The City of Winnipeg

Northeast Interceptor Sewer Geotechnical Baseline Report

Prepared by:
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Prepared for:
City of Winnipeg
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Date: April, 2018

Project #: 60509089

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Date
April 25, 2018

Mr. Stacy Cournoyer, P.Eng.
Senior Project Engineer
City of Winnipeg
110 - 1199 Pacific Avenue
Winnipeg, MB R3E 3S8

Our Reference: 60509089

Dear Mr. Cournoyer:

Regarding: Northeast Interceptor Sewer Red River Crossing - Geotechnical Baseline Report

We are pleased to submit this Geotechnical Baseline Report for the Northeast Interceptor Sewer to be constructed in northeast Winnipeg, Manitoba. The report presents the baseline subsurface soil, bedrock, and groundwater conditions and descriptions that the proponents shall use for their tender preparation.

This Geotechnical Baseline Report has been prepared by AECOM Canada Ltd. for the City of Winnipeg. The report has been prepared in general conformance with the guidelines and practices described in the Geotechnical Baseline Reports for Construction, Suggested Guidelines, published by ASCE, 2007.

If you have any questions concerning this report please contact the undersigned at (780) 486-7905.

Sincerely,
AECOM Canada Ltd

PROVINCE OF MANITOBA
F.L. ALOBAIDY
Member
41677
REGISTERED PROFESSIONAL ENGINEER
April 25, 2018

Faris Alobaidy, M.Sc., P.Eng.
Senior Geotechnical Engineer

FA:rd

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General Statement – Normal Variability of Subsurface Conditions

The scope of the investigation presented herein is limited to an investigation of the subsurface conditions as to the suitability of the proposed project. This report has been prepared to aid in the evaluation of the site and to assist the engineer in the design of the facilities. The description of the project represents an understanding of the significant aspects of the project relative to the design and construction of earth work, foundations, and similar. In the event of any changes in the basic design or location of the structures as outlined in this report or plan, AECOM Canada Ltd. should be given the opportunity to review the changes and to modify or reaffirm, in writing, the conclusions and recommendations of this report.

The analyses and recommendations represented in this report are based on the data obtained from the testholes drilled at the locations indicated on the site plans and from other information discussed herein. This report is based on the assumption that the subsurface conditions everywhere on the site are not significantly different from those encountered at the testhole locations. However, variation in the soil conditions between the testholes may exist. Also, general groundwater levels and conditions may fluctuate from time to time. The nature and extent of the variations may not become evident until construction. If subsurface conditions different from those encountered in the exploratory borings are observed or encountered during construction, or appear to be present beneath or beyond excavations, AECOM Canada Ltd. should be advised at once so that the conditions can be observed and reviewed and, where necessary, the recommendations reconsidered.

Since it is possible for conditions to vary from those identified at the testhole locations and from those assumed in the analysis and preparation of recommendations, a contingency fund should be included in the construction budget to allow for the possibility of variations which may result in modification of the design and construction procedures.

In order to observe compliance with the design concepts, specifications, or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated, it is recommended that all construction operations dealing with earthwork and the foundations be observed by an experienced geotechnical engineer. In addition, it is recommended that a qualified geotechnical engineer review the plans and specifications that have been prepared to check for substantial conformance with the conclusions and recommendations contained in the report.

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Quality Information

Report Prepared By:

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April 25, 2018

Report Reviewed By:

Faris Alobaidy, M.Sc., P.Eng.
Senior Geotechnical Engineer

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1. Introduction

AECOM has prepared this Geotechnical Baseline Report (GBR) for the Northeast Interceptor sewer (NE Interceptor) to be constructed in the community of Kildonan in northeast Winnipeg, Manitoba (the Project). The purpose of this GBR is to:

- Provide a baseline interpretation of the geotechnical aspects of the design and construction of the works;
- Set clear baselines for subsurface conditions anticipated to be encountered during construction;
- Provide all bidders with a single contractual interpretation in preparing bids;
- Describe the subsurface conditions along the NE Interceptor alignment; and,
- Assist in evaluating the requirements for excavation, temporary support, groundwater control, and ground movement for shaft and tunnel construction.

The GBR presents the subsurface conditions as baseline values and descriptions that the proponents shall use for their tenders. The GBR should be read in conjunction with the Geotechnical Data Report (GDR) prepared for NE Interceptor by AECOM dated April 25, 2018. The baselines presented in this GBR do not provide a warranty that subsurface conditions different from the baselines will not be encountered. The baselines, however, represent a contractual agreement between the City of Winnipeg (the City) and the Contractor to use for the resolution of claims made for “differing ground conditions”.

The baselines in this GBR also provide the City with the opportunity to allocate risks associated with the variability in the subsurface ground conditions during bidding stage. Risks associated with consistent or less adverse subsurface conditions than baselined subsurface conditions are allocated to the Contractor and risks associated with more adverse subsurface conditions than the baselined subsurface conditions are accepted by the City. The effective use of the baseline conditions will depend on adequate documentation of subsurface conditions encountered during tunnelling.

Proponents must consider this GBR as part of the Contract Documents and it must be read in conjunction with the Specifications and the Design Drawings prepared by AECOM for the City. The hierarchy of this document and other documents is indicated in the Project’s Contract Documents.

The baselines presented in this GBR apply to the excavation limits shown on the Design Drawings and Figures provided in this GBR. The baselines presented in this GBR do not apply to Contractor-modified portion(s) of the Project.

Some of the technical concepts, terms and descriptions in this GBR may not be fully understood by bidders. It is required that bidders have a geotechnical engineer with local experience, who is familiar with the topics in this GBR, to carefully review and explain this information so that a complete understanding of the information presented in this GBR can be developed prior to submitting a bid.

The GBR has been prepared in general conformance with the guidelines and practices described in the Geotechnical Baseline Reports for Construction, Suggested Guidelines, published by ASCE, 2007. The GBR has been prepared by AECOM for the City.

Certain elements of the Project are based on requirements that cannot be varied unless otherwise specified in this GBR. These include, but are not limited to, the following:

- Use of full-face microtunnelling methods.
- Adoption of ‘sealed’ methods of shaft construction – ‘sealed’ methods of shaft construction may include secant piles, pre-cast concrete or cast-in-place concrete caissons, or other methods. All sealed shafts are required to have a concrete base designed to prevent basal heave, resist hydrostatic pressures, and minimize ingress of fines and infiltration of groundwater.
- Micro-Tunnelling Boring Machine (MTBM) launch and receiving shafts shape and dimensions.
- Final siphon internal diameter.
- Alignment and invert of the proposed siphon.

Other elements of the project that are flexible and afford the Contractor latitude in planning its work and selecting means and methods, subject to approval of the City, include, but are not limited to, the following:

- Procurement, selection, and configuration of the Microtunnel Boring Machine (MTBM).
- Means of installing stub connections.
- Selection of the type of jacking pipe.
- Design of the jacking pipe, although there are minimum requirements that have to be satisfied.
- Type of sealed shaft support system.

2. Project Description

2.1 General

The description and dimensions for the various components of the project provided in this GBR are approximate and for illustration purposes only. The Contractor should refer to the Contract Documents/Drawings for accurate information on dimensions and project layout.

2.2 Project Location

The project site is located within the Kildonan area of the northeast portion of the City of Winnipeg. The proposed Northeast Interceptor Sewer (NEIS) alignment crosses the Red River directly south of the existing Kildonan Settlers Bridge as shown on **Figure 1**. The Kildonan Settlers Bridge bounds the project limits to the north of the proposed NEIS alignment.

2.2.1 Adjacent Structures

A high-rise residential development is located approximately 50 to 75 m southeast of the eastern inlet chamber of the NEIS alignment. The existing eastern inlet chamber directly bounds the existing parking lot of the high-rise residential development. The Kildonan Golf Course is situated approximately 55 m to the west /southwest of the proposed NEIS alignment and western siphon outlet chamber (situated between the Red River and Main Street).

The existing siphon sewer alignment is located directly south of the proposed NEIS alignment. Overhead electrical utility lines run near parallel to the existing siphon alignment at the Red River Crossing. Additional existing buried utilities such as a gas pipeline and telecommunication lines are present between the existing residential high-rise building and Chief Peguis Trail. The gas pipe line does not appear to cross the river or intersect the proposed eastern inlet chamber.

The proposed NEIS alignment relative to adjacent and pertinent features is shown on **Figure A1** included in **Appendix A**.

2.2.2 Winnipeg Climate

Winnipeg is located in central southern Manitoba at the bottom of the Red River Valley, a low lying flood plain with flat topography. Winnipeg has a humid continental climate with a wide range of temperatures throughout the year. The monthly average temperature ranges from -18°C in January to 20°C in July. Winter is defined as the time which the daily mean temperature remains below 0°C and typically lasts from the beginning of November to the beginning of April. Spring and autumn are defined as the time period the mean daily temperature ranges from 0° to 6°C and are typically short in duration, lasting only a couple of weeks. The average yearly precipitation in Winnipeg is 505 mm of precipitation per year although the precipitation can vary greatly. The average annual snow fall in Winnipeg is 115 cm, with the most snow typically accumulating in January and February.

2.3 Project Background

It is understood that the existing sewer siphon is under capacity and experiences surcharging during severe wet weather events and additional capacity is required to meet current and future wet weather flow conditions. In order to add additional capacity for the siphon, a trenchless solution is the proposed method for installation of additional conveyance capacity. The existing siphon is comprised of 500 mm and 800 mm steel pipes connecting to inlet and outlet chambers on the eastern and western riverbanks, respectively. The proposed NEIS alignment will be constructed on an almost parallel alignment north of the existing siphon.

2.4 Key Components of the Project

Construction of the NEIS will begin from the downstream siphon chamber (western siphon outlet chamber) located to the south/southeast of the Kildonan Settlers Bridge, and will terminate to the southwest of the Kildonan Settlers Bridge (eastern siphon inlet chamber) as shown on **Figure A1** in **Appendix A**. The proposed siphon will be connected to the existing 1800 mm monolithic concrete interceptor sewer via stub connections. A summary of the NEIS lengths, sizes and installation methods are provided in Table 2-1.

Table 2-1: Summary of NEIS Lengths, Sizes and Proposed Installation Methods

Location	Length (m)	Size (mm)	Installation Method
Start: 1+288.61 – Western Outlet Chamber End: 1+539.70 – Eastern Outlet Chamber	251.09	900- Carrier Pipe Casing Pipe (Optional)	Microtunnelling
Eastern and Western Outlet Chambers Sewer Connection	4.1 to 6.2	1200- Carrier Pipe 2400 – Casing Pipe (Optional)	Pipe Jacking

The NEIS will be installed using two (2) shafts to facilitate the trenchless forms of siphon installation. The shafts will be used to launch and/or retrieve the Microtunnelling Boring Machine (MTBM). The locations of the shafts are shown on **Figure A1** in **Appendix A**. Based on current geotechnical information and groundwater depths, it is understood that sealed methods of shaft construction are required to reduce the potential for dewatering.

Sealed methods of construction are to be utilized for all shafts, manholes and connection chambers as a means of minimizing lowering of the groundwater table, reducing the volume and duration of pumping operations, and reducing potential of ground subsidence due to lowering of groundwater table and lateral ground movements.

The inside diameters of the launching (western outlet chamber) and receiving (eastern inlet chamber) shafts are generally 8.5 m and 5.0 m, respectively.

In addition to the siphon chamber shafts, two temporary access shafts may be required at each of the sewer connection points to a depth equal to the invert of the existing siphon. The dimensions of these temporary shafts are a function of Contractors selected construction methodology and are subject to review by the Consultant.

3. Sources of Information

Reference should be made to the AECOM GDR for subsurface exploration and testing sources.

The following sources of information and references were referred to in preparation of this GBR.

