



**PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED 12-STOREY APARTMENT BUILDING
1 GRIFFITH STREET
WELLAND, ONTARIO**

Submitted To:

**1435587 Ontario Inc.
Kingsway Investments Ltd. And Lenders
105 Main Street East, Suite #1510
Hamilton, Ontario
L8N 1G6**

Attention: Mr. Ray Khanna

Submitted by:

**AMEC Environment & Infrastructure,
A division of AMEC Americas Limited
3300 Merrittville Highway, Unit 5
Thorold, Ontario
Canada, L2V 4Y6**

**July 2011
TG113018**

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1. AUTHORIZATION

Authorization to proceed with this investigation was received from Mr. Ray Khanna of Kingsway Investments Ltd. And Lenders, in the form of a signed Professional Service Agreement contract dated May 24, 2011.

2. INTRODUCTION

AMEC Environment & Infrastructure, a division of AMEC Americas Limited ('AMEC'), has been retained to carry out a geotechnical investigation for the proposed development located at 1 Griffith Street in Welland, Ontario. The site is shown on the Site Location Plan appended as Figure 1.

The investigation obtained subsurface information relating to the project site by means of sampled boreholes. The data gathered from this investigation is used to perform analysis and provide geotechnical engineering recommendations.

2.1 Site and Project Description

The subject property is located on the west end of Griffith Street on the east side of the Welland Canal. It is understood that the two existing buildings on the property will be demolished prior to construction of the new building. AMEC understands that the proposed development will consist of a 12-storey apartment building. For the purposes of this report, it is assumed that the proposed construction will include a full basement or a two-level underground garage. The finished floor elevations and the details of the foundation loads are unknown at this stage.

2.2 Terms of Reference

The findings of the investigation, together with AMEC's comments and recommendations, are presented in this report. The anticipated construction conditions are also discussed but only to the extent that they may influence the design decisions. Any construction methods discussed express AMEC's opinions only and are not intended to direct contractors on how to carry out the construction. Contractors should also be aware that the data and the interpretation presented in this report may not be sufficient to assess all the factors that may have an effect on construction.

This report was prepared with the assumption that the design will be in accordance with applicable standards and codes, regulations of authorities having jurisdiction, and good engineering practices.

Further, the recommendations and opinions expressed in this report are only applicable to the proposed project as described above.

An ongoing liaison with AMEC must be maintained during both the design and construction phases of the project to ensure that the recommendations in this report have been interpreted and implemented correctly. Also, if any further clarification and/or elaboration are needed concerning the geotechnical aspects of this project, AMEC should be contacted immediately.

2.3 Background and Geological Setting

Preliminary Map P796, Geological Series, Quaternary Geology of the Welland Area, Southern Ontario indicates that the subject site lies partly within an area of glaciolacustrine clay, silt, and minor sand and partly within Cultural features (Welland Canal).

3. FIELD AND LABORATORY INVESTIGATION

The investigation was carried out to obtain information about the soils at this site by means of sampled boreholes and laboratory testing.

3.1 Field Work

A total of 3 boreholes were drilled on June 7th and 10th, 2011. Boreholes 1 and 3 were drilled to a depth of 12.7 m and Borehole 2 to 30.4 m. The approximate locations of the boreholes are indicated on the Borehole Location Plan on Figure 2 in Appendix A.

The boreholes were advanced using a track-mounted drilling rig, designed for soil sampling purposes. Samples were obtained through the overburden soil by driving a standard split spoon-sampling device. In-situ testing was carried out in these boreholes to determine the relative density/consistency of the soils, in accordance with the requirements of the Standard Penetration Test (SPT) according to ASTM D1586.

Dynamic Cone Penetration Testing was conducted from approximately 25.3 m below existing grade to 30.4 m in Borehole 2. The number of blows required to drive the cone, using the same hammer and drop height as the Standard Penetration Test, for every 0.3 m of penetration was recorded.

Prior to carrying out any fieldwork, the appropriate utility companies were notified to carry out underground service clearances at the borehole locations. As well, a private utility locator was hired to locate privately-owned utilities.

Upon recovery, all of the samples retrieved were examined and placed in appropriate containers and returned to AMEC's laboratory for further examination and water content testing. AMEC personnel supervised the fieldwork, performed preliminary sample identification and prepared field borehole logs. The ground surface elevations at the borehole locations were surveyed and referenced to a temporary benchmark (TBM) described as "the top of stem valve of a fire hydrant located on south side of Regent Street, across from the south east corner of the site," as shown on the appended Figure 2. This TBM was assigned a local elevation of 100.0 m.

3.2 Physical Laboratory Work

The soil samples were returned to our laboratory for visual examination and classification. Water content tests were carried out on each of the recovered soil drill rig samples.

