AGRA
Earth & Environmental

AGRA Earth &
Environmental Limited
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December 14, 1995
Project: WX-03838

DWL Engineering Inc.
210 - 530 Kenaston Blvd.
Winnipeg, Manitoba
R3N 1Z4

Attention: Mr. Fred Kemp, P. Eng.

Dear Sir:

RE: GEOTECHNICAL INVESTIGATION
PROPOSED GREATER WINNIPEG WATER DISTRICT RAILWAY BRIDGE
EAST BRAINTREE, MANITOBA

1.0 INTRODUCTION

As authorized by Mr. Fred Kemp, P. Eng., acting on behalf of DWL Engineering Inc., AGRA Earth & Environmental Limited (AEE) performed a geotechnical investigation for a proposed bridge on the Greater Winnipeg Water District (GWWD) railway line at East Braintree, Manitoba. The scope of the geotechnical investigation was to determine the subsurface soil stratigraphy and auger refusal depths at each proposed abutment location in order that a preliminary indication of approximate steel H pile refusal depths could be established.

The work was undertaken in accordance with a proposal submitted to Fred Kemp Engineering Ltd., on August 9, 1995.

2.0 DESCRIPTION OF SITE AND PROPOSED FACILITIES

The site of the proposed railway bridge is located at the GWWD crossing of the Boggy River, about 2 km south of the Trans Canada Highway at the town of East Braintree, Manitoba. A wooden bridge was present at the site. The existing bridge was about 27 m long.

The design details for the proposed new bridge were not made available to AEE at the time of this report. However, it was understood that the bridge would be a single span steel structure. The new abutments would be located in close proximity to the existing abutments, which are about 7 m from the water's edge. No new fills were proposed.

3.0 RESULTS OF INVESTIGATION

3.1 Field Work

A total of 2 test holes were drilled at the site on November 21 and 22, 1995. The test holes were drilled with a track mounted drill rig equipped with 200 mm hollow stem augers and wire line sampling equipment. The drill rig was placed on the rails and the test holes were drilled between the railway ties immediately behind the existing bridge abutments. The test hole locations are shown on the attached plan, Figure 1.

Test Hole 1 was drilled at the proposed west abutment location and was augered to a depth of 24.8 m from grade, where auger refusal occurred on suspected boulders. After auger refusal had occurred, a dynamic cone was driven to a depth of 28.5 m from grade, where cone refusal occurred. It is not known if the cone refused on bedrock or on boulders within the glacial till.

Test Hole 2 was drilled at the proposed east abutment location and was drilled to a depth of 23.2 m from grade. At this depth, refusal of both the hollow stem augers and the standard penetration test (SPT) sampler occurred. Driving a dynamic cone was not attempted at this location.

Disturbed soil samples were recovered at selected depths within both test holes by means of a sampler used in the SPT. The SPT consists of the driving of a 50 mm diameter split barrel sampler a total of 450 mm into the soil using a drop weight weighing 63.5 kg and falling 76 cm. The number of blows required to drive the sampler the final 300 mm is recorded as the N value shown on the test hole logs. The N value is a measure of the relative density of cohesionless soils or relative consistency of cohesive soils. It can be correlated empirically to soil strength and stiffness parameters relevant to the design and performance of foundations. The dynamic cone driven at Test Hole 1 does not retrieve samples for visual classification. However, as with the SPT, the number of blows to drive the cone 300 mm are recorded and the cone penetration resistance is a measure of the relative density of the soil. It should be appreciated, however, that build up of frictional forces along the drill rods trailing the cone often lead to misleading values for the cone penetration, once the cone has penetrated substantially into the soil.

The recovered soil samples were visually classified at the time of drilling by AEE's field technician. The soil profiles, as determined at the time of drilling, are shown on the test hole logs, Figures 2 and 3.

3.2 Laboratory Testing

In the laboratory, moisture contents were determined for all soil samples obtained from the test holes, as a check on the relative moisture contents throughout the drilled depths and across the site.

4.0 SUBSOIL STRATIGRAPHY

The subsurface soil stratigraphy encountered at the site consisted of the following, as noted in descending order from the ground surface:

- granular fill (ballast)
- sand
- clay
- glacial till with sand layers

Rail ballast (crushed rock) was present at the ground surface at both test hole locations. The ballast was 3.1 m thick at Test Hole 1 and 2.7 m thick at Test Hole 2.

Underlying the fill soils was a poorly graded, fine sand. The sand contained some silt and was brown, moist and loose. At Test Hole 1, the sand layer extended to 6.6 m from grade and was saturated below a depth of 4.9 m. At Test Hole 2, the sand was moist throughout and extended to 4.0 m from grade.

A highly plastic clay was present below the sand at both test holes. The clay was moist to very moist, soft to very soft and grey. Silt lenses were present within the clay throughout the deposit. A decrease in moisture contents with depth (with no corresponding gain in soil strength) indicates an increase in silt content and a decrease in plasticity with depth. The clay layer extended to about 13.0 m from grade at Test Hole 1 and 14.5 m from grade at Test Hole 2.

At Test Hole 1, a glacial silt till was present below the clay, at 13.0 m from grade. The till was initially medium dense, low to non-plastic, sandy and moist. A layer of wet, loose, fine, silty sand was observed from 16.0 to 19.5 m from grade, and was underlain by additional glacial till. The composition of the glacial till underlying the sand was similar to that of the till above the sand, however was loose to medium dense and very sandy. Cobbles and boulders were present within the till below about 22.5 m from grade.

At Test Hole 2, a layer of sand was present immediately below the clay. The sand was fine, poorly graded, medium dense and grey. The sand extended to 17.0 m from grade after which a glacial silt till, similar to that described for Test Hole 1 was identified. Auger refusal occurred at 23.2 m from grade, however it could not be confirmed if refusal occurred on bedrock or on boulders within the till.

5.0 RECOMMENDATIONS

Driven steel H piles are considered to be a feasible foundation alternative for the proposed bridge. Bored piles are not considered suitable, given the loose, wet overburden soils present at the site. The depth to bedrock at the site likely negates the use of conventional driven precast concrete piles.

Refusal to drilling and/or the dynamic cone occurred at depths of 28.5 and 23.2 m below grade in Test Holes 1 and 2, respectively. It could not be determined if refusal occurred on bedrock or boulders in the till. Therefore, it is possible that piles would penetrate to greater depth.

Ideally, it would be desirable to drive a test pile at each abutment to establish refusal depth. If this is not practical, contract documents should recognize the potential requirement for additional pile length.

Steel H piles driven to practical refusal on the granite bedrock can be designed on the basis of an allowable capacity equal to 0.3 times the yield stress of the steel. Practical pile refusal can be considered to be about 15 blows per 25 mm of pile penetration, assuming that the piles are driven with a hammer having a minimum driving energy of 40 kJ per blow and consist of conventional HP310 sections. Actual refusal criteria should be established once the actual pile sizes and steel area is known. All piles should be fitted with rock points (driving shoes) for penetration into the underlying bedrock. Due to the long pile lengths necessary and the difficult driving conditions expected near the refusal depths (as a result of boulders), a light weight steel section is not recommended. A minimum HP 310 x 130 steel pile is recommended. Pile spacing should not be less than 2.5 pile diameters, centre to centre.

Full time monitoring of the pile driving operation by qualified geotechnical personnel is recommended in order to assess pile behaviour near and at refusal.

During the final design stage, the lateral load capacity of the steel H piles should be assessed. Due to the loose, soft overburden soils present at this location, and as substantial point resistance will likely not be achieved, the piles will not have a high lateral load capacity. It is likely that if substantial lateral loads are present, resistance may require the use of battered piles.

6.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering principles and practice. The findings and recommendations have been based on the results of field and laboratory investigations combined with an interpolation of soil and groundwater conditions between test hole locations. If conditions encountered during construction appear to be different than those shown by the test holes drilled at this site, this office should be notified in order that the recommendations can be reviewed.

Yours truly,

AGRA Earth & Environmental Limited

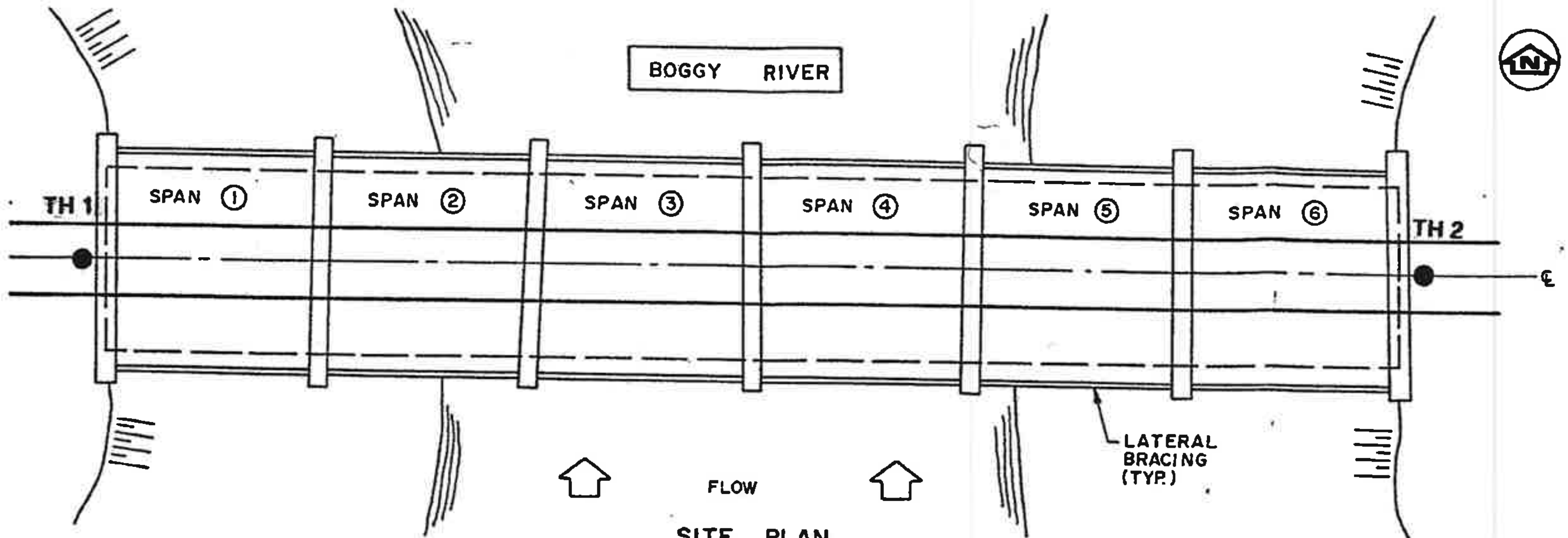
PROVINCE OF MANITOBA
15-12-95
H. D.
PANKRATZ
REGISTERED PROFESSIONAL ENGINEER

Harley Pankratz, P.Eng.
Manager; Winnipeg Operations

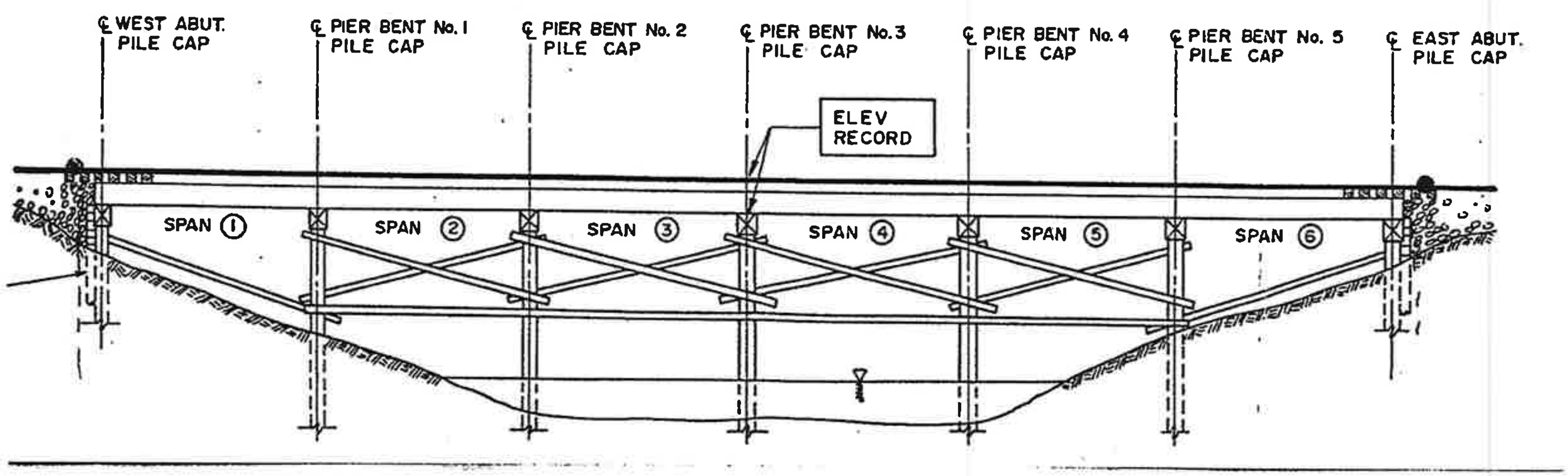
Reviewed by:
Brian A. Ross, P.Eng.
Vice President; Manitoba/Saskatchewan

3738REP1.HDP

Dist: (2) Addressee
(2) Fred Kemp Engineering



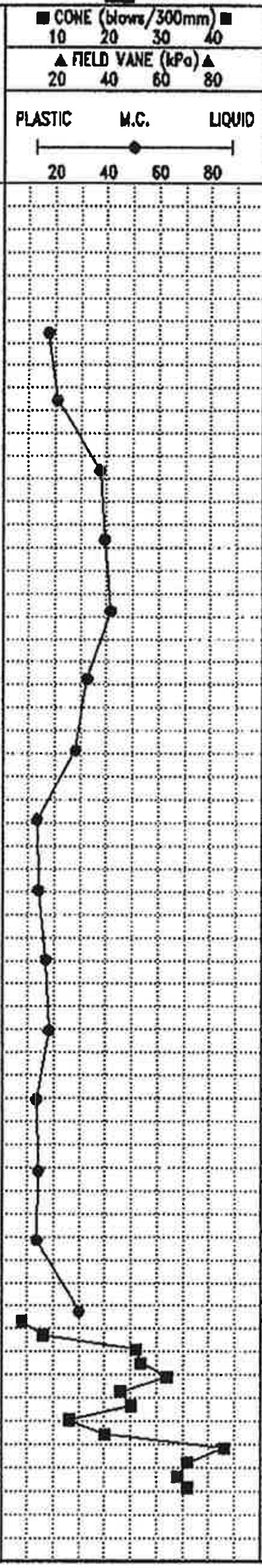
SITE PLAN
SCALE: 1/8" = 1'-0"



AGRA Earth & Environmental Limited		DELCAN WESTERN LTD.			TESTHOLE LOCATION PLAN EAST BRAINTREE BRIDGE GREATER WINNIPEG WATER DISTRICT EAST BRAINTREE, MB.
Drawn: BY OTHERS	Scale: NTS	Date: DEC 95	Project No: WX-03838	Figure: 1	

PROJECT: GEO. BRIDGE INVESTIGATION	DRILLER: PADDOCK DRILLING LTD.	TEST HOLE NO: 1
CLIENT: DELCAN WESTERN LTD.	DRILL: RM-30	PROJECT NO: WX-03838
LOCATION: EAST BRAINTREE, WEST ABUTMENT	AUGER: 200mm HSA	ELEVATION: 99.846 (m)

SAMPLE TYPE: SHELBY TUBE | CUTTINGS | SPT | CORE | NO RECOVERY | CONT. SAMPLE



DEPTH (m)	SAMPLE TYPE	SAMPLE NO	SPT(N)	USC	SOIL DESCRIPTION	COMMENTS	ELEVATION (m)
0.0–3.0				FI	FILL - BALLAST GRAVEL frozen to 0.6m		
3.3	SPT	1	5	SP	SAND - poorly graded, fine, moist, loose, brown, traces of silt - wet below 4.9m - wood pieces present between 6.4m & 6.6m		
4.8	SPT	2	6	SP			
6.3	SPT	3	3	CH	CLAY - high plastic, moist, soft, grey, traces of silt lensing.		
7.8	SPT	4	3	CH			
9.4	SPT	5	2	CH			
10.9	SPT	6	3	CH			
12.4	SPT	7	3	CH			
14.0	SPT	8	10	ML	SILT (GLACIAL TILL) - low to non-plastic, moist, loose to medium dense grey, some wet sand lenses present.		
15.5	SPT	9	13	ML			
17.0	SPT	10	8	SP	SAND - poorly graded, fine, wet, loose, brown		
18.6	SPT	11	8	SP			
20.1	SPT	12	6	ML	SILT (GLACIAL TILL) - low to non-plastic, moist, medium dense, grey, very sandy. - cobbles/boulders at 22.5m - wet at refusal		
21.6	SPT	13	9	ML			
23.1	SPT	14	12	ML			
24.7	SPT	15	12	ML			
24.8–28.5					Auger refusal at 24.8m from grade. Drove dynamic cone from 24.8m to 28.5m. Backfilled hole upon completion.		
28.5					Abrupt cone refusal at 28.5m from grade.		

Depth scale (m): 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0, 20.0, 21.0, 22.0, 23.0, 24.0, 25.0, 26.0, 27.0, 28.0, 29.0, 30.0

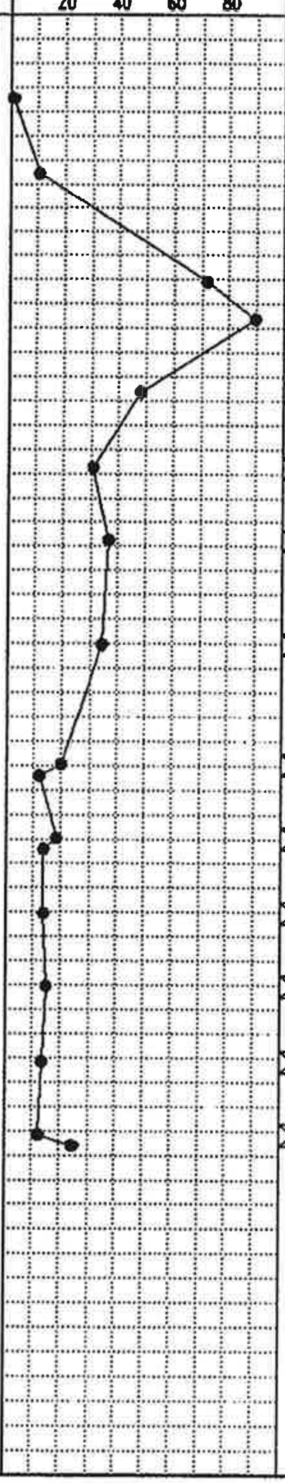
Elevation scale (m): 99.0, 98.0, 97.0, 96.0, 95.0, 94.0, 93.0, 92.0, 91.0, 90.0, 89.0, 88.0, 87.0, 86.0, 85.0, 84.0, 83.0, 82.0, 81.0, 80.0, 79.0, 78.0, 77.0, 76.0, 75.0, 74.0, 73.0, 72.0, 71.0, 70.0

AGRA Earth & Environmental Limited Winnipeg, Manitoba	LOGGED BY: DRS	COMPLETION DEPTH: 28.5 m
	REVIEWED BY: BAR	COMPLETE: 22/11/95
	Fig. No: 2	Page 1 of 1

95/12/14 10:27AM

PROJECT: GEO. BRIDGE INVESTIGATION	DRILLER: PADDOCK DRILLING LTD.	TEST HOLE NO: 2
CLIENT: DELCAN WESTERN LTD.	DRILL: RM-30	PROJECT NO: WX-03838
LOCATION: EAST BRAINTREE, EAST ABUTMENT	AUGER: 200mm HSA	ELEVATION: 99.918 (m)

SAMPLE TYPE: SHELBY TUBE | CUTTINGS | SPT | CORE | NO RECOVERY | CONT. SAMPLE



DEPTH (m)	SAMPLE TYPE	SAMPLE NO	SPT(N)	USC	SOIL DESCRIPTION	COMMENTS	ELEVATION(m)
0.0 – 2.8	SPT	1	5	Fl	FILL – BALLAST GRAVEL frozen to 0.6m		99.918
2.8 – 4.6	SPT	2	7	SP	SAND – poorly graded, fine, moist, loose, brown, traces of silt		
4.6 – 15.0	SPT, CUTTINGS	3, 4, 5, 6, 7, 8, 9	2, 2, 2, 3, 4	CH	CLAY – high plastic, very moist, very soft, grey – silt lenses present below 10.7m		
15.0 – 17.5	SPT	10, 11, 12, 13	15, 13	SP	SAND – poorly graded, fine, wet, medium dense grey – layers of SILT till present below 15.5m		
17.5 – 23.2	SPT	14, 15, 16, 17, 18	8, 7, 8, 21/210	ML	SILT (GLACIAL TILL) – low to non-plastic, moist, loose to medium dense, brown, sand and gravel – wet coarse grained sand from 23.1m to 23.2m		
23.2					Auger and SPT sampler refusal at 23.2m from grade. Backfilled hole upon completion.		