3.1 Publications

1. Bannatyne, B.B., 1975. High Calcium Limestone Deposits of Manitoba. Manitoba Branch Publication 75-1.
2. Baracos, A.G. Shields, D.H., and Kjartenson, B., 1983. Geological Engineering Report for Urban Development of Winnipeg. University of Manitoba, Department of Geological Engineering.
3. Broms, B.B., Bennemark, H., 1967. Stability of Clay at Vertical Openings. ASCE, Journal of Soil Mechanics and Foundation Engineering Division, SMI 93, 71-94.
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6. Deere, D., 1964. Technical Description of Rock Cores for Engineering Purposes. Rock Mechanics and Engineering Geology, V.1, No.1.
7. Gamble, J.C., 1971. Durability-Plasticity Classification of Shales and Other Argillaceous Rocks. PhD Thesis, University of Illinois, Urbana.
8. Graham, J., and Shields, D.H, 1985. Influence of geology and geological processes on the geotechnical properties of plastic clay. Engineering Geology.
9. Heuer, R.E., 1974. Important ground parameters in soft ground tunnelling. Proceedings of Specialty Conference on Subsurface Exploration for Underground Excavation and Heavy Construction, New York, ASCE.
10. Hollmann, F., Thewes, M., 2013. Assessment method for Clay Clogging and Disintegration of Fines in Mechanised Tunneling. TUST 37, pp. 96-106.
11. KGS Group, Acres Engineering, UMA Engineering, 2004. Appendix B, Floodway Channel Pre-Design, Floodway Expansion Project, Project Definition and Environmental Assessment, Preliminary Engineering Report.
12. Kirsten, H.A.D., 1988. Case Histories of groundmass characterization for Excavatability. ASTM STP 984, pp. 102-120.
13. Mayne, P.W., and Kulhawy, F.H., 1982. K_0-OCR Relationship in Soil. ASCE Journal of Geotechnical Engineering Division, Vol 108, No. GT6, pp. 851-872.
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16. Schmidt, B., 1966. Discussion of Earth Pressure At-Rest Related to Stress History. Canadian Geological Journal, Vol 3, No. 4, pp.239-242.
17. Peck, R.B., 1969. Deep Excavations and Tunnelling in Soft Ground. 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City State-of-the-Art Volume, pp. 225- 290.
18. Taylor, R.K. and Smith, T.J., 1986. The Engineering Geology of Clay Minerals: Swelling, Shrinkage and Mudrock Breakdown. Clay Minerals, Vol 21, pp. 235-260.
19. Terzaghi, K. 1950. Geologic aspects of soft ground tunnelling. Chapter 11 in Applied Sedimentation, R. Task and D. Parker, eds. New York: John Wiley & Sons.
20. Thewes, M., 1999. Adhesion of Clays During Tunneling with Slurry Shields. Dissertation, Vol 21, Bergischen University Wuppertal, Department of Civil Engineering.
21. Thewes M., Burger W., 2004. Clogging Risk for TBM Drives in Clay. Tunnels and Tunnelling International, pp.28-31, June 2004.
22. Thurber Engineering Ltd., Thurber Engineering Ltd., 2007. Geotechnical Baseline Report for 15th Street Siphon Upgrade, Calgary, Alberta – Tunnel Crossing of the Bow River. Report Prepared for Associated Engineering (File: 17-123-415, dated July 23, 2007).
23. Thurber Engineering Ltd., 2008. Geotechnical Baseline Report for Valley Ridge Feedermain Phase 1, Calgary, Alberta. Report Prepared for CH2M Hill Canada (File: 17-834-90, dated July 28, 2008).
24. Van Der Merwe, D.H., 1964. The Prediction of Heave from the Plasticity Index and Percentage of Clay Fraction of Soil. Journal of South African Institution of Civil Engineering, Vol 6, No. 6, pp. 103-107.

4. Geologic Setting

4.1 Regional Geology

The regional geology of the site has been outlined in the AECOM (March 2018) Geotechnical Data Report and should be referenced in conjunction with Section 4 of this Report for a more detailed outline of the regional geological setting. Additional information for the regional geology within the City of Winnipeg is included in the following references:

1. Baracos, A., Shields, D.H., and Kjartanson, B., 1983. Geological engineering report for urban development of Winnipeg. University of Manitoba.
2. Baracos, A., Graham, J., Kjartanson, B., and Shields, D.H., 1983. Geology and soil properties of Winnipeg. In ASCE Conference on Geologic Environment and Soil Properties, Houston TX: 39-56.
3. Baracos, A., 1977. Compositional and structural anisotropy of Winnipeg soils – a study based on scanning electron microscopy and X-ray diffraction analyses. Canadian Geotechnical Journal, 14: 125-137.
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5. Quigley, R.M., 1968. Soil mineralogy Winnipeg swelling clays. Can. Geotech. J. 5(2), pp. 120-122.
6. Render, F.W., 1970. Geohydrology of the metropolitan Winnipeg area as related to groundwater supply and construction. Canadian Geotechnical Journal, 7(3): 243-274.
7. Skaftfeld, K., 2014. Experience as a Guide to Geotechnical Practice in Winnipeg (Masters of Science Thesis). University of Manitoba, Winnipeg, Manitoba.

Site-specific geotechnical and geological information derived from the AECOM 2016 geotechnical investigation and past investigations (including results of the geotechnical drilling and laboratory test data) are also presented in the GDR.

4.2 Topography

The topography along the NEIS alignment varies significantly as the site is located at a river crossing. The elevation along the eastern riverbank varies between approximately 228.0 m and 226.2 m at its crest, and decreases sharply towards the centre of the river channel to an approximate elevation of 215.0 m. The ground surface along the crest of the western riverbank varies between 226.3 m and 227.75 m and in turn falls sharply to the centreline of the river channel. It is understood that the proposed excavation work required for shaft construction will not impact existing riverbank profiles as the siphon chambers are located away from the riverbank slopes. Any plans to disturb the riverbank slopes should be submitted to the Consultant for review prior to construction.

The ground surface profile along the NEIS alignment is shown on **Figure A2** in **Appendix A**.

5. Summary of Subsurface Investigation

As described in the AECOM (March 2018) GDR, AECOM conducted a geotechnical investigation in 2016 along the proposed NEIS alignment with the objective of characterizing the subsurface ground and groundwater conditions along the new alignment. The findings of the AECOM 2016 geotechnical investigation are summarized in the GDR, but the pertinent findings of the investigation are also presented below.

Several past geotechnical investigations that have been completed near the project site have also been referenced within the AECOM (March 2018) GDR. These past investigations have been carried out to support the design of the Kildonan Settlers Bridge and the North Kildonan Feedermain replacement projects. The findings of these investigations have also been summarized in to the following sections of this Report.

5.1 Geotechnical Investigation

5.1.1 AECOM 2016 Geotechnical Investigation

From August 19 to September 9, 2016, four (4) test holes (TH16-01 to TH16-04) were drilled at the approximate locations shown on **Figure A2** within **Appendix A**. Test holes TH16-01 and TH16-02 were drilled along the northwest embankment in the vicinity of the west chamber location, test hole TH16-03 was drilled within the Red River channel, and test hole TH16-04 was drilled in the vicinity of the east chamber location.

Drilling was completed by Maple Leaf Drilling using the following equipment: track-mounted Acker Renegade drill rig equipped with 125 mm solid stem augers and HQ sized (96 mm OD) core barrel for test holes TH16-01 and TH16-02, Cricket B20 equipped with BQ sized (60 mm OD) core barrel mounted on a floating barge for test hole TH16-03, and track-mounted Mobile B54X drill rig equipped with 125 mm solid stem augers and NQ sized (75.7 mm OD) core barrel for test hole TH16-04. Subsurface conditions observed during drilling were visually classified and documented by AECOM geotechnical personnel. Other pertinent information such as groundwater and drilling conditions were also recorded during the field investigation.

Disturbed soil samples collected from auger cuttings and split-spoon samplers, as well as relatively undisturbed Shelby Tube samples were obtained at regular intervals. Standard penetration tests (SPTs) were completed at selected intervals in the test holes and blow counts for 300 mm penetration (SPT "N" blow counts) were recorded. NQ and HQ rock core samples were logged in the field and collected for further analysis. Recovered soil and rock core samples were transported to AECOM's materials testing laboratory in Winnipeg for further visual examination and testing.

The bedrock cores were logged on-site recording type of bedrock, Total Core Recovery (TCR) and Rock Quality Designation (RQD).

Monitoring wells (50 mm diameter PVC pipes) were installed in two of the test holes to measure groundwater depths. A vibrating wire piezometer was additionally installed in one test hole within the glacio-lacustrine clay to monitor the groundwater response within the deeper cohesive soils. The test hole logs and groundwater instrumentation details and measurements are provided in the GDR.

5.1.1.1 Laboratory Testing

Soil samples collected during the preliminary and detailed geotechnical investigations were tested in AECOM's Material Testing Laboratories in Winnipeg, Manitoba for soil classification and estimation of engineering properties. The bedrock core samples were tested in Eng-Tech Consulting Ltd., Laboratories in Winnipeg, Manitoba to estimate uniaxial compressive strength (UCS). Details of the type and number of tests are presented in **Table 5-1**. The laboratory test results for test holes drilled along the NEIS alignment are provided in the GDR.

Table 5-1: Laboratory Testing – AECOM 2016 Geotechnical Investigation

Laboratory Test	Number of Tests Completed
Moisture Content Determination	54
Atterberg Limits (3 Points)	12
Grain Size Distribution (Hydrometer Method)	8
Unconfined Compressive Strength of Rock	6

5.2 Past Geotechnical Investigations

As described in the project GDR, several geotechnical investigations have been conducted by other engineering firms along the proposed NEIS alignment and at adjacent structures. These geotechnical investigations comprise of the following:

- TREK Geotechnical (2013) - Three (3) test holes drilled along the alignment of the North Kildonan feedermain.
- KGS (2012) - Five (5) test holes drilled along the alignment of the proposed NEIS alignment.
- Dyregrov & Burgess (1987) - Twenty seven (27) test holes drilled along the alignment of the existing North Kildonan Settlers Bridge.

Detailed information of the previous geotechnical investigations is provided in the GDR.

Based on AECOM's review of the existing geotechnical investigations, the KGS (2012) investigation is deemed most applicable given that the test holes were drilled along the proposed NEIS alignment. The findings of all of the geotechnical investigations and laboratory testing results have been referenced in the review of subsurface soil/rock conditions and groundwater conditions as summarized in the GDR.

5.3 Hydrogeological Investigation

Friesen Drillers conducted a hydrogeological investigation to determine the potential for aquifer depressurization which would allow for deep excavations at the project (as well as at locations within the tunnel). The hydrogeological investigation included; test well drilling, aquifer pump testing, and technical analysis. The results of the detailed hydrogeological investigation are presented in a separate report included within the GDR, and are also discussed in context of shaft excavation in Section 6.4 of this Report.

6. Ground Characterization

6.1 General Stratigraphy

The subsurface stratigraphy along the NEIS alignment generally comprises of mixed alluvial soils (sand, silt and clay) overlying (in descending order) glacio-lacustrine clay, glacial till deposits (sand and silt till), and carbonate bedrock (predominately limestone and dolomitic limestone). The bedrock surface was typically encountered at an elevation of between 209.5 m and 210.1 m. The composition of the alluvial soils is expected to vary with depth and between riverbanks (and at the proposed siphon outfall chamber locations). Cobbles and boulders should be expected within the glacial till deposit (typical of glacial till soils within the Winnipeg area).

For the purposes of outlining the site-specific subsurface stratigraphy, the following test holes are considered applicable for characterization of the subsurface ground and groundwater conditions along the NEIS alignment:

Table 6-1: Test Holes Along NEIS Alignment

Test Hole	Coordinates (m)	Ground Surface Elevation (m)	Overburden Thickness (m)	Elevation of Bedrock Surface (m)
AECOM (2016) TH16-01	5534868 N 636362 E	227.03	17.03	209.9
AECOM (2016) TH16-02	5534787 N 636578 E	228.05	17.85	210.2
AECOM (2016) TH16-03	5534783 N 636494 E	223.80	17.10	210.0
AECOM (2016) TH16-04	5534859 N 636384 E	226.33	16.13	210.3
KGS (2012)- TH12-01	5534788 N 636543 E	226.37	16.85	209.52
KGS (2012)- TH12-02	5534757 N 636604 E	228.37	18.28	210.09
KGS (2012)- TH12-02B	5534757 N 636604 E	228.37	18.30	210.07
KGS (2012)- TH12-03	5534926 N 636265 E	230.84	21.03	209.81
KGS (2012)- TH12-03B	5534926 N 636265 E	230.84	20.98	209.86

Detailed descriptions of the subsurface conditions encountered at the test holes locations are shown on the test holes logs in Appendix C and Appendix D of the GDR. A brief description of the subsurface soil/bedrock units encountered along the NEIS alignment and their engineering properties is provided in the following sections. A simplified interpreted stratigraphic cross-section along the NEIS alignment is shown on **Figures A2** in **Appendix A** of this GBR.

6.2 Overburden Characterization

To simplify the interpretation of the soil deposits within the project boundaries, the overburden soils above the bedrock have been divided into five major soil units (excluding the carbonate bedrock) as follows:

- Clay Fill
- Alluvial Deposit
    - Clay interlayer
    - Silt Interlayer
    - Sand Interlayer
    - Organics
- Glacio-lacustrine soil
- Glacial till soil
- Carbonate Limestone Bedrock (not classified as overburden material)

This division is based on the mechanical and hydrogeological characteristics of each of the soil units. Detailed descriptions of the strata and related field and laboratory data are provided in Section 3 of the GDR.

6.2.1 Clay Fill

Silty clay fill was encountered in four (4) test holes; KGS (2012) TH12-02, 02B, 03 and 3B along the NEIS alignment directly from ground surface. The fill was generally silty clay with trace to some sand and gravel and trace rootlets, was brown, moist, and of intermediate to high plasticity. The noted thickness of the fill was between 0.45 m and 1.70 m. One moisture content test was reported on a sample from test hole TH12-02 with a value of 20 percent.

6.2.2 Alluvial Clay or Alluvial Silt Interlayer - Cohesive

Alluvial clay or alluvial silt was encountered directly from the ground surface in AECOM (2016) test holes TH16-01, 02 and 04, and KGS (2012) test hole TH12-01. Elsewhere, the alluvial clay or alluvial silt was overlain by either alluvial sand or clay fill.

The alluvial clay was generally described as containing trace silt to silty, trace sand to sandy, trace gravel and trace organics. The alluvial clay was brown to dark grey, very soft to stiff (but mostly firm to stiff), dry to wet, and was of an intermediate plasticity. The alluvial silt was noted as containing trace clay to clayey, trace sand to sandy, and was dark brown to light brown, soft to stiff, dry to moist, and of low to intermediate plasticity.

