APPENDIX 'A'

GEOTECHNICAL REPORT

APPENDIX 'A' - GEOTECHNICAL REPORT

The geotechnical report is provided to aid in the Contractor's evaluation of the existing pavement structure and/or soil conditions. The information presented is considered accurate at the locations shown on the Drawings and at the time of drilling. However, variations in pavement structure and/or soil conditions may exist between test holes and fluctuations in groundwater levels can be expected seasonally and may occur as a result of construction activities. The nature and extent of variations may not become evident until construction commences.

STANTEC CONSULTING LTD.

Additional Geotechnical Investigation for Kenaston Underpass Project

Prepared For:
Stantec Consulting Ltd.

Prepared By:
UMA Engineering Ltd.
1479 Buffalo Place
Winnipeg, Manitoba, R3T 1L7

UMA Job Number: 4231 040 09 (4.6.1)

March, 2005

uma
An AECOM Company

March 7, 2005

UMA File: 4231 040 09 (4.6.1)

Stantec Consulting Ltd.
905 Waverley Street
Winnipeg, Manitoba
R3T 5P4

Attention:Mr. Gil Mourant, P.Eng.

Dear Sir:

Reference: Kenaston Underpass
Additional Geotechnical Investigation

Please find enclosed three copies our geotechnical report for the Kenaston Underpass.

Should you have any questions or wish to review any part of the report, please do not hesitate to contact Giovanni Militano or Jeff Tallin.

Sincerely,

UMA ENGINEERING LTD.

Larry Bielus, M.Sc., P.Eng.
Manager, Manitoba
Earth and Environmental
/dh

cc: Bruce Bigelow

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1.0 INTRODUCTION AND BACKGROUND

1.1 TERMS OF REFERENCE

This report presents the results of our geotechnical investigation for the Kenaston Boulevard underpass at the CN Rail tracks near Wilkes Avenue. Our November 3, 2004 letter to Mr. Gil Mourant, P.Eng. outlined our proposed scope for the investigation.

This investigation supplements the following investigations conducted for the project:

- Geotechnical Investigation for the Proposed Route 90 CN Underpass near Wilkes Avenue, by I.D. Engineering, September 27, 1987.
- Kenaston Blvd. Grade Separation – Geotechnical Investigation, by UMA Engineering Ltd., memorandum to Bruce Biglow, P.Eng., March 8, 2004.

1.2 SCOPE OF INVESTIGATION

This investigation was conducted to obtain the geotechnical information required to provide additional information and recommendations pertaining to the following components of the project:

- Rail bridge foundations, specifically an evaluation of the feasibility of rock socketed caissons as an alternative to driven precast concrete piles. Caissons extending to pier caps below the girders have been proposed as an alternative to full height piers supported on driven piles, because caissons would facilitate construction of the entire bridge prior to excavation of the underpass and minimize the length of time that the rail detour will be in service.
- Cut slopes for the underpass excavation such that acceptable levels of performance can be expected and that the risks to other infrastructure (watermains, sewer and gas lines) are mitigated.
- Design of the underpass drainage lift station foundations.
- The Kenaston Boulevard and CN detours and the slip ramp.
- A portion of Sterling Lyon Parkway West which was realigned since our initial geotechnical investigation for this portion of road.

An investigation of the granular materials used to construct the former CN intermodal terminal was also conducted to evaluate potential uses of these materials for the project. The results of this investigation were reported separately, in a memorandum dated January 31, 2005 to Bruce Biglow, P.Eng.

Memorandums dated February 4 and 14, 2005 to Bruce Biglow, P.Eng. provided our preliminary recommendations regarding the rail bridge foundations and lateral earth pressures.

Slope stability issues were first addressed by UMA in a memorandum to Gil Mourant dated September 7, 2004.

2.0 FIELD AND LABORATORY INVESTIGATION

2.1 TEST DRILLING PROGRAM

The test drilling program included test holes at the locations shown on Figures 1 and 2. Drilling was performed on December 6 through 13, 2004 by Paddock Drilling, using a truck mounted drill rig. Most of the test holes were drilled with 125 mm diameter solid stem augers. Hollow stem augers were used where sloughing conditions prevented drilling with solid stem augers. Table 2.1 summarizes the test drilling program.

