Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

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Geotechnical Investigation
Proposed Residential Development
Kenwood Avenue at Edison Avenue
Ottawa, Ontario

Prepared For

Uniform Urban Developments

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Report PG0949-1

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APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Uniform Urban Developments Limited (Uniform) to conduct a geotechnical investigation for a proposed residential development, which is to be located on the south side of Kenwood Avenue between Edison Avenue and Melbourne Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

- determine the subsoil and groundwater conditions at this site by means of eight (8) boreholes, and
- based on the results of the boreholes, provide geotechnical recommendations pertaining to the design of the proposed development, including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as it is understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. Therefore, the present report does not address environmental issues.

Paterson has completed a Phase I - Environmental Site Assessment (ESA) and a Designated Substances Survey (DSS) for the subject site. The findings and recommendations of the Phase I - ESA and DSS are presented under separate cover.

2.0 PROPOSED DEVELOPMENT

It is understood that the development will include the construction of twelve (12) single family dwellings, seven (7) semi-detached residential units, ancillary paved parking areas and access lanes.

Currently, a residential house and detached garage exist in the south portion of the subject site. Two (2) large buildings connected by a breezeway and a separate outbuilding are currently located along the east property line. A paved parking area and access lanes also exist on site. It is understood that the existing buildings will be demolished to permit the construction for the proposed development.

The subject property is bordered to the north by Kenwood Avenue, to the east by Edison Avenue, to the south by a residential apartment building, and to the west by Melbourne Avenue.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

The fieldwork program for the investigation was carried out on February 2 and 6, 2007. At that time, six (6) boreholes were advanced to depths varying between 1.5 to 3.6 m. Two (2) of the proposed boreholes were not completed due to the limited access along the east property line. The approximate borehole locations are shown on Drawing PG0949-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden. The bedrock was cored at three (3) locations using diamond drilling procedures.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. Rock cores were recovered using diamond drilling procedures. The cores were logged and placed in hard cardboard core boxes. All samples were transported to our laboratory. The depths at which the split-spoon, auger and core samples were recovered from the boreholes are shown as SS, AU, and RC, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling using an NW size casing was carried out in BHs 1, 2 and 3 to determine the nature of the bedrock. Recovery values and Rock Quality Designation (RQD) values were calculated for the drilled sections (core runs) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of rock pieces longer than 100 mm in one core run over the length of the core run. All these values are indicative of the quality of the bedrock.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Flexible standpipes were installed in BH 1, BH 2 and BH 3 to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

All samples will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were selected in the field by Paterson personnel in a manner to provide general coverage of the subject site and to address some environmental issues, taking into consideration the presence of underground services and access difficulties. The ground surface elevations at the boreholes were referenced to a temporary benchmark (TBM), which consists of the top of spindle of the fire hydrant located at the southeast corner of the intersection of Kenwood Avenue and Melbourne Avenue. An assumed geodetic elevation of 76.17 m was assigned to the TBM as shown on the survey plan provided by Uniform. The locations and ground surface elevations of the boreholes are presented on Drawing PG0949-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil and rock samples recovered from the subject site were examined in our laboratory to review the results of the field logging.

4.0 OBSERVATIONS

4.1 Surface Conditions

The subject site is relatively flat and at approximately grade with surrounding streets. A residential house and detached garage exist in the south portion of the subject site. Two (2) large buildings connected by a breezeway and a separate outbuilding are currently located along the east property line. A paved parking area and access lanes also currently exist on site.

4.2 Subsurface Profile

Generally, the soil conditions encountered at the boreholes consist of topsoil or asphaltic concrete overlying fill and/or sandy silt/silty sand. Weathered shale with frequent sandy silt seams was encountered below the overburden at all boreholes. Fresh to faintly weathered interbedded limestone and dolostone bedrock was encountered below the weathered shale at BH 1, BH 2 and BH 3. Practical refusal to augering was encountered at all boreholes, at depths varying between 1.4 ans 2.3 m. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at the boreholes.

Topsoil

Topsoil was encountered at ground surface at BH 1, BH 2, BH 3 and BH 5. The thickness of the topsoil varies between 25 and 180 mm. Topsoil layers, 110 and 130 mm thick, were also encountered below the fill at BH 4 and BH 6, respectively.

Pavement Structure

Asphaltic concrete was encountered at ground surface at BH 4 and BH 6. The thickness of the asphaltic concrete was observed to be 25 mm.

Crushed stone was encountered below the asphaltic concrete at both boreholes. The crushed stone extends to depths of 180 and 200 mm at BH 4 and BH 6, respectively.

Fill

Fill, consisting of silty sand with gravel or sandy silt with clay and gravel, was encountered underlying the pavement structure or topsoil at BH 2, BH 4 and BH 5. The fill extends to depths varying between of 0.5 m, at BH 5, to 1.5 m, at BH 2. Organic matter was observed in the upper 0.6 m of the sandy silt layer at BH 2.

Silty Sand/Sandy Silt

Silty sand/sandy silt was encountered below the topsoil at BH 1, BH 3, BH 5, and BH 6. This layer extends to depths varying between 0.7 at BH 1 and BH 3 to 1.1 m at BH 1. Gravel was observed within the silty sand/sandy silt layer at BH 1 and BH 3. Organic matter was observed within the sandy silt layer at BH 5.

Practical Refusal to Augering/Bedrock

Practical refusal to augering was encountered in all boreholes, at depths ranging from 1.4 m at BH 3 to 2.3 m at BH 5.

Based mainly on observations during the drilling operations, due to the very low recovery values of the split-spoon samples, it is inferred that weathered shale with frequent sandy silt seams was encountered at BH 1, and BH 3 to BH 6, at depths ranging from 0.7 m to 1.1 m.

Faintly weathered to sound bedrock is inferred below the weathered shale. At BH 1, BH 2 and BH 3, the bedrock was cored, below the practical refusal to augering depths, to determine the bedrock type and quality. Generally, the bedrock was found to consist of faintly weathered to fresh dolostone. The recovery values vary between 90 and 100%, and the RQD values range from 39 to 94%. These values are indicative of a poor to excellent quality.