All soil samples will be stored for 3 months after completion of this report. The samples will then

be discarded unless AMEC is instructed otherwise.

4. SUBSURFACE CONDITIONS

Descriptions of the soil conditions encountered at each location during this investigation can be found on the Logs of Boreholes 1 through 3 in Appendix A. It should be noted that the soil conditions are based on soils encountered at the borehole locations, which may vary between and beyond these locations.

Pavement Structure

Approximately 30 mm of asphalt over 330 mm of granular fill were encountered at surface in Borehole 3.

Fill

Brown silty clay to clayey silt fill was encountered at surface in Boreholes 1 and 2, and below the pavement structure in Borehole 3. This layer extended to a depth of 2.9 m in Boreholes 1 and 3, and 5.6 m in Borehole 2.

The fill materials contained topsoil, fine to medium gravel and sand, coarse gravel, cobbles, asphalt pieces, grey fissures, black staining, and traces of silt seams. SPT testing gave 'N' values ranging from 0 to 11 blows for 0.3 m, indicating very soft to stiff consistencies. Moisture contents were between 8 and 28 %.

Silty Clay to Clayey Silt

Underlying the fill materials, brown to grey/reddish brown silty clay to clayey silt was encountered in all of the boreholes. The layer extended to at least the maximum depths investigated in the boreholes.

The silty clay to clayey silt deposits contained traces of fine to medium gravel, coarse gravel pieces, grey fissures, and silt seams. SPT tests gave 'N' values ranging from 18 to 0 blows (with increasing depth) for 0.3 m, indicating very stiff to very soft consistencies. Natural water contents ranged between 19 and 42 %.

Field Vane tests were carried out in Borehole 2 at various depths through the weaker soils. The results are summarized below:

BH	Depth	Undrained Shear Strength	Sensitivity
2	11.6 m	90 kPa	1.5
2	16.2 m	70 kPa	1.8

Cone Penetration Refusal / Assumed Bedrock

Borehole 2 was terminated due to cone penetration refusal on assumed bedrock at a depth of 30.4 m. Preliminary Map No. P. 989, Paleozoic Geology of the Welland-Fort Erie Area, Southern

Ontario, indicates that the bedrock consists of argillaceous dolostone of the Salina Formation.

Groundwater

Upon completion of the drilling, Boreholes 1 and 2 remained dry and open. Water was measured in Borehole 3 at a depth of 6.7 m. It should be noted that the groundwater conditions may vary subject to weather and seasonal fluctuations.

5. DISCUSSION AND RECOMMENDATIONS

It is understood that a proposed 12-storey apartment building is proposed. For the purposes of this report, it is assumed that the proposed construction will include a full basement or a two level underground garage. The finished floor elevations and the details of the foundation loads are unknown at this stage, therefore, the recommendations should be considered preliminary only. Once these details have been established AMEC should be retained to review them and make any necessary comments. An additional detailed geotechnical investigation will be required to determine the design parameters and to confirm the ability of the subsurface soils to bear the design loads.

The results of the site-specific investigation indicate that the native soils consist of very stiff to soft (with increasing depth) silty clay to clayey silt underlain by assumed bedrock at 30.4 m. Please note that the fills depth varies from 2.9 to 5.6 m below existing grade.

5.1 Foundation Options

Based on the subsurface soil conditions, we have considered the following foundation options:

- Conventional spread and/or strip footings
- Structural slab or mat foundation
- Deep foundation alternatives

5.1.1 Conventional Spread and Strip Footings

All new foundations must be placed on native soils, below any fills, surficially wet, soft or loosened soils. Based on the subsurface information obtained from the investigation, the native silty clay to clayey silt are anticipated between 2.9 and 5.6 m below existing grade

Spread and/or strip footings (to a maximum of 1.2 m in width) founded on native silty clay to clayey silt soils may be designed using a factored ultimate limit state (ULS) bearing value of 200 kPa and service limit state (SLS) bearing value of 125 kPa. Based on these bearing values, the founding soil is likely not suitable to support a 12-storey building but preliminary values are provided in the event that changes are considered. The following table shows the maximum and minimum founding elevations at each borehole to support these bearing values.

Borehole Number	Anticipated Founding Elevation (m)
1	93.6 – 90.2
2	96.6 - 91.5
3	96.4 – 91.3

If strip footings are proposed in the vicinity of Borehole 1, it may be practical to dig 2 m wide trenches and fill them concrete from approximately 5.6 m deep (or as directed by the engineer in the field) to 3.0 m deep and place the basement footing on top of them.

