

APPENDIX A: GEOTECHNICAL REPORT



Tetra Tech Inc.

Louise Bridge Rehabilitation Preliminary Design Geotechnical Investigation Report

Prepared for:

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Our File No. 0002-171-00

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**RE: Louise Bridge Rehabilitation Preliminary Design
Geotechnical Investigation Report**

TREK Engineering Inc. is pleased to submit our final geotechnical investigation report for the above noted project.

Please contact the undersigned should you have any questions.

Sincerely,

TREK Engineering Inc.

Per:

A handwritten signature in blue ink, appearing to read "M. Van Helden", is written over a light blue horizontal line.

Michael Van Helden Ph.D., P.Eng.
Senior Geotechnical Engineer

Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	Nuno Mendonça	July 17, 2025	Draft Report
1	Michael Van Helden	February 25, 2026	Final Report

Authorization Signatures

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1.0 Introduction

This report summarizes the results of the geotechnical investigation completed by TREK Engineering Inc. (TREK) in support of Preliminary Design of the Louise Bridge Rehabilitation over the Red River in Winnipeg, Manitoba. The terms of reference for this work are included in our proposal to Tetra Tech Inc. (TT) dated December 13, 2024. TREK's scope of work for the project includes information review, site reconnaissance, supplemental sub-surface investigation and laboratory testing, embankment design and slope stability analysis, and provision of geotechnical design recommendations for the rehabilitated structure including assessment of existing foundations and potential foundation underpinning.

2.0 Background Information

The Louise bridge was constructed in 1909 and consists of a primary span of ± 210 m comprised of two stationary outer steel trusses (± 62 m long each) and one central, pivoting steel truss (± 85 m long). In addition, short approach spans are also present beyond the original south and north abutments. The bridge was originally constructed as a railway bridge, and was subsequently widened to serve as a roadway bridge. A few years later, an ice breaker structure was added to the pivot pier. Drawings for the pivot pier ice breaker foundations show that the substructures are supported by 7.6 m long timber piles. The length, spacing and number of piles supporting each bridge substructure is not shown on the plans. Piling specifications available for the pivot pier ice breaker substructure (installed after the original bridge and pier extensions) indicate that the piles were to consist of oak timber with minimum pile tip diameters of 100 mm (8 inches) and were to be driven to less than 6 mm (0.25 inches) per blow of a 2,000 lb drop hammer with drop height of 20 feet (rated energy of approximately 54 kJ). There is no information available to support the assumption that the original pier or pier extension piles were installed to the same specification, or with the same pile lengths.

KGS Group Inc. (KGS) conducted sub-surface investigations near the abutments in 2020 as part of the Bus Rapid Transit Eastern Corridor Study, which included a Condition Assessment and Load Rating of the Louise Bridge completed by WSP. The geotechnical study included two deep test holes near the abutments, riverbank instrumentation monitoring, slope stability analysis and recommendations for new bridge foundations for a replacement structure. The KGS report identified potential riverbank instabilities downstream of the bridge on both the north and south banks, although no specific evidence was provided (i.e. head scarp locations, site photos of instabilities, slope inclinometer monitoring showing movements). Riverbank stabilization, likely using rockfill columns, was recommended. Any new approach fills, or active transportation pathways will need to be considered in riverbank stability analysis, and any opportunities to increase the bridge spans to locate approach fills farther from the riverbanks will be advantageous in terms of slope stability.

A key aspect of the rehabilitation design is to determine if existing substructures can be re-used or will require underpinning or replacement. Pier SU-2 (south river pier) has a measurable tilt, as identified in AECOM's underwater inspections from 2014 and 2019, which we understand has been confirmed by Tetra Tech. Furthermore, inspection reports as far back as 1968 have indicated that one or both abutments may be gradually moving towards the river.

In a report prepared by Dillon Consulting in 1968, the south river pier (Pier #2) exhibited 6 mm of tilt towards the river, and the south abutment appeared to have moved 25 to 40 mm towards the river. Dillon concluded that the pier movements were most likely resulting from riverbank movements. We understand that no long-term structure monitoring is available to confirm these findings.

3.0 Site Conditions

A site visit was completed by TREK staff on June 11, 2025, to assess the general condition of the surrounding area (north and south riverbanks), condition of the existing structure, and Red River conditions in order to identify any potential issues of concern for the preliminary design of the Louise bridge rehabilitation.

The river near the bridge has a well-defined main channel which is gently meandering and well incised within relatively flat flood plain. The riverbank is a combination of vegetated areas with mid to large sized trees and grass as shown in Photo 1.

The south riverbank slopes steeply up from the river's edge, and then is relatively flat in the mid-bank area. There is no evidence of slope instabilities, however ongoing erosion is present along the shoreline beneath the structure and upstream and downstream of the structure (Photo 1).

On the north riverbank, upstream (east) of the bridge, a mid-slope instability extends approximately 30 m upstream from the bridge, with a head scarp approximately in line with the north abutment (Photo 3). The instability is visible in aerial photography as far back as 2002 (Google Earth). Within the instability, remnants of a dock structure (steel H-piles) are present along the shoreline; erosion is visible downslope and upslope of the piles. Shoreline erosion is visible extending downstream from the dock structure beneath the bridge and into the Ernie O'Dowda Park to the west (Photos 2 and 3)

On the north riverbank, downstream (west) of the bridge, global instabilities are present throughout Ernie O'Dowda Park, and it appears that slope movements have been progressing towards the bridge since 1992 (as noted in the previous KGS report).

Pier 2 (south river pier) is visibly tilted to the west and south (Photos 4 and 5). The concrete collar installed around the base of the pier in the 1960's does not appear to tilt as much as the upper portion of the pier, which may suggest that the pier movements pre-date the installation of the concrete collar. Further discussion on the history of pier movements is included in Section 5.2.



Photo 1- View of the South Riverbank (Downstream looking east)



Photo 2 - View of the North Riverbank (Upstream looking West)



Photo 3 - View of the North Riverbank (looking North)



DIRECTION
305 deg(T)

14n 635474
5529733

ACCURACY 4 m
DATUM WGS84

Looking west from
boat launch

2025-06-11
09:06:31-05:00

Photo 4 - Pier 2 Looking West from South East Shoreline

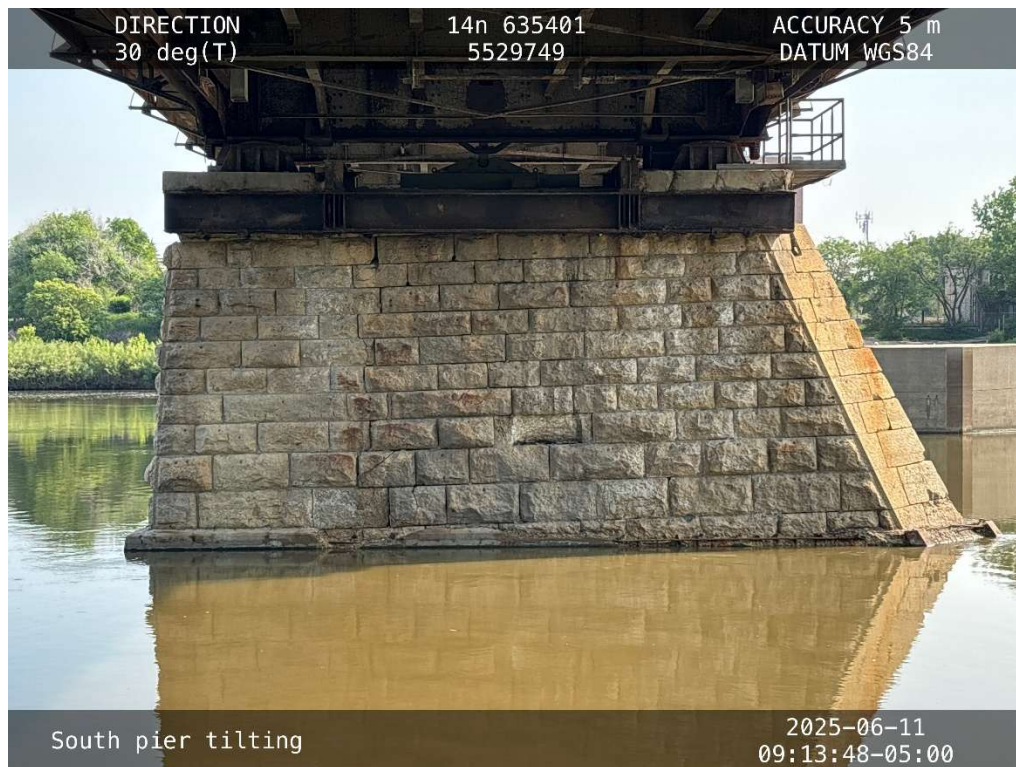


Photo 5 - Pier 2 Looking North from South Head Slope

4.0 Field Program

4.1 Sub-surface Investigation

A sub-surface investigation was completed under the supervision of TREK personnel to determine the soil stratigraphy and groundwater conditions at the site. Three test holes (TH25-01 to 03) and one probe hole (PH25-03A) were drilled by Paddock Drilling Ltd. from March 24, 2025, to April 4, 2025, using a CME 850 and a Mobile B57 track-mounted drill rig, equipped with 125 mm diameter solid stem auger and HQ coring. One test hole (TH25-03) and one probe hole (PH25-03A) were drilled near the south abutment, and two test holes (TH25-01 & 02) were drilled on either side of the north abutment of the bridge. Slope inclinometer casing SI25-01 was installed in TH25-01 to a depth of 16.2 m, while slope inclinometer casing SI25-03 was installed in a separate probe hole (PH25-03) located approximately 1.5 m east of TH25-03 to a depth of 18.4 m. Four Vibrating wire piezometers (VW's) were installed in both TH25-02 (two VW's) and TH25-03 (two VW's) at varying depths. All test holes were backfilled with a cement-bentonite grout mixture to surface.

Sub-surface soils encountered during drilling were visually classified based on the Unified Soil Classification System (USCS). Disturbed (auger grab and split spoon) samples were retrieved at regular intervals. Relatively undisturbed Shelby tube samples were obtained in clay, and HQ core samples were retrieved in till and bedrock. Standard Penetration Tests were performed at depths where split spoon samples were obtained.

Undrained shear strength testing was performed in the field on grab samples (i.e. auger cuttings / disturbed samples) using Torvane and/or Pocket Penetrometer testing devices.

All samples retrieved during drilling were transported to TREK's material testing laboratory in Winnipeg, Manitoba. Laboratory testing consisted of moisture content determination on all samples, unconfined compressive strength testing on Shelby tube samples, and uniaxial compressive strength testing on select rock core samples. Bedrock core samples were classified in general accordance with the Canadian Foundation Engineering Manual 5th Edition (2023).

The attached test hole logs include a description of the soil units encountered during drilling, groundwater conditions and a summary of the laboratory testing results. Laboratory testing results are included in Appendix B.

4.1.1 Soil Stratigraphy

A brief description of the soil units encountered at the test hole locations is provided in this report. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed information provided on the test hole logs.

North Abutment (TH25-01 and 02):

In general, the soil stratigraphy at the north abutment consisted of a 1.5 to 1.7 m thick layer of fill soils (clay and sand) overlying silty clay (lacustrine), silt till and dolomitic limestone. Silt within the silty clay was observed in TH25-02 and a sand layer below the silty clay in TH25-01 was also observed.

Clay fill extended to 1.7 and 1.2 m below ground surface (El. 224.2 and 229.9 m) in TH25-01 and 02, respectively. The clay fill is silty, contains trace to some sand, trace gravel, is black, moist soft and is of low to intermediate plasticity.

Sand fill was encountered below the clay fill in TH25-02 and extended to 1.5 m below ground surface (El. 229.9 m). The sand fill is silty, clayey, brown, dry to moist, loose and is of low plasticity.

Silty clay followed by clayey silt was encountered below the sand fill in TH25-02, to a depth of 3.7 m below ground surface. The silty clay contains trace sand, is moist, grey, very stiff and of high plasticity. The clayey silt contains trace to some sand, is light brown, moist, soft, and is of low plasticity.

Silty clay (lacustrine) was encountered below the fill soils (TH25-01) or the clayey silt (TH25-02) and extended to a depth of 12.8 m and 20.1 m below ground surface (El. 213.1 m & 211.0 m) in TH25-01 and 02 respectively. The silty clay contains trace sand, is brown becoming grey with depth, is moist and is of high plasticity. The silty clay is stiff to very stiff becoming soft to firm below 9.4 and 14.3 m depth in TH25-01 and 02, respectively.

Sand was encountered below the silty clay in TH25-01 and extended to a depth of 13.3 m below ground surface (El. 212.6 m). The sand is silty, contains trace gravel, is moist to wet, loose, and is poorly graded fine sand to fine gravel.

Silt till was encountered below the sand or silty clay and extended to the maximum depth of exploration of 16.8 m below ground surface (El. 209.1 m) in TH25-01 and to 27.5 m below ground surface (El.

203.6 m) in TH25-02. The silt till is sandy, contains trace clay, some gravel, is light grey, moist to wet, is very dense and is of no to low plasticity.

Dolomitic Limestone bedrock (Selkirk Member) was encountered below the silt till to the maximum explored depth of 27.5 m (El. 203.6 m) in TH25-02. The dolomitic limestone is slightly weathered (W2), is very fine grained, mottled cream to beige/light red, and weak to medium strong (R2-R3). The rock quality designation (RQD) of the dolomitic limestone in TH25-02 was measured to be 50 %, 84%, 87% and 90% in four rock cores with a compressive strength of 38.1, 36.2 and 38.4 MPa at depths of 28.8, 29.3 and 30.5 m (El. 202.3, 201.8 and 200.4 m).

South Abutment (TH25-03):

In general, the soil stratigraphy at the south abutment consisted of a 1.8 m thick layer of clay fill overlying alluvial clay, silt till and dolomitic limestone. Sand within the alluvial clay was also observed in TH25-03 and PH25-03A.

The clay fill is silty, contains trace to some sand, trace to some gravel, is dark grey to black, moist, soft to firm, and is of intermediate to high plasticity.

Alluvial clay was encountered below the clay fill and extended to a depth of 13.7 m below ground surface (EL.214.1 m). The alluvial clay is silty, contains some sand becoming sandy with depth, trace organics, is brown, moist, firm to stiff becoming softer with depth, and is of intermediate to high plasticity.

Sand was encountered within the alluvial clay from 3.2 to 6.6 m below ground surface (El. 224.6 to 221.2 m). The sand contains trace clay becoming clayey with depth, trace to some silt, is brown, moist becoming wet with depth, loose, and is poorly graded fine sand.

Silt till was encountered below the alluvial clay and extended to a depth of 30.6 m below ground surface (El. 197.2 m). The silt till contains trace clay, some sand, some gravel, trace boulders, is light grey, moist, loose to compact becoming dense to very dense with depth and is of no to low plasticity.

Dolomitic Limestone bedrock (Selkirk Member) was encountered below the alluvial clay to the maximum explored depth of 33.5 m (El. 194.3 m) in TH25-03. The dolomitic limestone is slightly weathered (W2), is very fine grained, mottled cream to beige, and is weak to medium strong (R2-R3). The rock quality designation (RQD) of the dolomitic limestone in TH25-03 was measured to be 62 and 93%, in two rock cores with a compressive strength of 37.6 and 46.6 MPa at depths of 31.5 and 32.0 m (El. 196.1 and 195.6 m).

4.1.2 Power Auger Refusal

Power auger refusal (PAR) was not observed due to the drilling method (HWT and HQ coring).

4.1.3 Groundwater and Sloughing Conditions

Groundwater seepage and sloughing conditions were not observed in any of the test holes due to the method of drilling (HWT and HQ coring). Table 1 summarizes the vibrating-wire piezometer installation details and groundwater measurements.

Table 1. Piezometer Installation Details and Groundwater Measurements

Piezometer ID	Piezometer Tip Elevation (m)	Soil Layer	Average Groundwater Elevation ¹ (m)	Maximum Groundwater Elevation ¹ (m)
VW25-02A (S/N)	212.6	TILL	224.1	224.4
VW25-02B (S/N)	218.1	CLAY	225.4	226.4
VW25-03A (S/N)	212.6	TILL	224.9	225.1
VW25-03B (S/N)	218.1	CLAY	223.5	223.7

Note 1: Piezometer readings recorded between April 4, 2025, and May 21, 2025

Groundwater monitoring data is included in Appendix C.

The groundwater observations are short-term and should not be considered reflective of (static) groundwater levels at the site which would require monitoring over an extended period to determine. It is important to recognize that groundwater conditions may vary seasonally, annually, or as a result of construction activities.

4.1.4 Slope Inclinator

Two slope inclinometers (SI25-01 & 03A) were installed at the site with one on the northeast side and one on the southeast side of the bridge. SI's TH20-01 and TH20-02 were installed downstream of the south and north riverbanks (approximately in the mid to the upper riverbank slopes) by KGS group in August of 2020. The instrument number, associated test hole ID, and tip elevation / soil unit are summarized in Table 2 below. Cumulative displacement plots are included in Appendix D.

Table 2. Slope Inclinator Installation Details

Test Hole ID (Surface Elev. m)	Instrument ID	Instrument Depth (m)	Soil Layer
TH25-01 ¹ (225.89 m)	SI25-01	16.2	TILL
TH25-03A¹ (227.84 m)	SI25-03A	18.4	TILL
TH20-01 (KGS Group) ² (226.25 m)	TH20-01	12.8	TILL
TH20-02 (KGS Group) ² (227.01 m)	TH20-02	16.9	TILL

Notes :

1) Slope Inclinator measurements recorded on April 4, 2025, and May 21, 2025.

2) Slope inclinometer measurements recorded from August 8, 2020, to December 19, 2024.

Based on the measurements to date, TH20-01 shows little to no movement and TH20-02 has experienced approximately 40 mm of cumulative movement at a depth of 3 m below ground surface since installation (August 2020) to December 2024. The cumulative movement in TH20-02 is consistent with the observed slope instabilities observed near the top of bank throughout Ernie O'Dowda Park as described above in section 3.0. SI25-01 and 02 have not experienced any significant movements since April 11, 2025. Two additional monitoring events are currently scheduled to occur prior and after the Red River drawdown in the fall.

5.0 Foundation Recommendations

Based on the sub-surface conditions and crossing alternatives, driven steel piles (H-piles at abutments) are considered feasible alternatives for underpinning foundations. An assessment of the existing timber pile capacity was also performed.

5.1 Limit State Design

The foundation recommendations provided in this report are based on limit state design. The National Building Code of Canada (NBCC, 2020), the Canadian Highway Bridge Design Code (CHBDC, 2019), and the American Association of State Highway and Transportation Officials (AASHTO, 2024) require the use of limit state design that utilize load and resistance factor design (LRFD) methodology. CHBDC (2019) also incorporates a consequence factor for geotechnical systems while NBCC (2020) does not.

Table 3 of this report summarizes the recommended ULS geotechnical resistance factors that can be used for the design of foundations based on the CHBDC, 2019. Depending on the relevant Code, resistance factors may depend upon the degree of understanding and verification testing completed during construction.

The following definitions from the 5th Ed. of the Canadian Foundation Engineering Manual (CFEM, 2023) are provided with additional notes to assist the reader with the limit state design terminology used in the building foundation recommendations.

Limit states: conditions beyond which a geotechnical system or component ceases to meet the criteria for which it was designed. The main ones are:

Serviceability limit states (SLS) – states corresponding to behaviour of the ground that causes unacceptable serviceability performance conditions, such as deformations, that restrict the intended use of the supported structure and (or) geotechnical system. Recommendations in this report are provided for evaluating the SLS that are developed based on limiting settlement to 25 mm or less. If a more stringent settlement tolerance is required, detailed settlement analysis should be conducted to refine the estimated settlement and/or adjust our recommendations.

Ultimate limit states (ULS) – states corresponding to a loss of stability of the geotechnical system and (or) failure of the supported structure.

Load factor – factor used to modify (usually increase) the characteristic load acting on and from a structure, for the limit state being considered.

Geotechnical resistance factor (ϕ)– multiplicative value that accounts for uncertainty in the geotechnical resistance to produce an acceptable and reliable geotechnical system.

Ultimate geotechnical resistance factor – resistance factor to be used at the ULS.

Characteristic (Nominal) geotechnical parameter – an appropriately conservative estimate of the mean value of a geotechnical parameter for individual strata within the zones of influence of applied loads.

Consequence factor (Y) - multiplicative factor applied to ultimate and serviceability geotechnical resistances, which accounts for consequences of exceeding the limit state under consideration.

Geotechnical resistance – load that the ground can support at a limit state. Different resistances can be defined, including:

Characteristic (nominal) ultimate geotechnical resistance – maximum load that the ground can support at the ULS, estimated using characteristic (nominal) geotechnical parameters.

Factored ultimate (ULS) geotechnical resistance – product of the consequence factor, the ultimate geotechnical resistance factor, and the characteristic (nominal) ultimate geotechnical resistance.

Table 3. ULS Geotechnical Resistance Factors for Foundations (CHBDC 2019)

Description	Resistance Factor for Low Degree of Understanding of Soil Conditions	Resistance Factor for Typical Degree of Understanding of Soil Conditions	Resistance Factor for High Degree of Understanding of Soil Conditions
Deep foundations in compression based on static analysis	0.35	0.40	0.45
Deep foundations in compression based on dynamic testing	0.40	0.50	0.55
Deep foundations in tension based on static analysis	0.20	0.30	0.40

5.2 Existing Timber Pile Foundations

Based on available drawings for the existing bridge, we understand that the original railway bridge was constructed first and was founded on timber piles. Subsequently, piers were extended to accommodate a roadway bridge, and the pier extensions (as shown Photo 5) were also founded on timber piles. Lastly, an icebreaker extension was added to the pivot pier and was also founded on timber piles.

The timber piles at piers 2, 3, 4 are 7.6 m (25ft) long (as per original bridge plan 4003), however the butt or tip diameters are not provided. Details on pile layout (number of piles and spacing), and installation criteria for the original bridge and pier extensions are not known. Abutment pile lengths and diameters are not provided on the existing drawings and are unknown at the time of this report.

Drawings for the pivot pier icebreaker foundations indicate that 7.6 m long 200 mm tip diameter timber piles were driven to practical refusal using a 2,000 lb drop hammer with a 20ft (6.1 m) drop height (maximum rated energy of 54kJ). The refusal criteria was ¼” (6 mm) or less per blow for the final set.

During the Red River Floodway expansion project (2007), dynamic pile load testing was performed on existing timber piles at the CN Redditt bridge over the Floodway.

The existing timber piles were installed to practical refusal in till, and the dynamic load tests (performed decades after pile installation) revealed unfactored ultimate capacities ranging from 780 kN to 930 kN with an average of 850 kN. In comparison, timber piles of this size have conventionally been designed for an allowable capacity of 266 kN based on the allowable fibre stress of the timber; this allowable capacity translates to an unfactored ultimate capacity of 800 kN (factor of safety of 3).

More recently, as part of the Arlington Bridge Rehabilitation Preliminary Design project (2024), TREK and Tetra Tech undertook static load testing of an existing timber pile supporting one of the bridge piers. The timber pile length is not known; however the diameter is comparable to the piles at the Floodway and the Louise Bridge. The pile tips may or may not have been embedded in till, and the upper till at the Arlington Bridge site tends to be loose to compact. As such, the end-bearing capacity of the pile was uncertain. The unfactored ultimate capacity of the timber test pile was approximately 400 kN.

Based on the CN Redditt and Arlington bridges load testing, timber piles supporting the Louise Bridge may range in unfactored ultimate capacity from 400 to 800 kN, depending upon whether or not they were driven to practical refusal on dense to very dense till. Given that the number of piles that support the structure is not known, the total foundation capacity cannot be estimated, and performance of the structure must guide the evaluation of the foundation load carrying capacity.

