Geotechnical Investigation
Proposed Benson Auto Parts Expansion
1871 Merivale Road
Ottawa, Ontario

Prepared for:

Benson Auto Parts

Attention: Mr. Gerry Benson

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TABLE OF CONTENTS

TABLES

APPENDICES

1 INTRODUCTION

LRL Associates Ltd. (LRL) has been retained by the Benson Auto Parts to complete a geotechnical investigation for a proposed expansion of their existing facility located at 1871 Merivale Road, Ottawa, Ontario.

The purpose of the investigation was to identify the subsurface conditions within the proposed expansion area by the completion of a limited borehole drilling program. Based on the factual information obtained, this report will provide guidelines on the geotechnical engineering aspects of the design of the project including construction considerations.

This report has been prepared in consideration of the terms and conditions noted above. Should there be any changes in the design features, which may relate to the geotechnical recommendations provided in the report, LRL Associates Ltd. should be advised in order to review the report recommendations.

2 PROJECT AND SITE DESCRIPTION

The site is located at civic address 1871 Merivale Road in Ottawa, Ontario. It is located on the northeast corner of Merivale Road and Jamie Avenue. The site currently contains a large and narrow building that houses Benson Auto Parts. The existing building is located along the north property line, with parking areas and access lanes as well as a small grassed area to the south of the site. Entrances to the site are available from both streets that its fronts. The site is generally flat throughout. The site is serviced by municipal water and sewer.

It is our understanding that the site development plan for this project will consist of the construction an expansion that will extend from the west side of the existing building and towards Merivale Road. It is understood that the final grading of the site will not be modified significantly from what is seen now. The new building will consist of a one to two-storey structure with a slab-on-grade and no basement. Finally, the existing access roads and parking areas layout may be modified slightly to accommodate the new expansion.

3 PROCEDURE

The fieldwork for this investigation was carried out on September 16th, 2014. A total of three (3) boreholes, labelled BH-1 through BH-3, were drilled within the proposed expansion area. The borehole locations were chosen based on the proposed expansion layout, existing underground services and where access of the drill rig was possible. The approximate location of the boreholes drilled on-site is shown on the latest expansion layout presented in **Appendix A.** Prior to the fieldwork, the borehole locations were cleared for the presence of any underground services and utilities.

The boreholes were advanced using a truck mounted drill rig equipped with continuous flight, hollow stem augers supplied and operated by George Downing Estate Drilling Inc. A “two man” crew experienced with geotechnical drilling operated the rig and equipment. The boreholes were advanced by auguring through the overburden down to a depth of 6.7m below ground surface (bgs).

Sampling of the overburden materials was carried out using a 50mm diameter drive open conventional split spoon sampler in conjunction with standard penetration testing (“N” value). Boreholes were backfilled using a combination of overburden cuttings and silica sand (if there were insufficient cuttings) with the addition of bentonite “hole-plug” below the water table, when needed. The boreholes drilled over paved areas with topped with 100mm of cold patch.

All soil samples collected from the boreholes were placed and sealed in plastic bags to prevent moisture loss. The recovered soil samples collected from the boreholes were classified based on visual and tactile examination of the materials recovered and the results of the in-situ testing. All soil samples were transported to our office for further examination by our geotechnical engineer. The fieldwork was supervised by a member of our engineering staff who oversaw the drilling of boreholes, coordinated the in-situ material testing, cared for the collected samples and logged the subsurface conditions at each borehole location.

A standpipe was installed in BH-2 at a depth of 3m in order to measure the static groundwater level. The standpipe consisted of 19mm diameter PVC piping that were slotted and placed within the overburden prior to backfilling.

4 Subsurface Soil and Groundwater Conditions

4.1 General

A review of the surficial geology maps for this area suggests that the site would be underlain by deltaic deposits, which are generally composed of sand and gravel. The drift thickness of the area would be 10-15m. The bedrock underlying this area would belong to the March Formation, which is generally composed of interbedded sandstone and dolostone.

The subsurface conditions encountered in the boreholes were classified based on visual and tactile examination of the materials recovered from the test holes and the results of the in-situ and laboratory testing. The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and LRL does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The subsurface soil conditions encountered at each borehole location are given in the Borehole Logs presented in **Appendix B**. A greater explanation of the information presented in the Borehole can be found in **Appendix D** of this report. These logs indicate the subsurface conditions encountered at a specific test locations only. Boundaries between zones on the logs are often not distinct, but are rather transitional and have been interpreted as such.

4.2 Topsoil

A thin layer (150mm) of topsoil was found directly at the surface in BH-3 considering it was drilled over a landscaped area. The topsoil is described as being a dark brown sandy loam.

The material classified as topsoil is based on colour and the presence of organic materials and is intended as identification for geotechnical purposes only and does not constitute a statement as to the suitability of this layer for cultivation and sustaining plant growth.

4.3 Pavement Structure

A pavement structure was found at the surface of BH-1 and BH-2 considering they were drilled within paved area of the site. The thickness of the asphaltic concrete was found to range from 75mm to 100mm, while the granular fill was found to be approximately 500mm in thickness.