The consistency of the alluvial clay or alluvial silt varied from very soft to stiff, but was mostly firm to stiff. Undrained shear strength values ranged from 5.6 kPa to 84 kPa (with an average of 58 kPa). Minimum, maximum, average, and standard deviations of undrained shear strength values are presented in **Appendix B, Table B1**. No Standard Penetration tests were performed in the alluvial clay or alluvial silt deposits along the NEIS alignment.

The distribution of the alluvial clay or alluvial silt is outlined in **Table 6-2**.

Table 6-2: Distribution of Alluvial Cohesive Soil Unit along NEIS Alignment

Location	Alluvial Clay		Alluvial Silt	
	Soil Thickness (m)	Elevation at Base (m)	Soil Thickness (m)	Elevation at Base (m)
Eastern Riverbank (Proposed Eastern Siphon Inlet Chamber)	0.75 to 2.90	217.67- 225.15	0.65 to 1.85	217.05 to 222.03
Western Riverbank (Proposed Eastern Siphon Outlet Chamber)	0.30 to 2.05	225.13 to 228.84	0.60 to 1.30	225.43 to 227.49
River Channel	Not Encountered		Not Encountered	

It should be noted that the alluvial deposits are highly variable in composition and distribution along the NEIS alignment.

6.2.3 Alluvial Sand Interlayer - Cohesionless

The alluvial sand consisted primarily of sand, although the gravel content increased further east away from the riverbank as shown in KGS test hole TH12-01. The alluvial sand contained trace clay to clayey, trace silt to silty, trace to some gravel, and was brown to grey, very loose to compact, moist to wet, and fine to medium grained. The alluvial sand thickness varied between 9.2 m and 12.1 m along the western riverbank, and 7.5 m and 10.2 m along the eastern riverbank. The alluvial sand was generally overlain by alluvial clay or alluvial silt.

The compactness condition of the soil varied from very loose to compact, but mostly very loose to loose. Minimum, maximum, average, and standards deviation of SPT N values are presented in **Appendix B, Table B2**. A histogram distribution diagram of SPT N values is illustrated in **Figure B1** of **Appendix B** of this GBR. Boulders and cobbles are expected in this unit. Baseline gradation envelope for this soil unit is illustrated on **Figure C1** in **Appendix C**.

6.2.4 Glacio-Lacustrine Soil

An intermediate to high plasticity silty clay was encountered at elevations ranging from approximately 212.3 m and 227.5 m. The thickness of the soil unit was thinnest beneath the eastern riverbank (between 0.60 m and 4.85 m) and increased in thickness further west (between 1.90 m and 15.85 m).

The glacio-lacustrine clay generally contained trace to some silt, trace sand to sandy, trace to some gravel, trace organics, and was brown to grey, very soft to very stiff, moist to wet, and of intermediate to high plasticity. The upper cohesive soil unit was damp to moist, and stiff in consistency. The consistency of the soil decreased with depth typically from a stiff clay becoming soft to firm with increasing depth. The lower cohesive soil unit was characterized as a grey, moist, highly plastic, soft to firm silty clay.

The consistency of this soil unit varied from soft to very stiff, but was mostly firm to stiff. Undrained shear strength values ranged from 28 kPa to 100 kPa (with an average of 45 kPa). Minimum, maximum, average, and standards deviation of undrained shear strength values are presented in **Appendix B, Table B3**. Insufficient numbers of Standard Penetration tests were carried out in the glacio-lacustrine clay to perform statistical analysis of the results.

6.2.5 Glacial Till Soil

A glacial till deposit was encountered in all test holes located along the NEIS alignment. The glacial till soil unit was recorded with a corresponding thickness of between 0.95 m and 2.05 m below the western riverbank, and between 2.19 m and 2.76 m below the eastern riverbank. The glacial till consisted of sand, with varying proportions of silt, clay and gravel. The glacial till was noted as light brown/tan in colour, damp, loose to very dense, and was of low plasticity to non-plastic. Although not encountered during the advancement of the AECOM 2016 test holes, the glacial till is known to contain cobble and boulder size obstructions.

The compactness of this soil unit varies from loose to very dense. The Standard Penetration Test (SPT) blow counts for 300 mm ranged from 8 to greater than 50 blows. Insufficient numbers of Standard Penetration tests were carried out in the glacial till to perform statistical analysis of the results.

6.2.6 Swelling Potential – Cohesive Soils (Glacio-Lacustrine Clay)

The swelling potential of cohesive soil unit and glacio-lacustrine clay unit was assessed following the criteria presented by Van Der Merwe (1964) and Taylor and Smith (1986) and using the I_P and percentage of clay size particles provided in the GDR. The swelling potential of alluvial cohesive soils and glacio-Lacustine clay was also assessed using the chart provided by Savage (2007) and Atterberg limits of the soils provided in the GDR.

The swelling potential of clay is highest when a sample has a percentage of clay size particles and a high plasticity index (see Section 3.2 of GDR). The estimated swelling potential of the glacio-lacustrine clay unit is considered to have a high to very high potential severity of an expansive soil based on the measured Atterberg limits, I_P and percentage of clay sized particles. Volumetric increases are usually in the 2 percent range with swelling pressure generally less than 75 kPa (Graham and Shields, (1985)).

<u>For baseline purposes related to selection of the overcut and lubricants for pipe jacking operations, the glacio-lacustrine clay unit shall be considered to have a high to very high swelling potential.</u>

6.2.7 Boulders

Cobbles and boulders were not directly observed during the geotechnical investigation (AECOM 2016 and KGS 2012) within the subsurface soils along the NEIS alignment. Geotechnical investigations conducted in the immediate area of the site encountered cobbles and boulders within the glacial till at an approximate elevation of between 209.0 m and 211.0 m.

<u>For baseline purposes related to the excavation of the access shafts, the Contractor shall assume that the glacial till may contain cobbles and boulders, and shall select suitable excavation equipment to progress the shafts through the glacial till soil unit.</u>

6.3 Bedrock Characterization

6.3.1 General

The majority of the bedrock encountered at the site, specifically along the proposed NEIS alignment, consists of dolomitic limestone and limestone. The bedrock surface elevation varied between 209.6 m and 210.3 m along the proposed NEIS alignment. The bedrock is generally white to tan, medium strong to very strong limestone to dolomite. The bedrock units encountered are consistent with geological maps of the area.

Details of bedrock RQD, TCR and UCS are provided in Section 3.2.5 of the GDR.

6.3.2 Rock Quality Designation (RQD)

RQD ranges from 27% to 100% which represents poor to excellent quality bedrock. Lower RQD values were typically found at depths closer to the bedrock surface, but RQD values are typically consistent between an elevation of 210.3 m and 193.3 m. RQD values at each test hole location are shown on **Figure A2** in **Appendix A**. Histogram distributions of RQD within test holes are presented in **Figure 6-1**. It should be noted that RQD classifications from the KGS (2012) geotechnical investigation are not included as these values were not reported.

<u>As a baseline to assist MTBM equipment selection, consider RQD to range from 27% to 100% with an average of 82%.</u>

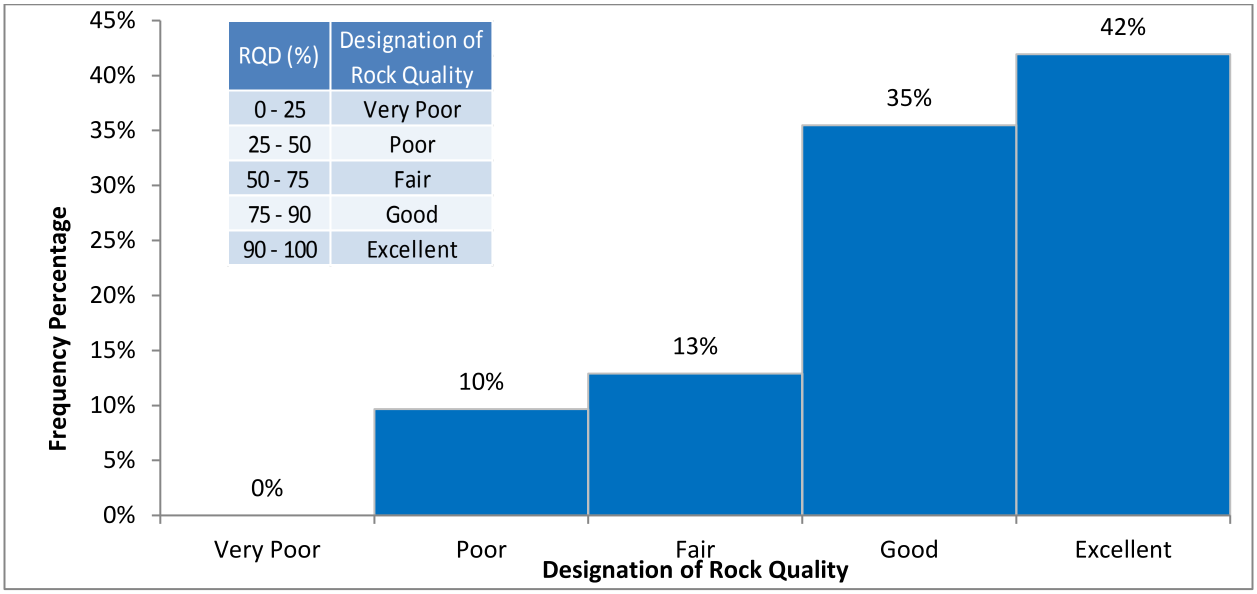


Figure 6-1: Histogram Distribution of RQD within Test holes

6.3.3 Unconfined Compressive Strength (UCS)

UCS testing was performed on samples of limestone and dolomitic limestone from the Red River Formation. The limestone is classified as medium strong to very strong, and the dolomitic limestone classified as strong in accordance with ISRM (1981). The measured UCS values for the limestone range between 39.7 MPa and 149.6 MPa, and between 77.8 MPa and 93.5 MPa for the dolomitic limestone. Histogram distributions of UCS values for the carbonate bedrock are illustrated in **Figure 6-2**. Minimum, maximum, average and standard deviation of measured UCS for samples of limestone are presented in **Table 6-3**.

<u>A baseline UCS of up to 150 MPa should be used for selecting cuttings tools for excavation (tunnels, shafts, etc.) within the bedrock portion including the both the limestone and dolomitic limestone bedrock</u>. <u>A baseline UCS of 40 MPa should be used for the design of tunnels and shafts and open-cut excavation support within the carbonate bedrock for the NEIS alignment.</u>

Table 6-3: Measured UCS Values – Bedrock (Limestone)

Bedrock Type	Minimum (MPa)	Maximum (MPa)	Average (MPa)	Standard Deviation (MPa)
Limestone	39.7	149.6	87.0	48.7
Dolomitic Limestone	77.8	93.5	85.7	-

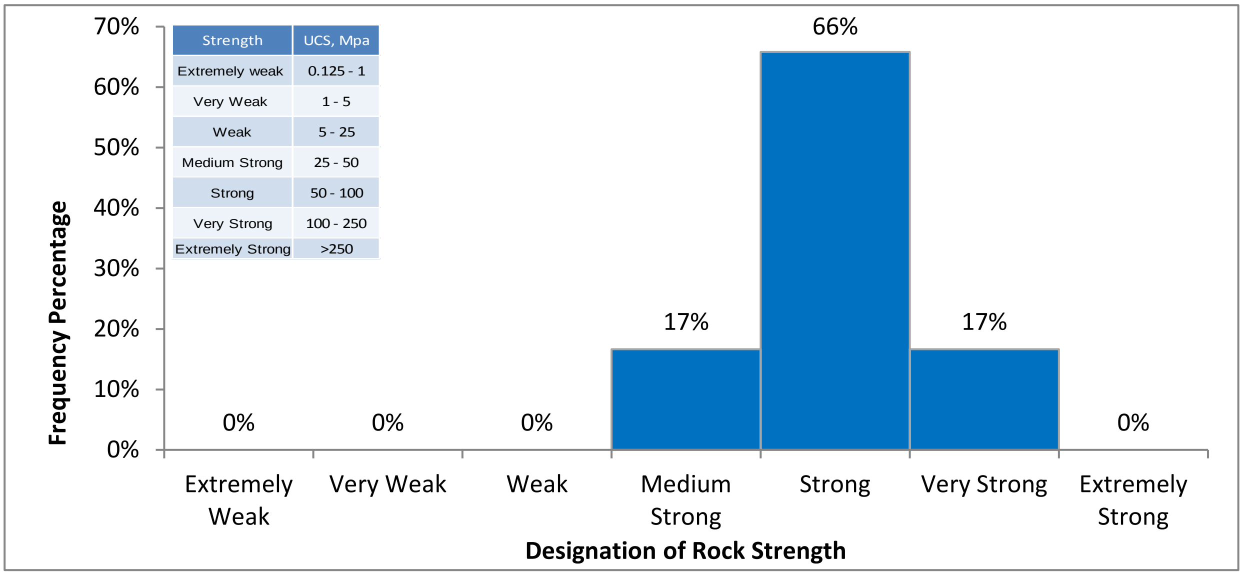


Figure 6-2: Histogram Distribution of UCS – Limestone and Dolomitic Limestone

6.3.4 Excavatability/Rippability of Bedrock – Limestone/Dolomitic Limestone

Excavation of bedrock will be required at the proposed siphon inlet/outlet locations. Rippability of bedrock was assessed using the Kirsten method (Kirsten 1988; ASTM STP 984). Rippability indices for bedrock were estimated using the factors provided in Kirsten (1988). The Rippability index for bedrock at the shaft locations varied from 25 to over 10,000.

