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REPORT

REPORT ON

Geotechnical Investigation Proposed Residential Building 101 Wurtemburg Street Ottawa, Ontario

Submitted to:
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1.0 INTRODUCTION

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

The purpose of this report was to review the results of a previous investigation carried out for the site and, based on an interpretation of the factual information, provide a general description of the subsurface conditions across the site. These interpreted subsurface conditions and available project details were then used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

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).

Plans are being prepared for the proposed development of the site with a high-rise residential building including up to three levels of underground parking. The building footprint (basement portion) will occupy most of the site and will measure approximately 36 metres by 15 metres in plan area; however the above-grade tower will be somewhat smaller.

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

The rear yard is essentially unvegetated and the ground level is about 1 metre lower than the front part of the property. The slope down to the Rideau River is quite densely vegetated but with relatively juvenile tree cover. The slope is approximately 12 metres high and inclined at just slightly flatter than 1 horizontal to 1 vertical (1H:1V). It appears that the slope toe forms the river bank (i.e., there is no apparent flood plain separating the slope toe from the river bank).

The building to the north (apparently an embassy building) is three storeys in height. 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 that 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.

Golder Associates previously carried out a geotechnical investigation on the 101 Wurtemburg site in 1989. The results of that investigation, along with geotechnical guidelines on the development 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 No. 891-2060).

Geotechnical investigations were also carried out on the 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 scope of this current report is to review the results obtained from the previous investigations on (and adjacent to) this site and develop foundation engineering guidelines in a manner consistent with Part 4 of the 2006 Ontario Building Code (OBC).

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 sandy soil, and 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. Published geologic mapping also indicates that a fault exists to the east of this site; the bedrock surface level and quality could therefore be somewhat irregular.

3.0 PROCEDURE

The field work for the previous investigation on this site by Golder Associates was carried out on February 27 and 28, 1989. During that time, one borehole (numbered borehole 1) was advanced at the approximate location shown on the Site Plan, Figure 2. One additional borehole (numbered borehole 1A) was put down adjacent to borehole 1 to obtain a Shelby tube sample of the silty clay.

The boreholes were advanced using a track-mounted continuous flight hollow-stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. Borehole 1 was advanced to a depth of about 19 metres below the existing ground surface, with regular sampling and in-situ testing, and then further extended to 27.5 metres depth, without sampling, prior to encountering practical refusal to augering. Borehole 1A was advanced to a depth of about 6.9 metres below the existing ground surface

Standard penetration tests were carried out at regular intervals to about 19 metres at borehole 1 and samples of the soils encountered were recovered using drive open sampling equipment. In-situ vane shear testing was carried out in the cohesive silty clay layer to determine the undrained shear strength of this material.

Two relatively undisturbed, 75-millimetre diameter thin-walled Shelby tube samples of the silty clay were obtained in borehole 1A using a fixed piston sampler.

A standpipe was sealed into the silty clay at borehole 1A to allow subsequent measurement of the stabilized groundwater level at the site. The water level in the standpipe was measured on March 7, 1989.

The field work was supervised by a member from our staff who located the boreholes, directed the drilling operations and in-situ testing, logged the boreholes, and took custody of the soil samples retrieved.

Upon completion of the drilling operations, samples of the soils obtained from the boreholes were transported to our laboratory for examination by the project engineer and for laboratory testing. The laboratory testing included natural water content determinations, Atterberg limit tests, grain size distribution tests, and oedometer consolidation testing.

The borehole locations were selected and subsequently surveyed by Golder Associates personnel using the existing features at the site. The ground surface elevations at the borehole locations were referenced to the top of the manhole cover on the west side of Wurtemburg Street, immediately opposite the site. The elevation of this point is given as 67.49 metres, geodetic datum, on the J.G. Payette Ltd. survey plan of the site.

One sample of the groundwater was recovered from borehole 1 and submitted to Accutest Laboratories Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered at the boreholes put down for the previous Golder investigation are shown on the Record of Borehole sheets in Appendix A. The results of the water content determinations and Atterberg Limit tests are also shown on the Record of Borehole sheets. The subsurface conditions encountered at the relevant boreholes put down for the previous McRostie investigations are shown on the borehole records in Appendix B. The results of basic chemical analysis carried out on a sample of groundwater from borehole 1 of the previous Golder investigation are provided in Appendix C.

In general, subsurface conditions on this site consist of a thick deposit of sensitive marine clay, underlain by sandy soil, and glacial till.

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 Associate (Report No. 891-2060). However, reference is also made to the results of the previous McRostie boreholes advanced on the adjacent sites (i.e., borehole 2 from Report No. SF1625 and borehole 4 from 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 present 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 at the previous McRostie borehole 2 to the north of the site.

4.3 Sensitive Silty Clay

The surficial fill materials at borehole 1 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 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 very stiff to 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 high 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 percent, reflecting medium 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 provided on Figure 3 and are summarized below.

Borehole/ Sample No.	Sample Depth/Elevation (m)	Unit Wt. (kN/m^3)	σ_P' (kPa)	σ_{VO}' (kPa)	C_c	C_r	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	σ_{VO}'	-	Computed existing vertical effective stress
C_c	-	Compression index	C_r	-	Recompression index
e_o	-	Initial void ratio	OCR	-	Overconsolidation ratio

A similar silty clay deposit was encountered at 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 at borehole 1.

4.4 Sand

A layer of silty fine sand was encountered beneath the silty clay at 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 at the McRostie boreholes, however the deposits were thicker. The sand layers at boreholes 2 and 4 were about 5.0 and 4.7 metres thick, respectively. The materials encountered at 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 trace clay. The glacial till was inferred to a depth of 27.5 metres (i.e., elevation 39.9 metres) at 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 results of grain size distribution testing on the glacial till are provided on Figure 4.

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 at 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 2 (north of the site) was advanced to the bedrock surface at about 20.7 metres depth (elevation 44.7 metres). Shale bedrock was then cored to about 23.9 metres depth (i.e., 3.2 metres into the bedrock) using rotary diamond drilling (i.e., rock coring) techniques.

4.7 Groundwater

The groundwater level in a standpipe sealed into the silty clay at 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 at 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.

Based on the above, there is potentially a downward flow gradient from the silty clay deposit to the sand layer.

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 engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the borehole information and project requirements, and is subject to the 'Important Information and Limitations of this Report' which follows the text but forms an integral part of this document.

The foundation engineering guidelines presented in this section have been developed in a manner consistent with Part 4 of the 2006 Ontario Building Code (OBC).

5.2 Excavations

It is understood that the proposed building will encompass essentially the full limits of the property and the lowest basement level will have a design floor elevation between 55.5 and 59 metres.

Considering that the excavation will likely need to extend a further 1 to 1.5 metres below the basement floor level to accommodate the foundations, the excavation is expected to extend to as much as about 13 to 14 metres below the existing ground surface (of about elevation 67 to 67.5 metres). The basement floor elevation will in fact vary across the building footprint due to the stepping parking levels and the possible use of parking elevators.

The excavation will be made through surficial fill materials and very stiff to firm silty clay. The deepest parts of the excavation will also probably extend into the sand layer and may potentially reach the glacial till.

No unusual problems are anticipated in excavating the overburden using conventional hydraulic excavating equipment.

The soils at this site would generally be classified as Type 3 soils in accordance with the Occupational Health and Safety Act (OHSA) and therefore, if open cut side slopes were considered (or feasible), they would need to be cut back at an inclination no steeper than 1H:1V. However the sand layer at depth, if encountered by the excavation and not first dewatered, would be classified as a Type 4 soil; side slopes of 3H:1V would be required in accordance with OSHA. It is expected however, given the close proximity of the adjacent roads and properties/buildings, that it will instead be necessary to shore this excavation. Guidelines on excavation shoring are provided in Section 5.3 of this report.

Some groundwater inflow into the excavation should be expected. For excavations within the silty clay, it is expected that, due to the relatively low hydraulic conductivity of this material, it should be possible to handle the groundwater inflow from this deposit by pumping from well filtered sumps in the floor of the excavation, using suitably sized pumps. However, for deeper excavations which extend into the sand layer (i.e., for excavations more than 11 metres deep), the groundwater level may be encountered and the rate of groundwater inflow could be significant, particularly given the close proximity of the river.

Therefore, for excavations which would reach the sand (and therefore likely encounter the groundwater level), some form of active dewatering will be required and the groundwater level will need to be lowered in advance of the excavation. Otherwise the rate of groundwater inflow to the excavation would be excessive, which might:

- Make excavation of the soils difficult;
- Flood the excavation or make it untrafficable;
- Disturb the subgrade and foundation bearing surfaces;
- Destabilize the excavation side slopes and/or shoring; and/or
- Make compaction of materials and the placement of concrete not feasible.

Even excavations which extend *close* to the groundwater level (e.g., within about 1 metre above it), may disturb the subgrade due to construction vibrations.

As such, some form of active dewatering of the sand layer would be required, so that the groundwater level could be maintained below the excavation level for the duration of the foundation construction. The design of the dewatering system should be entirely the responsibility of the excavation contractor. However it is envisaged that the system would consist of a series of well or well points that are installed into the sand layer.

The impact of temporary dewatering of the sand layer on nearby buildings would also need to be evaluated. A lowering of the groundwater level in the sand layer might result in a reduction in the piezometric level in the overlying clay layer, with the effect extending for some distance around the site (i.e., the sand layer would under-drain the clay). The lowering of the piezometric level would result in an increase in the effective stress level in the clay deposit (due to a reduction in the buoyant forces acting on the soil particles), which might induce settlements.

The installation of sheeting or some other form of hydraulic cut-off, installed through the sand layer to reach the surface of the glacial till, would assist with the dewatering and help limit the off-site impacts. However, the sand layer appears to be compact and therefore the driving of sheeting through the sand layer might be difficult.