Based on available geological mapping, the subject site is located in an area where bedrock consists of limestone and dolomite (dolostone) of the Gull River Formation, and is expected at depths between 0 to 2 m.

4.3 Groundwater

The groundwater levels were measured in the standpipes at BH 1, BH 2 and BH 3 on February 16, 2007. The measured groundwater levels are presented in Table 1.

It should be noted that the groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could be higher at the time of construction.

Table 1
Summary of Groundwater Level Readings

Borehole Number	Ground Elevation, m	Groundwater Levels, m		Recording Date
		Depth	Elevation	
BH 1	75.65	2.70	72.95	February 16, 2007
BH 2	74.93	1.09	73.84	February 16, 2007
BH 3	75.73	Dry @ 2.5 m	-	February 16, 2007

Note: The ground surface elevation at each borehole was referenced to a temporary benchmark (TBM), which consists of the top spindle of the fire hydrant located at the southeast corner of the intersection of Kenwood Avenue and Melbourne Avenue. A geodetic elevation of 76.17 m was assumed at the TBM.

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 Geotechnical Assessment

It is understood that the development will include the construction of twelve (12) single family dwellings, seven (7) semi-detached residential units, ancillary paved parking areas and access lanes across the subject site.

Currently, a house and a detached garage exist in the south portion of the subject site. Two (2) large buildings connected by a breezeway and a separate outbuilding are currently located along the east property line. A paved parking area and access lanes also currently exist on site. It is understood the existing buildings will be demolished for the proposed development.

Based on the results of the boreholes, it is expected that the proposed buildings will be founded on shallow footings placed on weathered shale bedrock and/or faintly weathered to fresh dolostone/limestone.

Footings, up to 2 m wide, placed on weathered bedrock can be designed based on an allowable bearing pressure of 175 kPa. Footing placed on faintly weathered or fresh bedrock can be designed based on an allowable bearing pressure of 1000 kPa.

Bedrock removal could be required for the construction of the foundations/basements of the proposed buildings, and to complete the service trenches. Hoe ramming could be used where only small quantities of bedrock need to be removed. Line drilling and controlled blasting could be used where large quantities of bedrock need to be removed. The blasting operations should be planned and carried out under the guidance of a professional engineer with experience in blasting operations.

Excavation through the overburden can be carried out with side slopes profiled at 1H:1V or shallower.

Groundwater infiltration into the shallow excavations anticipated at this site should be controllable using properly designed sumps and pumps.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

All fill and topsoil should be removed from within the proposed buildings' perimeters. Topsoil and deleterious fill such as those containing organic material should be stripped from under any paved areas and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the perimeters of the buildings. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum depth of 1.0 m below final grade.

Bedrock Removal

It is expected that the weathered shale can be excavated using an hydraulic shovel. Faintly weathered and fresh bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock need to be removed; line drilling and controlled blasting can be used where a large quantity of bedrock needs to be removed. Prior to considering blasting, the blasting effects on the existing buildings and structures should be considered.

As a general guideline, peak particle velocities should not exceed 50 mm/sec (measured at the structures) during the blasting operation to reduce the risks of damages to the existing structures.

The blasting operations should be carried out under the supervision of a licensed professional engineer who is also a blasting expert.

A pre-blast or preconstruction survey of the existing surrounding structures should be carried out prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

Fill Placement

Fill used for grading beneath the buildings (outside the zones of influence of the footings) and under the base and subbase layers of the paved areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type I or II. These materials should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the buildings (outside the zones of influence of the footings) and paved areas should be compacted to at least 95% of its standard Proctor maximum dry density (SPMDD).

Throughout the zones of influence of the footings, the fill should consist of OPSS Granular A crushed stone or Granular B Type II materials. These materials should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed within the zones of influence of the footings should be compacted to at least 98% of the material's SPMDD.

Where the footings are founded on faintly weathered to fresh bedrock, lean concrete should be used below the footing level, if required, to raise the founding level.

The zone of influence of a footing is considered to be the area beneath the footing limited by planes extending down and out from the bottom edges of the footing, at a slope of 1H:1V, to the in situ soil.

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to reduce voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a prefabricated drainage system (such as system Platon or Miradrain G100N) connected to a perimeter drainage system is provided.

If excavated rock is to be used as fill, it should be suitably fragmented to produce a well-graded material with a maximum particle size of 300 mm. This material should only be used structurally to build up the subgrade for roads and pavements. Where the fill is open-graded, a blinding layer of finer granular fill or a geotextile may be required to prevent adjacent finer materials from migrating into the voids, with associated loss of ground and settlements.

5.3 Foundation Design

Shallow Foundation

Founding conditions at the site are favorable for the single family dwellings and semi-detached structures proposed at this site. Generally, based on the soil profile encountered across the site, it is anticipated that weathered shale bedrock or faintly weathered to fresh dolostone will be encountered at the founding levels. These materials are suitable bearing strata upon which to found footings for the support of the proposed structures.

Footings, up to 2 m wide, placed on a clean, weathered bedrock can be designed using an allowable bearing pressure of 175 kPa. Footings placed on a clean, surface sounded bedrock fresh to faintly weathered dolostone/limestone can be designed using an allowable bearing pressure of 1,000 kPa.

A clean, weathered bedrock surface consists of one from which all topsoil, soils, deleterious materials and loose rock have been removed prior to concrete placement.

A clean, surface-sounded bedrock bearing surface should be free of all soil and loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support. Near vertical (1H:6V) slopes can be used for unfractured bedrock bearing media; a 1H:1V slope can be used for fractured/weathered bedrock.

Grade Raise/Settlement

Based on the soil conditions encountered at this site, grade raises are not considered to be a concern for buildings founded on bedrock.

Potential post-construction long term total and differential settlements for footings placed on weathered shale and designed based on an allowable bearing pressure of 175 kPa are estimated to be 25 and 20 mm, respectively. The potential long term post-construction total and differential settlements for footings placed on surface-sounded faintly weathered to fresh dolostone/limestone and designed based on an allowable bearing pressure of 1000 kPa are estimated to be negligible.

