Trow

APPENDIX 3

Preliminary
Geotechnical Investigation
Proposed Waterfront Development
Between Point Lands and
Prince of Wales Park
St. Clair River
Sarnia, Ontario

Prepared for:

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LNEN00007659A
September 29, 2004

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Project: LNEN00007659A

September 29, 2004

Mr. Terry McCallum
City of Sarnia
P.O. Box 3018
255 North Christina Street
Sarnia, Ontario N7T 7N2

Dear Mr. McCallum:

Preliminary
Geotechnical Investigation
Proposed Waterfront Development
Between Point Lands and
Prince of Wales Park
St. Clair River
Sarnia, Ontario

1. Introduction

1.1 Project Description

This report presents the results of a geotechnical investigation carried out at the location of the proposed Sarnia Waterfront Development. Trow was retained by Mr. Terry McCallum of the City of Sarnia to carry out this work. It is understood that the proposed development will consist of three separate waterfront properties along the St. Clair River; one on Devine Street in the vicinity of the former CN Lands (Area A), one at George and Front Streets (Area B), and one on the Point Lands (Area C). Firm development plans were not available at the time of this writing, however it is assumed that the development at each site will consist of one or more structures. These structures will be serviced by underground utilities. It is also assumed that new parking lots and laneways will be planned.

1.2 Terms of Reference

The purpose of this investigation was to examine the subsoil and groundwater conditions at each site, by carrying out a limited number of widely spaced sampled boreholes at the locations designated by Trow and shown on the attached Site Plans (Drawing 1 and 1a, 1b, and 1c). Subsurface information obtained from these boreholes will also be used for the environmental site assessment which will be reported under separate cover. Based on an interpretation of the factual borehole data, Trow was to provide preliminary engineering recommendations for the geotechnical design of the proposed development. More specifically, comments on foundation options, allowable design bearing pressure, excavation and groundwater conditions, earth pressures, backfill, floor slab construction, permanent drainage and pavement design were to be provided.

This report is provided on the basis of the terms of reference presented above and on the assumption that the design will be in accordance with all applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional test holes and reporting before the recommendations of this office may be relied upon. The information in this report in no way reflects on the environmental aspects of the soil. Should specific information in this regard be needed, additional testing may be required.

2. Site

The proposed waterfront development is to be located at three sites along the St. Clair River and will access off of Devine Street, Front Street, and Seaway Drive. A key map of these locations is shown on Drawing 1. Plans were not finalized at the time of this reporting, however it is assumed that one or two storey structures with or without basements will be planned. It is understood that paved parking lots, and access roads and laneways will also be required at each of the three sites. Existing subsurface services are present throughout the development.

At present Area 'A' is being used as a concrete ready mix facility and the site is occupied by buildings, an open inground holding tank for water, and stockpiled granular materials. This site has a gradual slope towards the St. Clair River. Area 'B' is a relatively flat lot graveled parking lot, and Area 'C' is a grassed parkway. Area 'C' has several high landscaped berms, up to an approximate height of 9 m present throughout the site. All sites back on to the St. Clair River, and a sheet pile wall separates the property from the river. The difference in elevation between the current grade at the rear (river side) of each site and the river varies with location but is generally 2.4 m in height.

3. Procedure

The fieldwork for this investigation was carried out on September 9 and 10, 2004. At that time ten (10) sampled boreholes were drilled at the approximate locations shown on Drawings 1a, 1b and 1c. The borehole locations were restricted by the presence of underground utilities. The boreholes were advanced to depths of 6.5 m (21.5 ft) using continuous flight auger equipment operated by London Soil Test Ltd. In each borehole disturbed samples were recovered at regular depth intervals of 2.5 to 5.0 ft. (0.75 to 1.5 m) using split spoon sampling equipment and Standard Penetration test methods. Water levels were observed in the open boreholes at the time of drilling.

The fieldwork was supervised by a member of the Trow engineering staff who directed the drilling and sampling operations, logged the boreholes, and observed groundwater levels. All recovered samples were transported to a Trow laboratory for detailed examination and routine testing including moisture content and unit weight determinations.

