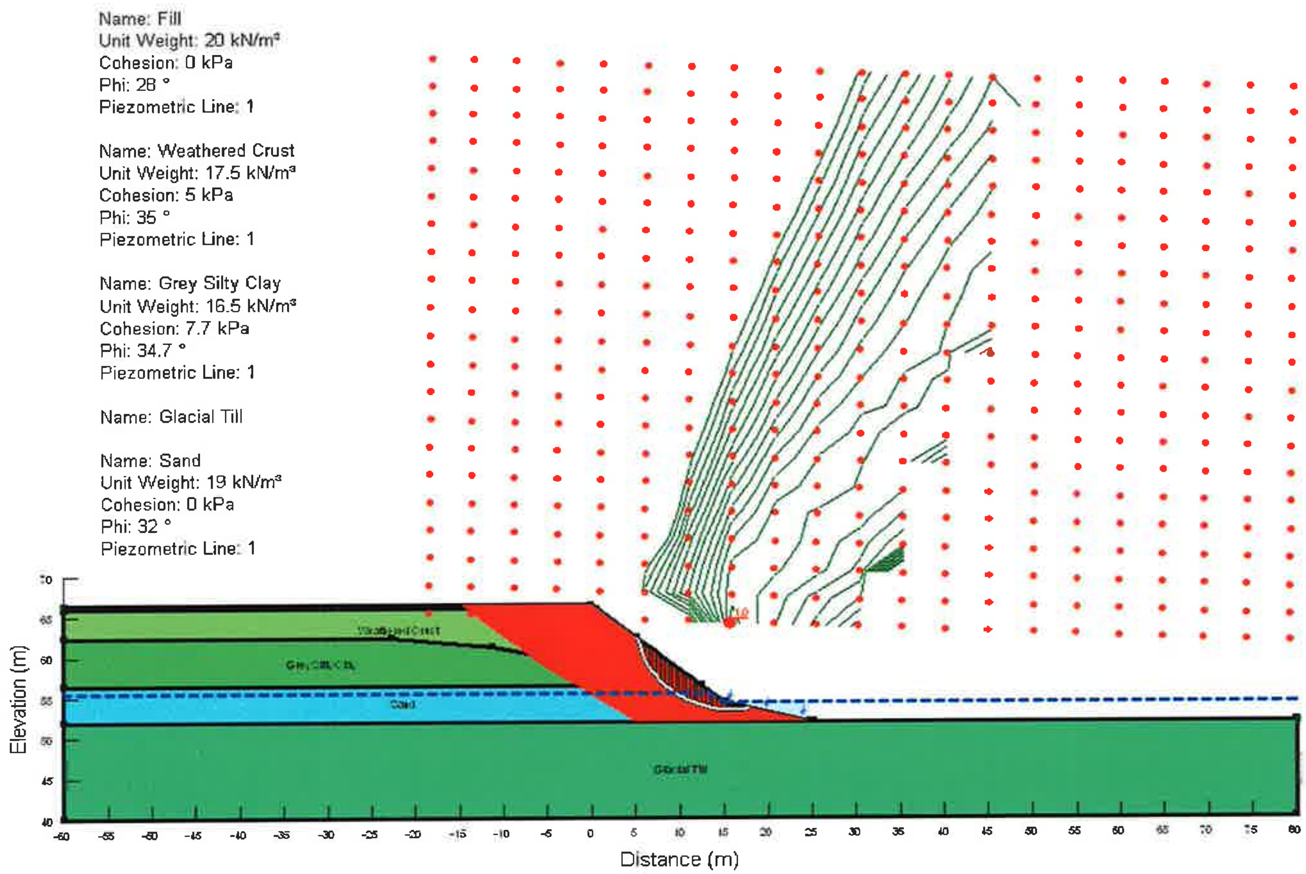


Figure 3	REVIEW: WMC Jul 27/10 CHECK: JR Jul 27/10 CADD DESIGN: KSL 06/20/10 REV. 0 SCALE AS SHOWN FILE No. PROJECT No. 10-1121-0003	TITLE **SLOPE STABILITY ANALYSIS** **CURRENT SLOPE GEOMETRY**

