APPENDIX 'A'

GEOTECHNICAL REPORT

Quality Engineering | Valued Relationships

Stantec Consulting Ltd.

Pembina Highway Overpass at Abinojii Mikanah Rehabilitation RFP 975-2024

Prepared for:
James Kennedy, P.Eng.
Stantec Consulting Ltd.
500-311 Portage Ave
Winnipeg, MB
R3B 2B9

Project Number: 1000-240-02

Date: February 19, 2025

Quality Engineering | Valued Relationships

February 19, 2025

Our File No. 1000-240-02

James Kennedy, P.Eng.
Stantec Consulting Ltd.
500-311 Portage Ave
Winnipeg, MB
R3B 2B9

RE: Pembina Highway Overpass at Abinojii Mikanah Rehabilitation RFP 975-2024

TREK Geotechnical Inc. is pleased to submit our Final Report for the road investigation for Pembina Highway Overpass at Abinojii Mikanah Rehabilitation.

Please contact the undersigned should you have any questions.

Sincerely,

TREK Geotechnical Inc.
Per:

Nelson John Ferreira, Ph.D., P.Eng.
Senior Geotechnical Engineer

Encl.

Revision History

Revision No.	Author	Issue Date	Description
1	TG	February 19, 2025	Final Report

Authorization Signatures

Prepared By:
Tyler Green
Intermediate Technician

PROVINCE OF MANITOBA
N. J. FERREIRA
Member
22892
REGISTERED PROFESSIONAL ENGINEER

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
TREK GEOTECHNICAL INC.
No. 4877

Reviewed By:
Nelson John Ferreira, Ph.D., P.Eng.
Senior Geotechnical Engineer

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1.0 Introduction

This report summarizes the results of the road investigation completed for the Pembina Highway Overpass at Abinojii Mikanah Rehabilitation and Related Works. The investigation was carried out along Pembina Highway and Pembina Highway Overpass ramps. Information collected describes the asphalt and concrete pavement structure. The investigation was carried out in accordance with the City of Winnipeg RFP No. 975-2024.

2.0 Road Investigation

The investigation included coring of pavement at 29 locations on Pembina Highway between Plaza Drive and Chancellor Drive. The investigation locations are shown on Figures 01 to 02 (attached) and the table below summarizes the investigation program per street.

Table 1 – Road Investigation Program

Pembina Highway Overpass at Abinojii Mikanah Rehabilitation	# of Locations	Investigation
Pembina Highway & Overpass Ramps – Plaza Drive to Chancellor Drive	29	29 Cores – 15 Compressive Strength

The road investigation was conducted between January 27, 2025 to January 31, 2025. The pavement structure (asphalt/concrete) was cored by Tyler Green of TREK Geotechnical Inc. (TREK) using a portable coring press equipped with a hollow 150mm diameter diamond core drill bits. Core samples were also retrieved and logged at TREK's material testing laboratory. A summary table of the concrete pavement cores, compressive strength of pavement cores and photographs of the cores are included in Appendix A.

Core logs noted on the summary tables are based on UTM coordinates obtained using a hand-held GPS, their location relative to the nearest address or intersection and measured distance from the edge of pavement, or other permanent features.

Fifteen concrete cores were selected for concrete compressive strength breaks and the length to diameter ratio was between 1.21 and 1.61 for all cores collected. The core compressive strength tests were tested in accordance with CSA A23.2-14C – wet condition. The measured compressive strengths were also corrected based on an adapted ACI 214.4R-03 Standard to estimate the in-place concrete strengths. The table below summarizes the compressive strength results while the compressive strength testing details and the correction factor methodology are included in Appendix A.

Table 1: Concrete Core Compressive Strength Results

Core ID	Uncorrected Compressive Strength (MPa)	Corrected Compressive Strength (MPa)
PC-02	72.00	80.06
PC-05	68.06	75.76
PC-07	75.65	82.05
PC-09	71.85	79.87
PC-11	51.52	56.97
PC-14	68.78	76.64
PC-15	64.47	71.82
PC-17	59.93	66.26
PC-18	61.74	68.73
PC-19	45.39	49.45
PC-20	61.17	67.68
PC-23	47.10	52.10
PC-24	65.48	73.14
PC-26	45.41	48.38
PC-28	69.41	83.98

3.0 Closure

The information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings of this report were based on information provided (field investigation).

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work, or a mutually executed standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

This report has been prepared by TREK Geotechnical Inc. (the Consultant) for the exclusive use of Stantec Consulting Ltd. (the Client) and their agents for the work product presented in the report. Any findings or recommendations provided in this report are not to be used or relied upon by any third parties, except as agreed to in writing by the Client and Consultant prior to use.

Figures

Z:\Projects\1000 Soils Lab\1000 Lab Projects\1000-240 Stantec\1000-240-02 975-2024 Pembina Overpass Coring\3 Survey and Dwg\3.4 CAD\3.4.3 Working Folder\Figs 01 & 02 2025-02-13 Pembina Overpass 0_A 1000-240-02.dwg, 2025-02-13 6:54:03 AM

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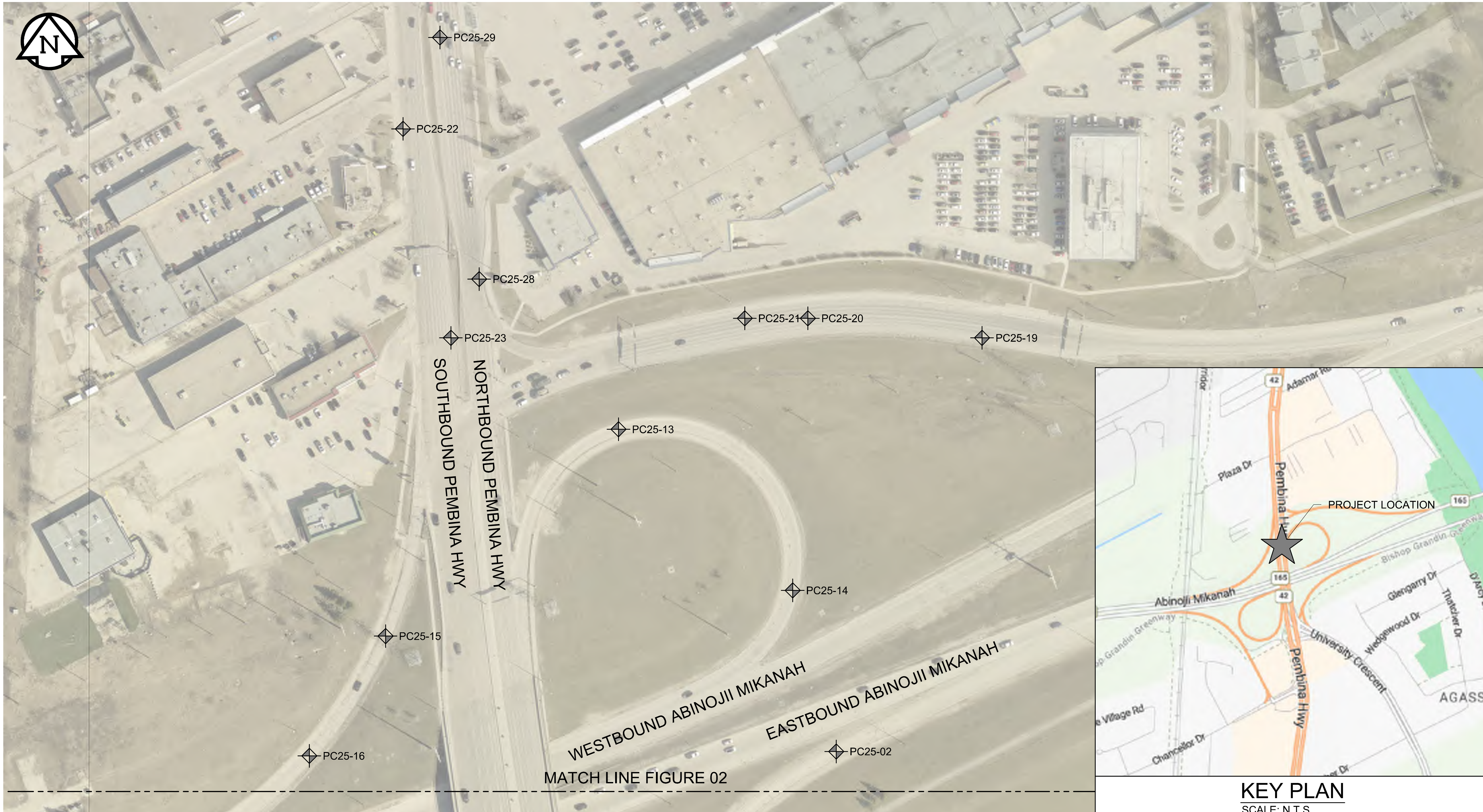
Stantec Consulting Ltd.
975-2024 Pembina Overpass Coring



0 25 50 75 m
SCALE = 1 : 1 750 (279 mm x 432 mm)

LEGEND: PAVEMENT CORE (TREK, 2025)

NOTES:
1. AERIAL IMAGERY FROM CITY OF WINNIPEG (2024).