The borehole locations were established in the field by Trow representatives. The ground surface elevation for boreholes at Area 'A' were referenced to a temporary site benchmark set on the finished floor of the existing maintenance shop [Assumed Elevation 182.3 m (598.1 ft)] and Area 'C' were referenced to a temporary benchmark set on the concrete dock on the northeast corner of Seaway Road [Assumed Elevation 177.6 m (582.7 ft)]. The ground surface elevations for Area 'B' were interpreted from an Ontario Base Map.

4. Subsurface Conditions

The information on subsurface conditions obtained from this investigation is consistent with findings from previous investigations on nearby sites.

4.1 Soil Stratigraphy

The detailed stratigraphy encountered in each borehole and the results of routine laboratory tests carried out on selected samples of the subsoil are given on the attached borehole logs (Appendix A). It must be noted that boundaries of soil indicated on the borehole logs are inferred from non continuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purposes of geotechnical design and should not be interpreted as exact planes of geological change. A brief summary of the soil stratigraphy encountered in the borehole locations follows. For ease of interpretation of borehole data, the development has been broken into three sites.

Area 'A' – Devine Street, west of Prince of Wales Park

Boreholes 101 to 105

GRANULAR FILL

All boreholes encountered granular fill material to depths varying from depths of 0.8 to 2.0 m (2.5 to 65. ft) below current grades. The granular fill material contains some deleterious materials with some black staining, is moist, and is in a very loose to compact state. The granular material is likely associated with the storage yard of the existing redi-mix concrete plant.

FILL

Beneath the surficial granular fill material, Boreholes 104 and 105 encountered a fill material comprised of trace to some silts and sands intermixed with cinders, wood, and containing areas of black staining. This soil matrix is very loose with some compact zones as demonstrated by blow counts of N=2 to 27 obtained in the Standard penetration test. The in-situ moist indicates the soil is moist to wet, laboratory results are reported on the borehole logs. Borehole 104 was terminated in this fill material.

SILTY SAND/SANDY SILT

Underlying the granular fill material, Boreholes 103 and 102 encountered a layer of silts and sands to depths of 4 m (13 ft) and to termination respectively. Blow counts of N=6 to 27 obtained from Standard Penetration testing within this deposit suggests that the relative density is loose to compact. This granular soil is moist with the moisture content increasing with increased depth.

SILTY CLAY

Beneath the fill materials and silty sand where encountered, Boreholes 101, 103 and 105 encountered and were terminated in clayey silt. This deposit is generally moist, and the laboratory results are recorded on the borehole logs. The competence of this deposit is demonstrated by Standard Penetration test blow counts of N=6 to 27, indicating a consistency of firm to very stiff.

Area 'B' – George and Front Streets

Boreholes 201 and 202

GRANULAR FILL

Boreholes 201 and 202 encountered a layer of sand and gravel fill material to an approximate depth of 1.8 m (6 ft) below current grades. This material is associated with the existing on grade graveled parking lot. The fill material contains some asphalt, trace organics, with some areas of black staining, is moist, and is loose as determined by blow counts of N=4 to 9 obtained in the Standard Penetration test.

SAND

Beneath the surficial material both boreholes encountered a layer of sand. The thickness of this material was approximately 1.2 m (4 ft) in Borehole 202, and Borehole 201 was terminated in this stratum. This material is loose to very loose as demonstrated by blow counts of N=2 to 8 obtained in Standard Penetration tests. The moisture content increases with increased depth, and the results are presented on the borehole logs.

SILTY CLAY

Borehole 202 encountered clayey silt below the sand stratum and was terminated in this material. This material is of glacial origin and the consistency is considered very soft to firm as demonstrated by results obtained in the Standard Penetration test of N=2 to 6. The soil is moist and the laboratory results of the natural moisture contents are presented on the borehole log.

Area 'C' – Point Lands, West of Sarnia Bay Marina, South of Seaway Road

Boreholes 301 to 303

TOPSOIL

Topsoil was encountered in each borehole and ranges in thickness from 50 to 150 mm (2 to 6 inches.