Based on the above, it is considered that excavation below the clay layer will be challenging and will add considerable cost to this project. Further hydrogeologic assessment of the dewatering requirements would be required. Measurement of *current* groundwater levels should also be considered (rather than using the historic groundwater level data).

5.3 Excavation Shoring

5.3.1 Shoring Options

As discussed in Section 5.2, it is expected that the excavation will extend to about 13 to 14 metres below the existing ground surface.

Vertical (or near vertical) excavation walls will be required for the excavation, due to the proximity of adjacent properties/buildings and Wurtemburg Street. The contractor should be responsible for the detailed design of the shoring. However the following general guidelines on possible concepts for the shoring are provided, to be used by the designers for:

- Assessing the costs of the shoring;
- Assessing possible impacts of the shoring design and construction on the design of the structure and site works; and,

- Evaluating, at the design stage, the potential for impacts of this shoring on the adjacent structures, services, and roadways.

The shoring method(s) chosen to support the excavation sides must take into account: the soil stratigraphy, the groundwater conditions, the methods adopted to manage the groundwater, the permissible ground movements associated with the excavation and construction of the shoring system, and potential impacts on adjacent structures and utilities.

In general, there are three basic shoring methods that are commonly used in local construction practice:

- Steel soldier piles and timber lagging;
- Driven steel sheet piles; and,
- Continuous concrete (secant pile or diaphragm) walls.

These three options are listed in order of generally increasing stiffness and ability to resist ground movements. Solider piles and lagging are suitable where the objective is to maintain an essentially vertical excavation wall and where the movements above and behind the shoring need only be sufficiently limited so that relatively flexible features (such as roadways) will not be adversely affected. Where the deflections need to be more strictly limited, such as where heavily loaded foundations lie within the zone of influence of the shoring, continuous concrete shoring can be required. Sheet piling provides an intermediate level of stiffness.

Underpinning of existing adjacent foundations, to transfer the foundation loads to below the excavation level, can also be required/justified, in addition to shoring of the excavation, if the settlements due to shoring movements would be unacceptable and/or if the loads from the adjacent foundations are large.

For all of the above systems, some form of lateral support to the wall is required for excavation depths greater than about 4 metres, which is the case for this site. Lateral restraint could be provided by means of tie-backs consisting of soil anchors or grouted bedrock anchors. The significant depth of the bedrock on this site makes rock anchors for tie-backs long and costly; consequently earth anchors could be considered (though not commonly used in Ottawa). However the use of soil or rock anchor tie-backs would require the permission of the adjacent property owners (including the City, who owns the adjacent roadways) since the anchors would be installed beneath their properties. Obtaining permission to install tie-backs beneath the adjacent diplomatic property (to the north) may be difficult. The presence of utilities beneath the adjacent streets or piles beneath the existing buildings which could interfere with the tie-backs should also be considered. In particular, the records for the adjacent building to the south indicate it to be supported on piled foundations.

Alternatively, interior struts can be considered, connected either to the opposite side of the excavation (if not too distant) or to raker piles and/or footings within the excavation. However internal struts could interfere with the construction of the foundations and superstructure.

The passive resistance provided to the socketed/embedded portion of the shoring also contributes to the lateral resistance.

Particular issues for this site include:

- The possible need to provide a hydraulic cut-off around the excavation, should the excavation extend to more than about 11 metres depth (and into the water-bearing sand layer).
- The potential to induce settlement or lateral loading on the foundations of the adjacent buildings, either due to groundwater level lowering or due to movements of the shoring and the retained soil.

If the excavation will be maintained above the sand layer, it is envisioned that the shoring system might consist of the following:

- Steel solder pile and timber lagging along the west side of the site (adjacent to Wurtemburg Street)
- Rigid steel sheet pile shoring along the north and south sides of the site, due to the close proximity of the existing adjacent structures.
- No shoring along the east side, where presumably the slope would be fully excavated.

Since the building to the south is understood to be supported on piles (but needs to be confirmed), underpinning of its foundations is not expected to be necessary. However the potential impacts on the building foundations to the north (which is a shorter and lighter building) will need to be considered/evaluated.

If however the excavation will extend below about 11 metres depth and into the sand layer, then shoring may also need to form a hydraulic cut-off. The driving of sheeting around the full perimeter of the excavation, down through the sand layer to the surface of the glacial till, might be considered. However driving sheeting through the sandy layer might be difficult. It may instead be necessary to install a continuous concrete shoring system, at significantly increased cost. Further evaluation of the shoring design would be required.

5.3.2 Lateral Earth Pressures

The shoring should be designed to account for lateral earth pressures resulting from the weight of the retained earth and other dead and surcharge loads. The earth pressure distribution used for shoring design is dependent upon the specific wall design and on the nature of the lateral support provided.

The selection of that design lateral earth pressure should therefore be the responsibility of the contractor who will be responsible for the shoring design. The following guidelines are provided only to assist the project's designers with assessing the general shoring requirements.

Cantilevered or single tie-back soldier pile and lagging walls should be designed to resist an earth pressure distribution having a (variable) magnitude with depth equal to:

$$\sigma_h(z) = K_a\,(\gamma z + q)$$

Where: $\sigma_h(z)$ = Lateral earth pressure on the shoring at depth 'z', kilopascals;

K_a = Active earth pressure coefficient, use 0.33;

γ = Unit weight of retained soil, use 20 kilonewtons per cubic metre;

z = Depth below top of shoring, metres;

q = Surcharge at ground surface to account for traffic, equipment, or stock piled materials. To be no less than 15 kilopascals.

For shoring consisting of tie-back solider piles and lagging or interlocking steel sheet piling supported by internal struts, or multiple levels of tie-backs, the system should be designed to resist a 'trapezoidal' earth pressure distribution having a magnitude of:

$$\sigma_h = 0.4\ (\gamma H + q)$$

Where:
σ_h = Lateral earth pressure on the shoring, kilopascals;

γ = Unit weight of retained soil, use 20 kilonewtons per cubic metre;

H = Height of shoring (i.e., depth of excavation), metres; and,

q = Surcharge at ground surface to account for traffic, equipment, or stock piled materials. To be no less than 15 kilopascals.

This lateral earth pressure would apply over a depth interval equal to 0.5H, centred on the excavation depth. Over the upper 0.25H and lower 0.25H, the lateral earth pressure would vary linearly from nil at ground surface and to nil at the bottom of the excavation.

The above lateral earth pressures have not been factored; factoring of these loads will be required if the shoring is being designed in accordance with Limit States Design.

The potential for the loads from the adjacent foundations to apply additional lateral pressure to the shoring system should be considered.

If a continuous concrete shoring system is used/required, at-rest earth pressures may apply. Additional geotechnical guidelines would need to be provided.

5.3.3 Ground Movements

Some unavoidable inward horizontal deformation and vertical settlement of the adjacent ground will occur as a result of excavation, installation of shoring, and deflection of the ground support system (including bending of the walls, compression of the struts and/or extension of the tie-backs), as well as deformation of the soil in which the toe of the shoring is embedded. The ground movements induced could affect the performance of buildings, surface structures or underground utilities adjacent to the excavation.

As a preliminary guideline, typical settlements behind soldier pile and lagging or sheet pile shoring are less than about 0.2 percent of the excavation depth, provided good construction practices are used (e.g., supports are installed as soon as the support level is reached) and voids are not left behind the lagging. This is only a preliminary assessment of the potential settlements and is provided only to assist the owner's designers with evaluating the potential impacts of the expected settlements. A detailed assessment of the expected settlements should be undertaken by the shoring contractor. Should the preliminary assessment carried out using this estimated settlement indicate unacceptably large settlements to adjacent structures, roadways, or utilities, then a more detailed assessment should be carried out at the design stage (prior to tender) to better assess the shoring requirements, or a more rigid form of shoring should be selected.

Alternatively, if a properly designed and supported diaphragm wall or secant pile wall system is used for the shoring system, the ground movements should be negligible.

The high-rise residential building on the south side of the site is in close proximity to the proposed development. However, based on the previous inspection reports by McRostie, it is understood that this building is supported on pile foundations and therefore impacts by the ground movements should be minimal.

Based on the current plans, it appears the structure to the north of the development will be some 5 metres away and is founded at a lower elevation. Therefore, provided that the excavation is properly supported (i.e., a properly designed and constructed shoring system is used), the excavation is not likely to have significant impacts on this structure. Notwithstanding this assessment, a pre-construction survey of this structure, as well as all structures within 100 metres of the development, should be carried out prior to commencement of the excavation.

The presence of buried utilities beneath Wurtemburg Street could impact on the shoring design in terms of:

- Possible conflicts with tie-back installation; and/or
- Possible restrictions on the acceptable shoring movements (particularly if older water mains are present).

Therefore, an inventory of these utilities should be made at an early stage in the design.

5.4 Foundations

In general, the subsurface conditions at this site consist of a thick deposit of sensitive silty clay, overlying a layer of sand, followed by glacial till. The bedrock surface is inferred to be at depth of about 27.5 metres (i.e., elevation of about 40 metres) below the existing ground surface.

It is considered that there are two feasible foundations systems for this building:

- Driven steel piles; or
- A raft foundation.

A raft foundation is considered feasible only if:

- The foundation loads are not excessive; and
- Either the founding level will be above the water-bearing sand layer or the groundwater level in the sand is lowered in advance of excavation.

Rock-socked cast-in-place concrete caissons might also be feasible for this site, however this foundation system might be significantly more expensive.

5.4.1 Pile Foundation

A piled foundation system could be used to transfer the foundation loads to competent bearing at depth (i.e., down to the bedrock surface).

