GEOTECHNICAL INVESTIGATION
PROPOSED BRIDGE REPLACEMENT
BUNN'S CREEK CROSSING
BONNER AVENUE
WINNIPEG, MANITOBA

Submitted to:

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100 years in Canada

TABLE OF CONTENTS

TABLE OF CONTENTS (cont)

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

As authorized by Mr. Darren Burmey, P. Eng. of Stantec Consulting Ltd., AMEC Earth & Environmental, a division of AMEC Americas Ltd. (AMEC), has completed a geotechnical investigation for a proposed bridge replacement at Bunn's Creek on the south side of Bonner Avenue in Winnipeg, Manitoba.

The Terms of Reference were presented in AMEC's proposal WPG2007.204, dated 8 June 2007. The purpose of the geotechnical investigation was to determine general subsurface soil and groundwater conditions at a single test hole location and, on this basis, provide geotechnical recommendations for the design and construction of suitable foundation alternatives for the bridge. As well, AMEC was to provide comments on any apparent slope issues and in the event of any potential concerns provide preliminary recommendations on how to address these items. It is understood that AMEC's report will be used as supporting documentation to provide a City of Winnipeg Waterway permit.

2.0 SITE CONDITIONS

The current timber bridge is located immediately south of Bonner Avenue and is used to traverse Bunn's Creek. The bridge is currently utilized as an access to the residents of Lots 330 and 334 as shown on Figure 1, as well as to access a park and trail along the south side of Bunn's Creek. The existing bridge is a three span timber bridge, about 11 m long and is apparently supported on driven timber piles. The creek forms an outside bend at the bridge location, with wood shoring located along the outside bend along Bonner Avenue to the north. Wood shoring is also used locally at the south abutment although the remainder of the slope is natural. It is assumed that the wood shoring along Bonner Avenue was originally installed to protect against erosion and riverbank instability. It is further understood that the shoring will remain in place following replacement of the bridge structure.

Generally, the creek ranges from 2 m to 6 m wide in the vicinity of the bridge and at the time of the investigation, was 0.1 to 0.5 m deep and flowing west towards the Red River. The existing survey data shows the overall slope height to be about 3 m, as measured from the centre of road along Bonner Avenue.

3.0 PROPOSED FACILITIES

Based on the information provided to AMEC by Stantec Consulting Ltd., it is understood that the proposed new bridge will be a concrete bridge instead of the existing timber bridge. The new bridge will be designed to be capable of sustaining a truck load of about 625 kN and will be supported by pile foundations. At this stage, the layout of the bridge is not known, however, it is understood that it will have similar size and design elevation as the current timber bridge, although would be assumed to be constructed with a single span.

4.0 FIELD INVESTIGATION

On 13 July 2007, one test hole (TH01) was drilled utilizing a truck mounted (B40LX) drill rig equipped with 125 mm diameter continuous flight solid stem augers, and operated by Maple Leaf Drilling of Winnipeg, Manitoba. Test hole logging and site supervision was provided by Mr. Lee Keong Tan, E.I.T. of AMEC on a full-time basis. The test hole was advanced to practical auger refusal, which occurred at 17.8 m below existing grade. The test hole location is shown on the Test Hole Location Plan in Figure 1.

All soils observed during test hole drilling were visually classified on site according to the Modified Unified Soil Classification System. Groundwater and drilling conditions, as well as any pertinent subsurface observations, were also recorded at the time of the investigation. Disturbed soil samples were taken at regular depth intervals, both from auger cuttings and a split spoon sampler. In addition, relatively undisturbed Shelby tube samples were collected continuously from 2.3 m to about 8.2 m below grade, to assess the possible presence of slickensides which are indicators of potential slope movement. Field testing completed during the field investigation consisted of pocket penetrometer tests (PP's) for all cohesive soils and Standard Penetration Tests (SPT's) performed in conjunction with spilt spoon sampling in the non-cohesive glacial till soils. The number of blows to drive the SPT sampler a distance of 0.3 m was recorded and is shown on the test hole logs as the SPT (N) value.

The test hole was backfilled with auger cuttings and a layer of bentonite at the completion of drilling, after verification of short-term sloughing and seepage conditions, with excess cuttings left adjacent to the test holes. The ground surface elevation of the test hole was determined and was referenced to the top nut on a fire hydrant located northwest of the site along the north side of Bonner Avenue (See Figure 1). The temporary benchmark was assigned an arbitrary local elevation of 100.00 m.

The test hole log is included in Appendix A (Figure 2) and shows the soil profile, results of the field and laboratory testing and comments relative to sloughing and seepage conditions encountered.

5.0 LABORATORY TESTING

Soil samples were transported to the AMEC's soils laboratory for examination and testing. All soil samples were visually examined by AMEC's Project Engineer in order to supplement and confirm the field classifications. All soil samples were tested to determine their natural moisture content. Atterberg Limits were completed on select samples to assess index properties and unconfined compressive strength and laboratory vane shear strength testing of selected Shelby tube samples was also undertaken. All of the laboratory results are shown on the test hole log in Appendix A.

6.0 SUBSURFACE CONDITIONS

6.1 SOIL PROFILE

The soil stratigraphy at the site, as noted in descending order from the ground surface at the test hole location, consisted of the following:

- Gravel Fill
- Clay Fill
- Alluvial Clay
- High Plastic Clay
- Silt Till

Gravel Fill

An approximate 150 mm thick layer of gravel fill was encountered at ground surface at the test hole location. The gravel fill was poorly graded, fine grained, wet, medium dense to dense, light brown and contained some sand to sandy and a trace amount of silt.

Clay Fill

Directly underlying the gravel fill was a layer of clay fill about 450 mm thick. The clay fill was high plastic, moist, very stiff, dark grey and contained some silt, sand and gravel and occasional silt and oxidation inclusions.

Alluvial Clay

The clay fill was underlain by native high plastic alluvial clay that extended to about 5.8 m below existing ground surface. The alluvial clay was silty, high plastic, moist, stiff, dark brownish grey and contained occasional sand and gravel inclusions, occasional oxidation, organic lenses and inclusions, as well as occasional rootlets. The alluvial clay became dark greyish brown below 1.2 m from ground surface. A poorly developed slickenside, which was inclinded at about 60 degree from the horizontal, was encountered in the Shelby tube sample that was collected from about 5.6 m below existing grade; however, no other evidence of slickensides was observed.