Table 2.1 – Test Hole Summary

Component	Test Holes	Termination Depth (elev. m)	Instrumentation ID (elev. m)	Comments
Sterling Lyon West	TH-04-21	4.6 m (229.2)		
	TH-04-22	4.6 m (229.5)	SP-04-22 (229.5)	SP in clay
	TH-04-23	4.6 m (229.3)		
	TH-04-24	4.6 m (229.4)		
	TH-04-25	4.6 m (229.1)	SP-04-22 (229.1)	SP in clay
Underpass Sideslopes	TH-04-28	4.6 m (229.4)	SP-04-28 (229.4)	
	TH-04-29	10.7 m (223.3)	VW-04-29A (224.1)	VW in clay
			VW-04-29B (227.4)	VW in clay
	TH-04-30	13.1 m (221.2)	SP-04-30A (221.2)	SP till
			VW-04-30B (225.8)	VW in clay
			VW-04-30C (229.7)	VW in clay
	TH-04-31	4.6 m (228.7)	SP-04-31 (228.7)	
Kenaston Detour and Slip Ramp	TH-04-32	4.6 m (229.5)		
	TH-04-33	4.6 m (229.4)		
	TH-04-34	4.6 m (229.3)	SP-04-34 (229.3)	SP in clay & silt
	TH-04-35	9.2 m (224.3)	PN-04-35 (224.3)	PN in clay
	TH-04-36	4.6 m (228.5)	SP-04-36 (228.5)	SP in clay & silt
LDS Lift Station	TH-04-41	13.4 m (220.6)		Auger to refusal in till
Underpass Bridge Foundation	TH-04-42	37.2 m (197)	SP-04-42 (207.8)	Auger to refusal in till at 13.4 m Cored bedrock to 37.2 m SP in bedrock

Notes:
TH – Test hole identification
SP - Standpipe piezometer, 25 mm diameter PVC pipe.
VW - Vibrating wire piezometer
PN - Pneumatic piezometer

Test hole logs are included in Appendix A. For completeness, the test hole logs from IDE's 1987 report and UMA's March 8, 2004 report are included in Appendices B and C, respectively.

Test holes TH-04-21 to 25 were drilled along the western end of Sterling Lyon Parkway, which had been realigned since our previous investigation, reported in March 2003.

Test holes TH-04-28 to 31 were drilled to install piezometers needed to characterize the groundwater regime at the site. The piezometric data was used in our analysis to determine safe sideslopes for the underpass cut.

The test holes along the Kenaston Boulevard detour and slip ramp (TH-04-32 and 34 to 36) were drilled to evaluate soil conditions along these roadways.

TH-04-41 was drilled at the location of the lift station to evaluate foundation options and construction conditions for this structure.

TH-04-42 was drilled to evaluate the feasibility of supporting the rail bridge on rock socketed caissons as an alternative to the driven precast concrete piles, used in the preliminary design. The test hole was advanced to power auger refusal in the till at a depth of 13.4 m and was completed by coring the till and bedrock to a depth of 37.2 m. The standpipe piezometer installed in this test hole measures the piezometric head in the bedrock. TH-04-41 supplements the four test holes (T.H.5, T.H.9, T.H.10, and T.H.10) drilled by IDE in the vicinity of the proposed rail bridge in 1987.

2.2 Laboratory Testing

Representative disturbed (auger cutting) samples and undisturbed (Shelby tube) samples were collected for testing at UMA's geotechnical laboratory. Testing included: determination of moisture content and Atterberg limits to assess consistency; Torvane and Labvane to evaluate undrained shear strength; and odometer tests to measure volume change parameters. At the time of this draft report the odometer tests had not been competed. Laboratory test results are included on the test hole logs in Appendix A.

2.3 MONITORING

The piezometers were monitored on December 16th and 23rd of 2004, and January 6th and 26th of 2005. Figures 3 and 4 are plots of the groundwater levels with time. The groundwater monitoring data is tabulated in Appendix D.

3.0 SOIL AND GROUNDWATER CONDITIONS

3.1 VICINITY OF UNDERPASS

3.1.1 Soil Profile

The generalized stratigraphy in the vicinity of the underpass is lacustrine silt and clay, over glacial till over limestone bedrock. At most locations topsoil and/or fills were encountered at the surface. Layers of silt of varying thickness were encountered in the upper 3.5 m of lacustrine deposits. A brief description of the subsurface conditions is as follows:

Topsoil and Fill

A thin (about 50 mm) layer of black organic clay or topsoil is present at most test holes at the surface. At TH-04-42 a 0.3 m thick layer of sand and gravel fill was encountered at the surface. Clay fill ranging in thickness from 0.2 m up to 2.0 m was present at most of the test hole locations. The clay fill is silty, medium plastic, very stiff, moist and brown.

Lacustrine Silt

Silt and clayey silt layers, varying in thickness from 50 mm to 2.9 m, occur in the upper 3.5 m of the lacustrine deposit. The silt and clayey silt layers tend to be soft and wet below about 1.5 m depth. The thicker silt layers below the water table produced seepage and sloughed during the test drilling.

Lacustrine Clay

The lacustrine clay is highly plastic with traces of silt inclusions. To a depth of about 4.5 m, it is stiff, brown and oxidized with sulphate inclusions and oxide stains. Below this depth it is grey and firm with silt inclusions generally less than 3 mm in thickness. The clay contained small stones and pebbles above the till contact.

Silt Till

Silt till underlies the grey clay at an average elevation of 223.4 m. In TH-04-42, the till was cored to the bedrock which was encountered at a depth of about 20 m below ground surface or elevation 214.4 m. The silt till is low plastic and contains traces of sand and trace gravel. The upper portion of the till (above approximately 11 to 13 m depth) contains some clay and is generally moist and loose, with moisture contents of 9 to 13 percent. Auger refusal with 400 mm diameter augers occurred at depths ranging from 13 to 14 m (IDE report 1987). The moisture content of the till below about 13 m ranges from 6 to 8 percent, representative of very dense till (hard pan).