4.4 Fill

Some fill was encountered below the topsoil layer within BH-3. The fill is described as being sand with some gravel and silt and traces of organics. The fill is brownish grey in colour and found to be in a compact state of packing. The fill was found to extend to a depth of 1.45m bgs.

4.5 Sand

Underlying the pavement structure and the fill material, a sand deposit was encountered in all boreholes completed for this project. In general, the deposit consists of fine grained sand containing some silt. The sand is brown in colour becoming grey with depth and is generally in a compact state of packing. The sand was moist becoming wet at around 4m bgs. The sand deposit was found to extend to approximately 5.6m bgs, where it transitions into medium grained sand. The medium grained sand contains traces of gravel and silt, and is generally wet and in a compact to dense state of packing.

All of the boreholes were terminated within the lower sand deposit at a depth of 6.7m bgs.

4.6 Groundwater Conditions

The presence of shallow overburden groundwater was established by installing a standpipe within BH-2 at depth of 3.0m bgs. The static water levels were measured using a water meter on September 29, 2014. At the time of the measurement, the standpipe in BH-2 was found to be dry.

Based on our observation during the drilling of the boreholes, the groundwater table is anticipated to be located around 4m bgs. The water levels obtained would not necessarily be representative of the high water level due to seasonal variations which can easily fluctuate with local and regional weather conditions (i.e.: rainfall, droughts and spring thawing). In addition, it can be locally affected by the presence of existing underground services.

5 Geotechnical Considerations

This section of the report provides geotechnical recommendations for the design aspect of the project based on our interpretation of the information gathered from the boreholes performed at this site and from the project requirements. It is our understanding that the site development plan for this project will consist of the construction an expansion that will extend from the west side of the existing building and towards Merivale Road. It is understood that the final grading of the site will not be modified significantly from is seen now. The new building will consist of a one to two-storey structure with a slab-on-grade and no basement. Finally, the existing access roads and parking areas layout may be modified slightly to accommodate the new expansion.

5.1 Foundations

Based on the subsurface soil conditions encountered at this site, it is recommended that foundations for the expansion be founded over the native undisturbed sand deposit below frost depth.

Conventional strip footings set over the native undisturbed sand deposit may be designed using a maximum allowable bearing pressure of **100kPa** for serviceability limit state (**SLS**) and **150kPa** for ultimate limit state (**ULS**) factored bearing resistance.

5.2 Settlement

The estimated total settlement of the foundations, designed using the recommended serviceability limit state capacity value given herein as well as other recommendations will be less than 25mm. The differential settlement between adjacent footings is anticipated to be 20mm or less.

5.3 Seismic Design

Based on the results of the geotechnical investigation, the soil at the site can be classified as a Class “D” as per the Site Classification for Seismic Site Response in the latest version of the Ontario Building Code. It is noted that a greater seismic site response class may be obtained from carrying out seismic velocity testing using a multichannel analysis of surface waves (MASW).

5.4 Potential for Soil Liquefaction

Based on the characterisation of the subsurface soil conditions established at this site, the potential of soil liquefaction is not considered to be a concern at this Site

5.5 Use of Structural Fill

Where excavation below the underside of the footing is performed, considerations shall be given to support the footings on structural fill. The structural fill shall be placed over undisturbed native soils in layers not exceeding 200mm and compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD). In order to allow the spread of load beneath the footings and to prevent under mining during construction, the structural fill must extend 1m beyond the outside edges of the footings and then outward and downward at 1 horizontal to 1 vertical profile (or flatter) over a distance equal to the depth of the structural fill below the footing. The recommended material to be used as structural fill to support the footings shall consist of imported granular material meeting Ontario Provincial Standards Specifications (OPSS) requirements for a Granular B Type II, or an approved equivalent material.

Prior to placing any structural fill or pouring of footings, it is required that any disturbed soils along the base of the footings be removed and that the subgrade soils be inspected and approved by the geotechnical engineer. Furthermore, the structural fill must be tested to ensure that the specified compaction level was achieved.

5.6 Frost Protection

All exterior footings located in any unheated portions of the proposed building should be protected against frost heaving by providing a minimum of 1.5 m of earth cover. Areas that are to be cleared of snow (i.e. sidewalks, paved areas, etc.) should be provided with at least 1.8 m of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided using a combination of earth cover and extruded polystyrene insulation. Detailed guidelines for footing insulation frost protection could be provided upon request.

In the event that foundations are to be constructed during winter months, the foundation soils are required to be protected from freezing temperatures using suitable construction techniques. The base of all excavations should be insulated from freezing temperatures immediately upon exposure, until heat can be supplied to the building interior and the footings have sufficient soil cover to prevent freezing of the subgrade soils.

5.7 Foundation Backfill

To prevent possible foundation frost jacking and lateral loading, the backfill material against any foundation walls, grade beams, isolated walls, or piers, should consist of free draining, non-frost susceptible material such as sand or sand and gravel meeting OPSS Granular B Type I grading requirements.

The foundation wall backfill should be compacted to 90% of its SPMDD using light compaction equipment, where no loads will be set over top. The compaction shall be increased to 95% under walkways, slabs or paved areas close to the foundation or retaining walls. Backfilling against foundation walls should be carried out on both sides of the wall at the same time where applicable.