PROJECT

SLOPE STABILTY ASSESSMENT
101 WURTEMBURG STREET
OTTAWA, ONTARIO

Golder Associates

Drawing file: Static Stability Analysis Output Figures.xlsx 26/07/2010 - 4:56 PM

File Name: 10-1121-0003 Sht 3-4 Drained Analysis_building.gsz
Created By: Cormier, Jacqueline
Date: 27/01/2010
Time: 4:49:32 PM
Method: Morgenstern-Price
PWP Conditions Source: Piezometric Line
Direction of movement: Left to Right

Drained Analysis

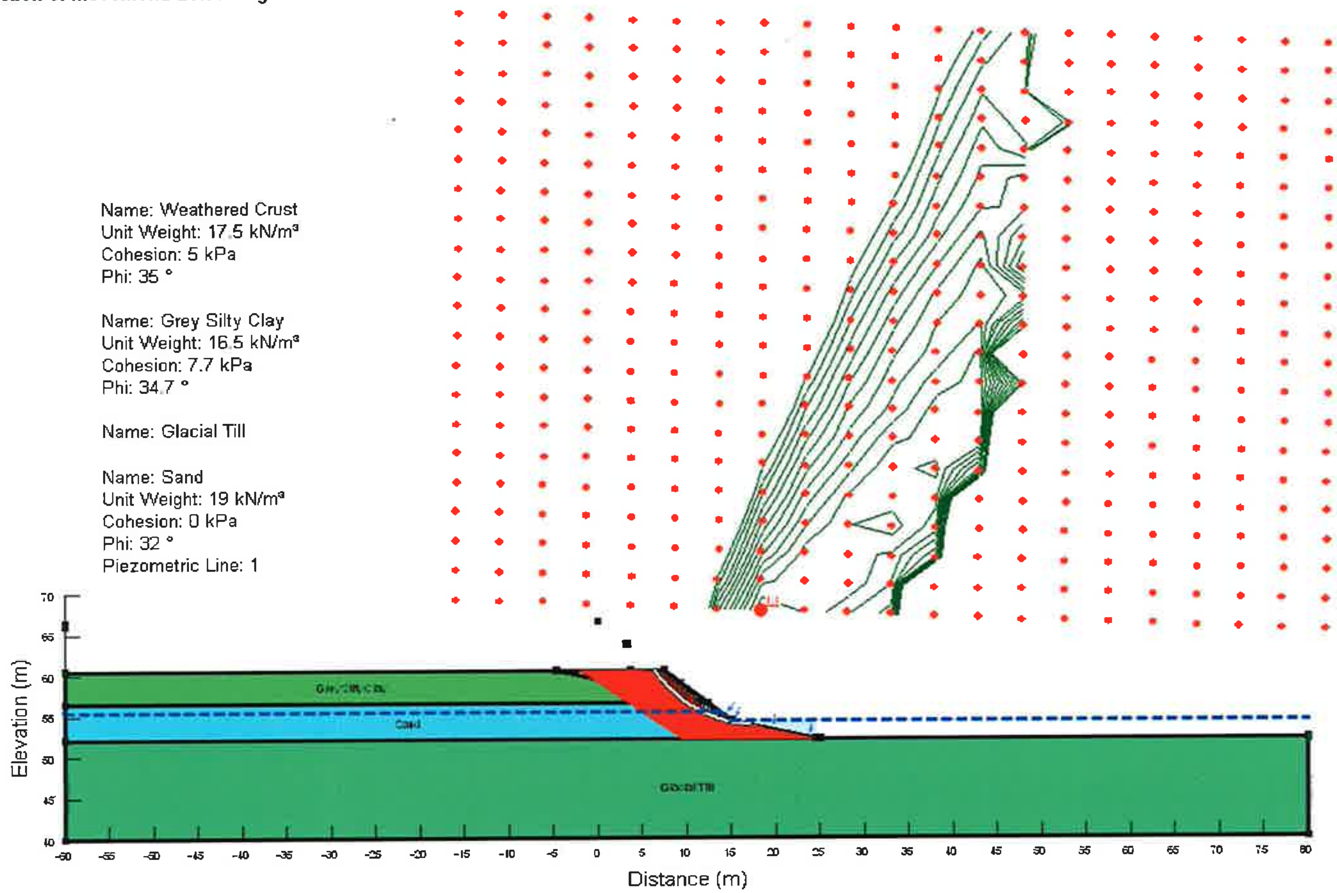


Figure 4	
REVIEW	MLC Jul 27/10
CHECK	KL Jul 27/10
CADD	
DESIGN	KSL 06/20/10
REV. 0	SCALE AS SHOWN
FILE No.	
PROJECT No.	10-1121-0003