Figure 01
Pavement Core Location Plan

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TREK GEOTECHNICAL

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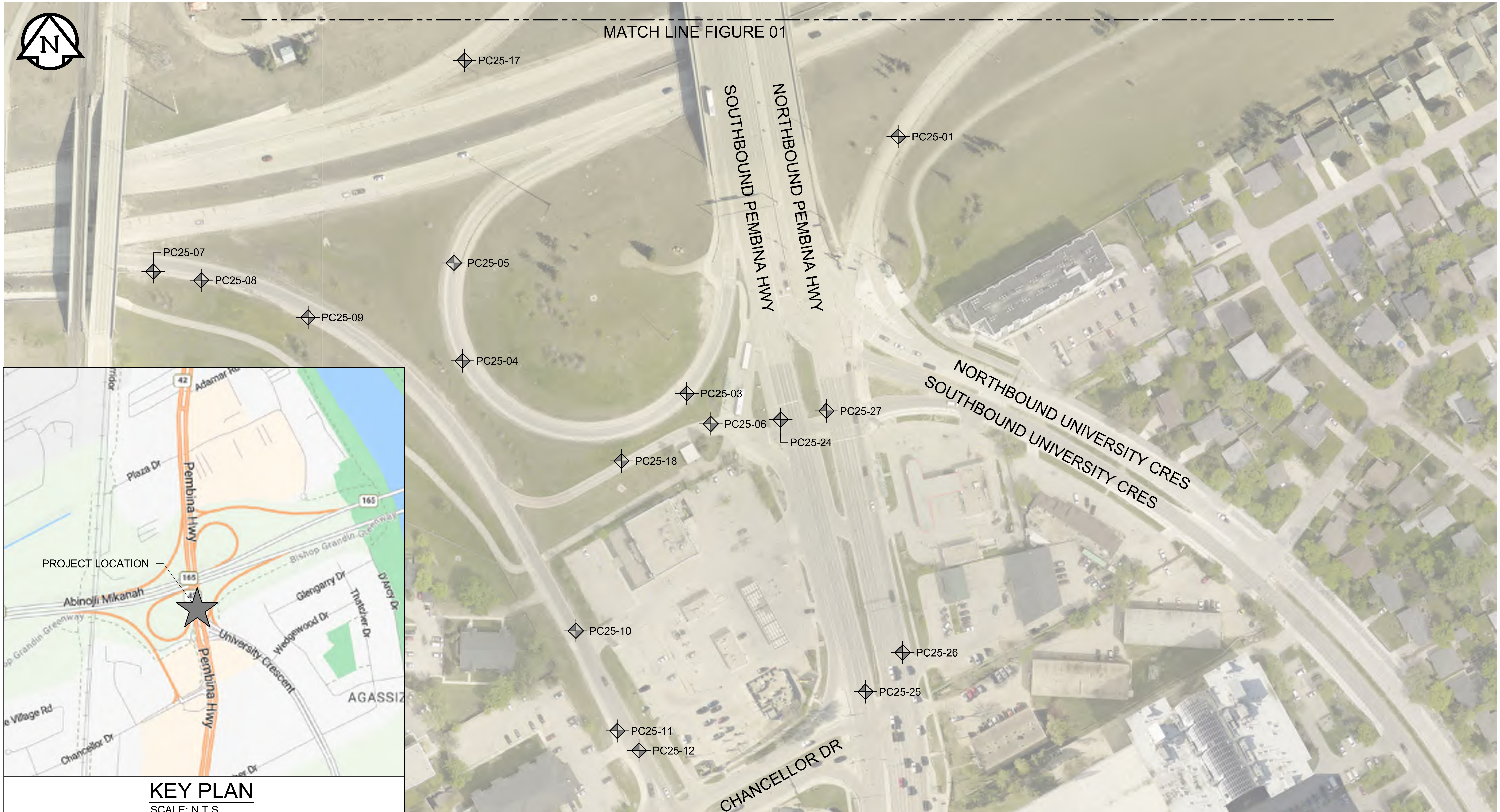
Stantec Consulting Ltd.
975-2024 Pembina Overpass Coring



KEY PLAN
SCALE: N.T.S.

0 25 50 75 m
SCALE = 1 : 1 750 (279 mm x 432 mm)

LEGEND: PAVEMENT CORE (TREK, 2025)

NOTES: 1. AERIAL IMAGERY FROM CITY OF WINNIPEG (2024).

Figure 02
Pavement Core Location Plan

Appendix A

Summary Table, Core Compressive Strength and Pavement Core Photos

Pembina Highway Overpass – Plaza Drive to Chancellor Drive

Pembina Highway Overpass at Abinojii Mikanah Rehabilitation and Related Works

975-2024 Pembina Overpass Coring

Pavement Core No.	Pavement Core Location	Pavement Surface		Pavement Structure Material		
		Type	Thickness (mm)	Type	Thickness (mm)	Corrected Compressive Strength (Mpa)
PC25-01	UTM : 5520143 m N, 632973 m E; Located on Pembina Overpass, Northbound Pembina Highway to Eastbound Abinojii Mikanah, 1.9 m North of South curb (Joint, Core #20)	Asphalt	0	Concrete	235	
PC25-02	UTM : 5520215 m N, 633052 m E; Located on Pembina Overpass, Northbound Pembina Highway to Eastbound Abinojii Mikanah, 2.9 m North of South curb (Compressive Strength, Core #21)	Asphalt	0	Concrete	235	80.06
PC25-03	UTM : 5520025 m N, 632876 m E; Located on Pembina Overpass, Southbound Pembina Highway to Eastbound Abinojii Mikanah, 1.5 m South of North (Joint, Core #09)	Asphalt	90	Concrete	200	
PC25-04	UTM : 5520040 m N, 632773 m E; Located on Pembina Overpass, Southbound Pembina Highway to Eastbound Abinojii Mikanah, 2.0 m South of North (Joint, Core #10)	Asphalt	75	Concrete	240	
PC25-05	UTM : 5520085 m N, 632769 m E; Located on Pembina Overpass, Southbound Pembina Highway to Eastbound Abinojii Mikanah, 2.3 m South of North (Compressive Strength, Core #11)	Asphalt	85	Concrete	225	75.76
PC25-06	UTM : 5520011 m N, 632887 m E; Located on Pembina Overpass, Bus Access Road, 1.4 m North of South curb (Joint, Core #07)	Asphalt	0	Concrete	230	
PC25-07	UTM : 5520081 m N, 632769 m E; Located on Pembina Overpass, Eastbound Abinojii Mikanah to Chancellor Drive, 3.4 m South of North shoulder (Compressive Strength, Core #06)	Asphalt	75	Concrete	200	82.05
PC25-08	UTM : 5520077 m N, 632653 m E; Located on Pembina Overpass, Eastbound Abinojii Mikanah to Chancellor Drive, 3.1 m South of North shoulder (Joint, Core #05)	Asphalt	75	Concrete	225	
PC25-09	UTM : 5520060 m N, 632702 m E; Located on Pembina Overpass, Eastbound Abinojii Mikanah to Chancellor Drive, 2.6 m South of North shoulder (Compressive Strength, Core #04)	Asphalt	35	Concrete	230	79.87
PC25-10	UTM : 5519916 m N, 632825 m E; Located on Pembina Overpass, Eastbound Abinojii Mikanah to Chancellor Drive, 2.6 m West of East curb (Joint, Core #03)	Asphalt	50	Concrete	200	
PC25-11	UTM : 5519870 m N, 632844 m E; Located on Pembina Overpass, Eastbound Abinojii Mikanah to Chancellor Drive, 2.0 m West of East curb (Compressive Strength, Core #02)	Asphalt	55	Concrete	225	56.97
PC25-12	UTM : 5519861 m N, 632854 m E; Located on Pembina Overpass, Eastbound Abinojii Mikanah to Chancellor Drive, 2.0 m East of West curb (Joint, Core #01)	Asphalt	110	Concrete	175	
PC25-13	UTM : 5520363 m N, 632952 m E; Located on Pembina Overpass, Northbound Pembina to Westbound Abinojii Mikanah, 2.5 m North of South curb (Joint, Core #18)	Asphalt	0	Concrete	200	
PC25-14	UTM : 5520289 m N, 633032 m E; Located on Pembina Overpass, Northbound Pembina Highway to Westbound Abinojii Mikanah, 2.1 m North of South curb (Compressive Strength, Core #19)	Asphalt	0	Concrete	230	76.64
PC25-15	UTM : 5520268 m N, 632845 m E; Located on Pembina Overpass, Southbound Pembina Highway to Westbound Abinojii Mikanah, 2.0 m West of East curb (Compressive Strength, Core #14)	Asphalt	40	Concrete	230	71.82

Pembina Highway Overpass at Abinojii Mikanah Rehabilitation and Related Works

975-2024 Pembina Overpass Coring

Pavement Core No.	Pavement Core Location	Pavement Surface		Pavement Structure Material		
		Type	Thickness (mm)	Type	Thickness (mm)	Corrected Compressive Strength (Mpa)
PC25-16	UTM : 5520213 m N, 632810 m E; Located on Pembina Overpass, Southbound Pembina Highway to Westbound Abinojii Mikanah, 1.8 m West of East curb (Joint, Core #13)	Asphalt	40	Concrete	260	
PC25-17	UTM : 5520178 m N, 632774 m E; Located on Pembina Overpass, Southbound Pembina Highway to Westbound Abinojii Mikanah, 2.0 m West of East curb (Compressive Strength, Core #12)	Asphalt	25	Concrete	225	66.26
PC25-18	UTM : 5519994 m N, 632846 m E; Located on Pembina Overpass, Bus Access Road, 2.0 m South of North curb (Joint, Core #08)	Asphalt	0	Concrete	230	68.73
PC25-19	UTM : 5519861 m N, 632854 m E; Located on Pembina Overpass, Westbound Abinojii Mikanah to Pembina Highway, 2.3 m North of South curb (Compressive Strength, Core #17)	Asphalt	90	Concrete	210	49.45
PC25-20	UTM : 5520414 m N, 633039 m E; Located on Pembina Overpass, Westbound Abinojii Mikanah to Pembina Highway, 4.5 m South of North curb (Compressive Strength, Core #16)	Asphalt	90	Concrete	210	67.68
PC25-21	UTM : 5520414 m N, 633010 m E; Located on Pembina Overpass, Westbound Abinojii Mikanah to Pembina Highway, 1.7 m North of South curb (Joint, Core #17)	Asphalt	40	Concrete	220	
PC25-22	UTM : 5520501 m N, 632853 m E; Located at #1875 Pembina Highway, Southbound Curb Lane, 1.9 m East of West curb (Joint, Core #29)	Asphalt	70	Concrete	210	
PC25-23	UTM : 5520405 m N, 632875 m E; Located at #1921 Pembina Highway, Southbound Median Lane, 2.0 m West of East curb (Compressive Strength, Core #28)	Asphalt	65	Concrete	225	52.10
PC25-24	UTM : 5520013 m N, 632919 m E; Located at Pembina Highway Bus Stop #60138, Southbound Curb Lane, 2.0 m East of West curb (Compressive Strength, Core #27)	Asphalt	115	Concrete	240	73.14
PC25-25	UTM : 5519888 m N, 632958 m E; Located at #2027 Pembina Highway, Southbound Median Lane, 1.7 m West of East curb (Joint, Core #26)	Asphalt	110	Concrete	200	
PC25-26	UTM : 5519906 m N, 632975 m E; Located at #2028 Pembina Highway, Northbound Curb Lane, 2.0 m East of West curb (Compressive Strength, Core #22)	Asphalt	70	Concrete	190	48.38
PC25-27	UTM : 5520017 m N, 632940 m E; Located at Pembina Highway 10 m North of bus access road, Northbound Median Lane, 2.0 m East of West curb (Joint, Core #23)	Asphalt	180	Concrete	190	
PC25-28	UTM : 5520432 m N, 632888 m E; Located at #1890 Pembina Highway, Northbound Curb Lane, 1.4 m West of East curb (Compressive Strength, Core #24)	Asphalt	170	Concrete	190	83.98
PC25-29	UTM : 5520543 m N, 632870 m E; Located at Pembina Highway 20 m South of Plaza Drive, Northbound Median Lane, 4.0 m East of West curb (Joint, Core #22)	Asphalt	90	Concrete	200	