High Plastic Clay

A layer of native, high plastic, lacustrine clay was encountered below the alluvial clay and extended to 15.5 m below existing grade. The high plastic clay was generally moist, stiff, mottled grey and brown, varied from containing some silt to being silty and had occasional silt inclusions. The native lacustrine clay became very moist, soft to firm, and grey with occasional silt till inclusions, with increasing depth. No evidence of slickensides was observed within the high plastic lacustrine clay zone.

Silt Till

Silt till was encountered underlying the lacustrine clay in the test hole and extended to the depth explored. The silt till was initially clayey, low plastic, moist, medium dense to dense, light brown and contained trace to some sand and trace gravel. A seepage zone was encountered at 15.9 m below exiting ground surface. The silt till became very dense and had some gravel to gravelly with a corresponding decrease in clay content at depth 17.1 m below grade.

6.2 GROUNDWATER CONDITIONS

The test hole was left open for approximately five to ten minutes after completion of drilling to observe short-term groundwater seepage and sloughing conditions. Although no sloughing was encountered, heavy water seepage occurred within the silt till, with the water level rising up to 5.5 m below existing ground surface prior to backfilling.

It should be noted that only short-term seepage and sloughing conditions were observed and that groundwater levels can fluctuate annually, seasonally or as a result of construction activity.

6.3 POWER AUGER REFUSAL

Practical auger refusal was achieved at 17.8 m below the existing ground surface in TH01. Based on auger resistance at the refusal depth and local experience elsewhere, it is inferred that boulders or very dense silt till prevented further advancement of the auger at this location.

7.0 DISCUSSION AND RECOMMENDATIONS

7.1 FOUNDATIONS

Based on the subsurface conditions encountered at the test hole location, a deep piled foundation system, consisting of either driven piles or cast-in-place concrete piles, is considered to be feasible for the proposed new concrete bridge. Based on the size of the proposed bridge, driven pre-cast concrete end bearing piles or drilled cast-in-place concrete friction piles are likely preferred, although driven steel piles may also be suitable.

It is expected that piles will also be subject to lateral loads, as a result of vehicles on the bridge, or soil movements being retaining walls or abutments. All piles will need to be capable of resisting these forces. Where driven pre-cast concrete piles are desired, consideration should be given to battered piles, since this pile type does not generally perform well in lateral loading.

The following sections provide recommendations for driven pre-cast and steel piles, as well as cast-in-place concrete friction piles.

7.1.1 Driven Pre-Cast Concrete Piles

Driven, hexagonal pre-cast concrete piles are a suitable foundation alternative at this site. Applicable design loads for various pre-cast concrete piles, driven to practical refusal, are summarized in Table I.

Table I: Allowable Pile Capacity Driven Pre-Cast Concrete Piles

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

The above noted design capacities are based on the concrete piles being installed with a hammer (drop or diesel) rated for a minimum energy of 40 kJ per blow.

Any piles that are damaged, excessively out of plumb or refuse prematurely due to encountering boulders in the till may need to be replaced, pending a review of their load carrying capacity and expected settlement by a qualified geotechnical engineer.

The following additional recommendations are provided and are applicable to the design and installation of driven pre-cast concrete piles for the proposed development:

1. The above allowable values pertain to end bearing resistance only. The pile cross sections must be designed to withstand the design loads and the driving forces during installation.
2. Pile spacing should not be less than 2.5 pile diameters, measured centre to centre. All piles driven within 5 pile diameters should be monitored for heave and, where heave is observed, piles should be re-driven. Piles that are re-driven should be driven to the refusal criteria outlined above (i.e. re-drive piles for 1 full set).
3. Pre-boring to a maximum depth of about 6 m from grade is recommended at all pile locations, to enhance pile plumbness and alignment, and to reduce the effects of pile heave during driving of adjacent piles. Pre-bore holes should have a "tight fit" to reduce the potential loss of lateral pile capacity resulting from pre-boring.
4. The driving of all piles should be documented and approved by qualified geotechnical personnel. The capacities shown in Table I should be confirmed and reported after driving.
5. Where pre-cast piles are subject to uplift forces, the allowable skin friction values provided in Table II should be utilized.
6. All piles should be driven continuously to their required design lengths once driving is initiated.

7. A void space (minimum of 150 mm thick) should be constructed below all pile caps and grade beams to accommodate the potentially expansive nature of the underlying soils. The void material should be a low compressible strength, biodegradable material.

The pre-cast concrete piles, driven to practical refusal into silt till or limestone bedrock will develop most of their capacity from end bearing resistance. Therefore, the reduction of capacity due to group action can be ignored. Under these conditions, the capacity of pile group can be taken as the number of the piles in the group multiplied by the allowable capacity of a single pile, provided that the above referenced pile spacing is adhered to. With respect to uplilft or lateral resistance, however, group reduction factors should be applied where groups of three or more piles are used. In this case, AMEC should be contacted to provide review of the foundation layout and to provide further design input with respect to the application of group reduction factors.

7.1.2 Cast-in-Place Concrete Friction Piles

Drilled cast-in-place concrete friction piles are also well suited for this site and are likely preferred over driven pre-cast concrete due to their superior resistance to lateral forces. Drilled cast-in-place concrete piles may be designed on the basis of the allowable skin friction values provided in Table II, applied to the pile circumference within the native high plastic clay.

Table II: Allowable Skin Friction Values – Drilled Cast-in-Place Concrete Piles

Depth Interval From Grade	Allowable Skin Friction	
	Compression	Tension (Uplift)
0 – 2.4 m	0 kPa	0 kPa
2.4 m – 13 m	16 kPa	11 kPa

Groups of two piles can be effectively utilized without a group reduction factor, whereas the total load carrying capacity of groups of three or more piles may be somewhat less than the sum of the individual pile capacities. Where groups of three or more piles are planned, this office should be contacted to review the proposed pile layout such that a suitable group reduction factor may be provided, if required, based on pile layout and spacing.

Further design and construction recommendations for concrete friction piles are provided below:

1. The contribution from end bearing should be ignored.

2. The piles should be spaced a minimum of three pile diameters, measured centre to centre.

3. All piles should have a minimum length of 9 m as measured from the adjacent final grade to resist uplift forces.

4. Pile should not be installed more than about 13 m below the existing ground surface, to avoid potential blow-in of groundwater on account of the high water pressure in the silt till.

5. All piles should be provided with adequate full length steel reinforcement designed by a structural engineer.

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

7. Concrete should be placed as soon as practical following the drilling of each pile to prevent any potential squeezing and groundwater blow up from the bottom clay zone.

8. Temporary steel casings should be available on site and utilized as required during construction to control ground water seepage and sloughing in the pile holes to maintain pile holes in a clean dry state.