TITLE

SLOPE STABILITY ANALYSIS
PROPOSED SLOPE GEOMETRY

PROJECT

SLOPE STABILTY ASSESSMENT
101 WURTEMBURG STREET
OTTAWA, ONTARIO

Golder Associates

APPENDIX A

List of Abbreviations and Symbols
Record of Borehole Sheets
Previous Investigations by Golder Associates and McRostie & Associates

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual Tube sample

II. 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.) drive open Sampler for a distance of 300 mm (12 in.)
DD- Diamond Drilling

Dynamic Penetration Resistance; 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, 60^0 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

Peizo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	**C_u or S_u** Kpa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_1	liquid limit
C	consolidaiton (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	modified Proctor compaction test
SPC	standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

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

I. GENERAL

π	= 3.1416
ln x, natural logarithm of x	
$\log_{10}$ x or log x, logarithm of x to base 10	
g	Acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma''$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1\sigma_2\sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma'=\gamma-\gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R= p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_1	liquid limit
w_p	plastic limit
I_p	plasticity Index=(w_1-w_p)
w_s	shrinkage limit
I_L	liquidity index=(w-w_p)/I_p
I_c	consistency index=(w_1-w)/I_p
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index-(e_{max}-e)/(e_{max}-e_{min}) (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction=tan δ
c'	effective cohesion
c_u,s_u	undrained shear strength (ϕ=0 analysis)
p	mean total stress $(\sigma_1+\sigma_3)/2$
p'	mean effective stress $(\sigma'_1+\sigma'_3)/2$
q	$(\sigma_1-\sigma_3)/2$ or $(\sigma'_1-\sigma_3)/2$
q_u	compressive strength $(\sigma_1-\sigma_3)$
S_t	sensitivity

Notes: 1. τ=c'σ' tan |'
2. Shear strength=(Compressive strength)/2

RECORD OF BOREHOLE 1

SHEET 1 of 1

LOCATION See Figure 2

BORING DATE

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

BORING METHOD: Power Auger, 200mm Diam (Hollow Stem)

DESCRIPTION	ELEV. / DEPTH (M)	SAMPLE NUMBER	TYPE	BLOWS/0.3M
Ground Surface	67.41			
Grey crushed stone (FILL)	0.06			
Loose to very loose brown fine to medium sand to sand and gravel (FILL)	0.46	1	50 DO	4
		2	50 DO	6
	64.33	3	50 DO	3
Very stiff to stiff grey brown SILTY CLAY (Weathered Crust)	3.08	4	50 DO	2
	62.38	5	50 DO	WH
Very stiff to firm grey SILTY CLAY	5.03	6	50 DO	WH
		7	50 DO	WH
		8	50 DO	WH
	56.10	9	50 DO	1
Compact brown to grey SILTY fine SAND	11.31	10	50 DO	24
	54.12			
Compact to loose dark grey silty sand to sandy silt, some clay, gravel and cobbles, occasional boulder and fine sand layer (GLACIAL TILL)	13.29	11	50 DO	14
		12	50 DO	7
		13	50 DO	5
	48.51	14	50 DO	15
Probably compact dark grey Glacial Till	18.90			
	42.42			
Probably dense dark grey Glacial Till	24.99			
	39.89			
End of Hole Refusal to Auger	27.52			

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — nat.V.- + Q.- ● rem.V.- ⊕ U.- O (20, 40, 60, 80)

0.00-0.06 ASPHALT

HYDRAULIC CONDUCTIVITY, k, CM/SEC

WATER CONTENT, PERCENT — Wp, W, Wl (20, 40, 60, 80)

ADDITIONAL LAB. TESTING: /MH (at approx. 14.3 m)