<u>Based on the estimated rippability indices, local experience with bedrock excavation and considering the relatively small size of the shaft excavations, an extremely hard ripping or blasting classification (or excavation with break hammer) shall be used as a baseline for bedrock excavation.</u>

6.4 Groundwater Conditions

The relationship between the Red River and the underlying Carbonate Aquifer is very dynamic, and typically the potentiometric surface of the aquifer is above the river level. Aquifer levels in the summer are characterized by a reduction in groundwater levels in the Carbonate Aquifer which tend to fall below the river level. The aquifer begins to rise during the fall as river levels decrease (Render, 1970).

Flood elevations for the Red River are shown in Section 3.3.2 of the GDR.

Groundwater levels fluctuate seasonally, and typically rise during the spring melt and after significant rainfall events and snowmelts.

6.4.1 Site Specific Groundwater Observations

Groundwater elevations were measured in the test holes during and after the completion of AECOM geotechnical investigation. The measured groundwater levels are also presented in Section 3.3.1 (Table 3-19) of the GDR.

Groundwater instrumentation along the NEIS alignment consists of; standpipe piezometers (6), vibrating wire (1) and pneumatic (1) piezometers installed as part of the AECOM 2016 and KGS 2012 geotechnical investigations. Instrumentation was installed into all soil units encountered along the NEIS alignment and the instruments were monitored between the periods of May 2013 and March 2017 by AECOM and others.

Groundwater levels measured within the glacial till and carbonate bedrock are generally consistent with each other, suggesting that there is a hydraulic connection between the two units. Variations in groundwater elevations within the glacial till/bedrock are mirrored within the glacio-lacustrine clay which is consistent with the current understanding of the regional groundwater dynamics that exist between the surficial soils and bedrock units.

There also appears to be a downward hydraulic gradient between the alluvial soils and the river levels which suggest that the more permeable alluvial soils are influenced by river levels in the channel.

The baseline groundwater elevations for shafts, tunnels and open cut sections are presented in **Section 8.2** of this report.

6.4.2 Groundwater Dewatering Rates

As outlined in Section 1.0 of this report, drawdown of the aquifer is not permitted to facilitate shaft construction as part of the project. Therefore the Contractor shall adopt ‘sealed’ methods of shaft construction. Sealed shafts are required to have a concrete base designed to prevent basal heave, resist hydrostatic pressures, and minimize ingress of fines and infiltration of groundwater. However, notwithstanding this requirement, the following provides a concise summary of the hydrogeological assessment undertaken by Friesen Drillers Ltd with respect to groundwater dewatering rates at the site which should be taken in to account as part of the design of sealed shafts.

To determine the required dewatering rates at each shaft location, Friesen Drillers Ltd., performed a hydrogeological assessment to assess local aquifer conditions. The findings and recommendations of this assessment are contained within the January 2018 Hydrogeological Assessment/Aquifer Characterization Report within Appendix F of the GDR.

The field assessment comprised the completion of two (2) short term pump tests at each proposed shaft location, with corresponding groundwater monitoring at nearby observation wells (approximately 30 m to 40 m from the pump test locations). The results of the pumping tests indicate that there are variable conditions across the proposed east and west chambers as reflected by the drawdown rates for each shaft location. The results of the pump tests indicate that prolonged dewatering in order to depressurize the carbonate aquifer could develop a hydraulic connection to the river. As a consequence, this would result in higher discharge rates in order to achieve the necessary drawdown within the carbonate aquifer. The report recommends a pumping rate of between 7.57×10^{-2} m^3/s and 2.78×10^{-1} m^3/s to lower groundwater to an elevation of 201 m.

6.5 Frost Penetration

The expected depth of frost penetration has been estimated assuming a design freezing index of 2680 °C days, taken as the coldest winter over a ten (10) year period. The estimated maximum depth of frost penetration is 2.4 m assuming no insulation cover.

6.6 Frost Susceptibility

The surficial alluvial soils encountered on site are highly frost susceptible. These soils have propensity to grow ice lenses and heave during freezing, and loss strength during thaw. Silts are particularly susceptible to frost action due to their grainsize range. The installation depths for the siphon and or any pipes should be situated below the frost penetration depth. Backfill material should consist of non-frost susceptible granular material.

6.7 Anticipated Ground Behaviour

6.7.1 Overburden

For the description of the anticipated behaviour of the overburden deposits, the Tunnelman Ground Classification System, developed by Terzaghi (1950) and modified by Heuer (1974), has been adopted. It should be noted that the Tunnelman ground classification terms provide a description of the behaviour of the different soil types at an unsupported vertical tunnel face under atmospheric conditions. As the tunnelling is to be constructed using MTBM, the Tunnelman descriptions have only been provided to give a general idea of soil face stability behaviour.

In non-plastic strata (granular soil units), the face stability is assessed based on a consideration of groundwater conditions, soil gradation, grain shape, and in-situ density. The Contractor shall be responsible for establishing operation of the MTBM such that surface settlement remains within limits provided by the Contract Drawings and Specifications.

In plastic soils (alluvial cohesive soil and glacio-lacustrine clay units), the anticipated face stability can be further described by assessing the undrained shear strength of the soil. The soil behaviour/tunnel face stability can be related to the stability number, N_s, where:

$N_s = (\gamma H - P_i)/S_u$

where:

γ is total unit weight of soil;

H is depth to tunnel axis;

P_i is support pressure within tunnel; and,

S_u is undrained shear strength of soil.

Typically, the following general relationships are used based on past experience (Clough and Schmidt, 1981):

$N_s<2$ Small ground movement and shield tunnelling is not required.

$2<N_s<4$ Shield generally used to restrain ground movements.

$4<N_s<6$ Increasing ground movement even with shield tunnelling.

$6<N_s$ Face may be unstable and face support required.

The baseline behaviour of the overburden soil units is presented in **Table 6-4**.

6.7.2 Bedrock

This section describes the anticipated behaviour of the bedrock at an unsupported vertical tunnel face under atmospheric conditions. The following description will apply to sections of shafts in bedrock and will also give a general idea of face stability behaviour in the tunnel sections where bedrock is encountered.

Wedge-shaped blocks will be released and fall into the tunnel excavation under the following conditions: (i) where nearly vertical joint sets intersect the tunnel at a shallow angle in combination with bedding planes and/or weak horizontal seams; and, (ii) where horizontal bedding planes intersect two inclined joints. This type of wedge instability is expected to occur on a localized basis and can be expected to occur at any time following tunnel excavation.

Roof slab fallout can occur in the bedrock where a clay-filled or open, weak horizontal seam is present in the tunnel crown. This type of fallout occurs along the tunnel until the weak seam pinches out or rises sufficiently above the crown.

Table 6-4: Anticipated Behaviour of Soil/Bedrock at Unsupported Vertical Tunnel/ Excavation Face

Soil Group	Soil Type and Description	Anticipated Ground Behaviour
Alluvial Cohesive Soil Unit	Clayey Silt, Silty Clay	Will be stable and exhibit Firm behaviour initially after excavation, but depending on the degree of fissuring will degrade into Slow Ravelling ground both above and below the groundwater table. The silt layers are known to be water bearing and are susceptible to strength loss when subjected to mechanical disturbance and sloughing from wetting. All open excavation side slopes should be covered with water proof material to prevent saturation of the soil and all surface runoff to be directed away from the excavations.
Glacio-Lacustrine Clay (Cohesive) Unit	Silty Clay	The upper layer of the glacio-lacustrine clay will be stable and exhibit Firm behaviour initial upon excavation and quickly in-turn become Slow Ravelling depending upon the degree of fissuring. The lower layer will begin to Squeeze and yield plastically with increased depth upon excavation. The shear strength of both the upper and lower silty clay will progressively decrease over a short period of time due to changes in effective stress and moisture conditions, resulting in Swelling and yielding conditions of the soil if left unsupported.
Alluvial Granular Soil Unit	Sand, Sand and Gravel	Above the groundwater table these soil types will be Fast Ravelling or exhibit cohesive running but will immediately Flow below the groundwater table even under a small groundwater head (< 1 m).
Glacial Till (Granular)	Sandy Silt, Silty Sand	Below the groundwater table, Fast Ravelling to Flowing conditions will occur. Unstable [Running or Flowing] conditions can be expected where cohesionless granular layers or pockets are present in the till. Cobbles and boulders will be encountered.
Bedrock	Competent Limestone/Dolomitic Limestone	The unweathered competent bedrock units will be stable and Firm upon excavation. Fast Ravelling conditions will be encountered depending upon the degree of rock fracturing and discontinuities within the bedrock formation.

7. Previous Tunnel Construction Experience

7.1 General

Tunnel construction experience in the Winnipeg area has been described by KGS (November 2017), and these references are fully listed in Section 2.3 of this GBR. Select case histories which have relevance to the design and construction of the current project, and lessons learned from construction of tunnels in the Winnipeg area are presented in the following sections.

While historically in the City of Winnipeg other forms of trenchless technologies have been used in the installation of buried pipe infrastructure, Micro-Tunnel Boring Machines (MTBM) have been increasingly used. The following project examples are considered relevant to the NEIS project.

7.2 Trunk Sewer & LDS Separation, Contract Two-Cockburn and Calrossie Combined Sewer Relief Project

The project included installation of approximately 1.3 km of 1.2 m diameter land drainage sewer using an MTBM at approximate depths of 8 m to 9 m below grade within the glacio-lacustrine clay. The project is located on Byng Place, Rockman Street, Parker Avenue and Heatherdale Avenue (approximately 600 m south from Taylor Avenue). The work was carried out by Marathon Drilling Co., using a Herrenknecht AVN1200 MTBM.

The shaft construction was completed using steel sheet piles and walers. The sheet piling was vibrated through the glacio-lacustrine clay to an approximate depth of 12 m below grade.

Constructions Issues

- During shaft construction and pipe installation, the following issues were encountered;
  - Unacceptable transfer of vibrations through the glacio-lacustrine clay that negatively impact adjacent structures.
  - The separation plant was unable to effectively separate the clay particles from the water. This resulted in a mud spoil too wet to be hauled off-site due to excessive moisture. Reportedly drier material was added to the spoil to allow for disposal. Marathon Drilling Co., modified the separation plant to optimize the water return. Accordingly modifications to the separation ratios were not successful (which included replacing finer screens with coarser screens) resulting in an excess of excavated material entering the recovery tank and increasing the chute size on the shaker deck. One partially successful solution consisted of the application of sprayer bars to force material through the screens and adding a scalping belt was considered.

7.3 Trunk Sewer & LDS Separation, Contract Four-Cockburn and Calrossie Combined Sewer Relief Project

Contract 4 of the project (Cockburn and Calrossie Combined Sewer Relief) included the installation of approximately 525 m of 2.7 m diameter land drainage sewer by trenchless installation methods. The land drainage sewer was installed within the glacio-lacustrine clay at approximately 8.0 m to 8.5 m below grade. The project is located on land adjacent to Manitoba Hydro and Shindico property along Wilton Street from the north side of Taylor Avenue. The work was carried out by Ward and Burke Microtunnelling. Ltd., using a Herrenknecht AVN2500 MTBM. The MTBM was ‘up-skinned’ to match the outside diameter of the 2.7 m reinforced concrete jacking pipe.

The use of dual centrifuges to remove the clay from the slurry was deemed effective comparatively to the slurry separation used within Contract 2. Lessons learned from Contract 2 of the Cockburn and Calrossie combined sewer relief project (see Section 7.2 of this report) were successfully implemented to mitigate the effects vibration as a result of the caisson installation.

During the tunnelling process, it was stated that a correlation was observed between the face pressure maintained at the MTBM and recorded surface settlements. Where the machine face pressure was near zero (prior to crossing the CN right-of-way), measured surface settlement along the centreline of the alignment exceeded the settlement tolerances (greater than 25 mm) for the project. Upon reassessment by the MTBM Contractor, an average face pressure of 55 kPa was maintained for the remaining drive length, and the initial settlement values were reduced by approximately 50 percent. Face pressure was increased by pumping bentonite slurry to the machine and tunnel to fully charge the annular overcut.

It was stated that contact grouting of the tunnel annulus was highly effective in restoring the surface to pre-tunnelling elevations. It is understood that grout port spacing of 15 m (every 5 pipes) for lubrication was used during the tunnelling process. However, it was reported that the bentonite lubrication initially used was not viscous enough to be displaced through the subsequent set of lubrication ports. Higher grouting pressures were sufficient to result in surface cracking indicating ground heave. This is a result of the friction force being too high, slurry mixture being too thick or the wide spacing of the lubrication ports. Accordingly grout bulkheads were created along the north and south sides of the CN crossing to create a seal and maintain the stabilized annular pressure under each set of the railway tracks. Upon reduction of the lubrication density during the second tunnelling drive, measured surface settlements along the centreline rebounded during contact grouting to 2 mm from the baseline readings.

The access shafts consisted of cast-in-place reinforced concrete caissons with sacrificial steel cutting shoes. It is understood that the shafts were installed by excavating the soil within the caisson as sinking occurred under the self-weight of the cutting shoe and concrete wall to depths of between 9 m and 9.5 m. Installation of the cast-in-place caissons was achieved without additional point or vibratory loads to sink the shafts. The report states that negligible vibrations were produced during the shaft installation.