5.8 Foundation Drainage

Considering that no basements are proposed and provided that the proposed finished floor surfaces of the new building are above the exterior finished grades, no perimeter foundation drainage system is required.

In order to minimize ponding of water adjacent to the foundation walls, roof water should be controlled by a roof drainage system and the exterior grade should be sloped to promote surface water away from the foundation walls.

5.9 Slab-on-Grade Construction

Slab-on-grade construction will be acceptable over the native sand only. Therefore, all organic, fill, asphaltic concrete, deleterious or otherwise objectionable materials encountered shall be removed from the building's footprint. Any loose sand would need to be compacted to 95% of its standard proctor maximum dry density (SPMDD). The exposed native subgrade surface should then be inspected and approved by geotechnical personnel.

Any underfloor fill needed to raise the general floor grade shall consist of Granular B Type I material or an approved equivalent, compacted to 95% of its SPMDD. The final lift shall be compacted to 98% of its SPMDD. A 200 mm layer of Granular A material shall be placed under the slab and compacted to at least 98% of the SPMDD.

In order to further minimize and control cracking, the floor slab shall be provided with wire mesh reinforcement and construction or control joints. The construction or control joints should be spaced equal distance in both directions and should not exceed 4.5 m. The wire mesh reinforcement shall be carried through the joints.

5.10 Retaining Walls and Shoring

The following **Table 1** below provides the suggested soil parameters for the design of retaining wall and/or shoring systems. For excavations near existing services and structures, the coefficient of earth pressure at rest (K_o) should be used.

Table 1: Material Properties for Shoring and Permanent Wall Design (Static)

Type of Material	Bulk Density (kg/m^3)	Pressure Coefficient		
		Active (Ka)	Passive (Kp)	At Rest (Ko)
Clay	18	0.45	2.22	0.80
Sand	19	0.33	3.00	0.50
Till	22	0.27	3.70	0.50
Granular B Type I	20	0.33	3.00	0.50
Granular B Type II	23.1	0.31	3.23	0.47
Granular A	23.5	0.27	3.70	0.43

The above values are for a flat surface behind the wall, a straight wall and a wall friction angle of 0 degree. The designer should consider any difference between these coefficients, and make appropriate corrections for a sloped surface behind the wall, angled wall or wall friction as required. The bearing capacity for the design of a retaining wall are the same as provided for the building structure provided it is founded over sand or properly prepared and approved structural fill.

Retaining walls should also be designed to resist the earth pressures produced under seismic conditions. The Canadian building code recommends the used of combined coefficients of static and seismic earth pressure, referred to as K_{AE} for active conditions and K_{PE} for passive conditions for routine design purposes.

The total active and passive loads under seismic conditions can be calculated using the following two equations;

$P_{AE} = ½\ K_{AE}\ γ\ H^2\ (1-k_V)$

$P_{PE} = ½\ K_{PE}\ γ\ H^2\ (1-k_V)$

Where;

K_{AE} = Combined Static and Seismic Active Earth Pressure Coefficient

K_{PE} = Combined static and seismic passive earth pressure coefficient

H = Total Height of the Wall (m)

K_h = horizontal acceleration coefficient

K_v = vertical acceleration coefficient

γ = bulk density (kg/m^3)

These equations are based on a horizontal slope behind the wall and a vertical back of the retaining wall and zero wall friction. For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values.

A = Zonal acceleration ratio = 0.2

K_h = Horizontal acceleration coefficient = 0.1

K_V = Horizontal acceleration coefficient = 0.067

The above value of K_h corresponds to ½ of the A value and the value K_V of corresponds to 0.67 of the K_h value. The angle of friction between the soil and the wall has been set at 0^o to provide a conservative estimate.

The following **Table 2** provides the parameters for seismic design of retaining structures.

Table 2: Material Properties for Shoring and Permanent Wall Design (Seismic)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II
Bulk Unit Weight, γ (kN/m3)	20	23.3
Effective Friction Angle (degrees)	30	32
Angle of Internal Friction Between wall and Backfill (degrees)	0	0
Yielding Wall		
Active Seismic Earth Pressure Coefficient (KAE)	0.37	0.33
Height of the Application of PAE from the base of the wall as a ration of its height (H)	0.36	0.37
Passive Seismic Earth Pressure Coefficient (KPE)	3.06	3.48
Height of the Application of PPE from the base of the wall as a ration of its height (H)	0.30	0.30

6 POTENTIAL OF CORROSIVE ENVIRONMENT

6.1 Sulphate Attack on Buried Concrete

One (1) soil sample collected from BH-2 (SS2) was submitted to Paracel Laboratories Ltd., an accredited chemical testing laboratory, for analysis on sulphate content within the recommended founding soil deposit. The laboratory analysis revealed a maximum sulphate concentration of 0.006% (60 µg/g) within the samples.

Based on the CAN/CSA - A23.1 standards (Concrete Materials and Methods of Concrete Construction), a sulphate concentration of 0.1% (1000 µg/g) or less in soil falls within the negligible category for sulphate attack on buried concrete. As such, buried concrete for footings and foundation walls will not require any special additive to resist sulphate attack and the use of normal Portland cement is acceptable. The laboratory Certificates of Analysis can be found in **Appendix C** of this report.