Pier 2 appears to have undergone considerable foundation settlement and longitudinal/transverse tilting. As noted in Section 6.0, we do not anticipate that south riverbank movements are contributing to pier movements. We understand from discussion with Tetra Tech that historical reports dating back to the 1940's mention the pier movement, including a report from 1957 which included a pier monitoring program. The concrete collar encasement of the timber cribbing was constructed in the 1960's and shows no appreciable signs of further settlement or tilting based on recent three-dimensional scans completed by Tetra Tech. The bridge was levelled in the 1960's during bearing replacement, and we understand from Tetra Tech that no appreciable settlement has occurred since then. As such, we are of the opinion that the existing pier structures are relatively stable (from a geotechnical perspective) to continue to support existing dead and live loading conditions. A structure and pier monitoring program is recommended to identify any further movements should they occur, in which case a more detailed assessment should be undertaken.

In addition, there is mention in historical reports regarding shifting of the abutments towards the river. A similar mechanism was identified at the St. Vital Bridge over the Red River, which was attributed to inadequate lateral pile capacity of vertical piles, as well as excessive lateral earth pressures driving movements due to increased soil saturation, swelling and frost heave forces. Existing abutment piles should be assessed for lateral and axial capacities relative to lateral and overturning forces, to determine if any remedial measures are required. Remedial measures may include foundation underpinning to supplement capacity and/or drainage measures to decrease earth pressures and alleviate swelling and frost action forces.

We understand that as part of the proposed pier rehabilitation, additional dead loads (e.g., due to concrete encasement of the piers) are expected. As such, pier underpinning will be required. Underpinning foundations should be designed to support all dead and live loads, and the existing foundations should not be relied upon following the rehabilitation works.

5.3 Driven Steel Piles

Steel H-piles driven to refusal on very dense silt till or bedrock are considered suitable to support the anticipated loads. This pile type will derive a majority of its geotechnical resistance in end bearing with a relatively small contribution from shaft adhesion.

Steel piles driven to refusal on bedrock are commonly designed for the ULS based on the structural strength of the pile section. The silt till at site varies from loose to compact to very dense and the bedrock ranges from fair to excellent rock quality. Based on experience with dynamic testing of steel piles on various sedimentary rock formations, we anticipate that the geotechnical resistance will likely govern design due to the relatively low rock strength and variable rock quality, and the possibility of piles reaching practical refusal in till. In this regard, driven steel piles installed for the Disraeli Bridge reconstruction reached practical refusal within till, with associated capacities confirmed by dynamic testing and CAPWAP of approximately 1,900 to 2500 kN, with an average of about 2,200 kN.

A wave equation pile driveability analysis was performed using GRLWEAP (Version 2023). The WEAP model was prepared using reasonable soil parameters based on the soil type model. A fully-plugged H-pile section was modeled using the gross “box” end-area of the pile. Based on the WEAP analysis it is anticipated that steel H-piles (likely HP310x110 sections) will be driven into very dense till for a final tip elevation ranging from approximately El. 203 m to 210 m, provided that a hydraulic hammer (e.g. Junttan HHK 5S) is used. If a single-acting diesel hammer (e.g. Pileco D19-42) is used, depending on the condition of the hammer, practical refusal may be reached shallower depth in till resulting in a lower capacity.

Piles driven to practical refusal based on the hammer energy and criteria described below are expected to develop a nominal pile capacity of 2,200 kN, resulting in a factored ULS pile capacity of 1,210 kN (based on a resistance factor of $\phi = 0.55$) and an SLS pile capacity of 730 kN. A minimum pile size of HP310x110 with Grade 350W steel is recommended.

Selection of pile sizes should take into consideration the pile sections and weights that are readily available. Under the SLS loads, pile settlement can be expected to be less than about 15 mm, including elastic compression of the pile. The majority of the settlement is expected to be due to elastic compression of the steel.

Based on our experience with dynamic testing of steel H-piles in Manitoba and the subsurface conditions encountered in the test hole drilled for this geotechnical investigation, we recommend that a resistance factor of 0.55 can be used for design provided that dynamic testing with CAPWAP analysis is performed during foundation installation. If dynamic testing with CAPWAP analysis is not performed a resistance factor of 0.45 must be used. Dynamic load testing will provide data on the driving energy delivered to the pile and the driving stresses (tensile and compressive) in the piles.

The CAPWAP analysis will utilize the data collected to provide a mobilized static pile capacity that can be compared to the unfactored and factored ULS pile capacities. The details of the dynamic load testing program can be finalized once the foundation layout has been established. Typically, about 2 to 5 percent of the piles should be tested during pile installation under restrike conditions; however, the number of piles to be tested will depend on the number and sizes of piles to be installed.

An average nominal (unfactored) skin friction of 30 kPa should be used to calculate uplift resistance; uplift resistance within the upper 3 m of the embedded pile length (or the depth of pre-bore, whichever is greater) should be neglected.

Additional Design Recommendations

1. The weight of the embedded portion of the pile should be neglected in design.
2. Pile spacing should be a minimum of 2.5 pile diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
3. The piles must be structurally designed to withstand the design loads, handling stresses, and driving stresses.
4. All piles should be fitted with driving tips (shoe) to help protect the pile tip during installation. The driving tip must be designed to withstand driving stresses and long-term design load cases. It is recommended to assume that pile compressive stresses during driving could potentially reach the yield strength of the steel for the design of driving shoes and splices. Hard-bite tips for H-piles are recommended.

Installation Recommendations

1. A pile driving system (*i.e.* pile-driving hammer) should be capable of delivering 45 kJ for driving H-piles (310x110 sections). The pile-driving hammer should have the capability of adjusting the fuel setting or stroke to deliver higher energy to the pile during driving if the energy is not sufficient to drive the pile to the required tip elevation. The driving system should also have the capability of adjusting the fuel setting or stroke to deliver lower energy to prevent pile damage upon sudden pile refusal. It should be noted that delivered energy is a function of the rated energy and the efficiency of the driving system and can be considered to be the net energy transferred to the pile head. The delivered energy should not be taken directly as the rated energy.
2. The efficiency of the driving system (ratio of delivered to potential energy) depends on the type and condition of hammer used, as well as the properties of the soil and pile. The driving system efficiency is typically about 50 to 60% for single-acting diesel hammers and about 85 to 90% for hydraulic drop hammers, although it is not uncommon for values to fall outside this range. TREK can assist in developing specifications for piling hammers once the pile section to be used is known. The actual stroke (for hydraulic hammers) or blow rate (for open-ended diesel hammers) should be monitored during driving at refusal to confirm that the required potential energy is developed.
3. The Contractor should be required to submit a proposed driving system for approval a minimum of 7 days prior to the start of pile driving. The pile driving system should be capable of installing the piles to the required capacity within specified allowable driving stresses. Acceptance of the

proposed driving system should be confirmed by driveability analysis (i.e. wave-equation analysis) prior to construction.

4. All piles driven within 5 pile diameters of one another should be monitored for pile heave. If heave is observed, all piles should be checked and piles exhibiting heave should be re-driven to one set of the specified refusal criteria.
5. Driving stresses in the pile should not exceed 90% of the yield stress of the pile material or the stresses specified by the pile manufacturer, whichever is the smaller value.
6. Pile verticality (plumbness) should be measured on all piles after driving completion has been achieved to check if verticality is within the limits of the structural design. It is common local practice to specify a maximum acceptable percentage that the pile can be out of vertical plumbness (e.g. 2% out of plumb) or out of the specified batter.
7. Observation of all driven pile installation should be performed by TREK personnel to confirm that the refusal criteria have been met and to record that pile installation has been completed according to the design.
8. Any piles damaged, out of plumb an excessive amount or reaching premature refusal may need to be replaced. The structural designer will have to assess non-conforming piles to determine if they are acceptable. Any piles exhibiting unusual driving behaviour (e.g. relaxation) or those driven out of alignment, plumbness or not meeting the refusal criteria should also be tested.

5.4 Micropiles

Micropiles are small diameter, drilled, grouted, and reinforced piles that are commonly used for underpinning structures with restricted access and low overhead clearance. They generally have a diameter in the range of 75 to 320 mm and can be installed using a variety of drilling techniques (e.g. single tube, rotary percussive) and grouting (e.g. gravity, pressurized). It is important that an experienced micropile contractor be retained to verify the correct design for the micropiles and determine the proper installation and grouting methods for the sub-surface conditions at the site; performance and capacity of micropiles are directly related to proper installation and grouting techniques. Construction of a micropile typically consists of advancing steel casing to the target depth in the bearing stratum followed by placement of a single bar of reinforcing steel (typically a high-strength threaded bar) and placing high-strength cement grout from the bottom of the casing either by tremie or pressurized injection techniques as it is removed from the drilled shaft. The steel casing can then be retracted either fully or partially, leaving in place a bond length (socket) within competent soils or rock.

Typically, design and construction of micropiles is the responsibility of the foundation Contractor that has experience with this specialized foundation type. Preliminary design of micropiles can be completed using the recommendations provided below.

Micropiles installed into bedrock (i.e. socketed into bedrock) will derive their capacity from shaft adhesion with the rock socket. Long piles may also benefit from some shaft adhesion above the bedrock, particularly in dense material such as glacial till. End-bearing resistance should be neglected in design. Table 4 provides the recommended factored ULS and SLS unit geotechnical resistances for micropiles

installed into good quality limestone bedrock. In this regard, bedrock was encountered at the site below Elev. 202.7 m (north abutment) and 197.2 m (south abutment).

Although not encountered in drilling at the site to date, the upper 1 to 2 m of bedrock in the area can be expected to be fractured, and therefore the bond length (i.e. rock socket length) should be developed 1.5 m below the bedrock contact depth; the fractured zone may vary across the site and the installation of micropiles will need to be adjusted to suit bedrock conditions encountered during piling. A resistance factor of 0.40 was applied to calculate the factored ULS geotechnical unit bearing resistances, based on a typical level of understanding. The pile head settlement under unfactored service loads can be calculated based on 5 mm or less of pile tip displacement plus elastic shortening of the pile.

Table 4. Recommended Factored ULS and SLS Geotechnical Resistances for Micropiles

Soil Type	Factored ULS Resistance (kPa)		SLS Resistance (kPa)
	Compression ($\phi = 0.40$)	Tension ($\phi = 0.30$)	
	Shaft Adhesion	Shaft Adhesion	Shaft Adhesion
Good Quality Limestone Bedrock	400	300	330

Notes: 1) Shaft adhesion based on gravity placed grout.

Micropile Design Recommendations:

1. Pile spacing from existing structures should be confirmed in consultation with a qualified micropile contractor.
2. The weight of the embedded portion of the pile may be neglected.
3. Shaft adhesion should be neglected within overburden soils and the upper 1.5 m of bedrock.
4. Micropiles must be installed in good to excellent quality (RQD > 75%) dolomitic limestone bedrock.
5. Micropiles should have a minimum design socket length of 3 m.
6. Micropiles should have a minimum spacing of 760 mm or 3 micropile diameters, whichever is greater measured center to center. If a closer spacing is required, TREK should be contacted to provide a reduction factor to account for potential group effects.
7. Micropile locations should be offset a sufficient distance from existing timber piles to prevent undermining of timber pile tips. The micropile contractor should be consulted relative to select and appropriate method of advancement in order to alleviate this concern.
8. Lateral resistance of vertical micropiles is limited due to their slenderness ratio and should be ignored. If lateral resistance is required some of the micropiles may need to be installed on an incline.
9. Corrosion of the cased length over the design life needs to be accounted for.
10. Higher bearing resistances are possible depending on the grouting technique (i.e. injection vs gravity). An experienced micropile contractor should be contacted to verify available grouting techniques and the corresponding bearing resistances.

Micropile Installation Recommendations:

1. The contractor should plan to leave steel casings in place above the rock socket to prevent sloughing

- or caving of the micropile shaft or necking of the pile within overburden soils.
2. The final foundations should be designed by the foundation contractor to suit the subsurface conditions at the site and the contractors micropile experience and equipment. A shop drawing, installation procedure and load testing for the micropiles, signed and sealed by an engineer licensed to practise in Manitoba, should be provided for review. The pile layout may need to be adjusted by the micropile contractor.
 3. Grout must be placed by gravity (tremie) methods. Grout placement by free-fall method must not be permitted under any circumstance.
 4. Load testing should be specified to be completed by the Contractor. At least one verification load test (up to 2.0 to 2.5 times the factored pile load) should be completed prior to production piling at each substructure unit. Additional proof testing (up to 1.5 times the factored pile load) should also be considered during production piling.

5.5 Frost Effects

Buried concrete sections subjected to freezing conditions should be designed to resist ad-freeze and uplift forces related to frost action acting along the vertical face of the member within the maximum depth of frost penetration (2.4 m). In this regard concrete members may be subject to an unfactored (nominal) ad-freeze bond stress of 65 kPa within the depth of frost penetration.

Ad-freeze and frost heave forces will be resisted by structural dead loads and uplift resistance. The following design recommendations apply to design against frost effects.

1. Frost-jacking of footings is not specifically addressed in CHBDC (2019). The following approach is recommended to achieve design outcomes consistent with local practice and observations of acceptable performance. A load factor (α) of 1.2 should be applied to the nominal ad-freeze bond stress for calculating the factored ad-freezing force, or to the frost-heave pressure acting on the underside of the structure (unless a void form is used). Factored structural dead loads should be added to the resistance. The factored geotechnical uplift resistance plus the factored structural dead loads must be greater than the factored ad-freezing forces.
2. Measures such as dowelling of the footing into bedrock should be considered to act as a frost jacking counteracting measures.
3. Use of non-frost susceptible soils such sand and gravel with minimal fines as backfill around footings and buried structures should be used to minimize ad-freeze forces.

5.6 Lateral Loads

The soil response (subgrade reaction) to lateral loads can be modeled in a simplified manner that assumes the soil around a pile can be simulated by a series of horizontal springs for the preliminary design of pile foundations. The soil behaviour can be estimated using an equivalent spring constant referred to as the lateral subgrade reaction modulus (k_s). Table 5 provides the recommended subgrade reaction modulus for the lateral load analysis. The majority of lateral resistance will typically be offered by the upper 5 to 10 m of soil, depending on the relative stiffness of the pile and soil units. Portions of

the pile that are installed into sill till can be assumed to be fixed in terms of lateral displacement and free to rotate (pinned connection at the toe).

Table 5. Recommended Values for Lateral Subgrade Reaction Modulus (Ks)

Soil	Approximate Depth below Ground/ Elevation (m)	K _s (kN/m ³)
Clay Fill	0 to 6 m ~Elev. 231 to 225 m	3,350/d
Clay (Lacustrine)	6 to 11 m ~Elev. 225 to 220 m	4020/d
Clay (Lacustrine)	11 to 20 m ~Elev. 225 to 211 m	3000/d
Clay (Alluvial)	3.5 to 14.0 m (~Elev. 254.8 to 245.1 m)	2350/d

Notes: d = pile diameter (m)

Using the lateral subgrade reaction modulus assumes a linear response to lateral loading and therefore is only appropriate under the following conditions:

- maximum pile deflections are small (less than 1% of the pile diameter),
- loading is static (no cycling), and
- pile material behaves linear elastically (does not reach yield conditions).

If one or more of these conditions are not met, a more rigorous analysis that includes non-linear behavior of the piles and surrounding soil is required. In this regard, as part of detailed design, a lateral pile analysis that incorporates the material and section properties of the piles, final lateral deflection criteria and a more realistic elastic-plastic model of the soil response to loading should be carried out by TREK to confirm the lateral load capacity of the piles.

5.7 Foundation Concrete

All foundation concrete should be designed by a qualified structural engineer for the anticipated axial (compression and uplift), lateral, and bending loads from the structure. Based on local experience gathered through previous work in Winnipeg, the degree of exposure for concrete subjected to sulphate attack is classified as severe according to Table 3, CSA A23.1-19 (Concrete Materials and Methods of Concrete Construction). Accordingly, all concrete in contact with the native soil should be made with high sulphate-resistant cement (HS or HSb). Furthermore, the concrete should have a minimum specified 56-day compressive strength of 32 MPa and have a maximum water to cement ratio of 0.45 in accordance with Table 2, CSA A23.1-19 for concrete with severe sulphate exposure (S2). Concrete that may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA A23.1-19.

6.0 Slope Stability Analysis

Slope stability analyses were conducted to determine the stability of the riverbank under short-term (extreme) and long-term (typical) conditions and to evaluate conceptual remedial alternatives to improve stability. The analysis cases included a back-analysis of the observed instability on the north

riverbank slope, and of the existing condition of the south riverbank slope. A remedial conceptual solution (rock columns + regrading) was evaluated for the north riverbank abutment slope.

Slope movements are observed on many sections of riverbank throughout the city following annual fall drawdown of the Red River or following spring flooding due to a time lag where groundwater levels in the till remain elevated as they are adjusting to lower river levels (short-term extreme groundwater conditions). The probability of riverbank instabilities occurring increases as values approach unity (FS=1.0). Riverbanks with a critical (minimum) Factor of Safety (FS) greater than 1.3 under these conditions are considered to be relatively stable; however, creep movements are still possible. A factor of safety greater than 1.3 was therefore selected as the design target under short-term (extreme) groundwater conditions. Creep movements are considered unlikely when the factor of safety exceeds 1.5, therefore a factor of safety target of 1.5 was selected for long-term (typical) groundwater conditions.

The stability analysis was conducted using a limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2021 software package (Geo-Slope International Inc./Seequent Ltd.). The slope stability model used the Morgenstern-Price method of slices to calculate factors of safety. Critical local and global slip surfaces were identified using a grid and radius slip surface method. The soil stratigraphy was based on the information provided in this report from the test holes drilled by TREK. Table 6 summarizes the parameters used for the soil units in the slope stability analyses, which are considered reasonable based on the available information. The groundwater conditions were modelled using a static piezometric groundwater line. Residual strengths were assigned to a zone of lacustrine clay just above the till to reflect strain weakening where past instabilities have been observed (north bank). Although no signs of instability are evident on the south bank, and lacustrine clay was not identified in the test hole logs, this layer has been included in the analysis of the south bank as well, as a conservative assumption.

Water levels in the river were modelled at Elev. 221.80 m and 223.70 m under short-term (extreme) and long-term (typical) groundwater conditions. Head slope groundwater levels were assumed to be approximately 2 m below ground surface (Elev. 228.2 m) for short-term conditions, and at the measured levels of Elev.226 m for long-term conditions.

Table 6. Soil Properties used in Slope Stability Analysis

Soil Description	Unit Weight (kN/m ³)	Cohesion (kPa)	Friction Angle (degrees)
Clay (Alluvial)	19.1	3	20
Clay (Lacustrine)	17.5	5	17
Fill (Sandy Clay)	18	2	25
Clay (Residual)	17.5	3	13
Rock Column	20	0	50
Sand	18	0	30
Silt	18	2	22
Silt and Sand Till	20	2	30
Bedrock	Impenetrable		

6.1 Stability Analysis Results

Calculated factors of safety for bridge head slopes are summarized in Table 7, with stability analysis results shown in the figures included in Appendix E (as referenced in the table).

Table 7. Slope Stability Analysis Results

Stability Case	Riverbank Slope	Ground-water Case	Ground-water Elevation (m)	Red Rivel Level (m)	Critical FS	Relative Improvement (%)	Model Number (Appendix A)
Back Analysis/Existing Conditions	North Abutment	Short-Term	228.2	221.8	1.02	N/A	E-1
		Long-Term	226	223.7	1.1	N/A	E-2
	South Abutment	Short-Term	228.2	221.8	1.35 (Global)/ 1.37 (Pier #2)	N/A	E-3
		Long-Term	226	223.7	1.78 (Global)/ 1.81 (Pier #2)	N/A	E-4
Rock Columns (Staggered 3 Rows) with 2.1 m diameter and 0.6 m clear spacing	North Abutment	Short-Term	228.2	221.8	1.34	31%	E-5
		Long-Term	226	223.7	1.48	34%	E-6

The back analysis of the north slope results in a slip surface closely matching the observed instability with a factor of safety of approximately unity under short-term extreme conditions, which confirms the need for riverbank stabilization works. A shear key equivalent to 3 staggered rows of rock columns with 2.1 m in diameter spaced 0.6 m was analyzed for the north slope, resulting in a relative improvement of at least 30% and factors of safety that satisfy the targets (1.3/1.5 for extreme/typical conditions).

The existing site conditions on the south riverbank slope were also analyzed, including a layer of lacustrine clay of residual strength above the till, which is considered a conservative assumption. The existing factors of safety for the south slope exceed the targets of 1.3/1.5 for extreme/typical conditions, and therefore stability improvement works are not required on the south bank. Should any modifications to the slope such as added embankment fill be considered, additional analyses should be performed to confirm the need for slope stabilization works.

6.2 Slope Stability Recommendations

South riverbank stability is considered acceptable, providing no major alternations (i.e., fill placement) is planned on the riverbank. North riverbank stability does not meet factor of safety targets, and therefore slope stabilization using rockfill columns is recommended. As shown in Figures E-5 and E-6 (see Table 7), three rows of 2.1 m diameter rockfill columns are proposed, in a staggered arrangement with 0.6 m clear spacing (2.7 m on centre). The rockfill columns are recommended to extend at least 30 m upstream and downstream of the bridge deck. Rockfill columns cannot be installed beneath the existing bridge due to inadequate overhead clearance. A proposed rockfill column layout is provided in Figure 03, where a minimum 1.5 m clearance is provided to the edge of the bridge deck, and any rockfill columns that would otherwise be placed beneath the bridge have been relocated outside the bridge deck. In total, 96 rockfill columns are required. Table 8 provides a Class 3 construction cost estimate for the proposed stabilization works; the cost estimate includes an allowance of 12% for engineering during detailed design or construction and a 15% contingency, but excludes City administration costs or permit fees. Note that the costs for temporary access that have been carried are lower than typical for this type of project, recognizing that rockfill column construction will be completed concurrently with pier underpinning works, and will predominantly utilize access roads and working platforms planned for those works.

The recommended stabilization works achieve the target factors of safety, provided that erosion of the head slopes does not occur over the life of the structure. As such, Class 450 (0.625 mm thick) riprap erosion protection is recommended to extend a minimum of 30 m upstream and downstream of the structure on the north bank, and extend from approximately the Elev. 224.7 m contour, down to the flat river bottom (about 10 m out from the winter river shoreline. Similarly, riprap erosion protection is also recommended on the south riverbank, extending at least 15 m upstream and downstream of the bridge, and extend about 5 m out from the winter river shoreline. The recommended riprap extent is shown on Figures 03 and 04.

7.0 Lateral Earth Pressures and Backfill

The magnitude of lateral earth pressures from retained soil acting against buried walls (e.g. abutments) will depend on the retained material type, method of placing and compacting the backfill, and the magnitude of rotation of the walls. We understand from discussions with Tetra Tech that the abutments have been subjected to gradual movement towards the river on the order of 25-50 mm, in particular the south abutment. It was concluded from the slope stability analysis presented previously that riverbank movements are not likely the cause of abutment movements. TREK has experience with numerous cases of bridge abutment movements that were attributed to freeze/thaw and swelling action resulting in annual buildup of lateral earth pressures, causing gradually accumulating permanent displacements, with the St. Vital Bridge being the most recent case. Furthermore, these bridge abutment structures have been supported by piled foundations installed vertically with very little lateral resistance, similar to the abutments at the Louise Bridge.

TREK anticipates that existing abutments may be backfilled with either granular or cohesive soils. If cohesive backfill was used, freeze/thaw and swelling forces may be contributing to abutment movements. If granular backfill was used and it is not free-draining, freeze/thaw forces may be contributing to abutment movements. Additional investigations are recommended to confirm the nature and moisture content of the backfill soils.