Depending upon the conditions encountered during construction, areas of localized deepening of footings may be required. It may be more economical to dig to an appropriate foundation bearing level and use a thicker footing (or mass fill beneath the standard footing with a lean concrete) than to step down the footing and foundation wall.

Ideally, all old foundations should be removed as part of the demolition, and the area remediated /backfilled with appropriately compacted material prior to beginning new construction. Otherwise, any new foundations placed in close proximity to foundations left in place from the previous structure must extend to at least the founding elevation of the old foundations. This is to ensure that no new foundations bear on backfill associated with the other / previous foundations.

Any exterior and interior footings in unheated areas should be covered with a minimum of 1.2 m of overburden soil (or equivalent insulation) below the exterior finished grade in order to provide for frost protection.

During construction, a geotechnical engineer should inspect and verify that these footings are founded on the native soil. The following table shows the estimated founding elevations at each borehole.

5.1.2 Structural slab or Mat Foundation

The building may be founded on a mat foundation consisting of reinforced concrete at an elevation of between 93.6 and 96.6 m using a factored ultimate limit state (ULS) bearing value of 130 to 150 kPa and service limit state (SLS) bearing value of 75 to 100 kPa. The coefficient of the subgrade reaction recommended for a mat foundation at this site is 5,000 kN/m³. The total and differential settlements are anticipated to be less than 25 mm and 19 mm, respectively. The mat foundation will be founded on the firm to stiff clays encountered on site.

Immediately after excavation, it is recommended that a minimum of 100 mm layer of concrete mud slab (specified strength of 10 MPa or higher) should be poured to protect the integrity of the founding surface from being disturbed.

5.1.3 Deep Foundations

Dynamic cone penetration testing was conducted at Borehole 2 from a depth of 25.3 m to approximately 30.4 m below existing grade. Should the Client consider driven piles as an option, it is recommended that an additional geotechnical investigation be carried out to determine the bearing stratum for piles and obtain rock core samples (if necessary) to determine the competency of the bedrock for design load of the structures.

Driven Piles

Preliminary calculations indicate that piles (300 mm or smaller) driven to refusal to an approximate depth of 30.4 metres could support up to 800 kN, depending on pile type and cross-section. H-piles are considered appropriate. Closed-end pipe piles (say, 200 to 250 mm diameter) are also considered suitable and are often less expensive. The pipe piles have the advantage that they can be easily inspected for plumb and damage prior to filling with concrete.

Driven piles have the definite advantage that there would be no concerns with groundwater. No obvious obstructions to pile driving operations were encountered in any of the boreholes. However, the vibrations generated from the driving operations, may cause damage to the existing addition and possibly other surrounding structures depending on their foundation design and bearing soil, particularly if they rest on fill. This should be taken into consideration.

Should the Client consider this option, AMEC would be pleased to provide further assistance in the detailed design of these pile options if requested.

5.1.4 Earthquake Consideration

The site may be classified as Site Class E (OBC 2006, Table 4.1.8.4.A).

5.2 Slope Stability

It must be noted that the property lies at the edge of the man-made Welland Canal which entail relative steep cuts through the weak silty clay to clayey silt material. It is therefore emphasized that a detailed slope stability assessment must be an integral part of the detailed geotechnical investigation.

5.3 Excavations and Groundwater Control

No significant problems are anticipated during excavation through the upper overburden soils.

All excavations must comply with the Occupational Health & Safety Act and Regulations for Construction Projects (OHSARCP). The side slopes of any excavations through the overburden, deeper than 1.2 metres, must be sloped as outlined in the Act. Based on the criteria in the Act, all fills must be considered as Type 3 soils, and possibly degrading to Type 4 if saturated. The very stiff to firm native silty clay to clayey silt is considered to be Type 2 soil.

No significant groundwater problems are anticipated during excavations for the foundations based on the conditions encountered in the boreholes. If the groundwater conditions at the time of the construction are significantly different than described in this report, the soils classifications

could change and the excavation requirements would need to be revised. It is anticipated that any minor seepage can be controlled by properly filtered sumps.

In general, temporary excavation side slopes of 1 vertical to 1 horizontal are expected to remain stable through the overburden, although some flattening of excavation sides may be required if saturated fills are encountered. All surface drainage should be directed away from any open excavations and trenches.

5.4 Perimeter Drainage

Perimeter drains and damp-proofing should be placed around any depressed floor areas (e.g. basement, garage, underground mechanical and electrical rooms). It is unnecessary to have perimeter drains and damp-proofing around the exterior footings if the outside finish grade is lower than the finish floor elevation (by say 200 mm).

The landscaping and parking areas should be designed and graded in such a way that surface water will be drained away from the proposed building.