It is not recommended to place the footings of a building partly on faintly weathered to fresh bedrock and partly on weathered bedrock or soil without providing means to accommodate differential settlements of up to 25 mm between the footings.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as Class C for the structures considered at this site. Reference should be made to the latest revision of the 2006 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of all topsoil, organic soils and fill within the footprint of the proposed buildings, the native soil surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular B Type I or Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab (outside the zones of influence of the footings). It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone. All backfill materials within the footprints of the proposed buildings (but outside the zones of influence of the footings) should be placed in maximum 300 mm thick loose layers and compacted to at least 95% of its SPMDD. Within the zones of influence of the footings, the backfill materials should be compacted to a minimum of 98% of its SPMDD.

Bedrock was encountered at shallow depths across the subject site. It is recommended that a minimum 0.3 m thick layer (native soil plus crushed stone layer) be present between the floor slabs and the bedrock surface to reduce the risks of bending stresses in the concrete slab. The bending stress could lead to cracking of the concrete slabs. This requirement could be waived if the bedrock surface is relatively flat within the footprint of the building. This recommendation does not refer to potential concrete shrinkage cracking which should be controlled in the usual manner.

5.6 Pavement

Car only parking areas and heavy traffic access areas are expected at this site. The subgrade material is expected to consist of silty sand/sandy silt. The proposed pavement structures are presented in Tables 2 and 3.

Table 2
Recommended Pavement Structure - Car Only Parking Areas

Thickness mm	Material Description
50	**WEAR COURSE** - HL 3 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil or fill.

Table 3
Recommended Pavement Structure - Access Lanes and Heavy Truck Parking Areas

Thickness mm	Material Description
40	**WEAR COURSE** - HL 3 Asphaltic Concrete
50	**BINDER COURSE** - HL 8 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
	SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil or fill.

Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or II material.

The pavement granulars (base and subbase) should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Deleterious fill and topsoil encountered at the subgrade level should be excavated. The excavation should be backfilled with acceptable materials placed in maximum 300 mm thick lifts and compacted to a minimum of 95% of the material's SPMDD.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity. Consideration should be given to installing subdrains at the catch basins. These drains should be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. The subdrain inverts should be approximately 300 mm below subgrade level.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed buildings. The system should consist of a 100 mm to 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed around the exterior perimeter of the structures. The clear stone should be wrapped in a non-woven geotextile. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I material, should be used for this purpose. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls unless a prefabricated drainage system (such as Miradrain G100N or System Platon) connected to a perimeter drainage system is provided.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures should be insulated against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or insulation equivalent) should be provided in this regard.

A minimum 2.1 m thick soil cover (or insulation equivalent) should be provided for other exterior unheated footings, such as those for isolated exterior piers.

Due to the high level of the bedrock across the site, consideration can be given to using rigid insulation boards to provide adequate frost protection. Typical foundation insulation details for footing are shown in Figures 2 and 3 presented in Appendix 2.

Option 1 (figure 2) consists of an insulation layer placed on the exterior face of the foundation wall, then extending horizontally over the top of the footing and outward for the specified dimension (L in Table A1). The footing is founded directly on the bearing medium, so the full bearing pressure of the bearing medium can be used.

Option 2 (figure 3) makes use of subfooting insulation. In preparation for the subfooting insulation, a levelling mat of clean (mortar) sand or lean concrete should be placed on the approved bearing surface. The insulation layer extends horizontally as shown in Figure 3. It should be noted that the bearing pressure for footings placed on the insulation boards should not exceed about 35% of the insulation's quoted compressive strength due to the time dependant creep characteristics of this material. The allowable design bearing pressures for footings placed on rigid insulation boards are provided in Table A2 in Appendix 2.

Care must be taken to ensure that the insulation is not damaged during construction. All joints should be carefully overlapped and/or glued where and if possible.

6.3 Excavation Side Slopes

The side slopes of the shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available to permit the building excavations to be undertaken by open-cut methods (i.e. unsupported excavations).

Above the groundwater level, within the overburden and the weathered shale bedrock, the excavation side slopes extending to a maximum depth of 3.5 m should be cut back at 1H:1V. In fresh to faintly weathered dolostone/limestone, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by "cut and cover" methods and excavations will not be left open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa (5th edition, March 31, 2006). Trench details should be as per Drawings W17, S6 and S7.

At least 150 mm of OPSS Granular A should be used for bedding for sewer pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's standard Proctor maximum dry density (SPMDD).

At least 150 mm of OPSS Granular A should be used for bedding for water pipes. The bedding material, which should extend to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular M. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to a minimum of 95% of the material's SPMDD.

It should generally be possible to re-use the moist, not wet, site excavated silty sand/sandy silt above the cover material if the excavation and filling operations are carried out in dry weather conditions and all stones greater than 300 mm in their longest dimension are removed.

Where hard surfaces are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below the finished grade) should match the soils exposed at the trench walls to reduce differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

Where blast-rock fill is used as trench fill, it should be well-graded and of maximum 300 mm in size. The use of a blinding layer or woven geotextile may be required for open-graded blast-rock.

6.5 Groundwater Control

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

The rate of flow of groundwater into the excavation through the overburden is expected to be low for the shallow excavations at this site. It is anticipated that pumping from open sumps will be sufficient to control the groundwater influx through the sides of the excavations.

6.6 Winter Construction

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level. Placing concrete directly on a cold bedrock surface is not recommended.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

7.0 MATERIALS TESTING AND OBSERVATION SERVICES

It is a requirement for the foundation design data provided herein to be applicable that a materials testing and observation program including the following aspects be performed by the geotechnical consultant.

- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Observation of the placement of the foundation insulation, if applicable.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3.0 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

Upon demand, a report confirming that the inspection program has been conducted in general accordance with our recommendations could be issued following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 STATEMENT OF LIMITATIONS

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations provided are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions which differ from those at the test locations be encountered at the site, we request immediate notification in order to permit a reassessment of our recommendations.