Depth scale (m): 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 9.0, 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0, 20.0, 21.0, 22.0, 23.0, 24.0, 25.0, 26.0, 27.0, 28.0, 29.0, 30.0

Elevation scale (m): 99.0, 98.0, 97.0, 96.0, 95.0, 94.0, 93.0, 92.0, 91.0, 90.0, 89.0, 88.0, 87.0, 86.0, 85.0, 84.0, 83.0, 82.0, 81.0, 80.0, 79.0, 78.0, 77.0, 76.0, 75.0, 74.0, 73.0, 72.0, 71.0, 70.0

AGRA Earth & Environmental Limited Winnipeg, Manitoba	LOGGED BY: DRS	COMPLETION DEPTH: 23.2 m
	REVIEWED BY: BAR	COMPLETE: 21/11/95
	Fig. No: 3	Page 1 of 1

95/12/14 10:00AM

Burmey, Darren

From: Pankratz, Harley [harley.pankratz@amec.com]
Sent: Wednesday, August 02, 2006 1:45 PM
To: Burmey, Darren
Subject: East Braintree
Attachments: WX03838 Geo Inv, Proposed Greater Winnipeg Water District Ra.pdf

Hi Darren

Here is the report we did in 95.

Regards

Harley

31 October 2006
Project No. WX15408

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

Attention: Mr. Darren Burmey, P. Eng.

Re: Geotechnical Evaluation
Proposed Greater Winnipeg Water District Bridge
East Braintree, Manitoba

1.0 INTRODUCTION

At the request of Mr. Darren Burmey, P. Eng., of Stantec Consulting Inc., AMEC Earth & Environmental (AMEC) completed a geotechnical evaluation of the existing Greater Winnipeg Water District (GWWD) bridge site located at East Braintree, Manitoba. AMEC (then AGRA Earth & Environmental Limited) previously completed a geotechnical investigation at the site, the report for which was submitted to DWL Engineering Ltd. in December 1995.

2.0 SCOPE OF WORK

As outlined in AMEC's proposal dated 3 August 2006, the scope of work for the project included the following tasks:

- Conduct a site visit to review existing site conditions, including the proposed bridge alignment as well as the riverbank conditions relative to signs of instability or erosion;
- Review the structural drawings for the proposed bridge and provide geotechnical input to the bridge designers;
- Review the existing geotechnical report and provide additional comments and recommendations as required to proceed with the foundation design for the project;
- Provide recommendations for fill placement and grading adjacent to the proposed bridge abutments; and
- Submit a geotechnical report summarizing our review.

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AMEC's site visit was completed on 5 October 2006, in conjunction with Mr. Darren Burmey, P. Eng. of Stantec and Mr. Cam MacInnes, P. Eng. of Unies Consulting.

3.0 SITE CONDITIONS

The existing bridge is located at mile 77.6 of the GWWD rail line. The bridge is located about 2 km south of the TransCanada Highway and runs in an approximate east/west direction over the Boggy River. The existing structure is a six span timber bridge, with short approach spans at each end, followed by four spans traversing the river channel. The length of the existing bridge is about 27 m. A site plan showing the existing bridge alignment is shown in Figure 1.

The Boggy River is relatively straight at the bridge crossing and flows northwards. The existing bank height ranges from about 3 to 5 m, with the rail bridge (top of rail track) just over 5 m above the water level at the time of the site visit. The rail line is built up above surrounding grades, most noticeably on the west side of the bridge where grades are about 2 to 3 m lower than the top of rail elevation. The riverbank is heavily treed upstream of the bridge, however, is relatively clear of significant vegetation downstream, where the Shoal Lake aqueduct crosses the river. A private bridge leading to a residence is located immediately north (downstream) of the aqueduct and GWWD bridge.

Immediately south of the bridge, a culvert drains into the River. The culvert was apparently connected to a drainage ditch located east of the site, which runs along the south side of the rail line and east of Highway 308. It was understood in discussions with the adjacent resident that a ditch was formerly present just south of the bridge, however, had been infilled and the culvert installed about 20 years ago.

There were no signs of significant slope instabilities at the bridge site, although a small failure scarp was evident just south of the bridge (see Photo 1), on the east side of the River, and in the vicinity of the above referenced culvert. At this location, it was evident that some construction equipment had recently accessed the river, apparently to clear debris from the upstream side of the bridge. There were indications of recently placed gravel within the river (see Photo 2), as well as obvious disturbance of the slope in this area. It was unclear if the small scarp observed (noted above) was a result of this recent re-grading, or if it was a pre-existing condition, possibly due to the historic fill placement above the culvert.

There was limited evidence of erosion at the river's edge, with no signs of concern. The existing bridge abutments are made of timber and were armoured at the toe with a zone of crushed rock which slopes from the water line to the abutment (see Photo 3). The abutments appeared to be in good condition, with no obvious signs of lateral movement towards the river (see Photo 4).

4.0 DISCUSSION

It was understood that the proposed new bridge will be of steel construction and will have a single span, with a total bridge length about 2 m shorter than the existing. As a result, it is preferred that the new abutments be about 2 m closer together than what currently exists. At this time, AMEC was unaware if each abutment would be moved equally, or if only one of the abutments would be moved. Given the existing conditions, there does not appear to be any geotechnical issues that would prevent moving the abutments closer to each other.

In the previous geotechnical report, driven steel H piles were recommended as the preferred foundation alternative. Auger refusal depths of 25 and 23 m from grade were encountered at the two test holes, however, depending on the actual conditions encountered during driving, it is possible that piles may be advanced past the refusal depths noted. Driven precast piles are also a potentially suitable option, however, the depth to anticipated pile refusal (25 m from top of rail) may preclude their use, as conventional pile lengths are limited to about 20 m. Where the piles can be cut off at the river bed elevation, further consideration to their use can be made. Depending on actual foundation loads, driven precast concrete piles, designed based on a combination of frictional resistance and limited end bearing may also be considered (i.e. driven to elevation, not refusal). This type of pile system would potentially have capacities in the order of 350 to 500 kN per pile, for a 400 mm diameter pile. Bored piles are not recommended as a result of the wet, sloughing soil conditions encountered.

Difficult driving conditions are anticipated near to refusal depths, given the apparent bouldery conditions near to the refusal depth. Where steel piles are used, the piles should be fitted with driving shoes to ensure adequate penetration to a suitable bearing layer, and to limit potential for damage to the pile tips. Care should be taken when nearing refusal, as pile damage could occur where piles are overdriven.

As outlined in the geotechnical report, steel piles can be designed based on an allowable capacity equal to 0.3 times the yield stress of the steel multiplied by the cross sectional area of the steel, assuming refusal in dense glacial till or bedrock is achieved. The actual pile capacities should be confirmed once the loads, pile sizes and driving criteria are established. It is recommended that the piles be driven with a hammer having a minimum driving energy of 40 kJ/blow, but it should also be verified that driving energies do not exceed about 600 joules/cm^2 of steel area. The remaining recommendations for driven steel piles are provided in the previously submitted geotechnical report.

Installation of all piles should be monitored on a full time basis by geotechnical personnel.

The following additional recommendations are provided for design and construction of the new bridge:

- Where the new abutments are placed down slope of the existing abutments, it is preferred that the old abutments be removed once the new abutments are installed. The space behind the new abutment wall should be backfilled with free draining granular fill, compacted to a minimum of 95% of standard Proctor maximum dry density.

- Active earth pressure coefficients should be used for the design of the retaining walls, with an earth pressure coefficient of 0.25. The active earth pressure coefficient assumes that granular materials are situated within a significant distance behind the wall (i.e. 1 to 2 m of new fill, plus existing rail embankment fill) and that some lateral movement of the wall will occur. For any embedded portions of the retaining wall, a passive earth pressure coefficient of 2.0 should be utilized within the native clay soils to a depth of 12 m from top of rail. Bulk unit weights of 23 kN/m^3 and 18 kN/m^3 should be used for the granular and clay soils, respectively.

- The fill between the abutment and the river should slope at a maximum slope of 4 horizontal to 1 vertical (4:1), extending to the bottom of the channel. This assumes the use of coarse granular soils.

- A heavy duty non-woven geotextile should be placed over the newly placed slope fill, anchored at both the top and bottom. Rockfill should be placed over the geotextile in a minimum 400 mm thick layer and should extend at least 5 m on either side of the new abutments. The existing rock at the toe of the abutments should be removed prior to new abutment construction and may be stockpiled for re-use. Rockfill material shall be sized according to the identified river flows, to avoid erosion during high water level conditions. AMEC would be pleased to provide further recommendations regarding rip-rap sizing and gradation once the design details and hydraulic study are complete.

- The existing slope to the south of the abutment (east side) should be re-graded to no steeper than 6:1 and all existing material within the channel should be removed after discussion with the Department of Fisheries and Oceans (DFO). The slope should be re-vegetated after the re-grading is complete. The remaining slopes may be left in their current condition.

- No fill materials should be stockpiled adjacent to the top of slope at any time during construction.

- All workings shall be performed in agreement with current guidelines and regulations from Manitoba Conservation and DFO.

Once the structural drawings are complete, AMEC requests the opportunity to review, in order to confirm that our assumptions are in keeping with the final design and to provide further input where necessary.

5.0 CLOSURE

This report has been prepared exclusively for Stantec Consulting Inc., and the Greater Winnipeg Water District for the proposed works described in the report. The findings and conclusions of the report were prepared in accordance with generally accepted engineering principles and practice. The findings and recommendations of the report were based on the results of field and laboratory investigations completed in 1995, combined with an interpolation of soil and groundwater conditions between test hole locations. If conditions are encountered during construction that are different than those shown in the 1995 report and as described above, AMEC should be notified so that the recommendations can be reviewed in light of the new information.

Please do not hesitate to call this office with any questions.

Sincerely,
AMEC EARTH & ENVIRONMENTAL

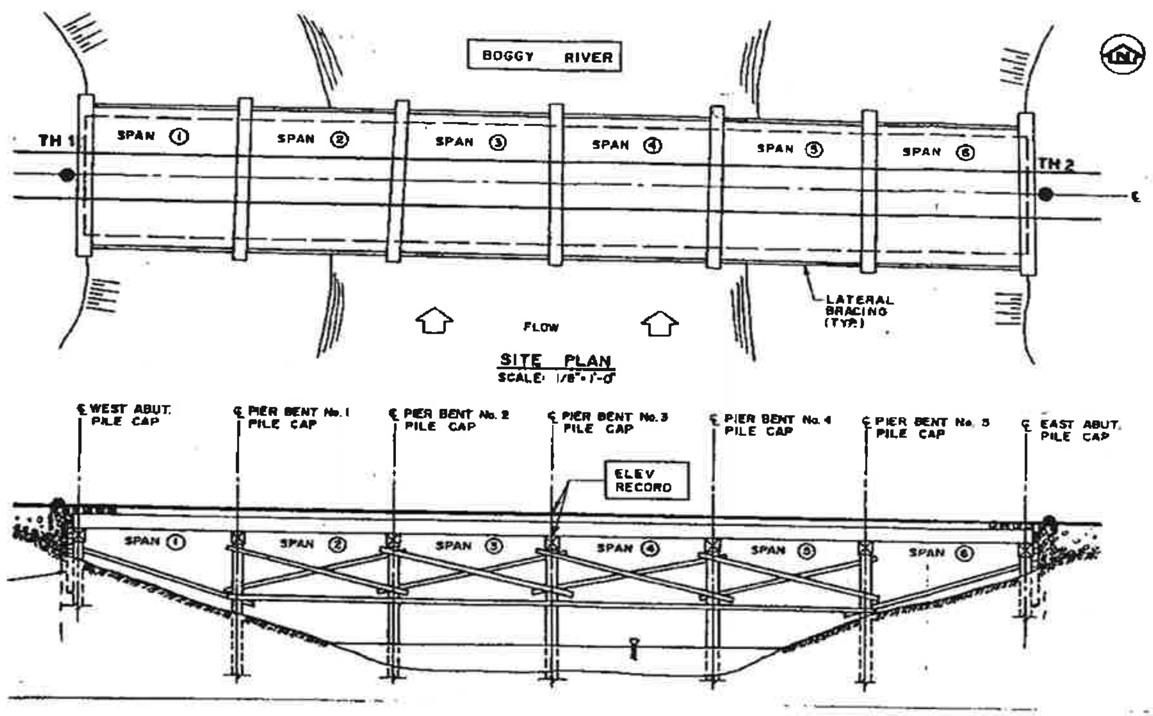
Harley Pankratz, P. Eng.
Vice President; Manitoba/Saskatchewan

PROVINCE OF MANITOBA
31-10-06
H. D.
PANKRATZ
REGISTERED PROFESSIONAL ENGINEER

APEGM
Certificate of Authorization
AMEC Earth & Environmental (MB)
No. 555 Expiry: April 30, 2007

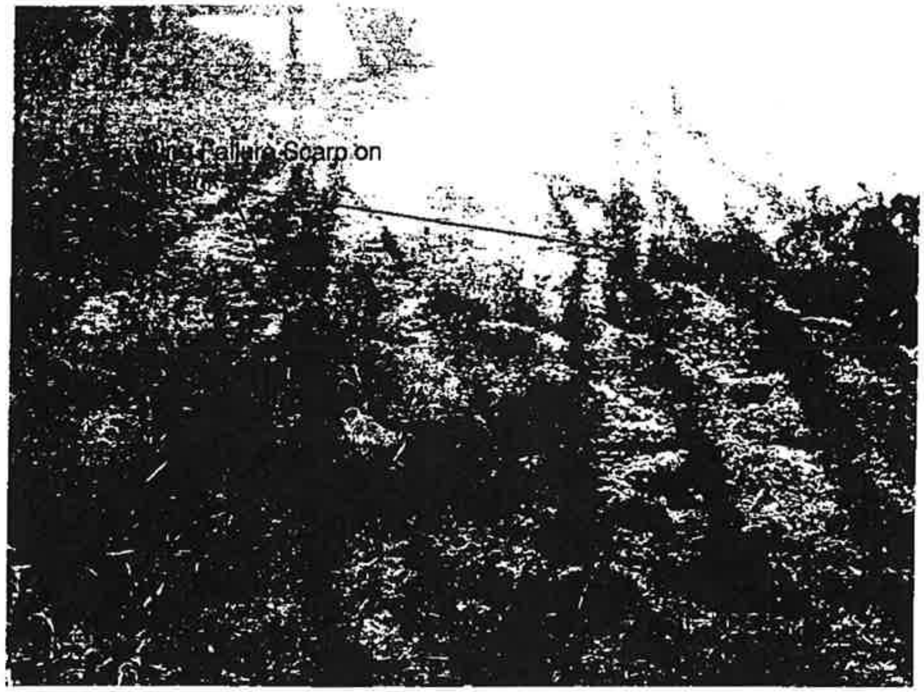
Reviewed By

Caius Priscu, PhD, P. Eng.
Regional Technical Leader

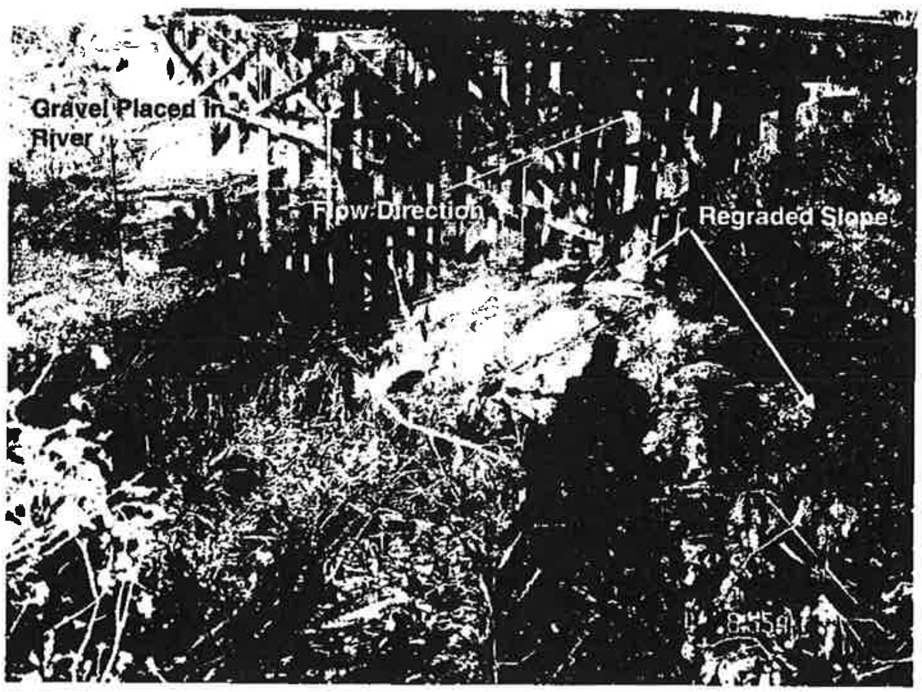


amec Earth & Environmental / STANTEC CONSULTING	SITE PLAN GWWD BRIDGE EAST BRAINTREE, MANITOBA