7 EXCAVATION AND BACKFILLING REQUIREMENTS

7.1 Excavation Requirements

It is anticipated that shallow excavation as part of this project would not exceed 2.0m bgs. Most of the shallow excavation will be through a pavement structure and primarily through native sand. According to the Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 213/91 and its amendments, the surficial overburden soil anticipated to be excavated into at this site can be classified as Type 3 for fully drained excavations. Therefore, shallow temporary excavation in the overburden soil classified as Type 3 can be cut at 1 horizontal to 1 vertical for a fully drained excavation starting at the base of the excavation and as per requirements of the OHSA regulations.

The listed slopes are for fully drained excavations. Much gentler slopes could be required if the excavations are not fully drained, where local water infiltration occurs and where the excavations are exposed for prolonged periods of time. Any excavated material stockpiled near an excavation or trench should be stored at a distance equal to or greater than the depth of the excavation/trench and construction equipment traffic should be limited near any open excavation.

It the event that the aforementioned slopes are not possible to achieve due to space restrictions, the excavation should be shored according to OHSA O. Reg. 213/91 and its amendments. A geotechnical engineer should design and approve the shoring and establish the shoring depth under the excavation profile. Refer to the parameters provided in **Tables 1 and 2** in **Section 5.10** for use in the design of any shoring structures.

7.2 Groundwater Control

Groundwater seepage and infiltration entering shallow (less than 2.0m) and temporary excavations performed within the overburden soil should be mitigated by pumping from sumps installed in the excavation. However, based on the groundwater measurement taken during this investigation, groundwater infiltration into an open excavation at the anticipated excavation depth should be minimal. Surface water runoff into the excavation should be avoided and diverted away from the excavation.

7.3 Trench Backfill

Should any new underground services be required as part of this project, they would be founded over the sand deposit. Bedding, thickness of cover material and compaction requirements for sewers and watermains should conform to the manufacture's design requirements and to the requirements and detail installations outlined in the Ontario Provincial Standard Specifications (OPSS), drawings OPSD 802-030 or 802.031 Class B or Class C for concrete pipes and OPSD 802.01 for flexible pipes as well as any recommendations from the City of Ottawa.

All service trenches should be backfilled using compactable material, free of organics, debris and large cobbles or boulders. Acceptable and compactable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 metres below finished grade) in order to reduce the potential for differential frost heaving. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming at minimum to OPSS Granular B Type I or an approved equivalent.

To minimize future settlement of the trench backfill and achieve an acceptable subgrade for the roadway, the trench should be compacted in maximum 300mm thick lifts to at least 95 SPMDD. The specified density may be reduced where the trench backfill is not located within or in close proximity to existing roadways or any other structures.

8 REUSE OF ON-SITE SOILS

The majority of the native soil found at this site that may be excavated will consist of sand which contains variable silt content. It is possible that the sand could be suitable for use as structural backfill or as backfill material directly against foundation walls however it must be tested once the excavation is complete to ensure that the material meets the contract specifications.

However, this material could be reused for general backfill material (service trenches and general landscaping and/or backfilling), if the material can be compacted according to the guidelines outlined above, at the time of construction.

The existing asphaltic concrete could be pulverised with the granular base and could be used as general trench backfill material or select subgrade material for the new parking areas, if required. Any imported material shall conform to OPSS Granular B - Type I.

It should be noted that the adequacy of a material for reuse as backfill will mainly depend on the water content of the material at the time of use and on the weather conditions at that time. Any excavated materials proposed for reuse should be stockpiled in a manner to promote drying and should be inspected and approved for reuse by a geotechnical engineer.

9 PAVEMENT DESIGN

For predictable performance of the pavement areas, any organic, soft or deleterious materials should be removed from the proposed pavement areas to expose native undisturbed subgrade soil. The exposed subgrade should be inspected and approved by geotechnical personnel and any evidently loose and unstable areas should be sub-excavated and replaced with suitable earth borrow approved by the geotechnical engineer. The subgrade should be shaped and crowned to promote drainage of the roadway. Following approval of the preparation of the subgrade, the granular subbase may be placed.

It is anticipated that the subgrade for the proposed road will consist of sand. The recommended pavement structures for the proposed light and heavy duty access roads and parking areas are provided below.

For light vehicle parking areas and access lanes, the pavement structure should consist of:

50 mm of hot mix asphaltic concrete (HL3) over

150 mm of OPSS Granular A base over

250 mm of OPSS Granular B Type II subbase

For heavy duty access roads, the pavement should consist of:

40 mm of hot mix asphaltic concrete surface layer (HL3) over

40 mm of hot mix asphaltic concrete binder layer (HL8) over

150 mm of OPSS Granular A base over

350 mm of OPSS Granular B, Type II subbase

The base and subbase granular materials should conform to OPSS Form 1010 material specifications. Prior to importing any granular material onto the site, it should be tested and approved by a geotechnical engineer prior to delivery to the site and should be compacted to 100% SPMDD. Compaction of the granular pavement materials should be carried out in maximum 200 mm thick loose lifts to 100% of its SPMDD using suitable vibratory compaction equipment.