An active pressure coefficient (K_a) of 0.3 is commonly used to calculate lateral loads from granular soils against retaining structures (i.e. abutment walls) which are free to translate horizontally by at least 0.2 percent of the retaining wall height. However, given our experience with similar structures and performance issues (McPhillips Street underpass, St. Vital bridge), it is possible that loading and vibrations from traffic may be increasing long-term lateral earth pressures on the abutments. As such, we recommend that an at-rest earth pressure coefficient (K_o) of 0.5 for granular soils should be used in the assessment. An appropriate surface surcharge should also be included in the earth pressure distribution to account for surface loads. The active pressure coefficient (K_a) can be used to calculate the component of lateral load on the retaining structure due to surcharge loads.

A passive earth pressure coefficient (K_p) of 3.0 can be used to calculate the resistance of granular backfills behind the abutments due to horizontal loads acting on the bridge. However, full passive resistance will only develop if the abutment can deflect at least 2 percent of the buried height of the abutment adjacent to the backfill.

Since drains are not indicated on the bridge abutment as-built drawings, a hydrostatic water pressure should be applied behind the abutment walls in calculation of lateral earth pressures, at a depth of 0.5 m below the roadway surface (in absence of any monitoring data).

Where clay backfill is present as backfill against abutment pile caps and walls, earth pressure coefficients of 0.55 (K_a), 0.71 (K_o) and 1.83 (K_p) should be used. Following the same approach described above for granular backfill, the at-rest earth pressure coefficient (K_o) should also be used for clay backfill behind abutment head walls.

Although difficult to quantify, existing abutment walls and pile caps may experience additional lateral earth pressures associated with swelling of clays and frost heave, both of which are exacerbated by poor drainage and presence of standing water.

For design of bridge rehabilitation, consideration should be given to installation of perimeter trench drains (subdrains) to collect groundwater that may be present around the abutments and discharge any intercepted water into the land-drainage sewer or at a suitable location on the lower riverbank. Removal of clay backfill behind and around abutment walls and pile caps, and replacement with free-draining granular soils should also be considered to reduce lateral earth pressures on the abutments. Lightweight fill (cellular concrete) could also be considered as backfill replacement to reduce earth pressures on the abutment walls and on the riverbank.

Where free draining backfill is added or confirmed, such as where a sub-drainage system is present at the base of the wall to prevent the build-up of hydrostatic pressures, the total lateral earth pressure force is the area of the triangular pressure distribution acting on a below grade wall which can be derived based on the following equation:

$$P = K\gamma D$$

where,

P = lateral earth pressure at depth D (kPa)

K = earth pressure coefficient (unitless)

γ = bulk unit weight of retained soil (20 kN/m³ for granular fill, 17.5 kN/m³ for clay)

D = depth below finished grade to where earth pressure is being calculated (m)

If cohesive soil is present or drainage is not expected, the following equation should be used:

$$P = K\gamma' D + \gamma_w D$$

where,

P = lateral earth pressure at depth D (kPa)

K = earth pressure coefficient (unitless)

γ' = effective unit weight of retained soil (10.2 kN/m³ for granular fill, 7.7 kN/m³ for clay)

D = depth below finished grade to where earth pressure is being calculated (m)

γ_w = unit weight of water (9.81 kN/m³)

Backfill (retained soil) should not be placed and compacted until the walls can support lateral earth pressures. Over-compaction of the backfill soils adjacent to buried walls may result in earth pressures that are considerably higher than those predicted in design. Compaction of the granular fills within about 1.0 m of the vertical walls should be conducted with a light hand operated vibrating plate compactor and the number of compaction passes should be limited to achieve a density between 90% and 92% of Standard Proctor Maximum Dry Density (SPMDD). Compensation for any settlement can be made in the final grading by placing additional fill adjacent to the structure and to provide positive drainage away from the structure. Backfill compacted in this manner (lightly) will ultimately settle by a maximum of about 2 to 4% of the fill depth. Beyond the 1 m offset, the granular fill should be compacted to at least 95% SPMDD in an unfrozen state in lifts not exceeding 200 mm loose thickness.

8.0 Excavations and Shoring

Excavations must be carried out in compliance with the current relevant regulations under the Manitoba Workplace Safety and Health Act to suit the planned and expected construction activities and schedule. Excavations greater than 3 m deep must be designed and sealed by a professional engineer. If space is limited or the stability of adjacent structures may be endangered by an excavation, a shoring system may be required to prevent damage to, or movement of, any part of adjacent structures, and the creation of a hazard to workers and the public.

Cantilevered shoring walls will be required for deep excavations where temporary shoring is necessary. Table 9 provides the recommended earth pressure coefficients and bulk unit weights of each soil layer for calculation of lateral earth pressures on cantilevered walls. Surcharge loads and hydrostatic water pressure should be incorporated into the design of cantilevered walls, as well as an adequate factor of safety against instability. If braced shoring is used, an appropriate earth pressure distribution should be selected by the shoring designer for a braced wall condition.

A certain amount of ground movement behind the shoring will occur and is largely unavoidable. The amount of movement that will occur cannot be accurately predicted, mainly because the movement is as much a function of installation procedures and workmanship as it is a function of theoretical considerations. It is anticipated that the design of temporary shoring will be the responsibility of the Contractor. The shoring design and shop drawings should be submitted by the Contractor for review and comment by TREK prior to construction. Performance of the excavation system should be monitored from the onset of installation to removal of the shoring system.

Dewatering measures should be provided and completed as necessary to maintain a dry excavation and permit proper completion of the work. Surface water should be diverted away from the excavation and the excavation should be backfilled as soon as possible following construction.

Table 9. Soil Parameters for Cantilevered Walls

Material	Elev. Range	Undrained Shear Strength	Effective Cohesion	Effective Friction Angle	Saturated Unit Weight	Effective Unit Weight	Earth Pressure Coefficients (Rankine ¹)		
Units >	(m)	(kPa)	(kPa)	(degrees)	(kN/m ³)	(kN/m ³)	Ko	Ka	Kp
Silty Clay (North Riverbank)	Above 211.0 m±	50	5	17	17.5	7.7	0.70	0.54	1.82
Alluvial clay (South Riverbank)	Above 214.1 m±	30	3	20	19	9.2	0.66	0.49	2.04
Silt and sand till	Below 214.1 or 211.0 m±	N/A	2	30	19.0	10.2	0.50	0.33	3.0

Note 1: The effective stress earth pressure coefficients assume the magnitude of wall rotation is sufficient to develop the full earth pressure. The values should be reduced to suit the allowable wall rotation. Refer to Section 20.2.5 of the Canadian Foundation Engineering Manual (5th Edition 2023).

9.0 Dewatering and Cofferdams

Cofferdams may be required to divert stream flow and/or maintain a dry construction area to facilitate the construction of pile caps for underpinning works. A cofferdam is defined as a temporary earth structure or structure made using engineered components (e.g., sheet piles, soldier piles/lagging, etc.), designed to isolate the work and enable construction under dry conditions. The water level in the river at the time of construction is a key risk factor, and therefore winter construction is recommended (to minimize this risk and reduce the robustness of cofferdams). The contractor should be responsible for diverting flow using a method acceptable to the engineer and in accordance with existing guidelines for in-stream work and should be responsible for its installation and removal.

Seepage cut-offs on all four sides of the work area are recommended. Cobbles and boulders are known to be present within till soils. Installation of sheet piles may encounter difficulties with maintaining pile alignment, separation of joints, and pile integrity in the event cobbles or boulders are encountered within the till.

10.0 Inspection Requirements

In accordance with Engineers and Geoscientists of Manitoba, a Professional Engineer, or delegated staff responsible to them must perform site reviews for the work presented in the documents they've sealed.

For conformance with the EGM requirements, TREK should be retained on a full-time basis to observe and document the installation of all pile foundations, shoring or engineered fills supporting the structure, and on an as-required basis for other components such as subgrade inspections and compaction testing.

TREK is familiar with the geotechnical conditions present and the underlying design assumptions of our foundation recommendations. TREK is therefore solely qualified to evaluate identify deviations from design assumptions during installation and to evaluate any design modifications deemed to be necessary should altered subsurface conditions be encountered.

11.0 Closure

The geotechnical 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 and laboratory testing). Soil conditions are natural deposits that can be highly variable across a site. If sub-surface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

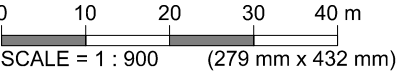
All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already 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 Engineering Inc. (the Consultant) for the exclusive use of Tetra Tech Inc. (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



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LEGEND:

TEST HOLE (KGS, 2020)

TEST HOLE (TREK, 2025)

NOTES:

1.

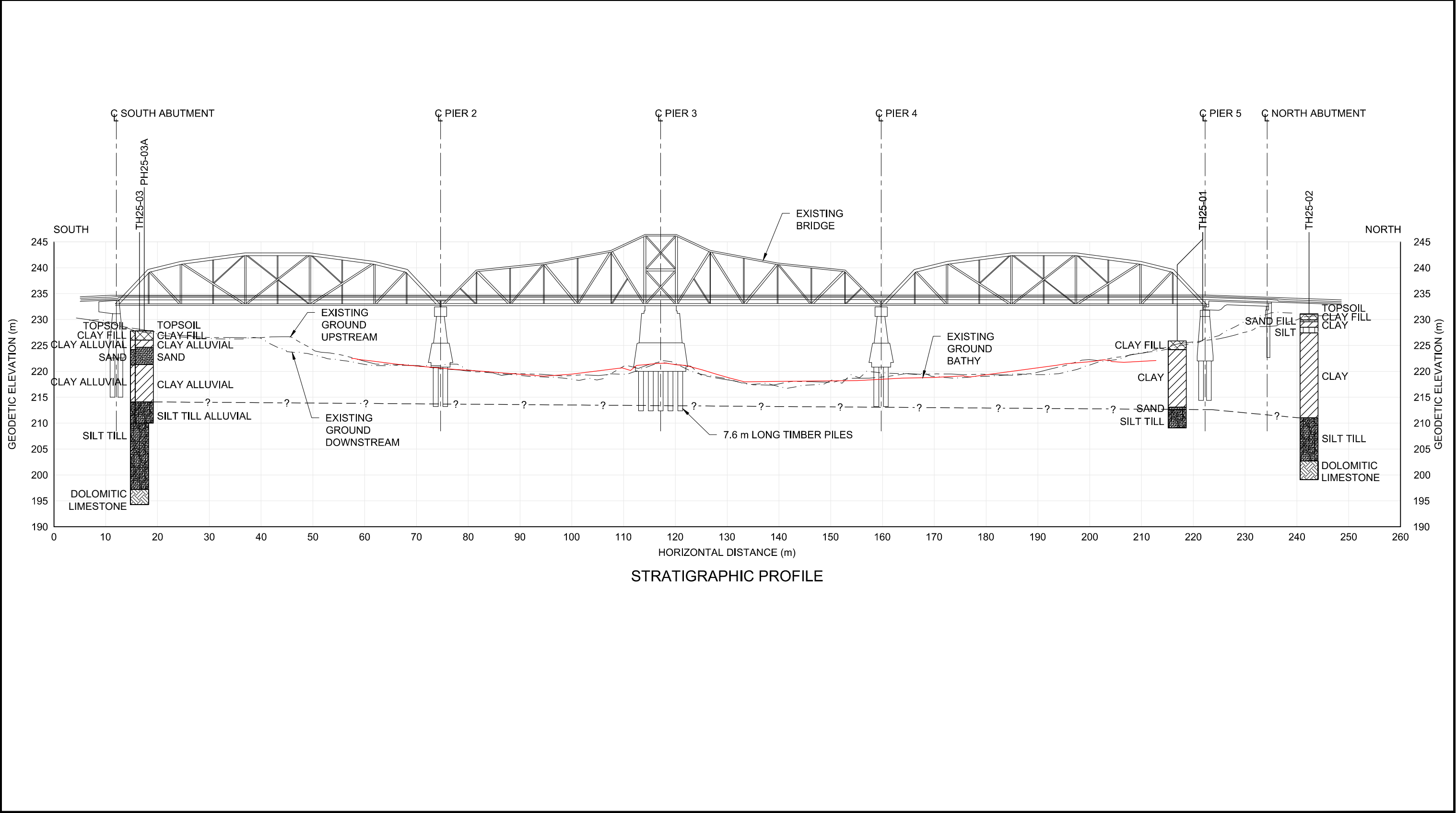
AERIAL IMAGERY FROM CITY OF WINNIPEG (2024).

2.

TEST HOLE LOCATIONS AND GEODETIC ELEVATIONS WERE ESTABLISHED USING AN RTK - GPS UNIT EQUIPPED WITH CAN-NET.

Figure 01

Test Hole Location Plan

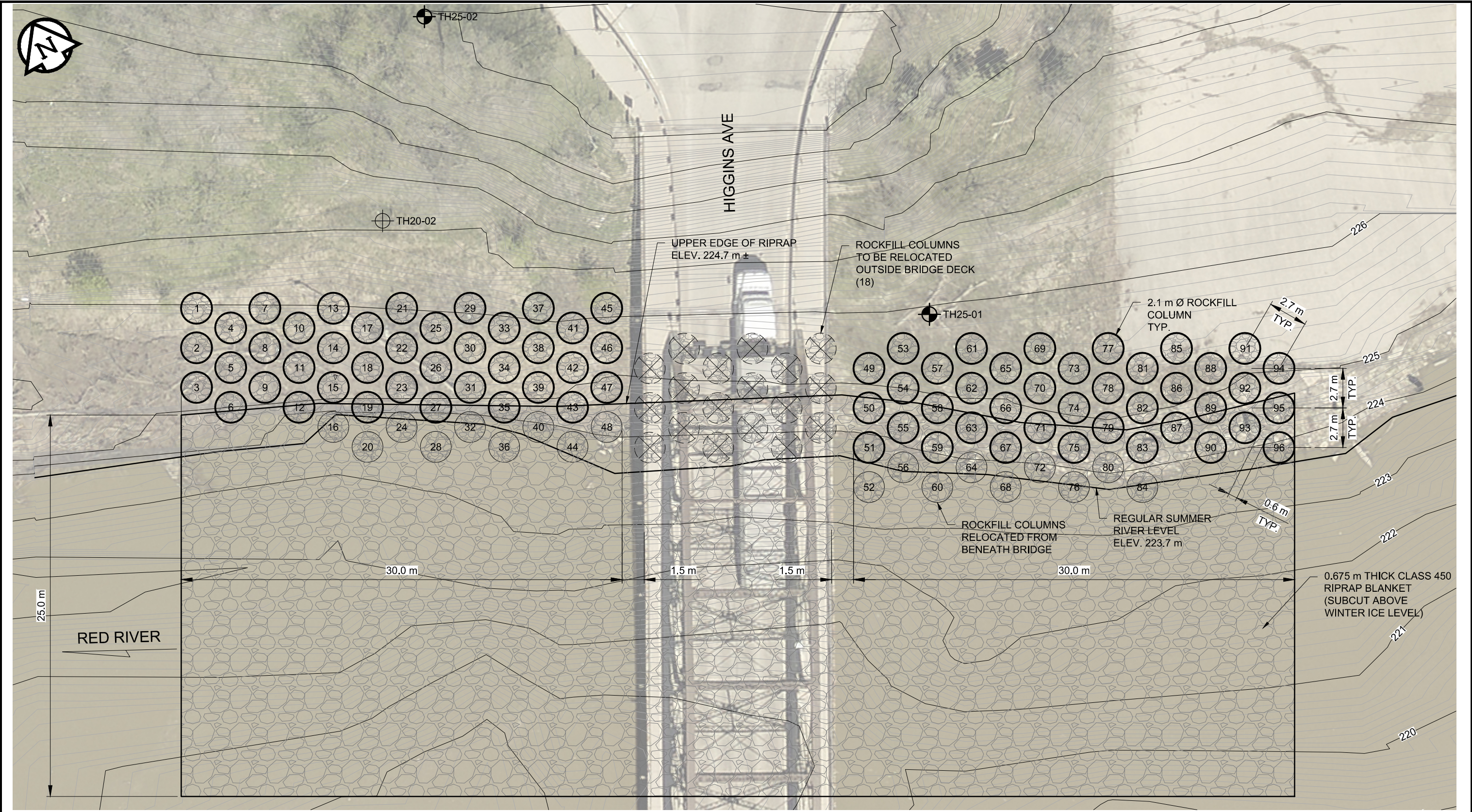


0 10 20 30 m
SCALE = 1 : 700 (279 mm x 432 mm)

Figure 02
Stratigraphic Profile



Z:\Projects\0002 Tetra Tech\0002 171 00 Louise Bridge Rehabilitation PD\3 Survey and Dwg\3.4 CAD\3.4.3 Working Folder\Figs 03 & 04 2026-02-25 Louise Bridge Rehab 0_A 0002-171-00.dwg, 2026-02-25 12:59:57 PM



0 5 10 15 m
SCALE = 1 : 250 (279 mm x 432 mm)

LEGEND:

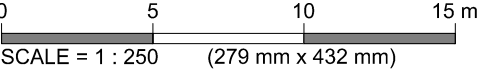
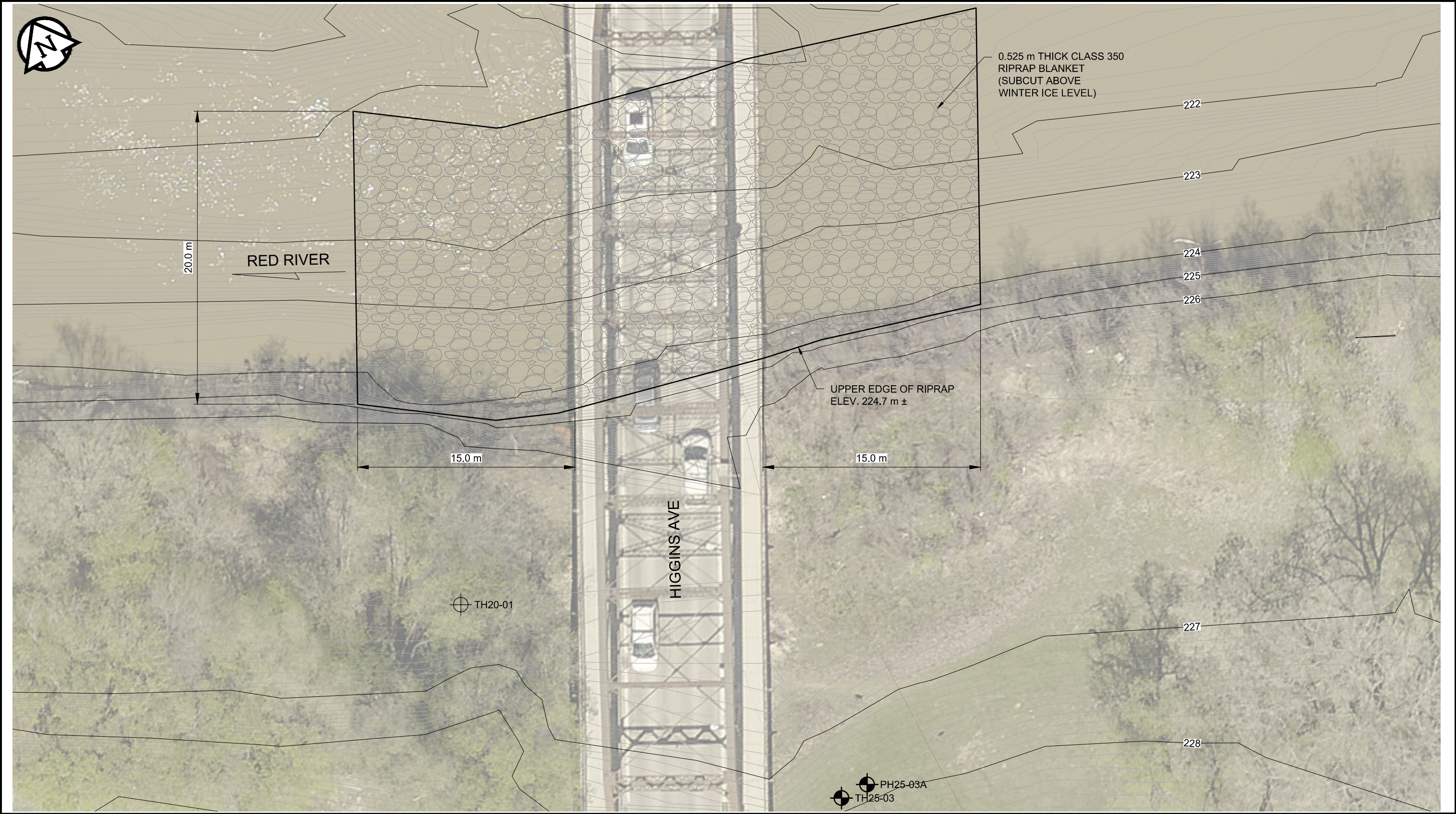
- TEST HOLE (TREK, 2025)
- TEST HOLE (KGS, 2020)
- EXISTING MAJOR CONTOUR (1.0 m INTERVAL)
- EXISTING MINOR CONTOUR (0.1 m INTERVAL)

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

Figure 03
Rockfill Column
Layout Plan



Z:\Projects\0002 Tetra Tech\0002 171 00 Louise Bridge Rehabilitation PD\3 Survey and Dwg\3.4.3 Working Folder\Figs 03 & 04 2026-02-25 Louise Bridge Rehab 0_A 0002-171-00.dwg, 2026-02-25 1:02:59 PM



LEGEND:

TEST HOLE (TREK, 2025)

TEST HOLE (KGS, 2020)

EXISTING MAJOR CONTOUR (1.0 m INTERVAL)

EXISTING MINOR CONTOUR (0.1 m INTERVAL)

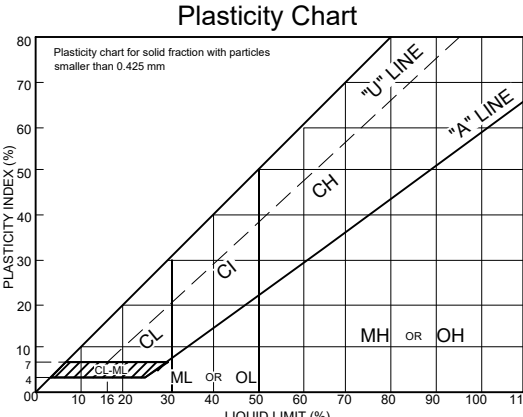
NOTES: 1. AERIAL IMAGERY FROM CITY OF WINNIPEG (2024).

Figure 04
Riprap
Layout Plan

Sub-Surface Logs

GENERAL NOTES

- Classifications are based on the Unified Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
- Descriptions on these test hole logs apply only at the specific test hole locations and at the time the test holes were drilled. Variability of soil and groundwater conditions may exist between test hole locations.
- When the following classification terms are used in this report or test hole logs, the primary and secondary soil fractions may be visually estimated.

Major Divisions			USCS Classification	Symbols	Typical Names	Laboratory Classification Criteria		Particle Size		Material			
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than 4.75 mm)	Clean gravel (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent..... GW, GP, SW, SP More than 12 percent..... GM, GC, SM, SC 6 to 12 percent..... Borderline cases requiring dual symbols*	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	mm	ASTM Sieve sizes	#10 to #4 #40 to #10 #200 to #40 < #200			
			GP		Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW						
			GM		Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4				Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols		
			GC		Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7						
	Sands (More than half of coarse fraction is smaller than 4.75 mm)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	mm	ASTM Sieve sizes	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425 < 0.075			
			SP		Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW						
			SM		Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4				Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols		
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7						
Fine-Grained soils (More than half the material is smaller than No. 200 sieve size)	Silt and Clays (Liquid limit less than 50)	ML		Inorganic silts and very fine sands, rock floor, silty or clayey fine sands or clayey silts with slight plasticity	Plasticity Chart 	mm	ASTM Sieve Sizes	Material	Boulders Cobbles Gravel Coarse Fine				
		CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays									
		OL		Organic silts and organic silty clays of low plasticity									
	Silt and Clays (Liquid limit greater than 50)	MH		Inorganic silts, micaceous or distomaceous fine sandy or silty soils, organic silts									
		CH		Inorganic clays of high plasticity, fat clays									
		OH		Organic clays of medium to high plasticity, organic silts									
	Highly Organic Soils	Pt		Peat and other highly organic soils						Von Post Classification Limit		Strong colour or odour, and often fibrous texture	

* Borderline classifications used for soils possessing characteristics of two groups are designated by combinations of groups symbols.
For example; GW-GC, well-graded gravel-sand mixture with clay binder.