7.4 Lessons Learned from Previous Tunneling Projects

Using the case histories from the projects above has permitted preparation of the following key lessons from previous tunnelling projects in Winnipeg.

- Adequate geotechnical investigation in accordance with the ASCE guidelines is critical for the successful completion of tunnelling projects, and an inadequate number and depth of test holes may result in significant delays and claims for differing ground conditions.

- Sufficient testing should be conducted on soil/bedrock samples to enable proper estimation of the parameters required for the design of tunnel and shafts, and selection of the MTBM and suitable tunnel support system. Inadequate soil/bedrock testing may result in a conservative design, or selection of MTBM and shaft system that are not suited to the soil/bedrock conditions.
- Groundwater inflow rates in the tunnel and shafts should be estimated by pumping tests or hydraulic conductivity testing, undertaken as part of the geotechnical and hydrogeological investigation.
- Selection of a suitable MTBM for soil/bedrock conditions is critical for the successful completion of tunnelling projects. The MTBM should be equipped with a combination of cutting tools for tunnels passing through variable subsurface conditions.
- Unsuitable applied face pressures during the tunnelling process will result in ground settlement within the glacio-lacustrine soils.
- Lubrication port spacing and bentonite lubrication mix design should be given extra considerations when working in the glacio-lacustrine soils which are considered to have a high stickiness potential.
- Contact grouting is an effective means in restoring the ground surface elevation to pre-tunnelling conditions.
- Suitable separation plants should be designed for clay soils.
- Vibrating loading does not quickly dissipate within the glacio-lacustrine soils (i.e., silty clays) and can result in structural damage to adjacent structures. Installation methods for sheet piling should be critically assessed and adequate vibration monitoring programs should be implemented to assess vibration at specific distances relative to the location of sheet piles.
- The concrete caisson shaft design and self-sinking installation methodology produced negligible vibrations through the glacio-lacustrine soils and was comparatively non-intrusive to the surrounding environment.

8. Design and Construction Considerations

8.1 General

General design and construction considerations which are applicable to all shafts and tunnel sections are provided in **Sections 8.2** and **8.3**.

8.2 Launching and Receiving Shafts

- The Contractor is responsible for the design of temporary support systems considered necessary for shafts in accordance with the Contract Documents.
- Two (2) shafts are planned for construction as part of the proposed NEIS tunnel section (STA. 1+288.6 m and STA. 1+539.7m). The launching and receiving shafts shall be located on the western and eastern side of the Red River, respectively. The shafts shall be large enough to accommodate launching and retrieving of the MTBM, while providing space required for the siphon construction as per Contract Drawings.
- Shafts will be used to launch and/or retrieve the MTBM and provide access and space for construction of the tunnel and permanent structures within the shafts. The shafts will be constructed in a combination of soil and bedrock.
- Due to proximity of buildings and utilities, use of temporary shoring will be required to support the excavation walls without impacting the adjacent structures.
- There are sanitary, water, storm, gas and electric lines and industrial buildings adjacent to the manhole location. Ground movements are anticipated around the trench excavation; therefore, the Contractor shall make an assessment of the potential adverse impacts and, where necessary, adopt suitable measures to prevent any damage to the utilities (underground and overhead) and buildings.
- The approximate depths (from top of the chamber wall to the top of the base slab) of the outlet shafts to be constructed ranges from 23.8 m to 24.1 m below ground surface.
- The anticipated behavior of each type of soil/bedrock to be encountered is provided in **Table 6-4** of this GBR.
- The baseline grain size distribution envelopes for the alluvial sand soil unit that will be encountered at the location of shafts are shown on **Figure C1** presented in **Appendix C** of this GBR.
- The baseline UCS for bedrock is provided in **Table 6-3** and rippability/excavatability indices for bedrock excavation is provided in **Section 6.3.4** of this GBR. The Contractor shall consider the UCS and rippability index of bedrock for selecting equipment for bedrock excavation.
- For each shaft location, baseline elevations, and percentage volume of each soil unit is presented in **Tables D1** and **Table D2** in **Appendix D** of this GBR.
- Temporary support and protection of the bedrock within the excavation should be provided as soon as possible after exposure in order to protect the bedrock from weathering, deterioration and spalling.

Seepage at joints in the bedrock is expected and should be controlled by relief of hydrostatic pressure build-up behind the temporary support.

- Temporary support systems are required to be designed for lateral earth pressure, lateral hydrostatic pressure, surcharge of equipment adjacent to the shaft, and should be capable of controlling ground movement in accordance with the Contract Documents. Shaft walls and base slab need to resist uplift forces due to buoyancy, and adequate foundation details should be provided to prevent ground instability due to soil piping and basal heave.
- For rigid shoring systems and circular shafts, the at-rest earth pressure, K_0, must be used for the design (refer to **Figure E-1** in **Appendix E**). All shoring designs should be in accordance with the 4th Edition of the Canadian Foundation Engineering Manual and must be reviewed by the design engineers. Surface surcharges from construction activities must be accounted for in the shoring design. If shoring is to be carried out over the winter months or if the excavation is to be left open for any period during below zero temperature, shored walls must be protected against frost penetration by means of insulation or heated hoarding. The drilling contractor should account for potential for presence of obstruction in the till layer and at the bedrock surface when installing the shoring system. Cobbles and boulders are frequently encountered in the till layer above the bedrock.
- The construction of the shafts by “sealed” construction methods is mandated for all of the shafts as stipulated in the Contract Documents. The Contractor is required to submit their methods of designing and constructing a sealed shaft temporary support system to the Consultant for review with respect to meeting the performance requirements defined in the Contract Documents.
- The Contractor shall be prepared to collect and discharge potential seepage within the shafts and meet the discharge requirements indicated in the Contract Documents.
- There is the potential for boulders within the glacial till soil units and competent bedrock within the shaft excavations. It will be necessary to use equipment that is robust enough to deal with these conditions during shaft excavation and shaft wall construction.
- The sealed shaft wall system selected by the Contractor shall be designed and constructed to allow for the entry and exit of the MTBM. This typically requires the incorporation of a “soft eye” reinforced with materials that can be cut by the MTBM along with a tunnel eye sealing system that prevents soil and groundwater ingress during MTBM breakout or breakthrough.
- The zone located outside of the shaft wall system at the break-in and break-out penetrations shall create a watertight zone where the MTBM can develop or dissipate earth pressure in the forward chamber of the MTBM and allow penetration through the shaft “soft eye”.

8.3 Tunnels

- The Contract Documents require the Contractor to design the jacking pipes and construct the tunnel using a MTBM which is capable of providing face support, installing and jacking pipes from the launching shaft immediately behind the MTBM.
- MTBM’s are to be used for the entire NEIS alignment in bedrock to install the 900 mm carrier pipe or casing pipe, under the Red River in accordance with the Contract Documents.
- The anticipated face stability behavior of each soil unit to be encountered is provided in **Table 6-4** of this GBR.
- **Figure A2** presented in **Appendix A** of this GBR provide the baseline interpreted ground conditions for the tunnel alignment only.

- The cutter head should be designed to breakdown boulders and cobbles into fragments that are easily ingestible by the conveyance system (screw convey, slurry lines, etc.) or easily broken by a rock crusher.
- The MTBM is required to be utilised in conjunction with jacking pipe that provides full ground support over the entire excavated length of tunnel.
- Where the tunnel will be excavated in bedrock, the MTBM should be capable of boring through medium strong to very strong carbonate bedrock.
- Watertight techniques are required to install the 900 mm siphon or casing pipes in accordance with the Contract Documents, and this shall prevent significant groundwater inflow. Local dewatering or compressed air may be required to provide access to the face of the MTBM for maintenance, change of cutters, etc.
- The groundwater flow into the tunnel should be collected and discharged according to the requirements indicated in the Contract Documents.
- Contact grouting shall be used to completely fill the annulus between the ground and the lining to provide ground support and reduce ground settlement. Grouting should be done immediately upon completion of each drive. To minimize surface settlement, all voids behind the lining must be completely filled with grout so that the tunnel lining is in direct contact with the ground.
- During microtunnelling operations, bentonite or other suitable lubricating fluid should be used in the annular gap surrounding the pipe to minimize ground deformation and buildup of soil friction.
- To maintain face stability during excavation and avoid ground loss at the face it is essential that the chamber pressure is maintained within an acceptable range. Further, it is essential to ensure that the forward progress of the machine matches to the amount of excavation being removed from the chamber.
- The tunnel sections for the stub connections are short and are approximately 4.1 m and 6.2 m in length. Suitable trenchless methods shall be considered for installation of pipes at these locations.

8.4 Geotechnical Design Parameters

The geotechnical design parameters provided in **Table 8-1** were interpreted from laboratory and field test results presented in the GDR and can be used to assist the Contractor with design of the jacking pipes, shaft support system and open-cut excavation support system. Earth pressure distributions for temporary shoring design are provided in **Appendix E** of this GBR.

Table 8-1: Recommended Geotechnical Design Parameters for Overburden Soils and Bedrock

Type of Ground	γ (kN/m^3)	E (MPa)	μ	ϕ' (deg	c' (kPa)	UCS (MPa)	S_u (kPa)	K_a	K_p	K_o
Alluvial Granular Soil Unit	18	15	0.33	24	0.0	-	-	0.42	2.38	0.59
Alluvial Cohesive Soil Unit	18	10	0.33	24	0.0	-	30.0	0.42	2.38	0.59
Glacio-Lacustrine Soil Unit	17	7.5	0.33	14	5.0	-	60.0 (to an elevation of 220.0 m) 40.0 below an elevation of 220.0 m.	0.61	1.64	0.76
Glacial Till Unit	19	40	0.33	28	0.0	-	-	0.36	2.78	0.53
Bedrock – Limestone/Dolomitic Limestone	23	1000	0.20	-	-	40	-	-	-	-

γ = bulk unit weight, E = elastic modulus; μ = Poisson's Ratio, ϕ' = effective friction angle; K_a = active earth pressure co-efficient, and K_p = passive earth pressure co-efficient, K_o = at-rest earth pressure co-efficient

8.5 Temporary Excavations

Temporary excavations will be required to facilitate the construction of the proposed NEIS alignment. All excavation work will be required to be performed in accordance with the Workplace Safety and Health Act and Part 26 of the Manitoba Workplace Safety and Health Regulation M.R. 217/2006.

Excavations performed adjacent to the existing bridge (approach fil and roadway) or associated infrastructure will require temporary shoring or bracing as outlined in Section 8.2 of this Report. Excavations deeper than 1.5 m are required to be designed and approved prior to construction by an experienced Professional Engineer with an expertise in Geotechnical engineering. The shoring design should account for all applicable surcharge loads. Opening and voids behind shoring lagging or sheet piles shall be backfilled with cement grout.

8.6 Impact on Existing Structures

Some degree of settlement, heave, and lateral movement will be an inevitable consequence of the construction of the shafts, tunnels, and there will also be some movement of adjacent structures and utilities. The Contractor shall undertake construction in a fashion which maintains movements of utilities and structures within acceptable pre-defined limits, shown on Contract Drawings, to ensure there will be no adverse impacts or damage to the adjacent infrastructure.

MTBM's are to be used for installation of the 900 mm siphon pipe or casing pipe. Minor ground loss can be expected at the face of the MTBM and from some convergence of the soil into the annular void surrounding the trailing pipes and this will cause some degree of ground movements and settlements longitudinally and transverse to the direction of tunnelling.

Suitable trenchless methods approved by the Consultant may also be used on the east and west sides of the Red River where stub connection lengths are less than 6.2 m between the shafts. Ground loss should be expected at the tunnel face and from some ground convergence into the annular space between the steel casing and excavated tunnel walls. This may cause some degree of ground movements and settlements longitudinally and transverse to the direction of tunneling. The Contractor shall select a suitable trenchless method at these locations to reduce the potential for ground movements.

To ensure that ground movements, settlements, and movements of adjacent utilities and buildings are maintained within acceptable limits it is expected that the Contractor will adopt the following measures:

1. Maintain the clearances indicated in the Contract Documents when tunnelling below or adjacent to utilities, buildings and the Red River.
2. Minimise the magnitude of ground loss due to microtunnelling and other trenchless operations by:
   - Utilising an appropriate MTBM;
   - Utilizing appropriate trenchless methods for two tunnel sections required for the stub connections on east and west sides of the Red River where tunnel lengths are less than 6.2 m;
   - Using experienced MTBM operators who will carefully control machine operating parameters for optimum results;
   - Limit the degree of radial overcut;
   - Fill the annulus with bentonite lubricant during microtunnelling operations, and with cement grout immediately following completion of the tunnel drive;
3. The Contractor should be highly experienced to avoid improper operation of the tunneling machine; and,
4. Install and monitor the instrumentation shown on the Contract Documents and undertake investigation of MTBM operation and adopt suitable corrective measures in the event that instrumentation readings equal or exceed pre-defined alert levels.

8.7 Groundwater Management and Spoil Disposal

The Contractor shall be familiar with local spoil disposal regulations, and include the cost of all monitoring, testing, analyses, permits, and treatment necessary to meet the disposal guidelines as part of the Tender.

The Contractor’s Environmental Construction Operations (ECO) Plan shall provide the methodology for managing impacted soils and groundwater, if encountered. The Contractor shall be responsible for managing and discharging groundwater in accordance with the applicable City of Winnipeg By-Laws and applicable provincial and federal regulatory requirements.