Drawn: N/A	Scale: NTS	Date: Oct 06	Project No.: WX15408	Figure: 1



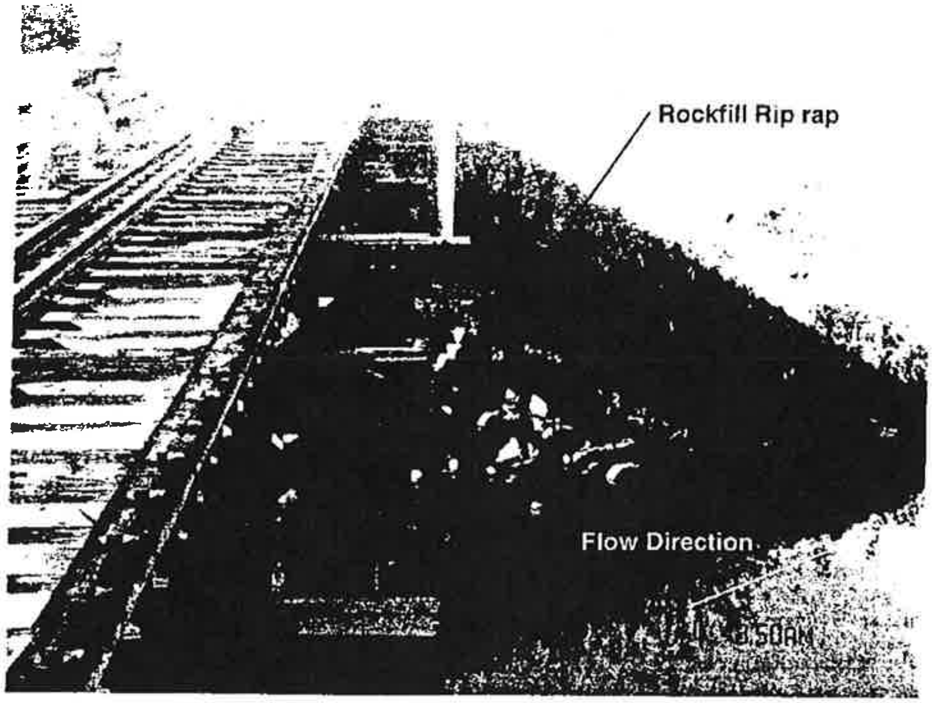
PHOTOGRAPH 1: Small Failure Scarp Noted South of The Bridge



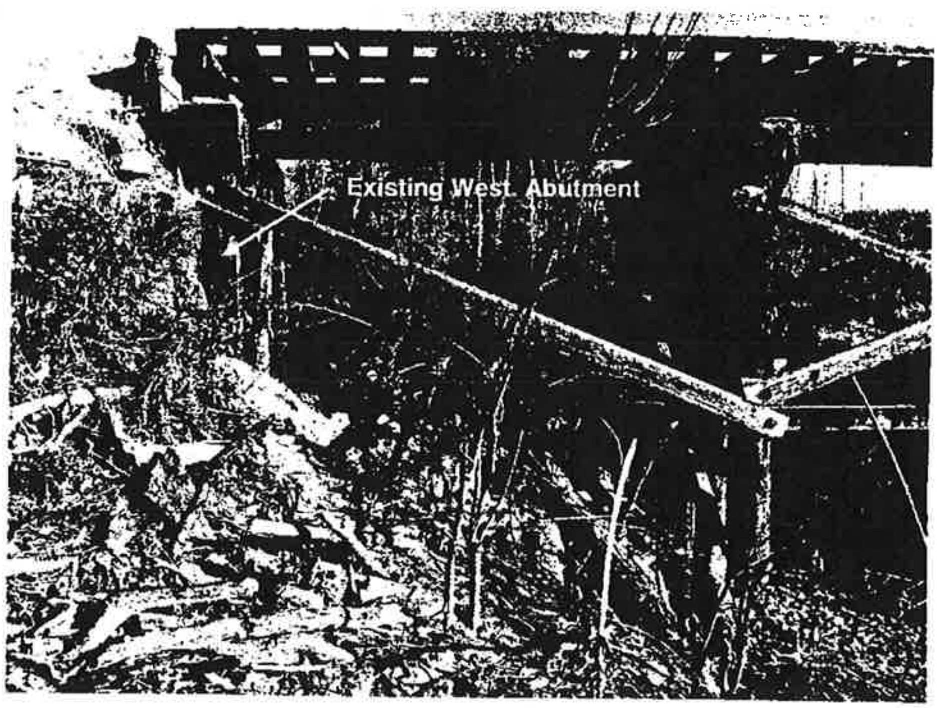
PHOTOGRAPH 2: Recently Placed Gravel In Channel

amec Earth & Environmental **STANTEC CONSULTING INC.**	**SITE PHOTOGRAPHS GEOTECHNICAL ASSESSMENT GWWD BRIDGE EAST BRAINTREE, MANITOBA**

Drawn: N/A	Scale: N/A	Date: OCT/06	Project No.: WX-15408	Figure: 2



PHOTOGRAPH 3: Existing Rockfill Armouring at Toe of Abutment



PHOTOGRAPH 4: Existing West Abutment

amec Earth & Environmental	**SITE PHOTOGRAPHS GEOTECHNICAL ASSESSMENT GWWD BRIDGE EAST BRAINTREE, MANITOBA**
STANTEC CONSULTING INC.	

Drawn: N/A	Scale: N/A	Date: OCT/06	Project No.: WX-15408	Figure: 3

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DRAFT REPORT
GEOTECHNICAL INVESTIGATION
GREATER WINNIPEG WATER DISTRICT
RAILWAY BRIDGE MILE 22.15
RM OF SPRINGFIELD, MANITOBA

Submitted to:

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5 February 2014

WX17312

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1.0 INTRODUCTION

At the request and authorization of Mr. Mike Boissonneault, P.Eng., Project Manager, and Senior Associate for Stantec Consulting Ltd (Stantec), AMEC Environment & Infrastructure, a division of AMEC Americas Limited (AMEC), completed a geotechnical investigation for the proposed replacement of the Greater Winnipeg Water District (GWWD) railway bridge located at Mile 22.15 in the RM of Springfield, Manitoba. The purpose of the geotechnical investigation was to verify the subsurface soil and groundwater conditions at the site in order to provide geotechnical recommendations for foundation design and construction. The scope of work for the project was outlined in AMEC's proposal number WPG2013.557, dated 3 December 2013. The geotechnical investigation was completed under subcontract to Stantec, Stantec Project Number and Subconsultant Agreement 113732050.

This report summarizes the field and laboratory testing programs, describes the subsurface conditions encountered at the test hole locations, and presents geotechnical engineering recommendations for: driven steel pile foundation alternatives; frost design considerations; abutment backfill and lateral earth pressures; and foundation concrete. Slope stability analyses, embankment settlement analyses, and pore pressure and fill staging analyses were not part of the scope of work for this geotechnical investigation. AMEC has assumed that these analyses are being undertaken by others as required for design.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Site Description

The GWWD Mile 22.15 site is located within the RM of Springfield, near the intersection of Edgewood Road and Centreline Road. Specifically, the site is located about 550 m west of Edgewood Road, where the GWWD Rail Line crosses Cook's Creek.

At Mile 22.15, the GWWD rail line consists of a single track with a siding located on the east side of the crossing. Currently the crossing consists of a double span wooden bridge, supported on timber piles. Head walls at the abutments consisted of wooden lagging supported by steel piles. Installation depths, for both the wooden foundation piles and steel abutment piles, as well as sizing details, was not known. Rip-rap appeared to have been placed on both sides of the crossing, both under the bridge and extending out from the bridge on both sides. The thickness, material type, total quantity, and total coverage area of the rip-rap could not be determined due to snow cover. Photos of the site at the time of the geotechnical investigation are provided in Appendix A.

Cook's Creek is oriented relatively perpendicular to the existing bridge structure. Drainage ditches providing drainage into Cook's Creek were present along both sides of the rail embankment. Generally, the site is surrounded by flat-lying farm fields, with the rail track elevated about 1 to 2 m above the surrounding fields and ditches, respectively. On the north side of the site, a siphon for the City of Winnipeg aqueduct is present.

At the time of the geotechnical investigation, the farm fields surrounding the site, as well as the rail embankment, were covered by snow. Access to the site was gained via the rail line, which had been closed to rail traffic by the City of Winnipeg at the request of Stantec in order to provide a safe work area for drilling.

2.2 Proposed Development

It is understood that the proposed development at Mile 22.15 consists of a full replacement of the existing wooden bridge. Exact details of the proposed bridge were not known, however AMEC understood that the new structure will be a single span structure of approximately equal size to the existing structure. Based on information provided by Stantec, AMEC understood that steel HP310x110 piles are the preferred foundation type. Foundation loads were not available at the time of this report.

3.0 GEOTECHNICAL INVESTIGATION PROGRAM

Prior to initiating drilling, AMEC notified public utility providers (i.e. Manitoba Hydro, MTS, City of Winnipeg, etc.) of the intent to drill in order to clear public utilities, and where required, met with said representatives on-site.

On 16 December 2013, AMEC supervised the drilling of two test holes (TH01 and TH02) at the approximate locations illustrated in Figure 1. The test holes were drilled using an Acker MP5 track mounted drill rig equipped with 125 mm diameter solid stem augers; operated by Maple Leaf Drilling Ltd. of Winnipeg, Manitoba.

During drilling, AMEC field personnel visually classified the soil stratigraphy within the test holes in accordance with the Modified Unified Soil Classification System (MUSCS); as well as noted any observed seepage and/or sloughing conditions. Grab samples were collected at selected depths and retained in sealed plastic bags for shipping, review, and select testing in AMEC's Winnipeg laboratory. Shelby tube samples were also collected at selected depths for possible laboratory testing. The in-situ relative consistency of cohesive overburden was evaluated within the test hole using pocket penetrometer readings. The recorded pocket penetrometer readings are shown on the test hole logs. The relative consistency of underlying till was evaluated using standard penetration tests (SPTs), where the number of blows to drive the SPT sampler 0.3 m into the soil was recorded. The recorded number of blows is shown on the test hole logs as the SPT (N) value.

Upon completion of drilling, the depth to slough and groundwater level within each test hole was obtained after an elapsed time of about 10 minutes. Subsequently, the test holes were backfilled to grade with bentonite and auger cuttings. Excess auger cuttings were left neatly on site. UTM coordinates of the test hole locations were obtained using a hand held Garmin GPS, and are summarized in Table 3-1.

Table 3-1: Testhole Coordinates (UTM)

Testhole ID	Northing	Easting	Local Elevation[1] (m)
TH01	5524052	662681	~ 99.7
TH02	5524050	662706	~ 99.7

1. Local elevation 100.0 m assigned to top of track.

Following completion of the field drilling program, a laboratory testing program was conducted on selected soil samples obtained from the test holes. The laboratory testing program consisted of moisture content determinations, three unconfined compressive strength tests, and one set of liquid limit and plastic limit determinations.

Detailed test hole logs summarizing the sampling, field testing, laboratory test results, and subsurface conditions encountered at the test hole locations are presented in Appendix B. Actual depths noted on the test hole logs may vary by ± 0.3 m from those recorded due to the method by which the soil cuttings are returned to the surface. Summaries of the terms and symbols used on the test hole logs and of the Modified Unified Soil Classification System are also presented in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 Stratigraphy

Consistent with the regional geology and anticipated conditions, the stratigraphy at the test hole location consisted of the following, in descending order from grade level:

- Sand Fill or Clay Fill
- Clay
- Silt (Till)

A brief description of each of the soil layers bulleted above is presented below: For detailed descriptions, the test hole logs in Appendix B should be consulted.

Sand Fill and Clay Fill

Weathered clay, with trace organics, likely comprising fill, was encountered at the surface of TH01 and extended to about 1.2 m below grade. The clay was generally described as medium to high plastic with some silt, frozen, and black with some organics and trace rootlets. A single moisture content determination on a thawed sample indicated an in-situ moisture content of about 37 percent.

Sand fill was encountered at the surface of TH02 and extended to about 0.5 m below grade. The sand was generally described as gravelly, poorly graded, medium to coarse grained, frozen, and brown. A single moisture content determination on a thawed sample indicated an in-situ moisture content of about 13 percent.

Clay

Clay was encountered beneath the organic clay (TH01) and sand fill (TH02) and extended to about 7 m below surface. The clay was silty, high plastic, moist, stiff becoming firm below 3.0 m, and dark grey. In-situ moisture contents within the clay ranged from 35 percent to 51 percent. Unconfined compressive strength tests were completed on one Shelby tube sample collected from TH01 and two Shelby tube samples collected from TH02; the results of which are summarized in Table 4-1.

Table 4-1: Summary of Unconfined Compressive Strength Tests

Test Hole	Depth (m)	UCS (kPa)	ε_{100} (%)	ε_{50} (%)	Bulk Density (kg/m^3)	Moisture Content (%)
TH01	4.6 – 5.2	103	4.5	0.9	1744	51.4
TH02	3.0 – 3.6	106	6.6	1.2	1830	42.4
TH02	6.1 – 6.7	95	5.1	1.2	1900	37.0

ε100 equals strain at maximum unconfined compressive stress.
ε50 equals strain at one half of maximum unconfined compressive stress

Silt(Till)

Glacial silt till was encountered beneath the clay in both test holes at about 7.0 m below grade, and extended to auger and practical refusal in TH01 at about 8.7 m below grade, and to auger refusal in TH02 at about 16.2 m below grade. The till comprised a low plastic silt matrix containing some sand, trace to some gravel, trace clay, and was wet becoming moist below 9 m. In-situ moisture contents within the till ranged from about 20 percent at the clay/till interface to about 8 to 10 percent below 9 m at TH02. Atterberg Limits testing on a sample of the silt collected from TH02 at about 12.2 m below grade indicated a liquid limit of about 16 percent, and a plastic limit of about 9 percent.

SPT ‘N’ values ranged from 16 and 9 near the top of the till (i.e. 7.6 m below grade) in TH01 and TH02 respectively; to in excess of 50 blows per foot at all other depths, suggesting compact to very dense conditions.

4.2 Groundwater and Sloughing Conditions

Seepage and sloughing conditions were noted during drilling, and the depths to the accumulated water levels within the test holes were measured about ten minutes after drilling.

Sloughing of the silt till was noted during solid stem auger drilling at TH01 below 7.0 m. Sloughing during drilling did not occur at TH02.

Slight seepage within the silt till at TH01 and moderate seepage within the silt till at TH02 were observed during drilling. Slight seepage within TH01 was observed between grade and 3.7 m below grade, as well as between 4.6 m and 8.1 m below grade.

Upon completion of drilling and removal of the augers, TH01 and TH02 remained open to 8.5 m below grade and 11.6 m below grade, respectively. The depth to accumulated water was measured in TH01 at 7.9 m below grade, and within TH02 at about 4.0 m below grade.

Seepage water within both boreholes is considered likely as originating as groundwater originating from the till. For design purposes per the recommendations outlined in this report, a groundwater table of 4 m below top of track (i.e. local elevation 96 m) is recommended. 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.

5.0 GEOTECHNICAL RECOMMENDATIONS

5.1 General Evaluation

The stratigraphy and soil conditions encountered within the test holes advanced at the site is considered typical of conditions within the region and are considered favourable for the proposed development.

From a foundations perspective, soil conditions are considered suitable for the use of the driven steel H-piles indicated as preferred by Stantec. Driven steel pipe piles are also regarded as a suitable pile foundation alternative. Other suitable pile alternatives included bored concrete piles bearing within the underlying silt till. However, given Stantec's indicated preference for driven Steel H-Piles and the fact that the existing structure is supported on a driven pile type, foundation recommendations here-in are limited to driven steel HP and pipe piles. Recommendations for other foundation alternatives can be provided upon request.

The following sections provide discussion and recommendations as they pertain to: driven steel piles; lateral earth pressures on below grade walls; frost design considerations; and foundation concrete.

5.2 Driven Steel Pile Foundations

As previously discussed, soil conditions at the site are considered suitable for the use of the driven steel H-piles indicated as preferred by Stantec, as well as pipe piles. Notwithstanding, the following conditions should be considered in final selection and design of piles:

- The underlying silt till at the site is very dense below 9 m and depending on selection of the pile type (i.e. H-Pile, open-ended, or closed-ended pipe), end bearing development could vary with pile type and location. H-piles are anticipated to penetrate deeper than open ended or closed ended pipe piles.
- High end-bearing development within the silt till could inhibit pile penetration below local elevation 91 m (i.e. beyond 9 m below test hole elevation) and the achievable embedment depth for tensile (uplift) resistance to transient uplift loads and frost. In this regard, pile type selection and sizing as well as selection of the piling hammer and appurtenances must consider both the compressive and tensile requirements of

the pile, and the ability to both achieve the required compressive capacity and achieve the minimum embedment depth required for uplift resistance.

AMEC understands that the foundation will be designed in accordance with the 2013 AREMA Manual for Railway Engineering. AMEC's interpretation of recommended practices outline in the manual is that foundation design employs allowable stress design (ASD) principles as opposed to Limit State or Load-Factor Resistance Design (LFRD) design principles. In this regard, parameters here-in have been presented for use in ASD. If parameters for alternative design principles (i.e. Limit States) are required, this office should be contacted for revisions.

5.2.1 Axial Compressive Resistance of Single Driven Steel Piles

The 'allowable' compressive resistance of a driven steel pile (H or pipe) as a function of embedment depth may be determined using the 'allowable' unit shaft friction and unit end bearing pressures recommended in Table 5-1.

Table 5-1: 'Allowable' Unit Shaft Friction & End Bearing Values for Driven Steel Piles

Elevation Range[1] (m)	Assumed Average Soil Type	Shaft Friction (kPa)	End Bearing (kPa)
99.7 to X[2]	Fill and Clay	0	--*
X to 93.0	Clay	16	--*
93.0 to 91.0	Silt Till (Compact)	18	350
91.0 to 84.0 m	Silt Till (Very Dense)	48	1,600

[1] The elevations presented assume top of track to be approximate local elevation 100.0 m.
[2]X = the elevation of the frost penetration front at the pile interface, determined in accordance with the recommended frost penetration depth presented in Section 5.4, to account for possible movement of the soil away from the perimeter of the pile.

The above 'allowable' unit shaft friction and 'allowable' unit end bearing values include a factor of safety of 2.5.

For all pipe pile types and sizes, shaft friction should only be applied to the exterior surface area of the pile. In the case of steel H piles, shaft friction may be applied to the exterior sides of the two flanges plus twice the depth of the web (i.e. the box perimeter). For pipe piles with a closed-end configuration, end bearing may be applied to the full cross-sectional area of the toe of the pile. For H-piles and open end pile configurations, the area over which end bearing may be applied varies with the pile diameter. For small diameter pipes piles (i.e. DN300 or smaller) and H-Piles, there is considered a higher potential for 'plugging' of the pile during installation, and as such, it is considered acceptable to apply the end bearing to the full cross-sectional area of the toe of the pile which may be taken as the area enclosed by the outer circumference of a pipe section, or the cross sectional area of a rectangle bounded by the flanges in the case of steel H sections. For larger pile sizes, 'plugging' of the pile during driving may be variable, and the end bearing values provided above should be re-evaluated by AMEC for large diameter pipe piles. However, for current design purposes, the unit end bearing values outlined above may be applied to the steel area of the toe of pipe piles larger than DN300. If during construction driving resistance is lower or higher than anticipated, 'soil plug' development and end bearing development may be quantified via dynamic pile testing by pile driving analyzer (PDA Testing) and CAPWAP[1] analysis.

Due to limitations on the driveability of the pile imposed by the yield strength of the pile, as a guide to initial design and selection of pile wall thickness and steel grade, it is recommended that the maximum design ‘allowable’ compressive resistance of a steel pile be limited to 0.25FyAs (i.e. a fraction of the unfactored structural yield capacity of the pile), where: Fy is the nominal yield stress of the steel, and As is the cross-sectional area of steel in the pile. The purpose of this restriction is to mitigate the risk of statically designing a pile that cannot be driven with enough energy or force to overcome dynamic soil resistance and subsequently develop the design static load resistance without yielding or damaging the pile. Subject to driveability analysis and evaluation of driving stresses at the pile design stage, the maximum ’allowable’ compressive stress could be increased to as much as 0.35FyAs.