The recommendations provided in this report should only be used by the design professionals associated with this project. They are not intended for contractors bidding or undertaking the work. The contractors should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Uniform Urban Developments Limited or their agents is not authorized without review by Paterson Group for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
March 5/07
G. J. A. COLLINS
PROVINCE OF ONTARIO

Glenn Collins, P.Eng.

Carlos P. Da Silva, P.Eng.

Report Distribution:

- ❑ Uniform Urban Developments Limited (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE & TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Residential Development, Kenwood Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, intersection of Kenwood Avenue and Melbourne Avenue. Assumed geodetic elevation = 76.17m.

REMARKS

BORINGS BY CME 75 Power Auger **DATE** 2 FEB 07

FILE NO. PG0949

HOLE NO. BH 1

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	75.65
25mm Topsoil Brown SANDY SILT, trace gravel	0.69	AU	1				
BEDROCK: Weathered, black shale fragments with sandy silt seams throughout		SS	2	100	50+	1	74.65
- Practical refusal to augering	1.96	SS	3	33	63+		
- Start of diamond drilling						2	73.65
BEDROCK: 0.3m of grey faintly weathered to fresh limestone over faintly weathered to fresh grey-black dolomite - horizontal fractures @ 75, 289, 330, 355 and 395mm from top of RC1		RC	1	90	50		
	3.56	RC	2	92	92	3	72.65
End of Borehole (GWL @ 2.70m-Feb. 16/07)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Residential Development, Kenwood Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, intersection of Kenwood Avenue and Melbourne Avenue. Assumed geodetic elevation = 76.17m.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 2 FEB 07

FILE NO. PG0949

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	74.93
TOPSOIL 0.18							
FILL: Brown sandy silt with organic matter 0.60							
FILL: Brown sandy silt with clay and gravel		SS	1	46	7	1	73.93
- Practical refusal to augering 1.52							
- Start of diamond drilling							
BEDROCK: 175mm of faintly weathered to fresh black shale over faintly weathered to fresh black-grey dolomite		RC	1	94	39	2	72.93
		RC	2	100	94	3	71.93
3.12 End of Borehole							
(GWL @ 1.09m-Feb. 16/07)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Residential Development, Kenwood Avenue
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant, intersection of Kenwood Avenue and Melbourne Avenue. Assumed geodetic elevation = 76.17m.

REMARKS

BORINGS BY: CME 75 Power Auger

DATE: 6 FEB 07

FILE NO. PG0949

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.15				
Brown SILTY SAND, trace gravel	0.71				
BEDROCK: Weathered, black shale fragments with sandy silt seams throughout - Practical refusal to augering	1.40				
- Start of diamond drilling BEDROCK: Faintly weathered to fresh, grey-black dolomite - 45 degree fracture @ 1575 and 1625mm from top of RC1	2.80	RC	1	100	87
End of Borehole (BH dry-Feb. 16/07)					

DEPTH (m)	ELEV. (m)
0	75.73
1	74.73
2	73.73

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Residential Development, Kenwood Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, intersection of Kenwood Avenue and Melbourne Avenue. Assumed geodetic elevation = 76.17m.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 6 FEB 07

FILE NO. PG0949

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	75.78		
25mm Asphaltic concrete over crushed stone FILL 0.18									
FILL: Brown silty sand with gravel 0.60		AU	1						
TOPSOIL 0.71									
BEDROCK: Weathered, black shale fragments with sandy silt seams throughout 1.47		SS	2	67	50+	1	74.78		
End of Borehole Practical refusal to augering @ 1.47m depth									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Residential Development, Kenwood Avenue
Ottawa, Ontario

DATUM: TBM - Top spindle of fire hydrant, intersection of Kenwood Avenue and Melbourne Avenue. Assumed geodetic elevation = 76.17m.

REMARKS

BORINGS BY: CME 75 Power Auger

DATE: 6 FEB 07

FILE NO. PG0949

HOLE NO. BH 5

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.08				
FILL: Brown silty sand with gravel	0.46				
Loose, brown SANDY SILT, trace organic matter	1.07	SS	1	25	5
BEDROCK: Weathered, black shale fragments with sandy silt seams	2.26	SS	2	44	50+
End of Borehole					
Practical refusal to augering @ 2.26m depth					

DEPTH (m)	ELEV. (m)
0	75.15
1	74.15
2	73.15

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

Piezometer Construction

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE & TEST DATA

Geotechnical Investigation
Proposed Residential Development, Kenwood Avenue
Ottawa, Ontario

DATUM TBM - Top spindle of fire hydrant, intersection of Kenwood Avenue and Melbourne Avenue. Assumed geodetic elevation = 76.17m.

REMARKS

BORINGS BY CME 75 Power Auger

DATE 6 FEB 07

FILE NO. PG0949

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	75.39		
25mm Asphaltic concrete over crushed stone FILL 0.20									
TOPSOIL 0.33									
Brown SANDY SILT 0.97		SS	1	44	50+				
BEDROCK: Weathered, black shale fragments with sandy silt seams 1.50						1	74.39		
End of Borehole Practical refusal to augering @ 1.50m depth									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SOIL DESCRIPTION (continued)

Cohesive soils can also classified according to their "sensitivity". The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called "mechanical breaks") are easily distinguishable from the normal in-situ fractures.