Photo 1: Pavement Core Sample at PC25-01

Photo 2: Pavement Core Sample at PC25-02

Photo 3: Pavement Core Sample at PC25-03

Photo 4: Pavement Core Sample at PC25-04

Photo 5: Pavement Core Sample at PC25-05

Photo 6: Pavement Core Sample at PC25-06

Photo 7: Pavement Core Sample at PC25-07

Photo 8: Pavement Core Sample at PC25-08

Photo 9: Pavement Core Sample at PC25-09

Photo 10: Pavement Core Sample at PC25-10

Photo 11: Pavement Core Sample at PC25-11

Photo 12: Pavement Core Sample at PC25-12

Photo 13: Pavement Core Sample at PC25-13

Photo 14: Pavement Core Sample at PC25-14

Photo 15: Pavement Core Sample at PC25-15

Photo 16: Pavement Core Sample at PC25-16

Photo 17: Pavement Core Sample at PC25-17

Photo 18: Pavement Core Sample at PC25-18

Photo 19: Pavement Core Sample at PC25-19

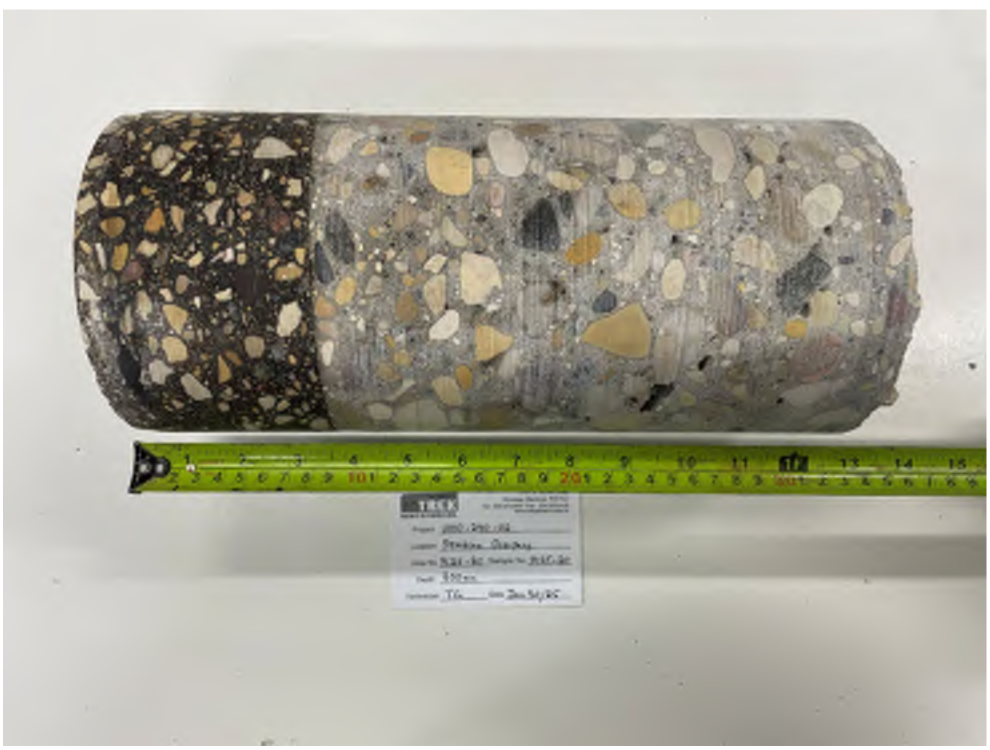


Photo 20: Pavement Core Sample at PC25-20

Photo 21: Pavement Core Sample at PC25-21

Photo 22: Pavement Core Sample at PC25-22

Photo 23: Pavement Core Sample at PC25-23

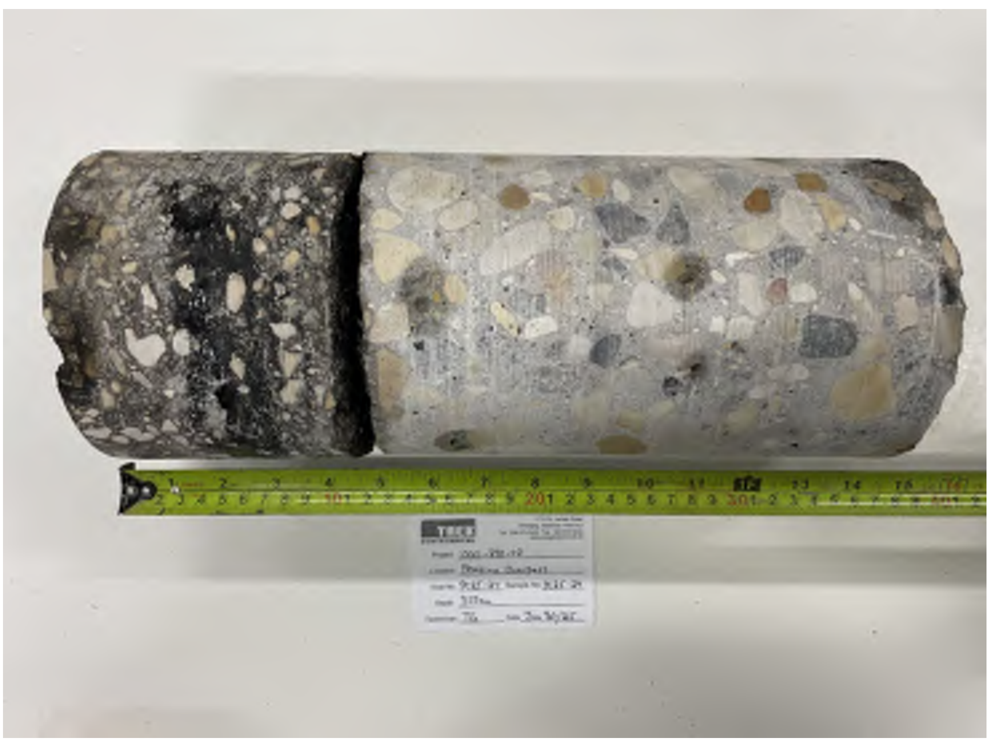


Photo 24: Pavement Core Sample at PC25-24

Photo 25: Pavement Core Sample at PC25-25

Photo 26: Pavement Core Sample at PC25-26

Photo 27: Pavement Core Sample at PC25-27

Photo 28: Pavement Core Sample at PC25-28

Photo 29: Pavement Core Sample at PC25-29

www.trekgeotechnical.ca
1712 St. James Street
Winnipeg, MB R3H 0L3
Tel: 204.975.9433 Fax: 204.975.9435

Concrete Core Compressive Strength Report

CSA A23.2-14C

Project No. 1000-240-02 **Date** February 7, 2025

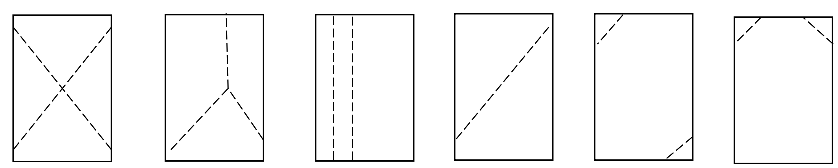
Project 975-2024 Pembina Overpass **Technician** T. Green

Client Stantec Consulting Ltd.

Core Location	Core ID	Date Received	Date of Break	Age at Break	Diam. (mm)	Length (mm)	Moisture Conditioning	Compressive Strength (MPa)		Break Type	Correction Factors*				
								Uncorrected f_{conc}	Corrected* f_c		$F_{l/d}$	F_{dia}	F_{mc}	F_D	F_{reinf}
UTM : 5520215 m N, 633052 m E: (Core 21)	PC-02	2025-01-27	2025-02-07	-	143	220	Soaked 48 h	72.00	80.06	1	0.9817	0.9804	1.0900	1.0600	1.0000
UTM : 5520085 m N, 632769 m E: (Core 11)	PC-05	2025-01-27	2025-02-07	-	144	224	Soaked 48 h	68.06	75.76	1	0.9827	0.9803	1.0900	1.0600	1.0000
UTM : 5520081 m N, 632769 m E: (Core 06)	PC-07	2025-01-28	2025-02-07	-	144	186	Soaked 48 h	75.65	82.05	1	0.9576	0.9803	1.0900	1.0600	1.0000
UTM : 5520060 m N, 632702 m E: (Core 04)	PC-09	2025-01-28	2025-02-07	-	144	221	Soaked 48 h	71.85	79.87	1	0.9814	0.9803	1.0900	1.0600	1.0000
UTM : 5519870 m N, 632844 m E: (Core 02)	PC-11	2025-01-28	2025-02-07	-	144	216	Soaked 48 h	51.52	56.97	1	0.9763	0.9803	1.0900	1.0600	1.0000
UTM : 5520289 m N, 633032 m E: (Core 19)	PC-14	2025-01-29	2025-02-07	-	144	226	Soaked 48 h	68.78	76.64	1	0.9838	0.9803	1.0900	1.0600	1.0000
UTM : 5520268 m N, 632845 m E: (Core 14)	PC-15	2025-01-29	2025-02-07	-	145	228	Soaked 48 h	64.47	71.82	1	0.9837	0.9802	1.0900	1.0600	1.0000
UTM : 5520178 m N, 632774 m E: (Core 12)	PC-17	2025-01-29	2025-02-07	-	145	216	Soaked 48 h	59.93	66.26	1	0.9762	0.9802	1.0900	1.0600	1.0000
UTM : 5519994 m N, 632846 m E: (Core 08)	PC-18	2025-01-29	2025-02-07	-	145	227	Soaked 48 h	61.74	68.73	1	0.9829	0.9802	1.0900	1.0600	1.0000
UTM : 5519861 m N, 632854 m E: (Core 17)	PC-19	2025-01-30	2025-02-07	-	144	198	Soaked 48 h	45.39	49.45	1	0.9619	0.9803	1.0900	1.0600	1.0000
UTM : 5520414 m N, 633039 m E: (Core 16)	PC-20	2025-01-30	2025-02-07	-	145	217	Soaked 48 h	61.17	67.68	1	0.9770	0.9802	1.0900	1.0600	1.0000
UTM : 5520405 m N, 632875 m E: (Core 28)	PC-23	2025-01-30	2025-02-07	-	145	219	Soaked 48 h	47.10	52.10	1	0.9768	0.9802	1.0900	1.0600	1.0000
UTM : 5520013 m N, 632919 m E: (Core 27)	PC-24	2025-01-30	2025-02-07	-	145	233	Soaked 48 h	65.48	73.14	1	0.9863	0.9802	1.0900	1.0600	1.0000
UTM : 5519906 m N, 632975 m E: (Core 22)	PC-26	2025-01-31	2025-02-07	-	145	177	Soaked 48 h	45.41	48.38	1	0.9408	0.9802	1.0900	1.0600	1.0000
UTM : 5520432 m N, 632888 m E: (Core 23)	PC-28	2025-01-31	2025-02-07	-	144	174	Soaked 48 h	69.41	83.98	1	0.9454	0.9803	1.0900	1.0600	1.1300

Comments

*Correction factors $F_{l/d}$, F_{dia}, F_{mc}, and F_D calculated as per ACI 214.4R-03, and correction factor F_{reinf} calculated as per Khoury et al. (2014): $f_c = f_{conc}F_{l/d}F_{dia}F_{mc}F_DF_{reinf}$



Type 1 Type 2 Type 3 Type 4 Type 5 Type 6

Reviewed by (print): Angela Fidler-Kliewer, C.Tech. Signature: Angela Fidler-Kliewer

Geotechnical Investigation and Foundation Report

Pembina Highway Overpass at Abinojii Mikanah (Bishop Grandin) Rehabilitation and Related Works – Parks Building Relocation

February 21, 2025

Prepared for:

City of Winnipeg

Prepared by:

Stantec Consulting Ltd.