Other Symbol Types

	Asphalt		Bedrock (undifferentiated)		Cobbles
	Concrete		Limestone Bedrock		Boulders and Cobbles
	Fill		Cemented Shale		Silt Till
			Non-Cemented Shale		Clay Till

LEGEND OF ABBREVIATIONS AND SYMBOLS

LL - Liquid Limit (%)	VW - Vibrating Wire Piezometer
PL - Plastic Limit (%)	SI - Slope Inclinator
PI - Plasticity Index (%)	▽ Water Level at Time of Drilling
MC - Moisture Content (%)	▼ Water Level at End of Drilling
SPT - Standard Penetration Test	▽ Water Level After Drilling as Indicated on Test Hole Logs
RQD- Rock Quality Designation	
Qu - Unconfined Compression	
Su - Undrained Shear Strength	

FRACTION OF SECONDARY SOIL CONSTITUENTS ARE BASED ON THE FOLLOWING TERMINOLOGY

TERM	EXAMPLES	PERCENTAGE
and	and CLAY	35 to 50 percent
"y" or "ey"	clayey, silty	20 to 35 percent
some	some silt	10 to 20 percent
trace	trace gravel	1 to 10 percent
with *	with silt, with sand	> 35 percent

* Used when the material is classified based on behaviour as a cohesive material

TERMS DESCRIBING CONSISTENCY OR COMPACTION CONDITION

The Standard Penetration Test blow count (N) of a non-cohesive soil can be related to compactness condition as follows:

<u>Descriptive Terms</u>	<u>SPT (N) (Blows/300 mm)</u>
Very loose	< 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	> 50

The Standard Penetration Test blow count (N) of a cohesive soil can be related to its consistency as follows:

<u>Descriptive Terms</u>	<u>SPT (N) (Blows/300 mm)</u>
Very soft	< 2
Soft	2 to 4
Firm	4 to 8
Stiff	8 to 15
Very stiff	15 to 30
Hard	> 30

The undrained shear strength (Su) of a cohesive soil can be related to its consistency as follows:

<u>Descriptive Terms</u>	<u>Undrained Shear Strength (kPa)</u>
Very soft	< 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	> 200

EXPLANATION OF ROCK CLASSIFICATION

In general accordance with the
Canadian Foundation Engineering Manual, 5th Edition (2023)

ROCK LITHOLOGY DESCRIPTION FORMAT	(underlined items described further in tables below)
PREDOMINANT ROCK TYPE (% by volume), <u>weathering state</u>, <u>grain size</u>, structure, colour, <u>strength</u>, <u>minor rock type</u> (% by volume) (Rock Formation). – Discontinuity descriptions of <u>roughness</u>, <u>aperture</u>, <u>evidence of water flow or infilling</u>, <u>joint spacing</u>, <u>rock quality designation</u>. <i>(items in italics are provided only if applicable)</i>	

INTACT ROCK STRENGTH CLASSIFICATIONS					(as per Table 4.21 of CFEM 2023)
Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field Estimate of Strength	Examples
R6	Extremely strong	>250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100-250	4-10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite, rhyolite, tuff
R4	Strong	50-100	2-4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium Strong	25-50	1-2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concrete, phyllite, schist, siltstone
R2	Weak	5-25	***	Can be peeled with a pocket knife with difficulty, shallow indentation made by a firm blow with the point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rock-salt
R1	Very weak	1-5	***	Crumbles under firm blows with point of a geological hammer, can be peeled with a pocket knife	Highly weathered or altered rock, shale
R0	Extremely weak	0.25-1	***	Indented by thumbnail	Stiff fault gouge
* Grade according to ISRM (1981).					
** All rock types exhibit a broad range of uniaxial comprehensive strengths reflecting heterogeneity in composition and anisotropy in structure. Strong rocks are characterized by well-interlocked crystal fabric and few voids.					
*** Rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results under point load testing.					

WEATHERING STATE CLASSIFICATIONS			(as per Table 4.18 of CFEM 2023)
Term	Description	Symbol	
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	W1	
Slightly Weathered	Slightly weathered Discolouration indicates weathering of rock material on discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker than in its fresh condition.	W2	
Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones	W3	
Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	W4	
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	W5	
Residual Soil	The rock is completely changed to a soil in which the original rock texture has been completely destroyed.	W6	

BEDDING THICKNESS CLASSIFICATION (as per Table 4.17 of CFEM 2023)

Class	Bedding plane spacing	Class	Bedding plane spacing
Very thickly bedded (massive)	>2 m	Very thinly bedded	>20 to 60 mm
Thickly bedded	>0.6 to 2 m	Laminated	6 to 20 mm
Medium bedded	>0.2 to 0.6 m	Thinly laminated	<6 mm
Thinly bedded	>60mm to 0.2 m		

JOINT APERTURE CLASSIFICATION (as per Table 4.22 of CFEM 2023)

Aperture	Description
<0.5 mm	Closed
0.5 to 10mm	Gapped
> 10 mm	Open

UNFILLED JOINT CLASSIFICATION (as per Table 4.23 of CFEM 2023)

Class	Description
1	Water flow not possible
2	No evidence of water flow
3	Evidence of water flow (e.g. rust staining)
4	Dampness
5	Seepage
6	Flow (volume per unit time)

FILLED JOINT CLASSIFICATION (as per Table 4.24 of CFEM 2023)

Class	Description
1	Filling is dry and low permeability
2	Filling is damp; no free water present
3	Filling is wet; drops of free water present
4	Filling shows outwash; continuous water flow present
5	Filling is locally washed out and there is considerable water flow along channels

JOINT SPACING CLASSIFICATION (as per Table 4.25 of CFEM 2023)

Class	Spacing width (m)
Extremely Close	<0.02
Very Close	0.02 to <0.06
Close	0.06 to <0.20
Moderately Close	0.20 to <0.6
Wide	0.6 to <2.0
Very Wide	2.0 to 6.0
Extremely Wide	>6.0

ROCK QUALITY DESIGNATION (RQD) CLASSIFICATION (as per Table 4.26 and Fig 4.5 of CFEM 2023)

RQD Class	RQD Value (%)
Very poor quality	< 25
Poor quality	25 to <50
Fair quality	50 to <75
Good quality	75 to <90
Excellent quality	90 to 100

$$RQD = \frac{\sum \text{Core pieces} > 100 \text{ mm}}{\text{Total length of core run}} \times 100$$

JOINT ROUGHNESS AND PROFILE (based on Fig 4.4 of CFEM 2023)

Description	Profile
Rough	
Smooth	
Slickensided	
	Stepped
Rough	
Smooth	
Slickensided	
	Undulating
Rough	
Smooth	
Slickensided	
	Planar

Roughness assessed on small scale (<20cm)
Profile assessed on large (metre) scale



Sub-Surface Log

Test Hole TH25-01

1 of 2

Client: Tetra Tech Inc.

Project Name: Louise Bridge Rehabilitation

Contractor: Paddock Drilling Ltd.

Method: 125 mm Solid Stem Auger / HQ Coring, Mobile B57 Track Mounted Rig

Project Number: 0002-171-00

Location: 14U, 5529907 m N, 635496 m E

Ground Elevation: 225.89 m

Date Drilled: 3 April 2025

Sample Type:

Grab (G)

Shelby Tube (T)

Split Spoon (SS) / SPT

Split Barrel (SB) / LPT

Core (C)

Particle Size Legend:

Fines

Clay

Silt

Sand

Gravel

Cobbles

Boulders

Backfill Legend:

Bentonite

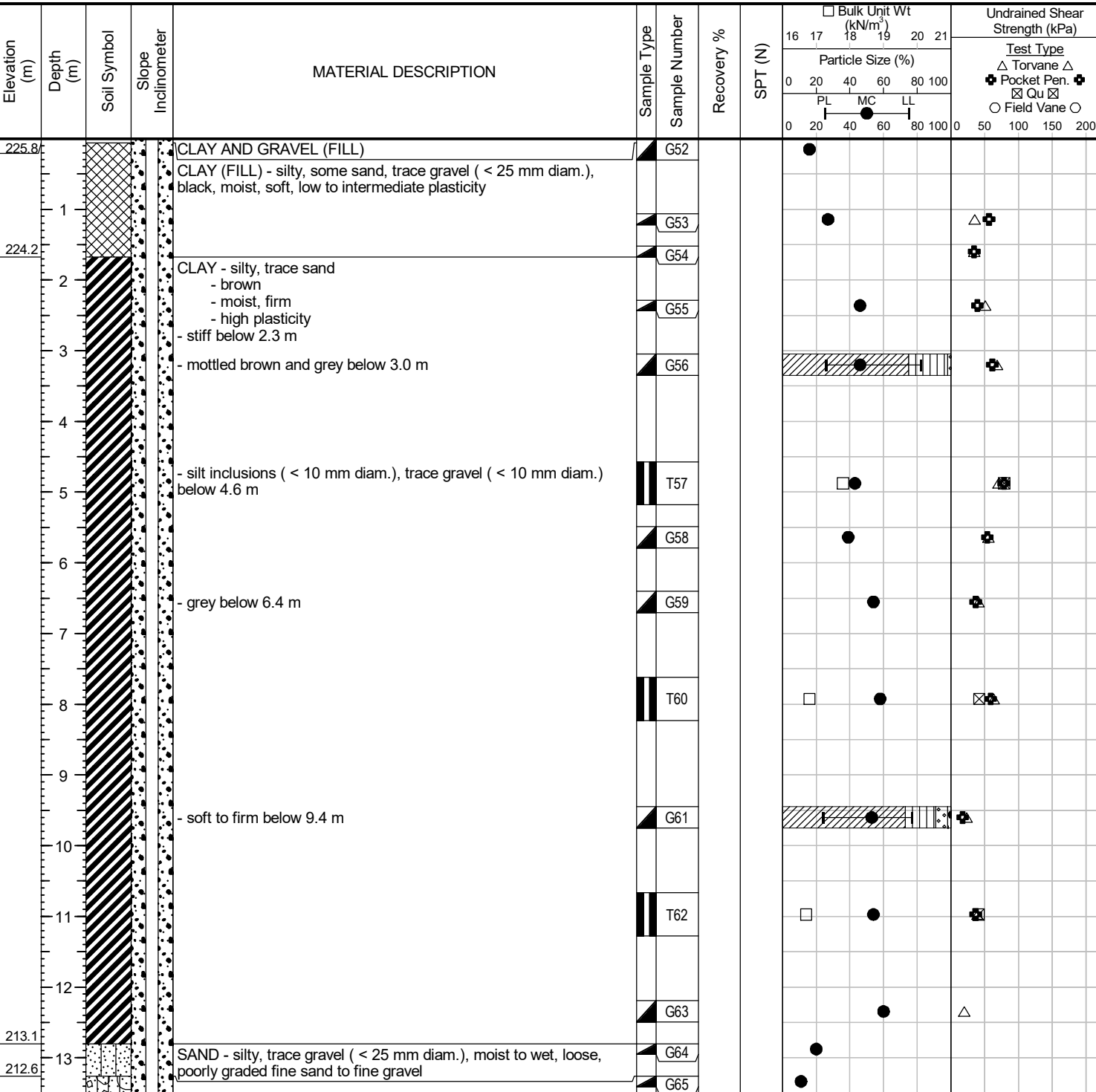
Cement

Drill Cuttings

Filter Pack Sand

Grout

Slough



Logged By: David Clark

Reviewed By: Michael Van Helden

Project Engineer: Nuno Mendonca

SUB-SURFACE LOG LOGS 2025-05-29 LOUISE BRIDGE REHABILITATION 0_C_DC 0002-0171-00.GPJ TREK.GDT 15/7/25



Sub-Surface Log

Elevation (m)	Depth (m)	Soil Symbol	Slope Inclinometer	MATERIAL DESCRIPTION	Sample Type	Sample Number	Recovery %	SPT (N)	Bulk Unit Wt (kN/m³) Particle Size (%) PL MC LL 16 17 18 19 20 21 0 20 40 60 80 100 0 20 40 60 80 100	Undrained Shear Strength (kPa)	Test Type
											△ Torvane △ ✦ Pocket Pen. ✦ ⊠ Qu ⊠ ○ Field Vane ○
	14			SILT (TILL) - sandy, trace clay, some gravel (< 50 mm diam.) - light grey - moist to wet, very dense - no to low plasticity	✕	SS66		80			
	15										
	16				✕	SS67		100			
209.1											

END OF TEST HOLE AT 16.8 m IN SILT (TILL)

Notes:

- 1) Seepage and sloughing observed at 12.8 m depth.
- 2) Drilling method was switched to HWT casing at 12.8 m depth. Water level not measured due to drilling method.
- 3) Slope Inclinometer SI25-01 installed to 16.2 m depth.
- 4) Test hole backfilled with bentonite-cement grout mix to 1.0 m depth and bentonite chips to surface.

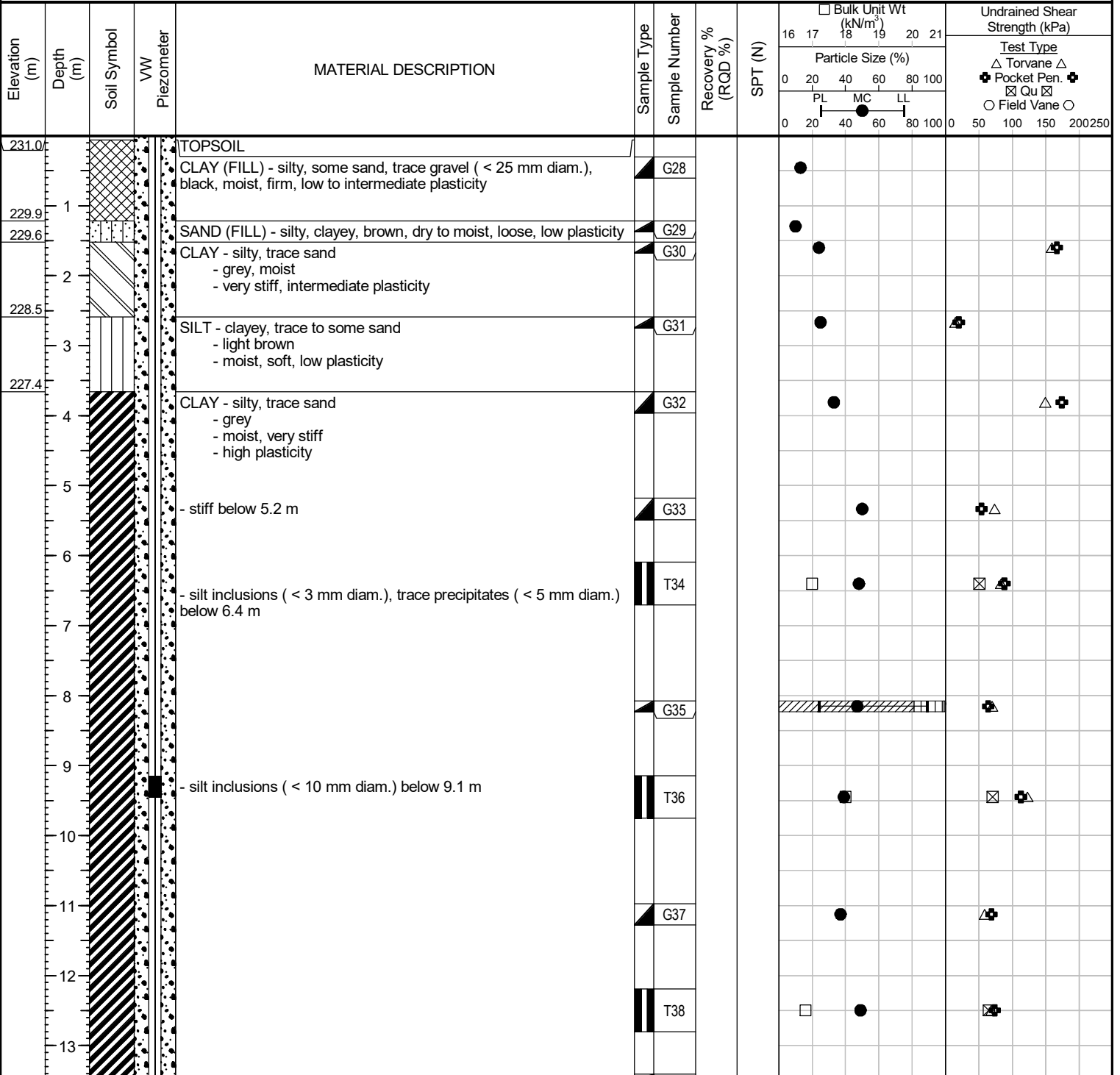
Sub-Surface Log

Test Hole TH25-02

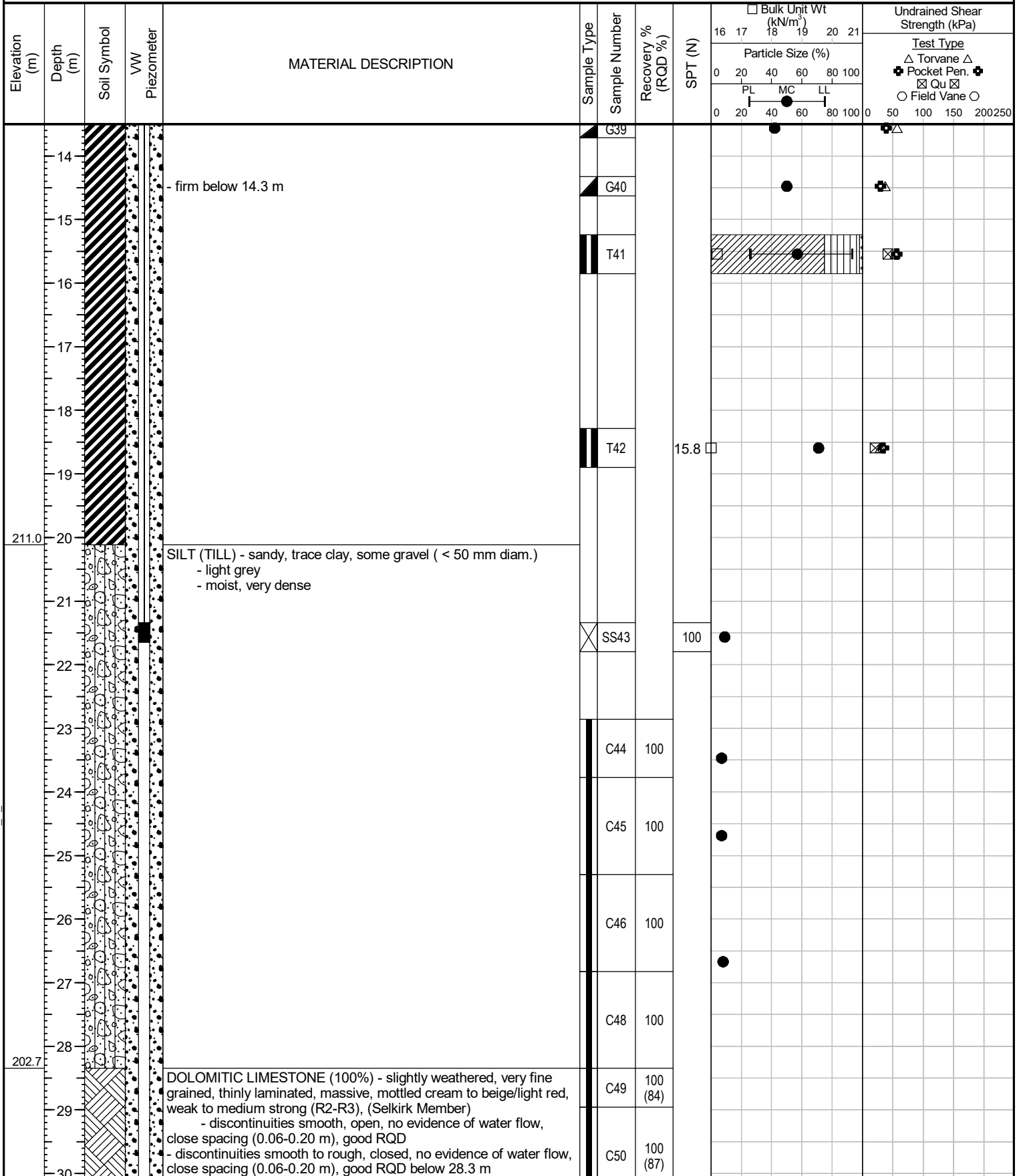
1 of 3

Client: Tetra Tech Inc. **Project Number:** 0002-171-00
Project Name: Louise Bridge Rehabilitation **Location:** 14U, 5529946 m N, 635475 m E
Contractor: Paddock Drilling Ltd. **Ground Elevation:** 231.09 m
Method: 125 mm Solid Stem Auger / HQ Coring, Mobile B57 Track Mounted Rig **Date Drilled:** 2 April 2025

Sample Type:  Grab (G)  Shelby Tube (T)  Split Spoon (SS) / SPT  Split Barrel (SB) / LPT  Core (C)
Particle Size Legend:  Fines  Clay  Silt  Sand  Gravel  Cobbles  Boulders
Backfill Legend:  Bentonite  Cement  Drill Cuttings  Filter Pack Sand  Grout  Slough



Logged By: David Clark **Reviewed By:** Michael Van Helden **Project Engineer:** Nuno Mendonca



Elevation (m)	Depth (m)	Soil Symbol	VW Piezometer	MATERIAL DESCRIPTION	Sample Type	Sample Number	Recovery % (RQD %)	SPT (N)	Bulk Unit Wt (kN/m ³)					Undrained Shear Strength (kPa)						
									16	17	18	19	20	21	Test Type					
									Particle Size (%)											
									0	20	40	60	80	100						
									PL	MC	LL									
									0	20	40	60	80	100	0	50	100	150	200	250
199.1	31			- compressive strength of 38.1 MPa at 28.8 m - discontinuities rough, closed, no evidence of water flow, close spacing (0.06-0.20 m), excellent RQD below 29.0 m - compressive strength of 36.2 MPa at 29.3 m - compressive strength of 38.4 MPa at 30.5 m		C51	100 (90)													

END OF TEST HOLE AT 32.0 m IN DOLOMITIC LIMESTONE

Notes:

- 1) Seepage and sloughing not observed due to drilling method.
- 2) Vibrating Wire Piezometers VW25-02A (SN-189698) installed at 21.3 m depth, and VW25-02B (SN-126131) installed at 9.1 m depth.
- 3) Test hole backfilled with bentonite-cement grout mix to 2.0 m depth and bentonite chips to surface.
- 4) Drilling method switched to HQ coring at 22.9 m.