Asphaltic concrete should conform to OPSS Form 1150 and be placed and compacted to at least 97% of the Marshall Density. The mix and its constituents should be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

9.1 Paved Areas and Subgrade Preparation

The proposed access lanes and parking areas should be stripped of any vegetation, topsoil, debris and other obvious objectionable material. Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade should be shaped, crowned and proof-rolled using heavy roller with any resulting soft areas sub-excavated down to an adequate bearing layer and replaced with approved backfill. Following approval of the preparation of the subgrade, the pavement structure may be placed.

If the roadway subgrade is disturbed or wetted due to construction operations or precipitation, the granular thicknesses given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a non-woven geotextile separator between the roadway subgrade surface and the granular subbase material.

The performance of the pavement structure is highly dependent on the subsurface groundwater conditions and maintaining the subgrade and pavement structure in a dry condition. To intercept excess subsurface water within the pavement structure granular materials, sub-drains with suitable outlets should be installed below the pavement structure subgrade, if adequate overland flow drainage is not provided (i.e. ditches). The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features. It is recommended that the lateral extent of the subbase and base layers not be terminated vertically immediately behind any proposed the curb/edge of pavement line but be extended beyond the curb.

For areas of the site that require the subgrade to be raised, the material should consist of OPSS Granular B Type 1 or approved equivalent. Any materials proposed for this use should be approved by the geotechnical engineer before placement. Materials used for raising the subgrade to the proposed roadway subgrade level should be placed in maximum 300 mm thick loose lifts and be compacted to at least 95% of the SPMDD using suitable compaction equipment. Any grade raise must respect the recommendations provided in the foundation section.

The preparation of subgrade should be scheduled and carried out in such a manner that a protective cover of overlying granular material is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment over the subgrade. Frost protection of the surface should be implemented (i.e. insulated tarps, etc.), if works are carried out during the winter months.

Transitions should be constructed between new and existing pavement structures where new access lanes will meet with existing road. In areas where the new pavement structure will abut existing pavement structure, the depths of granular materials should be tapered up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

10 INSPECTION SERVICES

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed development do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

All footing areas and any engineered fill areas for the proposed addition should be inspected by LRL Associates Ltd. to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations and slab-on-grade should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the pavement areas, watermain and sewers should be inspected and approved by geotechnical personnel. In-situ density testing should be carried out on the pavement granular materials and pipe bedding and backfill to ensure the materials meet the specifications from a compaction point of view.

11 REPORT CONDITIONS AND LIMITATIONS

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document or its use by a third party other than the client specifically listed in the report is neither intended nor authorized by LRL Associates Ltd. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or deep contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report.

The recommendations provided in this report are based on subsurface data obtained at the specific test locations only. Boundaries between zones presented on the borehole and test pit logs are often not distinct but transitional and were interpreted. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the recommendations given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The report recommendations are applicable only to the project described in the report. Any changes to the project will require a review by LRL Associates Ltd., to ensure compatibility with the recommendations contained in this project. We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact the undersigned.

Yours truly,

LRL Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
W. BALL
100192508
Oct 14, 2014
PROVINCE OF ONTARIO

Mario Elie, Senior Technologist
Project Manager

Will Ball, P. Eng.
Project Engineer

APPENDIX A

SITE AND BOREHOLE LOCATION PLAN

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PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED EXPANSION BENSON AUTO PARTS
1871 MERIVALE ROAD
OTTAWA, ONTARIO