9. A void space (minimum of 150 mm thick) should be constructed below all pile caps and grade beams to accommodate the potentially expansive nature of the underlying soils. The void material should be a low compressible strength, biodegradable material.

7.1.3 Driven Steel Piles

Driven steel piles, consisting of open or closed ended pipe or H sections, are also considered to be a suitable foundation system at the site, given the subsurface conditions encountered. Generally, the allowable geotechnical capacity of steel piles driven to refusal into competent soil/rock is often assumed equal to the allowable structural load capacity of the steel. The allowable structural load capacity of a steel pile is related to the allowable fibre stress of the steel and is generally determined by multiplying the cross-sectional area of the steel at the pile tip by 0.35 f_y, where f_y, is the yield strength of the steel. In reality, the allowable geotechnical capacity of driven steel piles achieved on the site will depend on various factors such as the selected driving hammer, refusal criteria and driving energy during pile installation. It is important to use an appropriate combination of driving hammer / energy and refusal criteria for each type of pile to ensure the allowable geotechnical capacity is in fact equal to or greater than the allowable structural capacity, while also ensuring that the pile structure will not be damaged by driving. AMEC can provide the actual refusal criteria, once the pile sizes, steel strength and hammer size are known

The following additional recommendations regarding steel pipe piles are provided.

1. The allowable capacities noted pertain to soil and bedrock resistance only. The pile cross sections must be designed to withstand the design loads, handling stresses and the driving forces during installation.

2. As a general guideline, driving energies should not exceed 600 Joules per blow per cm^2 of pipe wall cross sectional area to prevent pile damage. This limiting maximum driving energy assumes that only new steel will be used.

3. If drop hammers are used, the hammer should have a minimum mass equivalent to three times the mass of the pile.

4. Piles should be driven continuously to the identified refusal criteria, once driving is initiated.

5. The driving of all piles should be documented and approved by competent and experienced geotechnical personnel, working on a full time basis.

6. Pile driving should be monitored to determine if heaving of nearby piles occurs. Surveyed elevations and locations of adjacent driven piles should be obtained by the contractor and reported on a regular basis so that the extent of pile movements can be properly evaluated.

7. Pre-boring to a maximum depth of about 3 m below excavation grade is recommended at all of the pile locations, to reduce heave. Any pre-bore holes should be backfilled with sand to reduce the potential loss of lateral pile capacity resulting from pre-boring.

8. A void space (minimum of 150 mm thick) should be constructed below all pile caps and grade beams to accommodate the potentially expansive nature of the underlying soils. The void material should be a low compressive strength, biodegradable material.

9. All piles should be fitted with driving shoes to improve penetration and minimize the potential for pile damage.

10. Pile spacing should be a minimum of 3 pile diameters, measured centre to centre.

11. Piles may encounter cobbles, boulders and/or hard till deposits during driving. Care should be taken to prevent structural damage to piles during driving, to the extent possible. Under some circumstances where structural damage is imminent, however, some piles may need to be abandoned and replaced with supplementary piles. The decision to abandon and/or replace piles should be the site supervisor's responsibility.

12. Any piles that are damaged, excessively out of plumb or refuse prematurely due to encountering boulders in the till may need to be replaced, pending a review of their load carrying capacity and expected settlement.

13. Open-end steel pipe piles may be filled with tremie concrete to increase the structural strength and lateral resistance of the pile.

7.2 LATERAL LOADS

Bridge structures can be subject to high lateral loads and therefore, lateral resistance of the foundation will be required. In the absence of pressuremeter testing or a full scale load test, the following equation should be utilized to estimate the coefficient of horizontal subgrade reaction in order to calculate lateral pile capacities where only relatively small deflections of the foundations are tolerable (i.e. less than about 10 mm). For larger allowable deflections, the design may be limited by the strength of the soil and should be reviewed in detail by this office.

For cohesive soils, $$k_s = \frac{67\tau_u}{d}$$

Where: k_s = coefficient of horizontal subgrade reaction
d = pile diameter (m)
τ_u = undrained shear strength of the clay (kPa)

In this regard, AMEC recommends that a value of $\tau_u = 50$ kPa be utilized for the clay deposits below a depth of 1 m, to a maximum depth of about 13 m from grade. A reduced $\tau_u = 30$ kPa should be used below the 13 m depth. It should be noted the coefficients computed by this method are subject to a high degree of uncertainty and must be used with caution.

Where lateral loads and pile deflections are critical, it is recommended that a p-y analysis be undertaken to model the non-linear response of the soil. In this type of analysis, the strength-deformation characteristics are modeled by load-displacement curves developed for the various soil layers. This is an iterative procedure performed using LPILE software. Outputs for a given lateral load include deflections and bending moments at frequent depth increments along the pile shaft. This analysis, with group action effects, could be conducted by AMEC upon request. Battered piles can also be analyzed using this software.

Alternatively, lateral loads could be resisted by a system of battered piles only.

7.3 ROADWAY APPROACHES

It was assumed that a gravel approach is planned for the roadway to the bridge abutments on the south side of the bridge, as presently exists. In the absence of design traffic volumes, and based on the assumption that the roadway is subject to light truck and passenger vehicle traffic only, it is recommended that the gravel thickness be not less than 400 mm (150 mm of base and 250 mm of subbase). AMEC can provide a more detailed review on request, if traffic data is known.

Subgrade preparation for new approach areas should be performed as follows:

1. Ensure that all existing fill materials are removed to expose the native clay subgrade. Care should be taken to prevent disturbance of the subgrade and to ensure that the excavations are not advanced more than required. Further excavate as required to

achieve the design subgrade elevation. The exposed subgrade is expected to consist primarily of high plastic, stiff to very stiff clay.

2. During and subsequent to excavation, the subgrade should not be disturbed or subject to frost, desiccation, inundation or heavy tire loads.

3. After excavation, the exposed subgrade should be proof-rolled with a heavy non-vibratory sheepsfoot / pad foot compactor to determine the uniformity of the subgrade conditions and to identify the presence of any soft or weak zones

4. Where soft zones are encountered, these should be either fully excavated or bridged with geotextile and 150 mm down crushed rock as directed by the geotechnical engineer. Where proof rolling does not identify the presence of underlying zones of highly compressible material, the sub-grade surface should be scarified, moisture conditioned and uniformly compacted to a minimum of 95% of standard Proctor maximum dry density (SPMDD).

5. Excavation, proof-rolling and preparation of the subgrade should be undertaken under the direction of qualified geotechnical personnel.