Project Number: 0002-171-00

Location: 14U, 5529720 m N, 635400 m E

Ground Elevation: 227.81 m

Date Drilled: 30 March 2025

Backfill Legend: Bentonite Cement Drill Cuttings Filter Pack Sand Grout Slough



Elevation (m)	Depth (m)	Soil Symbol	VW	Piezometer	Slope Inclinometer	MATERIAL DESCRIPTION	Sample Type	Sample Number	Recovery % (RQD %)	SPT (N)	Bulk Unit Wt (kN/m ³)					Undrained Shear Strength (kPa)					
											16	17	18	19	20	21	Test Type				
											Particle Size (%)										
											0	20	40	60	80	100	△ Torvane △	✦ Pocket Pen. ✦	⊠ Qu ⊠	○ Field Vane ○	
												PL	MC	LL	0	50	100	150	200	250	

214.1						- boulder at 13.7 m	✕	SS16A		100											
	14					SILT (TILL) - trace clay, some sand, some gravel (< 25 mm diam.)															
	15					- light grey															
	16					- moist, loose to compact															
	17					- no to low plasticity															
	18					- very dense below 18.0 m															
	19																				
	20						✕	SS18		100											
	21					- boulder at 21.0 m	✕	SS19		100											
	22																				
	23					- boulder at 22.9 m															
	24																				
	25																				
	26																				
	27																				
	28																				
	29																				
	30																				

Elevation (m)	Depth (m)	Soil Symbol	VW	Slope Inclinometer	MATERIAL DESCRIPTION	Sample Type	Sample Number	Recovery % (RQD %)	SPT (N)	Bulk Unit Wt (kN/m ³)					Undrained Shear Strength (kPa)				
										Particle Size (%)					Test Type				
										16 17 18 19 20 21									
										0 20 40 60 80 100									
										PL MC LL									
										0 20 40 60 80 100									
										0 50 100 150 200 250									
197.2					DOLOMITIC LIMESTONE (100%) - slightly weathered, very fine grained, thinly laminated, massive, mottled cream to beige, weak to medium strong (R2-R3), (Selkirk Member)		C26	100 (62)											
31					- discontinuities rough, closed, no evidence of water flow, damp, no free water, close spacing (0.06-0.20 m), fair RQD														
32					- compressive strength of 37.6 MPa at 31.5 m														
33					- discontinuities rough, gapped, no evidence of water flow, damp, close spacing (0.06-0.20 m), excellent RQD below 32.0 m		C27	100 (93)											
194.3					- compressive strength of 46.6 MPa at 32.0 m														

END OF TEST HOLE AT 33.5 m IN DOLOMITIC LIMESTONE

Notes:

- 1) Seepage and sloughing not observed due to drilling method.
- 2) Drilling method changed to HQ coring at 15.2 m.
- 3) Vibrating Wire Piezometers VW25-03A (SN-188353) installed at 15.5 m depth, and VW25-03B (SN-183291) installed at 7.9 m depth.
- 4) Slope Inclinometer SI25-03 installed 2.0 m east of TH25-03 to a depth of 18.5 m.
- 5) Test hole backfilled with bentonite-cement grout mix to 2.0 m depth and bentonite chips to surface.

CLIENT	WSP CANADA GROUP LIMITED	PROJECT NO.	17-0183-001
PROJECT	Eastern Corridor Study - Louise Bridge	SURFACE ELEV.	226.25 m
LOCATION	Winnipeg, Manitoba	TOC STICK-UP / ELEV.	0.91 m / 227.16 m (Inclinometer)
DESCRIPTION	South abutment, mid bank, 4 m west of the bridge	DATE DRILLED	7/27/2020
DRILL RIG / HAMMER	Acker MP5 Track Mounted Drill Rig with Auto-Hammer	UTM (m)	N 5,529,747
METHOD(S)	0.0 m to 12.8 m: 125 mm ø SSA		E 635,391

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEV (m)	PIEZO. LOG		SAMPLE TYPE	NUMBER	BLOWS/0.15 m	N-VALUE	PL MC LL Cu TORVANE (kPa) ◆ Cu POCKET PEN (kPa) ★ SPT (N) BLOWS/0.30 m ▲			
					WATER LEVEL	DIAGRAM								
226			CLAY FILL - Brown, damp, stiff, high plasticity.											
	1			225.3				S1						
	5		POORLY GRADED SAND (SP) - Black, damp, medium to coarse grained, some organics.	224.7				S2						
	2		CLAY (CI) - Grey, damp, firm, intermediate plasticity, some fine grained sand.					S3						
	3		- Soft, some medium grained sand, some silt below 3.0 m.					S4						
	4		- LL=47, PL=18, PI=29 at 3.8 m.					S5						
	5							S6						
	6		POORLY GRADED SAND (SP) - Dark grey, moist, loose, fine to medium grained, some silt.	220.8				S7						
	7			219.2				S8	2	2				
	8		CLAY (CI) - Grey, damp, firm, intermediate plasticity, trace silt inclusions, some fine to coarse grained sand.					S9	1					
	9		- Some fine gravel below 7.6 m.					S10						
	10		- LL=39, PL=15, PI=24 at 7.9 m.					S11						
	11			217.1				S12						
	12		CLAY (CH) - Grey, damp, stiff, high plasticity, some fine to medium grained sand, cemented.	216.6				S13	2	10				
	13		SILT (ML) - Light grey, damp, soft, low plasticity, trace fine to medium grained sand.					S14	4					

WATER LEVELS ▼ Upon Completion of Drilling on 7/27/2020 None Encountered

CONTRACTOR
Maple Leaf Drilling Ltd.

INSPECTOR
N. BRAY

APPROVED
T. ERNST

DATE
9/24/2020

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	WATER LEVEL	PIEZO. LOG		SAMPLE TYPE	NUMBER	BLOWS/0.15 m	N-VALUE	PL MC LL Cu TORVANE (kPa) ◆ Cu POCKET PEN (kPa) ★ SPT (N) BLOWS/0.30 m ▲			
					DIAGRAM	DEPTH (m)								
216			ELEV (m) 215.9					S15						
	35		CLAY (CH) - Grey, moist, stiff, high plasticity, trace silt inclusions (1 to 5 mm dia.), cemented.		Pneumatic 038336	10.6								
	11		- Soft below 11.0 m.											
215			- LL=75, PL=20, PI=55 at 11.3 m.					S16						
	12		ELEV (m) 214.4					S17						
214	40		SILT TILL - Light brown, dry, very dense, non-plastic, some fine to medium grained, some fine gravel.											
			ELEV (m) 213.4		Pneumatic 038335	12.6		S18	9 16 45	61				
			Notes: 1. End of test hole at 12.8 m. 2. Power auger refusal encountered in till at a depth of 12.8 m. 3. Slope inclinometer installed to a depth of 12.7 m. 4. Locking protective cover installed at surface. 5. Test hole backfilled with grout.			12.7								
213														
	45													
212														
	15													
211	50													
	16													
210														
	55													
209	17													
	18													
208	60													
	19													
207	65													
	20													
206														
	21													
205	70													

WATER LEVELS ▼ Upon Completion of Drilling on 7/27/2020 None Encountered

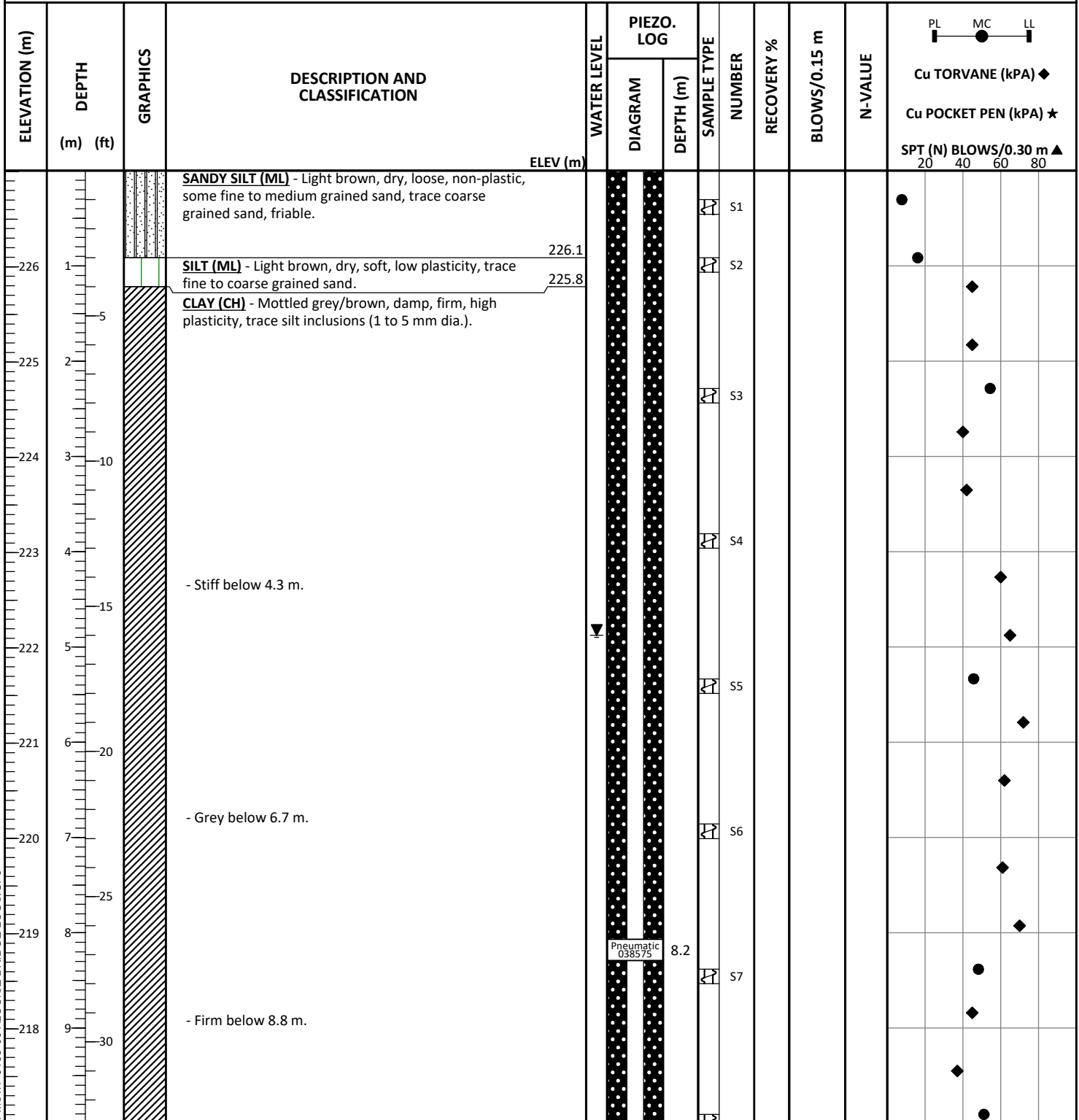
CONTRACTOR
Maple Leaf Drilling Ltd.

INSPECTOR
N. BRAY

APPROVED
T. ERNST

DATE
9/24/2020

CLIENT	WSP CANADA GROUP LIMITED	PROJECT NO.	17-0183-001
PROJECT	Eastern Corridor Study - Louise Bridge	SURFACE ELEV.	227.01 m
LOCATION	Winnipeg, Manitoba	TOC STICK-UP / ELEV.	0.99 m / 228.00 m (Inclinometer)
DESCRIPTION	North abutment, mid bank, 17 m west of the bridge	DATE DRILLED	7/28/2020
DRILL RIG / HAMMER	Acker MP5 Track Mounted Drill Rig with Auto-Hammer	UTM (m)	N 5,529,926 E 635,466
METHOD(S)	0.0 m to 16.8 m: 125 mm ø SSA 0.0 m to 16.8 m: 200 mm ø HSA - switched due to sloughing		



WATER LEVELS	▼ Upon Completion of Drilling 4.88 m on 7/28/2020	CONTRACTOR Maple Leaf Drilling Ltd.	INSPECTOR N. BRAY
		APPROVED T. ERNST	DATE 9/24/2020

ELEVATION (m)	DEPTH (m) (ft)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	WATER LEVEL	PIEZO. LOG		SAMPLE TYPE	NUMBER	RECOVERY %	BLOWS/0.15 m	N-VALUE	PL MC LL Cu TORVANE (kPa) ◆ Cu POCKET PEN (kPa) ★ SPT (N) BLOWS/0.30 m ▲			
					DIAGRAM	DEPTH (m)						20	40	60	80
				ELEV (m)											
			- Increased moisture, soft below 10.4 m.					S8							
216	11														
			- LL=81, PL=24, PI=57 at 11.4 m.					S9							
215	12					12.1									
			- No silt inclusions below 12.2 m.												
214	13							S10							
213	14			213.0				S11							
			CLAY TILL - Grey, moist, soft, low to intermediate plasticity, trace to some fine to coarse grained sand, trace fine gravel.												
212	15			212.1				S12							
			SILT TILL - Light brown, dry to damp, dense, low plasticity, some fine to coarse grained sand, some fine gravel.					S13	89	17 15 20	35				
211	16							S14							
210	17			210.1		16.6		S15	100	50/ 130mm	+100				>>
			Notes: 1. End of test hole at 16.9 m. 2. Power auger refusal encountered in till at a depth of 16.9 m. 3. Slope inclinometer installed to a depth of 16.7 m. 4. Locking protective cover installed at surface. 5. Test hole backfilled with grout.			16.7									
209	18														
208	19														
207	20														
206	21														

WATER LEVELS ▼ Upon Completion of Drilling 4.88 m on 7/28/2020

CONTRACTOR
Maple Leaf Drilling Ltd.

INSPECTOR
N. BRAY

APPROVED
T. ERNST

DATE
9/24/2020



Louise Bridge Rehabilitation PD
TH25-02
Rock Core: C49
Depth: 28.3 m to 29.0 m



Project Number: 0002 171 00	Date: April 2, 2025	Location: 5529946 N, 635475 E El. 231.09 m	Rock Core Elevation 202.79 m to 202.09 m	Created By: DC	Reviewed By: MVH
---------------------------------------	-------------------------------	---	--	--------------------------	----------------------------



Louise Bridge Rehabilitation PD
TH25-02
Rock Core: C50 & C51
Depth: 29.0 m to 32.0 m



Project Number:
0002 171 00

Date:
April 2, 2025

Location:
5529946 N, 635475 E
El. 231.09 m

Rock Core Elevation
202.09 m to 199.09 m

Created By:
DC

Reviewed By:
MVH



Louise Bridge Rehabilitation PD
TH25-03
Rock Core: C26
Depth: 30.5 m to 32.0 m



Project Number:
0002 171 00

Date:
March 30, 2025

Location:
5529720 N, 635400 E
El. 227.81 m

Rock Core Elevation
197.31 m to 195.81 m

Created By:
DC

Reviewed By:
MVH



Louise Bridge Rehabilitation PD
TH25-03
Rock Core: C27
Depth: 32.0 m to 33.5 m



Project Number:
0002 171 00

Date:
March 30, 2025

Location:
5529720 N, 635400 E
El. 227.81 m

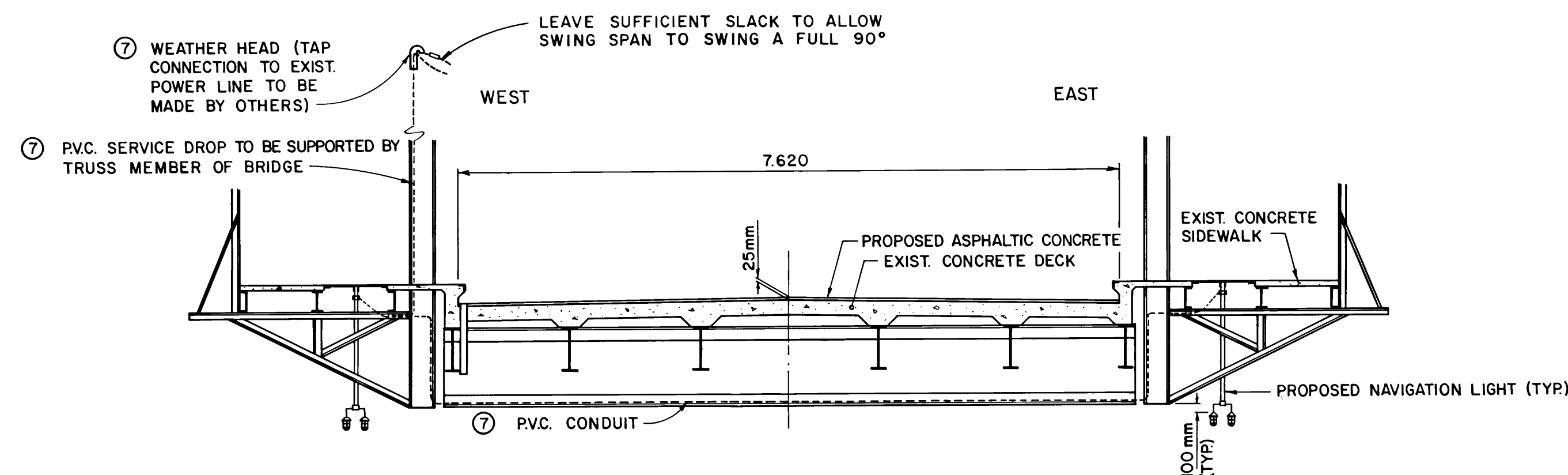
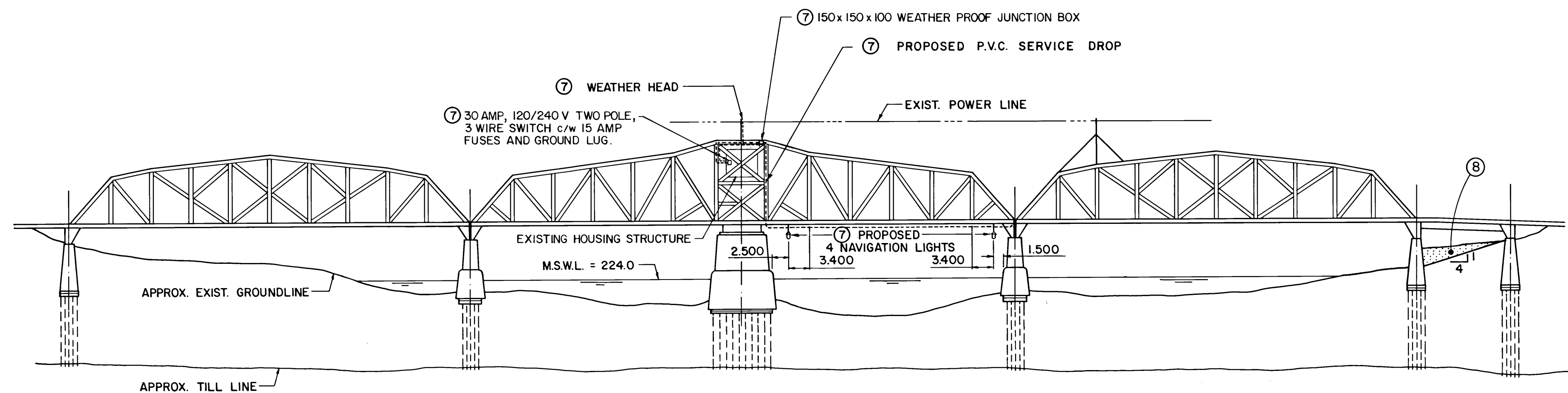
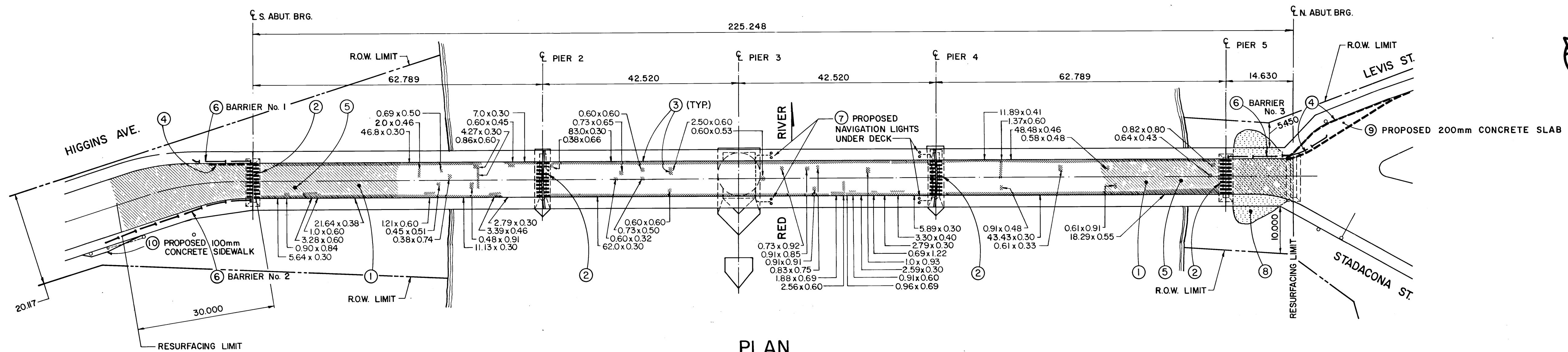
Rock Core Elevation
195.81 m to 194.31 m

Created By:
DC

Reviewed By:
MVH

Appendix A

Existing Drawings



NOTE:
- FOR NAVIGATION LIGHT DETAIL SEE DWG. No. 04
- ALL P.V.C. CONDUIT TO BE 25mm Ø AND SUPPORTED AT INTERVALS OF NO MORE THAN 750mm. (METHOD OF SUPPORT MUST BE APPROVED BY THE ENGINEER.)
- ALL P.V.C. SHALL HAVE 2 - #12 WIRES AND 1 - #12 GROUND WIRE.

REFERENCE DRAWING No.

02
04
04

SD 206
SD 207
09
06
04

02
06
06

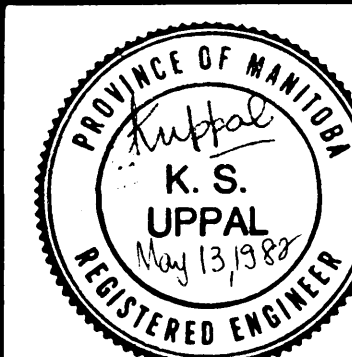
PROPOSED WORKS

- ① PLANE EXISTING ASPHALTIC CONCRETE OVERLAY ON BRIDGE DECK AND SOUTH APPROACH.
- ② RENEW BRIDGE DECK EXPANSION JOINTS.
- ③ RENEW DETERIORATED AREAS OF CONCRETE BRIDGE DECK SURFACE. (AREAS SHOWN ARE APPROXIMATE AREAS OF CONCRETE BRIDGE DECK SURFACE DETERIORATION. FINAL AREAS TO BE RENEWED TO BE IDENTIFIED AFTER REMOVAL OF EXISTING ASPHALTIC CONCRETE OVERLAY.)
- ④ RENEW CONCRETE CURBS AT APPROACHES.
- ⑤ RESURFACE BRIDGE DECK AND SOUTH APPROACH WITH ASPHALTIC CONCRETE OVERLAY.
- ⑥ INSTALL BALANCED ALUMINUM TRAFFIC BARRIER.
- ⑦ INSTALL NEW NAVIGATION LIGHT SYSTEM AND SUBSEQUENTLY REMOVE EXISTING NAVIGATION LIGHT SYSTEM.
- ⑧ EXCAVATE EMBANKMENT AT PIER 5.
- ⑨ PROPOSED 200mm CONCRETE SLAB.
- ⑩ PROPOSED 100mm CONCRETE SIDEWALK.

NOTE:
- ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE SHOWN.