Limestone Bedrock

Dolomitic limestone bedrock underlies the till at elevation 214.4 m at TH-04-04-42 and was cored to elevation 197 m . The upper 6 m of bedrock to about elevation 208 m is characterized as poor to very poor quality, unsound rock. This 6 m of bedrock is weathered, broken and highly fractured and has a Rock Quality Designation (RQD) between 20 and 46 percent. A 30 mm thick solution cavity, infilled with cemented silt, was identified at depth of about 25.7 m.

Below a depth of about 26.5 m (between elevations 208 m and 197 m) the bedrock is considered to be good quality rock with an RQD of 57 and 94 percent.

Carbonate bedrock can contain solution cavities, zones of poor broken rock and other discontinuities problematic to construction. Because these features occur unpredictably, it is not possible to fully identify their frequency or distribution during a geotechnical investigation.

3.1.2 Groundwater

The results of the groundwater monitoring are tabulated in Appendix D. Figure 3 is a plot of piezometric head measured in the piezometers versus time, for the vibrating wire and standpipe piezometers near the rail crossing. The data shows total head decreases with depth in the clay from a head of 232.0 m in the silt layers near the ground surface to a head of 228.3 m in the till. The head in the bedrock stabilized to between 224.9 m and 225.2 m during the monitoring period.

Figure 4 plots piezometric levels in the silt and clay near the ground surface (SP-04-3-28, SP-04-34, SP-04-36, SP-04-31). These piezometers stabilized at elevations between 231.0 m and 232.4 m or 1.5 to 2 m below ground surface, in February 2005.

The reported groundwater conditions were obtained during the period of this investigation from December 2004 to February 2005. It should be appreciated that the groundwater conditions may vary seasonally, over time, or as a result of construction activities on the site.

3.2 STERLING LYON PARKWAY WEST

The westerly extension of the proposed alignment of Sterling Lyon Parkway is generally open grassland and agricultural land. The CN rail line to the former WIT yard runs approximately parallel to the proposed road and is built up above grade, with ditches on either side. The proposed realignment will reduce the degree of curvature near the west connection of Sterling Lyon Parkway with Wilkes Avenue. The updated soil stratigraphy consists of topsoil and/or clay fill over lacustrine clay containing one or more silt layers up to 1.3 m thick. In TH-04-24 and TH-04-25 the silt layers extended to depths of 1.5 m to 2.5 m, which is similar to the occurrence of silt in TH51 to TH54 drilled previously in the general area but peat was not encountered at the surface in the recent test holes. Silt was not encountered in TH-04-21 to 23 near the west end of the former WIT yard. TH-04-21 to 25 should be used to assess subgrade conditions along the new alignment.

Two shallow standpipe piezometers (SP-04-22, SP-04-25) were installed along the Sterling Lyon Parkway West alignment. SP-04-3-25 is connected to the wet silt layers, while SP-04-22 is not. In both cases, their groundwater elevation is 232.9 m about one metre below ground surface. As previously noted, groundwater conditions may vary seasonally, annually, or as a result of construction activities.

The recommendations in Section 5 of our March 8, 2004 report are applicable to design and construction of the road.

4.0 UNDERPASS SLOPE STABILITY

4.1 Review Of Previous Stability Analysis

UMA has reviewed the sideslope recommendations and parameters used to conduct the stability analysis by IDE in 1987. The stability analysis conducted by IDE used strength parameters measured on samples of clay from the site and assumed groundwater conditions, as no site specific groundwater data was obtained. Table 4.1 summarizes IDE's sideslope recommendations for unimproved slopes (slopes cut without attempts to lower groundwater levels).

Table 4.1 – Cut Slope Recommended by IDE in 1987

Height of Cut Slope	Recommended Sideslope (Horiz.:Vert.)	Computed Factor of Safety
Less than 3 m	4:1	1.5 – 1.53
3 m to 5 m	5:1	1.32 – 1.5
5 m to 7 m	5:1 with a 5 m wide berm 3.5 m above the toe of slope	1.34 – 1.47

As an initial step in our investigation into sideslope gradients, we have verified the above calculated factors of safety by replicating the slope stability model and input parameters reported in IDE's 1987 report. The IDE report does not specifically state the design factor of safety, but based on the minimum factors of safety reported, it can be inferred that a factor of safety of about 1.3 was considered acceptable.

4.2 Design Factor Of Safety

We have consulted with the Water and Waste Department regarding the safety of slopes adjacent to the watermains to be re-routed around the underpass excavation. It is our understanding a watermain break due to slope movements could result in rapid flooding of the underpass and unacceptable risks to the project and the public. In order to maintain an acceptably low probability of slope failure and mitigate risk to the pipes, a design factor of safety (FS) of 1.5 for the cut slopes of the underpass excavation has been selected. In our September 7, 2004 memorandum a factor of safety of 1.5 was also recommended for slopes in the vicinity of Manitoba Hydro's gas mains. However, it is our understanding that the gas mains will be rerouted so as to be well removed from the underpass slopes.