6. Fill materials required between the exposed sub-grade and the underside of the granular sub-base material should consist of additional compacted granular sub-base or compacted clay fill, placed and compacted in maximum 150 mm thick lifts to a minimum of 95% of SPMDD. All clay fill should be placed on the wet side of optimum moisture content.

7. The sub-base and base courses should be placed and uniformly compacted in maximum 150 mm thick lifts to a minimum of 100% of SPMDD.

The ground surface should be crowned to promote drainage to the sides of the roadway and then graded to drain to the adjacent creek.

8.0 RIVERBANK SLOPE STABILITY ASSESSMENT

8.1 STABILITY

A site reconnaissance of the site was conducted on 9 July 2007. At the time of the site reconnaissance, neither signs of distress related to current slope movements nor evidence of historic movements were observed. It should be appreciated, however, that tension cracks or other evidence of slope instabilities may have been obscured by the long grass and other vegetation. In this regard, a follow-up confirmatory site visit in Fall 2007 is recommended, once the vegetation has reduced.

To evaluate long term slope conditions, a slope stability model was developed using the topographical information provided by City of Winnipeg. Although the survey information is not detailed, the contours suggest that the slope of the riverbank along the south side of the creek varies from about 3.5:1 to 4.5:1. On the north side, there is a vertical retaining wall which controls the local slope angle. In keeping with AMEC's proposal, it has been assumed that this vertical retaining wall will remain and therefore the north side of the creek has not been evaluated at this time.

A numerical model was created by AMEC using Slope/W and Seep/W, software packages of Geo-Studio 2004, established by Geo-Slope International Ltd of Alberta. The purpose of the model development was to determine the approximate existing stability of the south riverbank and to identify the need for any slope modifications during new bridge re-construction. As the slope at the west side of the bridge (south side of creek) was steeper than on the east side of the bridge, this cross section was utilized in the numerical model (labelled as Cross Section A-A).

Typically, back analysis is undertaken to verify a factor of Safety (FS) equal to 1.0, at a verified failure surface. However, given that no failure surfaces were provided, a range of typical Winnipeg soil properties were used to assess the likely range of existing FS. The ranges utilized for the study are shown in the following table.

Table III: Summary of Upper and Lower Bound Soil Properties

Soil Type	Unit Weight (kN/m^3)	Lower Bound		Upper Bound	
		Cohesion (kPa)	Internal Friction Angle, ϕ' (Degree)	Cohesion (kPa)	Internal Friction Angle, ϕ' (Degree)
Clay Fill	18	0	16	0	23
Native Alluvial High Plastic Clay	18	1	12	4	21
Lacustrine High Plastic Clay 2	18	2	12	4	19
Silt Till	Assume to be a hard layer for analysis purposes				

In the model, the groundwater phreatic surface was assumed to be located at about 2.0 m below the existing ground surface at the top of the riverbank then decreasing gradually to the current water level in the creek. While a shallow perched groundwater level coincident with the ground surface throughout the bank could be utilized, this is not considered to be a realistic assumption given the anticipated flood stage of this creek. In addition to the local shallow groundwater, a second phreatic surface at about 5.5 m from grade was used in the model. This phreatic surface was based on the observed groundwater level in the glacial till. The model which was developed utilizing the observed soil profile and the measured/assumed phreatic surfaces are shown in Figure B1, Appendix B.

The results of the lower bound and upper bound analyses are shown as Figures B2 and B3, using Cross Section A-A as the applicable bank section. As can be seen, the lower bound values predict a FS of close to unity (i.e. FS = 1.08) and the upper bound values predict a FS of 2.57, neither of which is considered to be realistic based on the site observations and expected conditions.

Based on this initial data, a number of iterations were undertaken to identify a representative FS, in keeping with the site observations and anticipated FS in the order of at least 1.5. In this regard, a FS of 1.5 was identified when using the following probable operating soil properties:

Table IV: Summary of Selected Soil Properties

Soil Type	Unit Weight (kN/m^3)	Cohesion (kPa)	Internal Friction Angle, ϕ' (Degree)
Clay Fill	18	0	23
Native Alluvial High Plastic Clay	18	2	14
Lacustrine High Plastic Clay 2	18	4	19
Silt Till	Assume to be a hard layer for analysis purposes		

This model is shown as Figure B4 and is considered to represent a reasonable approximation of the operating soil shear strength parameters for this location. On this basis, the slope model confirms the site observations which indicate that the south riverbank is stable in its current configuration.

AMEC recommends that any changes to the slope configuration should ensure positive additions to stability of the riverbank (i.e. slope flattening, reduced loading, end-bearing piles), in particular adjacent to the bridge. Furthermore, construction of the bridge should ensure the following:

- No stockpiles of construction materials should be allowed within 25 m of the top of bank;
- Any open excavations near the creek should not be allowed to retain water;
- Any grade changes in the vicinity of the riverbank should be evaluated by AMEC as part of the final design; and
- No increase in grades or steepening of the riverbank should be allowed.

8.2 PERMANENT ABUTMENT WALLS

Deep excavations at the riverbank are not expected for the proposed bridge replacement, however, it is assumed that limited excavations will be necessary for abutment construction and that soil will need to be retained behind the abutments over the long term. Temporary excavations should be not steeper than 1:1.

Earth pressure coefficients for use in design of the abutment should assume a triangular pressure distribution and an active earth pressure co-efficient of $K_a = 0.5$. Passive resistance on the downstream side of the abutment should be assumed to be equal to zero. Traffic loads should also be accounted for in the design of the abutment.

8.3 EROSION PROTECTION

Erosion protection is generally required downslope of the abutments to reduce the potential for localized over-steepening of the riverbank due to erosion. At this location, the outside bend of the creek, which is most susceptible to erosion, is protected by a retaining wall which appears to have performed satisfactorily. On the inside bend of the creek, erosion protection may not be essential; however, is recommended as a minimum on either side of the bridge abutment (extending 10 m from the bridge in either direction). The erosion protection should consist of large diameter limestone riprap, in a layer 300 to 400 mm thick, extending from just below to about 0.6 m above the normal summer water level. The limestone riprap should be well graded, durable, crushed and consistent with typical local practise. The limestone riprap should be underlain by a high quality non-woven geotextile. Rip-rap may also be warranted along the shoring, to avoid any potential undermining.