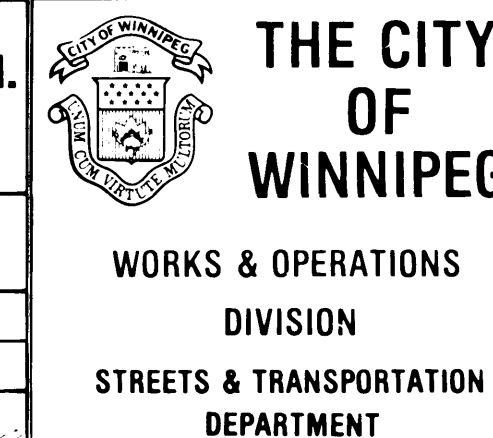
RECORD DRAWING

NO.	REVISIONS	DATE	APP.
1	REVISED TO RECORD DRAWING	6/06/85	JS



ENGINEER'S SEAL

the Underwood McLellan Ltd. Consulting Engineers and Planners	
DESIGNED BY: J.J.	DRAWN BY: W.F.S.
CHECKED BY: K.S.U.	DATE: MAY 1982 JOB No. 0265-202-01
APPROVED BY: [Signature]	DATE: May 13/82



THIS DRAWING IS METRIC	
LOUISE AND REDWOOD BRIDGES ASPHALT RESURFACING OF BRIDGE DECK AND RELATED WORKS	
LOUISE BRIDGE GENERAL ARRANGEMENT AND LOCATION OF PROPOSED WORKS	
AUTHORIZED BY: [Signature]	DATE: 8/20/84
ACCEPTED BY: [Signature]	DATE: 8/20/84
SCALE: AS SHOWN	CONTRACT DRAWING NO. 02 BRIDGE DRAWING NO. B107-82-02

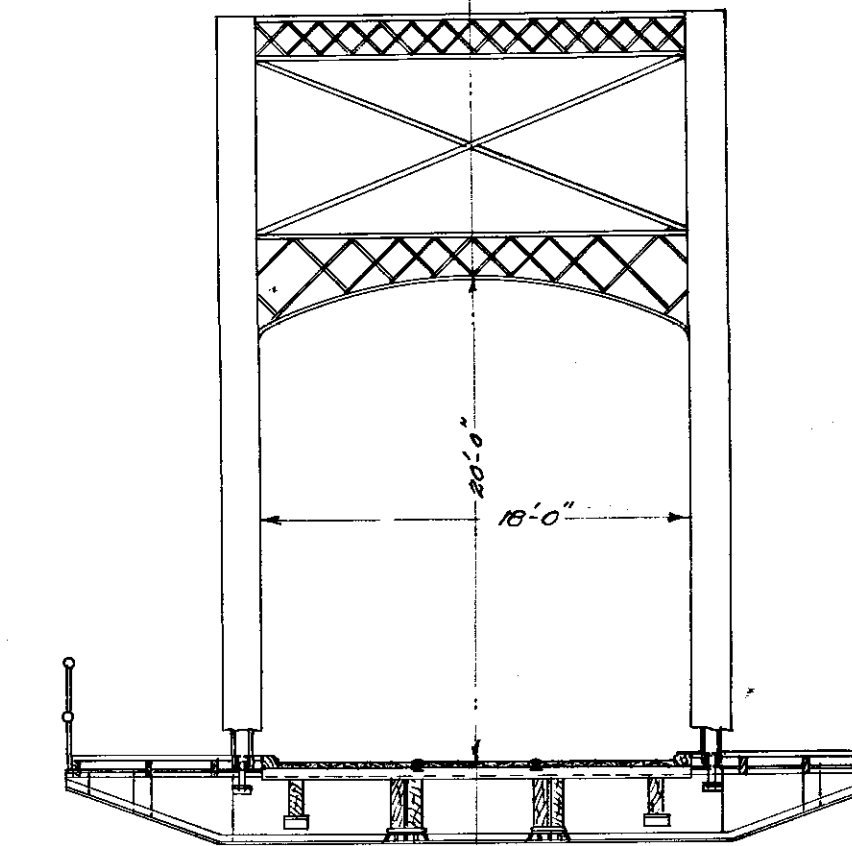
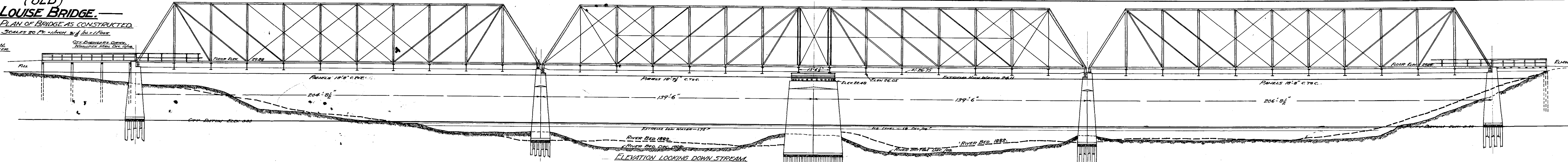
(OLD)
— LOUISE BRIDGE. —

PLAN OF BRIDGE AS CONSTRUCTED

5 SCALES 20 FT. = 1/4 INCH & 1/8 IN. = 1 FOOT

H. N. PUTTAN.
CITY ENGINEER.

CITY ENGINEER'S OFFICE.
WINNIPEG MAN. DEC. 17



#4003

~~SECRET~~

Plan 4001

Specification

Straighten & plumb the old ice breaker fender on the E. side of the bridge and do all the necessary work to make the connection between the old & new work substantial and effective.

Drive forty two bearing piles, as shown on plan, of not less than eight inches diameter at small ends, until the last blow, or its equivalent, of a hammer weighing two thousand pounds falling twenty feet, will not depress any pile more than one fourth of an inch. Also drive sheet piles of not less than twenty six feet in length six inches in diameter at small ends, in close contact all around the inside bearing piles & tie them in the iron straps & bolts as shown.

Cap the bearing piles & build the cribwork thereon of pine, oak or tamarac sawn or hewn to dimensions not less than eight inches wide for inside cribs and not less than ten inches wide for outside walls of cribs; splice the timbers on to each other at all intersections, & well spike them together with good wrought iron spikes of lengths equal to the thickness of the timbers in contact at each intersection. Also drive additional wrought iron spikes of sufficient lengths through the timbers of the outside walls at every three feet.

Fill in between the piles, & also fill the cribs, with large stones or rip rap.

8" tip diameter

Final set $\frac{1}{4}$ " or less per blow

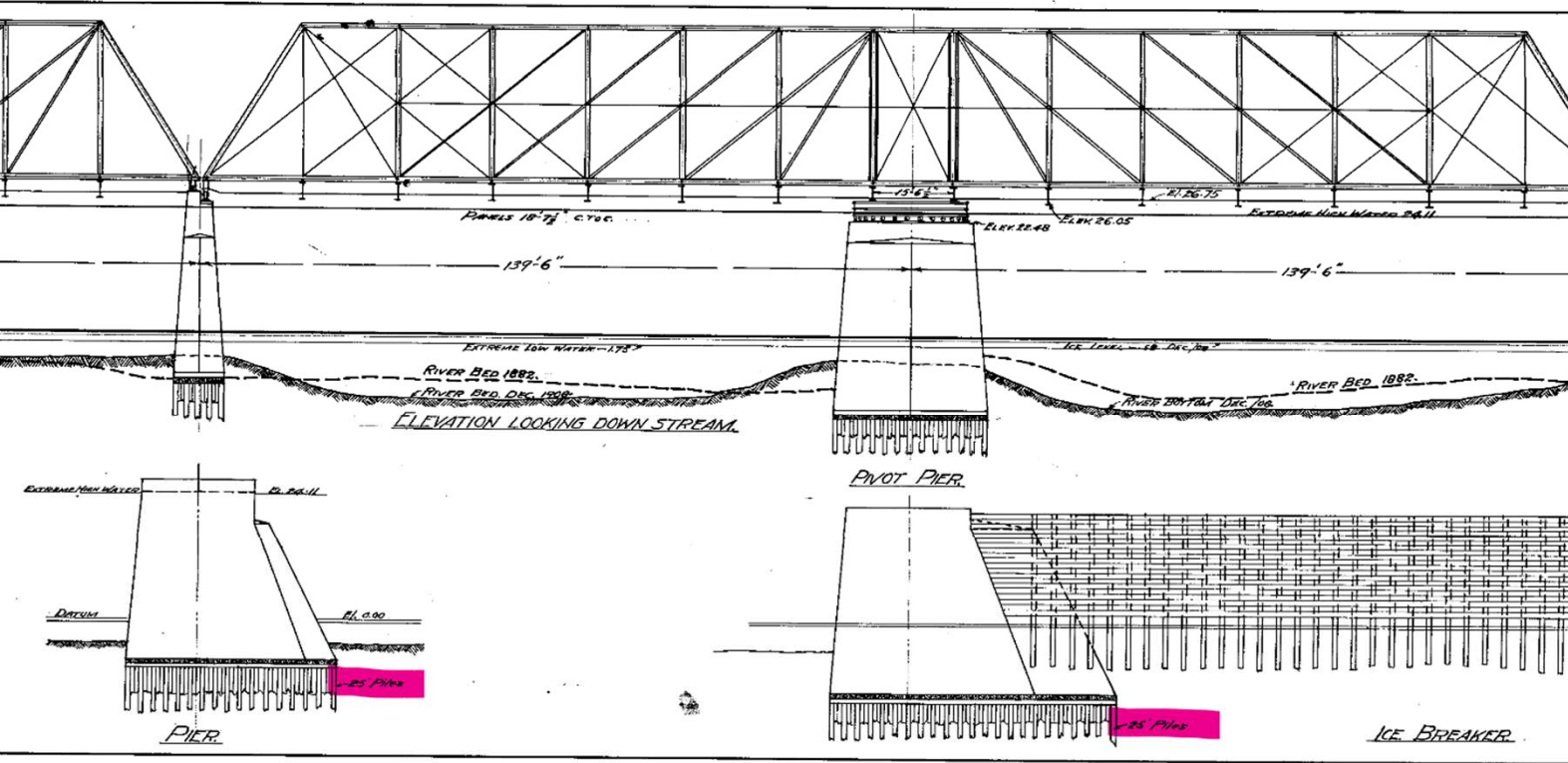
2,000 lb hammer, 20ft

drop = 40,000 ft-lbs = 54

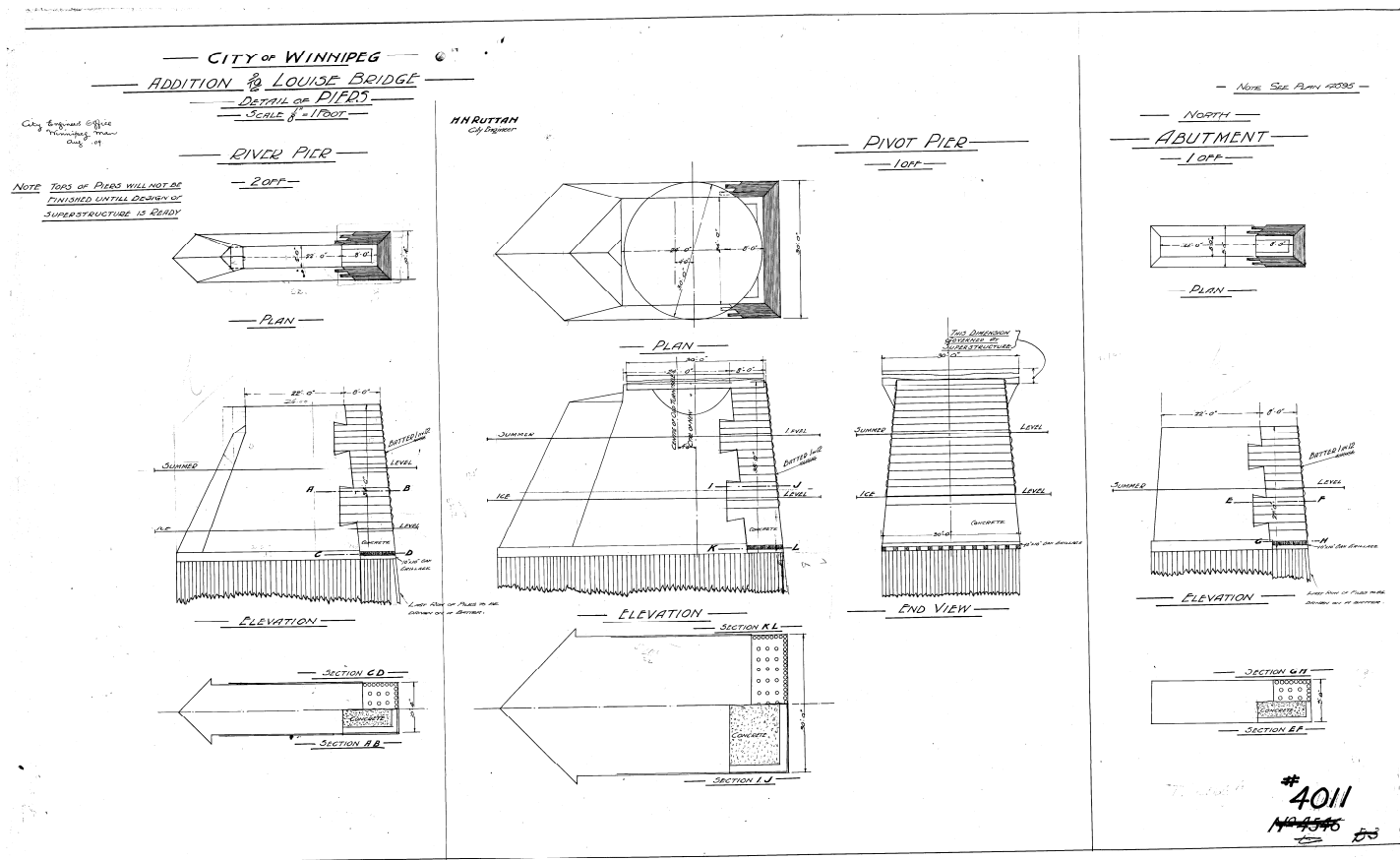
kj

Plan 4003 – general arrangement

25 ft long timber piles



Plan 4011 – addition to piers – 1909?



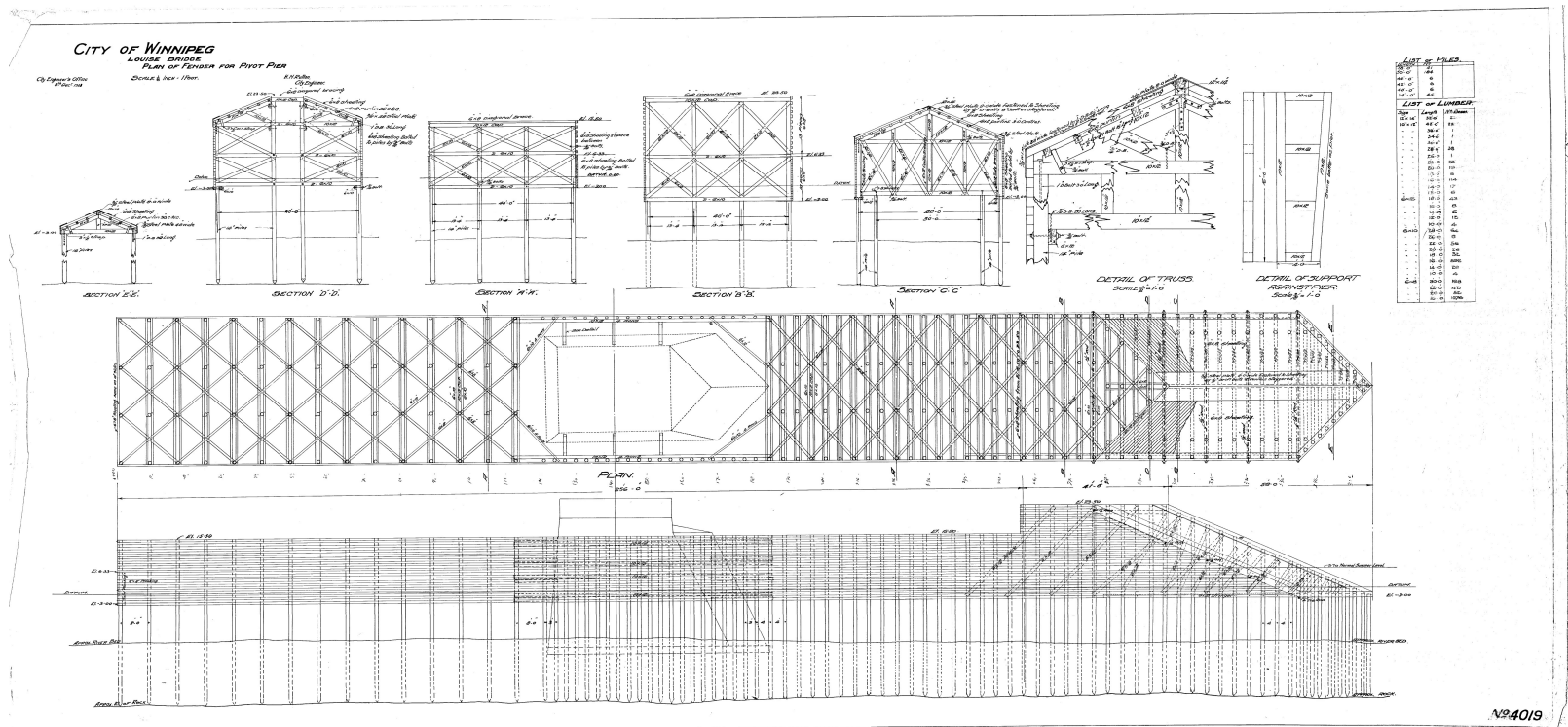
No pile lengths or diameters provided

Plan 4019 – pivot pier – 1913?

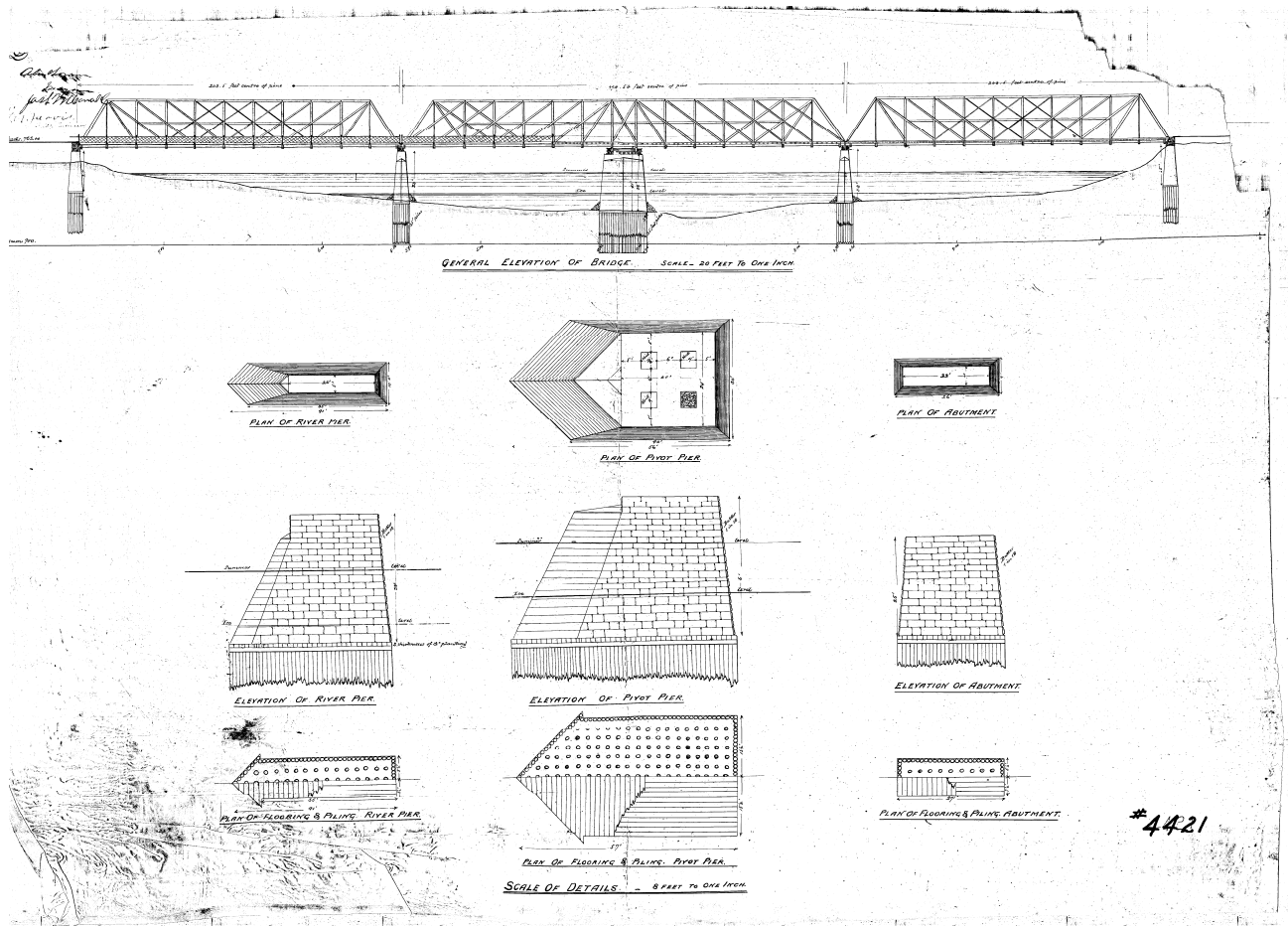
Pile lengths from 34 to 58 ft long

LENGTH	TY'S	
58'-0"	41	
50'-0"	184	
44'-0"	6	
42'-0"	6	
40'-0"	6	
34'-0"	44	

Size	Length	N ^o . Pieces
12x14	32'-0"	2
10x12	42'-0"	22
"	36'-0"	1
"	34'-0"	1
"	30'-0"	1
"	28'-0"	38
"	26'-0"	1
"	22'-0"	23
"	20'-0"	19
"	18'-0"	6
"	16'-0"	114
"	14'-0"	17
"	12'-0"	6
6x12	18'-0"	43
"	16'-0"	8
"	14'-0"	6
"	12'-0"	12
"	10'-0"	4
6x10	28'-0"	64
"	26'-0"	8
"	22'-0"	56
"	20'-0"	26
"	18'-0"	32
"	16'-0"	592
"	14'-0"	28
"	12'-0"	4
6x8	30'-0"	198
"	22'-0"	42
"	20'-0"	52
"	16'-0"	1076



Plan 4421 – gen. elev. & pile layout



No pile lengths or
diameters provided

Appendix B

Laboratory Testing



Quality Engineering | Valued Relationships

MEMORANDUM

Date April 25, 2025
To David Clark, TREK Geotechnical
From Angela Fidler-Kliewer, TREK Geotechnical
Project No. 0002-171-00
Project Louise Bridge Rehabilitation
Subject Laboratory Testing Results – Lab Req. R25-102

Distribution Nuno Mendonca, Michael Van Helden

Attached are the laboratory testing results for the above noted project. The testing included moisture content determinations, Atterberg Limit and grain size distribution (Hydrometer method) and unconfined compressive strength and related testing on shelly tube samples.

Regards,

Angela Fidler-Kliewer, C.Tech.

Attach.