Additional comments for design and construction of driven steel piles are as follows:

- Static pile design parameters pertain to soil resistance only. The pile cross sections must be designed to withstand the design loads and the driving forces during installation.
- Although not commonly employed for the installation of driven piles, if a pre-bore was required (i.e. for ground disturbance clearance or contractor preference), shaft friction must be neglected over the depth of the pre-bore.
- Piles must be spaced a minimum of three pile diameters apart, as measured from centre-to-centre, in order to act individually as single piles in vertical compression when used in a small pile group of three piles or less. Where pile groups larger than 3 piles are required, the pile group should be reviewed by AMEC.
- Once the pile configuration is known, AMEC recommends that a driveability analysis (i.e. WEAP) be completed prior to proceeding to construction, and concurrent with selection of the pile driving equipment, to confirm the ability of the hammer and appurtenances to drive the piles to the design capacity and embedment depth without damage. Similarly, the driveability analysis can be extended to develop termination criteria for use in pile installation monitoring. It should be noted that driveability analyses should be completed using ultimate soil parameters.
- All piles driven within five pile diameters should be monitored for heave and, where heave is observed, piles should be re-driven. Piles that are re-driven should be advanced to at least the original elevation.
- All piles should be driven continuously to practical refusal once driving is initiated.
- Any piles that are have been damaged, are excessively out of plumb, or have refused prematurely may need to be replaced, pending a review by a qualified geotechnical engineer of their load carrying capability and estimated settlement.
- All pile caps and grade beams should be underlain by a minimum 150 mm thick void form to accommodate the expansive nature and potential frost heave of the underlying soil.

[1] PDA : Pile Driving Analyzer, CAPWAP: software to analyze PDA Test data

- Prior to the pile installation, the piles should be inspected to confirm that the material specifications are satisfied. As a minimum, steel H-piles should meet the requirements of CAN/CSA-G40.20/G40.21, Grade 350 W, and pipe piles should have a minimum yield strength of 310 MPa (i.e. ASTM A252 Grade 3 steel). The piles should be free from protrusions, which could create voids in the soil around the pile during driving.
- Monitoring of the pile installation by an experienced inspector is recommended to verify that the piles are installed in accordance with design assumptions and the driving criteria are satisfied. For each pile, a complete driving record in terms of the number of blows per 300 mm of penetration should be recorded by the inspector and reviewed during pile installation by the designer.

5.2.2 Tensile (Uplift) Resistance (Single Pile)

In the case of driven steel piles, the uplift resistance of a single pile will be provided by the sustained downward load on the pile (if applicable) and shaft friction along the length of pile embedded below the depth of frost penetration. In the case of straight shaft (i.e. driven steel) piles, the soil component of the ‘allowable’ uplift resistance to *sustained* tensile loads will be provided by shaft friction and can be determined using 70% of the shaft friction values outlined in Table 5-1. For pipe piles, only the exterior surface area of the pile in contact with the soil should be used in the calculation of the frictional resistance. In the case of steel H piles, the surface area should include the exterior sides of the two flanges plus twice the depth of the web. For frost and *transient* uplift loads, such as those due to wind gusts, no reduction of the shaft friction values in Table 5-1 is required. Transient loads would not be additive to the uplift forces due to frost action.

Although not commonly employed for the installation of driven steel piles, if a pre-bore was required (i.e. for ground disturbance clearance or contractor preference), shaft friction must be neglected over the depth of the pre-bore.

5.2.3 Lateral Resistance (Single Pile)

Piles resist laterally applied loads by deflecting until the necessary resistance is mobilized in the adjacent soils. The majority of lateral load resistance for flexible piles is generally provided within the upper 4 to 5 m of the soil profile (i.e. the typical point of inflection for the pile). The maximum bending moment typically occurs at 1.5 m to 3.0 m below grade depending on the applied loading and soil resistance. The allowable lateral capacity depends upon the properties of the soil and pile material, pile sizes, fixity of the top of the pile, depth of embedment, height of load application above ground, vertical load applied and tolerable deflections.

Where the lateral load capacities or magnitude of movements of piles are critical, it is recommended that the lateral deflections and design capacities of piles or groups of piles be evaluated using Reese’s method of p-y curves. This method models the strength-deformation characteristics using load-displacement curves for the various soil strata, and the non-linear behaviour of the soil. With the method of p-y curves, solutions may be obtained through an iterative procedure performed using LPILE Software for single piles, and extended to pile groups by using GROUP Software to analyze the behaviour of piles in a group subjected to both axial and lateral loadings. The analytical procedure provides lateral pile deflections, generated bending moments, shear forces, and the soil reaction computed at close intervals over the depth of the pile.

Based on conditions observed within the appended test hole logs, the stratigraphy and soil parameters outlined in Table 5-2 are considered suitably representative of the average subsurface conditions expected to influence the lateral behaviour of driven steel piles at the Site.

Table 5-2: LPile Input Parameters

Elevation Range (m)[1]	Soil Type / Model	Effective Unit Weight (kN/m^3)[2]	Friction Angle (•)	Cohesion (kPa)	E50 (%)	p-y subgrade modulus, k (kPa/m)
100.0 to 96.0	Clay	19	n/a	50	0.012	Default
96.0 to 93.0	Clay	9	n/a	50	0.012	Default
93.0 to 84.0	Silt Till	10	35	0	n/a	Default

[1] The elevations presented assume top of track to be approximate local elevation 100.0 m.
[2] Groundwater level of 4.0 m below top of track was assumed.

The use of zero lateral resistance for cohesive soils (i.e. clay soils) located within 1 m of final surface is recommended for the serviceability condition where there exists the potential for formation of a permanent gap between the pile and the soil due to installation, desiccation, or frost effects.

Lateral pile analysis of a prescribed pile configuration was not part AMEC's scope of work for this investigation. Notwithstanding, lateral pile analysis could be conducted by AMEC for specified pile configurations on request.

5.2.4 Single Pile Settlement

The settlement of a single pile depends on the applied load, strength-deformation properties of the foundation soils, load transfer mechanism, load distribution over the pile embedment depth, and the relative proportions of the load carried by shaft friction and end-bearing. Assuming good workmanship, inclusive of good excavation, the predicted settlement of piles at working loads equal to a maximum given by the 'allowable' pile capacity are 0.5 to 1 percent of the shaft diameter plus the elastic shortening of the pile due to the compressive load acting on the pile.

5.2.5 Pile Group Effects

Generally, piles will behave individually in compression (i.e. group efficiency equals 1.0) when a minimum centre-to-centre spacing of 5 pile diameters is provided between adjacent piles, and will behave individually laterally when the center-to-center spacing is greater than 3 diameters in the direction transverse to loading (side-by-side), and greater than 8 diameters in the direction parallel to loading (in-line). However, for circumstances in which piles are closely spaced and/or the piles are connected by a rigid pile cap forcing equal settlement behaviour at the pile heads, interaction between the piles will occur and should be considered in design.

Notwithstanding the above, AMEC does not anticipate that large groups of four or more closely spaced piles will be required. Consequently, recommendations pertaining to the axial and lateral load resistances of pile groups are not provided here-in. If pile groups are required by design, AMEC should be notified and a review of possible group interactions effects evaluated.

5.3 Lateral Earth Pressures on Below Grade Walls (i.e. Wing Walls)

5.3.1 Soil Design Parameters

Below grade walls (i.e. wing walls) will be required to resist lateral pressures from the surrounding soil, water, and any additional surcharge loading (i.e. fill, live surface loads, etc.). Table provides recommended design values for the bulk unit weight, angle of internal friction, and ‘at rest’, active, and lateral earth pressure coefficients for moderately to well compacted native clay, compacted sand fill, and compacted gravel fill soils.

Table 5-3: Earth Pressure Coefficients and Soil Unit Weights

Soil Type		Active Pressure Coefficient K_a	“At Rest” Earth Pressure Coefficient K_o	Passive Pressure Coefficient[a] K_p	Total Soil Unit Weight (kN/m^3)	Friction Angle (deg) Between Soil and Concrete
Gravel Fill	Well Compacted	0.25	0.40	2.67	23	25
	Moderately Compacted	0.30	0.47	2.17	22	21
Sand Fill	Well Compacted	0.30	0.47	2.17	21	21
	Moderately Compacted	0.36	0.53	1.85	20	18
Common Clay Fill and Clay	In-situ and Well Compacted	0.53	0.70	1.26	19	12
	Moderately Compacted	0.59	0.75	1.13	18	10

The passive earth pressure coefficients provided in Table 5-3 include a reduction factor of 1.5 to account for the partial mobilization of passive resistance that is consistent with the small wall displacements expected under operational conditions. Relatively large wall displacements would be necessary to realize full passive resistances.

With respect to subsurface drainage and groundwater conditions over the depth of the foundation structure, the phreatic surface at the site should be taken as 4 m below top of track. The use of free draining backfill and the provision of drainage behind vertical subsurface walls is strongly recommended, and will further serve to mitigate frost action on vertical walls extending through the zone of frost penetration.

The magnitude and distribution of the lateral earth pressures on below grade structures will depend on such factors as the rigidity of the below grade structure; the degree of compaction of the backfill against the structure; the backfill soil type; the slope angle at the structure/soil interface; and the subsurface drainage and groundwater conditions over the height of the structure. It is anticipated that a sloped excavation will be implemented for construction of below grade foundation structures, which will necessitate the placement of backfill behind below grade structure walls. The magnitude and distribution of the lateral earth pressures (P) on below grade structures will depend on the degree of compaction of the backfill. In addition to earth pressures, lateral stresses generated by any applicable surcharge loads also need to be evaluated in the design. Recommended earth pressure distributions for moderate to well compacted backfill cases, as well as for line or point surcharge loads, are discussed in Section 5.3.2.

5.3.2 Calculation of Earth Pressure Distributions and Load Factors

5.3.2.1. Moderate to Well Compacted Backfill Case

Where subgrade support on the surface of the retained soil behind a wall is required, as it is for headwalls, the backfill against the wall will need to be compacted to at least 95 percent Standard Proctor maximum dry density. The use of free draining backfill behind below grade structures is strongly recommended in order to maintain drained conditions behind the structure. Assuming drained conditions, the design earth pressure distribution should adopt a combined trapezoidal/triangular distribution as shown on Figure 2 to account for the induced lateral pressures due to compaction. Figure 2 also provides the relationships to be used in the calculation of the compaction induced earth pressures, and tabulated loads (P) generated by typical compactors. The earth pressure coefficients to be used in the calculation of the lateral pressures should be those applicable to the backfill types given in Table 5-3.

If sub-drainage is not provided and it is possible by design for a perched groundwater to develop within the retained soil (i.e. “bathtub” effect associated with gravel fill soils surrounded by low permeable fine grained soil types), the hydrostatic component should be included in addition to the earth pressure given in Figure 2.

5.3.2.2. Surcharge Loads

In addition to earth pressures, lateral stresses generated by surcharge loads, such as point loads from locomotives, also need to be evaluated in the design. For line or point surcharge loads, the lateral pressures should be determined using the relationships given in Figure 4. In the case of uniformly distributed surcharge loads, such as those acting on the surface of the retained soil, the induced lateral earth pressure may be determined by multiplying the surcharge load by the appropriate earth pressure coefficient.

5.4 Frost Design Considerations

5.4.1 Frost Penetration Depth

The upper stratigraphy at the test hole locations, and across the site, is considered moderately frost susceptible in the presence of water, and as such, frost effects should be considered for foundations or surface structures sensitive to movement. Based on historical temperature data for the area, a design frost penetration of 2.4 m below final grade is recommended in unheated areas that will not have regular snow or vegetative ground cover. It should be noted that this recommended frost penetration depth extends both vertically and laterally behind final surface (i.e. extends 2.4 m behind the headwall).

5.4.2 Pile Foundations

Frost forces applied to pile foundations include adfreeze pressures acting along the pile shafts within the depth of frost penetration. If pile caps are used and extend beyond the perimeter of the underlying pile, then frost heave forces acting on the undersides of the pile caps, as well as any connecting supports (i.e. lateral tie between the piles) will also need to be considered.

5.4.2.1. Frost Heave

To reduce the potential of frost heave pressures, a void-forming product should be installed beneath the underside of the pile caps and any other structural element located within the depth of frost penetration. The recommended minimum thickness of the void should be 150 mm. Alternatively, a compressible material may be used in lieu of a void forming material, and the uplift pressures may be taken as the crushing strength of the compressible medium. It is recommended that a frost heave of 150 mm be assumed in determining the required thickness for the void-filler and the associated uplift pressures associated with the thickness used.

The finished grade adjacent to each pile cap should be capped with well compacted clay and sloped away so that the surface runoff is not allowed to infiltrate and collect in the void space or in the compressible medium.

The use of void-forming product below the groundwater is unfeasible. In instances where groundwater is located within the recommended depth of frost penetration, the underside of foundation elements such as pile caps should extend below the depth of frost penetration to mitigate frost heave development on the underside of the foundation element.

5.4.2.2. Adfreeze Stresses

Resistance to adfreeze and frost heave forces will be provided by the sustained vertical loads on the foundation, the buoyant weight of the foundation and dead weight of the structure, and the soil uplift resistance component provided by the length of the pile extending below the depth of frost penetration. In the case of straight shaft (i.e. driven steel) piles, the adfreeze force acting on the pile may be determined assuming an unfactored unit adfreeze stress of 65 kPa applied to the exterior surface of the pile and supported foundation elements (i.e. pile caps) located within the zone of frost penetration. The uplift resistance of the pile below the depth of frost may be determined using the Tensile (Uplift) Resistance recommendations presented in Section 5.2.2.

5.5 Foundation Concrete

Where concrete elements outlined in this report and all other concrete in contact with the local soil will be subjected in service to weathering, sulphate attack, a corrosive environment, or saturated conditions, the concrete should be designed, specified, and constructed in accordance with concrete exposure classifications outlined in the latest edition of CSA standard A23.1, Concrete Materials and Methods of Concrete Construction. In addition, all concrete must be supplied in accordance with current Manitoba and National Building Code requirements.

Based on significant data gathered through previous work in Southern Manitoba, water soluble sulphate concentrations in the soil are typically in the range of 0.2% to 2.0%. As such, the degree of sulphate exposure at the site may be considered as ‘severe’ in accordance with current CSA standards, and the use of sulphate resistance cement (Type HS or HSb) is recommended for concrete in contact with the local soil. Furthermore, air entrainment should be incorporated into any concrete elements that are exposed to freeze-thaw to enhance its durability.

It should be recognized that there may be structural and other considerations, which may necessitate additional requirements for subsurface concrete mix design.

5.6 Construction Monitoring and Testing

All engineering design recommendations presented in this report are based on the assumption that an adequate level of testing and monitoring will be provided during construction and that all construction will be carried out by a suitably qualified contractor experienced in foundation and earthworks construction. An adequate level of testing and monitoring is considered to be:

- for earthworks: full-time monitoring and compaction testing.
- for deep foundations: design review and full time monitoring during construction.
- for concrete construction: testing of plastic and hardened concrete in accordance with the latest editions of CSA A23.1 and A23.2; and review of concrete supplier’s mix designs for conformance with prescribed and/or performance concrete specifications.

AMEC requests the opportunity to review the design drawings, and the installation of the foundations, to confirm that the geotechnical recommendations have been correctly interpreted. AMEC would be pleased to provide any further information that may be needed during design and to advise on the geotechnical aspects of specifications for inclusion in contract documents.

6.0 CLOSURE

The findings and recommendations presented in this report were based on geotechnical evaluation of the subsurface conditions observed during the site investigation described in this report. If conditions other than those reported in this report are noted during subsequent phases of the project, or if the assumptions stated herein are not in keeping with the design, this office should be notified immediately in order that the recommendations can be verified and revised as required. Recommendations presented herein may not be valid if an adequate level of inspection is not provided during construction, or if relevant building code requirements are not met.

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 in 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 has been prepared for the exclusive use of Stantec Consulting Ltd., and their agents, for specific application to the project described in this report. The data and recommendations provided herein should not be used for any other purpose, or by any other parties, without review and written advice from AMEC. Any use that a third party makes of this report, or any reliance or decisions made based on this report, are the responsibility of those parties. AMEC accepts no responsibility for damages suffered by a third party as a result of decisions made or actions based on this report.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, either expressed or implied, is made.

Respectfully submitted,
AMEC Environment & Infrastructure,
A Division of AMEC Americas Limited

DRAFT FOR CLIENT REVIEW

Reviewed by:

Kelly Johnson, P. Eng.
Senior Geotechnical Engineer

Harley Pankratz, P.Eng.
Vice President, Eastern Prairies/Northern Alberta

FIGURES

CLIENT LOGO

CLIENT

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amec

DWN BY: MD

CHK'D BY: TG

DATUM: NAD83

PROJECTION: UTM Zone 14 U

SCALE: AS SHOWN

GEOTECHNICAL INVESTIGATION GREATER WINNIPEG WATER DISTRICT RAILWAY BRIDGE MILE 22.15

TEST HOLE LOCATION PLAN

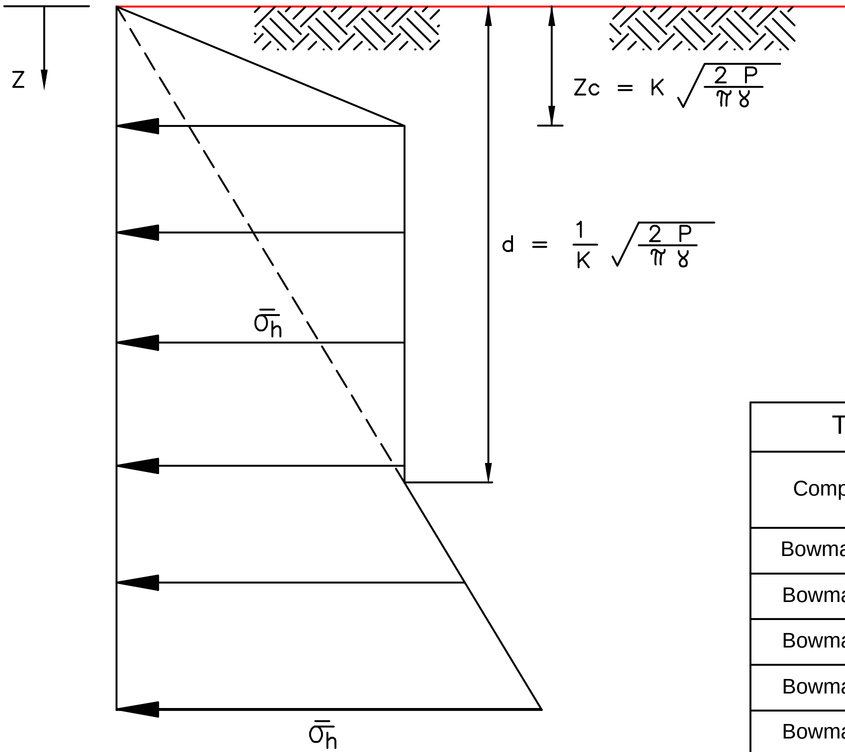
REV. NO.: A

DATE: FEBRUARY 2014

PROJECT NO: WX17312

FIGURE No. FIGURE 1

P:\JOBS\17300'S\17310'S\17312 STANTEC - GWWD BRIDGES\DRAWINGS\WX17312.DWG



EARTH PRESSURE DISTRIBUTION

FOR $Zc \leq Z \leq d$

$$\bar{\sigma}_h = \sqrt{\frac{2P\gamma}{\pi}}$$

FOR $Z > d$

$$\bar{\sigma}_h = K \cdot \gamma \cdot Z$$

TYPICAL COMPACTOR LOADS (P)			
Compactor	LOAD (P) kN/m	Compactor	LOAD (P) kN/m
Bowmag TSE	31	Bowmag BW122PD	36
Bowmag 60S	32	Bowmag 142PDB	47
Bowmag 65S	22	Bowmag 172PDB	93
Bowmag 75S	33	Dynapac LR100	42
Bowmag 90S	39	Dynapac 2100V	93
Bowmag 75AD	20	Dynapac CA121D	53
Bowmag 100AD	20	Dynapac CA121PD	54
Bowmag 120AD	34	Dynapac CA151	80
Bowmag 130AD	36	Dynapac CA151D	80
Bowmag BW122D	30	Dynapac CA151PD	96

$$P \text{ (ROLLER LOAD)} = \frac{\text{DEAD WT. OF ROLLER + CENTRIFUGAL FORCE}}{\text{WIDTH OF ROLLER}}$$