8.4 FOUNDATION CONCRETE

The degree of exposure of the soils to sulphate attack is classified in A23.1-04 (Concrete Materials and Methods of Concrete Construction) as moderate, severe or very severe. Based on significant data gathered through previous work in the Winnipeg and surrounding area and in accordance with the Manitoba Building Code, the degree of exposure for soil in Winnipeg is commonly classified as severe. Therefore, all concrete in contact with the native soils should be made with sulphate resistant cement (CSA Type HS). Furthermore, the concrete should have a minimum specified 56-day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Tables 2 and 3, CSA-A23.1-04. Concrete exposed to freeze-thaw cycles should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA-A23.1-04.

8.5 TESTING AND MONITORING

The engineering design recommendations presented within this report are based on the assumption that an adequate level of testing and monitoring will be provided during construction and that qualified contractors experienced in foundations and earthworks will carry out the construction. An adequate level of testing and monitoring are considered to be full-time monitoring and design review during the installation of piled foundations and regular monitoring and compaction testing for earthworks related to the abutment areas. AMEC further requests the opportunity to review drawings and specifications related to any foundations, earthworks or other designs based on the recommendations provided in this report to confirm that said recommendations have been correctly interpreted.

In additional, as for riverbank slope stability, a site visit a recommended one year after the completion of the construction to verify the performance of the bridge and the riverbank slope is in good stake.

9.0 CLOSURE

The findings and recommendations of this report were based on the results of field and laboratory investigations, combined with an interpolation of soil and ground water conditions encountered at one (1) test hole location. If conditions are encountered that appear to be different from those shown by the test hole drilled at this site and described in this report, or if the assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be reviewed and adjusted, if necessary.

The site investigation was conducted for the sole purpose of assessing geotechnical conditions. Although no environmental issues were identified during the fieldwork, this does not indicate that no such issues exist. If the owner or other parties have any concern regarding the presence of environmental issues, then an appropriate level environmental assessment should be conducted.

Soil conditions, by their nature, can be highly variable across a site. The placement of fill and prior construction activities on a site can contribute to the variability especially near surface soil conditions. A contingency should always be included in any construction budget to allow for the possibility of variation in soil conditions, which may result in modification of the design and construction procedures.

This report was prepared exclusively for Stantec Consulting Ltd. and their agents for the proposed development as described in the report. The data and recommendations provided herein should not be used for any other purpose, or by any other parties, without review written authorization of AMEC. The use of this report by third parties is done so at the risk and responsibility of those parties. The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering principles and practice. No other warranty, expressed or implied, is given.

Yours truly,
AMEC Earth & Environmental

PROVINCE OF MANITOBA
11-9-07
H. D.
PANKRATZ
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No. 555 **Date:** 11-9-07

APPENDIX A

TEST HOLE LOGS

AMEC Earth & Environmental
440 DOVERCOURT DRIVE
WINNIPEG, MANITOBA

amec

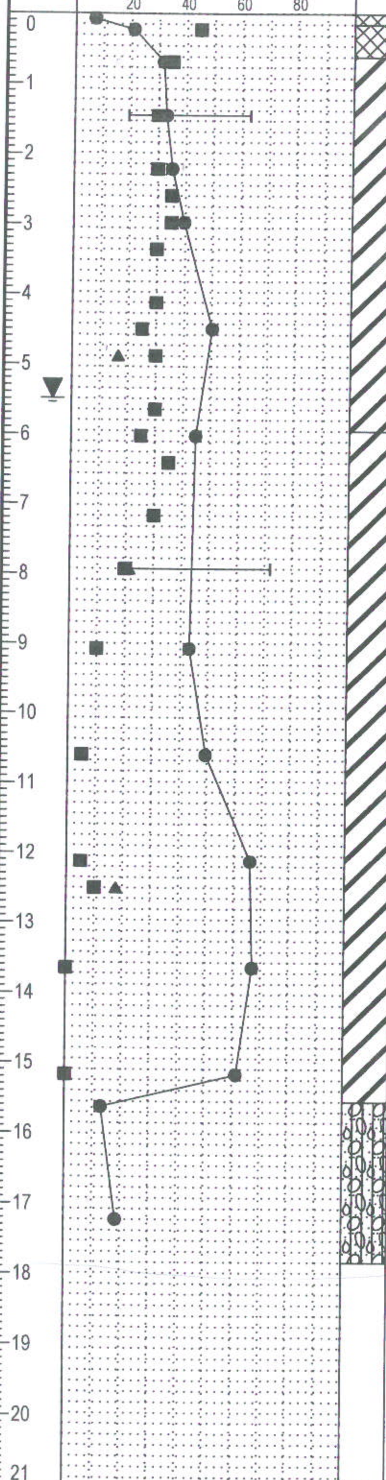
CLIENT LOGO	CLIENT: STANTEC CONSULTING LTD.		
PROJECT: PROPOSED BRIDGE REPLACEMENT BUNN'S CREEK AT BONNER AVENUE WINNIPEG, MANITOBA	DWN BY: AL	DATUM: N/A	DATE: JULY 2007
	CHK'D BY: TG	REV. NO.: A	PROJECT NO: WX15591
TITLE: PROPOSED TEST HOLE LOCATION PLAN	PROJECTION: N/A	SCALE: NTS	FIGURE No. 1

PROJECT: Proposed Bridge Replacement	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH01**
CLIENT: Stantec Consulting Ltd.	DRILL TYPE: M40LX	PROJECT NO: WX 15591
LOCATION: Bunn's Creek @ Bonner Ave., Wpg, MB.	DRILL METHOD: 125 mm SSA	ELEVATION: 99.54 m

SAMPLE TYPE: Shelby Tube | No Recovery | SPT (N) | Grab Sample | Split-Pen | Core

BACKFILL TYPE: Bentonite | Pea Gravel | Drill Cuttings | Grout | Slough | Sand

Depth (m) | ▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400 | ■ POCKET PENETROMETER (kPa) ■ 100 200 300 400 | PLASTIC M.C. LIQUID 20 40 60 80 | SOIL SYMBOL | MUSCS | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE NO | SPT (N) | COMMENTS | ELEVATION (m)