Project/File:

132500075

Disclaimer

The conclusions in the Report titled Geotechnical Investigation and Foundation Report are Stantec's professional opinion, as of the time of the Report, and concerning the scope described in the Report. The opinions in the document are based on conditions and information existing at the time the scope of work was conducted and do not take into account any subsequent changes. The Report relates solely to the specific project for which Stantec was retained and the stated purpose for which the Report was prepared. The Report is not to be used or relied on for any variation or extension of the project, or for any other project or purpose, and any unauthorized use or reliance is at the recipient's own risk.

Stantec has assumed all information received from City of Winnipeg (the "Client") and third parties in the preparation of the Report to be correct. While Stantec has exercised a customary level of judgment or due diligence in the use of such information, Stantec assumes no responsibility for the consequences of any error or omission contained therein.

This Report is intended solely for use by the Client in accordance with Stantec's contract with the Client. While the Report may be provided by the Client to applicable authorities having jurisdiction and to other third parties in connection with the project, Stantec disclaims any legal duty based upon warranty, reliance or any other theory to any third party, and will not be liable to such third party for any damages or losses of any kind that may result.

Prepared by J. Sears
Signature

Jack Sears, E.I.T.
Printed Name

Reviewed by Kevin Baylis
Signature

Kevin Baylis, M.Eng., P.Eng.
Printed Name

Reviewed by Aron Piamsalee
Signature

Aron Piamsalee, M.Sc., P.Eng.
Printed Name

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
Stantec Consulting Ltd.
No. 1301

PROVINCE OF MANITOBA
2025-02-21
K.D.
BAYLIS
Member
33363
REGISTERED PROFESSIONAL ENGINEER

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List of Appendices

1 Introduction

The City of Winnipeg has retained Stantec Consulting Ltd. (Stantec) to provide engineering services for the proposed new (relocated) Parks building to be located at the existing Abinojii Mikanah bus turnoff near 1995 Pembina Hwy in Winnipeg, Manitoba. These services are part of a larger scope of work for the Pembina Highway Overpass at Abinojii Mikanah (Bishop Grandin) Rehabilitation and Related Works. As part of the overall project and as requested by our client, a geotechnical site investigation was performed to support the design for the proposed building. Use of this report is subject to the Statement of General Conditions provided in **Appendix A**.

The purpose of this report is to outline the geotechnical engineering services provided for the proposed development. The work that has been performed as part of the geotechnical scope of work has included the following:

- Review available existing geotechnical background information.
- Arrange for public and private utility locates at the subject site prior to the site investigation.
- Complete a geotechnical drilling program consisting of drilling one borehole, soil sampling, and laboratory testing to identify the existing soil and groundwater conditions at the site.
- Prepare a geotechnical report (this report) including:
  - A general project and site description;
  - A site plan with borehole locations;
  - Borehole records with information on stratigraphic and groundwater conditions;
  - Results of field investigation and laboratory analysis;
  - Foundation recommendations including foundation types, frost protection requirements with limit states design (LSD) parameters and inspection requirements for the proposed building;
  - Drainage recommendations;
  - Design review, construction monitoring and testing requirements; and,
  - Site soil classification for seismic response as per the Manitoba Building Code which has adopted the 2020 National Building Code of Canada.

2 Proposed Development

The Pembina Highway Overpass at Abinojii Mikanah (Bishop Grandin) Rehabilitation and Related Works project includes construction of a new roadway through the footprint of an existing Parks building. The existing Parks building is located at 1995 Pembina Highway and consists of an unheated single storey storage facility without a basement, having an approximate footprint of 70 m^2 (approx. 750 ft^2).

It is our understanding that the existing Parks building will be relocated approximately 60 m west of its existing location, to the area identified as "Option 2" on Sketch 01 provided in **Appendix B**.

3 Investigation Program

The investigation program for this project consisted of utility clearances, a borehole drilling and sampling program, and a laboratory testing program which are outlined in the following subsections.

3.1 Utility Clearances

Prior to completing the geotechnical drilling and sampling program, Stantec coordinated with Click Before You Dig Manitoba to locate the public utility services as well as Structure Scan Inc. to locate the private utility services at the project site. The City of Winnipeg Water and Waste Engineering department required Stantec to expose a 750 mm Fort Gary & St.Vital feedermain using “soft digging” techniques prior to drilling. Badger Daylighting Inc. exposed the feedermain using hydrovac equipment on January 21, 2025 under the supervision of Stantec personnel. The hydrovac holes were backfilled with clean sand.

3.2 Drilling and Sampling Program

A geotechnical drilling and sampling program was performed on January 24, 2025, with drilling services provided by Maple Leaf Drilling Ltd. under continuous Stantec geotechnical personnel supervision.

The drilling was performed using a track mounted drill rig. One borehole, identified as BH25-01, was drilled to power auger refusal in the vicinity of the proposed building, with auger refusal encountered at a depth of 13.4 m. The approximate borehole location is shown in plan view on Sketch 01 provided in **Appendix B**.

The drilling program consisted of advancing 125 mm diameter solid stem augers through overburden materials to the depths explored. Overburden soil samples were retrieved directly from the auger flights at approximate 0.75 m to 1.5 m intervals and at locations of material changes. One undisturbed soil sample was also obtained with a 75 mm diameter Shelby tube. Field pocket torvane readings were taken to estimate the undrained shear strength of the cohesive soils. All samples were visually inspected in the field for material types and transferred to our Winnipeg laboratory for further inspection and testing. Upon completion of drilling, the groundwater and soil sloughing conditions were recorded in the borehole prior to backfilling with soil cuttings and bentonite chips.

A description of the soil stratigraphy at the borehole location is as provided within **Section 4.1** of this report as well as on the detailed borehole log included in **Appendix C**.

3.3 Laboratory Testing

A laboratory testing program was performed on select soil samples from the drilling program to evaluate the relevant engineering properties of the subsurface materials relative to the development of geotechnical recommendations. Index testing included moisture contents on all collected soil samples (ASTM D2216), as well as particle size analyses (ASTM D7928) and Atterberg limits testing (ASTM D4318) on select representative samples. The results of the laboratory testing are presented in **Section 4.2**, and shown on the detailed borehole records within **Appendix C** where applicable. The individual laboratory testing results sheets are provided in **Appendix D**.

4 Investigation Results

The subsurface conditions were based on the investigation results obtained during the field investigation and laboratory testing program. The pertinent results from these programs are summarized in the following sections.

4.1 Stratigraphy

In general, the stratigraphy of the borehole drilled at the site consisted of a surficial layer of topsoil, underlain by fat clay and silt till to the depth explored in the borehole. A description of the soil stratigraphy is summarized below and on the detailed borehole records included in **Appendix C**. Also included in **Appendix C** are summary sheets outlining the symbols and terms used on the borehole records.

4.1.1 Topsoil

Topsoil was encountered at the surface and consists of black clay with trace silt. The thickness of the topsoil was approximately 300 mm. One sample had a moisture content of 28%.

4.1.2 Fat Clay

A fat clay layer was encountered below the topsoil and extended to a depth of approximately 13.1 m. The fat clay was brown in colour with trace silt, becoming grey below a depth of 2.1 m. The moisture content of the fat clay ranged from 33% to 57%. From handheld torvane testing performed in the field, the undrained shear strength of the fat clay ranged between 10 kPa and 88 kPa, classifying the material as very soft to stiff in consistency. From Atterberg Limits testing completed on a sample taken at a depth of 4.3 m, the clay had a Liquid Limit of 96 and Plasticity Index of 68, classifying the clay as fat (i.e. of high plasticity). Based upon the laboratory results, the activity of the fat clay is 0.96, classifying the minerology of the material to be predominately illite.

4.1.3 Silt Till

A layer of silt till was encountered below the fat clay. The silt till extended to the depths explored in the borehole, corresponding to the depth of power auger refusal at 13.4 m. The silt till was grey in colour, sandy, and contained trace clay and trace gravel.

The moisture content of the silt till on one sample taken was 10%. Standard Penetration Tests were not conducted due to soil sloughing observed at 8.5 m.

4.2 Laboratory Test Results

Index testing included moisture contents on all collected soil samples (ASTM D2216), as well as particle size analyses (ASTM D7928) and Atterberg limits testing (ASTM D4318) on a select representative sample. The particle size analysis and Atterberg limits testing results are shown in **Table 1** and **Table 2,** respectively. Moisture content, particle size analyses and Atterberg limits tests results are also shown on the borehole records included within **Appendix C**. The laboratory summary sheets for all testing performed have been included within **Appendix D**.

Table 1. Particle Size Analysis Results

Borehole ID	Sample Depth (m)	Soil Type	Particle Size			
			Gravel (%) 75 to 4.75 mm	Sand (%) < 4.75 to 0.075 mm	Silt (%) < 0.075 to 0.002 mm	Clay (%) < 0.002 mm
BH25-01	4.3	Fat Clay (CH)	0.0	0.6	29.9	69.5

Table 2. Atterberg Limits Test Results

Borehole ID	Sample Depth (m)	Soil Type	Liquid Limit	Plastic Limit	Plasticity Index	Activity
BH25-01	4.3	Fat Clay (CH)	96	28	68	0.96

4.3 Groundwater and Sloughing Conditions

Groundwater and soil sloughing conditions were recorded upon completion of the drilling as shown in **Table 3**.

Table 3. Observed Groundwater and Sloughing Conditions

Borehole No.	Observed Depth of Groundwater Seepage (m)	Depth of Groundwater Upon Completion of Drilling (m)	Observed Depth of Soil Sloughing (m)
BH25-01	9.1	dry	8.5

It should be noted that only short-term seepage and sloughing conditions were checked for in the open borehole and that groundwater levels can fluctuate during the year and can be dependent on precipitation, drainage, and local/regional groundwater regimes.