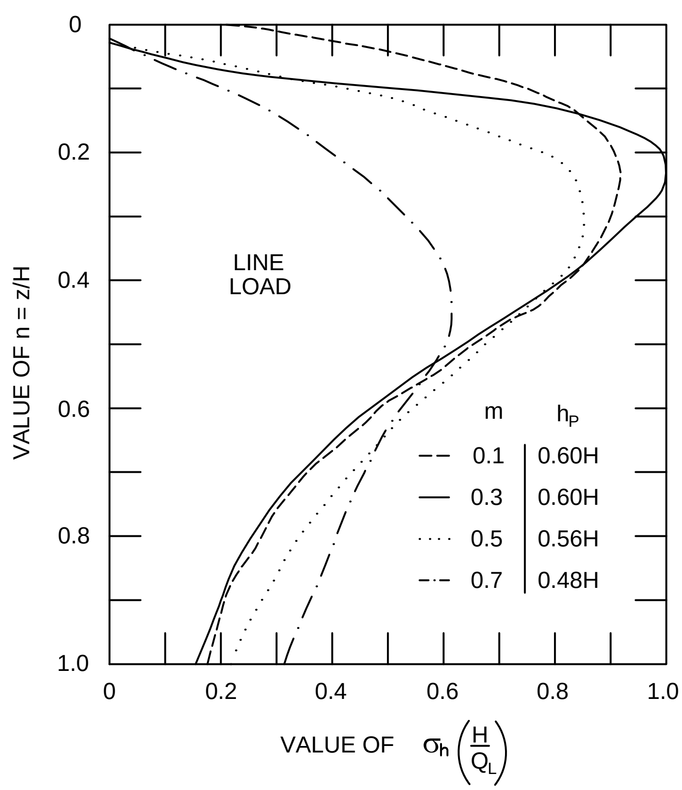
TYPICAL VALUES GIVEN IN TABLE

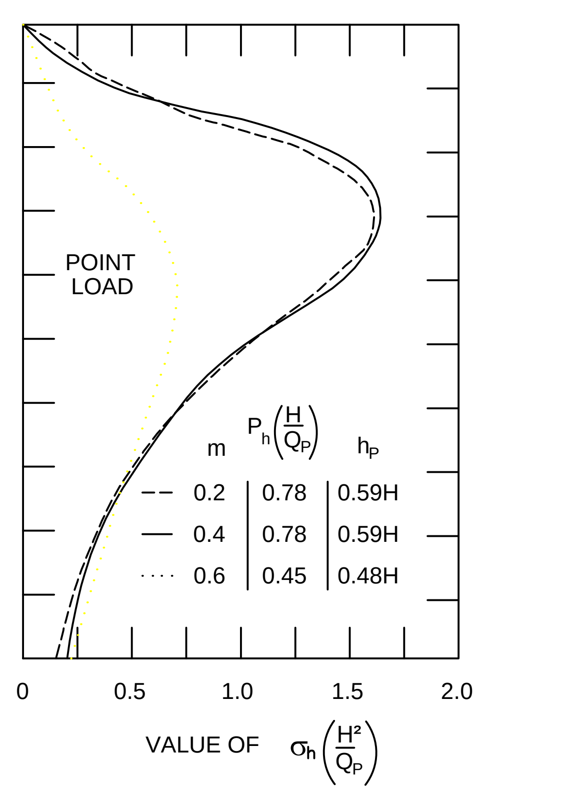
EARTH PRESSURE COEFFICIENTS

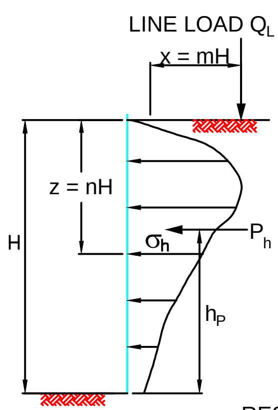
$K = K_o$ ("AT REST") OR K_a (ACTIVE CASE)
(SEE TEXT OF REPORT)

γ = SOIL UNIT WEIGHT
(SEE TEXT OF REPORT)

CLIENT: STANTEC CONSULTING LTD.	DWN BY: MD	GEOTECHNICAL INVESTIGATION GREATER WINNIPEG WATER DISTRICT RAILWAY BRIDGE MILE 22.15	DATE: FEBRUARY 2014
	CHK'D BY: KJ		PROJECT No.: WX17312
AMEC Earth & Environmental 5681-70 STREET, EDMONTON, ALBERTA, T6B 3P6 PHONE 780-436-2152, FAX 780-435-8425	DATUM: -		REV. No.: -
	PROJECTION: -	LATERAL EARTH PRESSURES INDUCED BY COMPACTION	FIGURE No.: FIGURE 2
	SCALE: NOT TO SCALE		







FOR m ≤ 0.4:

$$\sigma_h\left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

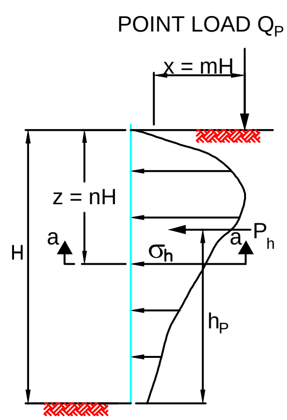
$$P_h = 0.55\ Q_L$$

FOR m > 0.4:

$$\sigma_h\left(\frac{H}{Q_L}\right) = \frac{1.28m^2n}{(m^2 + n^2)^2}$$

$$\text{RESULTANT}\ P_h = \frac{0.64\ Q_L}{(m^2 + 1)}$$

PRESSURES FROM LINE LOAD
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



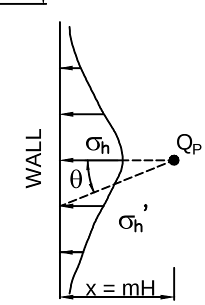
FOR m ≤ 0.4:

$$\sigma_h\left(\frac{H^2}{Q_P}\right) = \frac{0.28n^2}{(0.16 + n^2)^3}$$

FOR m > 0.4:

$$\sigma_h\left(\frac{H^2}{Q_P}\right) = \frac{1.77m^2n^2}{(m^2 + n^2)^3}$$

$$\sigma_h' = \sigma_h\ \cos^2(1.1\theta)$$



SECTION a-a
PRESSURES FROM POINT LOAD
(BOUSSINESQ EQUATION
MODIFIED BY EXPERIMENT)

CLIENT: STANTEC CONSULTING LTD.	DWN BY: MD	GEOTECHNICAL INVESTIGATION GREATER WINNIPEG WATER DISTRICT RAILWAY BRIDGE MILE 22.15	DATE: FEBRUARY 2014
	CHK'D BY: KJ		PROJECT No.: WX17312
amec AMEC Earth & Environmental 5681-70 STREET, EDMONTON, ALBERTA, T6B 3P6 PHONE 780-436-2152, FAX 780-435-8425	DATUM: -		REV. No.: -
	PROJECTION: -	LATERAL PRESSURES DUE TO SURCHARGE POINT AND LINE LOADS	FIGURE No.: FIGURE 3
	SCALE: NOT TO SCALE		

P:\Jobs\17300's\17310's\17312 Stantec - GWWD Bridges\Drawings\LAT PRESSURE - FIG 3.dwg - F3 - Jan, 31, 2014 10:40 AM - matthews.dolores

APPENDIX A

Photo 1: Existing Bridge Foundations, Looking Southeast.

Photo 2: Existing Bridge Foundations, Looking North of east.

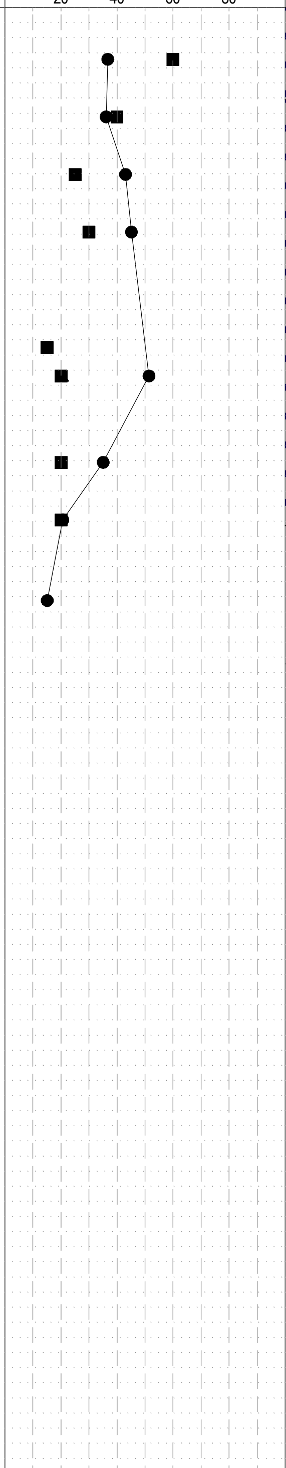
APPENDIX B

PROJECT: GWWD Bridges	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH01**
CLIENT: Stantec	DRILL TYPE: Acker MP5 Track Rig	PROJECT NO: WX17312
LOCATION: Anola, Manitoba	DRILL METHOD: 125 mm SSA	ELEVATION: 99.7 m

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

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

▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400
■ POCKET PENETROMETER (kPa) ■ 100 200 300 400
PLASTIC M.C. LIQUID 20 40 60 80



Depth (m)	SOIL SYMBOL / MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS
0 – 1.2	CH	CLAY - some silt, medium to high plastic, frozen, very stiff, black, some organics, trace rootlets	Grab	1		TH Coordinates (UTM): 5524052mN 662681mE
1.2 – 6.9	CH	CLAY - silty, high plastic, moist, stiff to very stiff, grey - stiff, mottled brown, occasional sulphate inclusions below 1.5 m - firm below 3.0 m - firm to stiff, occasional to frequent silt till inclusions below 4.6 m - frequent silt till inclusions below 6.1 m	Grab, Grab, Grab, Grab, Grab, Shelby Tube, Grab, Grab	2, 3, 4, 5, 6, 7, 8		Unconfined Compressive Strength (S6): Max Stress: 103 kPa Strain @ Failure: 4.5% Bulk Density: 1744 kg/m^3 Dry Density: 1151 kg/m^3
6.9 – 8.7	ML	SILT (TILL) - trace clay, some sand, trace gravel, low plastic, moist, compact, tan - some gravel, no clay, wet below 7.6 m - moist below 8.1 m	Grab, SPT, Grab, SPT	9, 10, 11, 12	16, 50/50	

AUGER AND PRACTICAL SPT REFUSAL AT 8.7 m BELOW GRADE ON A SUSPECTED BOULDER

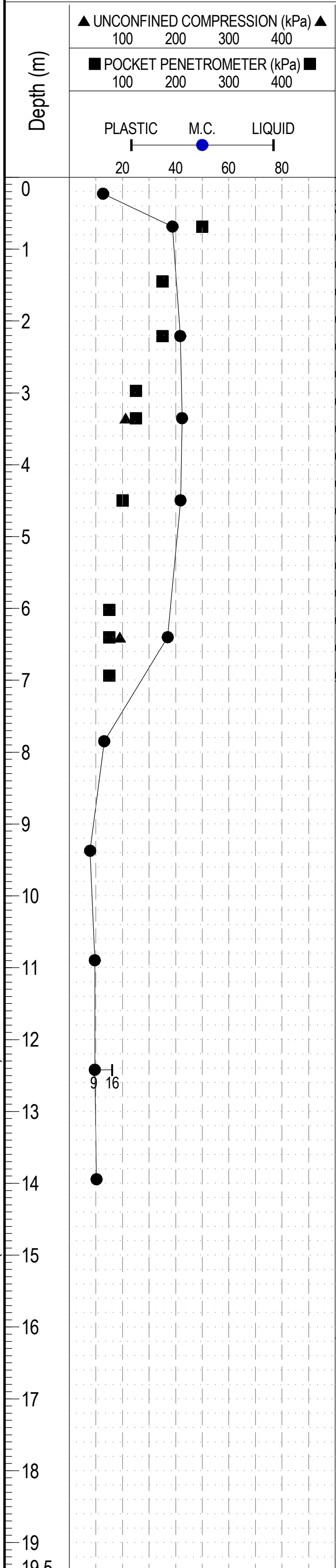
NOTES:
- Slight seepage was observed below 6.9 m. No sloughing was observed during drilling
- Test hole remained open to 8.5 m below grade with water measured at 7.9 m 10 min after completion
- Test hole was backfilled with auger cuttings and bentonite

Depth (m): 0 – 19.5; ELEVATION (m): 99 – 81

amec	AMEC Environment & Infrastructure Winnipeg, Manitoba	LOGGED BY: HWP	COMPLETION DEPTH: 8.7 m
		REVIEWED BY: TG	COMPLETION DATE: 16 December 2013
		Figure No. B1	Page 1 of 1

PROJECT: GWWD Bridges	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH02**
CLIENT: Stantec	DRILL TYPE: Acker MP5 Track Rig	PROJECT NO: WX17312
LOCATION: Anola, Manitoba	DRILL METHOD: 125 mm SSA	ELEVATION: 99.7 m

SAMPLE TYPE	Shelby Tube	No Recovery	SPT (N)	Grab Sample	Split-Pen	Core
BACKFILL TYPE	Bentonite	Pea Gravel	Drill Cuttings	Grout	Slough	Sand



SOIL SYMBOL	MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS	ELEVATION (m)
	SP	SAND (FILL) - gravelly, poorly graded, medium to coarse grained, compact (inferred), frozen, brown	Grab Sample	1		TH Coordinates (UTM): 5524050mN 662706mE	99
	CH	CLAY - silty, trace organics, high plastic, moist, stiff, grey to black	Grab Sample	2			
		- occasional grey silt lenses below 1.4 m - mottled brown, occasional sulphate inclusions below 1.5 m	Grab Sample	3			98
			Grab Sample	4			97
			Grab Sample	5			
		- grey, occasional oxidation below 3.7 m	Shelby Tube	6		Unconfined Compressive Strength (S6): Max Stress: 106 kPa Strain @ Failure: 6.6% Bulk Density: 1830 kg/m^3 Dry Density: 1284 kg/m^3	96
		- trace gravel, occasional silt till inclusions below 4.6 m	Grab Sample	7			95
		- frequent silt till inclusions below 5.3 m					94
		- firm below 5.9 m	Grab Sample	8			
			Shelby Tube	9		Unconfined Compressive Strength (S6): Max Stress: 95 kPa Strain @ Failure: 5.1% Bulk Density: 1900 kg/m^3 Dry Density: 1387 kg/m^3	93
	ML	SILT (TILL) - trace clay, some sand, some gravel, low plastic, moist, loose, tan	Grab Sample	10			
			Grab Sample	11			92
			SPT (N)	12	9		91
		- very dense below 9.1 m	Grab Sample	13			
			SPT (N)	14	57		90
			Grab Sample	15			89
			SPT (N)	16	74/17		88
			Grab Sample	17			
			SPT (N)	18	50/5.5		87
			Grab Sample	19			
			SPT (N)	20	50/4		86
							85
							84
		AUGER REFUSAL AT 16.2 m BELOW GRADE IN SUSPECTED VERY DENSE TILL					83

NOTES:
- Moderate sloughing and seepage was observed below 7.0 m during drilling
- Test hole remained open to 11.6 m below grade with water measured at 4.0 m 10 min after completion
- Test hole backfilled with auger cutting and bentonite

ELEVATION (m): 82, 81

AMEC Environment & Infrastructure Winnipeg, Manitoba	LOGGED BY: HWP	COMPLETION DEPTH: 16.2 m
	REVIEWED BY: TG	COMPLETION DATE: 16 December 2013
	Figure No. B2	Page 1 of 1

EXPLANATION OF TERMS AND SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of field investigation and subsequent laboratory testing are described in these pages.

It should be noted that materials, boundaries and conditions have been established only at the borehole locations at the time of investigation and are not necessarily representative of subsurface conditions elsewhere across the site.

TEST DATA

Data obtained during the field investigation and from laboratory testing are shown at the appropriate depth interval.

Abbreviations, graphic symbols, and relevant test method designations are as follows:

*C	Consolidation test	*ST	Swelling test
D_R	Relative density	TV	Torvane shear strength
*k	Permeability coefficient	VS	Vane shear strength
*MA	Mechanical grain size analysis and hydrometer test	w	Natural Moisture Content (ASTM D2216)
		w_l	Liquid limit (ASTM D 423)
N	Standard Penetration Test (CSA A119.1-60)	w_p	Plastic Limit (ASTM D 424)
N_d	Dynamic cone penetration test	E_f	Unit strain at failure
NP	Non plastic soil	γ	Unit weight of soil or rock
pp	Pocket penetrometer strength	γ_d	Dry unit weight of soil or rock
*q	Triaxial compression test	ρ	Density of soil or rock
q_u	Unconfined compressive strength	ρ_d	Dry Density of soil or rock
*SB	Shearbox test	C_u	Undrained shear strength
SO_4	Concentration of water-soluble sulphate	→	Seepage
		▼	Observed water level

* The results of these tests are usually reported separately

Soils are classified and described according to their engineering properties and behaviour.

The soil of each stratum is described using the Unified Soil Classification System[1] modified slightly so that an inorganic clay of "medium plasticity" is recognized.

The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual[2].

Relative Density and Consistency:

Cohesionless Soils

Relative Density	SPT (N) Value
Very Loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Cohesive Soils

Consistency	Undrained Shear Strength c_u (kPa)	Approximate SPT (N) Value
Very Soft	0-12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Standard Penetration Resistance ("N" value)
The number of blows by a 63.6kg hammer dropped 760 mm to drive a 50 mm diameter open sampler attached to "A" drill rods for a distance of 300 mm after an initial penetration of 150 mm.

1 "Unified Soil Classification System", Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S. Army. Vol. 1 March 1953.

2 "Canadian Foundation Engineering Manual", 3rd Edition, Canadian Geotechnical Society, 1992.