A suitable pile type would be concrete filled steel pipe piles (driven closed-ended) or H-piles, with the piles end-bearing on bedrock. For this site, the piles would be driven to practical refusal on the bedrock surface which appears to be at elevation of about 40 metres (i.e., 14 metres from the underside of the foundations).

A minimum 0.6 metre thick granular working mat should be provided for pile driving equipment to protect the silty clay subgrade.

5.4.1.1 Axial Resistance

As one possible design example, the ULS factored *structural* resistance of a 245-millimetre diameter steel pipe pile with a wall thickness of 9 millimetres may be taken as 1,500 kilonewtons. The ULS factored *geotechnical* resistance of the pile should equal or exceed the structural resistance if the piles are installed using an appropriate set criteria and using a hammer of sufficient energy. Note: The pile capacity/size to be used in the design may also be controlled by the dynamic testing program (see later discussion in this section).

For piles end-bearing on or within bedrock, SLS conditions generally do not govern the design since the stresses required to induce 25 millimetre of movement (i.e., the typical SLS criteria) exceed those at ULS. Accordingly, the post-construction settlement of structural elements which derive their support from piles bearing on bedrock should be negligible.

Pipe piles should be equipped with a base plate having a thickness of at least 20 millimetres to limit damage to the pile tip during driving.

The piles should be driven no closer than three pile widths/diameters centre to centre.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile, and length of pile; the criteria must therefore be established at the time of construction and after the piling equipment is known. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles will have adequate capacity, but are also not overdriven and damaged. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

Relaxation of the piles following the initial set could result from several processes, including:

- Softening of the bedrock into which the piles are driven;
- The dissipation of negative excess pore water pressures in the overburden material above the bedrock surface; and,
- The driving of adjacent piles.

Provision should therefore be made for restriking all of the piles at least once to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first restrike should receive additional restriking until the design set is met. All restriking should be performed after 48 hours of the previous set.

For the dense glacial till and potential shale bedrock, several rounds of restriking could potentially be required. The need for multiple restrikes could be reduced by using a reduced geotechnical capacity for the piles.

Some of the piles may not fully penetrate the bouldery glacial till to reach the bedrock surface; some of the piles may instead “hang up” at a shallower depth in the glacial till. In that case, pre-drilling of the glacial till could be considered. Alternatively, these particular piles may need to be designed for a reduced capacity. The ULS factored axial resistance of these piles will depend on the depth to which they penetrate and the set that is achieved. The capacities of these piles may have to be confirmed in the field by carrying out load testing.

Due to their smaller cross section, H-piles might have more success in penetrating the glacial till and reaching the bedrock surface. However the integrity of pipe piles following driving may be more readily inspected (by visual examination of the pile interiors) than for H-piles, and therefore damaged piles can be more easily identified. As well, H-piles are significantly more expensive. The option of using H-piles could however be discussed with the piling contractor.

It is recommended that dynamic monitoring and capacity testing (known as PDA testing) be carried out (by the contractor) at an early stage in the piling operation to verify both the transferred energy from the pile driving equipment and the load carrying capacity of the piles. As a preliminary guideline, the specification should require that at least 10 percent of the piles be included in the dynamic testing program. CASE method estimates of the capacities should be provided for all piles tested. These estimates should be provided by means of a field report on the day of testing. As well, CAPWAP analyses should be carried out for at least one third of the piles tested, with the results provided no later than one week following testing. The final report should be stamped by a professional engineer licensed in the province of Ontario.

The purpose of the PDA testing will be to confirm that the contractor's proposed set criteria is appropriate and that the required pile geotechnical capacity is being achieved. It will therefore be necessary for the pile to have sufficient structural capacity to survive that testing, which could require a stronger pile section than would otherwise be required by the design loading.

For example, for the PDA testing to be able to record/confirm a factored geotechnical resistance of 1,500 kilonewtons, it will be necessary to successfully proof load the tested piles to 3,000 kilonewtons during the PDA testing (per the resistance factor of 0.5 to be applied to PDA test results, as specified in Commentary K of the National Building Code of Canada (NBCC)). However that proof load may exceed the actual structural capacity of the piles. If the piles would fail (structurally) at a lower load, then the full geotechnical capacity cannot be confirmed (and piles loaded to 3,000 kilonewtons will have been damaged and will need to be wasted).

The following options could therefore be considered:

- Piles with a higher *structural* capacity could be specified (i.e., piles with a ULS factored structural resistance higher than the factored geotechnical resistance, and higher than required by the design loading), so that the piles can be successfully tested to the required loading, and so that the geotechnical capacity can then be confirmed by the PDA testing. This option could significantly increase the cost of the piled foundations.
- A reduced ULS factored geotechnical resistance could be used for the design, such that the piles would have sufficient structural capacity to be tested/loaded to twice the design geotechnical resistance. This option would also increase the cost for the piled foundations, by increasing the number of piles that would be required.
- The PDA results could be used/evaluated using 'Working Stress Design' (WSD) criteria (i.e., using a factor of safety of 2), rather than using 'Limit States Design' (LSD) methods. The pile capacities assessed using PDA test results have, in fact, traditionally been established using WSD methods, by applying a factor of safety (typically 2.0) against the ultimate pile capacity determined by the testing, and then checking the resulting capacity versus the working/service load.

  However, in compliance with the OBC and the NBCC, LSD methods are now being used (by applying a resistance factor against the ultimate pile capacity determined by the testing) and these two methods do not yield a consistent pile design. However, for field verification of the set criteria and geotechnical capacity, the structural engineer (and owner) could potentially accept a WSD methodology, given its traditional use.

- Static load testing could be carried out, rather than PDA testing, to confirm the ULS geotechnical resistance of the piles since the OBC/NBCC specifies a resistance factor of 0.6 for static load tests (instead of 0.5). However it may still not be feasible to prove the *full* factored geotechnical resistance.

The foundation and piling specifications should be reviewed by Golder Associates prior to construction tendering and the contractor's submission (i.e., shop drawings, equipment, procedures, and set criteria) should be reviewed by the geotechnical consultant prior to the start of piling.

Piling operations should be inspected on a full time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

5.4.1.2 Resistance to Lateral Loading

Lateral loading could be resisted fully or partially by the use of battered piles and/or rock anchors.

If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. The SLS resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equations given below.

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where:
- n_h = The constant of horizontal subgrade reaction, as given below;
- z = The depth, metres; and,
- B = The pile diameter/width, metres.

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where:
- s_u = The undrained shear strength of the soil kilopascals; and,
- B = The pile diameter/width, metres.

The constant of horizontal subgrade reaction depends on the soil type and the soil density/consistency around the pile shaft. For the design of resistance to lateral loads, the values indicated in the table below may be used. All values quoted are unfactored geotechnical parameters.

Elevation (m)	Soil Type	n_h (MPa/m)	S_u (kPa)
64.3 to 62.4	Very Stiff to Stiff Weathered Silty Clay	-	60
62.4 to 56.1	Firm to Very Stiff Grey Silty Clay	-	40
56.1 to 54.1	Compact Sand	4.4	-
54.1 to 39.9	Loose to Dense Glacial Till	4.4	-

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading using a reduction factor, R, as follows:

Pile Spacing in Direction of Loading d = Pile Diameter or Width	**Subgrade Reaction Reduction Factor, R**
8d	1.00
6d	0.70
4d	0.40
3d	0.25

For establishing the ULS factored *structural* resistance, the shear force and bending moment distribution in the piles under factored loading can be established using the procedures and parameters given above for evaluating the SLS response of the pile.

The ULS *geotechnical* resistance to lateral loading may be calculated using the passive earth pressure theory mobilized in front of the piles. For the silty clay on this site, the lateral resistance is assumed to vary linearly with a magnitude of $2S_u$ at pile cap level to a magnitude of $9S_u$ at a depth equal to three pile diameters below the underside of the pile cap, where S_u is the undrained shear strength of the silty clay. Below a depth equal to three pile diameters, the lateral resistance is assumed to be constant at $9S_u$, to the bottom of the silty clay deposit.

The ULS lateral passive resistance from the sand and glacial till may be assumed to act over the pile shaft to a depth equal to six pile diameters below the underside of the pile cap, except where the silty clay thickness exceeds that depth. The resistance can be calculated as follows:

Above the water table: $P_p(z) = 3\, d\, K_p\, \gamma\, z$

Below the water table: $P_p(z) = 3dK_p\, \gamma\, D_w + 3dK_p\, (z - D_w)\, (\gamma - \gamma_w)$

Where:

$P_p(z)$ = ULS lateral resistance at depth ‘z’ below ground surface, kilonewtons per meter;

γ = Average unit weight of overlying soil, use 20 kilonewtons per cubic metres

K_p = Coefficient of passive earth pressure, use 3.7;

D_w = Depth to groundwater table below ground surface (metres), assume at top of sand;

γ_w = Unit weight of water, use 9.8 kilonewtons per cubic metres; and,

d = Pile diameter/width, metres.

The ULS lateral resistance of a pile group may be estimated as the sum of the individual pile resistances across the face of the pile group, perpendicular to the direction of the applied lateral force.

The ULS resistances obtained using the above parameters represent unfactored values; a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.

If the soil in front of the piles will be relied upon to resist lateral loading, then it will be important that the pile splices be capable of developing the full bending moment resistance of the pile section. The welding detail of the pile splices should be designed on that basis. Ideally, pile splices within the upper portion of the piles should be avoided.

Additional lateral resistance can also be provided by socketing the piles into the bedrock.

The passive resistance against the foundation wall will also provide additional resistance to lateral loading, as is discussed in Section 5.8 of this report.

If the uplift resistance of the piles will need to be relied upon, additional geotechnical guidelines will need to be provided. Similarly, guidelines can be provided on vertical or inclined rock anchors, if required.