5 Geotechnical Recommendations

5.1 Frost Considerations

5.1.1 Frost Penetration Depth

The depths of frost penetration have been estimated for a range of annual air freezing indices identified in **Table 4** below. The mean annual freezing index is based on published climate normal from Environment Canada between 1991 and 2020 for the Winnipeg, Manitoba area (Winnipeg Richardson Airport). The ten-year return annual freezing index was calculated using the mean value and recommendations outlined in the Canadian Foundation Engineering Manual 5th Edition (CFEM).

Table 4. Estimated Frost Penetration Depth

Parameter	Period	
	Mean	10-Year Return
Annual Air Freezing Index (°C-Days)	1,725	2,325
Estimated Frost Penetration – Concrete pavement (m) (n=0.85)	1.7	1.9
Estimated Frost Penetration – Snow cover (m) (n=1.0)	1.8	2.1

For foundation design considerations, the CFEM recommends using the ten-year return annual freezing index to predict frost penetration. A potential frost penetration depth of approximately 2.1 m should be assumed for design considerations.

The United States Army Corp of Engineers (USACE) frost design soil classification system is a widely used system that places soils in one of four categories. The categories (or Frost Groups), from F1 to F4, reflect an increase in frost susceptibility and decrease in strength during thaw. The fat clay material would have a frost group rating of F4, and the frost susceptibility classification may be considered very high.

5.1.2 Adfreeze

Frozen soil in contact with unheated foundation elements can develop an adfreeze bond which can result in uplift forces on the foundations. The CFEM recommends adfreeze bond stresses for fine grained soils as follows:

- 65 kPa for fine-grained soils frozen to wood or concrete; and
- 100 kPa for fine-grained soils frozen to steel.

As a conservative estimate, this adfreeze stress should be applied to the perimeter of a pile or any other foundation element to a depth of 2.1 m below final grade.

The uplift forces from adfreeze and frost heave stresses are resisted by the permanent dead load of the structure plus the uplift resistance of the foundation elements below the frost penetration. Piles in unheated areas should contain full length reinforcing steel to resist the tensile forces related to frost jacking.

5.2 Foundation Design

Based on the soil conditions encountered at the borehole locations and our understanding of the proposed development, the proposed building may be supported on cast-in-place (CIP) concrete friction piles or a thickened edge slab. The recommended foundation type will depend on the City of Winnipeg's tolerance for movements in the structure.

5.2.1 Limit States Design

In accordance with the 2020 National Building Code of Canada (NBCC), the use of Limit States Design (LSD) is required for the design of buildings and their structural components including foundations. The limit states of LSD are classified into two groups; the Ultimate Limit States (ULS) and the Serviceability Limit States (SLS).

The ULS case is primarily concerned with the collapse mechanisms for the structure and hence, safety. For foundation design, the ultimate limit state consists of:

- Exceeding the load-carrying capacity of the foundation;
- Sliding;
- Uplift;
- Large deformation of foundation leading to an ultimate limit state being induced in the superstructure of building;
- Overturning; and
- Loss of overall stability.

The factored resistance at the ULS is the ultimate geotechnical resistance multiplied by the appropriate resistance factor outlined in the NBCC. For LSD, the factored resistance must be greater than or equal to the factored applied load as per the following general equation:

$\Phi R_n \geq \sum_i \alpha_i S_{ni}$

Where,

Φ = geotechnical resistance factor at ULS
R_n = ultimate geotechnical resistance
α_i = load factor at ULS
S_{ni} = load effect at ULS

The SLS case considers mechanisms that restrict or constrain the intended use of the occupancy of the structure. These are typically associated with movements that interrupt or hinder the purpose of the structure. For foundation design, the serviceability limit state consists of:

- Excessive movements; and
- Unacceptable vibrations.

The SLS case is addressed by determining the maximum available resistance to keep the foundation under service loads within tolerable limits as provided by the structural engineer. Unfactored permanent and transitory loads are used for calculating total deformation in non-cohesive soils. Unfactored permanent loads and appropriate portions of transitory loads are used for the initial and time-dependent deformations of cohesive soils. Therefore, the foundation loads and serviceability tolerances must be known to properly determine the SLS resistance values. In cases where tolerable movements are not provided by the structural engineer, the tolerable limit of total settlement for foundations subject to compression is typically assumed to be 25 mm.

5.2.2 Cast-in-Place Concrete Friction Piles

Cast-in-place concrete friction piles may be used for the proposed structure. The undrained shear strength (s_u) and adhesion coefficient (α) values within the fat clay are outlined in **Table 5**. The cast-in-place concrete friction piles may be designed based on the shaft resistance values shown in **Table 6**. The values in **Table 6** are estimated by the shear strength profile and the alpha method, as outlined in the CFEM.

Table 5. Undrained Shear Strength and Adhesion Coefficient Parameters

Depth Range	Soil Unit	Undrained Shear Strength, s_u (kPa)	Adhesion Coefficient (α)
0 to 2.1 m	Frost/Active Zone	0	N/A
2.1 to 5.0 m	Firm to Stiff Clay	45	0.7
5.0 to 12.0 m	Soft Clay	15	1

Table 6. Geotechnical Shaft Resistance for Cast-In-Place Friction Piles

Depth Range	Factored Geotechnical Shaft Resistance in Axial Compression at ULS[1] (kPa)	Factored Geotechnical Shaft Resistance in Axial Tension at ULS[2] (kPa)	Factored Uplift Resistance to Frost Adfreeze Forces at ULS[3] (kPa)
0 to 2.1 m	0	0	0
2.1 to 5.0 m	12	9	30
5.0 to 12.0 m	6	4	15

Note:
1 As per the 2020 NBCC, a resistance factor of 0.4 was used to calculate the factored geotechnical shaft resistance in axial compression at ULS.
2 As per the 2020 NBCC, a resistance factor of 0.3 was used to calculate the factored geotechnical shaft resistance in axial tension at ULS.
3 As per the CFEM of this report, a resistance factor of 1.0 was used to calculate the factored uplift resistance to frost adfreeze forces at ULS.

For friction piles, less than 15 mm of settlement is required to mobilize shaft resistance, and therefore, the SLS case does not govern pile design.

The shaft resistance value is applied to the pile circumference within the clay stratum over the depth intervals indicated in **Table 5**. For structures built at existing grade, the frictional support in the upper 2.1 m should be excluded in the calculation of the pile capacity. The contribution from end bearing should be ignored in pile capacity calculations.

The total resistance to the seasonal frost adfreeze forces will be the total dead weight of the structure plus the applicable frost adfreeze uplift resistance per pile.

To avoid pile group effects, the minimum pile spacing should be three pile diameters measured centre to centre. If pile spacing is less than three pile diameters, additional analyses will be required to evaluate the settlement and capacity of the pile group. Settlement calculation for a pile group is based on the foundation load and the consolidation properties of the soil below the base of the piles. The capacity of a pile group is reduced as the pile spacing is decreased.

Groundwater and soil sloughing upon completion of drilling was observed at a depth of 9.1 m and 8.5 m respectively, during the geotechnical investigation. Seepage and soil sloughing may occur during pile installation. Temporary sleeves should be available during pile installation to control soil sloughing and groundwater seepage. Groundwater, if encountered in the pile holes, should be removed prior to concrete placement with the use of a pumping system. The pile holes should be inspected during installation and the concrete for the piles should be poured immediately after drilling to minimize any potential problems related to soil sloughing and groundwater seepage. It is recommended that pile lengths do not exceed 12.0 m below existing grade (measured at the location of borehole BH25-01) to reduce the risk of encountering till and the associated potential for significant groundwater seepage and soil sloughing from this till layer during pile installation.

A minimum void space of 150 mm should be provided beneath all structural elements to accommodate potential heave of the soil. Pile inspection by qualified Stantec geotechnical personnel should be provided during foundation construction to confirm that the piles are constructed in accordance with the project specifications.

5.2.3 Thickened-Edge Slab

A thickened-edge slab bearing on properly compacted granular fill may be used to support the proposed Parks building; however, will be subject to movements as outlined in this section.

5.2.3.1 Design

The concrete slab should be thickened (i.e. thickened-edge slab) and contain additional reinforcement along the perimeter walls and interior supports (if applicable). The minimum combined thickness of granular fill materials recommended beneath the thickened-edge slab (thickened and non-thickened portions) is 500 mm, consisting of 150 mm of Granular C Base Course material, overlying 350 mm of Granular C – 50 mm Sub-base. The materials should comply with the current City of Winnipeg Design and Construction Specifications CW 3110.

The thickened-edge slab may be designed based on the parameters provided in **Table 7**.

Table 7. Thickened-Edge Slab Design Parameters

Structure	Bearing Material	Factored Bearing Resistance ϕ = 0.5 (ULS)	Serviceability Limit Pressure ϕ = 1.0 (SLS)
New Parks Building	Fat Clay (CH)	100 kPa	80 kPa

The factored geotechnical resistance at ULS is based on a resistance factor of 0.5 and a minimum embedment depth of 0.3 m at the base of the slab. The SLS value is based on a tolerable total settlement of 25 mm and a resistance factor of 1.0.

Due to the presence of clay soils at the site, volume changes related to the moisture content changes in the subsoil may occur. The magnitude of foundation movement related to volume change in the subsoil depends on several factors and is difficult to predict but is estimated to be in the range of 80 to 120 mm. Differential movements in the thickened and non-thickened sections of the thickened edge slab are anticipated due to the difference in loading conditions and natural variability of clay soils. Therefore, concrete cracking at the interface between the thickened and non-thickened portion of the thickened-edge slab should be expected. If insulation is not included in the design, potential frost heave could also occur in addition to volume changes related to moisture content changes and could be in the range of up to 150 mm. If the estimated movements are unacceptable, the CIP concrete friction pile foundation type should be adopted. To minimize volume change of the clay, measures should be taken to prevent drying or wetting of the subgrade during construction.

5.2.3.2 Insulation

The use of rigid insulation can be used to reduce the risk of freezing and associated frost heave of the soils underlying the thickened-edge slab within the unheated structure. The rigid insulation should be placed directly beneath the entire floor area of the thickened-edge concrete slab and extend horizontally at least 2.44 m beyond the outside edge of the thickened-edge slab. The thickness of the insulation should be a minimum of 200 mm.

The following items related to rigid insulation should be considered during the structural design process:

- The use of high compressive strength rigid insulation is required to limit creep settlements within the insulation under the applied footing loads and should be specified by the structural engineer. The 25 mm settlement for the SLS resistances provided in **Table 7** do not account for any potential creep settlement within the rigid insulation.
- The structural engineer should consider the modulus of subgrade reaction (k) of the insulation to design the rigidity of the slab. For reference, literature reports k-values from about 7 MPa/m to 42 MPa/m. The structural engineer should consider specifying a minimum k-value for the rigid insulation.
- Consideration should be made to install a hydrocarbon resistant liner, to protect the rigid insulation from potential hydrocarbon leaks.