RQD %	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.) Rock core samples are obtained with the use of standard diamond drilling bits

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = $D60 / D10$

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sand and gravels not meeting the above requirements are poorly-graded or uniformly-graded. Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

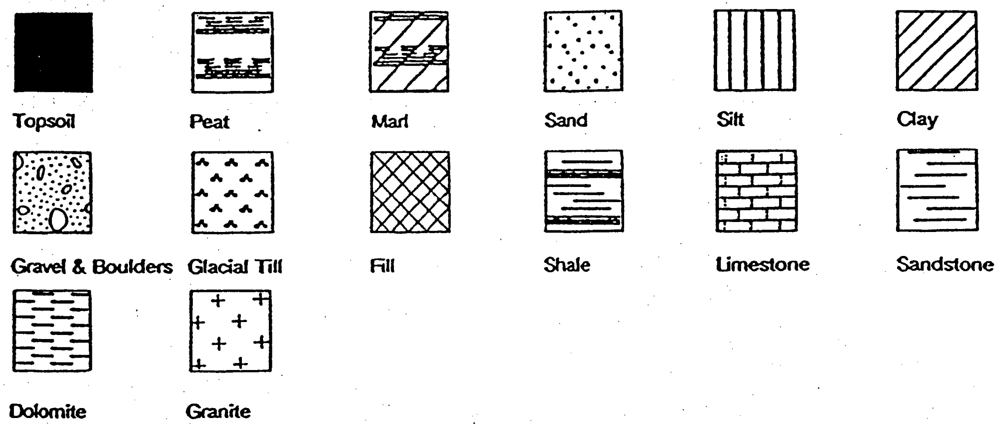
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidation ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

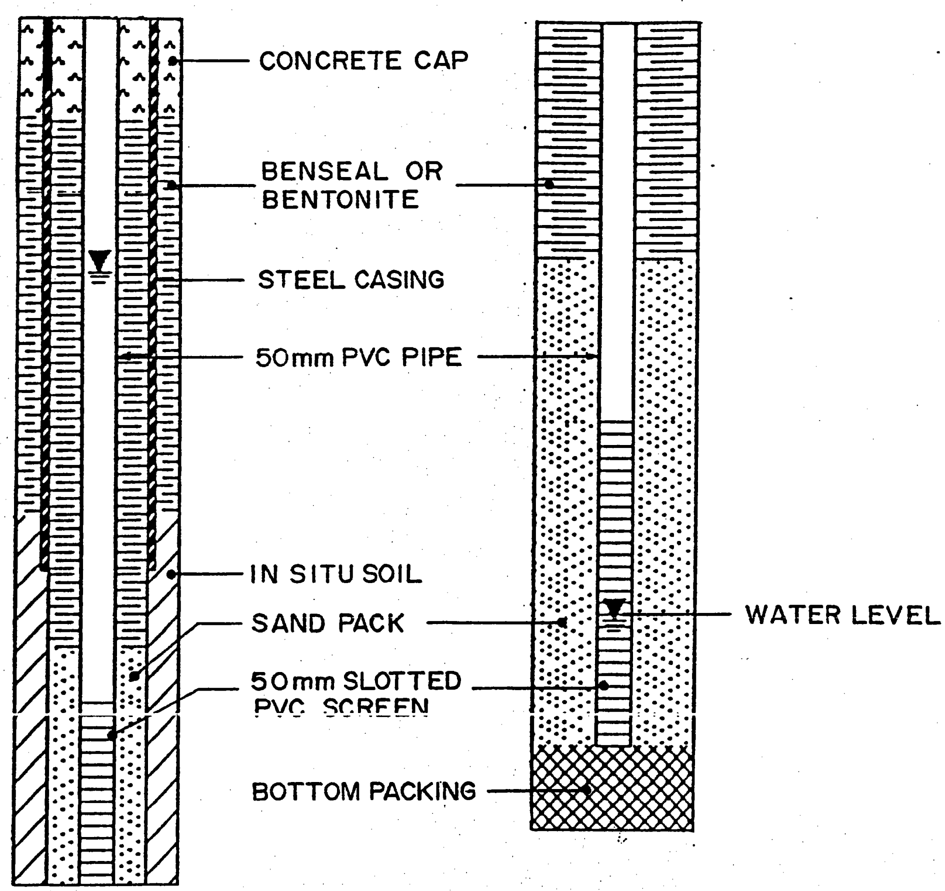
SYMBOLS AND TERMS (continued)