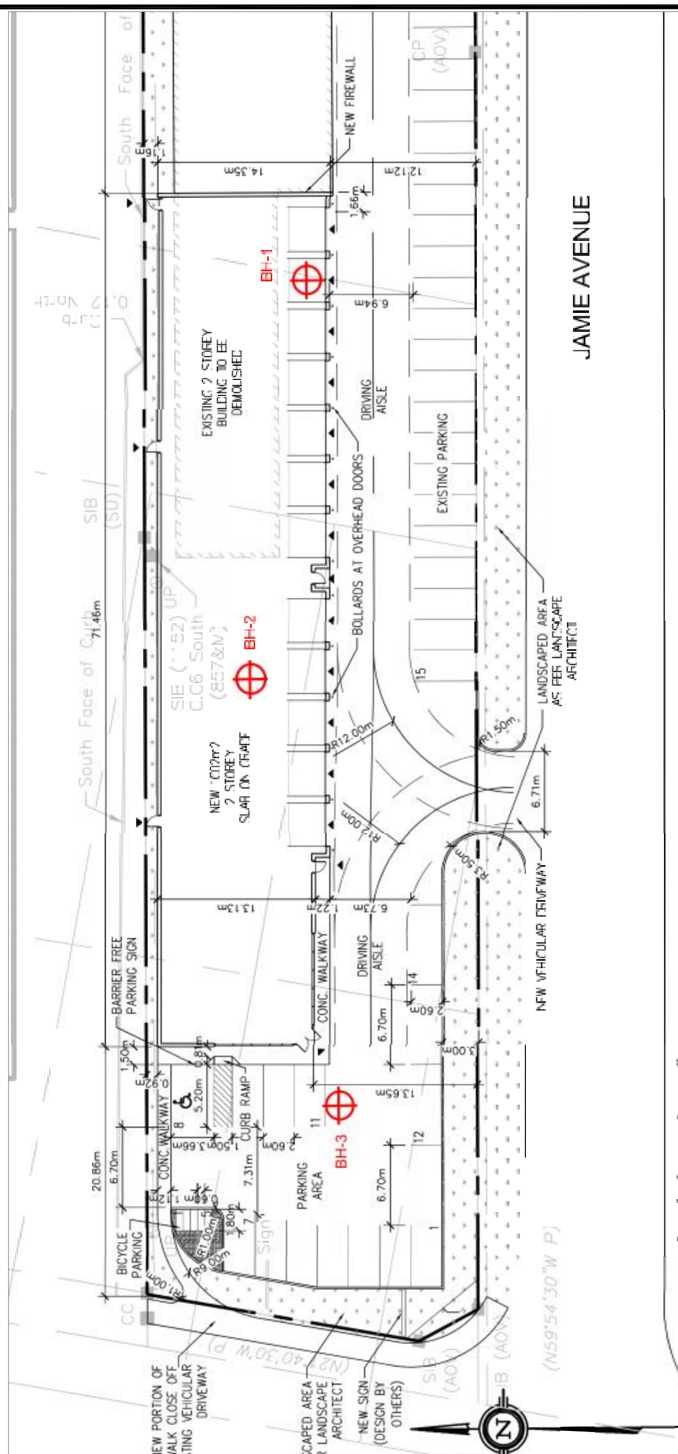
DRAWING TITLE

SITE AND BOREHOLE LOCATION PLAN

CLIENT: Mr. Gerry Benson

DATE: OCTOBER 2014

PROJECT: 140828



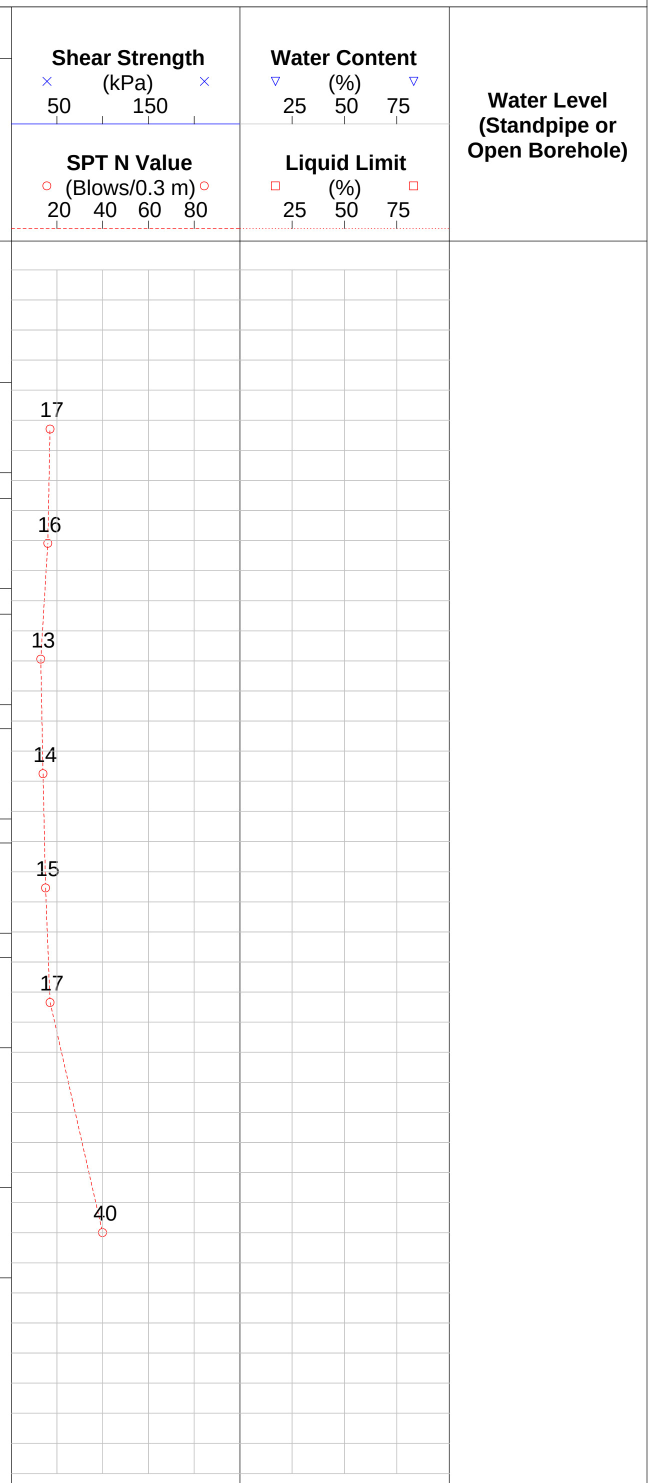
APPENDIX B

BOREHOLE LOGS

Borehole Log: BH-1

Project No.: 130828
Project: Expansion of Benson Auto Parts
Client: Benson Auto Parts
Location: 1871 Merivale Road, Ottawa, Ontario
Date: September 16, 2014
Field Personnel: WB
Driller: Goerge Downing Estate Drilling
Drilling Equipment: CME 55, Truck Mount
Drilling Method: HSA

Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
Ground Surface	0.00 / 0.00				
PAVEMENT STRUCTURE 100mm Asphaltic Concrete 500mm Granular Fill	-0.60 / 0.60				
SAND Fine grained, some silt, brown becoming grey with depth, moist to wet below approximately 4m, compact		SS	1	17	75
		SS	2	16	75
		SS	3	13	100
		SS	4	14	100
		SS	5	15	100
	-5.63 / 5.63	SS	6	17	50
SAND Medium grained, traces of gravel and silt, grey, wet, dense	-6.70 / 6.70	SS	7	40	50
End of Borehole					



Easting: 443412
Northing: 5020312
Site Datum: None
Groundsurface Elevation: N/A
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES:

Borehole Log: BH-2

Project No.: 130828

Project: Expansion of Benson Auto Parts

Client: Benson Auto Parts

Location: 1871 Merivale Road, Ottawa, Ontario

Date: September 16, 2014

Field Personnel: WB

Driller: Goerge Downing Estate Drilling

Drilling Equipment: CME 55, Truck Mount

Drilling Method: HSA

SUBSURFACE PROFILE / SAMPLE DATA

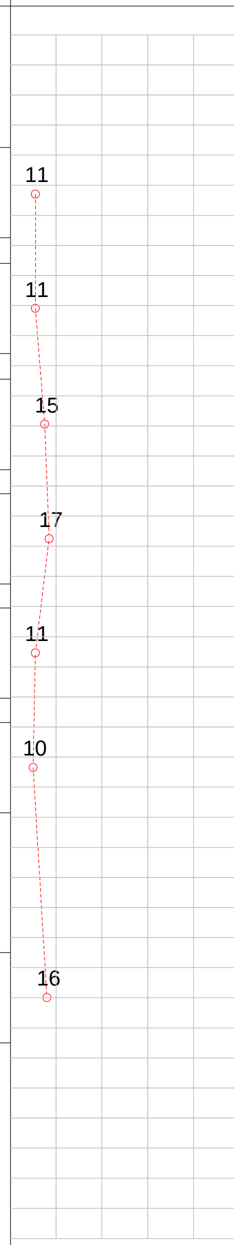
Depth (m)	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	Water Level (Standpipe or Open Borehole)
0	Ground Surface	0.00 / 0.00					
0.00–0.60	**PAVEMENT STRUCTURE** 75mm Asphaltic Concrete 500mm Granular Fill	-0.60 / 0.60					
0.60–5.63	**SAND** Fine grained, some silt, brown becoming grey with depth, moist to wet below approximately 4m, compact	-5.63 / 5.63	SS	1	11	75	Dry at 3m (September 29, 2014)
			SS	2	11	75	
			SS	3	15	75	
			SS	4	17	50	
			SS	5	11	50	
			SS	6	10	50	
5.63–6.70	**SAND** Medium grained, traces of gravel and silt, grey, wet, compact	-6.70 / 6.70	SS	7	16	50	
6.70	End of Borehole						

Shear Strength (kPa): 50, 150

Water Content (%): 25, 50, 75

SPT N Value (Blows/0.3 m): 20, 40, 60, 80

Liquid Limit (%): 25, 50, 75



Easting: 443376

Northing: 5020298

Site Datum: None

Groundsurface Elevation: N/A

Top of Riser Elev.: N/A

Hole Diameter: 200 mm

NOTES:

Borehole Log: BH-3

Project No.: 130828
Project: Expansion of Benson Auto Parts
Client: Benson Auto Parts
Location: 1871 Merivale Road, Ottawa, Ontario
Date: September 16, 2014
Field Personnel: WB
Driller: Goerge Downing Estate Drilling
Drilling Equipment: CME 55, Truck Mount
Drilling Method: HSA

SUBSURFACE PROFILE / SAMPLE DATA

Depth (m)	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	SPT N Value (Blows/0.3 m)
0	Ground Surface	0.00 / 0.00					
0–0.15	**TOPSOIL** 150mm Dark Brown Sandy Loam						
0.15–1.45	**FILL** Sand, some gravel and silt, traces of organics, brownish grey, moist, compact		SS	1	12	50	12
1.45–5.63	**SAND** Fine grained, some silt, brown becoming grey with depth, moist to wet below approximately 4m, compact	-1.45 / 1.45	SS	2	14	50	14
			SS	3	13	50	13
			SS	4	16	50	16
			SS	5	10	50	10
			SS	6	14	50	14
5.63–6.70	**SAND** Medium grained, trace gravel and silt, grey, wet, compact	-5.63 / 5.63	SS	7	14	50	14
6.70	End of Borehole	-6.70 / 6.70					

Shear Strength (kPa): 50, 150
Water Content (%): 25, 50, 75
SPT N Value (Blows/0.3 m): 20, 40, 60, 80
Liquid Limit (%): 25, 50, 75
Water Level (Standpipe or Open Borehole)

Easting: 443344
Northing: 5020263
Site Datum: None
Groundsurface Elevation: N/A
Top of Riser Elev.: N/A
Hole Diameter: 200 mm