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISIONS			SYMBOLS: USCS	SYMBOLS: GRAPH	SYMBOLS: COLOUR	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75um)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (TRACE OR NO FINES)	GW		RED	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = D_{60}/D_{10} > 4$; $C_c = (D_{30})^2/(D_{10} x D_{60})$ = 1 to 3
			GP		RED	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY GRAVELS (WITH SOME OR MORE FINES)	GM		YELLOW	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4
			GC		YELLOW	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (TRACE OR NO FINES)	SW		RED	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = D_{60}/D_{10} > 6$; $C_c = (D_{30})^2/(D_{10} x D_{60})$ = 1 to 3
			SP		RED	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (WITH SOME OR MORE FINES)	SM		YELLOW	SILTY SANDS, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4
			SC		YELLOW	CLAYEY SANDS, SAND-CLAY MIXTURES	ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75um)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML		GREEN	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)
		$W_L > 50\%$	MH		BLUE	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SAND OR SILTY SOILS	
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL		GREEN	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS	
		$30\% < W_L < 50\%$	CI		GREEN-BLUE	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	
		$W_L > 50\%$	CH		BLUE	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$W_L < 50\%$	OL		GREEN	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY
		$W_L > 50\%$	OH		BLUE	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGHLY ORGANIC SOILS			PT		ORANGE	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE

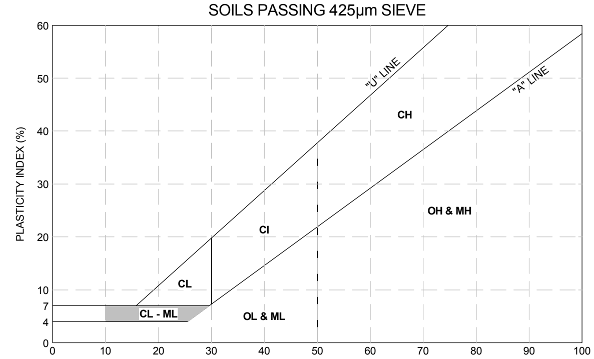
SPECIAL SYMBOLS

LIMESTONE		OILSAND	
SANDSTONE		SHALE	
SILTSTONE		FILL (UNDIFFERENTIATED)	

SOIL COMPONENTS

FRACTION	U.S. STANDARD METRIC SIEVE SIZE		DEFINING RANGES OF PERCENT BY WEIGHT OF MINOR COMPONENTS	
GRAVEL	PASSING	RETAINED	PERCENT	DESCRIPTOR
COARSE	76mm	19mm	35 - 50	AND
FINE	19mm	4.75mm		
SAND			30 - 35	Y / EY
COARSE	4.75mm	2.00mm		
MEDIUM	2.00mm	425µm	10 - 20	SOME
FINE	425µm	75µm		
FINES (SILT OR CLAY BASED ON PLASTICITY)	75µm		1 - 10	TRACE

PLASTICITY CHART FOR
SOILS PASSING 425µm SIEVE



NOTES:
1. ALL SIEVE SIZES MENTIONED ARE U.S. STANDARD ASTM E.11.
2. COARSE GRAINED SOILS WITH TRACE TO SOME FINES GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH TRACE TO SOME CLAY.
3. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

OVERSIZED MATERIAL

ROUNDED OR SUBROUNDED:

COBBLES 76mm to 200mm
BOULDERS > 200mm

NOT ROUNDED:

ROCK FRAGMENTS ? 76mm
ROCKS > 0.76 CUBIC METRE IN VOLUME

AMEC Environment & Infrastructure
a Division of AMEC Americas Limited

DRAFT REPORT
GEOTECHNICAL INVESTIGATION
GREATER WINNIPEG WATER DISTRICT
RAILWAY BRIDGE MILE 41.3
RM OF SPRINGFIELD, MANITOBA

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5 February 2014

WX17312

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1.0 INTRODUCTION

At the request and authorization of Mr. Mike Boissonneault, P.Eng., Project Manager, and Senior Associate for Stantec Consulting Ltd (Stantec), AMEC Environment & Infrastructure, a division of AMEC Americas Limited (AMEC), completed a geotechnical investigation for the proposed replacement of the Greater Winnipeg Water District (GWWD) railway bridge located at Mile 41.3 in the RM of Springfield, Manitoba. The purpose of the geotechnical investigation was to verify the subsurface soil and groundwater conditions at the site in order to provide geotechnical recommendations for foundation design and construction. The scope of work for the project was outlined in AMEC's proposal number WPG2013.557, dated 3 December 2013. The geotechnical investigation was completed under subcontract to Stantec, Stantec Project Number and Subconsultant Agreement 113732050.

This report summarizes the field and laboratory testing programs, describes the subsurface conditions encountered at the test hole locations, and presents geotechnical engineering recommendations for: driven steel pile foundation alternatives; frost design considerations; abutment backfill and lateral earth pressures; and foundation concrete. Slope stability analyses, embankment settlement analyses, and pore pressure and fill staging analyses were not part of the scope of work for this geotechnical investigation. AMEC has assumed that these analyses are being undertaken by others as required for design.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Site Description

The GWWD Mile 41.3 site is located within the RM of Springfield, near the intersection of Forestry Road and Road 32E. Specifically, the site is located about 560 m west of Centerline Road, where the GWWD Rail Line crosses Cook's Creek.

At Mile 41.3, the GWWD rail line consists of a single track. Currently the crossing consists of a double span wooden bridge, supported on timber piles. Head walls at the abutments consisted of wooden lagging supported by steel piles. Installation depths, for both the wooden foundation piles and steel abutment piles, as well as sizing details, were not known. Rip-rap appeared to have been placed on both sides of the crossing, both under the bridge and extending out from the bridge on both sides. The thickness, material type, total quantity, and total coverage area of the rip-rap could not be determined due to snow cover.

Cook's Creek is oriented relatively perpendicular to the existing bridge structure. Drainage ditches providing drainage into Cook's Creek were present along both sides of the rail embankment. Generally, the site is surrounded by flat-lying farm fields, with the rail track elevated about 2 to 3 m above the surrounding fields. The elevated berm on which the rail track is located extended beyond the bridge site on both sides of the crossing. To the east of the rail bridge, Road 32E crosses the rail line at an at-grade crossing.

At the time of the geotechnical investigation, the farm fields surrounding the site, as well as the rail embankment, were covered by snow. Access to the site was gained via the rail line, which had been closed to rail traffic by the City of Winnipeg at the request of Stantec in order to provide a safe work area for drilling.

2.2 Proposed Development

AMEC understood that the proposed development at Mile 41.3 consists of a full replacement of the existing wooden bridge. Exact details of the proposed bridge were not known, however AMEC understood that the new structure will be a single span structure of approximately equal size to the existing structure and that abutment locations would not change significantly from their current location. Based on information provided by Stantec, AMEC understood that steel HP310x110 piles are the preferred foundation type. Foundation loads were not available at the time of this report.

3.0 GEOTECHNICAL INVESTIGATION PROGRAM

Prior to initiating drilling, AMEC notified public utility providers (i.e. Manitoba Hydro, MTS, City of Winnipeg, etc.) of the intent to drill in order to clear public utilities, and where required, met with said representatives on-site.

On 17 December 2013, AMEC supervised the drilling of two test holes (TH01 and TH02) at the approximate locations illustrated in Figure 1. The test holes were drilled using an Acker MP5 track mounted drill rig equipped with 125 mm diameter solid stem and 175 mm diameter hollow stem augers; operated by Maple Leaf Drilling Ltd. of Winnipeg, Manitoba.

During drilling, AMEC field personnel visually classified the soil stratigraphy within the test holes in accordance with the Modified Unified Soil Classification System (MUSCS); as well as noted any observed seepage and/or sloughing conditions. Grab samples were collected at selected depths and retained in sealed plastic bags for shipping, review, and select testing in AMEC's Winnipeg laboratory. Shelby tube samples were also collected at selected depths for possible laboratory testing. The in-situ relative consistency of cohesive overburden was evaluated during drilling using pocket penetrometer readings. The recorded pocket penetrometer readings are shown on the test hole log. The relative consistency of sand and of the underlying till was evaluated using standard penetration tests (SPTs), where the number of blows to drive the SPT sampler 0.3 m into the soil was recorded. The recorded number of blows is shown on the test hole logs as the SPT (N) value.

Upon completion of drilling, the depth to slough and groundwater level within each test hole was obtained after an elapsed time of about 10 minutes. Subsequently, the test holes were backfilled to grade with bentonite and auger cuttings. Excess auger cuttings were left neatly on site. UTM coordinates of the test hole locations were obtained using a hand held Garmin GPS, and are summarized in Table 3-1.

Table 3-1: Testhole Coordinates (UTM)

Testhole ID	Northing	Easting	Local Elevation[1] (m)
TH01	5516021.5	690232.9	~ 99.7
TH02	5516028.5	690256.9	~ 99.7

1. Local elevation 100 m equals approximate top of track

Following completion of the field drilling program, a laboratory testing program was conducted on selected soil samples obtained from the test holes. The laboratory testing program consisted of moisture content determinations, and one set of liquid limit and plastic limit determinations.

Detailed test hole logs summarizing the sampling, field testing, laboratory test results, and subsurface conditions encountered at the test hole locations are presented in Appendix A. Actual depths noted on the test hole logs may vary by ± 0.3 m from those recorded due to the method by which the soil cuttings are returned to the surface. Summaries of the terms and symbols used on the test hole logs and of the Modified Unified Soil Classification System are also presented in Appendix A.

4.0 SUBSURFACE CONDITIONS

4.1 Stratigraphy

Consistent with the regional geology and anticipated conditions, the stratigraphy at the test hole location consisted of the following, in descending order from grade level:

- Sand and Gravel Fill
- Sand or Clay
- Silt (Till)

A brief description of each of the soil layers bulleted above is presented below: For detailed descriptions, the test hole logs in Appendix A should be consulted.

<u>Sand and Gravel Fill</u>

Sand and gravel fill was encountered at the surface of both test holes, and extended to about 3.1 m below grade at TH01 and about 1.5 m below grade at TH02. The fill was generally described as poorly graded, medium to coarse grained, frozen, and brown. In-situ moisture contents completed on a total of six thawed samples from both boreholes ranged from about 3 percent to 6 percent.

Significant sloughing of the sand and gravel fill layer was noted in TH01, and necessitated a switch from solid stem auger to hollow stem auger drilling beyond 4.6 m below grade to control slough into the open bore. Solid stem auger drilling was not attempted at TH02, which was immediately drilled with hollow stem from grade.

Sand

Sand was encountered beneath the sand and gravel fill at TH02 and extended to about 4.6 m below surface. The sand was generally described as poorly graded, fine to medium grained with trace fines becoming silty with some clay below about 3.1 m, moist becoming wet below about 3.1 m, and grey. SPT N value of 22 blows and 11 blows were obtained within the layer at about 1.5 m and 3.1 m below grade, respectively, indicative of compact relative density. In-situ moisture content results within the layer ranged from about 4 percent within the clean sand at the top of the layer to about 30 percent within the wet silty sand below 4.6 m.

Clay

Clay was encountered beneath the sand and gravel fill at TH01 and extended to about 4.6 m below surface. The clay was silty with trace organics and rootlets, high plastic, moist, firm becoming stiff below 3.6 m, and dark grey. In-situ moisture contents of about 38 percent and 30 percent were obtained on two samples obtained at about 3.5 m below grade and 4.5 m below grade, respectively.

Although not observed in TH02, it is advised that a thin layer of clay (i.e. less than 1.5 m thick) could have gone unnoticed due to the Hollow Stem Auger technique employed, and in this regard, may exist above the silt till at TH02.

Silt(Till)

Glacial silt till was encountered beneath the clay in TH01 and the sand at TH02 at about 4.6 m below grade, and extended to the test hole termination depth (defined by practical SPT refusal) of 8.1 m at both test holes. The till comprised a low to non plastic silt matrix containing some sand, some gravel, trace clay, and was moist to damp. In-situ moisture contents within the till ranged from about 16 percent to about 8 percent. Atterberg Limits testing on a sample of the silt collected at about 8.0 m below grade indicated a liquid limit of about 17 percent, and a plastic limit of about 9 percent.

SPT ‘N’ values ranged from 23 at the top of the till in TH02 to in excess of 50 blows per foot at all other locations, suggesting dense to very dense conditions.

4.2 Groundwater and Sloughing Conditions

Seepage and sloughing conditions were noted during drilling, and the depths to the accumulated water levels within the test holes were measured about ten minutes after drilling.

Sloughing of the sand and gravel was noted during solid stem auger drilling at TH01, and eventually necessitated a switch in drilling technique from solid stem augers to hollow stem auger to control slough into the open bore. Hollow stem auger drilling from grade at TH02 prevented sloughing of the sand and gravel fill.

Slight seepage within TH01 was observed between grade and 3.7 m below grade, as well as between 4.6 m and 8.1 m below grade. Similarly, seepage was observed in TH02 below 1.5 m below grade after the augers were removed.

Upon completion of drilling and removal of the hollow stem augers, the test holes remained open to between 6.4 m and 6.7 m below grade. The depth to accumulated water was measured at 5.8 m below grade within both boreholes.

Seepage water within both boreholes is considered likely as originating as perched groundwater within the upper sand and sand fill, as well as groundwater originating from thin sand stringers within the till that went undetected as a result of the hollow stem auger and rotary drill technique. For design purposes per the recommendations outlined in this report, a groundwater table of 3.3 m below top of track (i.e. local elevation 96.7 m) is recommended. 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.

5.0 GEOTECHNICAL RECOMMENDATIONS

5.1 General Evaluation

The stratigraphy and soil conditions encountered within the test holes advanced at the site is considered typical of conditions within the region and are considered favourable for the proposed development.

From a foundations perspective, soil conditions are considered suitable for the use of the driven steel H-piles indicated as preferred by Stantec. Driven steel pipe piles are also regarded as a suitable pile foundation alternative. Other suitable pile alternatives included bored concrete piles bearing within the underlying silt till; however, bored piles would necessitate casing through the sand fill and clay to control potential slough conditions. Given Stantec's indicated preference for driven Steel H-Piles, foundation recommendations here-in are limited to driven steel HP and pipe piles. Recommendations for other foundation alternatives can be provided upon request.

The following sections provide discussion and recommendations as they pertain to: driven steel piles; lateral earth pressures on below grade walls; frost design considerations; and foundation concrete.

5.2 Driven Steel Pile Foundations

As previously discussed, soil conditions at the site are considered suitable for the use of the driven steel H-piles indicated as preferred by Stantec, as well as pipe piles. Notwithstanding, the following conditions should be considered in final selection and design of piles:

- The underlying silt till at the site below 6 m is very dense and depending on selection of the pile type (i.e. H-Pile, open-ended, or closed-ended pipe), end bearing development could vary with pile type and location. H-piles are anticipated to penetrate deeper than open ended or closed ended pipe piles.
- High end-bearing development within the silt till could inhibit pile penetration local elevation 94 m (i.e. beyond 6 m below test hole elevation) and the achievable

embedment depth for tensile (uplift) resistance to transient uplift loads and frost. In this regard, pile type selection and sizing must consider both the compressive and tensile requirements of the pile, and the ability to both achieve the required compressive capacity and achieve the minimum embedment depth required for uplift resistance.

AMEC understands that the foundation will be designed in accordance with the 2013 AREMA Manual for Railway Engineering. AMEC's interpretation of recommended practices outline in the manual is that foundation design employs allowable stress design (ASD) principles as opposed to Limit State or Load-Factor Resistance Design (LFRD) design principles. In this regard, parameters here-in have been presented for use in ASD. If parameters for alternative design principles (i.e. Limit States) are required, this office should be contacted for revisions.

5.2.1 Axial Compressive Resistance of Single Driven Steel Piles

The 'allowable' compressive resistance of a driven steel pile (H or pipe) as a function of embedment depth may be determined using the 'allowable' unit shaft friction and unit end bearing pressures recommended in Table 5-1.

Table 5-1: 'Allowable' Unit Shaft Friction & End Bearing Values for Driven Steel Piles

Elevation Range (m)	Assumed Average Soil Type	Shaft Friction (kPa)	End Bearing (kPa)
99.7 to X[2]	Sand Fill	Linearly increasing with depth from: 0 to 12	--*
X to 95.1	Sand or Clay	12	--*
95.1 to 93.7	Silt Till	24	560
93.7 to 91.6 m	Silt Till	48	1,800

[1] The elevations presented assume top of track to be approximate local elevation 100.0 m.
[2]X = the elevation of the frost penetration front at the pile interface, determined in accordance with the recommended frost penetration depth presented in Section 5.4, to account for possible movement of the soil away from the perimeter of the pile.

The above 'allowable' unit shaft friction and 'allowable' unit end bearing values include a factor of safety of 2.5.

For all pipe pile types and sizes, shaft friction should only be applied to the exterior surface area of the pile. In the case of steel H piles, shaft friction may be applied to the exterior sides of the two flanges plus twice the depth of the web (i.e. the box perimeter). For pipe piles with a closed-end configuration, end bearing may be applied to the full cross-sectional area of the toe of the pile. For H-piles and open end pile configurations, the area over which end bearing may be applied varies with the pile diameter. For small diameter pipes piles (i.e. DN300 or smaller) and H-Piles, there is considered a higher potential for 'plugging' of the pile during installation, and as such, it is considered acceptable to apply the end bearing to the full cross-sectional area of the toe of the pile which may be taken as the area enclosed by the outer circumference of a pipe section, or the cross sectional area of a rectangle bounded by the flanges in the case of steel H sections. For larger pile sizes, 'plugging' of the pile during driving may be variable, and the end bearing values provided above should be re-evaluated by AMEC for large diameter piles. However, for current design purposes, the unit end bearing values outlined above may be applied to the steel area of the toe of piles larger than DN300. If during construction driving resistance is lower or higher than anticipated, 'soil plug' development and end bearing development may be quantified via dynamic pile testing by pile driving analyzer (PDA Testing) and CAPWAP1 analysis.

Due to limitations on the driveability of the pile imposed by the yield strength of the pile, as a guide to initial design and selection of pile wall thickness and steel grade, it is recommended that the maximum design 'allowable' compressive resistance of a steel pile be limited to 0.25FyAs (i.e. a fraction of the unfactored structural yield capacity of the pile), where: fy is the nominal yield stress of the steel, and As is the cross-sectional area of steel in the pile. The purpose of this restriction is to mitigate the risk of statically designing a pile that cannot be driven with enough energy or force to overcome dynamic soil resistance and subsequently develop the design static load resistance without yielding or damaging the pile. Subject to driveability analysis and evaluation of driving stresses at the pile design stage, the maximum 'allowable' compressive stress could be increased to as much as 0.35FyAs.

Additional comments for design and construction of driven steel piles are as follows:

- Static pile design parameters pertain to soil resistance only. The pile cross sections must be designed to withstand the design loads and the driving forces during installation.
- Although not commonly employed for the installation of driven piles, if a pre-bore was required (i.e. for ground disturbance clearance or contractor preference), shaft friction must be neglected over the depth of the pre-bore.
- Piles must be spaced a minimum of three pile diameters apart, as measured from centre-to-centre, in order to act individually as single piles in vertical compression when used in a small pile group of three piles or less. Where pile groups larger than 3 piles are required, the pile group should be reviewed by AMEC.
- Once the pile configuration is known, AMEC recommends that a driveability analysis (i.e. WEAP) be completed prior to proceeding to construction, and concurrent with selection of the pile driving equipment, to confirm the ability of the hammer and appurtenances to drive the piles to the design capacity and embedment depth without damage. Similarly, the driveability analysis can be extended to develop termination criteria for use in pile installation monitoring. It should be noted that driveability analyses should be completed using ultimate soil parameters.
- All piles driven within five pile diameters should be monitored for heave and, where heave is observed, piles should be re-driven. Piles that are re-driven should be advanced to at least the original elevation.
- All piles should be driven continuously to practical refusal once driving is initiated.