5.2.3.3 Construction Recommendations

Construction recommendations of the thickened-edge slab on fat clay subgrade will depend on whether insulation is included in the design or not. The construction recommendations for the two alternatives are described below.

Without Insulation:

- Remove topsoil and other materials to a depth of 150 mm below the underside of the thickened-edge slab, to the horizontal extents of the proposed slab.
- Proof roll the exposed subgrade to identify soft or weak areas at the subgrade level.
- Where soft or weak material is encountered at the subgrade level, it should be excavated and replaced with Granular C – 50 mm Sub-base and compacted to 100% of Standard Proctor maximum dry density (SPMDD).
- Place Granular C - Base Course material in maximum 150 mm lifts up to the underside of the proposed slab and compact to at least 100% of SPMDD.
- Place slab reinforcement and pour concrete in accordance with structural recommendations.

The subgrade must not be allowed to freeze during construction and there should be no frost present in the subgrade soils prior to concrete placement for the thickened-edge slab. It is recommended that inspection by qualified geotechnical personnel be conducted during construction to identify any soft or weak material at the subgrade level that should be removed and replaced.

With Insulation:

- Remove topsoil and other materials to a depth of 375 mm below the underside of the thickened-edge slab, to the horizontal extents of the proposed rigid insulation as described in **Section 5.2.3.2**.
- Proof roll the exposed subgrade to identify soft or weak areas at the subgrade level.
- Where soft or weak material is encountered at the subgrade level, it should be excavated and replaced with Granular C – 50 mm Sub-base compacted to 100% of Standard Proctor maximum dry density (SPMDD).
- Place Granular C – Base Course material in maximum 150 mm lifts up to 25 mm below the underside of the proposed rigid insulation and compact to at least 100% of SPMDD.
- Place a 25 mm thick layer of dry, clean levelling sand directly beneath the rigid insulation to provide a level surface.
- Place the rigid insulation on the leveling sand and in accordance with the manufacturer’s instructions. The portion of rigid insulation extending laterally beyond the thickened-edge slab should be placed at a 3% slope, to promote drainage away from the thickened-edge slab.
- The rigid insulation extending beyond the thickened-edge slab footprint may be backfilled with local fat clay fill in maximum 150 mm lifts to at least 95% of SPMDD and graded as per recommendations in **Section 6**. Care must be taken to avoid damaging rigid insulation during backfilling operations.
- Place slab reinforcement and pour concrete in accordance with structural recommendations.

The subgrade must not be allowed to freeze during construction and there should be no frost present in the subgrade soils prior to concrete placement for the thickened-edge slab. It is recommended that inspection by qualified geotechnical personnel be conducted during construction to identify any soft or weak material at the subgrade level that should be removed and replaced.

5.2.3.4 Granular Fill Requirements

The granular materials should comply with the gradation and material requirements outlined in the City of Winnipeg Design and Construction Specifications CW 3110 for Granular C Base Course and Granular C – 50 mm Sub-base. The gradation requirements for the granular fill materials are shown in **Table 8**.

Table 8. Granular C Materials Gradation Limits (City of Winnipeg CW3110)

Sieve Size (mm)	% Passing	
	Granular C – 50 mm Sub-base	**Granular C – Base Course**
75	100%	-
50	97 to 100%	-
37.5	100%	-
28	-	-
25	-	100%
20	-	97 to 100%
10	-	-
5	20 to 60%	28 to 65%
2.5	-	22 to 60%
1.25	-	-
0.63	-	-
0.315	-	3 to 22%
0.08	3 to 12%	2 to 10%

Sieve analysis and compaction testing of the Granular C Base Course and Granular C – 50 mm Sub-base materials should be conducted during construction to confirm that the materials and the compaction comply with the City of Winnipeg CW3110 specification requirements.

5.2.4 Foundation Concrete

Based on our experience in the area, the class of exposure for the concrete in contact with native Winnipeg fat clay soils is considered to be severe (S-2 in CSA A23.1-09). The requirements for concrete exposed to severe sulphate attack are provided in **Table 9**.

Table 9. Foundation Concrete Requirements

Parameter	Design Requirement
Class of Exposure	S-2
Compressive Strength	35 MPa at 56 days
Air Content	4 to 7%
Water-to-Cementing Materials Ratio	0.45 max.
Cement	Type HS or HSb

Concrete in contact with the native soils should meet the above requirements.

6 Drainage

All sources of water should be directed away from the proposed building and the ground surface around the proposed building should be graded to promote drainage away from the foundation and therefore minimize water accumulation and potential foundation issues. Final site grading should ensure that all surface runoff is directed away from the proposed store using a minimum gradient of 2%. To compensate for potential settlement of backfill materials adjacent to the proposed store, the grade should be increased to 5% for the first 2 m (horizontally) from the proposed building.

7 Seismic Site Classification

As per the 2020 NBCC, the Seismic Site Class is based on the average shear wave velocity of the ground profile within the top 30 m. In accordance with the 2020 NBCC, when shear wave velocities calculated from in-situ measurements are not available (as per this case), the Seismic Site Class can be assessed based on soil properties within the top 30 m. The soil properties from the borehole records and the ground profile criteria in Table 4.1.8.4.-B of the 2020 NBCC were considered to assess the Seismic Site Class. From the available information, the Seismic Site Class may be considered “Site Class E” (soft soil).

Seismic Hazard Values specific to the project location and associated with this Seismic Site Class can be obtained using Earthquakes Canada’s “2020 National Building Code of Canada Seismic Hazard Tool”. Seismic design parameters associated with the above recommended Seismic Site Class should be reviewed by a structural engineer and incorporated into the design as required by the 2020 NBCC.

8 Design Review, Construction Monitoring and Testing Requirements

Stantec should be retained to review the foundation plans and specifications for conformance with the intent of this report. During construction, the designer should consider that a representative from our firm be involved with the following tasks:

- Inspection of foundation installation;
- Inspection of subgrade conditions for the thickened-edge slab (if applicable);
- Testing of concrete;
- Sieve analysis and field density tests during placement and compaction of granular fill materials; and,
- Inspection during proof rolling of subgrade.

The purpose of the foundation and subgrade inspection services would be to provide Stantec the opportunity to observe the soil conditions encountered during construction, evaluate the applicability of the information presented in this report to the soil conditions encountered, and provide appropriate changes in design or construction procedures if conditions differ from those described herein. The purpose of the concrete testing is to ensure this material complies with the specification requirements. The purpose of the sieve analysis, proof rolling and field density tests is to confirm the fill materials are suitable and have been compacted to the specified density.

9 Closure

This report was prepared for the exclusive use of the City of Winnipeg for specific application to the Pembina Highway Overpass at Abinojii Mikanah (Bishop Grandin) Rehabilitation and Related Works – Parks Building Relocation project. Use of this report is subject to the Statement of General Conditions included in **Appendix A**. It is the responsibility of the City of Winnipeg who is identified as “the Client” within the Statement of General Conditions, to review the conditions and notify Stantec should any of them not be satisfied.

We trust that this report meets your present requirements. If you have any questions or require additional information, please contact us. This report has been prepared by Jack Sears, E.I.T., and reviewed by Kevin Baylis, M.Eng., P.Eng. and Aron Piamsalee, M.Sc., P.Eng.

We appreciate the opportunity to assist you in this project.

Appendix A Statement of General Conditions

Stantec

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This professional work product (“hereinafter referred to as the Report”) has been prepared for the sole benefit of the Client in accordance with Stantec’s contract with the Client. While the Report may be provided by the Client to applicable authorities having jurisdiction and to other third parties in connection with the project, Stantec disclaims any legal duty based upon warranty, reliance, or any other theory to any third party, and will not be liable to such third party for any damages or losses of any kind that may result.

BASIS OF THIS REPORT: This Report relates solely to the site-specific project for which Stantec was retained and the stated purpose for which the Report was prepared. The information, opinions, conclusions and/or recommendations made in this Report are in accordance with Stantec’s present understanding of the site-specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time the scope of work was conducted and do not take into account any subsequent changes. If the proposed site-specific project differs or is modified from what is described in this Report or if the site conditions are altered, this Report is no longer valid unless Stantec is requested by the Client to review and revise the Report to reflect the differing or modified project specifics and/or the altered site conditions. This Report is not to be used or relied on for any variation or extension of the project, or for any other project or purpose or site, and any unauthorized use or reliance is at the recipient’s own risk.

STANDARD OF CARE: Preparation of this Report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