4.3 Slope Stability Model

The underpass will have maximum cut depth, from top of pavement on Kenaston Boulevard to top of slope near existing ground surface, of approximately 6.5 m. Based on the preliminary design drawings, a 4.0 m wide bench to accommodate a sidewalk and bicycle path, and to improve slope stability, will be cut into the slope about 1/3 of the way up the slope from the toe. The slope stability calculations were conducted with combinations of sideslope gradients and cut depths from 3.5 m to 6.5 m. This approach is consistent with the analysis reported on in our September 7, 2004 memorandum to Gil Mourant, P.Eng.

Shear strength parameters of c=5 kPa and $\varnothing=14^0$ were used for the lacustrine clay. These shear strength parameters were judged to represent the lower bound shear strength at large strains for Winnipeg clay based on testing for the Floodway Expansion in 2003 and 2004. The shear strength parameters of c=5 kPa and $\varnothing=14^0$ are lower than the parameters of c=5 kPa and $\varnothing=17^0$ used by IDE in 1987, but are considered appropriate based on a comparison of the shear strength test results for the site (IDE 1987) and the testing conducted for the Floodway.

The till below the clay was modeled as a hard layer so that all trial slip surfaces in the slope stability model were forced through the lacustrine clay. Given the unpredictable distribution of silt and fill layers near the surface, these materials were not treated as independent soil layers. However since both the fills and silt layers are expected to have higher strengths than the clay, this was considered to be a conservative simplification.

The piezometric heads measured in the piezometers at the site and the head distribution used in the stability calculations are shown on Figure 5. It was reasoned that the total heads measured during the investigation in the clay would be near the median heads for the site, even though the groundwater levels in the bedrock and the surficial silt could fluctuate seasonally. A trend line representing the average distribution of heads measured in the clay was interpreted to represent the total head profile below the top of the slope and beyond the limits of the excavation. The piezometric head distribution below bottom of the cut in Figure 5 is the predicted long-term piezometric head distribution between the toe of the slope and the till. Beneath this type of excavation the long term pore water pressures are higher than the pore water pressures immediately after construction and therefore govern in design.

The above described piezometric head distribution is lower than the piezometric head distribution used in the calculations reported by IDE 1987, which assumed that the piezometric head increases hydrostatically with depth.

4.4 SLOPE STABILITY RESULTS

Figure 6 is a typical output of the slope stability calculations. The results of the stability calculations for cuts of 3.5 m, 5.0 m, and 6.5 m, with a 4.0 m wide bench at the 1/3 height, are summarized on Figure 7. Based on these calculations, cuts between 5.0 m and 6.5 m deep should have 5H:1V and a 4.0 m wide bench. Slopes of 4H:1V with a 4.0 m wide bench can be used for cuts of 5.0 m or less.

5.0 HEAVE ESTIMATES

We have conducted a preliminary estimation of rebound based on odometer test results from the Brady Road Landfill and the Bishop Grandin Underpass at Pembina Highway. The rebound at the deepest point of the Kenaston underpass excavation is calculated to be in the order of 200 mm. Although a portion of this rebound will occur during and immediately after excavation, it is recommended that the pavement and other infrastructure be designed to accommodate 200 mm of rebound. These rebound estimates will be reviewed based on test results conducted on samples from the site once available.

Most of the rebound occurs in the one to two meters of clay, immediately below the excavation. Granular materials below the pavement will not undergo post-construction expansion and a thick granular layer below the pavement will reduce rebound. Uneven rebound may occur if the thickness of granular materials below the pavement is not uniform. In addition, differential rebound is possible where pavements cross deep wide trenches backfilled with compacted granular material.

6.0 RETAINING STRUCTURES

It is understood that the rail bridge abutments and wing walls will be designed to retain lateral loads induced by earth backfill and surcharge loads from train loads. In addition, a low retaining wall (up to 2 m high) may be required to minimize the encroachment of the overpass excavation in the vicinity of the DND building northwest of the bridge.

6.1 ABUTMENTS AND WING WALLS

It is understood that the bridge abutments and wing walls will be ridged structures supported on pile foundations. The lateral earth pressures transferred to these structures will be a function of the backfill material, method of placing and compacting the backfill, and the magnitude of horizontal deflection of retaining wall after the backfill is placed. Table 6.1 lists lateral earth pressure coefficient for the active, passive and at rest cases for backfill materials likely to be considered for this project.

Table 6.1 – Lateral Earth Pressure Coefficients

Backfill Material	Active Coefficient (K_a)	At Rest Coefficient (K_o)	Passive Coefficient (K_p)
Sand	0.40	0.50	2.5
Coarse Gravel & Crushed Rock	0.25	0.35	4.0

The active pressure coefficients (K_a) should be used to calculate lateral loads on retaining structures which are free to translate or deflect horizontally by at least 0.2 percent of the retaining wall height. The at-rest earth pressure coefficients (K_o) should be used behind retaining structures which are not free to translate. Cohesive soils, including lakebed clay from the Red River Valley, are not recommended for backfill behind retaining structures, because these soils have high swelling potentials and could generate excessive lateral earth pressures.