[1] PDA : Pile Driving Analyzer, CAPWAP: software to analyze PDA Test data

- Any piles that have been damaged, are excessively out of plumb, or have refused prematurely may need to be replaced, pending a review by a qualified geotechnical engineer of their load carrying capability and estimated settlement.
- All pile caps and grade beams should be underlain by a minimum 150 mm thick void form to accommodate the expansive nature and potential frost heave of the underlying soil.
- Prior to the pile installation, the piles should be inspected to confirm that the material specifications are satisfied. As a minimum, steel H-piles should meet the requirements of CAN/CSA-G40.20/G40.21, Grade 350 W, and pipe piles should have a minimum yield strength of 310 MPa (i.e. ASTM A252 Grade 3 steel). The piles should be free from protrusions, which could create voids in the soil around the pile during driving.
- Monitoring of the pile installation by an experienced inspector is recommended to verify that the piles are installed in accordance with design assumptions and the driving criteria are satisfied. For each pile, a complete driving record in terms of the number of blows per 300 mm of penetration should be recorded by the inspector and reviewed during pile installation by the designer.

5.2.2 Tensile (Uplift) Resistance (Single Pile)

In the case of driven steel piles, the uplift resistance of a single pile will be provided by the sustained downward load on the pile (if applicable) and shaft friction along the length of pile embedded below the depth of frost penetration. In the case of straight shaft (i.e. driven steel) piles, the soil component of the 'allowable' uplift resistance to *sustained* tensile loads will be provided by shaft friction and can be determined using 70% of the shaft friction values outlined in Table 5-1. For pipe piles, only the exterior surface area of the pile in contact with the soil should be used in the calculation of the frictional resistance. In the case of steel H piles, the surface area should include the exterior sides of the two flanges plus twice the depth of the web. For frost and *transient* uplift loads, such as those due to wind gusts, no reduction of the shaft friction values in Table 5-1 is required. Transient loads would not be additive to the uplift forces due to frost action.

Although not commonly employed for the installation of driven steel piles, if a pre-bore was required (i.e. for ground disturbance clearance or contractor preference), shaft friction must be neglected over the depth of the pre-bore.

5.2.3 Lateral Resistance (Single Pile)

Piles resist laterally applied loads by deflecting until the necessary resistance is mobilized in the adjacent soils. The majority of lateral load resistance for slender piles is generally provided within the upper 4 to 5 m of the soil profile (i.e. the typical point of inflection for the pile). The maximum bending moment typically occurs at 1.5 m to 3.0 m below grade depending on the applied loading and soil resistance. The allowable lateral capacity depends upon the properties of the soil and pile material, pile sizes, fixity of the top of the pile, depth of embedment, height of load application above ground, vertical load applied and tolerable deflections.

Where the lateral load capacities or magnitude of movements of piles are critical, it is recommended that the lateral deflections and design capacities of piles or groups of piles be evaluated using Reese's method of p-y curves. This method models the strength-deformation characteristics using load-displacement curves for the various soil strata, and the non-linear behaviour of the soil. With the method of p-y curves, solutions may be obtained through an iterative procedure performed using LPILE Software for single piles, and extended to pile groups by using GROUP Software to analyze the behaviour of piles in a group subjected to both axial and lateral loadings. The analytical procedure provides lateral pile deflections, generated bending moments, shear forces, and the soil reaction computed at close intervals over the depth of the pile.

Based on conditions observed within the appended test hole logs, the stratigraphy and soil parameters outlined in Table 5-2 are considered suitably representative of the average subsurface conditions expected to influence the lateral behaviour of driven steel piles at the Site. Clay has conservatively been assumed above the till between 3.1 m and 4.6 m below grade.

Table 5-2: LPile Input Parameters

Elevation Range (m) [1]	Soil Type	Effective Unit Weight (kN/m^3)[2]	Friction Angle (•)	Cohesion (kPa)	E50 (%)	p-y subgrade modulus, k (kPa/m)
100.0 to 96.7	Sand Fill	20	28	0	n/a	Default
96.7 to 95.1	Clay	9	n/a	50	0.015	Default
95.1 to 91.6	Silt Till	10	35	0	n/a	Default

[1] The elevations presented assume top of track to be approximate local elevation 100.0 m.
[2] Groundwater level of 3.3 m below top of track was assumed.

The use of zero lateral resistance or skin friction in the upper part of the pile for sandy soils has not been recommended because the sand is cohesionless and therefore a permanent gap between the pile and the soil due to installation or frost effects is not expected.

Lateral pile analysis of a prescribed pile configuration was not part AMEC's scope of work for this investigation. Notwithstanding, lateral pile analysis could be conducted by AMEC for specified pile configurations on request.

5.2.4 Single Pile Settlement

The settlement of a single pile depends on the applied load, strength-deformation properties of the foundation soils, load transfer mechanism, load distribution over the pile embedment depth, and the relative proportions of the load carried by shaft friction and end-bearing. Assuming good workmanship, inclusive of good excavation, the predicted settlement of piles at working loads equal to a maximum given by the 'allowable' pile capacity are 0.5 to 1 percent of the shaft diameter plus the elastic shortening of the pile due to the compressive load acting on the pile.

5.2.5 Pile Group Effects

Generally, piles will behave individually in compression (i.e. group efficiency equals 1.0) when a minimum centre-to-centre spacing of 5 pile diameters is provided between adjacent piles, and will behave individually laterally when the center-to-center spacing is greater than 3 diameters in the direction transverse to loading (side-by-side), and greater than 8 diameters in the direction parallel to loading (in-line). However, for circumstances in which piles are closely spaced and/or the piles are connected by a rigid pile cap forcing equal settlement behaviour at the pile heads, interaction between the piles will occur and should be considered in design.

Notwithstanding the above, AMEC does not anticipate that large groups of four or more closely spaced piles will be required. Consequently, recommendations pertaining to the axial and lateral load resistances of pile groups are not provided here-in. If pile groups are required by design, AMEC should be notified and a review of possible group interactions effects evaluated.

5.3 Lateral Earth Pressures on Below Grade Walls (i.e. Wing Walls)

5.3.1 Soil Design Parameters

Below grade walls (i.e. wing walls) will be required to resist lateral pressures from the surrounding soil, water, and any additional surcharge loading (i.e. fill, live surface loads, etc.). Table provides recommended design values for the bulk unit weight, angle of internal friction, and ‘at rest’, active, and lateral earth pressure coefficients for moderately to well compacted native sand and compacted granular fill soils.

Table 5-3: Earth Pressure Coefficients and Soil Unit Weights

Soil Type		Active Pressure Coefficient K_a	“At Rest” Earth Pressure Coefficient K_o	Passive Pressure Coefficient[a] K_p	Total Soil Unit Weight (kN/m^3)	Friction Angle (deg) Between Soil and Concrete
Gravel Fill	Well Compacted	0.25	0.40	2.67	23	25
	Moderately Compacted	0.30	0.47	2.17	22	21
Sand Fill	Well Compacted	0.30	0.47	2.17	21	21
	Moderately Compacted	0.36	0.53	1.85	20	18

The passive earth pressure coefficients provided in Table 5-3 include a reduction factor of 1.5 to account for the partial mobilization of passive resistance that is consistent with the small wall displacements expected under operational conditions. Relatively large wall displacements would be necessary to realize full passive resistances.

With respect to subsurface drainage and groundwater conditions over the depth of the foundation structure, the phreatic surface at the site should be taken as 3 m below existing grade. The use of free draining backfill and the provision of drainage behind vertical subsurface walls is strongly recommended, and will further serve to mitigate frost action on vertical walls extending through the zone of frost penetration.

The magnitude and distribution of the lateral earth pressures on below grade structures will depend on such factors as the rigidity of the below grade structure; the degree of compaction of the backfill against the structure; the backfill soil type; the slope angle at the structure/soil interface; and the subsurface drainage and groundwater conditions over the height of the structure. It is anticipated that a sloped excavation will be implemented for construction of below grade foundation structures, which will necessitate the placement of backfill behind below grade structure walls. The magnitude and distribution of the lateral earth pressures (P) on below grade structures will depend on the degree of compaction of the backfill. In addition to earth pressures, lateral stresses generated by any applicable surcharge loads also need to be evaluated in the design. Recommended earth pressure distributions for light to moderate and moderate to well compacted backfill cases, as well as for line or point surcharge loads, are discussed in Section 5.3.2.

5.3.2 Calculation of Earth Pressure Distributions and Load Factors

5.3.2.1. Moderate to Well Compacted Backfill Case

Where subgrade support on the surface of the retained soil behind a wall is required, as it is for headwalls, the backfill against the wall will need to be compacted to at least 95 percent Standard Proctor maximum dry density. The use of free draining backfill behind below grade structures is strongly recommended in order to maintain drained conditions behind the structure. Assuming drained conditions, the design earth pressure distribution should adopt a combined trapezoidal/triangular distribution as shown on Figure 2 to account for the induced lateral pressures due to compaction. Figure 2 also provides the relationships to be used in the calculation of the compaction induced earth pressures, and tabulated loads (P) generated by typical compactors. The earth pressure coefficients to be used in the calculation of the lateral pressures should be those applicable to the backfill types given in Table above.

If sub-drainage is not provided and it is possible by design for a perched groundwater to develop within the retained soil (i.e. “bathtub” effect associated with gravel fill soils surrounded by low permeable fine grained soil types), the hydrostatic component should be included in addition to the earth pressure given in Figure 2.

5.3.2.2. Surcharge Loads

In addition to earth pressures, lateral stresses generated by surcharge loads, such as point loads from locomotives, also need to be evaluated in the design. For line or point surcharge loads, the lateral pressures should be determined using the relationships given in Figure 4. In the case of uniformly distributed surcharge loads, such as those acting on the surface of the retained soil, the induced lateral earth pressure may be determined by multiplying the surcharge load by the appropriate earth pressure coefficient.

5.4 Frost Design Considerations

5.4.1 Frost Penetration Depth

The upper stratigraphy at the test hole locations, and across the site, is considered moderately to highly frost susceptible in the presence of water, and as such, frost effects should be considered for foundations or surface structures sensitive to movement. Based on historical temperature data for the area, a design frost penetration of 2.4 m below final grade is recommended in unheated areas that will not have regular snow or vegetative ground cover. It should be noted that this recommended frost penetration depth extends both vertically and laterally behind final surface (i.e. extends 2.4 m behind the headwall).

5.4.2 Pile Foundations

Frost forces applied to pile foundations include adfreeze pressures acting along the pile shafts within the depth of frost penetration. If pile caps are used and extend beyond the perimeter of the underlying pile, then frost heave forces acting on the undersides of the pile caps, as well as any connecting supports (i.e. lateral tie between the piles) will also need to be considered.

5.4.2.1. Frost Heave

To reduce the potential of frost heave pressures, a void-forming product should be installed beneath the underside of the pile caps and any other structural element located within the depth of frost penetration above the groundwater table. The recommended minimum thickness of the void should be 150 mm. Alternatively, a compressible material may be used in lieu of a void forming material, and the uplift pressures may be taken as the crushing strength of the compressible medium. It is recommended that a frost heave of 150 mm be assumed in determining the required thickness for the void-filler and the associated uplift pressures associated with the thickness used.

The finished grade adjacent to each pile cap should be capped with well compacted clay and sloped away so that the surface runoff is not allowed to infiltrate and collect in the void space or in the compressible medium.

The use of void-forming product below the groundwater is unfeasible. In instances where groundwater is located within the recommended depth of frost penetration, the underside of foundation elements such as pile caps should extend below the depth of frost penetration to mitigate frost heave development on the underside of the foundation element.

5.4.2.2. Adfreeze Stresses

Resistance to adfreeze and frost heave forces will be provided by the sustained vertical loads on the foundation, the buoyant weight of the foundation and dead weight of the structure, and the soil uplift resistance component provided by the length of the pile extending below the depth of frost penetration. In the case of straight shaft (i.e. driven steel) piles, the adfreeze force acting on the pile may be determined assuming an unfactored unit adfreeze stress of 65 kPa applied to the exterior surface of the pile and supported foundation elements (i.e. pile caps) located within the zone of frost penetration. The uplift resistance of the pile below the depth of frost may be determined using the Tensile (Uplift) Resistance recommendations presented in Section 5.2.2.

5.5 Foundation Concrete

Where concrete elements outlined in this report and all other concrete in contact with the local soil will be subjected in service to weathering, sulphate attack, a corrosive environment, or saturated conditions, the concrete should be designed, specified, and constructed in accordance with concrete exposure classifications outlined in the latest edition of CSA standard A23.1, Concrete Materials and Methods of Concrete Construction. In addition, all concrete must be supplied in accordance with current Manitoba and National Building Code requirements.

Based on significant data gathered through previous work in Southern Manitoba, water soluble sulphate concentrations in the soil are typically in the range of 0.2% to 2.0%. As such, the degree of sulphate exposure at the site may be considered as ‘severe’ in accordance with current CSA standards, and the use of sulphate resistance cement (Type HS or HSb) is recommended for concrete in contact with the local soil. Furthermore, air entrainment should be incorporated into any concrete elements that are exposed to freeze-thaw to enhance its durability.

It should be recognized that there may be structural and other considerations, which may necessitate additional requirements for subsurface concrete mix design.

5.6 Construction Monitoring and Testing

All engineering design recommendations presented in this report are based on the assumption that an adequate level of testing and monitoring will be provided during construction and that all construction will be carried out by a suitably qualified contractor experienced in foundation and earthworks construction. An adequate level of testing and monitoring is considered to be:

- for earthworks: full-time monitoring and compaction testing.
- for deep foundations: design review and full time monitoring during construction.
- for concrete construction: testing of plastic and hardened concrete in accordance with the latest editions of CSA A23.1 and A23.2; and review of concrete supplier’s mix designs for conformance with prescribed and/or performance concrete specifications.

AMEC requests the opportunity to review the design drawings, and the installation of the foundations, to confirm that the geotechnical recommendations have been correctly interpreted. AMEC would be pleased to provide any further information that may be needed during design and to advise on the geotechnical aspects of specifications for inclusion in contract documents.

6.0 CLOSURE

The findings and recommendations presented in this report were based on geotechnical evaluation of the subsurface conditions observed during the site investigation described in this report. If conditions other than those reported in this report are noted during subsequent phases of the project, or if the assumptions stated herein are not in keeping with the design, this office should be notified immediately in order that the recommendations can be verified and revised as required. Recommendations presented herein may not be valid if an adequate level of inspection is not provided during construction, or if relevant building code requirements are not met.

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 in 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 has been prepared for the exclusive use of Stantec Consulting Ltd., and their agents, for specific application to the project described in this report. The data and recommendations provided herein should not be used for any other purpose, or by any other parties, without review and written advice from AMEC. Any use that a third party makes of this report, or any reliance or decisions made based on this report, are the responsibility of those parties. AMEC accepts no responsibility for damages suffered by a third party as a result of decisions made or actions based on this report.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, either expressed or implied, is made.

Respectfully submitted,
AMEC Environment & Infrastructure,
A Division of AMEC Americas Limited

Reviewed by:

DRAFT FOR CLIENT REVIEW

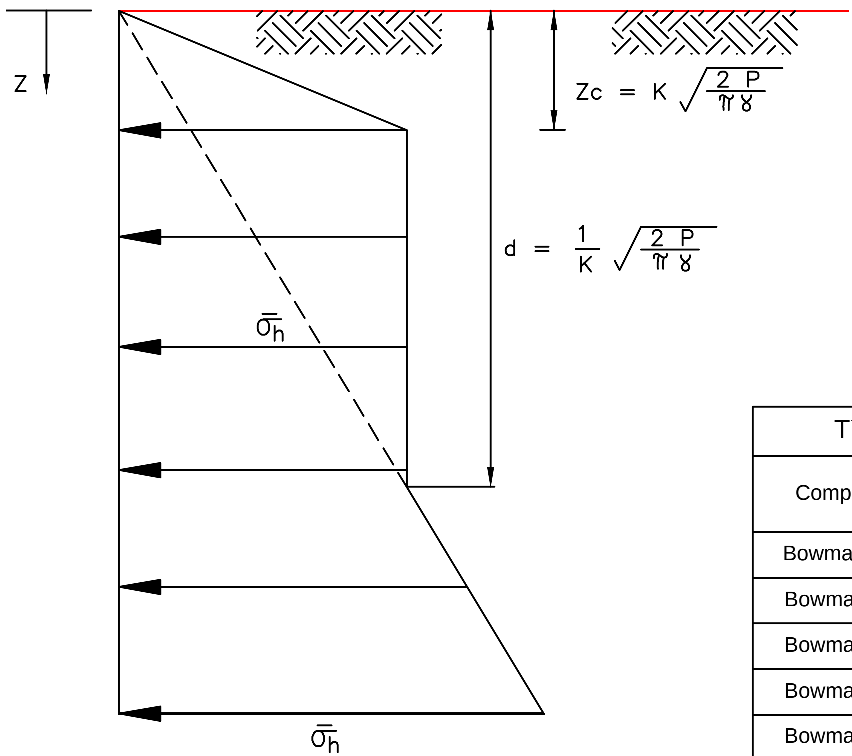
Kelly Johnson, P. Eng.
Senior Geotechnical Engineer

Harley Pankratz, P.Eng.
Vice President, Eastern Prairies/Northern Alberta

FIGURES

CLIENT LOGO	CLIENT STANTEC CONSULTING LTD.	DWN BY: MD	GEOTECHNICAL INVESTIGATION GREATER WINNIPEG WATER DISTRICT RAILWAY BRIDGE MILE 41.3	REV. NO.: A
		CHK'D BY: TG		DATE: FEBRUARY 2014
AMEC Environment & Infrastructure 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	amec	DATUM: NAD83		PROJECT NO: WX17312
		PROJECTION: UTM Zone 14 U	TEST HOLE LOCATION PLAN	FIGURE No. FIGURE 1
		SCALE: AS SHOWN		

P:\JOBS\17300'S\17310'S\17312 STANTEC - GWWD BRIDGES\DRAWINGS\WX17312.DWG



$$Z_c = K\sqrt{\frac{2P}{\pi\gamma}}$$

$$d = \frac{1}{K}\sqrt{\frac{2P}{\pi\gamma}}$$

EARTH PRESSURE DISTRIBUTION

FOR $Z_c \leq Z \leq d$

$$\bar{\sigma}_h = \sqrt{\frac{2P\gamma}{\pi}}$$

FOR $Z > d$

$$\bar{\sigma}_h = K \cdot \gamma \cdot Z$$

TYPICAL COMPACTOR LOADS (P)			
Compactor	LOAD (P) kN/m	Compactor	LOAD (P) kN/m
Bowmag TSE	31	Bowmag BW122PD	36
Bowmag 60S	32	Bowmag 142PDB	47
Bowmag 65S	22	Bowmag 172PDB	93
Bowmag 75S	33	Dynapac LR100	42
Bowmag 90S	39	Dynapac 2100V	93
Bowmag 75AD	20	Dynapac CA121D	53
Bowmag 100AD	20	Dynapac CA121PD	54
Bowmag 120AD	34	Dynapac CA151	80
Bowmag 130AD	36	Dynapac CA151D	80
Bowmag BW122D	30	Dynapac CA151PD	96

$$P \text{ (ROLLER LOAD)} = \frac{\text{DEAD WT. OF ROLLER + CENTRIFUGAL FORCE}}{\text{WIDTH OF ROLLER}}$$