5.4.2 Raft Foundations

A foundation alternative for the proposed structure at this site would be a 'raft' foundation. A raft foundation would need to be sufficiently rigid so that the building loads would be relatively uniformly distributed over the entire building footprint.

If a raft foundation is considered for the proposed structure, additional geotechnical investigation will be required to confirm the uniformity of the subsurface conditions, particularly at the rear of the property.

The available bearing resistance for support of the raft foundation will depend on the founding level, since it impacts on both the bearing stratum and on the compensating effect of the weight of the excavated soil. As currently proposed, the founding level will vary across the footprint, ranging from about elevation 54 to 58 metres. That varying founding level is not ideal for construction of a raft foundation in terms of limiting differential settlements and for subgrade preparation. The founding stratum would also vary, with sand or glacial till supporting the deeper portions and silty clay supporting the shallower portions. An additional complication is the need to first dewater the sand layer, if it is to be used as a bearing stratum; otherwise the groundwater inflow would disturb the sand.

Based on the above, further evaluation and design input will be required if a raft foundation is to be considered. Only preliminary guidelines are therefore provided at this stage.

For the proposed shallower founding level, the excavation of the native soils will result in an unloading of the underlying soils by about 170 kilopascals. Therefore, the raft foundation can be designed using an SLS gross contact stress of 170 kilopascals, provided the current groundwater level is maintained (i.e., water-tight construction is used below the natural groundwater level). This design would result in essentially no net stress increase and, as such, the total settlement of the raft should be small.

However, if the weight of the building would exceed this SLS resistance, it is considered that the net stress increase on the silty clay at the founding depth could be limited to no more than 100 kilopascals without the building experiencing excessive settlements. Therefore, the corresponding SLS gross resistance under the raft would be 270 kilopascals. This value also assumes that the current groundwater level is maintained (i.e., water-tight construction is used for the below the groundwater level).

The ULS factored bearing resistance that may be used for the design of the raft foundation is 300 kilopascals.

The post-construction total and differential settlements of the raft will depend, in part, upon the length of time that passes between the excavation being made and the building load being applied, since the clay will 'rebound' (i.e., swell) following removal of the weight of the overlying soil. That rebound will be recovered as settlement once the structure loads are imposed on the raft. The post-construction settlements will be larger for corresponding longer lengths of time between excavating and re-loading.

The *total* settlement of the raft foundation is expected to be in the order of 25 to 50 millimetres, depending in part upon that duration and noting that the larger settlement estimate would correspond to a period of several months of unloading. The corresponding *differential* settlements across the length or width of the structure are estimated at about half the total settlement (assuming a uniform founding level), but will also depend greatly on the stiffness of the raft.

Having a stepping/varying founding level would increase the differential settlements, particularly if the bearing stratum differs across the raft foundation. That arrangement should ideally be avoided.

Further, the SLS resistance for the case of a net stress increase corresponds to a settlement resulting from consolidation of the silty clay. Consolidation of silty clay is a process which takes months or longer and, as such, results from sustained loading. Therefore, the foundation loads to be used in conjunction with the SLS resistance given above should be the full dead load plus sustained live load. The factored dead load plus full factored live load should be used in conjunction with the ULS factored bearing resistance.

The SLS resistance and corresponding settlement estimates are dependant upon the soil at or below founding level not being disturbed during construction. The silty clay and sand subgrades will be very sensitive to disturbance by construction traffic, especially in the presence of water. It should therefore be planned to place a mud slab of lean concrete at the base level immediately upon completion of excavation, to minimize disturbance of the subgrade material.

It should also be noted that the localized differential settlements (i.e., raft slab deflections) within/beneath individual bays (such as directly beneath a column versus the mid-span of the bay) will depend upon the relative stiffness between the raft slab and the underlying subgrade. The deflections and the resulting forces and bending moments in the slab to be used in its structural design could be determined by structural analysis using a modulus of subgrade reaction, k_s, for the subgrade.

It should be noted however that the modulus of subgrade reaction is not a fundamental soil property and its value depends, in part, on the size and shape of the loaded area. For the analysis of the contact stress distribution beneath a raft foundation, its value would depend on the size of the areas over which increased/concentrated contact stresses are anticipated (analogous to equivalent footings beneath the columns); the size of these areas is in turn related to the value the modulus of subgrade reaction, i.e., they are inter-related.

Accordingly, the analysis of the raft slab should ideally involve an iterative analysis between the determination of the contact stress distribution by the structural engineer and the geotechnical determination of the modulus of subgrade reaction value, until the two are consistent with each other. For initial analyses, the modulus of subgrade reaction may be assumed to be in the range of 3 to 20 megapascals per metre. This range reflects both uncertainty in the size of the loaded area as well as variability in the properties of the subgrade soils. The structural design of the slab at any location should be determined based on whichever value causes the larger effect, since either the maximum and minimum modulus values may govern for different locations and load effects (e.g., shear force versus bending moments).

5.4.2.1 Sliding Resistance

The parameter values in the following table may be used to calculate the lateral resistance at the foundation-soil interface:

Founding Material	Drained Parameters		Undrained Shear Strength (kPa)
	Effective Interface Friction, tan δ* (degrees)	Effective Cohesion (kPa)	
Firm to Very Stiff Grey Silty Clay	0.36	7.7	40
Compact Sand	0.43	0	-
Loose to Dense Glacial Till	0.43	0	-

For foundations on the *silty clay*, separate parameters apply for short term (undrained) and long term (drained) loading. Both conditions should be checked and resistance values for both conditions are provided in the above table.

Should there be a case where the foundation would be supported on engineered fill, a tan δ* value of 0.5 may be used at the foundation-engineered fill interface.

The resistances obtained using the above parameters represent unfactored values; a resistance factor should be applied in calculating the horizontal resistance.

5.5 Frost Protection

All perimeter and exterior foundation elements or interior foundation elements in unheated areas should be provided with a minimum of 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings/pile caps adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover.

It is expected that these requirements will be satisfied for all of the foundation elements of the structure due to the deep founding levels required to accommodate the three levels of underground parking.

Insulating the bearing surface with high density insulation could be considered as an alternative to earth cover for frost protection. Further details can be provided if and when required.

5.6 Seismic Design

For design in accordance with Part 4 of the 2006 OBC, the seismic site response classification could be assigned a Site Class D.

However, this is a preliminary value, since the combination of cohesive and granular soils at this site makes interpretation of the Site Class using only borehole results uncertain. There are also uncertainties because there is no blow count data available below about elevation 48 metres and because the presence of bedrock below about elevation 40 metres is not confirmed. Consideration should therefore be given to carrying out shear wave velocity testing of the upper 30 metres of soil and/or rock below founding level to more accurately evaluate the Site Class.

In addition, the soils at this site are not considered to be liquefiable.

5.7 Basement Floor Slab

It is understood that the finished basement floor slab of the lowest level of the structure would be at about elevation 55.5 to 59 metres, which would be within the unweathered grey silty clay.

The geotechnical design guidelines for the floor slab will depend on the foundation system, the foundation depth, and on the potential for impacts on adjacent structures (i.e., settlements) due to drawdown of the groundwater level. As discussed previously, water-tight construction may be required below the groundwater level; otherwise the drawdowns could induce consolidation of the clay deposit and settlements of surrounding structures.

Further study will be required to evaluate the need for water-tight construction and, if so, below what level the foundations need to be water-tight. There appears to be a downward hydraulic gradient at this site. The groundwater level in the sand layer is not known with certainty (since it was not directly measured by any piezometers on the site) but, based on nearby information, is inferred to be slightly above the river level. For the purposes of this assessment, a groundwater level at the top of the sand layer (i.e., about elevation 56.1 metres) is considered reasonable. The groundwater level in the overlying clay deposit was previously measured (in 1989) to be at about elevation 61.4 metres. This water level approximately corresponds to the basement level of the adjacent building to the south of this site.

It is expected that, since the sand deposit appears to under-drain the clay deposit (and likely discharges to the river), any effects due to lowering of the groundwater level in the sand deposit would be localized. Therefore significant and far-reaching effects due to groundwater level lowering would probably be avoided, provided the groundwater level in the sand deposit is not lowered. However, the potential impacts due to drying of the clay beneath the floor of the adjacent structure would need to be evaluated (note: since the structure to the south is pile-supported, *foundation* settlements should not be an issue).

Based on the above, it is expected that water-tight construction would only be required below about elevation 56.1 metres. However further evaluation of this issue is required, and confirmation of the current groundwater levels in the clay and sand would be of value for that assessment.

5.7.1 Slab on Grade

If the structure is to be supported on piles and will have a floor level above elevation 56.1 metres, it is considered that conventional construction can be used for this building basement floor slab.

It is not yet known whether the lowest level of the parking garage will have a concrete floor or a paved driving surface. In either case, any loose, wet, and disturbed material should be removed from beneath the entire structure footprint.

To prevent hydrostatic pressure build up beneath the floor and potential groundwater infiltration, it is suggested that the granular base for the floor be drained. Provision should be made for at least 300 millimetres of 6 millimetre clear crushed stone to underlie the floor. Where a concrete floor slab will be provided, the clear stone chip can form the base layer for that floor. Where a paved surface will be provided, the clear stone chip should be covered with 150 millimetres of Ontario Provincial Standards (OPSS) Granular ‘A’ followed by 50 millimetres of asphaltic concrete.

Any bulk fill required to raise the grade to the underside of the clear stone should consist of OPSS Granular ‘B’ Type I or II.

The underslab fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

Rigid 100 millimetre diameter perforated pipes should be installed within the clear stone at 6 metre centres. The perforated pipes should discharge to an outlet such as a sump from which the water is pumped.