NOTES:

APPENDIX C

Laboratory Certificate of Analysis – Chemical Tests

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e: paracel@paracellabs.com
www.paracellabs.com

Certificate of Analysis

LRL Associates Ltd.
5430 Canotek Road
Ottawa, ON K1J 9G2
Attn: Will Ball

Phone: (613) 446-7777
Fax: (613) 446-1427

Client PO:
Project: 130828
Custody: 101546

Report Date: 7-Oct-2014
Order Date: 2-Oct-2014
Order #: 1440256

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1440256-01	130828, BH-2, SS#2

Approved By: Mark Foto

Mark Foto, M.Sc. For Dale Robertson, BSc
Laboratory Director

Order #: 1440256

Certificate of Analysis

Report Date: 07-Oct-2014
Order Date:2-Oct-2014

Client: **LRL Associates Ltd.**
Client PO:
Project Description: 130828

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	6-Oct-14	7-Oct-14
Solids, %	Gravimetric, calculation	4-Oct-14	4-Oct-14

PARACEL LABORATORIES LTD.

Certificate of Analysis

Report Date: 07-Oct-2014
Order Date:2-Oct-2014

Client: **LRL Associates Ltd.**

Client PO:

Project Description: 130828

	Client ID:	130828, BH-2, SS#2	-	-	-
	Sample Date:	16-Sep-14	-	-	-
	Sample ID:	1440256-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	84.4	-	-	-
Anions					
Sulphate	5 ug/g dry	60	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	ND	5	ug/g						

PARACEL LABORATORIES LTD.

Order #: 1440256

Certificate of Analysis

Report Date: 07-Oct-2014

Client: **LRL Associates Ltd.**

Order Date:2-Oct-2014

Client PO:

Project Description: 130828

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	21.7	5	ug/g dry	21.6			0.5	20	
Physical Characteristics									
% Solids	39.5	0.1	% by Wt.	39.6			0.2	25	

Certificate of Analysis

Report Date: 07-Oct-2014
Order Date:2-Oct-2014

Client: **LRL Associates Ltd.**
Client PO:
Project Description: 130828

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Sulphate	122	5	ug/g	21.6	100	78-111			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

APPENDIX D

SYMBOLS AND TERMS USED IN BOREHOLE AND TEST PIT LOGS

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Symbols and Terms Used on Borehole and Test Pit Logs

The following explains the data presented in the borehole and test pit logs.

1. Soil Description

The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves some judgement and LRL Associates Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice. Boundaries between zones on the logs are often not distinct but transitional and were interpreted.

a. Proportion

The proportion of each constituent part, as defined by the grain size distribution, is denoted by the following terms:

Term	Proportions
"trace"	1% to 10%
"some"	10% to 20%
prefix (i.e. "sandy" silt)	20% to 35%
"and" (i.e. sand "and" gravel)	35% to 50%

b. Compactness and Consistency

The state of compactness of granular soils is defined on the basis of the Standard Penetration Test. See Section 2c for more details. The consistency of clayey or cohesive soils is based on the shear strength of the soil, as determined by field vane tests and by a visual and tactile assessment of the soil strength.

The state of compactness of granular soils is defined by the following terms:

State of Compactness Granular Soils	Standard Penetration Number "N"
Very loose	0 – 4
Loose	4 – 10
Compact or medium	10 - 30
Dense	30 - 50
Very dense	over - 50

The consistency of cohesive soils is defined by the following terms:

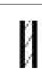
Consistency Cohesive Soils	Undrained Shear Strength (Cu) (kPa)
Very soft	under 10
Soft	10 - 25
Medium or firm	25 - 50
Stiff	50 - 100
Very stiff	100 - 200
Hard	over - 200

2. Sample Data

a. Elevation depth

This is a reference to the geodesic elevation of the soil or to a benchmark of an arbitrary elevation at the location of the borehole or test pit. The depth of geological boundaries is measured from ground surface.

b. Type

Symbol	Type	Letter Code
	Auger	AU
	Split spoon	SS
	Shelby tube	ST
	Rock Core	RC

c. Sample Number

Each sample taken from the borehole is numbered in the field as shown in this column.

LETTER CODE (as above) – Sample Number

d. Blows (N) or RQD

This column indicates the Standard Penetration Number (N) as per ASTM D-1586. This is used to determine the state of compactness of the soil sampled. It corresponds to the number of blows required to drive 300 mm of the split spoon sampler using a 622 kg*m/s^2 hammer falling freely from a height of 760 mm. For a 600 mm long split spoon, the blow counts are recorded for every 150 mm. The "N" index is obtained by adding the number of blows from the 2nd and 3rd count. Technical refusal indicates a number of blows greater than 50.

In the case of rock, this column presents the Rock Quality Designation (RQD). The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 10 cm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given below.

Rock Quality Designation (RQD) (%)	**Description of Rock Quality**
0 –25	very poor
25 – 50	poor
50 – 75	fair
75 – 90	good
90 – 100	excellent

e. Recovery (%)

For soil samples this is the percentage of the recovered sample obtained versus the length sampled. In the case of rock, the percentage is the length of rock core recovered compared to the length of the drill run.

3. General Monitoring Well Data