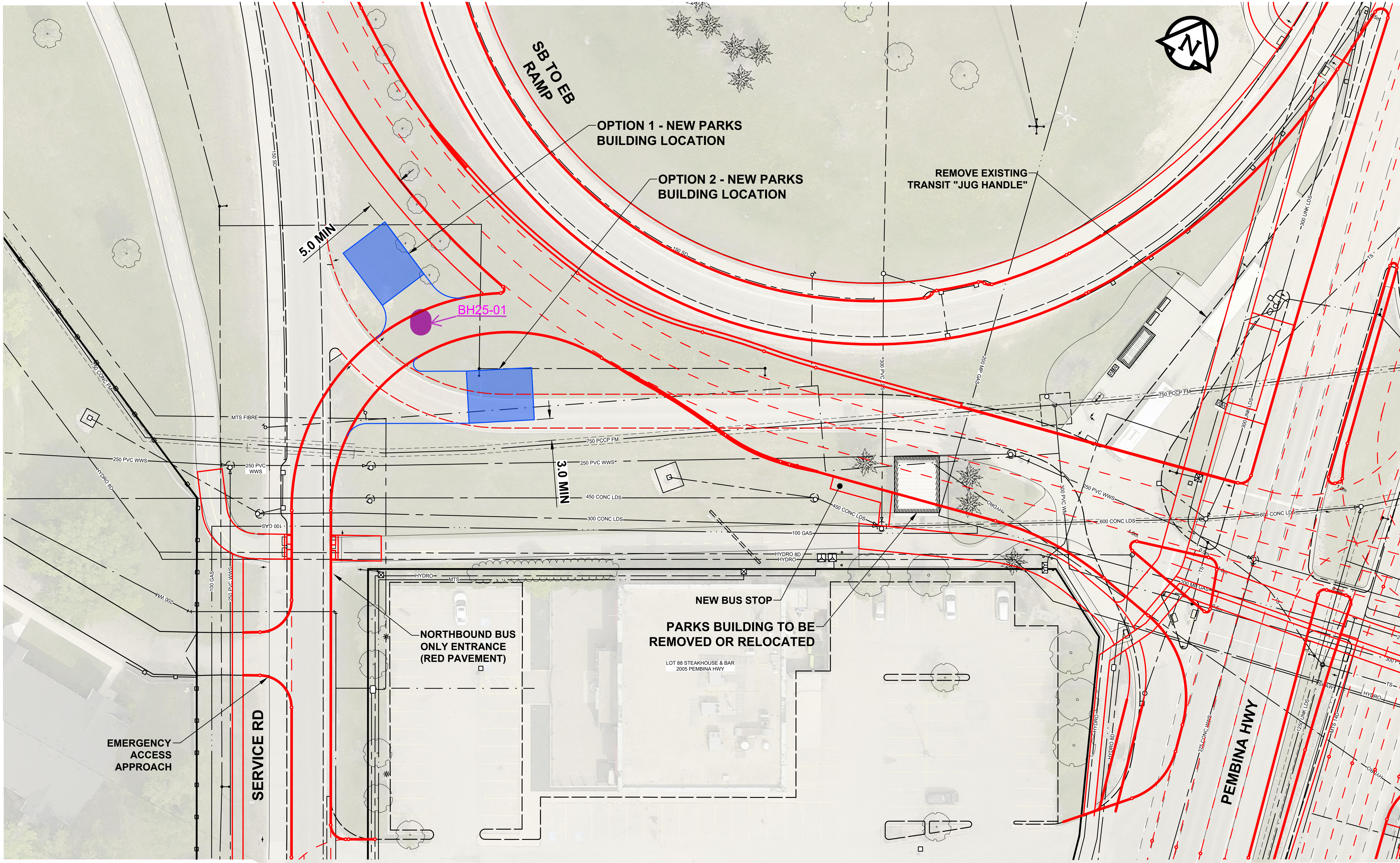
PROVIDED INFORMATION: Stantec has assumed all information received from the Client and third parties in the preparation of this Report to be correct. While Stantec has exercised a customary level of judgment or due diligence in the use of such information, Stantec assumes no responsibility for the consequences of any error or omission contained therein.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this Report are based on site conditions encountered by Stantec at the time of the scope of work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behaviour. Extrapolation of in-situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this Report or encountered at the test and/or sample locations, Stantec must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the Report conclusions or recommendations are required. Stantec will not be responsible to any party for damages incurred as a result of failing to notify Stantec that differing site or subsurface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec geotechnical engineers, sufficiently ahead of initiating the next project stage (e.g., property acquisition, tender, construction, etc.), to confirm that this Report completely addresses the elaborated project specifics and that the contents of this Report have been properly interpreted. Specialty quality assurance services (e.g., field observations and testing) during construction are a necessary part of the evaluation of subsurface conditions and site work. Site work relating to the recommendations included in this Report should only be carried out in the presence of a qualified geotechnical engineer; Stantec cannot be responsible for site work carried out without being present.

Appendix B Sketch 01



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Stantec
500 - 311 Portage Avenue, Winnipeg MB Canada R3B 2B9
Tel. 204.489.5900 Fax. 204.453.9012 www.stantec.com

SCALE: 1:500

0 5 10 20 30m

PEMBINA HIGHWAY OVERPASS AT ABINOJII MIKANAH

PARKS BUILDING RELOCATION

SKETCH 01

VERSION 1.0

Appendix C Borehole Record

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis

Rootmat	vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of visible and invisible fragments of decayed organic matter
Till	unstratified glacial deposit which may range from clay to boulders
Fill	material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure

Desiccated	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of regular alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

Terminology describing soil types

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris)

Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on Page 2. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

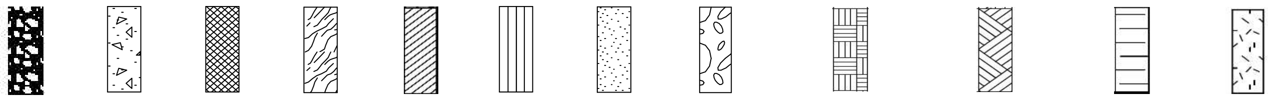
Terminology describing consistency of cohesive soils

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kg/cm^2 or kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Asphalt | Concrete | Fill | Organics | Clay | Silt | Sand | Gravel Cobbles Boulders | Undifferentiated Bedrock | Sedimentary Bedrock | Metamorphic Bedrock | Igneous Bedrock

SAMPLE TYPE

AS, BS, GS	Auger sample; bulk sample; grab sample
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
SO	Sonic tube
SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby Tube or thin wall tube
SV	Shear vane
RC *HQ, NQ, BQ, etc.*	Rock Core; samples obtained with the use of standard size diamond coring bits.

WATER LEVEL

Measured: in standpipe, piezometer, or well

Inferred: seepage noted, or; measured during or at completion of drilling

RECOVERY FOR SOIL SAMPLES

The recovery is recorded as the length of the soil sample recovered in the direct push, split spoon sampler, Shelby Tube, or sonic tube.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test (SPT): the number of blows of a 140-pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50 for 75 mm or 50/75 mm). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60-degree apex cone connected to ‘A’ size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Total Core Recovery (TCR) denotes the sum of all measurable rock core recovered in one drill run. The value is noted as a percentage of recovered rock core based on the total length of the drill run.

Solid Core Recovery (SCR) is defined as total length of solid core divided by the total drilled length, presented as a percentage. Solid core is defined as core with one full diameter.

Rock Quality Designation (RQD) is a modified core recovery that incorporates only pieces of solid core that are equal to or greater than 10 cm (4") along the core axis. It is calculated as the total cumulative length of solid core (> 10 cm) as measured along the centerline of the core divided by the total length of borehole drilled for each drill run or geotechnical interval, presented as a percentage. RQD is determined in accordance with ASTM D6032.

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock quality

Rock Mass Quality	Rock Quality Designation Number (RQD)	Alternate (Colloquial) Rock Mass Quality	
Very Poor Quality	0-25	*Very Severely Fractured*	*Crushed*
Poor Quality	25-50	*Severely Fractured*	*Shattered or Very Blocky*
Fair Quality	50-75	*Fractured*	*Blocky*
Good Quality	75-90	*Moderately Jointed*	*Sound*
Excellent Quality	90-100	*Intact*	*Very Sound*

Terminology describing rock strength

Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

Terminology describing rock with respect to discontinuity and bedding spacing

Spacing (mm)	Discontinuities Spacing	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Stantec

BOREHOLE RECORD

BH25-01

CLIENT: **City of Winnipeg**
PROJECT: **Pembina Highway Overpass at Abinojii Mikanah Rehabilitation**
LOCATION: **1995 Pembina Hwy, Winnipeg, MB**
DATE BORED: **January 24, 2025**

BH COORDINATES
[NAD 83 UTM 14 U]
5519986 N 632803 E
WATER LEVEL: **N/A**

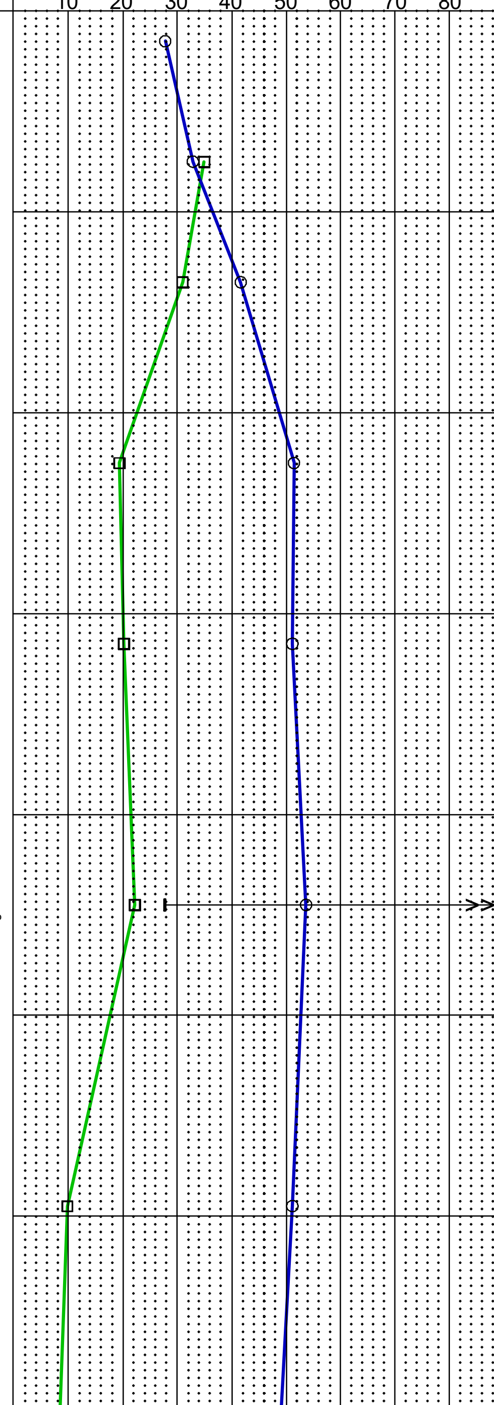
PROJECT NO.: **132500075**
BH ELEVATION: **N/A**
DATUM: **NAD 83**

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION (USCS)	STRATA PLOT	SAMPLES: TYPE	SAMPLES: NUMBER	SAMPLES: RECOVERY (mm) or TCR %	SAMPLES: N-VALUE or RQD %	OTHER TESTS / REMARKS
		Surface: grass						
0		**TOPSOIL:** black clay containing trace silt		AS	AS1			
		Firm to stiff brown fat **CLAY (CH)** - trace silt						
1				AS	AS2			
				AS	AS3			
2		- grey below 2.1 m		AS	AS4			
3				AS	AS5			
4				AS	AS6			Sieve/Hydro at 4.3 m G 0% S 1% M 30% C 70%
5				ST	ST7			
6				AS	AS8			
		- soft below 6.1 m						
7								

UNDRAINED SHEAR STRENGTH, Cu (kPa)
▲ LABORATORY TEST ◆ FIELD VANE TEST
★ POCKET PENETROMETER □ POCKET SHEAR VANE
50 kPa 100 kPa 150 kPa 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
✖ SPT (N-value) BLOWS/0.3m

Water Content (%) and Blow Count
10 20 30 40 50 60 70 80



BACKFILL

ELEVATION (m)

BACKFILL SYMBOL: ASPHALT, GROUT, CONCRETE, BENTONITE, DRILL CUTTINGS, SAND, SLOUGH

Drilling Contractor: Maple Leaf Drilling	Logged By: JS
Drilling Method: 125 mm SSA	Reviewed By: KB
Completion Depth: 13.4 m	Page 1 of 2

BOREHOLE RECORD

BH25-01

CLIENT: **City of Winnipeg**
PROJECT: **Pembina Highway Overpass at Abinojii Mikanah Rehabilitation**
LOCATION: **1995 Pembina Hwy, Winnipeg, MB**
DATE BORED: **January 24, 2025**

BH COORDINATES
[NAD 83 UTM 14 U]
5519986 N 632803 E
WATER LEVEL: **N/A**

PROJECT NO.: **132500075**
BH ELEVATION: **N/A**
DATUM: **NAD 83**

UNDRAINED SHEAR STRENGTH, Cu (kPa)
▲ LABORATORY TEST ◆ FIELD VANE TEST
★ POCKET PENETROMETER □ POCKET SHEAR VANE
50 kPa 100 kPa 150 kPa 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L
✖ SPT (N-value) BLOWS/0.3m
Water Content (%) and Blow Count
10 20 30 40 50 60 70 80

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION (USCS)	STRATA PLOT	TYPE	NUMBER	RECOVERY (mm) or TCR %	N-VALUE or RQD %	OTHER TESTS / REMARKS
7								
				AS	AS9			
8								
9				AS	AS10			
10				AS	AS11			
		- very soft below 10.7 m						
11								
12				AS	AS12			
13								
		Grey silt **TILL** - sandy, trace gravel, trace clay		AS	AS13			
14								

End of Borehole
• Auger refusal at a depth of 13.4 m within the till.
• Minor groundwater seepage was observed at a depth of 9.1 m.
• Borehole sloughed at 8.5 m upon completion of drilling.
• Borehole backfilled with bentonite chips and auger cuttings.