GRANULAR FILL

Beneath the surficial topsoil veneer, granular fill was encountered to an approximate depth of 760 mm (2.5 ft) below current grade in each borehole.

SAND

Beneath the surficial topsoil and granular fill materials, each borehole encountered and was terminated in a sand stratum. This alluvial deposit was silty in the upper level with some gravel. Blow counts of N=4 to 58 obtained from Standard penetration testing within the deposit indicate the relative density of this soil varies with depth and location and is loose to very dense. Generally this stratum is compact with some loose and very dense zones. This material is moist in the upper level and the natural moisture content increases with increased depth. The moisture contents are recorded on the borehole logs.

4.2 Groundwater

Insufficient time was available to establish the stabilized groundwater level. However, the groundwater level is expected to be where the soil turns grey near 3.0 m, 2.0 m, and 3.0 to 4.5 m depth below current grades for Areas 'A', 'B', and 'C' respectively; and is expected to be near the elevation of phreatic surface of the St. Clair River.

Short term groundwater level observations and measurements, and moisture contents of recovered soil samples are recorded on the borehole logs. Fluctuations in water level should be expected with high water levels expected during periods of wet weather. Capillary rise effect is also expected within fine grained deposits.

4.3 Methane Gas

M.S.A. Explosimeter readings taken in the upper levels of the open boreholes at the time of drilling did not detect any significant concentration of methane gas.

5. Preliminary Discussion

A subsurface investigation has been completed for the proposed waterfront development. At present, the proposed sites are either landscaped, graveled parking, or work/storage yard. Some existing structures will have to be demolished in Area 'A', and it is assumed that the high landscaped berms will be removed on Area 'C'. There are underground utilities and services located within the sites.

Foundation options, groundwater and excavation conditions, and other items related to building construction and pavement structures are discussed in the sections which follow. The groundwater depth below current grades vary from area to area. Seasonal groundwater fluctuation of 0.6 to 0.9 m (2 to 3 ft) can be expected.

The following preliminary comments are provided for use as general guidelines. More detailed geotechnical information for buildings and pavement structures at specific locations on each site can be obtained when additional details and finalized plans become available. Additional test holes and reporting may be required when preliminary design information becomes available before the recommendations of this report may be relied upon.

5.1 Foundations

From the borehole findings and past experience, foundation support on conventional footings set on the natural soils is practical in most areas. Consideration can also be given to footing support on a raft of engineered fill placed on the natural undisturbed soil for restoring subgrade purpose in areas where it is desired to raise the grades. A deep foundation option such as driven pile support may also be considered.

Each foundation scheme should be considered in the design stage from feasibility, economical and practical standpoints. A combination of options may also be considered. Fulltime geotechnical inspection must be provided by Trow to validate the bearing capacity and confirm the current soil conditions of the site during construction.

5.1.1 Conventional Shallow Foundations

The proposed buildings can be supported on conventional shallow foundations set below all topsoil and fill, on the natural undisturbed soil. An allowable bearing pressure of 145 kPa (3000 psf) can be used for design of footings set on the natural undisturbed soils in Areas 'A' and 'C'; and an allowable bearing pressure of 50 kPa (1000 psf) in Area 'B'. In Area 'A' competent soil was not encountered near the waterfront in Borehole 104, and the depth to competent soil in Borehole 105 was too deep for conventional foundations to be feasible. A list of the depths at which competent soil was found at the borehole locations for each of the three proposed areas is presented in Tables B1, B2, and B2 in Appendix B. The bearing capacities listed in Appendix B can be assumed provided that the lower, less competent natural soils are not overstressed. Refer to Appendix B 'Check for Overstressing' for additional details in this regard.

The actual founding levels will be affected by the existing structures and services. A review of the levels and locations of the existing services and structures should be carried out as part of the design verification.