Depth (m)	MUSCS	SOIL DESCRIPTION	SAMPLE NO	SPT (N)	COMMENTS
0	GP	GRAVEL (FILL) - some sand to sandy, poorly graded, fine grained, wet, dense, light brown, trace silt	1, 2		
	CH	CLAY (FILL) - high plastic, moist, very stiff, dark grey, some silt, sand and gravel, occasional silt and oxidation inclusions	3		
	CH	CLAY (ALLUVIAL) - silty, high plastic, moist, stiff, brownish grey, occasional sand and gravel inclusions, occasional oxidation inclusions, occasional rootlets, organics lenses and inclusions - dark greyish brown below 1.2m - occasional oxidation and sulphate inclusions below 2.7m - occasional sulphate lenses (approximately 60 degree from horinzontal) at 3.8m - frequent sulphate lenses between 4.0 to 4.1m - occasional sulphate pockets (<2cm dia) below 4.6m	4, 5, 6, 7, 8, 9, 10, 11, 12		Lab Vane @ Sample 11: 86 kPa
	CH	CLAY - some silt to silty, high plastic, moist, stiff, mottled grey and brown, trace sand, occasional silt inclusions - firm to stiff, dark grey, occasional sand and gravel inclusions, occasional silt pockets (<2cm dia) - very moist, soft to firm below 13 m	13, 14, 15, 16, 17, 18, 19, 20, 21, 22		Notes: Observed 60 degree from horizontal slickenslide encountered at 5.6m Lab Vane @ Sample 16: 104 kPa Lab Vane @ Sample 20: 52 kPa
	ML	SILT (TILL) - clayey, low plastic, moist, medium dense to dense, light brown, trace to some sand, trace gravel - a seepage zone below 15.9m - very dense, some gravel to gravelly below 17.1m	23, 24	71/287	Notes: Hard drilling conditions at 16.8m

AUGER REFUSAL AT 17.8m BELOW EXISTING GROUND SURFACE ON INFERRED BOULDERS

NOTES:

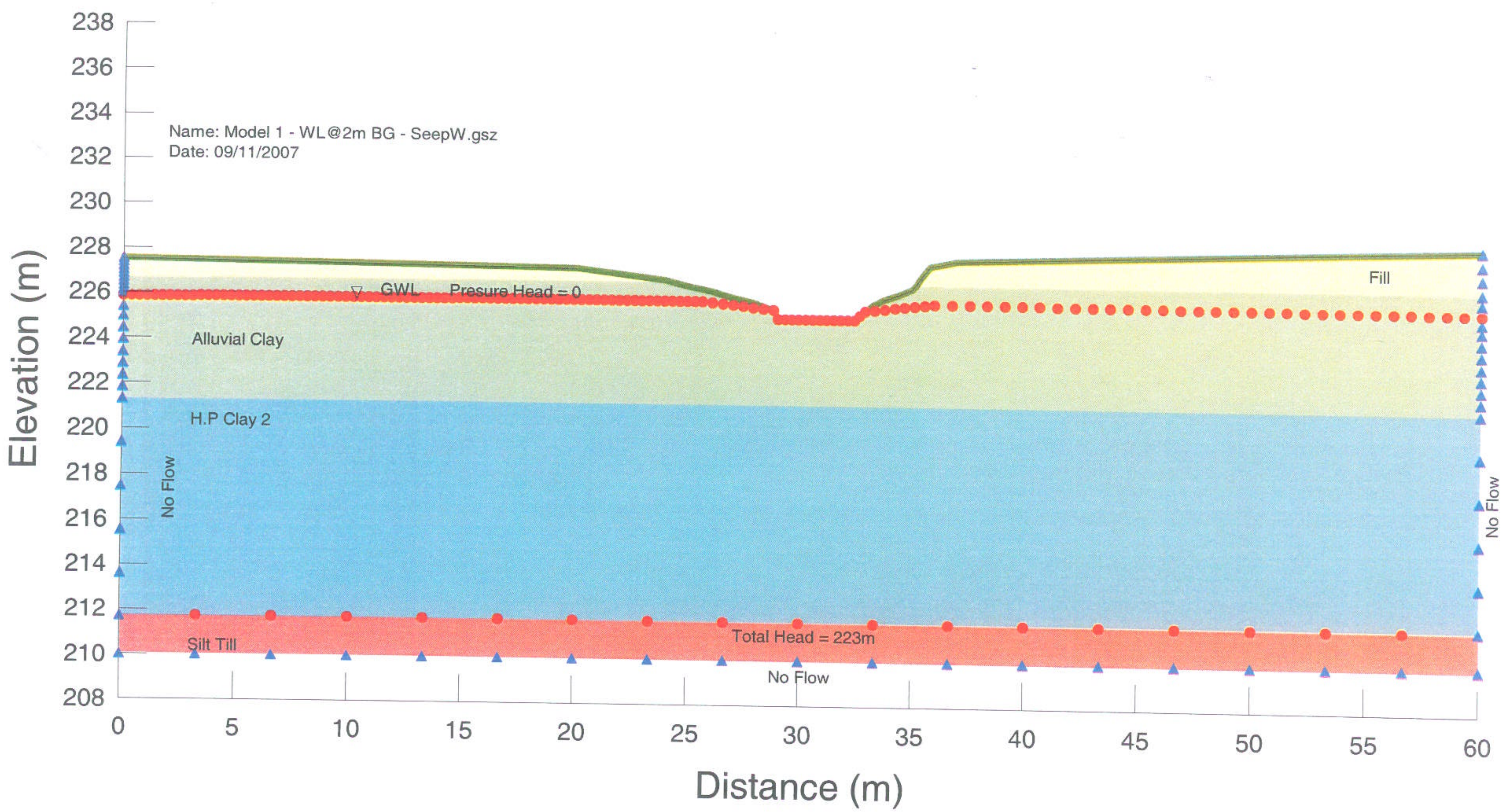
Significant seepage but no sloughing were encountered in silt till upon completion of drilling. Test hole open to 17.8m and water level at 5.5m below grade upon completion of drilling. Test hole backfilled with bentonite and auger cuttings upon completion of drilling.

amec	AMEC Earth and Environmental Winnipeg, Manitoba	LOGGED BY: LKT REVIEWED BY: HP Figure No. 2
		COMPLETION DEPTH: 17.8 m COMPLETION DATE: 13 July 2007