If this form of floor construction is determined to be feasible even below the surface of the sand, then there would be the potential for loss of ground and plugging of the drainage system due to the loss of soil particles from the subgrade soils into the underslab clear stone fill resulting from the groundwater inflow. In that case, a Class II nonwoven geotextile having a Filtration Opening Size (FOS) not exceeding 100 microns in accordance with OPSS 1860 should be placed on the subgrade, with overlaps of at least 0.5 metres between rolls. The placement of the geotextile should be inspected and approved by qualified geotechnical personnel.

5.7.2 Raft Slab

Guidelines on the design of the raft slab were provided in Section 5.4.2 of this report.

Depending on the founding level, the raft may need to be designed to resist upward hydrostatic forces. The buoyant uplift forces on the structure should be evaluated to check for net uplift. For design purposes, it is recommended that a groundwater level at about elevation 56.1 metres be used. However, further geotechnical input on this issue will need to be provided.

In preparation for construction of the raft slab, any disturbed or deleterious materials must be removed from within the proposed building area. A mud slab should then be poured on the subgrade to protect it from disturbance.

5.8 Foundation Wall Backfill

The basement walls may be poured using formwork or alternatively the excavation shoring could serve as formwork. On the east side of the building, where no shoring will be required, and open-cut excavation will need to be backfilled. The geotechnical recommendations depend on the specific condition for each wall and the method that is selected. The need for a water-tight foundation will also impact on the backfill design.

5.8.1 Open Cut Excavations

The soils at this site are generally frost susceptible and should not be used as backfill against exterior, unheated, or well insulated foundation elements within the depth of potential frost penetration (1.5 metres) to avoid problems with frost adhesion and heaving. Free draining backfill materials are also required if hydrostatic water pressure against the basement walls (and potential leakage) is to be avoided. The foundation and basement walls should therefore be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS Granular 'B' Type I or II.

To avoid ground settlements around the foundations, which could affect site grading and drainage, all of the backfill materials should be placed in maximum 0.3 metre thick lifts, compacted to at least 95 percent of the material's standard Proctor maximum dry density.

Unless water-tight construction is required, the basement wall backfill should be drained by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in a geotextile, which leads by positive drainage to a storm sewer or to a sump from which the water is pumped.

5.8.2 Shored Excavations

Where shoring will be provided, the basement walls may be poured using formwork or alternatively the excavation shoring could serve as formwork.

If a drained foundation is to be provided, then the following guidelines apply:

- Where basement walls will be poured against shoring, vertical drainage such as Miradrain must be installed on the face of the shoring to provide the necessary drainage. The top edge of the Miradrain should be sealed or covered with a geotextile to prevent the loss of soil into the void between the sheet and geotextile of the Miradrain.
- Where the basement walls will be constructed using formwork, it will be necessary to backfill a narrow gallery between the shoring face and the outside of the walls. The backfill should consist of 6 millimetre clear stone 'chip', placed by a stone slinger or chute. In no case should the clear stone chip be placed in direct contact with other soils. For example, surface landscaping or backfill soils placed near the top of the clear stone backfill should be separated from the clear stone with a geotextile.
- Both the drain pipe for the wall backfill and/or the Miradrain should be connected to a perimeter drain at the base of the excavation which is connected to a sump pump.
- Conventional damp proofing of the basement walls is appropriate with the above design approach. For concrete walls poured against shoring, damp proofing using an interior treatment such as Crystal Lok is suggested.

If a water-tight foundation is to be provided, then the following guidelines apply:

- Where basement walls will be poured against shoring, no drainage layer is required. However the upper portion of the walls, within the depth of frost penetration (1.5 metres), should be backfilled with free draining non-frost susceptible granular fill to avoid frost adhesion and heaving.
- Where the basement walls will be constructed using formwork, it will be necessary to backfill a narrow gallery between the shoring face and the outside of the walls. The backfill could consist of unshrinkable fill; however the upper portion of the walls, within the depth of frost penetration (1.5 metres), should be backfilled with free draining non-frost susceptible granular fill to avoid frost adhesion and heaving. The structure would also need to be designed to resist the temporary fluid pressure and buoyant uplift forces. Alternatively the gallery could be backfilled with 6 millimetre clear stone 'chip', placed by a stone slinger or chute. In no case should the clear stone chip be placed in direct contact with other soils. For example, surface landscaping or backfill soils placed near the top of the clear stone back fill should be separated from the clear stone with a geotextile.
- No drainage system should be provided at the base of the wall.
- The need for damp proofing should be discussed with the architect and structural engineer.

5.8.3 Lateral Earth Pressures

The magnitude of the lateral earth pressures will depend on the backfill materials and backfill conditions adjacent to the foundation walls. If the backfill materials consist of compacted sand or sand and gravel (OPSS Granular 'B' Type I or II), then the lateral earth pressures may be taken as:

$$\sigma_h(z) = K_o\,(\gamma z + q)$$

Where: $\sigma_h(z)$ = Lateral earth pressure on the wall at depth z, kilopascals;

K_o = At-rest earth pressure coefficient, use 0.5;

γ = Unit weight of retained soil, use 20 kilonewtons per cubic metre;

z = Depth below top of wall, metres; and

q = Uniform surcharge at ground surface to account for traffic and equipment (not less than 15 kilopascals), plus any surcharge due to adjacent foundation loads.

If a water-tight structure will be provided, then the water pressures will need to be considered for that portion below the groundwater level. Further input would need to be provided.

These lateral earth pressures would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). For preliminary design, the total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(z) = K_o\,\gamma\,z + (K_{AE} - K_o)\,\gamma\,(H\text{-}z)$$

Where: $\sigma_h(d)$ = Lateral earth pressure at depth z, kilopascals;

K_o = At-rest earth pressure coefficient, use 0.5;

K_{AE} = Seismic earth pressure coefficient, use 0.8;

γ = Unit weight of backfill soil, use 20 kilonewtons per cubic metre;

z = Depth below the top of the wall, metres; and

H = Total height of the wall, metres.

Increased hydrodynamic groundwater pressures would also need to be considered if the structure is water-tight and extends below the groundwater level. However, more sophisticated analyses may need to be carried out at the detailed design stage.

It should be noted that all of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Ultimate Limit States design purposes.

Lateral seismic forces on the adjacent buildings could potentially add additional seismic earth pressures to this structure. Additional input on this issue can be provided, if required.

It has been assumed that the underground parking levels will be maintained at minimum temperatures but will not be permitted to freeze. If these areas are to be unheated, additional guidelines for the design of the basements walls will need to be provided.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible materials beyond the wall backfill. To reduce the severity of this differential heaving, the backfill adjacent to the wall should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 metres below finished exterior grade level at a slope of 3H:1V, or flatter, away from the wall. The granular fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

The pavement or hard surfacing in these areas could be expected to perform better in the long term if the granular backfill against the foundation walls is drained by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in geotextile, which leads by gravity drainage to a positive outlet.

The passive resistance offered by the foundation wall backfill soils (and those retained by the shoring) could also be considered in evaluating the lateral resistance applied to the structure. The magnitude of that lateral resistance could depend, in part, on the backfill materials and backfill conditions adjacent to the foundation walls. Movement of the backfill and wall is also required to mobilize the passive resistance. Further guidelines on the available resistance can be provided, if required.

It should be noted that, because the grading on the west side of the building will be higher than the east side, there will be a net easterly force (towards the river) due to the different earth pressures.

5.8.4 Corrosion and Cement Type

One sample of groundwater from borehole 1 was previously submitted to Accutest Laboratories Ltd. for chemical analysis related to potential corrosion of buried ferrous elements and sulphate attack on buried concrete elements. The results of the testing are provided in Appendix C.

The chemical analysis shows that the sulphate concentration in the groundwater sample is about 343 milligrams per litre. According to CSA-A23.1-09, Table 3, the results corresponds to a moderate degree of exposure to sulphate attack. It is therefore recommended that concrete made with sulphate resistant Portland cement should be used for substructures (exposure class S-3). The results also indicate a potential for corrosion of exposed ferrous metal, which may be due to high chloride concentration.

5.9 Riverbank Slope

As previously described, the east side of the site consists of the slope of the Rideau River. The slope is approximately 12 metres high and inclined at just slightly flatter than 1H:1V. The stability of the existing slope was previously assessed by Golder Associates and the slope stabilization guidelines for developing the site are provided in a separate report titled "Slope Stability Assessment, Proposed Development Site, 101 Wurtemburg Street, Ottawa, Ontario" (Report No. 10-1121-0003) dated July 2010.

That report recommended that the slope be reconstructed using reinforcing (i.e., using an MSE wall system). Golder Associates should be retained to review the design of that slope system, to confirm its compatibility with the building foundation design.

6.0 CONSTRUCTION CONSIDERATIONS

It is recommended that the final shoring design be reviewed and accepted by a geotechnical engineer prior to construction and that periodic inspection of the shoring installation procedures be carried out to ensure compatibility with the building design.

If the proposed building is to be supported by a pile foundation, piling operations should be inspected on a full time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

Inspection of the prepared subgrade for floor slabs and control on the placing and compaction of the granular fill for floor slabs should be carried out to document that the materials used conform to specifications from both a grading and compaction point of view.

If a raft foundation is considered for the proposed building, all raft foundation areas should be inspected by geotechnical personnel to ensure that a suitable subgrade has been reached and properly prepared. In order to avoid disturbance of the sensitive clay subgrade, it is recommended that the clay subgrade be protected by a mud slab of lean concrete as soon as each portion of the excavation has been completed and inspected.

Should construction be carried out during freezing temperatures, freezing of the soil behind the temporary support walls could place additional stress on the walls/rakers. Frost penetration could also affect existing adjacent foundations negatively. Accordingly, the soils behind the support walls should be protected from freezing temperatures by methods such as a combination of heaters and tarpaulins.