The base of excavations terminating in wet sandy or silty soil may tend to exhibit instability. To minimize disturbance to the base, no section of the excavation should be left open overnight. Given the sensitive nature of these subgrade soils care must be exercised during excavating operations. If extensive concrete steel reinforcement is required at the footings, a skim coat of lean mix concrete will be recommended over the base of excavation.

5.1.2 Engineered Fill

Footings can be supported on a raft of engineered fill placed on the natural undisturbed soil for restoring subgrade purposes in areas where it may be desired to raise grades in Areas 'A' and 'C'. Engineered fill is not a feasible alternative in Area B due to overstressing of the weaker underlying soil. If engineered fill is used in Areas 'A' and 'C', after the grades have been finalized, checks should be made to ensure the underlying soil is not overstressed. Refer to Appendix B 'Check for Overstressing' for additional details in this regard.

An allowable bearing pressure of 145 kPa (3000 psf) can be used for footings set on the engineered fill in Areas 'A' and 'C' provided that a minimum thickness of fill equivalent to one footing width for strip footing and one half footing width for spread footings is used. General requirements for foundations set on engineered fill are detailed in Drawing No. 2. The engineered fill material should consist of an approved granular fill material such as an OPSS Granular 'B'.

5.1.3 Raft Foundation

The use of a raft foundation may also be considered suitable for Area 'B' of this proposed development. This foundation scheme would require the subexcavation of any fill material and organic soils. An allowable bearing capacity of 50 kPa (1000 psf) can be assumed for a raft foundation set on the undisturbed natural sand at a depth of 2 m below current grade. The excavations are expected to extend to, or slightly below, the stabilized groundwater level.

The base of the raft foundation should be placed at least 1.2 m (4 ft) below finished grade, to provide suitable frost protection; and designed to resist hydrostatic uplift forces caused by the groundwater condition. The mass of the raft foundation and proposed building are expected to be sufficient to counteract the buoyancy forces caused by hydrostatic uplift. However, the raft foundation should be designed by a professional structural engineer.

Additional recommendations for excavations, dewatering, and hydrostatic and uplift forces can be provided if this foundation option is considered. An ongoing liaison with this office will be required.

5.1.4 Driven Piles

Based on information obtained from this investigation and previous investigations in the area, deep foundation options such as driven piles may also be considered for these sites if a higher bearing capacity is required. This foundation option, if used with basementless construction, would allow the unsuitable soils to remain inplace and would not involve extensive groundwater dewatering.

The proposed structure may be supported on a structural slab or pile caps set onto pressure treated timber piles, or high capacity H-piles driven into the competent natural soils or bedrock anticipated to be up to 36 m (120 ft) below current grade. The actual pile driving criteria would depend on the driving hammer, the block, and the pile size. This criteria would need to be finalized when this information is available. Variation in actual field values for individual piles should be anticipated. Inspection of the pile installation is recommended to ensure proper pile placement and penetration into firm bearing soils, to minimize the occurrence to under-driving, and to maintain adequate records of the installation.

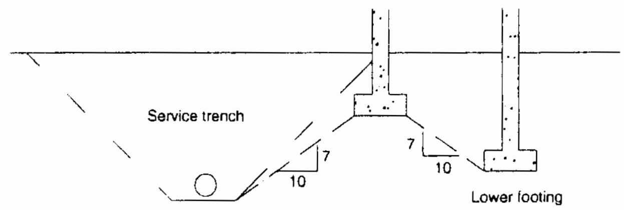
The effect of vibrations due to the pile driving on adjacent existing structures must be taken into account. This may present problems to existing adjacent structures if they are founded on the loose or soft soils. For instance, this may require the piles to be preaugered. It is suggested that vibration measurements be done while driving the first few piles to determine if any special measures are needed during installation of the piles. A preconstruction condition survey of all adjacent services and structures is also recommended.

If a deep foundation option such as driven piles is to be considered additional site investigation and reporting will be required to confirm the depth to competent soil and to provide appropriate geotechnical parameters. An ongoing liaison with this office will be required.