PIEZOMETER OR STANDPIPE INSTALLATION

15 — 0 — 5 — 10 PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE 1 : 150

Golder Associates

LOGGED S.Leighton

CHECKED

RECORD OF BOREHOLE 1A

SHEET 1 of 1

LOCATION See Figure 2

BORING DATE Feb. 28, 1989

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa (nat.V.- + Q.- ● rem.V.- ⊕ U.- O) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, CM/SEC / WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface									Seal Backfill
2	Power Auger 200mm Diam (Hollow Stem)	For detailed soil descriptions refer to RECORD OF BOREHOLE 1									
4											
6					1	75 DO	PH				
					2		10			/C	
		End of Hole						⊕ +			Caved
8											W.L. in Standpipe at elev. 61.37 March 7, 1989
10											
12											
14											
16											
18											
20											
22											
24											
26											
28											
30											

+ PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

1 : 150

Golder Associates

LOGGED S.Leighton

CHECKED CRM

McROSTIE SETO GENEST
& ASSOCIATES LTD. & ASSOCIÉS LTÉE
CONSULTING ENGINEERS – INGÉNIEURS CONSEILS
OTTAWA CANADA

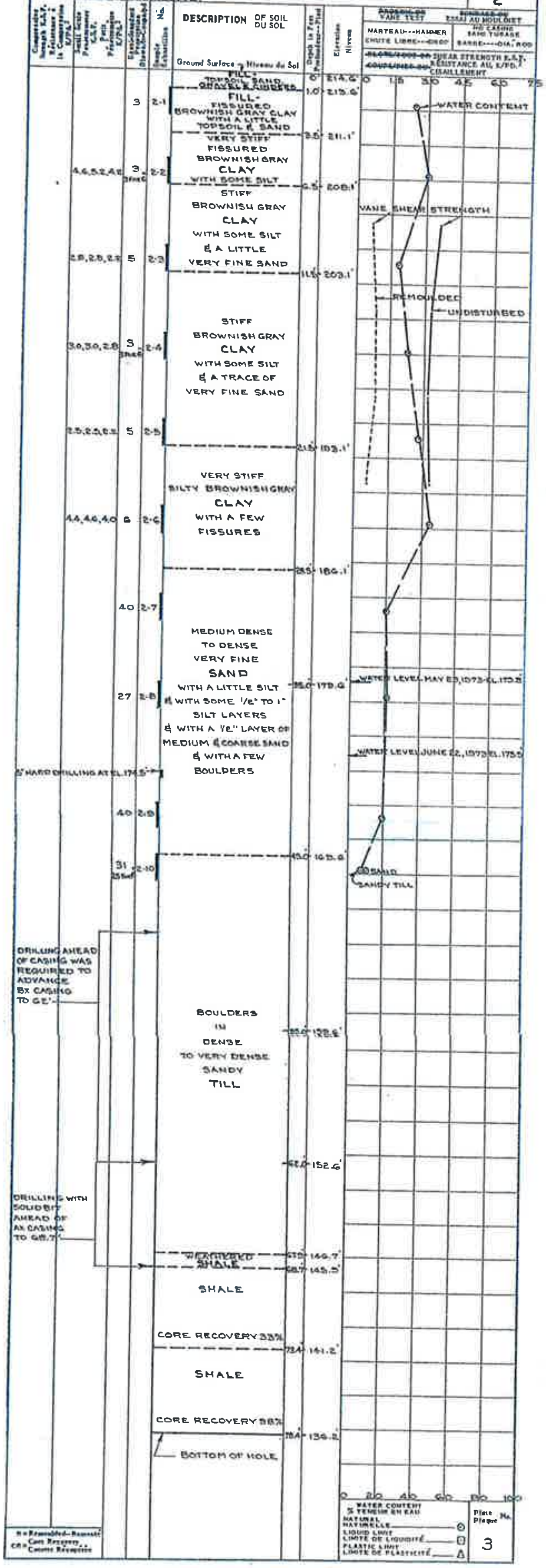
SOIL PROFILE & TEST SUMMARIES
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

WURTEMBURG ST. No. 2

ELEVATION OF GROUND SURFACE (ZERO DEPTH) / NIVEAU DU SOL (PROFONDEUR ZÉRO) 214.6' DATE MAY 15, 1973

HOLE / FORAGE No. 2

NOTES B.M. (EL. 237.0X) GEODETIC TABLET No. 3466, NORTHWEST CORNER OF RIDEAU & CHARLOTTE.