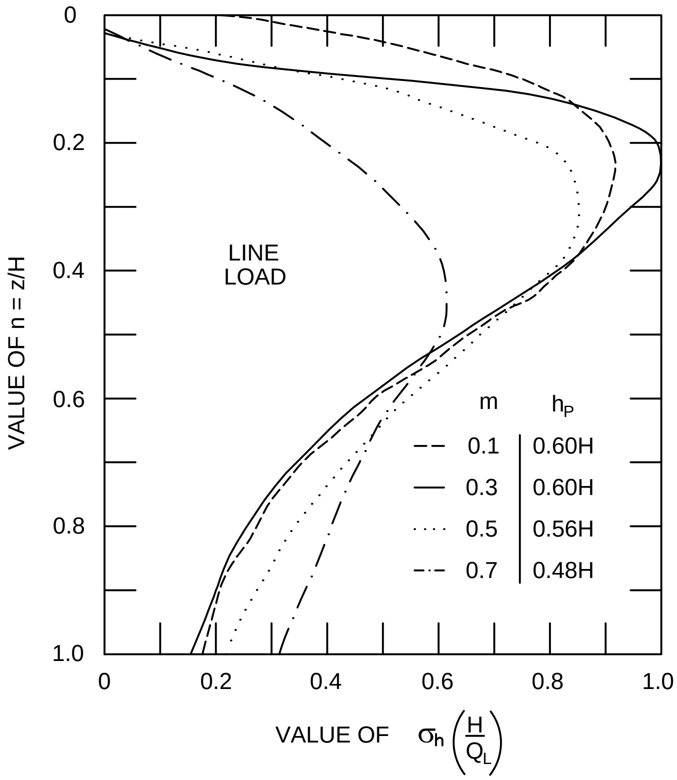
TYPICAL VALUES GIVEN IN TABLE

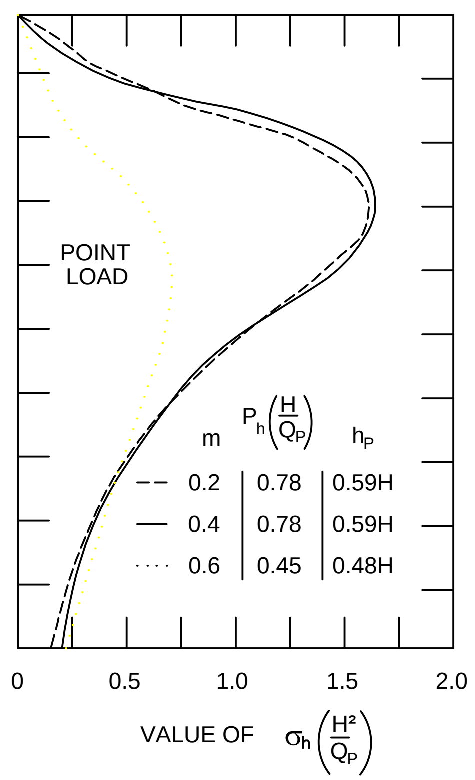
EARTH PRESSURE COEFFICIENTS

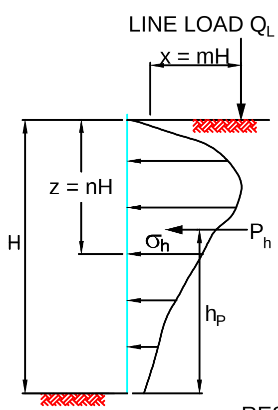
$K = K_o$ ("AT REST") OR K_a (ACTIVE CASE)
(SEE TEXT OF REPORT)

γ = SOIL UNIT WEIGHT
(SEE TEXT OF REPORT)

CLIENT: STANTEC CONSULTING LTD.	DWN BY: MD	GEOTECHNICAL INVESTIGATION GREATER WINNIPEG WATER DISTRICT RAILWAY BRIDGE MILE 41.3	DATE: FEBRUARY 2014
	CHK'D BY: KJ		PROJECT No.: WX17312
AMEC Earth & Environmental 5681-70 STREET, EDMONTON, ALBERTA, T6B 3P6 PHONE 780-436-2152, FAX 780-435-8425	DATUM: -		REV. No.: -
	PROJECTION: -	LATERAL EARTH PRESSURES INDUCED BY COMPACTION	FIGURE No.: FIGURE 2
	SCALE: NOT TO SCALE		







FOR m ≤ 0.4:

$$\sigma_h\left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

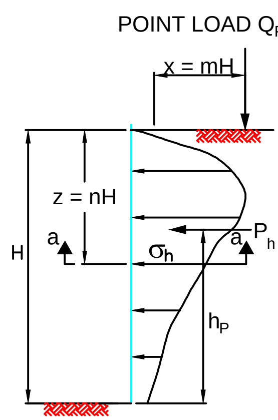
$P_h = 0.55\ Q_L$

FOR m > 0.4:

$$\sigma_h\left(\frac{H}{Q_L}\right) = \frac{1.28m^2n}{(m^2 + n^2)^2}$$

RESULTANT $P_h = \dfrac{0.64\ Q_L}{(m^2 + 1)}$

PRESSURES FROM LINE LOAD
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



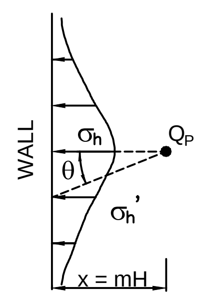
FOR m ≤ 0.4:

$$\sigma_h\left(\frac{H^2}{Q_P}\right) = \frac{0.28n^2}{(0.16 + n^2)^3}$$

FOR m > 0.4:

$$\sigma_h\left(\frac{H^2}{Q_P}\right) = \frac{1.77m^2n^2}{(m^2 + n^2)^3}$$

$$\sigma_h' = \sigma_h \cos^2(1.1\theta)$$



SECTION a-a
PRESSURES FROM POINT LOAD
(BOUSSINESQ EQUATION
MODIFIED BY EXPERIMENT)

CLIENT: STANTEC CONSULTING LTD.	DWN BY: MD	GEOTECHNICAL INVESTIGATION GREATER WINNIPEG WATER DISTRICT RAILWAY BRIDGE MILE 41.3	DATE: FEBRUARY 2014
	CHK'D BY: KJ		PROJECT No.: WX17312
AMEC Earth & Environmental 5681-70 STREET, EDMONTON, ALBERTA, T6B 3P6 PHONE 780-436-2152, FAX 780-435-8425	DATUM: -		REV. No.: -
	PROJECTION: -	LATERAL PRESSURES DUE TO SURCHARGE POINT AND LINE LOADS	FIGURE No.: FIGURE 3
	SCALE: NOT TO SCALE		

APPENDIX A

PROJECT: GWWD Bridge Mile 41.3	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH01**
CLIENT: Stantec Consulting Ltd.	DRILL TYPE: Acker MP5 Track Rig	PROJECT NO: WX17312
LOCATION: RM of Springfield, Manitoba	DRILL METHOD: 125 mm SSA	ELEVATION: 99.7 m

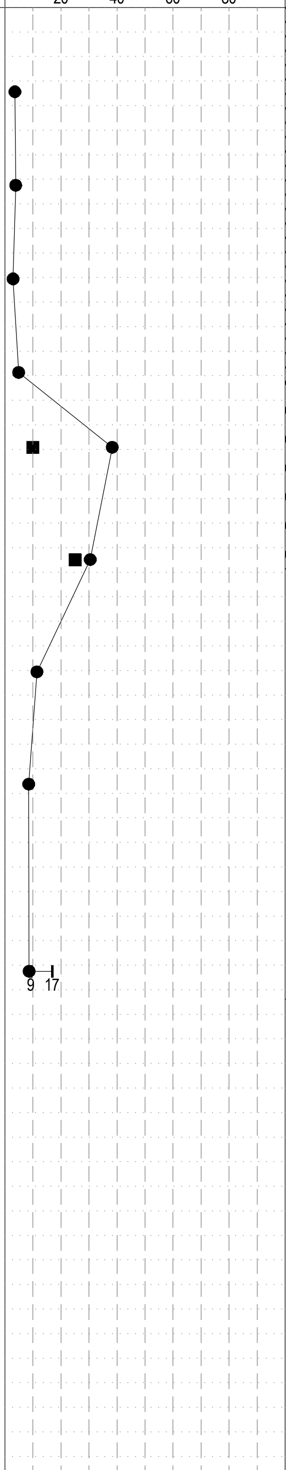
SAMPLE TYPE: Shelby Tube, No Recovery, SPT (N), Grab Sample, Split-Pen, Core

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

▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400

■ POCKET PENETROMETER (kPa) ■ 100 200 300 400

PLASTIC M.C. LIQUID 20 40 60 80



Depth (m)	SOIL SYMBOL	MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS	ELEVATION (m)
0		SP	SAND (FILL) - and gravel, poorly graded, medium to coarse grained, dense (inferred), frozen, brown	Grab Sample	1		TH Coordinates (UTM): 5516021.5mN 690232.9mE	99
1				Grab Sample	2			98
2				Grab Sample	3			97
3		CH	CLAY - silty, high plastic, moist, firm, dark grey, trace rootlets, trace organics	Grab Sample	4			
			- stiff, occasional black organic streaking below 3.6 m	Grab Sample	5			96
4				Grab Sample	6			
		CL-ML	SILT (TILL) - some sand, some gravel, low to non plastic, moist to damp, very dense, grey	Shelby Tube	7		**- Drill method switched to 175 mm HSA below 4.6 m to control sloughing of overlying sand**	95
5				SPT (N)	8	71		94
6				SPT (N)	9	50/140		93
7				SPT (N)	10	82		92
8			PRACTICAL SPT REFUSAL AT 8.1 m BELOW GRADE IN VERY DENSE TILL					91

NOTES:
- Significant sloughing was observed from 0 to 3.0 m below grade.
- Slight seepage was observed between 0 to 3.7 m and 4.6 m to 8.1 m below grade.
- Test hole remained open to 6.4 m below grade with water measured at 5.8 m 10 min after completion.
- Test hole was backfilled with auger cuttings and bentonite.

Depth scale: 9, 10, 11, 12; Elevation scale: 90, 89, 88

AMEC Environment & Infrastructure Winnipeg, Manitoba	LOGGED BY: HWP	COMPLETION DEPTH: 8.1 m
	REVIEWED BY: KJ	COMPLETION DATE: 17 December 2013
	Figure No. A1	Page 1 of 1

PROJECT: GWWD Bridge Mile 41.3	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH02**
CLIENT: Stantec Consulting Ltd.	DRILL TYPE: Acker MP5 Track Rig	PROJECT NO: WX17312
LOCATION: RM of Springfield, Manitoba	DRILL METHOD: 175 mm HSA	ELEVATION: 99.7 m

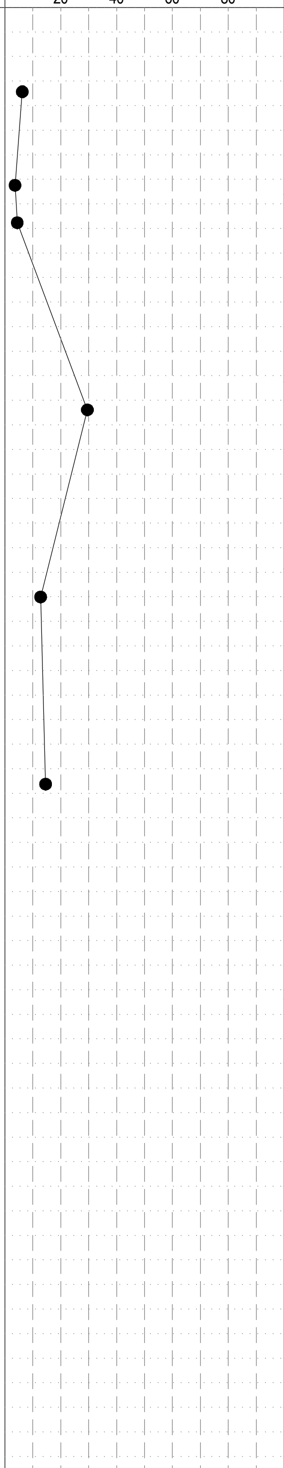
SAMPLE TYPE: Shelby Tube, No Recovery, SPT (N), Grab Sample, Split-Pen, Core

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

▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400

■ POCKET PENETROMETER (kPa) ■ 100 200 300 400

PLASTIC M.C. LIQUID 20 40 60 80



Depth (m)	SOIL SYMBOL	MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS	ELEVATION (m)
0		SP	SAND (FILL) - and gravel, poorly graded, medium grained, compact (inferred), frozen, brown - frozen till 0.8 m below grade	Grab Sample	1		TH Coordinates (UTM): 5516028.5mN 690256.9mE	99
				Grab Sample	2			
		SP	SAND - trace silt and clay fines, poorly graded, fine to medium grained, moist, compact, grey, trace rootlets	SPT	3	22		98
3		SM	SAND - silty, some clay, fine grained, wet, compact, grey	SPT	4	11		96
		CL-ML	SILT (TILL) - some sand, some gravel, trace clay, low to non plastic, compact, grey, occasional black sand pockets approx. 25 mm dia.	SPT	5	23		95
			- non plastic, very dense below 6.1 m	SPT	6	50/140		93
				SPT	7	90/280		92

PRACTICAL SPT REFUSAL AT 8.1 m BELOW GRADE IN VERY DENSE TILL

NOTES:
- Moderate seepage was observed below 1.5 m
- Test hole remained open to 6.7 m below grade with water measured at 5.8 m 10 min after completion
- Test was backfilled with auger cuttings and bentonite

Depth (m): 0 1 2 3 4 5 6 7 8 9 10 11 12

Elevation (m): 99 98 97 96 95 94 93 92 91 90 89 88

AMEC Environment & Infrastructure **Winnipeg, Manitoba**	LOGGED BY: HWP	COMPLETION DEPTH: 7.6 m
	REVIEWED BY: KJ	COMPLETION DATE: 17 December 2013
	Figure No. A2	Page 1 of 1

EXPLANATION OF TERMS AND SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of field investigation and subsequent laboratory testing are described in these pages.

It should be noted that materials, boundaries and conditions have been established only at the borehole locations at the time of investigation and are not necessarily representative of subsurface conditions elsewhere across the site.

TEST DATA

Data obtained during the field investigation and from laboratory testing are shown at the appropriate depth interval.

Abbreviations, graphic symbols, and relevant test method designations are as follows:

*C	Consolidation test	*ST	Swelling test
D_R	Relative density	TV	Torvane shear strength
*k	Permeability coefficient	VS	Vane shear strength
*MA	Mechanical grain size analysis and hydrometer test	w	Natural Moisture Content (ASTM D2216)
		w_l	Liquid limit (ASTM D 423)
N	Standard Penetration Test (CSA A119.1-60)	w_p	Plastic Limit (ASTM D 424)
N_d	Dynamic cone penetration test	E_f	Unit strain at failure
NP	Non plastic soil	γ	Unit weight of soil or rock
pp	Pocket penetrometer strength	γ_d	Dry unit weight of soil or rock
*q	Triaxial compression test	ρ	Density of soil or rock
q_u	Unconfined compressive strength	ρ_d	Dry Density of soil or rock
*SB	Shearbox test	C_u	Undrained shear strength
SO_4	Concentration of water-soluble sulphate	→	Seepage
		$\underline{\blacktriangledown}$	Observed water level

* The results of these tests are usually reported separately

Soils are classified and described according to their engineering properties and behaviour.

The soil of each stratum is described using the Unified Soil Classification System[1] modified slightly so that an inorganic clay of "medium plasticity" is recognized.

The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual[2].

Relative Density and Consistency:

Cohesionless Soils

Relative Density	SPT (N) Value
Very Loose	0-4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Cohesive Soils

Consistency	Undrained Shear Strength c_u (kPa)	Approximate SPT (N) Value
Very Soft	0-12	0-2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Standard Penetration Resistance ("N" value)
The number of blows by a 63.6kg hammer dropped 760 mm to drive a 50 mm diameter open sampler attached to "A" drill rods for a distance of 300 mm after an initial penetration of 150 mm.

1 "Unified Soil Classification System", Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S. Army. Vol. 1 March 1953.

2 "Canadian Foundation Engineering Manual", 3rd Edition, Canadian Geotechnical Society, 1992.

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISIONS			SYMBOLS: USCS	SYMBOLS: GRAPH	SYMBOLS: COLOUR	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75um)	GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm	CLEAN GRAVELS (TRACE OR NO FINES)	GW		RED	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = D_{60}/D_{10} > 4$; $C_c = (D_{30})^2/(D_{10} x D_{60})$ = 1 to 3
			GP		RED	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY GRAVELS (WITH SOME OR MORE FINES)	GM		YELLOW	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4
			GC		YELLOW	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7
	SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm	CLEAN SANDS (TRACE OR NO FINES)	SW		RED	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = D_{60}/D_{10} > 6$; $C_c = (D_{30})^2/(D_{10} x D_{60})$ = 1 to 3
			SP		RED	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (WITH SOME OR MORE FINES)	SM		YELLOW	SILTY SANDS, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4
			SC		YELLOW	CLAYEY SANDS, SAND-CLAY MIXTURES	ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75um)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML		GREEN	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)
		$W_L > 50\%$	MH		BLUE	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SAND OR SILTY SOILS	
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL		GREEN	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS	
		$30\% < W_L < 50\%$	CI		GREEN-BLUE	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	
		$W_L > 50\%$	CH		BLUE	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$W_L < 50\%$	OL		GREEN	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY
		$W_L > 50\%$	OH		BLUE	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGHLY ORGANIC SOILS			PT		ORANGE	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE

SPECIAL SYMBOLS

LIMESTONE		OILSAND	
SANDSTONE		SHALE	
SILTSTONE		FILL (UNDIFFERENTIATED)	

SOIL COMPONENTS

FRACTION	U.S. STANDARD METRIC SIEVE SIZE		DEFINING RANGES OF PERCENT BY WEIGHT OF MINOR COMPONENTS	
GRAVEL	PASSING	RETAINED	PERCENT	DESCRIPTOR
COARSE	76mm	19mm	35 - 50	AND
FINE	19mm	4.75mm		
SAND			30 - 35	Y / EY
COARSE	4.75mm	2.00mm		
MEDIUM	2.00mm	425µm	10 - 20	SOME
FINE	425µm	75µm		
FINES (SILT OR CLAY BASED ON PLASTICITY)	75µm		1 - 10	TRACE

OVERSIZED MATERIAL

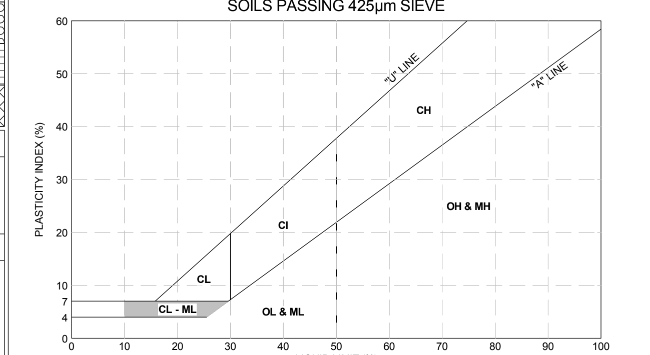
ROUNDED OR SUBROUNDED:

COBBLES 76mm to 200mm
BOULDERS > 200mm

NOT ROUNDED:

ROCK FRAGMENTS ? 76mm
ROCKS > 0.76 CUBIC METRE IN VOLUME

PLASTICITY CHART FOR SOILS PASSING 425µm SIEVE



NOTES:

1. ALL SIEVE SIZES MENTIONED ARE U.S. STANDARD ASTM E.11.
2. COARSE GRAINED SOILS WITH TRACE TO SOME FINES GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH TRACE TO SOME CLAY.
3. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

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