5.2 Foundations General

The foundation unit(s) for any basements or a raft foundation will likely be at or below the groundwater level. The basement slab and foundation units for basements not set a minimum of 500 mm above the stabilized water table should be waterproofed to minimize groundwater ingress into the basement. The foundation must be designed in accordance with the current Ontario Building Code and to resist hydrostatic pressure and uplift forces. Water stops and properly constructed joints will be required to provide a consistent surface to receive the water proofing system.

Footings at different elevations should be located such that the higher footings are set below a line drawn up at 10 horizontal to 7 vertical from the near edge of the lower footing. The lower footings should be constructed first to avoid undermining of higher footings.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

All footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.2 m (4 ft) of soil cover or equivalent insulation.

Provided that the footing bases are not disturbed due to construction activity, precipitation, freezing and thawing action, etc., and the aforementioned bearing pressures are not exceeded, the total and differential settlements of footings designed in accordance with the recommendations of this report and with careful attention to construction detail are expected to be within 25 mm and 19 mm respectively.

It should be noted that the recommended bearing capacities have been calculated by Trow from the test pit information for the design stage only. This investigation and comments are necessarily on-going as new information of underground conditions becomes available. For example, when more specific information is available with respect to conditions between test pits, when the foundation construction is underway. The interpretation between the test pits and the recommendations of this report must therefore be checked through field inspections provided by Trow to validate the information for use during the construction stage.

5.3 Excavations

The anticipated depth of excavation for any proposed buildings with or without a basement will likely be less than 2.5 m. The shallow excavations to install footings for the proposed structures and underground services are expected to extend through and terminate within the native sands, silts, and silty clay.

Provided surface water is directed away, significant water flow into excavations is not expected for the excavation depth anticipated (i.e. up to 2.5 m) in Area 'A'. Any seepage of water in the surficial fill in Area 'A' should be controllable by standard sump pumping techniques in oversized excavation. To assist in maintaining stability of side slopes, particularly in areas of silty soils, it is suggested that the cuts be blanketed with free draining granular soil. Perimeter ditching will be required to form 'buffers' along the base of the excavation. Some suggestions for excavations slightly below the groundwater table in granular soils, and for the control of groundwater into excavations using trenching technique are provided on Drawing 3.

If deeper excavations are required, or for shallow excavation depths that are anticipated to be at or below the groundwater table (such as Areas 'B' & 'C'), positive groundwater control techniques will be required to maintain a dry and stable base. An on going liaison will be required from this office.

Side slopes of temporary excavations must conform to the Ontario Health and Safety Regulations. The majority of the natural sands and silts above the groundwater table and the silty clay can be classified as Type 2 soil, and fill materials and the natural sands and silts below the groundwater table can be classified as Type 3 soil. For guidance in this regard, temporary slopes cut at 45 degrees to the horizontal should remain stable through the short construction period above the ground water table. In wet sandy soil, and the loose fill, side slopes may slough to flatter angles of about 2 to 3H to 1V.

If the above slopes cannot be achieved due to space restrictions, the foundations and underground services may be installed within confines of a prefabricated or engineered support system.

5.4 Site Preparation and Backfill

Preparation of the subgrade in the building and parking areas of each site is a key aspect of the construction.

It is expected that existing and possibly abandoned structures and services will have to be demolished or removed; and the remainder of the construction zones on the site will be stripped to prepare a uniform subgrade. Topsoil or any unsuitable material should be removed prior to placement of any fill materials. Special measures will be required to ensure an acceptable subgrade condition in settlement sensitive zones such as building and parking areas.

Old excavations for previous structures and services etc., where they occur in settlement sensitive zones such as parking lots and underfloor fill zones, must be backfilled with approved material compacted to at least 98 percent Standard Proctor dry density. Existing foundations and structures should be demolished to at least 0.9 m below subgrade levels. A program of inspection and testing should be carried out by this office to ensure that backfill is adequately compacted. Rubble from demolishing the existing foundation should not be dumped in settlement sensitive zones but may be used to raise the grade in landscaped areas, or should be removed from the site. Organic-contaminated or other deleterious material should be discarded. Removal and disposal of deleterious materials should conform to the Ministry of Environment guidelines.