The earth pressures against retaining structures will increase due to surcharges such as train loads behind the wall. As such, an appropriate surface surcharge should also be included in the pressure distribution to account for surface loads. The active pressure (K_a) coefficients can be used to calculate the component of lateral load on the retaining structure due to surcharge loads.

The passive earth pressure coefficients can be used to calculate the resistance of granular backfills behind the abutments due to horizontal loads acting on the bridge. However, full passive resistance will only develop if the abutment can deflect at least 0.6 percent of the buried height of the abutment adjacent to the backfill.

The lateral earth pressure coefficients are approximate for the types of backfill material likely to be selected. For design, it is recommended that the earth pressure coefficients for sand be used unless it is considered economical to use a well graded and well drained coarse gravel or crushed limestone product, which may be more costly to manufacture. Once the design is complete, a product that meets the design criteria can be specified. Regardless, it is recommended that granular backfill be free draining with a maximum of 5 percent fines (maximum of 5 percent finer than 0.080 mm) so as to minimize frost effects which could increase lateral forces.

The backfill adjacent to the retaining structures should be sloped to provide surface drainage away from the structure. To prevent the build-up of hydrostatic pressures behind the retaining structures, a sub-drainage system consisting of filter-wrapped drainage pipe backfilled with washed gravel should be used at the base of the backfill.

Over compacting backfills may result in earth pressures considerably higher than predicted in design. Compaction of the granular fills within about 1.5 m of the retaining walls and abutment should be conducted with a light hand operated vibrating plate compactor and the number of compaction passes should be limited. A maximum compacted density of 92% of standard Proctor maximum dry density (SPMDD) should be specified for backfills adjacent to retaining structures. Backfilling procedures should be reviewed during construction to verify that they are consistent with the design assumptions.

6.2 Retaining Wall at DND Building

It is understood that a retaining wall up to about 2 m in height will be required to minimize encroachment of the cut slope on the DND building located northwest of the underpass. Before specific recommendations can be provided for retaining wall system a number of design objectives and requirements for the retaining wall must the established.

It may be possible to design a low retaining wall using modular concrete blocks such as Allan Blocks, a bin wall such as supplied by Armtec or a geo-grid reinforced slope such as Terrawall by Maccaferri. Slope stability considerations will limit maximum height of these types of walls, because the retaining wall will be located along the top of the excavation, effectively increasing the overall slope gradient and lowering the factor of safety of the cut. If the retaining wall compromises slope stability significantly, it will be necessary to consider a retaining wall supported on a pile foundation or a cantilever retaining wall to transfer loads well below potential failure surfaces in the slope.

It is understood that the retaining wall must be designed to support surcharge loads from heavy trucks and will likely require a guardrail. In this regard, a retaining wall constructed from lightweight blocks such as those used for landscaping projects may not be sufficiently robust. A bin wall or a heavy block product such as the Dura-Hold block from Barkman Concrete may be more suitable choices, as these products are more commonly used for transportation and industrial applications.

Once the design objectives are determined, a review of available systems can be performed to identify suitable systems and provide specific geotechnical recommendations, such as slope stability, global stability and, bearing capacity applicable to the specified system. The Contractor or Contractor's supplier usually prepares the detailed design for these types of retaining wall systems following the design procedures and specifications set out by the supplier. A geotechnical review of the retaining wall design should be conducted for compliance with project specifications, specifically to verify that acceptable factors of safety against bearing capacity, overturning and sliding are attained.

Modular block retaining structures should be constructed on a reinforced granular base. The reinforced base should extend about 2.0 m beyond the front of the modular block wall, depending on the height of the wall and foundations conditions encountered. A non-woven geotextile should be placed on the subgrade below the initial layer of granular fill. The granular base below the wall should be placed in maximum 150 mm thick lifts and compacted to a minimum of 98% SPMDD. The successive layers of granular fill and geo-grid should be installed in accordance with the manufacturer's specifications.

The ground surface behind the retaining walls should be graded to prevent infiltration into the backfill behind the walls. A drainage collection system (i.e. weeping tiles) should be incorporated into the wall backfill to prevent the build-up of water pressures behind the wall.

Qualified geotechnical personnel should be on site to review subgrade preparation and verify that good construction practices are followed during construction of the retaining wall.

7.0 FOUNDATIONS

7.1 BRIDGE FOUNDATIONS

It is understood that the total load on each pier will be in the order of 15,000 kN. A number of foundation alternatives have been considered for the proposed underpass bridge. Based on the soil and groundwater conditions, driven precast concrete piles or cast-in-place concrete piles founded into dense silt till are expected to be the most suitable foundation alternative. Driven steel pipe piles are also considered feasible, provided refusal can be achieved in the dense till. If this option is considered it should be evaluated with a test pile to determine refusal depth, refusal criteria, and capacity.

Caissons to limestone bedrock are considered to be technically feasible, but could be associated with a considerable level of uncertainty due to potentially poor bedrock and construction difficulties, and are therefore not recommended. The quality of the upper portions of the bedrock, as previously described, is very poor to poor and loads must be transferred by extending the caissons to the sound rock below. Construction difficulties associated with extending the caissons to the required depth may prove cost prohibitive. Furthermore, if sound rock is not encountered at a caisson location, it may not be feasible to relocate the caisson. Should they be considered, additional bedrock drilling and a full scale caisson excavation combined with a detailed down-hole evaluation will be required to assess the constructability, provide design parameters and improve confidence in this alternative.