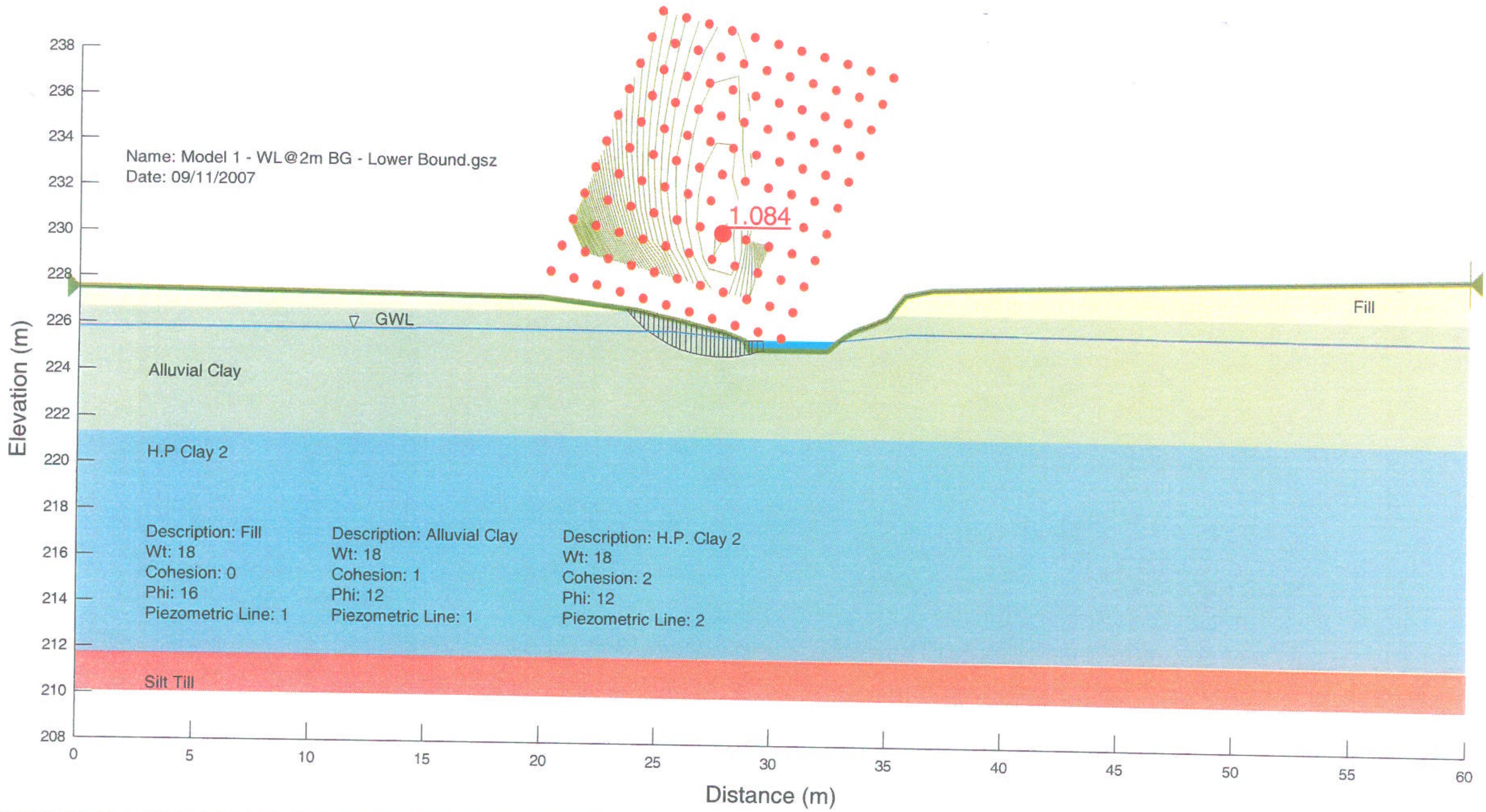
15591 - BUNN'S CREEK.GPJ 07/09/11 04:00 PM (GEOTECHNICAL)

APPENDIX B

SLOPE STABILITY ASSESSMENT
&
LATERAL EARTH PRESSURES



amec Earth & Environmental *A Division of AMEC Americas Limited*		PROPOSED BRIDGE REPLACEMENT STANTEC CONSULTING LTD.		SLOPE STABILITY ANALYSIS CROSS SECTION A-A SEEPAGE MODEL BUNN'S CREEK@ BONNER AVENUE WINNIPEG, MANITOBA
Drawn by: LKT	Scale: As Shown	Date: Aug/07	Proj. No: WX15591	Figure: B1



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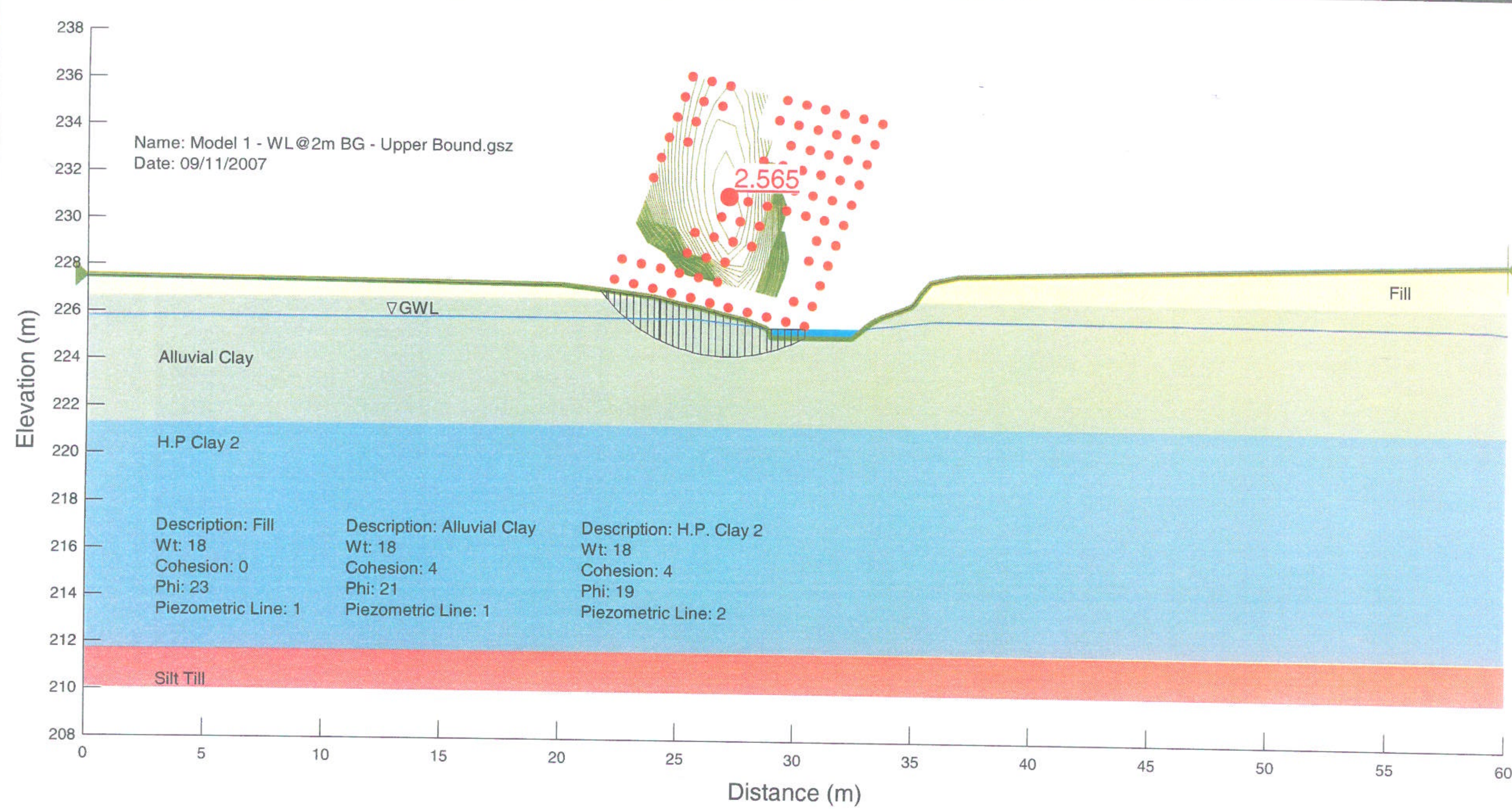
PROPOSED BRIDGE REPLACEMENT