Backfill to satisfy underfloor, parking grade requirements, and in service trenches, etc. should be clean (i.e. free from topsoil and other undesirable contaminants), compactible and within 3 percent of the optimum moisture as determined in the Standard Proctor test. This material should be placed in loose lifts not exceeding 12 inches (300 mm) and uniformly compacted to at least 98 percent of the Standard Proctor density. Due to the sensitive nature of the silty soils in some areas, non vibratory equipment may be required when compacting the initial lifts.

The upper level of excavated natural silty sand or silty clay, and gravel fill (where encountered)to be excavated, if maintained at moisture contents existing at the time of this investigation, should be compactible and may be used for construction backfill. It is expected that as the excavation depth increases, the soils may be too wet to allow for adequate compaction and have to be discarded or used for landscaping. The native silty clay is sensitive to moisture changes and if allowed to become wet may become impossible to compact to high densities, and may have to be discarded. It is recommended that any wet granular soils and the silty clay be stockpiled separately to minimize excess moisture transferring to the till from the granular soils. Stockpiling of natural material for long periods of time should therefore be avoided. Where free draining material is required, and for backfill in confined areas, the use of OPSS Granular 'B' is recommended.

5.5 Slab On Grade Construction

The lowest floor can be cast as slab-on-grade on the natural undisturbed soil provided the subgrade is stripped of all topsoil, contaminated fill, and other obviously unsuitable material. The exposed area should be thoroughly proofrolled with a heavy roller and any soft spots detected by this or any other means should be dug out and made good with compactible fill. Fill needed to satisfy the design grade should conform to the requirements outlined in the preceding sections.

A moisture barrier consisting of at least 200 mm (8 inches) thick compacted layer of 19 mm (3/4 in.) clear stone, should be placed between the prepared subgrade and the floor slab. Alternatively, a less desirable option is to place 300 mm of compacted granular base material.

5.6 Permanent Drainage Requirements

Suggestions for perimeter drainage are presented on Drawings 4 and 5 for two building configurations, assuming the underside of footing level is a minimum of 500 mm above the groundwater table. As indicated these drains can be eliminated if the lowest floor level is at least 300 mm (12 in) above the exterior grade and this grade is contoured away from the building.

If a substructure is planned, perimeter drains with sumps may be required in some areas, of high groundwater table. Waterproofing with core drains on the wall will be recommended. Underfloor drains will be required for subgrade set on wet sandy soils. This can be best assessed when finalized design plans become available.

5.7 Earth Pressures

If a basement is planned for any proposed structures the substructure walls should be damp-proofed and designed to resist the lateral earth pressure calculated from the following equation:

$$p = K(\gamma h + q) + s$$

where p = lateral earth pressure in kPa (psf) acting at depth h;

K^* = earth pressure coefficient, assumed to be 0.35;

γ = unit weight of backfill, a value of 20.4 kN/m^3 (130 pcf) may be assumed h = depth to point of interest in m (ft) and,

q = equivalent value of any surcharge on the ground surface.

S = surcharge from slope and parking lot

The above expression assumes that the perimeter drainage system prevents the build up of any hydrostatic pressure behind the wall.

* A higher K value may be required for an inclined slope behind the foundation wall.

5.8 Parking Areas and Internal Access Roads

It is assumed that parking lots and site access roads and laneways will be planned.

5.8.1 Subgrade Preparation and Support

The long term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to insure uniform subgrade moisture and density conditions are achieved.

As part of the subgrade preparation, the proposed parking areas and associated access laneways should be stripped of topsoil, fill, and other obviously unsuitable material. For wet sandy and silty some form of subgrade improvement may be required in some areas. Also, fill material was encountered in each area of the proposed development. Ideally, all existing fill should be removed from the parking areas. Depending on the final design grade in this area, a program including partial removal of the existing fill can be applied. The subgrade conditions can be best assessed on site by a geotechnical engineer during construction when final design plans are available.