STRATA PLOT



MONITORING WELL AND PIEZOMETER CONSTRUCTION

Monitoring Well Construction **Piezometer Construction**



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - TYPICAL FOUNDATION INSULATION DETAILS-OPTION 1

FIGURE 3 - TYPICAL FOUNDATION INSULATION DETAILS-OPTION 2

TABLE A1 - INSULATED FOUNDATION DESIGN DATA SUMMARY

TABLE A2 - ALLOWABLE BEARING PRESSURES ON INSULATION BOARDS

DRAWING PG0949-1 - TEST HOLE LOCATION PLAN

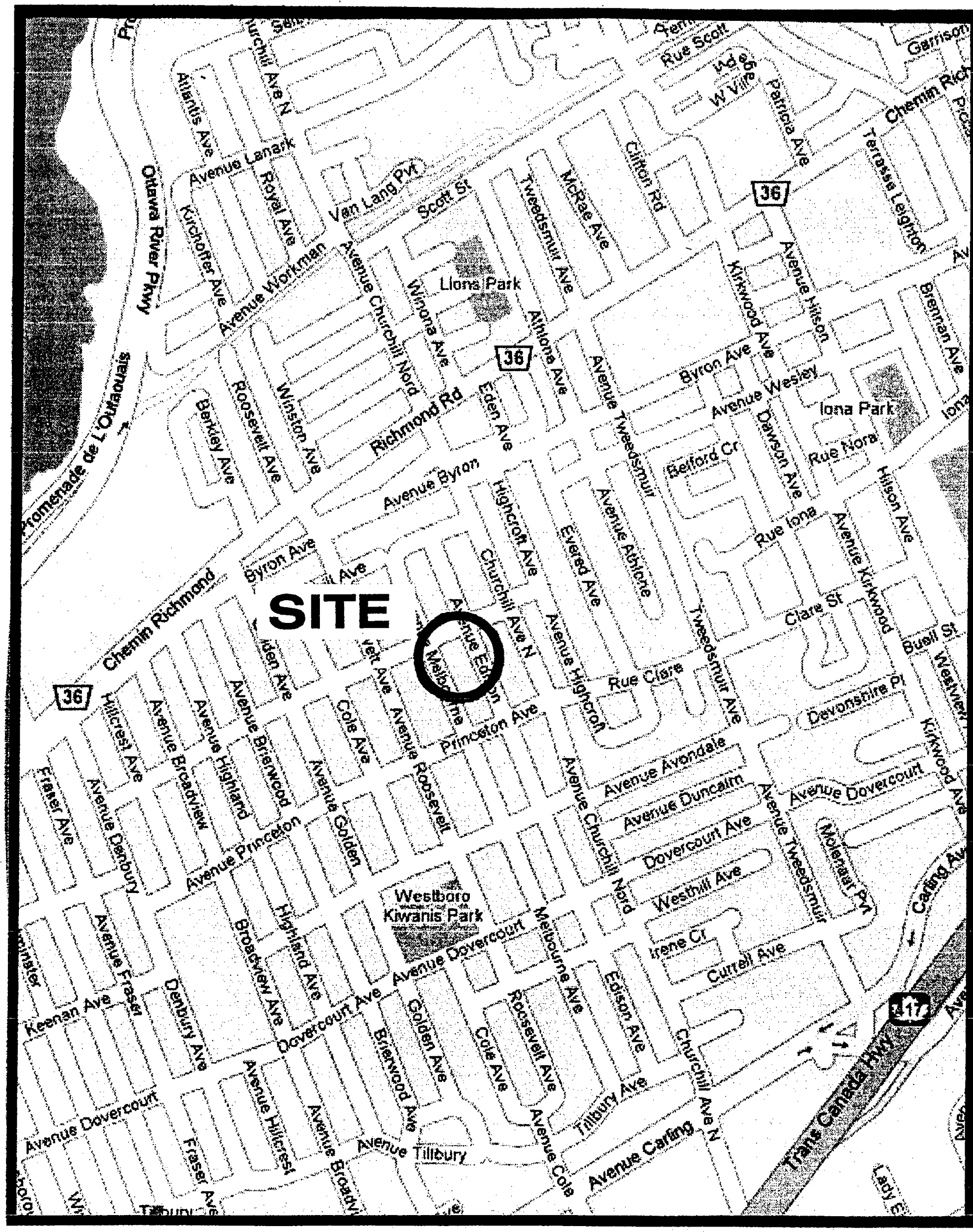


FIGURE 1
KEY PLAN

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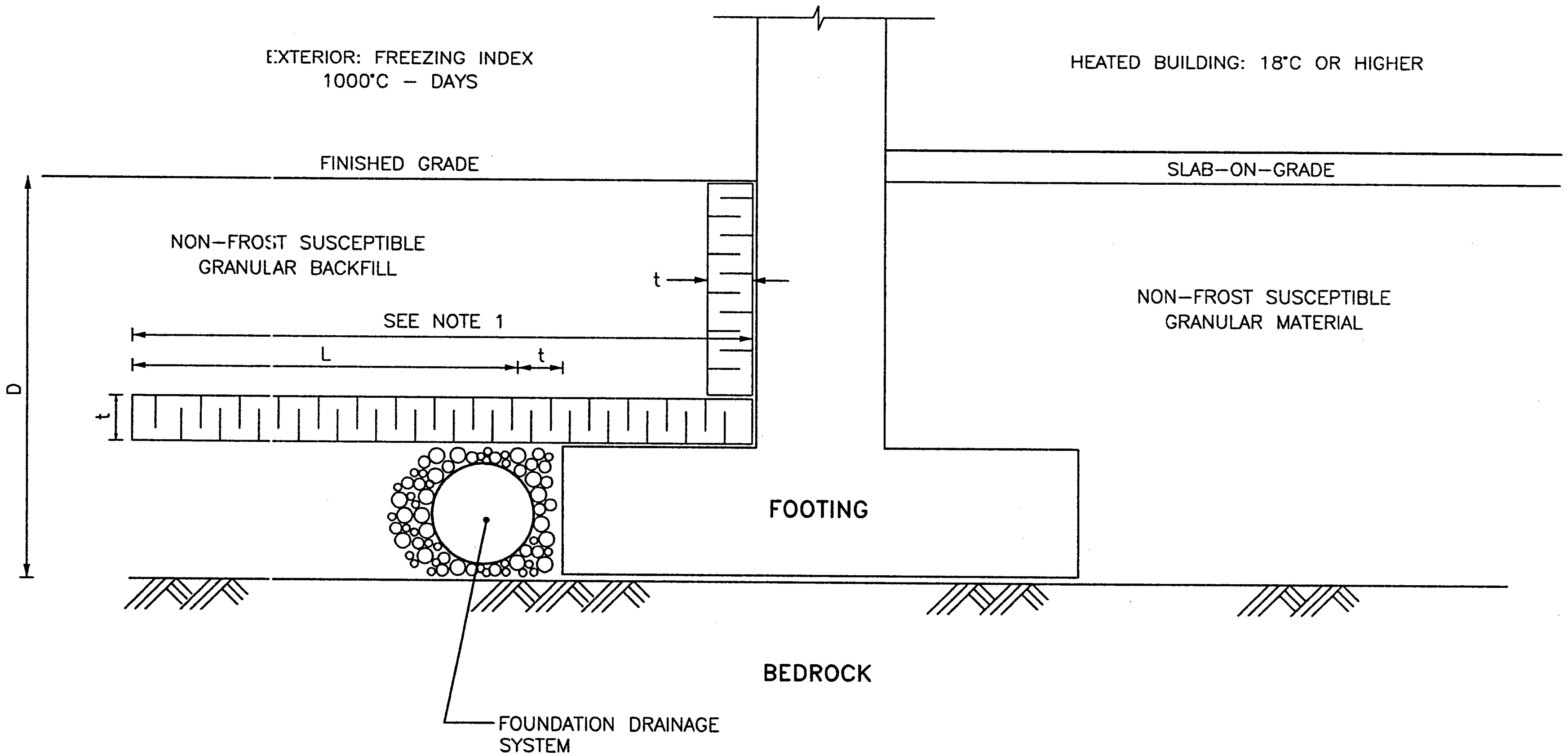
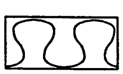


LEGEND:



STYROFOAM TYPE FROM TABLE A2 OR EQUIVALENT

STYROFOAM TYPE 'SM' OR EQUIVALENT

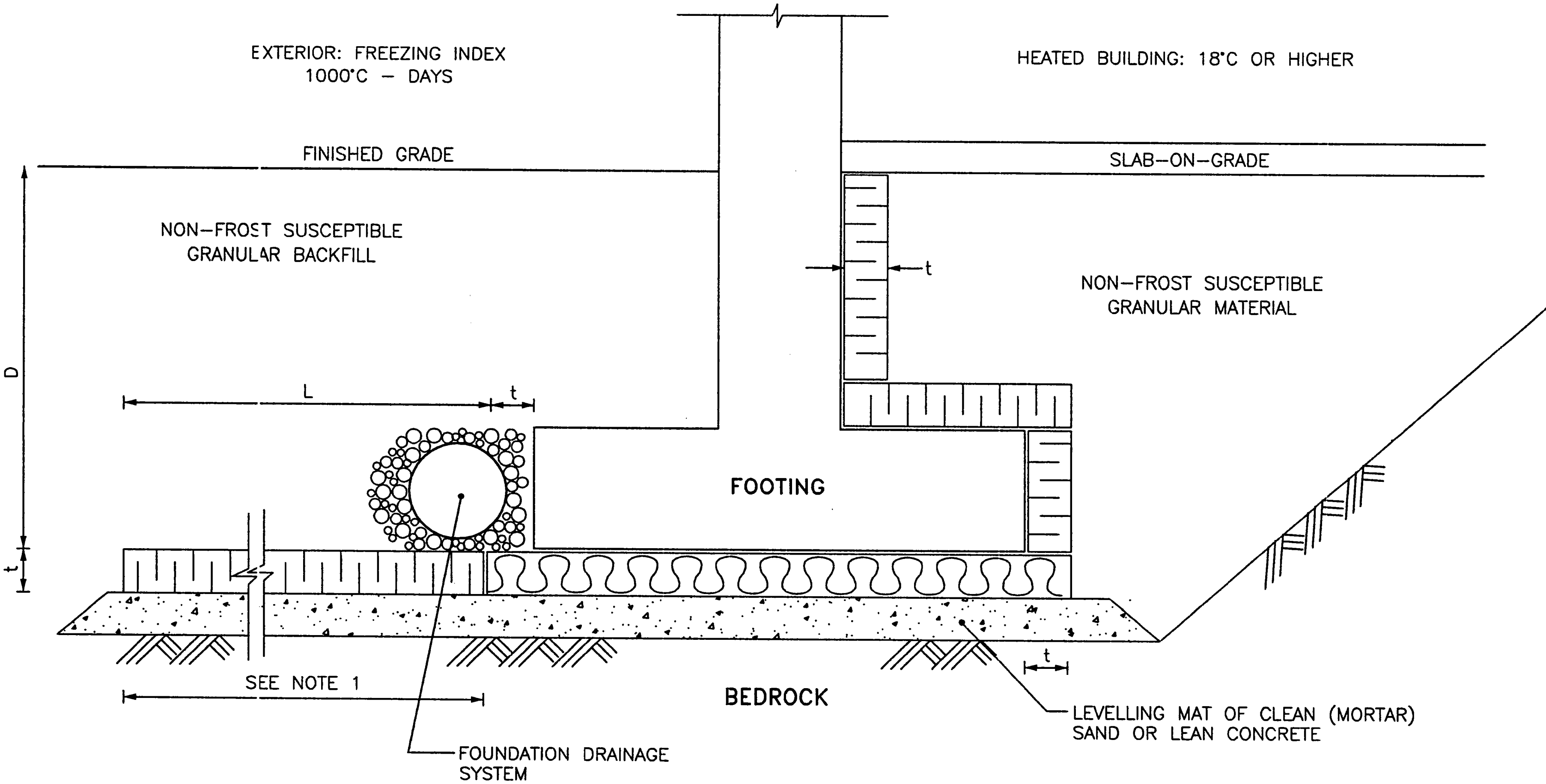
NOTES:

1) USE HIGH LOAD 40 OR EQUIVALENT IF PLACED BENEATH PAVED SURFACE AND D IS <1.0m.

2) DIMENSIONS L & t CAN BE DETERMINED FOR A GIVEN SOIL COVER, D, FROM TABLE A1 OF THE ACCOMPANYING REPORT.

3) NOT FOR CONSTRUCTION PURPOSES.

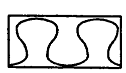
patersongroup consulting engineers	FIGURE 2 TYPICAL FOUNDATION INSULATION DETAIL-OPTION 1 HEATED FOOTINGS FOUNDED ON BEDROCK	Report No.: PG0949-1		Scale: N.T.S.
		Date: 02/2007	Drawn: MPG	Approved: GC



NOTES:

1) USE HIGH LOAD 40 OR EQUIVALENT IF PLACED BENEATH PAVED SURFACE AND D IS <1.0m.
2) DIMENSIONS L & t CAN BE DETERMINED FOR A GIVEN SOIL COVER, D, FROM TABLE A1 OF THE ACCOMPANYING REPORT.
3) NOT FOR CONSTRUCTION PURPOSES.

LEGEND:


STYROFOAM TYPE FROM TABLE A2 OR EQUIVALENT


STYROFOAM TYPE 'SM' OR EQUIVALENT

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FIGURE 3
TYPICAL FOUNDATION INSULATION DETAIL–OPTION 2
HEATED FOOTINGS FOUNDED ON BEDROCK

Report No.:	Scale:	
PG0949–1	N.T.S.	
Date: 02/2007	Drawn: MPG	Approved: GC

TABLE A1
INSULATED FOUNDATION DESIGN DATA SUMMARY

Thermal * Condition	Soil Cover Provided D (mm)	Insulation Dimensions (mm) **	
		t (thickness)	L (extension)
Heated (See Figures 2 and 3)	Less than 600	Not Recommended	
	600 to 750	50	1800
	750 to 900	50	1200
	900 to 1200	40	1200
	1200 to 1500	25	900
Unheated	Less than 300	Not Recommended	
	300 to 600	75	2400
	600 to 750	75	1800
	750 to 900	75	1500
	900 to 1200	65	1200
	1200 to 1800	50	600