5.5 Lateral Earth Pressure

Based on a well-compacted and drained backfill soil, the foundation walls should be designed to resist the horizontal pressure acting on the wall. The horizontal pressure 'p', at a depth 'h', may be estimated using the following:

$$p = K (Yh + q)$$

where

K = lateral earth pressure coefficient

= 1.0 for clay

= 0.5 for granular backfill against a wall restrained at both top and bottom

= 0.3 for granular backfill against a wall restrained at bottom only

Y = unit weight

= 22 kN/m³ for free-draining crushed limestone

= 20 kN/m³ for clay

h = depth below final grade (m)

q = surcharge load (kPa), if present, plus 15 kPa due to heavy compaction equipment

5.6 Soil Reuse and General Backfill

The soils removed by excavation are expected to consist of asphalt, granular fills, topsoil, and silty clay to clayey silt (fill and native).

The asphalt and topsoil are not considered suitable for reuse, unless they are used in landscaped areas.

The granular fills are considered suitable for reuse as mass fill, if they can be stockpiled separately.

The silty clay to clayey silt (fill and native) is considered suitable for reuse as fill, provided it can be properly compact. Generally, heavy compaction equipment must be used in order to break-up and thoroughly re-compact clayey soils to prevent post-construction settlement. Poorly compacted backfill will settle and will be reflected in any overlying rigid surfaces such as pavement, concrete curb & gutter, and sidewalks. Also, it is generally difficult to adequately place and compact cohesive soils in small, confined areas such as trenches or foundation excavations. Therefore, imported granular materials should be used in these locations.

The soils must also be at suitable moisture contents for re-compaction. Typically, the optimum moisture content for clay soils for recompaction is about 20%. The moisture contents recorded for the Silty Clay to Clayey Silt ranged from 8 to 42%. Therefore, some reconditioning may be required if the soils are exceedingly wet.

Backfill materials around the building perimeter should be in compliance with all relevant standards and codes. The backfill materials should be placed evenly in 300 mm loose lifts and compacted to at least 95% of the materials' SPMDD.

6. CLOSURE

The Report Limitations on the following page are an integral part of this report. The geotechnical aspects of the final design drawings and specifications should be reviewed by this office to confirm that the intent of this report has been met. During construction, sufficient subgrade, inspections, excavation monitoring and materials testing should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes and to monitor conformance to the pertinent project specifications.

We trust that this report is complete within our present terms of reference. If you have any questions, please do not hesitate to contact our office.

Yours truly,

AMEC ENVIRONMENT & INFRASTRUCTURE

Prepared By:

Reviewed By:



Rod de Castro, P.Eng.
Geotechnical Engineer



Michael Patterson, P.Eng.
Senior Geotechnical Engineer

Cc: 3 copies – Client
1 copy – File

REPORT LIMITATIONS

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Geotechnical Engineer be retained during the construction to confirm that the subsurface conditions across the site do not deviate materially from those encountered in the testholes.