Caissons bearing on dense till are a feasible alternative. The recommendations provided below are based on conventional end bearing capacities for caissons bearing on dense till. It may be possible to increase the design bearing capacity if an investigation to evaluate the compressibility of the till is conducted prior to design.

Recommendations for driven precast concrete piles and end bearing cast-in-place concrete caissons are provided below.

7.1.1 Driven Precast Concrete Piles

Driven precast concrete piles are well suited to support heavy foundation loads at this site. These piles should be driven to practical refusal into the dense glacial till. Provided that a hammer with a rated energy of 40 kJ per blow is utilized, the piles may be assigned the capacities shown in Table 7.1.

Table 7.1 – Allowable Pile Capacity Driven Precast Concrete Piles

Pile Diameter (mm)	Maximum Allowable Capacity (kN)	Final Refusal (blows/25 mm)
300	450	5
350	625	8
400	800	12

Precast concrete piles driven to practical refusal will develop the majority of their capacity from toe resistance, and therefore, no reduction in pile capacity is necessary due to group action. The design capacity of a pile group can be taken as the number of piles in the group multiplied by the allowable capacity per pile. Total and differential settlement for single piles subject to static compressive loads at the maximum allowable capacity is expected to be less than about 10 mm.

Further design and construction recommendations for driven precast concrete piles are summarized below:

1. The contribution from skin friction should be ignored.

2. The weight of the embedded portion of the pile may be neglected in the design.

3. The above allowable values pertain to soil resistance only. The piles must be designed to withstand the design loads and the driving forces during installation.

4. Pile spacing should not be less than 2.5 pile diameters, measured center to center. All piles driven within 5 pile diameters of adjacent piles should be monitored for heave, and should be re-driven if heave is observed. Piles that are re-driven should be driven to the refusal criteria outlined above (i.e. re-drive piles for one full set).

5. Piles may be pre-drilled to a maximum depth of 6 m to enhance pile plumbness and alignment, and to reduce the effects of pile heave during driving of adjacent piles. The diameter of the pre-drilling auger should not exceed the nominal diameter of the precast concrete pile. The pre-drill depth should be limited such that all piles are driven a minimum of 3 m beyond the pre-drill depth.

6. Piles that are damaged, more than 2 percent out of plumb, or refuse prematurely due to boulders in the till above the expected refusal depth may need to be replaced, pending a review of their acceptability by a qualified geotechnical engineer.

7. All piles should be driven continuously to their required design lengths, once driving is initiated.

8. Pile driving should be monitored and documented by qualified geotechnical personnel to verify that the piles are installed according to the specifications.

7.1.2 Hand Cleaned Cast-In-Place Concrete Caissons

Hand cleaned caissons (straight-shaft or belled) bearing in dense undisturbed silt till can be considered for the heavily loaded bridge foundations. Provided that the caisson bases are advanced into undisturbed dense silt till (about 13 to 14 m below ground surface) and hand cleaned to remove all loose, softened and disturbed soil, the caissons may be designed using an allowable end-bearing pressure of 750 kPa. At this bearing pressure, settlement is expected to be in the range of 5 to 15 mm for 0.75 m base diameter caissons.

Based on available subsurface information, the ground conditions are favourable for belled caissons. However, there are uncertainties inherent with this foundation system which could increase the installation costs. Expanding the caisson bases with a belling tool may be difficult in the dense till or if boulders are encountered. If water bearing zones are encountered it may be necessary to sleeve the shafts and extent the length of the caissons. If this option is considered, a test caisson and bell are recommended to evaluate construction conditions and verify the feasibility of this option.

The following additional recommendations are provided for the design and installation of cast-in-place end bearing caissons:

1. Caissons should be designed on end bearing capacity. Skin friction along the caisson shaft should be ignored.

2. Neighbouring caissons should be spaced no closer than 2.5 base or bell diameters, measured centre to centre. Closer spacing may be considered provided the potential for increased settlements is evaluated.

3. The bell to shaft ratio for belled caissons should be a maximum of 3.

4. A minimum shaft diameter of 0.7 m will be required to facilitate manual cleaning of the caissons bases. Full length, tight fitting sleeves will be required to protect down-hole workers. All caisson excavations and protection of down-hole workers should comply with the requirements of Manitoba Workplace Safety and Health.

5. Water should not be allowed to pond on the bearing surface at the base of the caisson.

6. Once the bearing surface has been suitably prepared, it should be evaluated by qualified geotechnical personnel to confirm that the bearing conditions are consistent with those identified in this report and that proper construction practices are being employed.

7. To minimize rebound of the caisson base, the caissons should be poured as soon as possible after excavating.

8. In order to resist tensile forces due to freezing and swelling of the soil, all caissons should be reinforced to a minimum depth of 8 m.