BACKFILL SYMBOL: ASPHALT, GROUT, CONCRETE, BENTONITE, DRILL CUTTINGS, SAND, SLOUGH

Drilling Contractor: Maple Leaf Drilling	Logged By: JS
Drilling Method: 125 mm SSA	Reviewed By: KB
Completion Depth: 13.4 m	Page 2 of 2

Appendix D Laboratory Data

LABORATORY
199 Henlow Bay
Winnipeg MB R3Y 1G4
Tel: (204) 488-6999

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS ASTM D4318

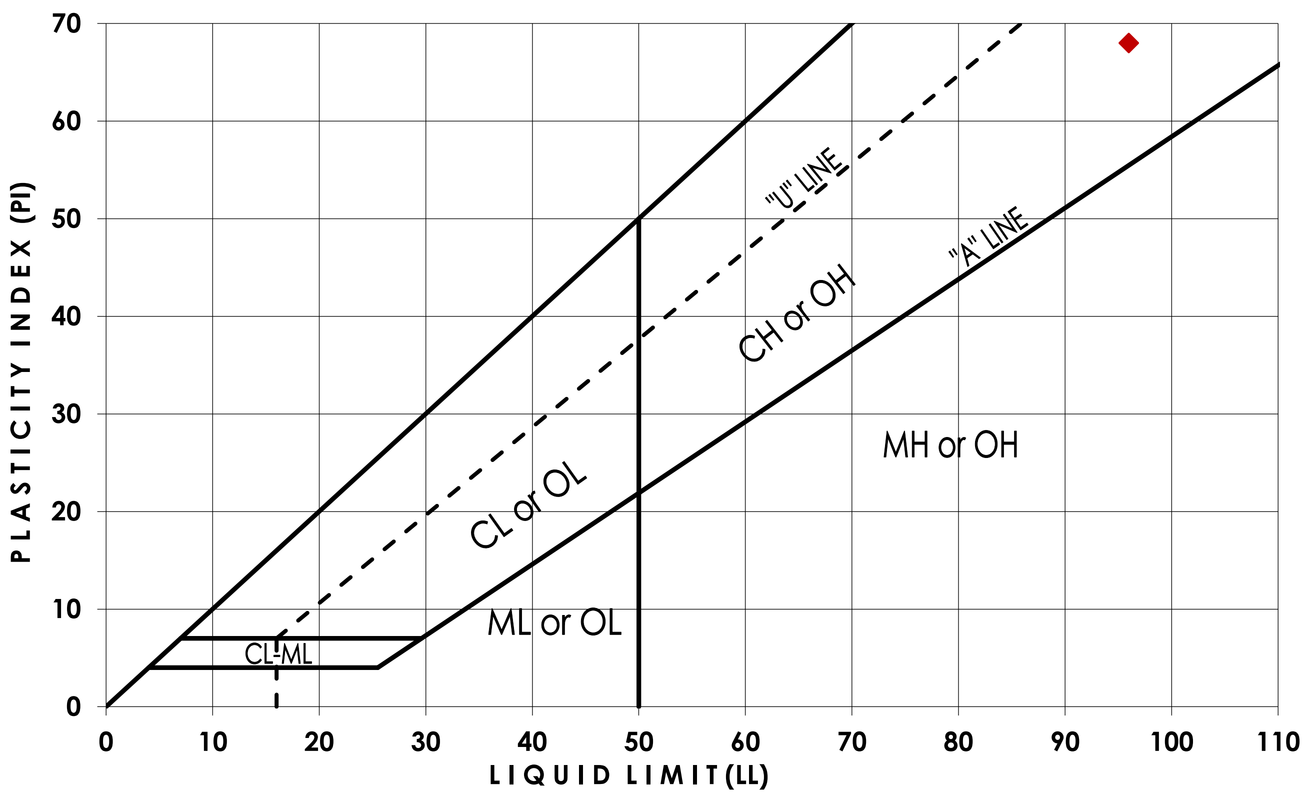
Client: City of Winnipeg

Project No.: 132500075
Project Name: Pembina Hwy Overpass Abinojii

Date Samples Received: 2025.Feb.04
Material Type: Fat Clay
Tested By: Larry Presado

Symbol	Testhole No.	Depth (m)	Liquid Limit	Plastic Limit	Plasticity Index	USCS
◆	BH25-01	4.3	96	28	68	CH

Plasticity Chart



Reviewed By: Kevin Baylis, M. Eng., P.Eng.
Date Reviewed: February 17, 2025

LABORATORY
199 Henlow Bay
Winnipeg MB R3Y 1G4
Tel: (204) 488-6999

PARTICLE SIZE ANALYSIS

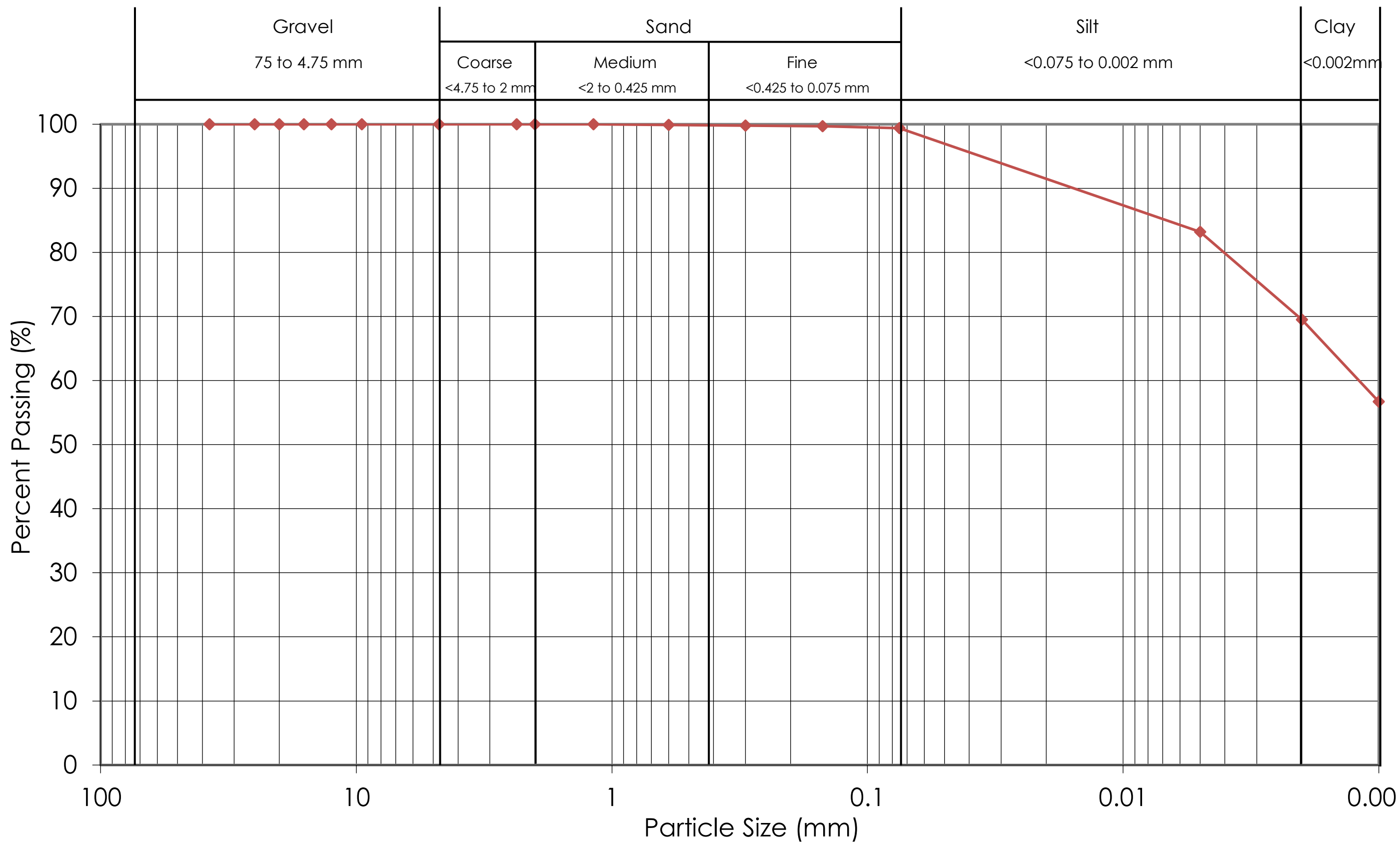
ASTM D422

Client Name: City of Winnipeg

Project No.: 132500075
Project Name: Pembina Hwy Overpass Abinojii

Date Samples Received: 2025.Feb.04
Tested By: Larry Presado

Material Type: Fat Clay



Symbol	Sample ID	Gravel, % 75 to 4.75 mm	Sand, % Coarse <4.75 to 2.0 mm	Sand, % Medium <2.0 to 0.425 mm	Sand, % Fine <0.425 to 0.075 mm	Silt, % <0.075 to 0.002 mm	Clay, % <0.002 mm	Colloids, % < 0.001 mm
◆	BH25-01	0.0	0.0	0.1	0.5	29.9	69.5	56.7

NT*: Sample not tested for colloids.

Reviewed By: Kevin Baylis, M.Eng., P.Eng.
Date Reviewed: February 17, 2025