After the proposed subgrade is stripped, the area should then be proofrolled in the presence of a geotechnical engineer. Soft or spongy subgrade areas detected by proofrolling, or any other means, should be subexcavated and replaced with approved backfill compacted to at least 98 percent of Standard Proctor maximum dry density. All materials required to backfill service trenches or to raise the grades to design elevations should conform to backfill requirements outlined in the previous sections of this report. Preferably native compactable material should be used to ensure uniform subgrade and moisture conditions are achieved.

5.8.2 Drainage and Grading

Good surface drainage provisions will optimize pavement performance and the need for adequate drainage cannot be over-emphasized. The subgrade in areas to be paved should be crowned and shaped to promote drainage. The subbase layer should have positive drainage to perforated subdrains to intercept excess subsurface moisture and prevent subgrade softening. This is important for any silty clay subgrade, particularly in any heavy duty pavement areas. Assuming that cross fills in the order of two percent have been provided, short subdrains extending from catchbasins should be satisfactory. These drains should be operational prior to the placement of the pavement structure. The preferred drainage detail is given on Drawing 6. The location and extent of subdrainage required within paved areas should be reviewed by this office in conjunction with the proposed grading.

5.8.3 Flexible Pavement Design

The following information on pavement thickness is provided for preliminary design purposes only. The recommended pavement structures provided in Table 1 are based upon an estimate of the natural subgrade soil properties determined from visual examination and textural classification of the soil samples. Consequently, the recommended pavement structures should be considered for preliminary design purposes only. A function design life of about ten years has been used to establish the pavement recommendations. This represents the number of years to the first major rehabilitation, assuming regular maintenance is carried out. If required, a more refined pavement structure design can be performed based on specific traffic data and design life requirements and will involve specific laboratory tests to determine frost susceptibility and strength characteristics of the subgrade soils, as well as specific data input from the client.

TABLE 1

SUGGESTED FLEXIBLE PAVEMENT THICKNESS DESIGN

Layer	Material	Compaction	Recommended Thickness (mm)			
			Light Duty Cars		Heavy Duty Trucks	
Asphaltic Concrete	HL3	97% Marshall	30		40	
	HL8		40		60	
Granular Base	OPSS Granular 'A' or equivalent	100% Standard Proctor Density	150		150	
Granular Sub-base *	OPSS Granular 'B' or equivalent	100% Standard Proctor Density	Subgrade Soil		Subgrade Soil	
			Sands	Silty clay or Fill	Sands	Silty clay or fill
			300	350	400	450

Notes:

* A program of partial removal of the unsuitable subgrade soil may be needed. An implementation of geotextile may be required to provide subgrade strengthening and separation.

1. Granular base/sub-base compacted to 100% Standard Proctor maximum dry density. Asphaltic concrete compacted per MTO/OPS requirements
2. A program of in-place density testing must be carried out to verify that satisfactory levels of compaction are being achieved in the trench fill.
3. To minimize the effects of differential settlements of pavement structure, it is recommended that wherever practical, placement of binder asphalt be delayed for about 6 months after the

granular sub-base is put down. The surface course asphalt should be delayed for a further one year.

4. If construction is undertaken under adverse weather conditions (wet/freezing) subgrade preparation and granular sub-base requirements should be reviewed by the geotechnical engineer.
5. These recommendations on thickness for flexible pavement design are not intended to support heavy and concentrated construction traffic particularly when only a portion of the present section is installed.

5.9 Methane Gas

M.S.A. Explosimeter testing carried out in the open test holes at the time of drilling did not detect any significant concentration of methane gas. Based on the present information, no special methane gas abatement measures are anticipated for this site.

6. General Comments

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

Trow Associates Inc. should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not afforded the privilege of making this review, Trow Associates Inc. will assume no responsibility for interpretation of the recommendations in this report.

Should you have any questions regarding this report, please do not hesitate to contact this office.

Yours truly,
Trow Associates Inc.

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Distribution: City of Sarnia – Mr. Terry McCallum

Attachments:
Drawings – 1 to 6
Appendix A – Borehole Logs
Appendix B - Founding Levels for Conventional Foundations
- Check for Overstressing