7.1.3 Lateral Pile Loading

It is understood that the bridge piers and abutments will be subjected to lateral forces. Driven precast concrete piles may be battered to increase lateral load carrying capacity. The lateral load capacity of a battered pile is considered to be the horizontal component of the axial load. The lateral capacity of other foundation systems can be estimated once details of the foundation are determined. If requested a lateral pile analysis can be performed.

7.1.4 Pile Caps, Abutments and Piers

The pile caps, bridge piers, and abutments should be cast with a minimum 200 mm void form between the underside of the concrete and the soil surface below to prevent uplift forces on the structure caused by heave of the excavation, swelling of the clay, and frost action. The void form should be a compressible and biodegradable material to prevent soil or debris from sloughing into the void.

7.2 Stormwater Lift Station

The stormwater lift station is to be located east of Kenaston Boulevard and south of the CN mainline. Based on the preliminary design drawings, the pumping station will be a one storey, cast-in-place concrete structure with a wet well. The structure will occupy an area of about 70 m^2. The lower level will be about 4 m below existing ground surface and the wet well will extend to about 14 m below ground surface. At this depth the underside of the wet well will be at elevation 219.5 m or about 4 to 5 m below the clay-till contact.

The upper portion of the pumping station should be supported on driven precast concrete piles or cast-in-place concrete caissons founded on dense till. The wet well can be supported directly on the till with a slab or raft foundation.

7.2.1 Wet Well Foundation

A thickened slab or raft foundation bearing on the till is feasible for the wet well. A thickened slab or raft foundation is preferred over individual footings, which would be more susceptible to differential settlements.

Provided that the bearing components of the thickened slab or raft foundation are founded in the medium dense to dense till, they may be designed using an allowable net bearing pressure of 400 kPa. Regardless of bearing capacity considerations, the bearing components of the foundation should have a minimum dimension width of 0.6 m. Long term total and differential settlements of the slab foundation constructed in accordance with the recommendations in this section are expected to be less than 25 mm.

Preparation of the bearing surface should include removal of all loosened and disturbed soils. The exposed bearing surface should consist of medium dense to dense till. Care should be taken to avoid excessive disturbance of the bearing surface during preparation. Once the bearing surface has been suitably prepared, it should be evaluated by qualified geotechnical personnel to confirm that the conditions are consistent with those identified in this report and that proper construction practices are being employed.

A layer of lean mix concrete (mud slab) or a layer of crushed limestone at the base of the excavation may be required to form a working surface prior to construction of the wet well. Where granular fill is used, it should be placed in maximum 150 mm lifts and uniformly compacted to a minimum of 100% of standard Proctor maximum dry density (SPMDD, ASTM D698).

7.2.2 Driven Precast Concrete Piles

Driven precast concrete piles are well suited for support of the pumphouse at this site. The driven precast concrete piles should be designed and installed in accordance with the recommendations in Section 7.1.1 of this report. The piles should be installed prior to construction of the wet well to limit the potential for disturbance of the wet well excavation during pile driving.

7.2.3 Mechanically Cleaned Cast-In-Place Concrete Caissons

End bearing caissons as outlined in Section 7.1.2 of this report may be used to support the upper portion of the lift station. However, the cost of hand cleaning the bells may be uneconomical and mechanically cleaning the bells, which will not require workers to enter the caissons, may be more cost effective. Provided that the caisson bases are advanced into undisturbed dense silt till (similar elevation as the wet well foundation), the caissons may be designed for an allowable net bearing pressure of 400 kPa. The belling operation should be initiated such that the base of the bells are in dense till.

As with the hand cleaned caissons, uncertainties and construction difficulties which could affect installation and cost of the caissons include: potential water bearing zones which must be sealed with casing, and difficult belling if boulders are encountered, or if the bells are attempted in the dense till.

The following additional recommendations are provided for the design and installation of mechanically cleaned cast-in-place concrete belled caissons:

1. Caissons should be designed on end bearing capacity only. Skin friction along the caisson shaft should be ignored.

2. Neighbouring caissons should be spaced no closer than 2.5 base diameters, measured centre to centre.

3. The bell to shaft ratio should be a maximum of 3.

4. Water should not be allowed to pond on the till surface.

5. Full length, tight fitting sleeves may be required to prevent caving, seal off seepage, particularly in areas of thick fill and surface silt.

6. To minimize rebound of the caisson base, the caissons should be poured immediately after excavation.

7. Caisson installation should be monitored by qualified geotechnical personnel to confirm that the bearing conditions are consistent with those identified in this report and that proper construction practices are being employed.

8. In order to resist tensile forces, all piles should be reinforced to a minimum depth of 8 m.

7.2.4 Wet Well Buoyancy

The wet well should be designed to resist hydraulic uplift pressure of 86 kPa acting across the entire area of its base. This pressure is based on the buoyancy forces due to a total head in the till of 228.3 m, or 8.8 m above the underside of the wet well.

7.2.5 Wet Well Excavation

Based on the preliminary design drawings it is understood that the wet well excavation will measure about 3 m by 6 m.

UMA has investigated the potential for two modes of failure of the base of the excavation for the wet well. These are: piping of the till, and uplift or base heave of the excavation. Failure by one of these modes would result in rapid flooding of the excavation and dangerous conditions for workers.