Review Control:

<i>Prepared By:</i> AFK	<i>Reviewed By:</i> AFK	<i>Checked By:</i> NJF
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LABORATORY REQUISITION

CLIENT Tetra Tech Inc.
PROJECT NAME Louise Bridge Rehabilitation

PROJECT NO: 0002-171-00
FIELD TECHNICIAN: David Clark

TEST HOLE NUMBER	SAMPLE NUMBER	DEPTH OF SAMPLE (ft)	Sample Date	MOISTURE	VISUAL CLASS.	ATTEBERG LIMITS	HYDROMETER	GRADATION	STD. PROCTOR	UNCONFINED AND AUXILIARY TESTS	SO ₄ - Water Soluble Sulfate (CSA-A23.2-3B)	Soil Description/Comments
TH25-01	G52	0.0 - 1.0		X								CLAY FILL
TH25-01	G53	3.5 - 4.0		X								↓
TH25-01	G54	5.0 - 5.5										CLAY
TH25-01	G55	7.5 - 8.0										↓
TH25-01	G56	10.0 - 11.0		X		X						↓
TH25-01	T57	15.0 - 17.0								X		
TH25-01	G58	18.0 - 19.0		X								
TH25-01	G59	21.0 - 22.0		X								
TH25-01	T60	25.0 - 27.0								X		
TH25-01	G61	31.0 - 32.0		X		X	X			X		
TH25-01	T62	35.0 - 37.0								X		
TH25-01	G63	40.0 - 41.0		X								↓
TH25-01	G64	42.0 - 42.5		X								SAND
TH25-01	G65	43.5 - 44.0		X								SILT TILL
TH25-01	SS66	45.0 - 46.5		X								↓
TH25-01	SS67	50.0 - 51.5		X								Boulder/Rock
TH25-02	G28	1.0 - 2.0		X								CLAY FILL
TH25-02	G29	4.0 - 4.5		X								SAND FILL
TH25-02	G30	5.0 - 5.5		X								CLAY
TH25-02	G31	8.5 - 9.0		X								SILT
TH25-02	G32	12.0 - 13.0		X								CLAY
TH25-02	G33	17.0 - 18.0		X								↓
TH25-02	T34	20.0 - 22.0								X		
TH25-02	G35	26.5 - 27.0		X		X	X					
TH25-02	T36	30.0 - 32.0								X		
TH25-02	G37	36.0 - 37.0		X						X		
TH25-02	T38	40.0 - 42.0								X		
TH25-02	G39	44.0 - 45.0		X								
TH25-02	G40	47.0 - 48.0		X								
TH25-02	T41	50.0 - 52.0				X	X			X		
TH25-02	T42	60.0 - 62.0								X		
TH25-02	SS43	70.0 - 71.5		X								SILT TILL (rocks sample)
TH25-02	C44	75.0 - 78.0		X								↓
TH25-02	C45	78.0 - 83.0		X								
TH25-02	C46	83.0 - 88.0		X								

REQUESTED BY: David Clark REQUEST TO: DC, NM, MVH
REQUISITION DATE: APR 14/25 DATE REQUIRED: APR 25/25
COMMENTS: _____

REQUISITION NO. R25-102

LABORATORY REQUISITION

CLIENT Tetra Tech Inc.
PROJECT NAME Louise Bridge Rehabilitation

PROJECT NO: 0002-171-00
FIELD TECHNICIAN: David Clark

TEST HOLE NUMBER	SAMPLE NUMBER	DEPTH OF SAMPLE (ft)	Sample Date	MOISTURE	VISUAL CLASS.	ATTERBERG LIMITS	HYDROMETER	GRADATION	STD. PROCTOR	UNCONFINED AND AUXILIARY TESTS	SO ₄ - Water Soluble Sulfate (CSA-A23.2-3B)	Soil Description/Comments
TH25-02	C48	88.0 - 93.0										Bedrock
TH25-02	C49	93.0 - 95.0										↓
TH25-02	C50	95.0 - 100.0										
TH25-02	C51	100.0 - 105.0										↓
TH25-03	G01	2.0 - 3.0		X								CLAY FILL
TH25-03	G02	6.0 - 7.0		X								CLAY (ALLUVIAL)
TH25-03	G03	8.5 - 9.0		X								
TH25-03	T04	10.0 - 12.0		X						X		CLAY / SAND unable to do Qu
TH25-03	G05	13.0 - 13.5		X								SAND
TH25-03	G06	14.5 - 15.0		X								↓
TH25-03	G07	16.0 - 17.0		X								
TH25-03	G08	19.5 - 20.0		X								↓
TH25-03	T09	20.0 - 22.0		X						X		SAND / CLAY unable to do Qu
TH25-03	G10	24.0 - 25.0		X		X						CLAY
TH25-03	G11	27.0 - 28.0		X						⊗		NO RECOVERY
TH25-03	T12	30.0 - 32.0		X								
TH25-03	G13	33.0 - 34.0		X								
TH25-03	G14	37.0 - 38.0		X								
TH25-03	T15	40.0 - 42.0		X						X		UNABLE TO DO Qu
TH25-03	SS16A	45.0 - 46.5		X								SILT TILL (no Recovery)
TH25-03	C16	50.0 - 55.0		X								↓
TH25-03	C17	55.0 - 60.0		X								
TH25-03	SS18	65.0 - 66.5		X								
TH25-03	SS19	70.0 - 71.5		X								
TH25-03	C20	71.5 - 75.0		X								
TH25-03	C21	75.0 - 80.0		X								No recovery
TH25-03	C22	80.0 - 85.0		X								↓
TH25-03	C23	85.0 - 90.0		X								
TH25-03	C24	90.0 - 95.0		X								↓
TH25-03	C25	95.0 - 100.0		X								Bedrock
TH25-03	C26	100.0 - 105.0		X								↓
TH25-03	C27	105.0 - 110.0		X								

C23, C24 - all contains Rocks so no moisture was performed on these samples

REQUESTED BY: David Clark REPORT TO: DC/NM /MVH
REQUISITION DATE: APR 14/25 DATE REQUIRED: APR 25/25
COMMENTS:

REQUISITION NO.
R25-10-2



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

Moisture Content Report ASTM D2216-98

Project No. 0002-171-01
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Sample Date 24-Mar-25
Test Date 14-Apr-25
Technician Z. Dypianco

Test Hole	TH25-01	TH25-01	TH25-01	TH25-01	TH25-01	TH25-01
Depth (m)	0.0 - 0.3	1.1 - 1.2	3.0 - 3.4	5.5 - 5.8	6.4 - 6.7	9.4 - 9.8
Sample #	G52	G53	G56	G58	G59	G61
Tare ID	F58	H30	D227	A100	H1	E60
Mass of tare	8.8	8.7	6.7	9.1	8.3	6.8
Mass wet + tare	210.8	211.4	415.4	210.4	213.4	418.2
Mass dry + tare	183.3	168.6	285.9	154.0	141.7	276.3
Mass water	27.5	42.8	129.5	56.4	71.7	141.9
Mass dry soil	174.5	159.9	279.2	144.9	133.4	269.5
Moisture %	15.8%	26.8%	46.4%	38.9%	53.7%	52.7%

Test Hole	TH25-01	TH25-01	TH25-01	TH25-01	TH25-01	TH25-02
Depth (m)	12.2 - 12.5	12.8 - 13.0	13.3 - 13.4	13.7 - 14.2	15.2 - 15.7	0.3 - 0.6
Sample #	G63	G64	G65	SS66	SS67	G28
Tare ID	M48	B12	E99	N23	P33	Z08
Mass of tare	7.1	6.7	8.5	8.7	8.4	8.5
Mass wet + tare	218.4	213.4	162.7	216.7	156.9	210.7
Mass dry + tare	139.0	179.5	146.9	198.4	151.2	187.7
Mass water	79.4	33.9	15.8	18.3	5.7	23.0
Mass dry soil	131.9	172.8	138.4	189.7	142.8	179.2
Moisture %	60.2%	19.6%	11.4%	9.6%	4.0%	12.8%

Test Hole	TH25-02	TH25-02	TH25-02	TH25-02	TH25-02	TH25-02
Depth (m)	1.2 - 1.4	1.5 - 1.7	2.6 - 2.7	3.7 - 4.0	5.2 - 5.5	8.1 - 8.2
Sample #	G29	G30	G31	G32	G33	G35
Tare ID	Z02	F51	M16	F125	C5	D218
Mass of tare	7.1	8.4	7.0	9.0	8.8	6.8
Mass wet + tare	210.4	215.0	212.6	211.7	212.5	410.6
Mass dry + tare	191.8	174.9	172.2	161.6	144.9	282.1
Mass water	18.6	40.1	40.4	50.1	67.6	128.5
Mass dry soil	184.7	166.5	165.2	152.6	136.1	275.3
Moisture %	10.1%	24.1%	24.5%	32.8%	49.7%	46.7%



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

Moisture Content Report ASTM D2216-98

Project No. 0002-171-01
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Sample Date 24-Mar-25
Test Date 14-Apr-25
Technician Z. Dypianco

Test Hole	TH25-02	TH25-02	TH25-02	TH25-02	TH25-02	TH25-02
Depth (m)	11.0 - 11.3	13.4 - 13.7	14.3 - 14.6	21.3 - 21.8	23.5 - 23.8	24.7 - 24.8
Sample #	G37	G39	G40	SS43	C44	C45
Tare ID	QT78	QT79	B7	D217	D208	W74
Mass of tare	8.2	8.2	6.8	6.8	6.9	8.6
Mass wet + tare	212.2	213.7	217.2	141.5	177.9	220.6
Mass dry + tare	157.3	152.5	147.3	131.0	166.9	206.0
Mass water	54.9	61.2	69.9	10.5	11.0	14.6
Mass dry soil	149.1	144.3	140.5	124.2	160.0	197.4
Moisture %	36.8%	42.4%	49.8%	8.5%	6.9%	7.4%

Test Hole	TH25-02	TH25-03	TH25-03	TH25-03	TH25-03	TH25-03
Depth (m)	26.7 - 26.8	0.6 - 0.9	1.8 - 2.1	2.6 - 2.7	4.0 - 4.1	4.4 - 4.6
Sample #	C46	G01	G02	G03	G05	G06
Tare ID	D210	F109	Z136	E35	W98	E01
Mass of tare	7.0	8.6	8.3	8.5	8.6	7.4
Mass wet + tare	223.4	243.8	255.7	239.3	239.5	336.3
Mass dry + tare	208.2	213.7	208.6	196.4	211.0	277.5
Mass water	15.2	30.1	47.1	42.9	28.5	58.8
Mass dry soil	201.2	205.1	200.3	187.9	202.4	270.1
Moisture %	7.6%	14.7%	23.5%	22.8%	14.1%	21.8%

Test Hole	TH25-03	TH25-03	TH25-03	TH25-03	TH25-03	TH25-03
Depth (m)	4.9 - 5.2	5.9 - 6.1	7.3 - 7.6	8.2 - 8.5	10.1 - 10.4	11.3 - 11.6
Sample #	G07	G08	G10	G11	G13	G14
Tare ID	M85	F69	QT36	D214	D221	W103
Mass of tare	7.0	8.6	8.0	6.8	6.8	8.5
Mass wet + tare	244.9	238.6	432.3	256.4	262.2	240.0
Mass dry + tare	189.6	178.0	320.6	192.4	203.2	183.7
Mass water	55.3	60.6	111.7	64.0	59.0	56.3
Mass dry soil	182.6	169.4	312.6	185.6	196.4	175.2
Moisture %	30.3%	35.8%	35.7%	34.5%	30.0%	32.1%



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Moisture Content Report ASTM D2216-98

Project No. 0002-171-01
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Sample Date 24-Mar-25
Test Date 14-Apr-25
Technician Z. Dypianco

Test Hole	TH25-03	TH25-03	TH25-03	TH25-03	TH25-03	TH25-03
Depth (m)	15.2 - 16.8	16.8 - 18.3	19.8 - 20.3	21.3 - 21.8	21.8 - 22.9	24.4 - 25.9
Sample #	C16	C17	SS18	SS19	C20	C22
Tare ID	B18	B114	QT69	D204	QT31	K11
Mass of tare	6.8	8.7	8.1	6.8	8.2	8.8
Mass wet + tare	89.3	782.9	207.5	234.5	151.8	408.2
Mass dry + tare	84.4	738.9	194.1	217.2	140.9	395.6
Mass water	4.9	44.0	13.4	17.3	10.9	12.6
Mass dry soil	77.6	730.2	186.0	210.4	132.7	386.8
Moisture %	6.3%	6.0%	7.2%	8.2%	8.2%	3.3%

Project No. 0002-171-01
Client Tetra Tech
Project Louise Bridge Rehabilitation

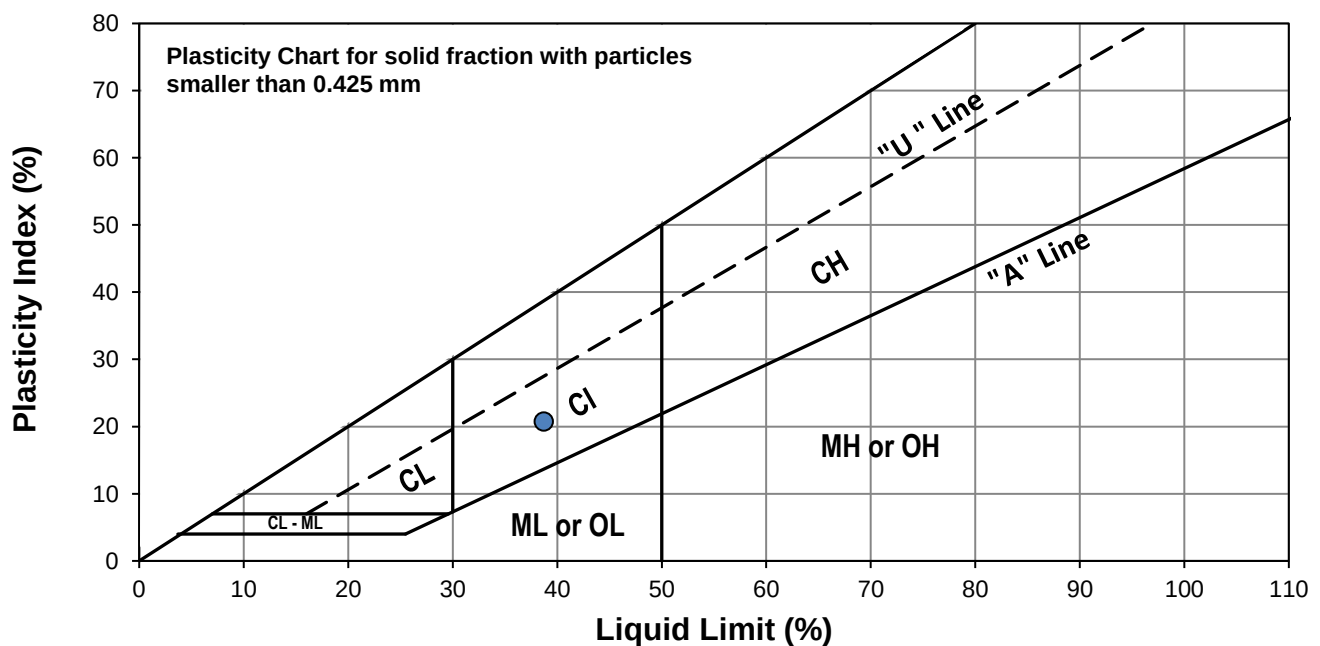
Test Hole TH25-03
Sample # G10
Depth (m) 7.6 - 7.6
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician M. Thomsen



Liquid Limit 39
Plastic Limit 18
Plasticity Index 21

Liquid Limit

Trial #	1	2	3		
Number of Blows (N)	19	26	30		
Mass Tare (g)	14.062	14.151	13.784		
Mass Wet Soil + Tare (g)	28.886	27.019	26.151		
Mass Dry Soil + Tare (g)	24.650	23.440	22.754		
Mass Water (g)	4.236	3.579	3.397		
Mass Dry Soil (g)	10.588	9.289	8.970		
Moisture Content (%)	40.008	38.529	37.871		



Plastic Limit

Trial #	1	2	3	4	5
Mass Tare (g)	14.153	14.148			
Mass Wet Soil + Tare (g)	24.884	23.868			
Mass Dry Soil + Tare (g)	23.273	22.365			
Mass Water (g)	1.611	1.503			
Mass Dry Soil (g)	9.120	8.217			
Moisture Content (%)	17.664	18.291			

Note: Additional information recorded/measured for this test is available upon request.

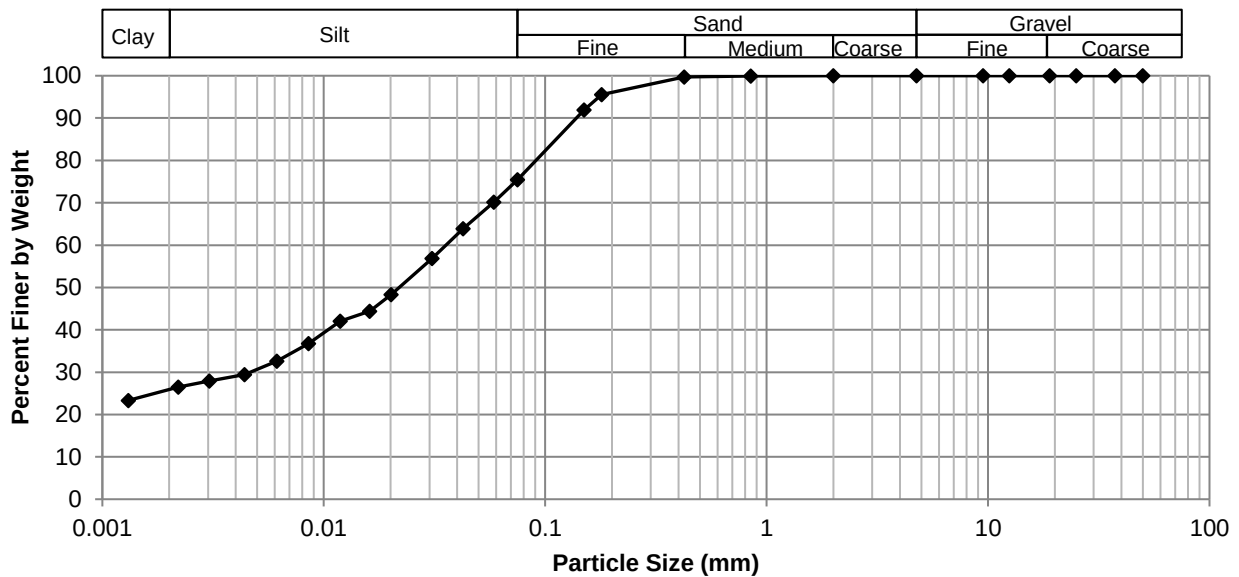
Project No. 0002-171-01
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation



Test Hole TH25-03
Sample # G10
Depth (m) 7.3 - 7.6
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Gravel	0.0%
Sand	24.5%
Silt	49.7%
Clay	25.8%

Particle Size Distribution Curve



Gravel		Sand		Silt and Clay	
Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing
50.0	100.00	4.75	100.00	0.0750	75.47
37.5	100.00	2.00	100.00	0.0587	70.18
25.0	100.00	0.850	99.93	0.0426	63.93
19.0	100.00	0.425	99.70	0.0309	56.89
12.5	100.00	0.180	95.58	0.0202	48.29
9.50	100.00	0.150	91.92	0.0161	44.39
4.75	100.00	0.075	75.47	0.0119	42.04
				0.0086	36.73
				0.0061	32.57
				0.0044	29.45
				0.0030	27.93
				0.0022	26.50
				0.0013	23.34

Project No. 0002-171-01
Client Tetra Tech
Project Louise Bridge Rehabilitation

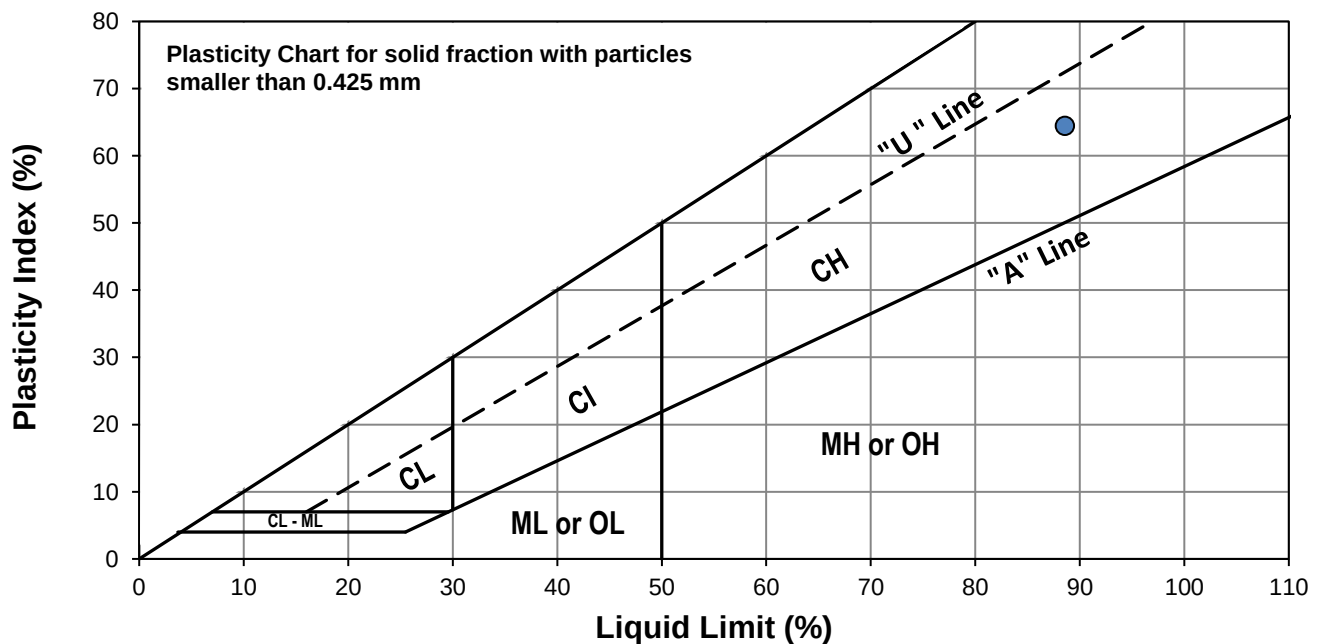
Test Hole TH25-02
Sample # G35
Depth (m) 8.1 - 8.2
Sample Date 24-Mar-25
Test Date 16-Apr-25
Technician A. Bhullar



Liquid Limit 89
Plastic Limit 24
Plasticity Index 64

Liquid Limit

Trial #	1	2	3		
Number of Blows (N)	16	23	35		
Mass Tare (g)	13.845	13.964	14.108		
Mass Wet Soil + Tare (g)	27.302	28.221	30.387		
Mass Dry Soil + Tare (g)	20.807	21.467	22.927		
Mass Water (g)	6.495	6.754	7.460		
Mass Dry Soil (g)	6.962	7.503	8.819		
Moisture Content (%)	93.292	90.017	84.590		



Plastic Limit

Trial #	1	2	3	4	5
Mass Tare (g)	14.061	13.741			
Mass Wet Soil + Tare (g)	20.803	20.004			
Mass Dry Soil + Tare (g)	19.480	18.799			
Mass Water (g)	1.323	1.205			
Mass Dry Soil (g)	5.419	5.058			
Moisture Content (%)	24.414	23.824			

Note: Additional information recorded/measured for this test is available upon request.