Prior to construction, it is recommended that a pre-construction survey of existing structures adjacent to the site be carried out to document their condition and the presence of any existing defects. It may also be prudent to install glass ‘tell-tales’ across any existing cracks in adjacent buildings and to monitor the ‘tell-tales’ frequently during construction in order to provide a warning of any movement of previously distressed areas.

Vibration monitoring should also be carried out at the adjacent building, particularly during the installation of the shoring or piled foundations.

A Permit-to-Take-Water (PTTW) from the Ministry of Environment (MOE) should be obtained for this project, particularly if excavation to the sand layer is planned. A hydrogeolgic assessment of the dewatering requirements and impacts would be required.

If a raft foundation is to be considered, additional investigation will be required to confirm the uniformity of the subsurface conditions (and the current groundwater level). If a piled foundation is to be used, confirming the depth to bedrock (and therefore the pile length) should be considered, to assist with project budgeting. Geophysical testing may also be required so that the seismic Site Class evaluation can be confirmed/refined.

We trust this report contains sufficient information for your present purposes. Should you have any questions concerning this report, please do not hesitate to contact the undersigned.

Yours truly,

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
M. I. CUNNINGHAM
PROVINCE OF ONTARIO

Christine Ko, P.Eng.
Geotechnical Engineer

Mike Cunningham, P.Eng.
Associate

CK/MIC/tm
n:\active\2010\1121 - geotechnical\10-1121-0003 wurtemburg st\10-1121-0003 rpt-002 - prelim geotech.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 cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

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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 cannot 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.

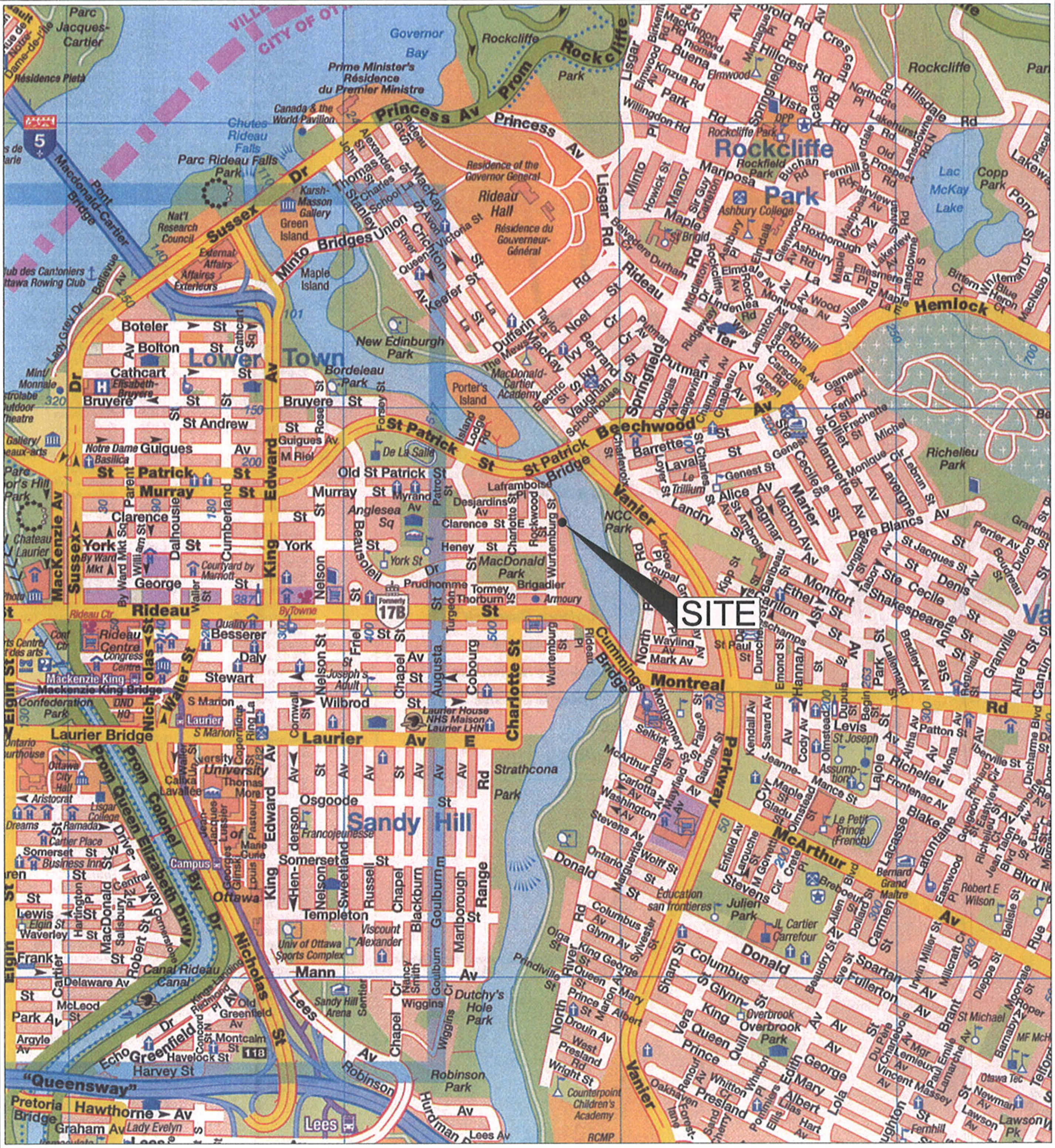
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater 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.

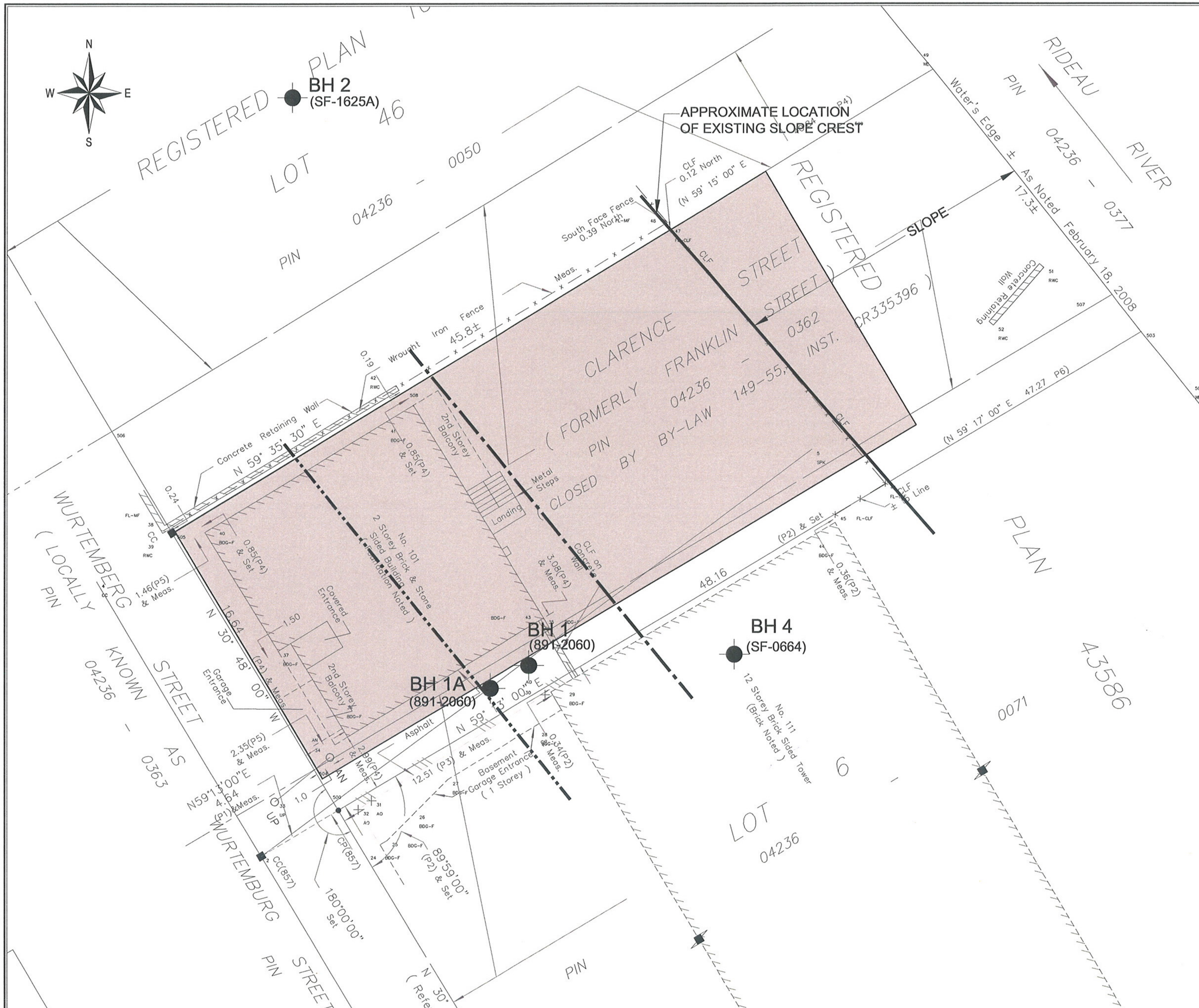


NOTE

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 CK	GEOTECHNICAL INVESTIGATION WURTEMBURG STREET - OTTAWA, ONTARIO	FIGURE 1
PROJECT No. 10-1121-0003 REV.	REVIEW MK		