In order to maintain safe working conditions in the excavation and to protect against instability of the excavation base, water shall not be allowed to accumulate anywhere within the excavations. Effective drainage and sump pump systems will be required below the base of the excavation to maintain a firm, dry working surface. The Contractor shall design the internal drainage system to efficiently collect groundwater seepage and all water inflow draining in to the excavation shall be pumped out and treated or use a watertight concrete slab designed to resist uplift. The Contractor shall be responsible to prevent surface runoff from entering the excavation.

Disposal of all construction generated spoil shall conform to all applicable laws and regulation and be in accordance with Contract requirements.

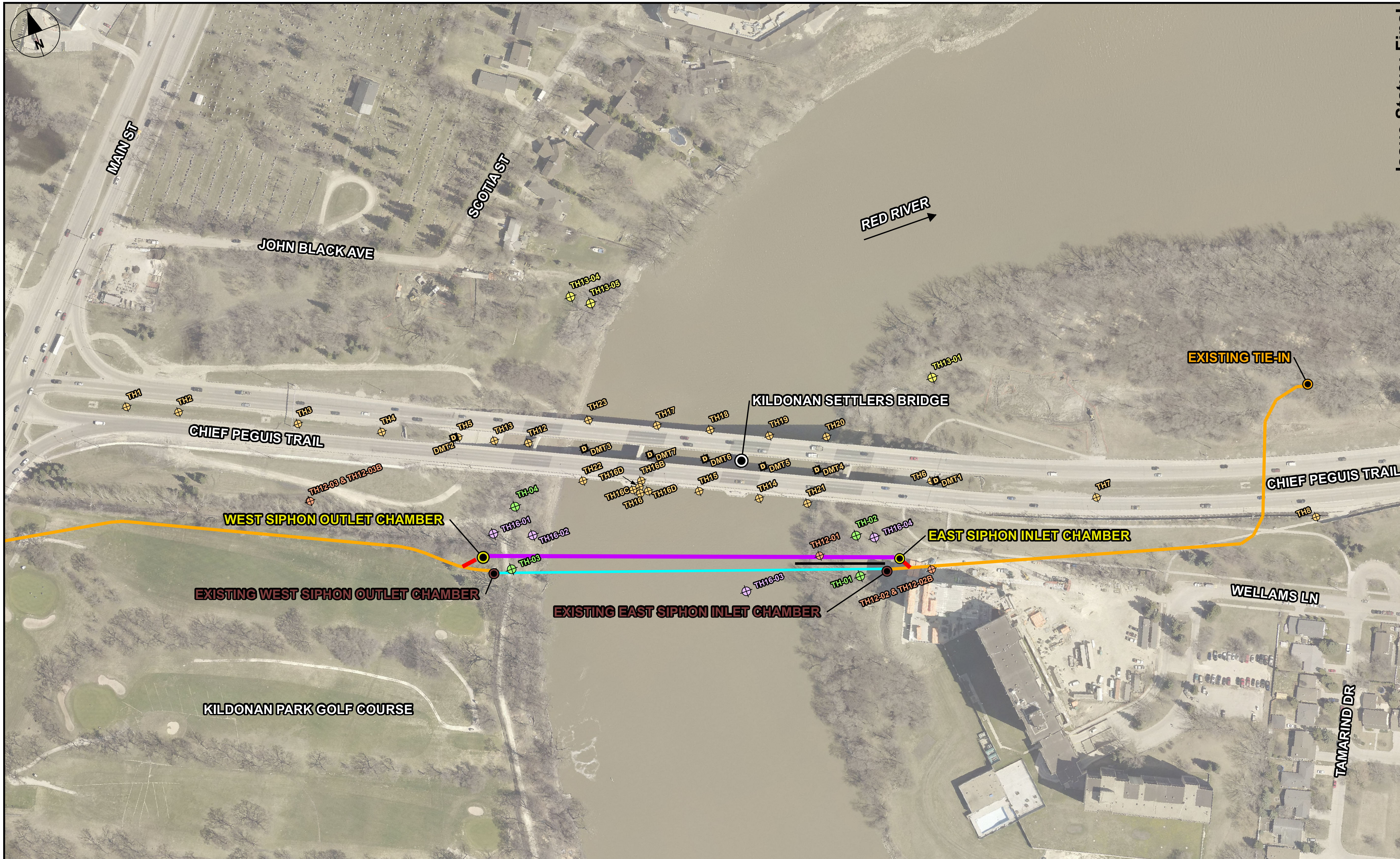
9. Instrumentation Program

The potential impact of tunnel construction on adjacent structures should be monitored and instrumentation designed for the project location to monitor ground movements, settlement of any structures within the zone of influence, tunnel convergence, ground vibration, and level of noise. Details of instrumentation design, Review Level and Alert Level and amount of displacement/distortion that necessitate response for each level are provided in the Contract Documents (if any).

Appendix A

Figures

- Figure A1: Site Location Plan and NEIS Alignment
- Figure A2: Baseline Ground Conditions along NEIS Alignment



LEGEND

- NEIS ALIGNMENT (MICROTUNNELING)
- CONNECTION TO EXISTING 1800 MONO CONC INTERCEPTOR
- EXISTING 1800 MONO CONC INTERCEPTOR
- EXISTING 1200 CSP CS OUTFALL
- EXISTING 500 & 800 STEEL SIPHON
- TEST HOLE (DYREGROV, 1988)
- DILATOMETER TESTING (DYREGROV, 1998)
- TEST HOLE (KGS, 2012)
- TEST HOLE (TREK, 2013)
- TEST HOLE (AECOM, 2016)
- TEST HOLE (FRIESEN DRILLERS LTD., 2017)

NOTE: LOCATION OF DYREGROV TEST HOLES AND DILATOMER ARE APPROX.

20 0 20 40 m

1:2,250

NAD 1983 UTM Zone 14N

AECOM

FIGURE: A1

SITE LOCATION PLAN AND NEIS ALIGNMENT

NORTHEAST INTERCEPTOR
GEOTECHNICAL BASELINE REPORT
CITY OF WINNIPEG, WATER AND WASTE DEPARTMENT

ANSI B 279.4mm x 431.8mm

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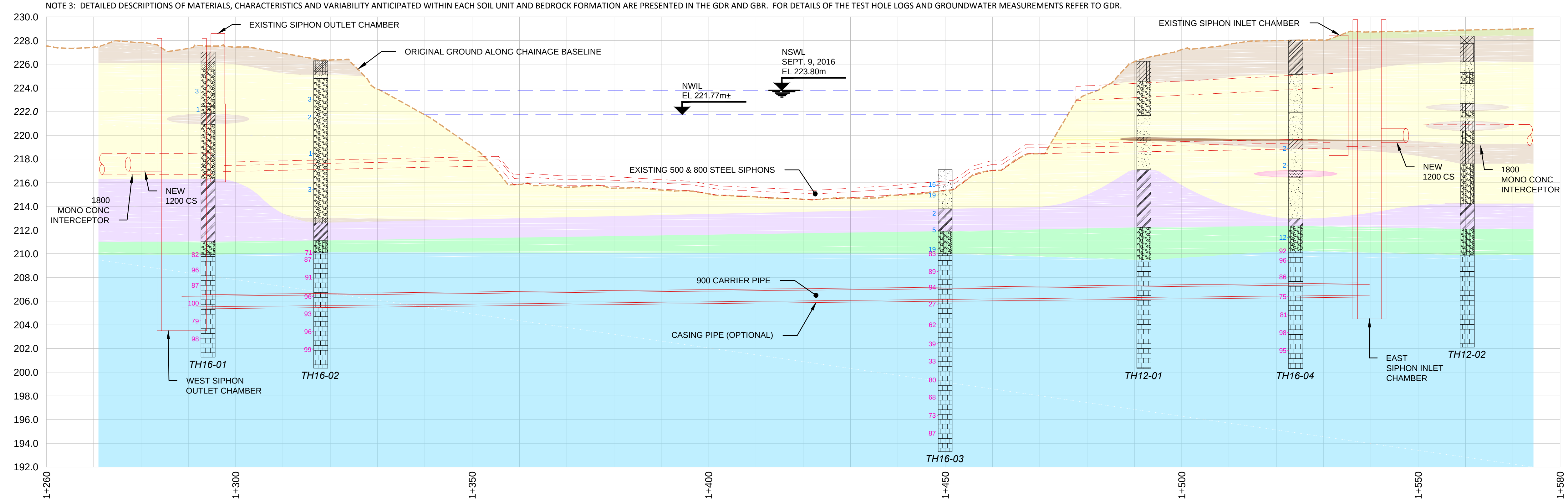
LEGEND

- COHESIVE FILL
- SAND
- SAND & GRAVEL
- SILTY SAND
- LIMESTONE BEDROCK
- ORGANIC SOIL
- SILTY CLAY/ CLAYEY SILT
- INORGANIC SILT
- INORGANIC HIGH PLASTIC CLAY
- CLAYEY SAND
- INORGANIC MEDIUM PLASTIC CLAY
- COHESIVE FILL (CLAY FILL)
- GRANULAR ALLUVIAL SOILS (SILTY SANDS/ SANDS/ GRAVELS)
- COHESIVE ALLUVIAL SOILS (SILT/ CLAY)
- GLACIO-LASCUSTRINE CLAY
- CARBONATE BEDROCK (LIMESTONE/ DOLOMITE)
- GRANULAR GLACIAL TILL
- ORGANICS
- 66 SPT (N) VALUES
- 100 RQD VALUES

NOTE 1: THIS FIGURE SHOULD BE USED FOR BASELINE PURPOSES ONLY AND SHOULD BE READ IN CONJUNCTION WITH THE GEOTECHNICAL BASELINE (GBR) AND DATA REPORT (GDR). THIS FIGURE PROVIDES BASELINE STRATIGRAPHIC CROSS SECTION ALONG THE TUNNEL SECTIONS ONLY. FOR BASELINE STRATIGRAPHIC CONDITIONS AT THE SHAFT LOCATIONS, REFER TO GBR.
NOTE 2: SUBSURFACE CONDITIONS ARE KNOWN ONLY AT THE TEST HOLE LOCATIONS. GROUND CONDITIONS BETWEEN TEST HOLES ARE INFERRED AND SIMPLIFIED. THE ACTUAL GROUND CONDITIONS BETWEEN THE TEST HOLES MAY VARY FROM THE INFERRED CONDITIONS.
NOTE 3: DETAILED DESCRIPTIONS OF MATERIALS, CHARACTERISTICS AND VARIABILITY ANTICIPATED WITHIN EACH SOIL UNIT AND BEDROCK FORMATION ARE PRESENTED IN THE GDR AND GBR. FOR DETAILS OF THE TEST HOLE LOGS AND GROUNDWATER MEASUREMENTS REFER TO GDR.



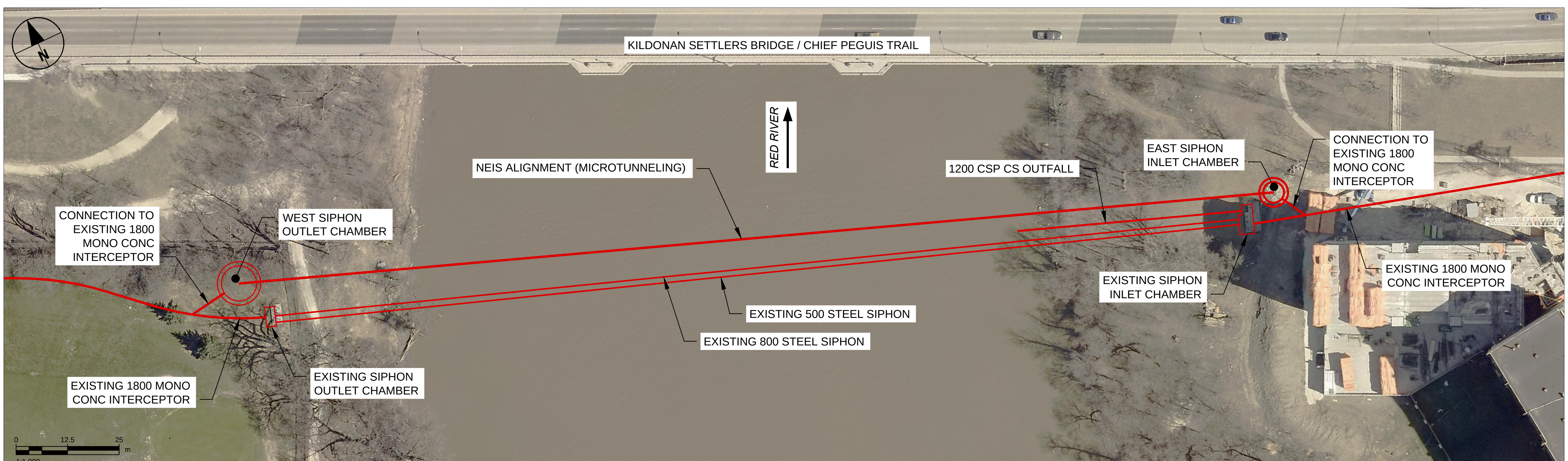


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FIGURE: A2

BASELINE GROUND CONDITIONS ALONG NEIS ALIGNMENT

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Appendix B

SPT N Values and Undrained Shear Strength- Variation and Histogram Distribution

- Table B1 and Figure B1: Variation and Histogram Distribution of Undrained Shear Strength for Alluvial Clay or Alluvial Silt Soil
- Table B2 and Figure B2: Variation and Histogram Distribution of SPT N Values for Alluvial Sand Soil Unit
- Table B3 and Figure B3: Variation and Histogram Distribution of Undrained Shear Strength for Glacio-Lacustrine Soil Unit