STANTEC CONSULTING LTD.

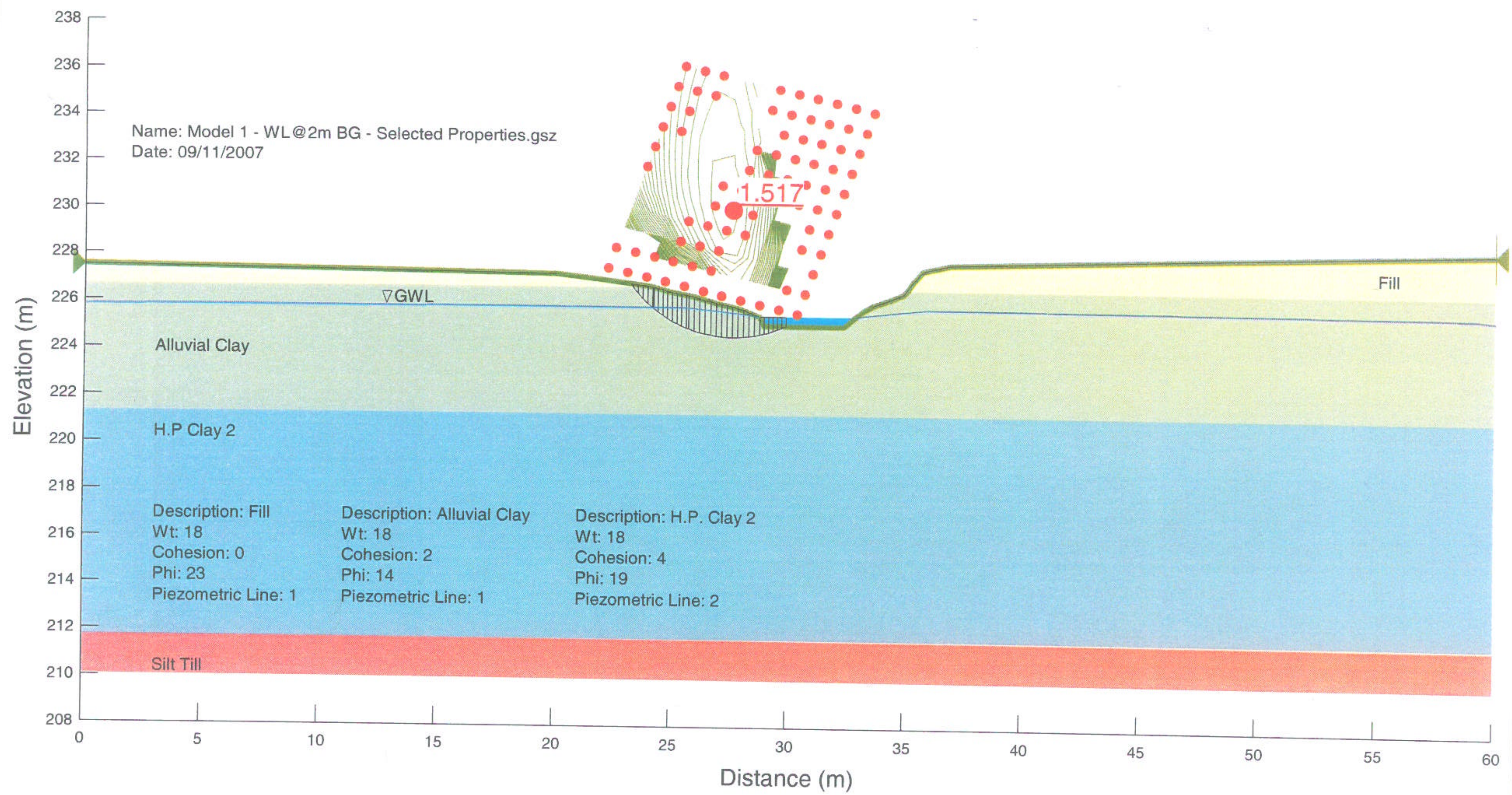
SLOPE STABILITY ANALYSIS
CROSS SECTION A-A
ESTIMATED LOWER BOUND SOIL PROPERTIES

BUNN'S CREEK@ BONNER AVENUE
WINNIPEG, MANITOBA

Drawn by: LKT	Scale: As Shown	Date: Aug/07	Proj. No: WX15591	Figure: B2



amec *Earth & Environmental* *A Division of AMEC Americas Limited*		**PROPOSED BRIDGE REPLACEMENT** **STANTEC CONSULTING LTD.**		**SLOPE STABILITY ANALYSIS** **CROSS SECTION A-A** **ESTIMATED UPPER BOUND SOIL PROPERTIES** **BUNN'S CREEK@ BONNER AVENUE** **WINNIPEG, MANITOBA**
Drawn by: LKT	Scale: As Shown	Date: Aug/07	Proj. No: WX15591	Figure: B3



amec Earth & Environmental A Division of AMEC Americas Limited		PROPOSED BRIDGE REPLACEMENT STANTEC CONSULTING LTD.			SLOPE STABILITY ANALYSIS CROSS SECTION A-A SELECTED SOIL PROPERTIES BUNN'S CREEK@ BONNER AVENUE WINNIPEG, MANITOBA
Drawn by: LKT	Scale: As Shown	Date: Aug/07	Proj. No: WX15591	Figure: B4	