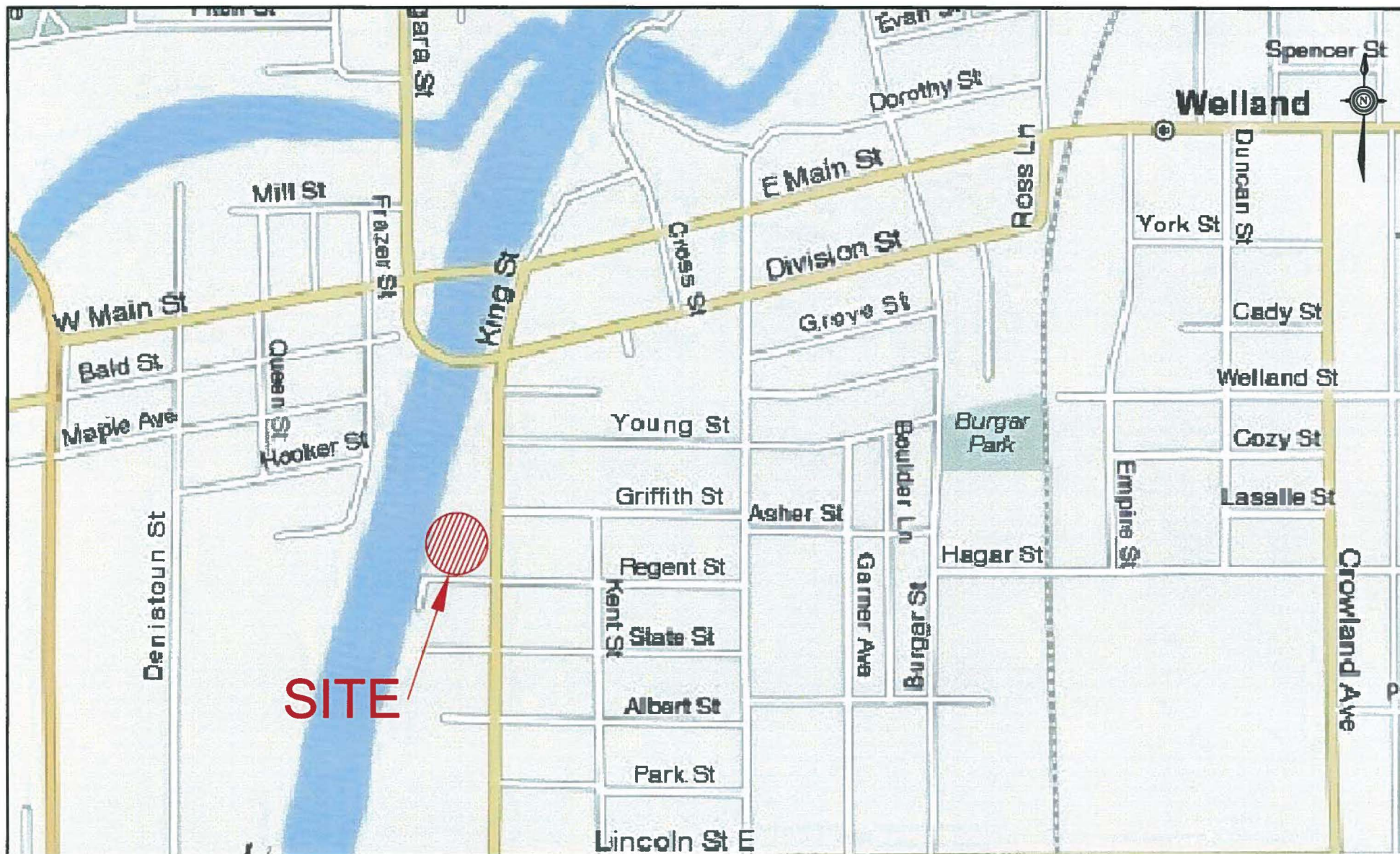
The design recommendations given in this report are applicable only to the project described in the text, and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report relating to potential construction problems and possible methods of construction are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

The benchmark and elevations mentioned in this report were obtained strictly for use by this office in the geotechnical design of the project. They should not be used by any other party for any other purpose.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AMEC Americas Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

APPENDIX A



Reference: Base Map from <http://www.mapquest.com>

For Illustration Purposes Only. NOTE: All Locations Approximate.

CLIENT:

KINGSWAY INVESTMENTS LTD. AND LENDERS

DWN BY: DG

CHK'D BY: RD

DATUM: NAD 83

PROJECTION: UTM Zone 17

SCALE: UNKNOWN

PROJECT:

GEOTECHNICAL INVESTIGATION
1 GRIFFITH STREET,
WELLAND, ONTARIO

TITLE:

SITE LOCATION PLAN

REV. NO.: A

DATE: JUNE 2011

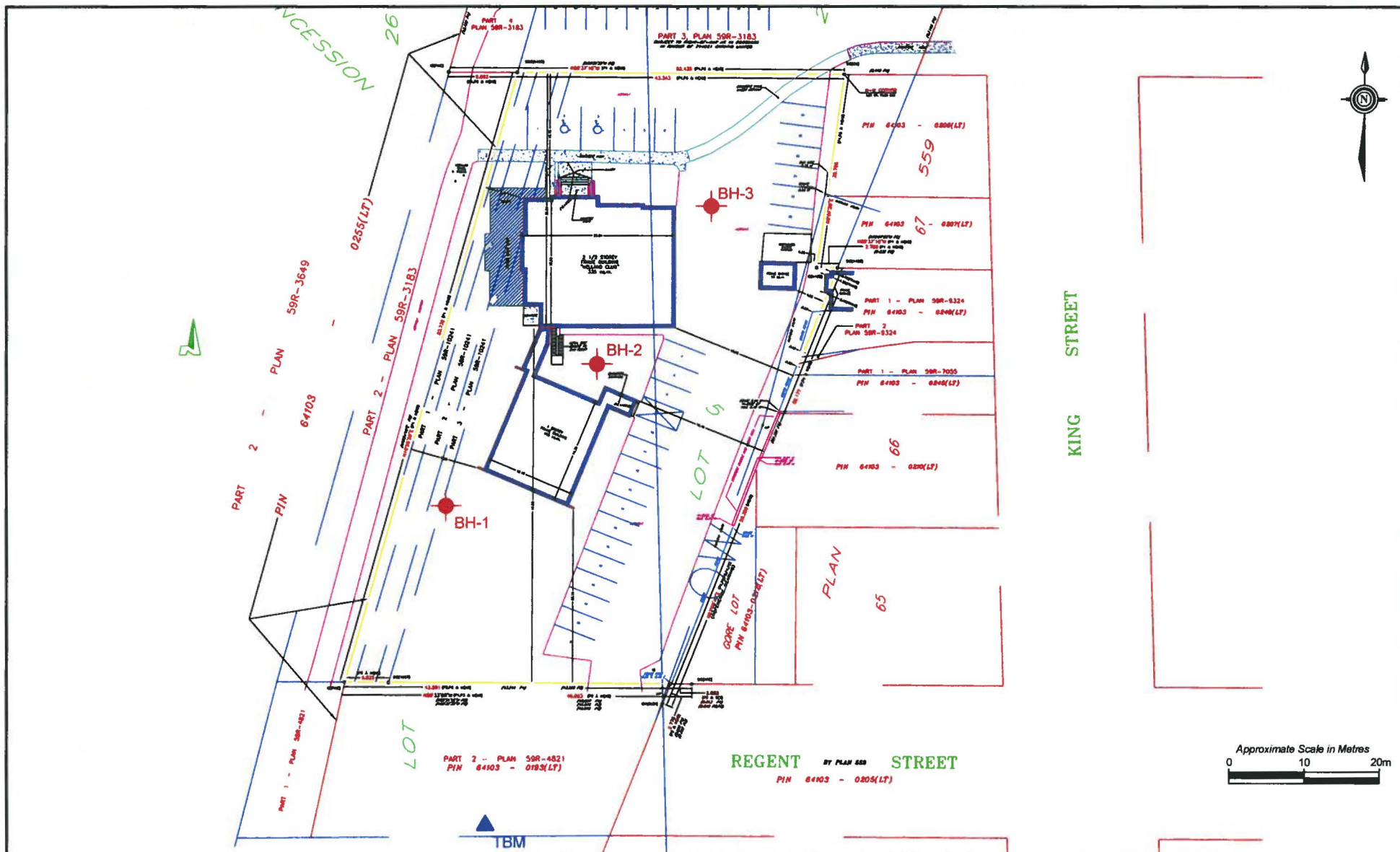
PROJECT NO.: TG113018

FIGURE No.: 1

AMEC Earth & Environmental

a division of AMEC Americas Limited
3300 Merrittville Hwy, Unit 5
Thorold, Ontario
L2V 4Y6





Reference: Base Map provided by Client.

For Illustration Purposes Only. NOTE: All Locations Approximate.

CLIENT: KINGSWAY INVESTMENTS LTD. AND LENDERS	LEGEND: ● BH-1 BOREHOLE LOCATION ▲ TBM TEMPORARY BENCHMARK	DWN BY: DG CHK'D BY: RD DATUM: NAD 83 PROJECTION: UTM Zone 17 SCALE: AS SHOWN	PROJECT: GEOTECHNICAL INVESTIGATION 1 GRIFFITH STREET, WELLAND, ONTARIO	REV. NO.: A DATE: JUNE 2011 PROJECT NO.: TG113018
AMEC Earth & Environmental a division of AMEC Americas Limited 3300 Merrittville Hwy, Unit 5 Thorold, Ontario L2V 4Y6	amec		TITLE: BOREHOLE LOCATION PLAN	FIGURE No.: 2

GENERAL REPORT NOTES

DEFINITIONS OF PENETRATION RESISTANCE

Standard penetration resistance 'N': -- The number of blows required to advance a standard split spoon sampler 30 cm into the subsoil, driven by means of a 63.5 kg hammer falling freely a distance of 70 cm.

Dynamic penetration resistance: -- The number of blows required to advance a 50 mm, 60 degree cone, fitted to the end of drill rods, 30 cm into the subsoil, the driving energy being 475 Joules per blow.

SAMPLE TYPE ABBREVIATIONS USED IN BOREHOLE LOGS

S.S.	Split spoon	T.W.	Thinwall open	R.C.	Rock core
A.S.	Auger sample	T.P.	Thinwall piston	W.S.	Washed sample
	P.H.	Sample pushed hydraulically	P.M.	Sample pushed manually	

SOIL TEST SYMBOLS USED IN BOREHOLE LOGS

□	Standard penetration resistance	▼	Laboratory vane	○	Unconfined compression
■	Dynamic penetration resistance	▲	Field vane	●	Undrained triaxial
		X	Penetrometer	S	Sensitivity