Table B1: Variation of Undrained Shear Strength for Alluvial Clay or Alluvial Silt Soil

Undrained Shear Strength (kPa)			
Minimum	**Maximum**	**Average**	**Standard Deviation**
5.6	84.0	58	31.4

Figure B1: Histogram Distribution of Undrained Shear Strength Values for Alluvial Clay or Alluvial Silt Soil

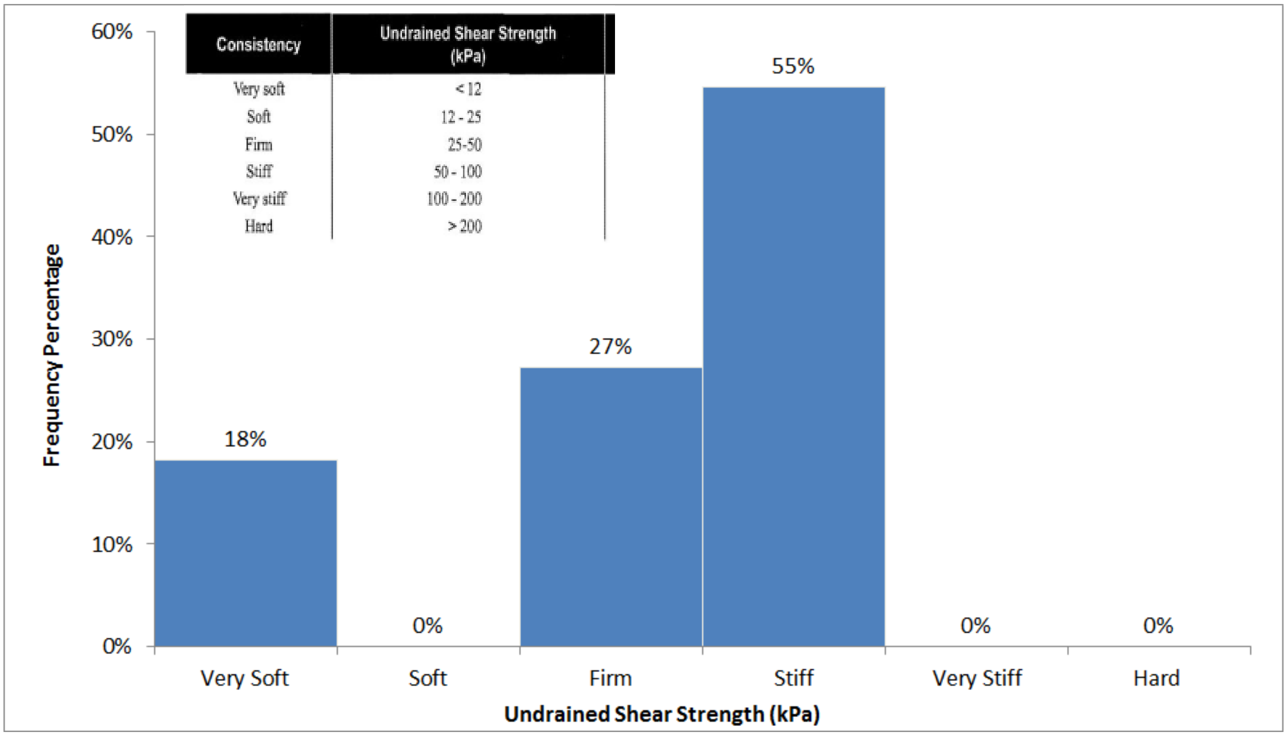


Table B2: Variation of SPT N Values for Alluvial Sand Soil

Undrained Shear Strength (kPa)			
Minimum	**Maximum**	**Average**	**Standard Deviation**
1	19	6	7

Figure B2: Histogram Distribution of SPT N Values for Alluvial Sand Soil

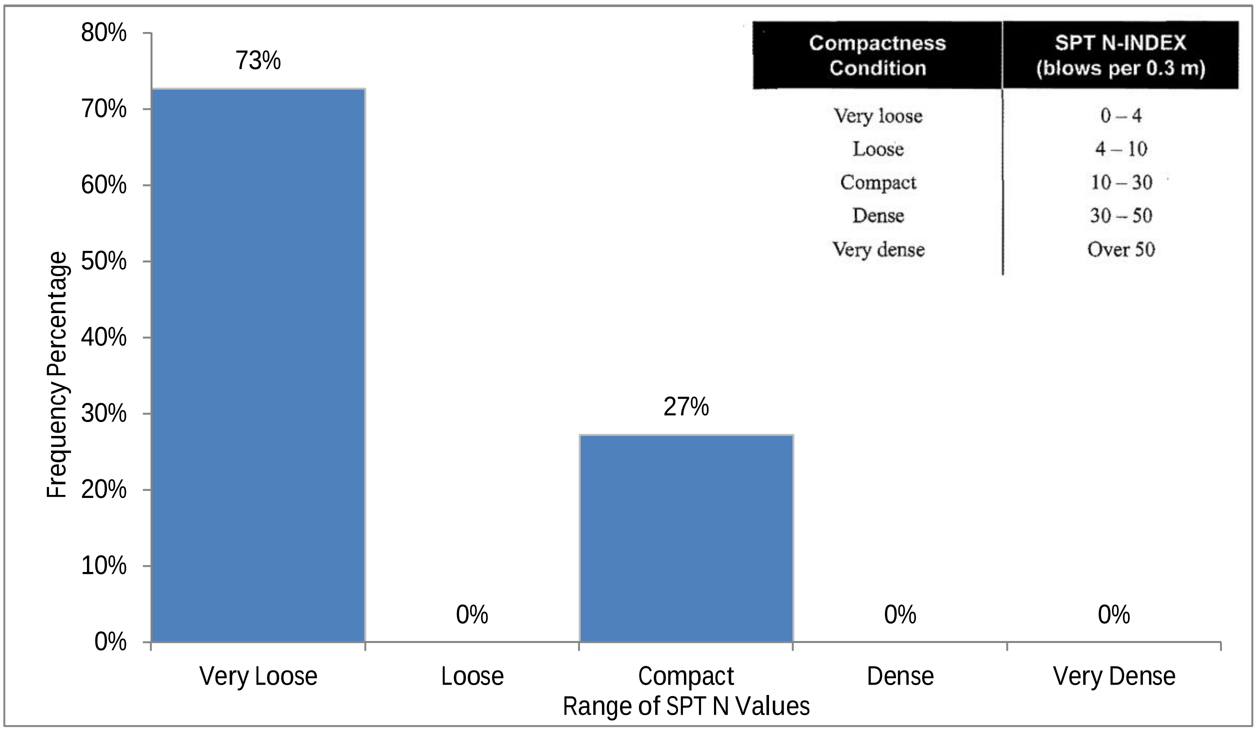
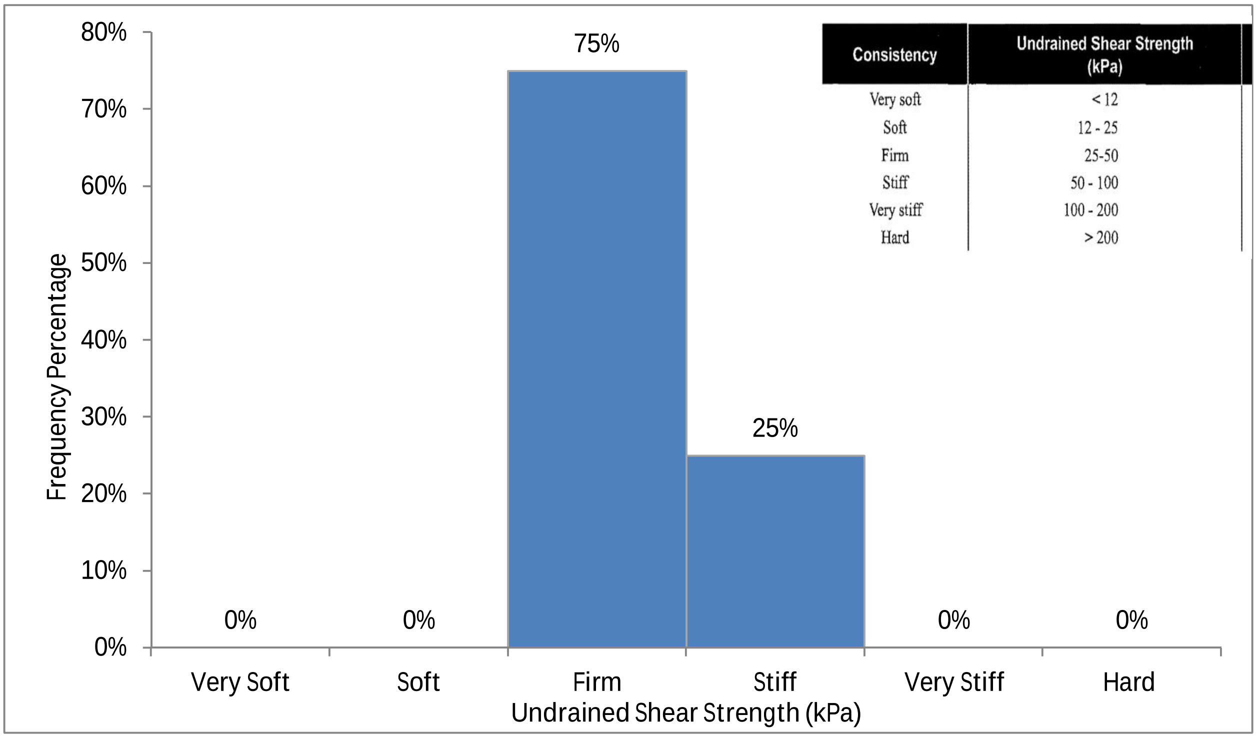


Table B3: Variation of Undrained Shear Strength for Glacio-Lacustrine Clay Soil Unit

Undrained Shear Strength (kPa)			
Minimum	**Maximum**	**Average**	**Standard Deviation**
28.0	100.0	45.1	22.7

Figure B3: Histogram Distribution of Undrained Shear Strength Values for Glacio-Lacustrine Clay Soil Unit



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Appendix C

Baseline Grain Size Distribution Envelopes

- Table B1: Baseline Grain Size Distribution Envelope for Alluvial Sand Soil Unit

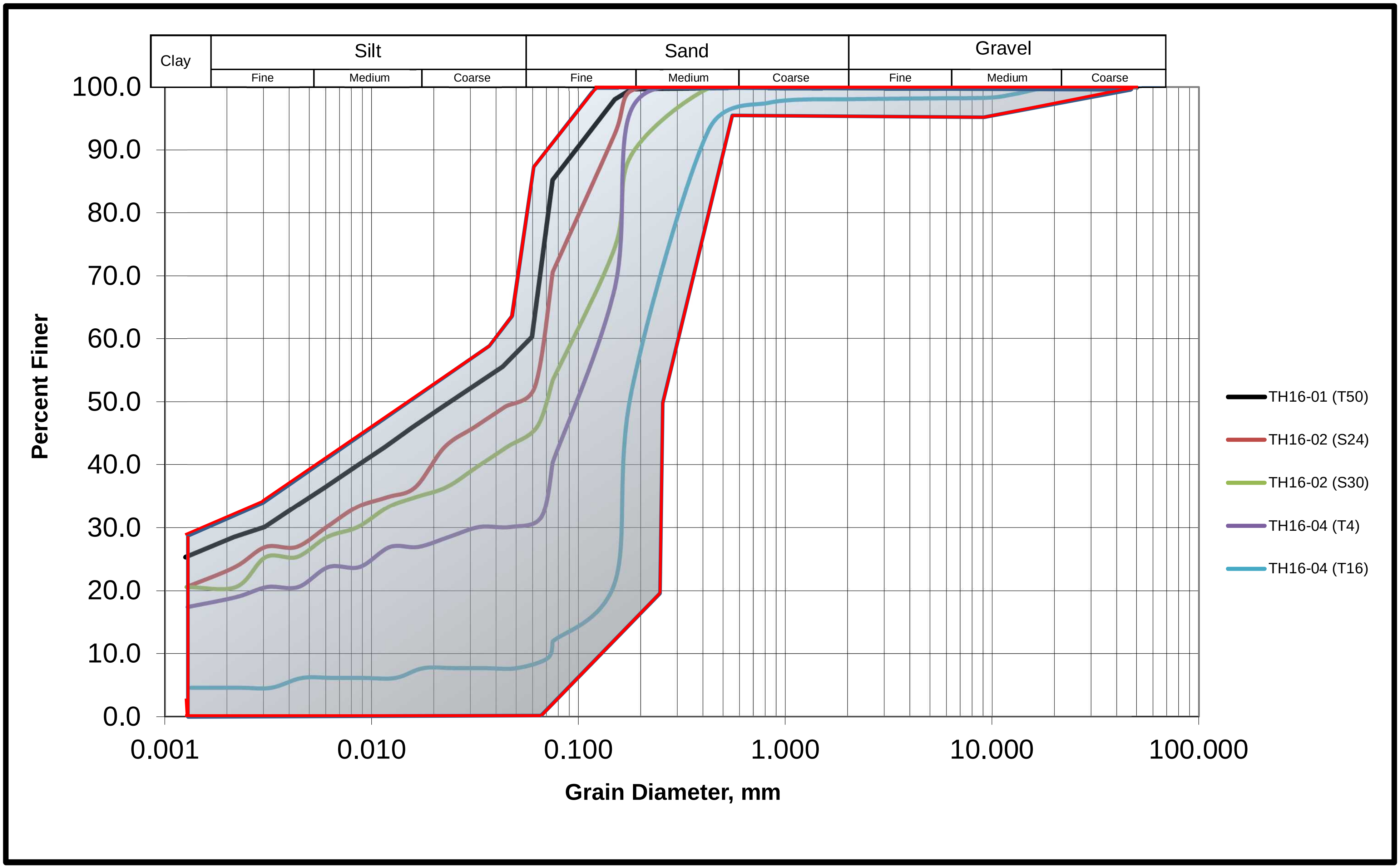

Figure C1: Baseline Grain Size Distribution Envelope for Alluvial Sand Soil Unit

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Appendix D

Baseline Ground Conditions at Siphon Outlet/Inlet Chambers

- Table D1: Baseline Elevations of Expected Soil Units- Eastern Siphon Inlet Chamber
- Table D2: Baseline Elevations of Expected Soil Units- Western Siphon Outlet Chamber