The potential for failure of the excavation base arises from the fact that the wet well excavation will be approximately 5 m above the top of the bedrock, while the head in the bedrock is about 4.5 m above the wet well excavation level.

Piping

The potential for piping (also referred to as boiling) increases as the hydraulic gradient approaches the critical hydraulic gradient and is significantly influenced by the degree of fracturing and/or fissuring of the till. The critical hydraulic gradient, for an intact cohesionless soil, is calculated according to the following equation:

$$i_c = \frac{\gamma'}{\gamma_w}$$

where $\gamma' = \gamma - \gamma_w$

The saturated unit weight (γ) of the till is documented to vary between about 21.2 and 24.3 kN/m^3 (Kjartanson et. al., 1983). Given that a majority of the till below the proposed slab elevation is dense, the saturated unit weight has been assumed to be 23 kN/m^3, which results in a critical hydraulic gradient (i_c) of about 1.34.

The hydraulic gradient across the till is calculated as:

$$i = \frac{\Delta h}{l}$$

where: Δh is the difference between the head at the base of the till (i.e. in the bedrock) and the floor of the excavation,

and l is the thickness of the till between top of the bedrock and the floor of the excavation.

Based on the groundwater pressures observed in January and February 2005, the hydraulic gradient across the till is 1.04, which is lower than the calculated critical hydraulic gradient of 1.34. The factor of safety against piping is about 1.3, which is considered acceptable. However, if the piezometric head in the upper carbonate aquifer reaches an elevation of about 226.7 m, (about 1.5 m higher than in January and February 2005) piping could occur. Although the head which could cause this failure is generally higher than the typical peak levels observed in Water Resources Branch well G05OC019, it is recommended that the piezometric head in the bedrock be monitored during construction, so that if it rises above unacceptable levels corrective action can be taken.

If water can flow along a direct connection between the bedrock and the excavation, such as through a discontinuity or fracture in the till, the potential for a piping failure increases significantly. The degree to which fractures may contribute to a boiling failure cannot reasonably be evaluated prior to construction. Therefore, it will be necessary to monitor the excavation for evidence of the onset of piping. If seepage rates increase and cannot be controlled, it will probably be necessary to terminate work and implement a dewatering system to lower the water pressure head in the bedrock aquifer.

Base Heave

A rupture failure or base heave failure can result if the pressure in the bedrock aquifer exceeds the overburden pressure at the till-bedrock contact beneath the excavation. If this condition is exceeded, there is a potential for base heave and flooding of the excavation. The pressure head in the aquifer at the till-bedrock contact is 107 kPa. (based on the head in the aquifer in January and February 2005). The 5 m of till above the bedrock remaining after excavation, exerts a downward pressure of 115 kPa, which is greater than the uplift pressure and therefore a base heave failure would not be expected. The shearing resistance of the till will further contribute to uplift resistance and it is felt that the base of the excavation will be sufficiently stable.

Base stability decreases with increasing size of the excavation and base stability should be reviewed if the excavation is to be larger than 3 m by 6 m as understood at the time of this report.

The excavation will need to be monitored for evidence of base instability and the development of unsafe working conditions. Should such conditions develop alternatives to complete the excavation safely, possibly including dewatering of the bedrock, will need to be implemented.

Given that the potential for base heave increases with increasing depth of excavation it is recommended that the excavation depth be kept as shallow as possible.

7.2.6 Temporary Shoring

A temporary shoring system will be required to excavate the wet well shaft. Predictions of earth pressures and deflections are complex. The yield of one part of a flexible system increases pressure in the more rigid components. Therefore, the pressures in the vicinity of supports are higher that in unsupported areas, and the loads on individual supports are dependent on the stiffness of the supports. Strutted temporary shoring systems such as soldier piles with timber lagging or sheet pile walls have been successfully used in the Winnipeg area and should be designed to accommodate the earth pressures shown in Figure 8. Given that the excavation will extend into the dense till, it may be difficult to drive the sheet piling to the required depth.

Once the detailed design of the temporary shoring is completed, a joint structural and geotechnical review should be conducted to assess the appropriate use of earth pressures and conformance with the project requirements.

7.3 FOUNDATION CONCRETE

Winnipeg soils are known to contain sulfates in concentrations high enough to be detrimental to concrete. The degree of exposure of concrete to sulphate in Winnipeg is commonly classified as severe (CSA-A23.1-M2000). Accordingly, all concrete in contact with the soils at this location should be made with sulphate resistant cement (CSA Type 50). Furthermore, the concrete should have a minimum specified 28 day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Table 12, CSA-A23.1-M2000. Concrete exposed to freeze-thaw cycles should be adequately air entrained to improve freeze-thaw durability in accordance with Table 10, CSA-A23.1-M2000.

Respectfully submitted,

UMA ENGINEERING LTD.

Mar 8/2005

Giovanni Militano, M.Sc., P.Eng.
Geotechnical Engineer
Earth and Environmental

Jeff Tallin, M.Sc., P.Eng.
Senior Geotechnical Engineer
Earth and Environmental