CONVENTIONAL SOIL DESCRIPTIONS

COHESIVE (CLAYS ETC.)			GRANULAR (SANDS ETC.)	
<u>Consistency</u>	<u>'N' blows/30cm</u>	<u>c kPa</u>	<u>Denseness</u>	<u>'N' blows/30 cm</u>
Very Soft	0 - 2	0 - 12	Very Loose	0 - 4
Soft	2 - 4	12 - 25	Loose	4 - 10
Firm	4 - 8	25 - 50	Compact	10 - 30
Stiff	8 - 15	50 - 100	Dense	30 - 50
Very Stiff	15 - 30	100 - 200	Very Dense	>50
Hard	>30	> 200		

ABBREVIATIONS FOR MOISTURE CONDITIONS

sdtp - slightly drier than the plastic limit.
dtp - drier than the plastic limit.
apl - about the plastic limit.

swtp - slightly wetter than the plastic limit.
wtp - wetter than the plastic limit.
mwtp - much wetter than the plastic limit.

NOTE

The soil conditions, profiles, comments, conclusions and recommendations found in this report are based upon the samples recovered during the field work. Soils are heterogeneous materials and, consequently, variations (possibly extreme) may be encountered at site locations away from boreholes. During construction, competent, qualified inspection personnel should verify that no significant variations exist from the conditions described in this report.

1 OF 1

PROJECT GI - Proposed 12 Storey Apartment Building

LOCATION [As shown on Figure 2]