Table D1: Baseline Elevations of Expected Soil Units- Eastern Siphon Inlet Chamber (Sta.1+288.61)

Soil Unit	Parameter	Baseline
Cohesive Clay Fill	Top Elevation (m)	228.37
	Bottom Elevation (m)	228.05
	Total Thickness (m)	0.32
	% of Shaft Volume (before excavation)	1.3
Cohesive Alluvial Clay or Alluvial Silt	Top Elevation (m)	228.05
	Bottom Elevation (m)	225.15
	Total Thickness (m)	2.86
	% of Shaft Volume (before excavation)	12.0
Cohesionless Alluvial Sand Soil*	Top Elevation (m)	225.15
	Bottom Elevation (m)	212.95
	Total Thickness (m)	12.20
	% of Shaft Volume (before excavation)	51.2
Cohesive Glacio-Lacustrine Soil	Top Elevation (m)	212.95
	Bottom Elevation (m)	212.35
	Total Thickness (m)	0.60
	% of Shaft Volume (before excavation)	2.5
Granular Glacial Till	Top Elevation (m)	212.35
	Bottom Elevation (m)	210.25
	Total Thickness (m)	2.10
	% of Shaft Volume (before excavation)	8.8
Carbonate Bedrock	Top Elevation (m)	210.25
	Bottom Elevation (m)	204.50 (1)
	Total Thickness (m)	5.75
	% of Shaft Volume (before excavation)	24.2

Shaft Top Elevation taken at Chamber: 228.30 m; Shaft Bottom Elevation: 204.50 m; *- Soil Unit is interbedded with a 3.2 m thick cohesive alluvial soil layer between 216.5 m and 219.7 m. An organic layer of 0.60 m thickness is present at an approximate elevation of between 216.50 m and 217.10 m; (1)- Full thickness of carbonate bedrock not proven.

Table D2: Baseline Elevations of Expected Soil Units- Western Siphon Outlet Chamber (Sta.1+539.70)

Soil Unit	Parameter	Baseline
Cohesive Clay Fill	Top Elevation (m)	227.50
	Bottom Elevation (m)	225.50
	Total Thickness (m)	2.00
	% of Shaft Volume (before excavation)	8.30
Cohesive Alluvial Clay or Alluvial Silt	Top Elevation (m)	225.50
	Bottom Elevation (m)	216.40
	Total Thickness (m)	9.10
	% of Shaft Volume (before excavation)	37.9
Cohesionless Alluvial Sand Soil*	Top Elevation (m)	216.40
	Bottom Elevation (m)	211.00
	Total Thickness (m)	5.40
	% of Shaft Volume (before excavation)	22.5
Granular Glacial Till	Top Elevation (m)	211.00
	Bottom Elevation (m)	209.90
	Total Thickness (m)	1.10
	% of Shaft Volume (before excavation)	4.60
Carbonate Bedrock	Top Elevation (m)	209.90
	Bottom Elevation (m)	203.50
	Total Thickness (m)	6.40
	% of Shaft Volume (before excavation)	26.7

Shaft Top Elevation taken at Chamber: 227.60 m; Shaft Bottom Elevation: 203.50 m; *- Soil Unit is interbedded with a 0.9 m thick cohesive alluvial soil layer between 220.9 m and 221.8 m; (1)- Full thickness of carbonate bedrock not proven.

Appendix E

Earth Pressure Distribution for Temporary Shoring

- Table E1: Earth Pressure Distribution for Temporary Shoring- Granular Alluvial Soil Unit
- Table E2: Earth Pressure Distribution for Temporary Shoring- Cohesive Alluvial Soil Unit and Cohesive Glacio-Lacustrine Soil Unit.

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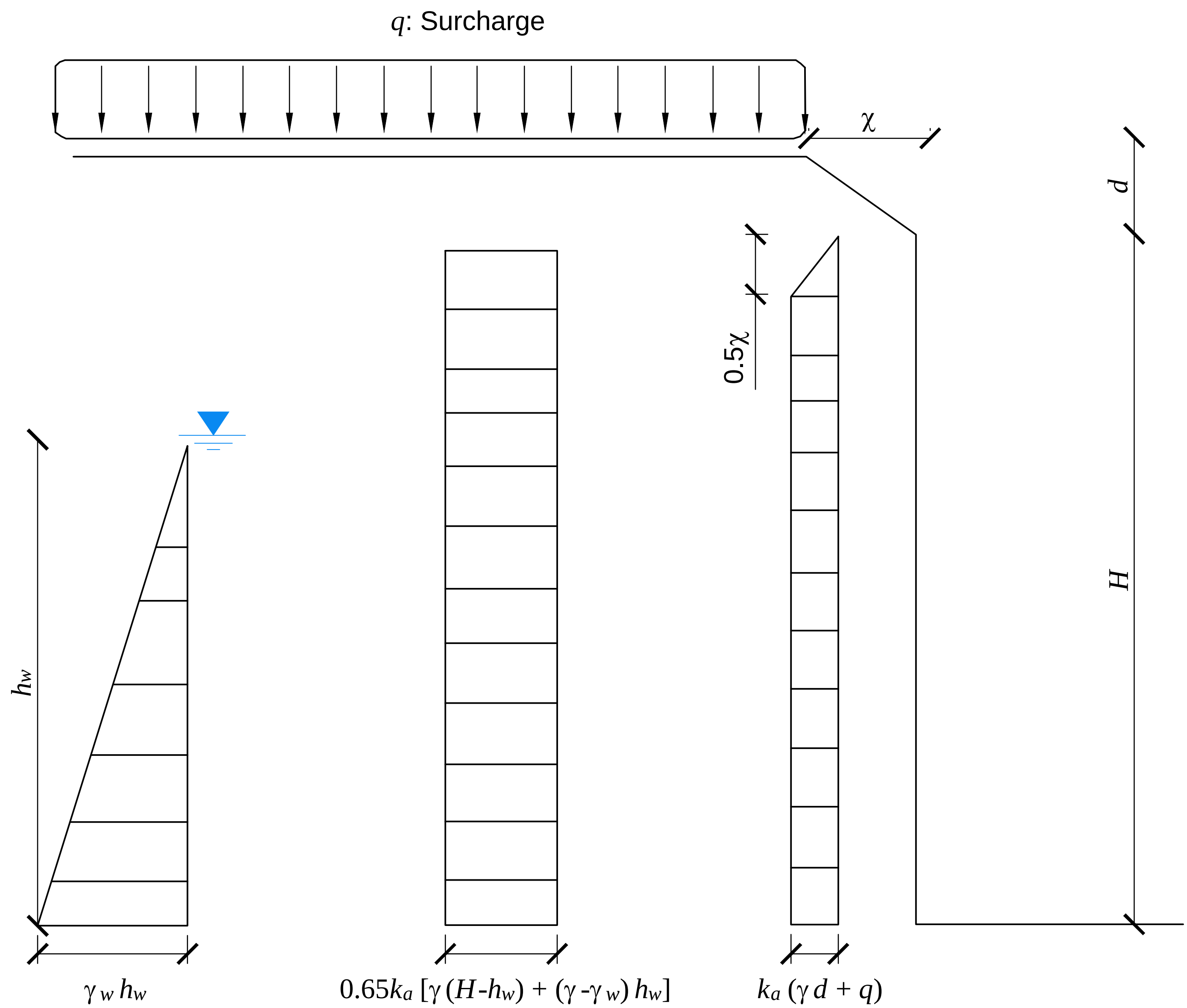

γ: Unit Weight of Soil
γ_w: Unit Weight of Water
k_a: Active Earth Pressure Coefficient

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EARTH PRESSURE DISTRIBUTION FOR TEMPORARY SHORING, GRANULAR ALLUVIAL SOIL UNITS

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Figure: E-1

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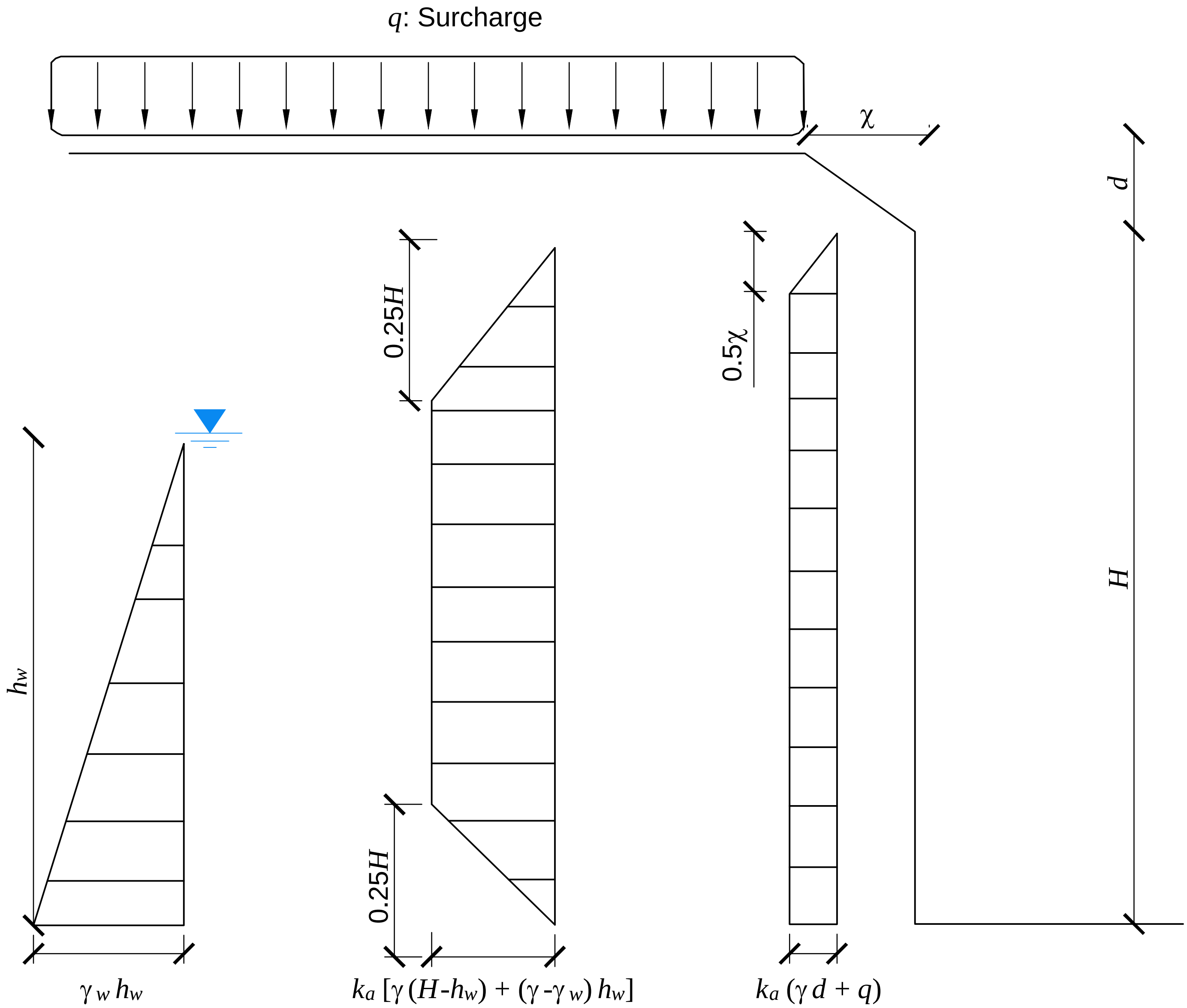

γ: Unit Weight of Soil
γ_w: Unit Weight of Water
k_a: Active Earth Pressure Coefficient

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EARTH PRESSURE DISTRIBUTION FOR TEMPORARY SHORING, COHESIVE ALLUVIAL AND COHESIVE GLACIO-LACUSTRINE SOIL UNITS

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Figure: E-2