McROSTIE & ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

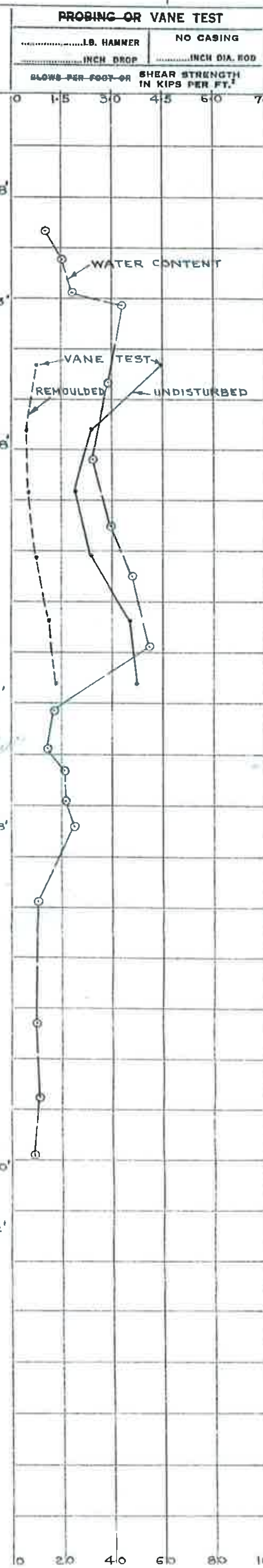
WURTEMBURG OPPOSITE HENEY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 224.8' DATE APRIL 26, 1963

REMARKS SEE PLATE No. 2

HOLE No. 4

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	224.8'
		6	4-1	LOOSE FINE SAND WITH A LITTLE GRAVEL		
		4 for 6"	4-2		6.5'	
		9 for 6"	4-3A	MEDIUM DENSE TO LOOSE FINE SAND WITH A LITTLE SILT		
		5 for 6"	4-3B		8.5'	216.3'
	5.0, 5.5, 5.0	6 for 6"		VERY STIFF TO STIFF SILTY BROWNISH GRAY CLAY WITH A FEW FISSURES		
	5.4, 5.6, 5.8 3.5, 3.8, 4.0 R-0.8		4-5		20'	204.8'
	3.4, 3.2, 3.4 3.5, 3.0, 3.6 3.4, 3.5, 3.0 2.6, 2.8, 2.7 R-0.2		4-6	STIFF SILTY GRAY CLAY		
	2.6, 2.4, 2.7 3.4, 3.2, 3.2 3.8, 3.6, 3.8 4.4, 3.6, 3.8 3.6, 3.4, 3.8 R-0.2		4-7		30'	
	3.8, 4.0, 3.8 3.6, 4.0, 4.2 3.5, 3.8, 3.7 R-0.0		4-8	STIFF TO VERY STIFF SILTY GRAY CLAY		
	4.6, 4.6, 4.9 4.6, 4.6, 4.6 4.0, 4.6, 4.8 4.0, 4.2, 4.0 R-0.0		4-9		39.5'	185.3'
		46	4-10	DENSE VERY FINE SAND	41.5'	
		17	4-11	BOULDERS IN MEDIUM DENSE GRAVELLY FINE SAND	45.6'	
		10 for 6" 49	4-12	DENSE VERY FINE SAND & SILT	48.5'	
		45 11 for 6"	4-13	MEDIUM DENSE VERY FINE SAND & SILT	50'	174.8'
		67	4-14	DENSE FINE TO VERY FINE SAND	55'	
		22	4-15	BOULDERS IN MEDIUM DENSE SILTY TILL		
		33 for 3"	4-16			
		16	4-17			
		22 16 for 6"	4-18		76'	
		26 5 for 6"	4-19	LOOSE SILTY TILL	76.8'	148.0'
				BOULDERS	81.6'	143.2'
				BOTTOM OF HOLE		



At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
32 Steacie Drive
Kanata, Ontario, K2K 2A9
Canada
T: +1 (613) 592 9600

Golder Associates