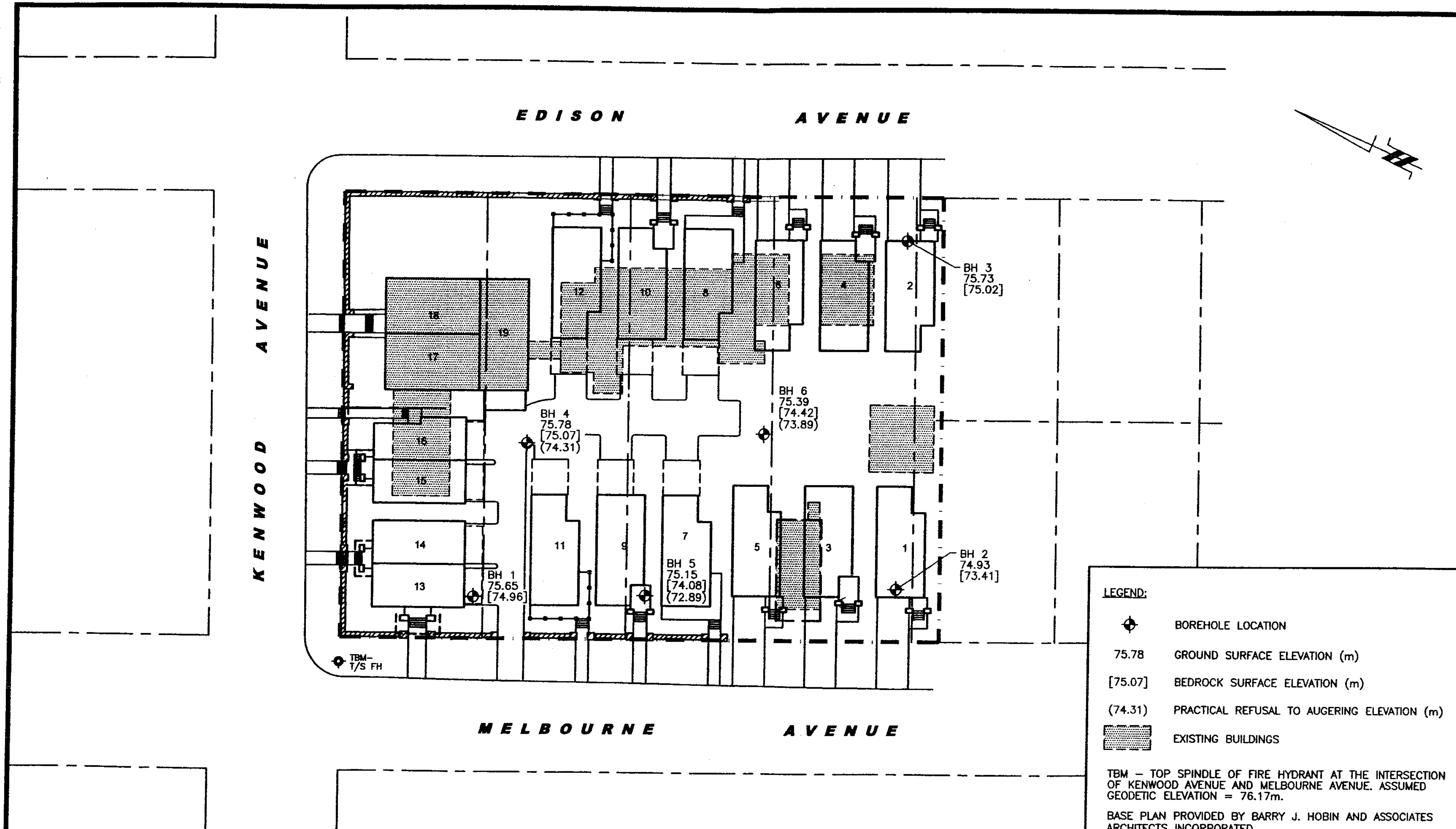
Notes: * All designs are based on a freezing index of 1000°C-days and a building temperature, for heated footings, of 18°C.

** Refer to Figures 2 and 3 for the definition of the insulation dimension terms t and L.

TABLE A2
ALLOWABLE BEARING PRESSURES ON INSULATION BOARDS

Insulation Type	Rated Compressive Strength		Allowable Bearing Capacity	
	psi	**kPa**	**psi**	**kPa**
Dow Chemical:				
SM	30	210	10	70
HL-40	40	280	13	90
HL-60	60	410	20	140
HL-100	100	690	33	230
Celfortec Inc.:				
Celfort 200	20	140	7	45
Celfort 300	30	210	10	70
Foamular 400	40	280	13	90
Foamular 600	60	410	20	140
Foamular 1000	100	690	33	230

Notes:
1. Specified allowable bearing capacities incorporate Dow Chemical Company's recommended factor of safety of 3.0 for sustained loading conditions where the number of loading cycles will be less than 1000. (i.e. dead loads and continuous live loads, where creep of the insulation will occur).
2. All insulation types have a thermal resistance value, RSI, of 0.87 (R=5 per inch).



LEGEND:

⊕	BOREHOLE LOCATION
75.78	GROUND SURFACE ELEVATION (m)
[75.07]	BEDROCK SURFACE ELEVATION (m)
(74.31)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
	EXISTING BUILDINGS

TBM – TOP SPINDLE OF FIRE HYDRANT AT THE INTERSECTION OF KENWOOD AVENUE AND MELBOURNE AVENUE. ASSUMED GEODETIC ELEVATION = 76.17m.

BASE PLAN PROVIDED BY BARRY J. HOBIN AND ASSOCIATES ARCHITECTS INCORPORATED.

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Scale:	1:500
Des.:	DG
Dwn:	MPG
Chkd:	GC

UNIFORM URBAN DEVELOPMENTS
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT, KENWOOD AVE.
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

Dwg. No.	PG0949-1
Report No.:	PG0949-1
Date:	02/2007

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