ORIGINATED BY NA

CLIENT Kingsway Investments Ltd. And Lenders

COMPILED BY NA

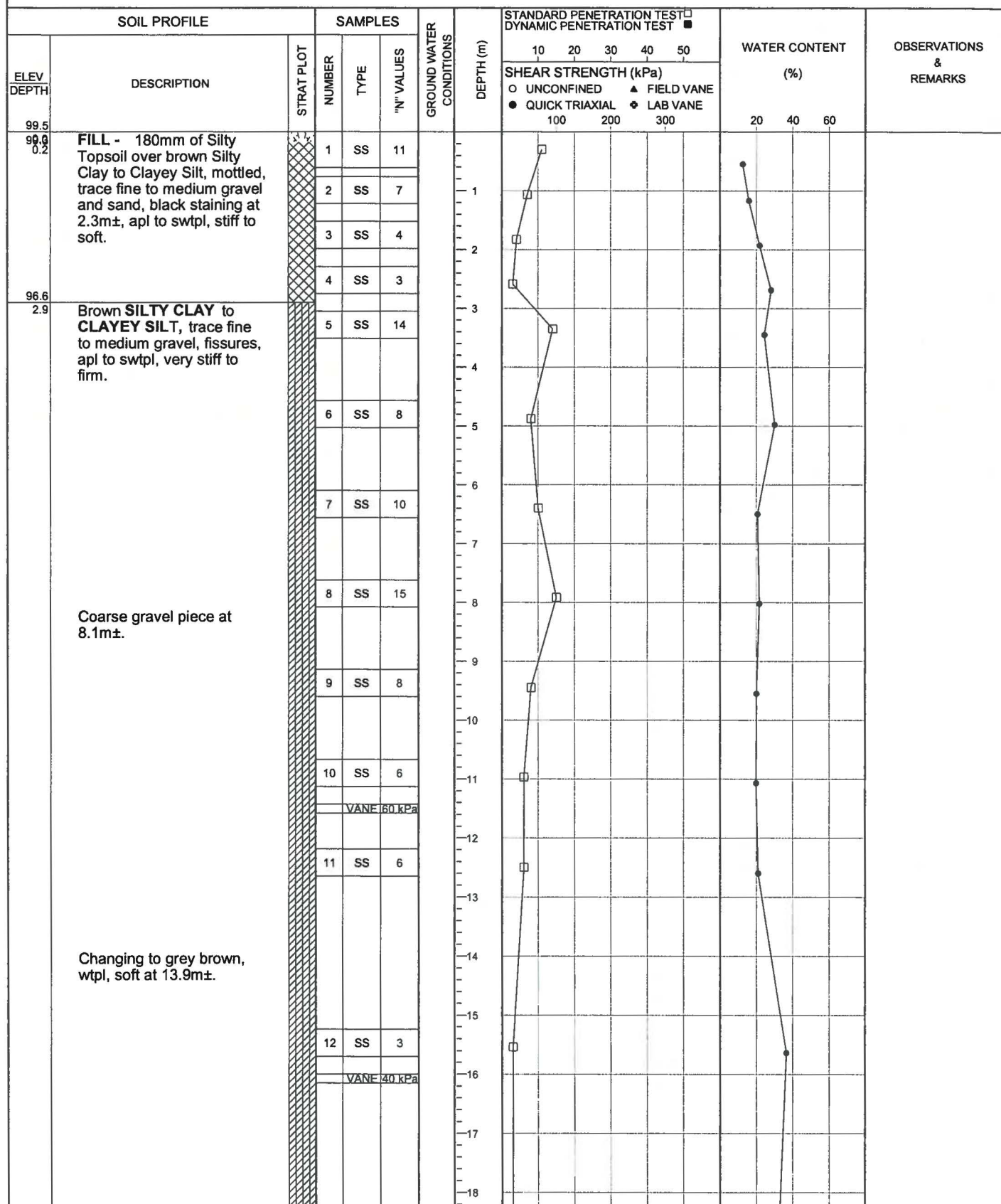
JOB NO. TG113018 DATE 7 June 2011

CHECKED BY RD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH (m)	STANDARD PENETRATION TEST DYNAMIC PENETRATION TEST					WATER CONTENT (%)	OBSERVATIONS & REMARKS		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)								
								10	20	30	40	50			○ UNCONFINED	▲ FIELD VANE
99.2 98.9	FILL - 100mm of Silty Topsoil over brown Silty Clay to Clayey Silt, mottled, trace fine gravel, coarse gravel pieces, black staining and asphalt pieces to 1.4m±, apl to wtpl, firm.		1	SS	8											
			2	SS	7											
			3	SS	5											
			4	SS	4											
			5	SS	6											
			6	SS	5											
93.6 5.6	Red brown SILTY CLAY to CLAYEY SILT, trace fine to medium gravel, apl to swtpl, very stiff to firm.		7	SS	18											
			8	SS	10											
			9	SS	15											
			10	SS	7											
			11	SS	7											
86.6 12.7	BOREHOLE TERMINATED														Upon Completion: Borehole open and dry.	

1 OF 2

PROJECT	<u>GI - Proposed 12 Storey Apartment Building</u>	LOCATION	<u>[As shown on Figure 2]</u>	ORIGINATED BY	<u>NA</u>
CLIENT	<u>Kingsway Investments Ltd. And Lenders</u>			COMPILED BY	<u>NA</u>
JOB NO.	<u>TG113018</u>	DATE	<u>10 June 2011</u>	CHECKED BY	<u>RD</u>



RECORD OF BOREHOLE No 2

2 OF 2

PROJECT GI - Proposed 12 Storey Apartment Building LOCATION [As shown on Figure 2] ORIGINATED BY NA
 CLIENT Kingsway Investments Ltd. And Lenders COMPILED BY NA
 JOB NO. TG113018 DATE 10 June 2011 CHECKED BY RD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH (m)	STANDARD PENETRATION TEST □ DYNAMIC PENETRATION TEST ■		WATER CONTENT (%)	OBSERVATIONS & REMARKS		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					
								○ UNCONFINED ● QUICK TRIAXIAL	▲ FIELD VANE ◆ LAB VANE				
						10	20	30	40	50	20	40	60

See page 1 of 2			13	SS	3		19	□					
							20						
							21						
			14	SS	5		22	□					
							23						
							24						
			15	SS	0		25	□					
							26						
							27						
							28						
							29						
							30						

74.2 25.3	Dynamic Cone Penetration						25	■					
							26	■					
							27	■					
							28	■					
							29	■					
							30	■					
					</								

RECORD OF BOREHOLE No 3

1 OF 1

PROJECT GI - Proposed 12 Storey Apartment Building LOCATION [As shown on Figure 2] ORIGINATED BY NA
 CLIENT Kingsway Investments Ltd. And Lenders COMPILED BY NA
 JOB NO. TG113018 DATE 7 June 2011 CHECKED BY RD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DEPTH (m)	STANDARD PENETRATION TEST ☐ DYNAMIC PENETRATION TEST ☒		WATER CONTENT (%)	OBSERVATIONS & REMARKS				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
								10	20			30	40	50	UNCONFINED ☐
99.3								100	200	300	20	40	60		
90.0	PAVEMENT STRUCTURE - 30mm of Asphalt over 330mm of Granular . FILL - Brown Silty Clay to Clayey Silt, mottled, trace sand and gravel, cobbles, fissures, grey/black staining to 1.4m±, traces of silt seams at 2.1m±, apl to wtpl, firm to very soft. Red/brown SILTY CLAY to CLAYEY SILT , trace fissures, fine to medium gravel, apl to wtpl, stiff to firm. Silt seam at 7.6m±, wet.		1	SS	4									Hydro carbon petroleum odours from 0.8 to 2.1m±.	
98.2			2	SS	0										
			3	SS	2										
			4	SS	11										
			5	SS	8										
			6	SS	12										
			7	SS	14										
			8	SS	7										
			9	SS	8										
			10	SS	7										
86.7	BOREHOLE TERMINATED													Upon Completion: Borehole open and water level at 6.7m±.	
12.7															