PLOT DATE: March 30, 2011
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

GEOTECHNICAL INVESTIGATION
WURTEMBERG STREET
OTTAWA, ONTARIO

TITLE

SITE PLAN

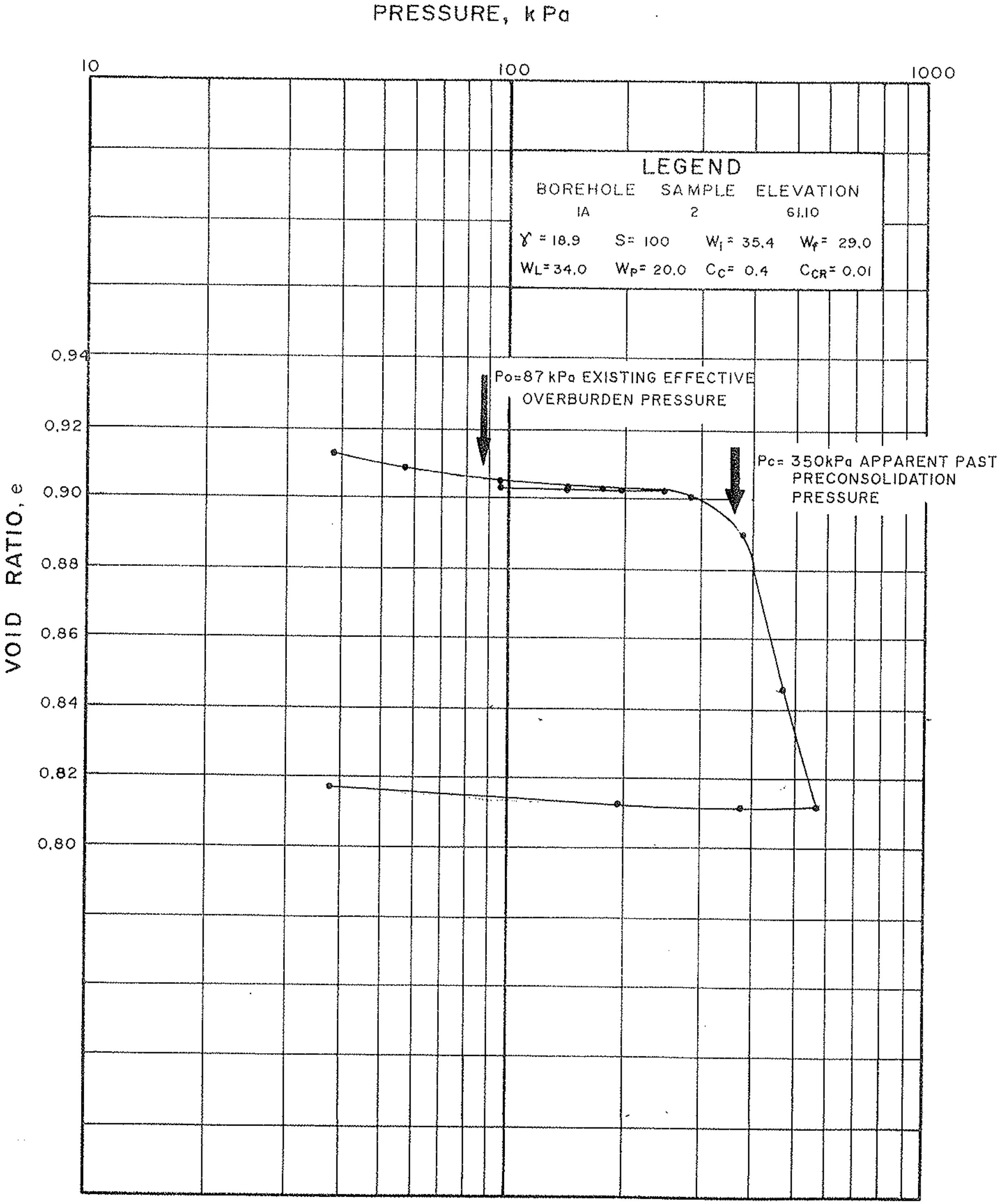
Golder Associates
Ottawa, Ontario, Canada

PROJECT No.	10-1121-0003		FILE No.	1011210003-02.dwg
DESIGN			SCALE 1:200	REV.
CAD	J.M.	14 JAN. 10	DRAWING No.	
CHECK	CK	30/3/11		2
REVIEW	MLC	30/3/11		

PLOT DATE: March 30, 2011
FILENAME: N:\Active\2010\1121 - Geotechnical\10-1121-0003 Wurtemburg St\ACAD\1011210003-02.dwg

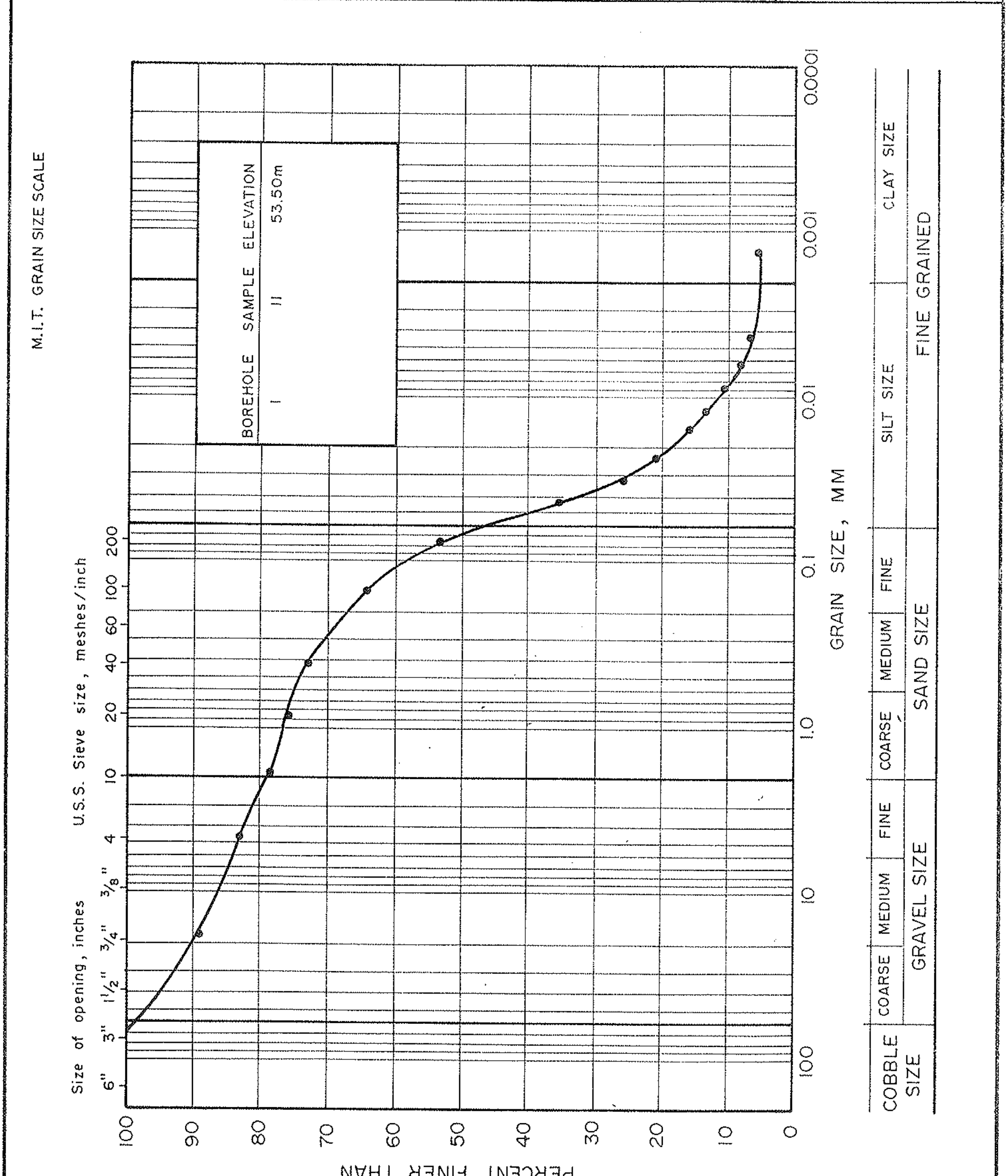
VOID RATIO - PRESSURE CURVES
CONSOLIDATION TEST

FIGURE 3



GRAIN SIZE DISTRIBUTION
GLACIAL TILL

FIGURE 4



Golder Associates

APPENDIX A

List of Abbreviations and Symbols
Record of Borehole Sheets - Previous Investigation by Golder 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_l	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_l	liquid limit
w_p	plastic limit
I_p	plasticity Index=(w_l-w_p)
w_s	shrinkage limit
I_L	liquidity index=(w-w_p)/I_p
I_c	consistency index=(w_l-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. $\tau = c'\sigma' \tan \phi'$
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

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

Boring method: Power Auger, 200mm Diam (Hollow Stem)

Description	Elev. / Depth (m)
Ground Surface	67.41
Grey crushed stone (FILL)	0.06
Loose to very loose brown fine to medium sand to sand and gravel (FILL)	0.48
Very stiff to stiff grey brown SILTY CLAY (Weathered Crust)	64.33 / 3.08
Very stiff to firm grey SILTY CLAY	62.38 / 5.03
Compact brown to grey SILTY fine SAND	56.10 / 11.31
Compact to loose dark grey silty sand to sandy silt, some clay, gravel and cobbles, occasional boulder and fine sand layer (GLACIAL TILL)	54.12 / 13.29
Probably compact dark grey Glacial Till	48.51 / 18.90
Probably dense dark grey Glacial Till	42.42 / 24.99
End of Hole Refusal to Auger	39.89 / 27.52

0.00-0.06 ASPHALT

Number	Type	Blows/0.3m
1	50 DO	4
2	50 DO	6
3	50 DO	3
4	50 DO	2
5	50 DO	WH
6	50 DO	WH
7	50 DO	WH
8	50 DO	WH
9	50 DO	1
10	50 DO	24
11	50 DO	14
12	50 DO	7
13	50 DO	5
14	50 DO	15

Additional lab testing (sample 11): /MH

15 -⊕- 5 PERCENT AXIAL STRAIN AT FAILURE 0 10

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.- ○) 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									
	Power Auger 200mm Diam (Hollow Stem)	For detailed soil descriptions refer to RECORD OF BOREHOLE 1									Seal Backfill
6					1	75 DO	PH				
					2		10	⊕ +		/C	Caved
		End of Hole									W.L. in Standpipe at elev. 61.37 March 7, 1989

-◇- PERCENT AXIAL STRAIN AT FAILURE

DEPTH SCALE

1 : 150

Golder Associates

LOGGED S.Leighton

CHECKED [illegible]

APPENDIX B

Record of Borehole Sheets - Previous Investigations by McRostie & Associates

McROSTIE SETO GENEST

& ASSOCIATES LTD. & ASSOCIÉS LTÉE

CONSULTING ENGINEERS – INGÉNIEURS CONSEILS

OTTAWA CANADA

SOIL PROFILE & TEST SUMMARIES

PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

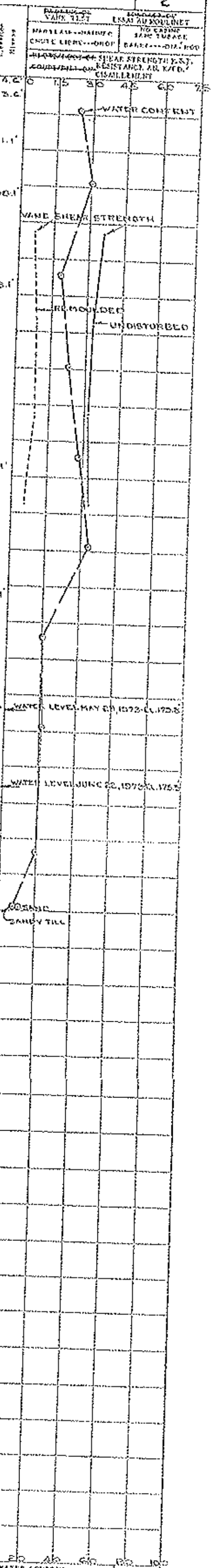
WURTEMBURG ST. No. 2

ELEVATION OF GROUND SURFACE (ZERO DEPTH) / NIVEAU DU SOL (PROFONDEUR ZÉRO) 214.6' DATE MAY 12, 1972

NOTES B.M. (EL. 222.02) GEODETIC TABLET No. 3662, NORTHWEST CORNER OF RIDEAU & CHARLOTTE

HOLE / FORAGE No. 2

Cohesive Strength K.S.F. / Résistance à la Compression K/P.C.	Vane Shear Test K.S.F. / Essai Scissomètre K/P.C.	Standard Penetration Blows/Ft.	Sample No.	DESCRIPTION OF SOIL / DU SOL	Depth in Feet / Profondeur – Pied	Elevation / Niveau
				Ground Surface – Niveau du Sol		
				FILL – TOPSOIL, SAND, GRAVEL & CINDERS	0'	214.6'
		3	2-1	FILL – FISSURED BROWNISH GRAY CLAY WITH A LITTLE TOPSOIL & SAND	1.0'	213.6'
	4.6, 5.2, 4.0	3 SHOE	2-2	VERY STIFF FISSURED BROWNISH GRAY CLAY WITH SOME SILT	3.5'	211.1'
	2.0, 2.0, 2.2	5	2-3	STIFF BROWNISH GRAY CLAY WITH SOME SILT & A LITTLE VERY FINE SAND	6.5'	208.1'
	3.0, 3.0, 2.8	3 SHOE	2-4	STIFF BROWNISH GRAY CLAY WITH SOME SILT & A TRACE OF VERY FINE SAND	11.5'	203.1'
	2.5, 2.5, 2.0	5	2-5			
	4.4, 4.6, 4.0	6	2-6	VERY STIFF SILTY BROWNISH GRAY CLAY WITH A FEW FISSURES	21.5'	192.1'
		40	2-7	MEDIUM DENSE TO DENSE VERY FINE SAND WITH A LITTLE SILT & WITH SOME 1/2" TO 1" SILT LAYERS & WITH A 1/2" LAYER OF MEDIUM & COARSE SAND & WITH A FEW BOULDERS	28.5'	186.1'
		27	2-8		36.0'	178.6'
HARD DRILLING AT EL.176.5'						
		40	2-9			
		31 SHOE	2-10	BOULDERS IN DENSE TO VERY DENSE SANDY TILL	45.0'	169.6'
DRILLING AHEAD OF CASING WAS REQUIRED TO ADVANCE BX CASING TO 62'					55.0'	159.6'
DRILLING WITH SOLID BIT AHEAD OF AX CASING TO 68.7'					62.0'	152.6'
				WEATHERED SHALE	67.9'	146.7'
				SHALE – CORE RECOVERY 33%	68.7'	145.9'
				SHALE – CORE RECOVERY 98%	73.4'	141.2'
				BOTTOM OF HOLE	78.4'	136.2'



R.F. Remoulded – Remanié; C.R. Core Recovery – Carotte Récupérée

Plate / Planche No. 3

ASSOCIATES LTD.
CONSULTING ENGINEERS
OTTAWA CANADA

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

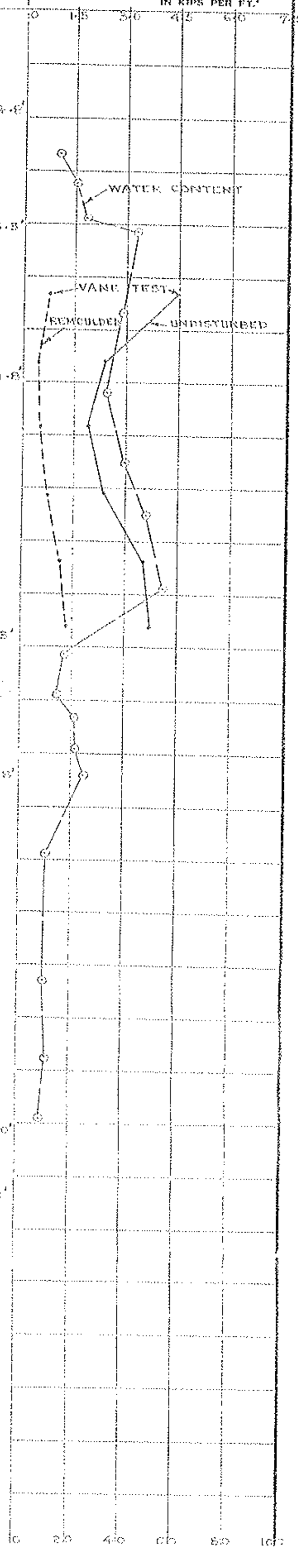
UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION
				GROUND SURFACE	0'	224.8'
		6	4-1	LOOSE FINE SAND WITH A LITTLE GRAVEL		
		4 for 6"	4-2			
					6.5'	
		9 for 6"	4-3A	MEDIUM DENSE TO 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, 5.6 3.5, 3.8, 4.6 R-0.8		4-5			
	5.4, 3.2, 3.4 3.5, 3.0, 3.0 3.4, 3.5, 3.0 2.6, 2.8, 2.7 R-0.2		4-6		20'	204.8'
	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	STIFF SILTY GRAY CLAY		
	3.8, 4.0, 3.2 5.6, 4.0, 4.2 3.5, 3.8, 2.7 R-0.0		4-8		30'	
	4.6, 4.6, 4.8 4.6, 4.6, 4.6 4.0, 4.6, 4.8 4.0, 4.2, 4.0 R-0.0		4-9	STIFF TO VERY STIFF SILTY GRAY CLAY		
					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		
		45 11 for 6"	4-13		48.0'	
				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			
		26 5 for 6"	4-19		76'	
				LOOSE SILTY TILL	76.8'	148.0'
				BOULDERS		
					81.6'	143.2'
				BOTTOM OF HOLE		

PROBING OR VANE TEST

.........LB. HAMMER ....INCH DROP

NO CASING ....INCH DIA. ROD

BLOWS PER FOOT OR SHEAR STRENGTH IN KIPS PER FT.²



APPENDIX C

Results of Chemical Analysis
Accutest Report No. A9-0396

LAB REPORT NO.: A9-0396

ACCUTEST LABORATORIES LTD.
146 Colonnade Rd., Suite 202, Nepean, Ontario K2E 7Y3 (613) 727-5692

REPORT OF ANALYSES

Client: Golder Associates

Attn: Mr. Randy Morey

Date: April 17, 1989

Project: 891-2060
Claridge/Apt INV/Ott

Parameter	Units	Sample BH 1 C 55	Sample	Sample	Sample	Sample
Fe	mg/L					
Mn	mg/L					
Hardness	mg/L $CaCO_3$					
Alkalinity	mg/L $CaCO_3$					
pH		6.83				
Conductivity	umhos /cm	1980-				
F	mg/L					
Na	mg/L					
$N-NO_3$	mg/L					
$N-NO_2$	mg/L					
$N-NH_3$	mg/L					
SO_4	mg/L	343				
CL	mg/L	113				
Phenols	mg/L					
Turbidity	NTU					
Colour	Pt/Co Units					
Ca	mg/L					
Mg	mg/L					
Tannin & Lignin	mg/L					
Total Nitrogen	mg/L					

ANALYST: [signature]

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. 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 who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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