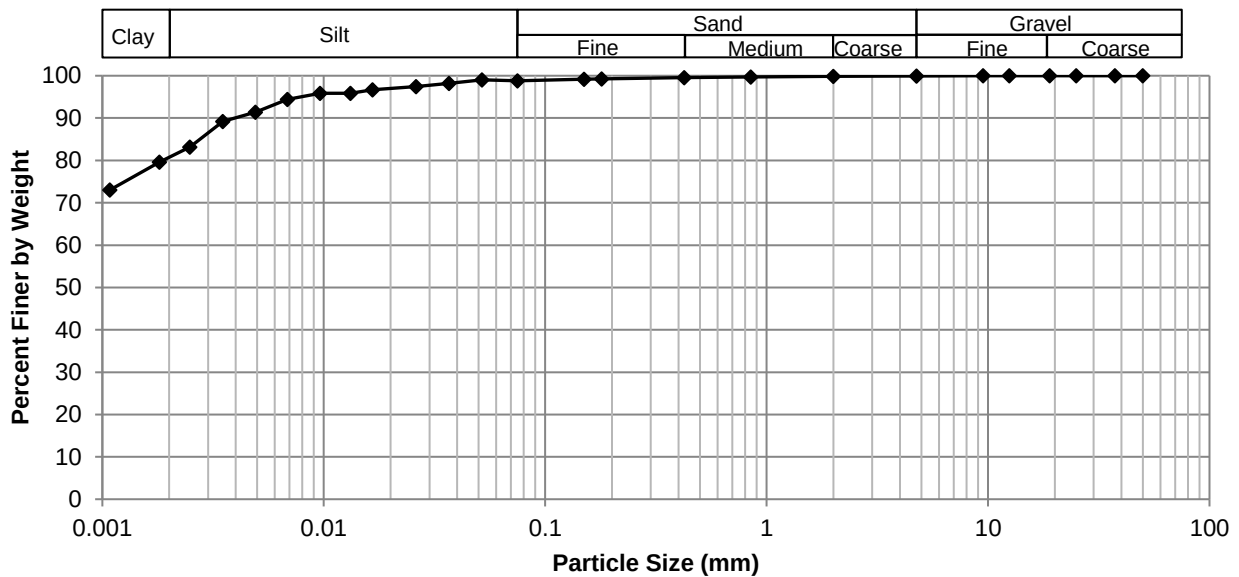
Project No. 0002-171-01
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation



Test Hole TH25-02
Sample # G35
Depth (m) 8.1 - 8.2
Sample Date 24-Mar-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Gravel	0.1%
Sand	1.1%
Silt	18.2%
Clay	80.6%

Particle Size Distribution Curve



Gravel		Sand		Silt and Clay	
Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing
50.0	100.00	4.75	99.92	0.0750	98.81
37.5	100.00	2.00	99.86	0.0519	99.01
25.0	100.00	0.850	99.67	0.0368	98.23
19.0	100.00	0.425	99.52	0.0262	97.45
12.5	100.00	0.180	99.24	0.0166	96.67
9.50	100.00	0.150	99.15	0.0132	95.89
4.75	99.92	0.075	98.81	0.0096	95.89
				0.0069	94.39
				0.0049	91.39
				0.0035	89.17
				0.0025	83.17
				0.0018	79.58
				0.0011	73.03

Project No. 0002-171-01
Client Tetra Tech
Project Louise Bridge Rehabilitation

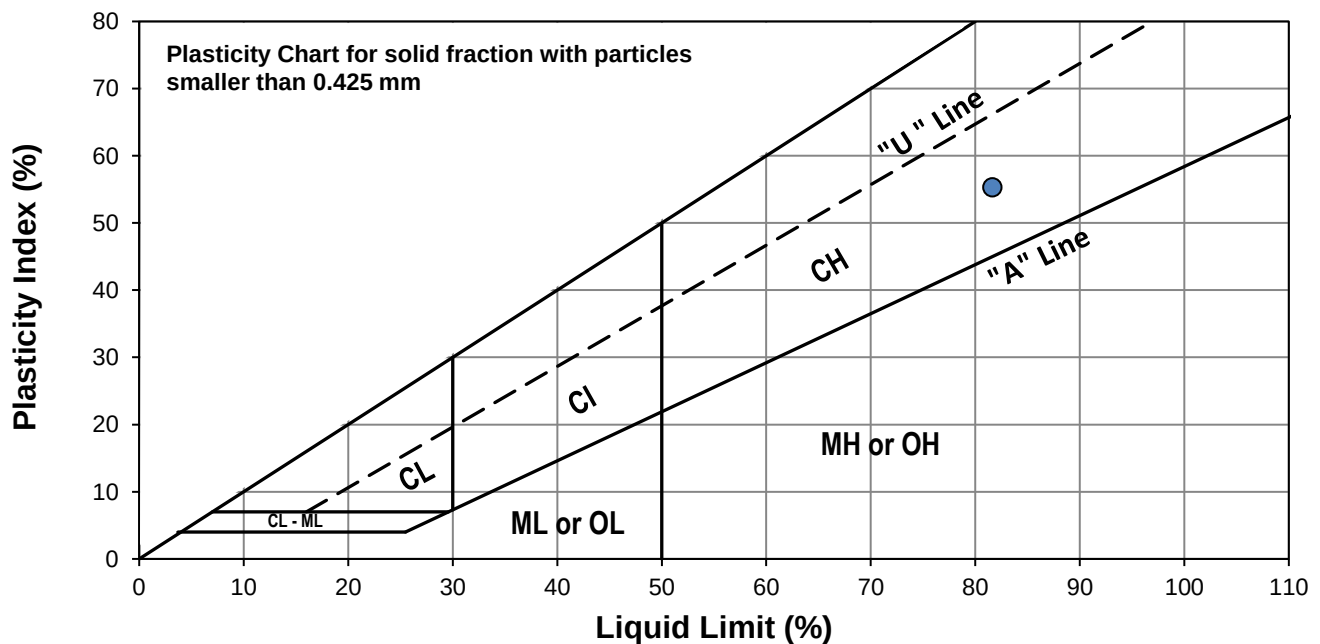
Test Hole TH25-01
Sample # G56
Depth (m) 3.0 - 3.4
Sample Date 24-Mar-25
Test Date 16-Apr-25
Technician I.Araquil



Liquid Limit 82
Plastic Limit 26
Plasticity Index 55

Liquid Limit

Trial #	1	2	3		
Number of Blows (N)	18	23	32		
Mass Tare (g)	14.009	14.043	14.006		
Mass Wet Soil + Tare (g)	23.031	25.216	23.821		
Mass Dry Soil + Tare (g)	18.892	20.164	19.483		
Mass Water (g)	4.139	5.052	4.338		
Mass Dry Soil (g)	4.883	6.121	5.477		
Moisture Content (%)	84.763	82.536	79.204		



Plastic Limit

Trial #	1	2	3	4	5
Mass Tare (g)	14.099	14.062			
Mass Wet Soil + Tare (g)	21.307	22.148			
Mass Dry Soil + Tare (g)	19.811	20.456			
Mass Water (g)	1.496	1.692			
Mass Dry Soil (g)	5.712	6.394			
Moisture Content (%)	26.190	26.462			

Note: Additional information recorded/measured for this test is available upon request.

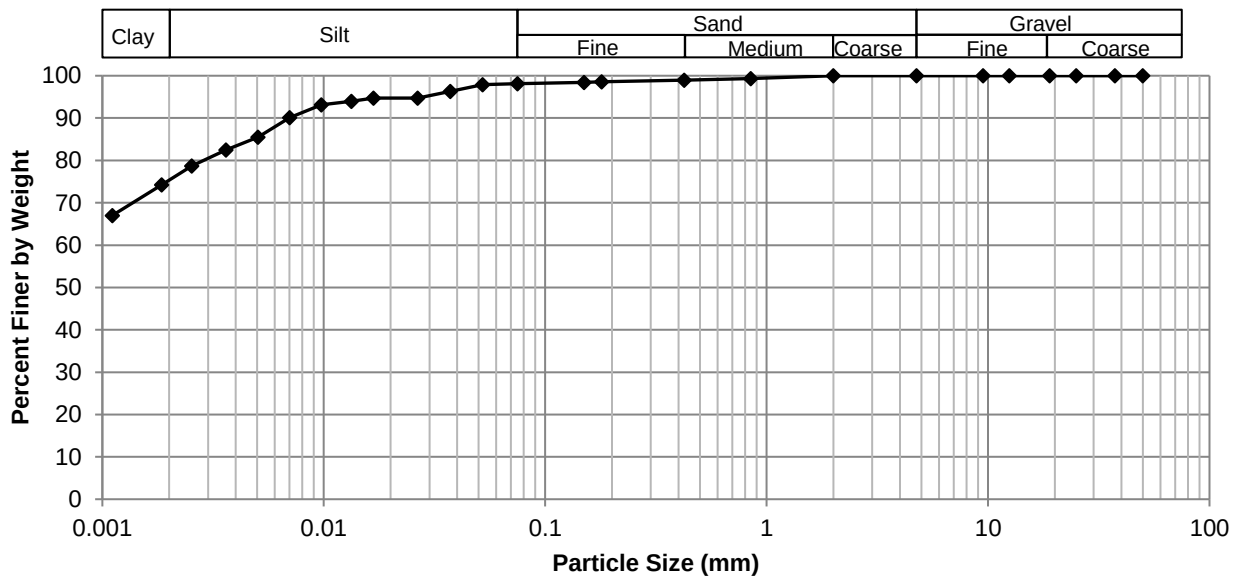
Project No. 0002-171-01
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation



Test Hole TH25-01
Sample # G56
Depth (m) 3.0 - 3.4
Sample Date 24-Mar-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Gravel	0.0%
Sand	1.9%
Silt	22.9%
Clay	75.2%

Particle Size Distribution Curve



Gravel		Sand		Silt and Clay	
Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing
50.0	100.00	4.75	100.00	0.0750	98.12
37.5	100.00	2.00	100.00	0.0522	97.86
25.0	100.00	0.850	99.34	0.0372	96.30
19.0	100.00	0.425	98.99	0.0266	94.73
12.5	100.00	0.180	98.54	0.0168	94.73
9.50	100.00	0.150	98.45	0.0133	93.95
4.75	100.00	0.075	98.12	0.0098	93.17
				0.0070	90.09
				0.0051	85.49
				0.0036	82.46
				0.0025	78.73
				0.0019	74.27
				0.0011	67.01

Project No. 0002-171-01
Client Tetra Tech
Project Louise Bridge Rehabilitation

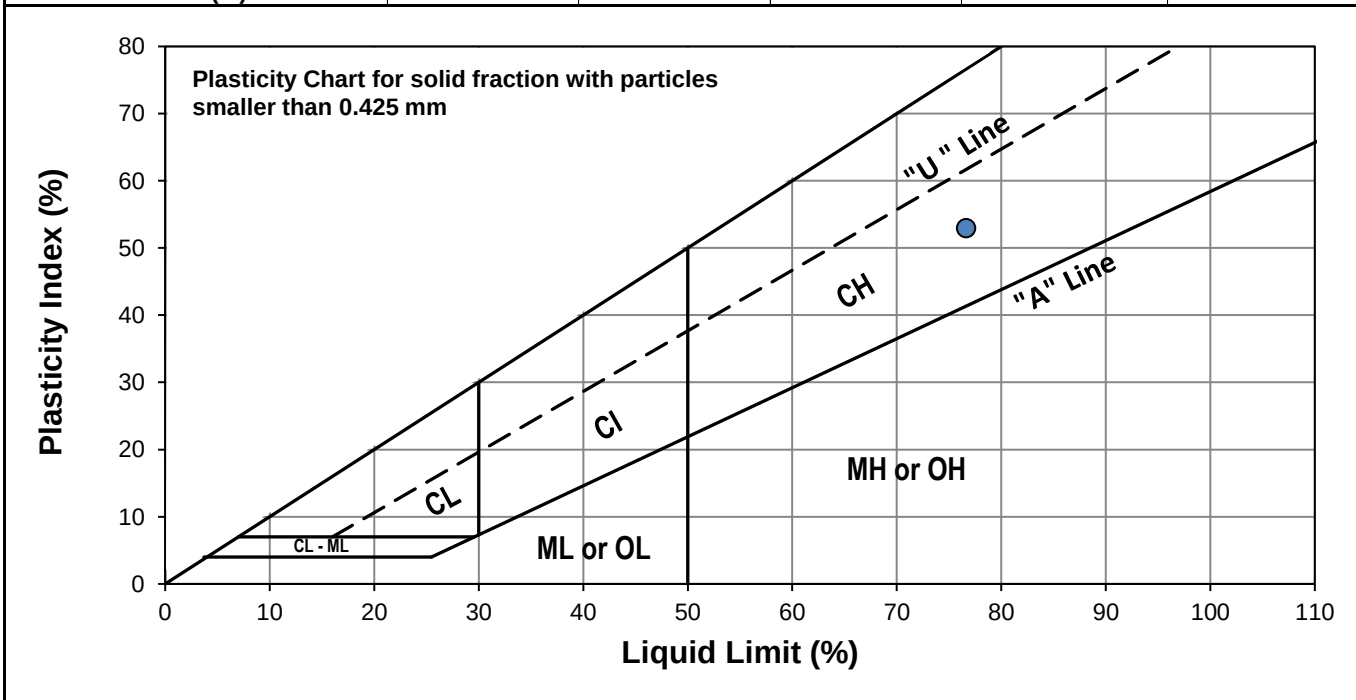
Test Hole TH25-01
Sample # G61
Depth (m) 9.4 - 9.8
Sample Date 24-Mar-25
Test Date 16-Apr-25
Technician I. Araquil



Liquid Limit 77
Plastic Limit 24
Plasticity Index 53

Liquid Limit

Trial #	1	2	3		
Number of Blows (N)	18	23	32		
Mass Tare (g)	14.042	14.037	14.029		
Mass Wet Soil + Tare (g)	24.055	23.780	22.334		
Mass Dry Soil + Tare (g)	19.672	19.536	18.761		
Mass Water (g)	4.383	4.244	3.573		
Mass Dry Soil (g)	5.630	5.499	4.732		
Moisture Content (%)	77.851	77.178	75.507		



Plastic Limit

Trial #	1	2	3	4	5
Mass Tare (g)	13.965	14.042			
Mass Wet Soil + Tare (g)	20.945	22.457			
Mass Dry Soil + Tare (g)	19.607	20.847			
Mass Water (g)	1.338	1.610			
Mass Dry Soil (g)	5.642	6.805			
Moisture Content (%)	23.715	23.659			

Note: Additional information recorded/measured for this test is available upon request.

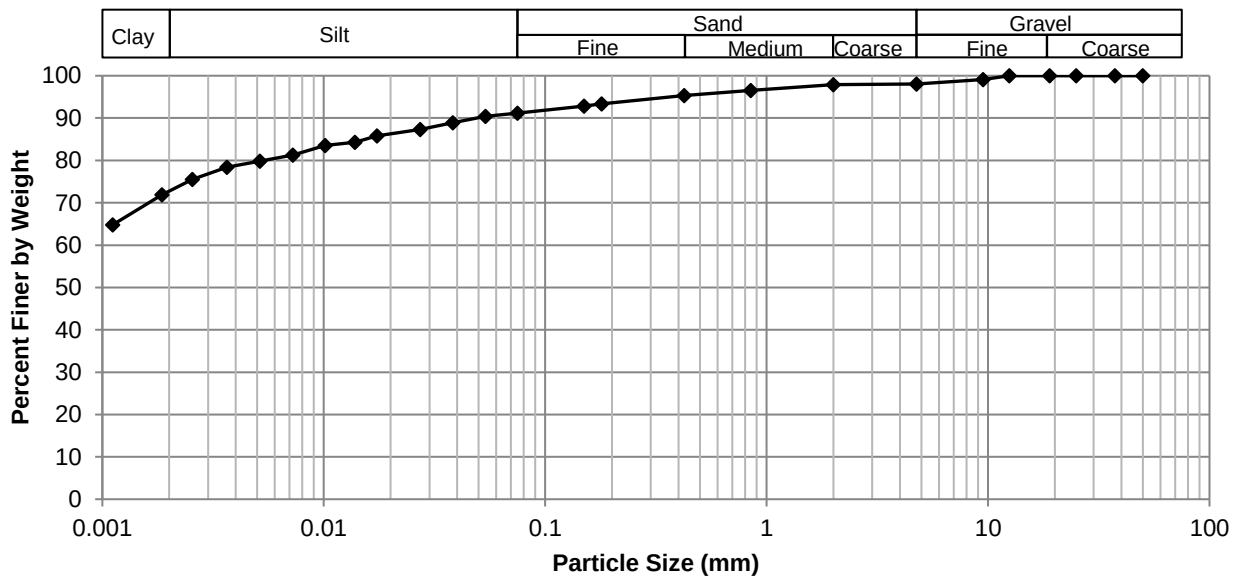
Project No. 0002-171-01
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation



Test Hole TH25-01
Sample # G61
Depth (m) 9.4 - 9.8
Sample Date 24-Mar-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Gravel	1.9%
Sand	6.9%
Silt	18.5%
Clay	72.6%

Particle Size Distribution Curve



Gravel		Sand		Silt and Clay	
Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing
50.0	100.00	4.75	98.07	0.0750	91.18
37.5	100.00	2.00	97.93	0.0538	90.40
25.0	100.00	0.850	96.50	0.0383	88.87
19.0	100.00	0.425	95.30	0.0273	87.34
12.5	100.00	0.180	93.34	0.0174	85.81
9.50	99.11	0.150	92.85	0.0139	84.28
4.75	98.07	0.075	91.18	0.0102	83.51
				0.0072	81.26
				0.0051	79.82
				0.0037	78.38
				0.0026	75.50
				0.0019	71.91
				0.0011	64.79

Project No. 0002-171-01
Client Tetra Tech
Project Louise Bridge Rehabilitation

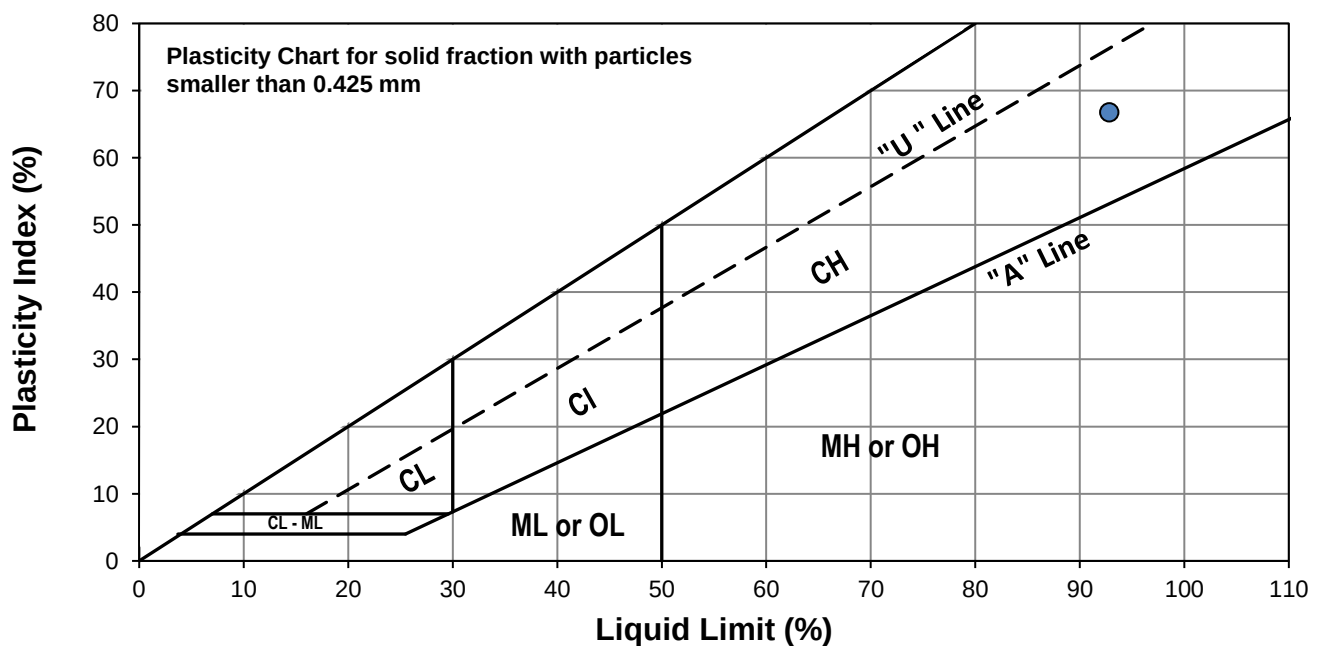
Test Hole TH25-02
Sample # T41
Depth (m) 15.2 - 15.8
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician M. Thomsen



Liquid Limit 93
Plastic Limit 26
Plasticity Index 67

Liquid Limit

Trial #	1	2	3		
Number of Blows (N)	15	27	33		
Mass Tare (g)	14.082	13.906	14.084		
Mass Wet Soil + Tare (g)	24.229	24.552	24.475		
Mass Dry Soil + Tare (g)	19.237	19.446	19.534		
Mass Water (g)	4.992	5.106	4.941		
Mass Dry Soil (g)	5.155	5.540	5.450		
Moisture Content (%)	96.838	92.166	90.661		



Plastic Limit

Trial #	1	2	3	4	5
Mass Tare (g)	14.123	14.086			
Mass Wet Soil + Tare (g)	22.990	23.272			
Mass Dry Soil + Tare (g)	21.150	21.380			
Mass Water (g)	1.840	1.892			
Mass Dry Soil (g)	7.027	7.294			
Moisture Content (%)	26.185	25.939			

Note: Additional information recorded/measured for this test is available upon request.

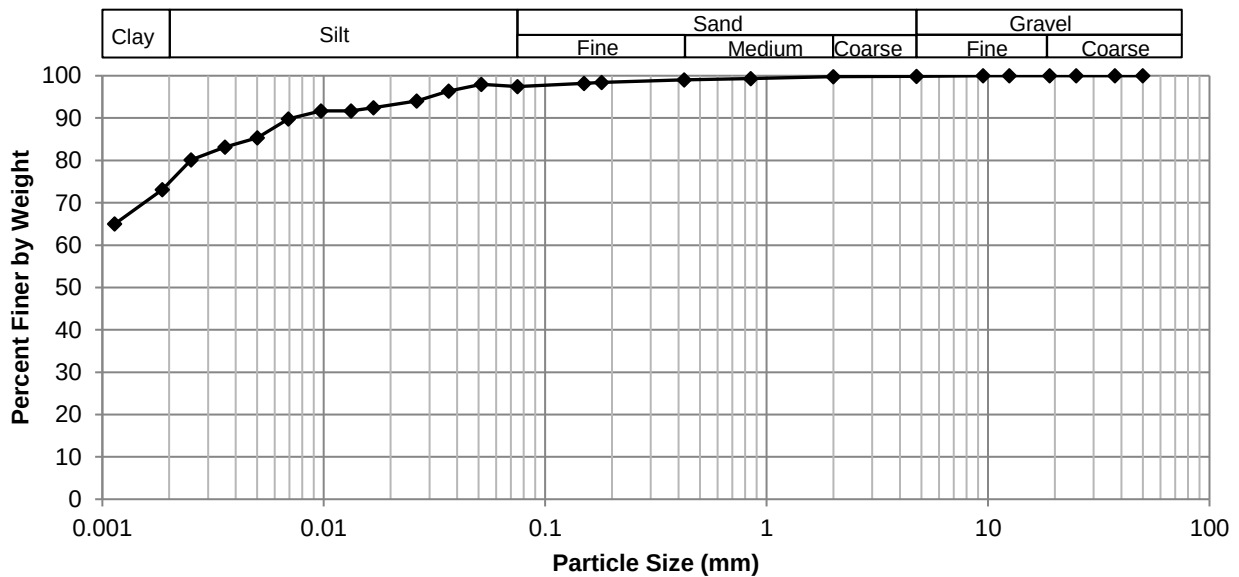
Project No. 0002-171-01
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation



Test Hole TH25-02
Sample # T41
Depth (m) 15.2 - 15.8
Sample Date 01-Apr-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Gravel	0.1%
Sand	2.4%
Silt	22.9%
Clay	74.6%

Particle Size Distribution Curve



Gravel		Sand		Silt and Clay	
Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing	Particle Size (mm)	Percent Passing
50.0	100.00	4.75	99.88	0.0750	97.45
37.5	100.00	2.00	99.75	0.0515	97.95
25.0	100.00	0.850	99.37	0.0367	96.39
19.0	100.00	0.425	99.03	0.0263	94.05
12.5	100.00	0.180	98.41	0.0168	92.49
9.50	100.00	0.150	98.21	0.0133	91.71
4.75	99.88	0.075	97.45	0.0097	91.71
				0.0069	89.84
				0.0050	85.35
				0.0036	83.17
				0.0025	80.11
				0.0019	73.12
				0.0011	65.00



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-03
Sample # T04
Depth (m) 3.0 - 3.7
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Tube Extraction

Recovery (mm)	620	
Bottom	A	B
3.31 m	3.23 m	3.15 m
		Top
		3.05 m
Moisture Content Visual	Moisture Content Visual PP/TV	Bulk
80 mm (A)	80 mm (B)	100 mm (B)

Visual Classification	A	B
Material	SAND	CLAY and SAND
Composition	trace silt	silty
		trace gravel (<10mm diam.)
Color	beige	brown
Moisture	dry to moist	moist
Consistency	loose	stiff
Plasticity	non plastic	low to intermediate plasticity
Structure		
Gradation	well graded	
Torvane	A	B
Reading	-	0.55
Vane Size (s,m,l)	m	m
Undrained Shear Strength	-	53.9 (kPa)
Pocket Penetrometer	A	B
Reading	1	1.20
	2	1.00
	3	1.20
	Average	1.13
Undrained Shear Strength	-	55.6 (kPa)

Moisture Content	A	B
Tare ID	W94	EX01
Mass tare (g)	8.8	6.8
Mass wet + tare (g)	238.2	356.2
Mass dry + tare (g)	228.2	289.2
Moisture %	4.6%	23.7%
Unit Weight		
Bulk Weight (g)	-	638.60
Length (mm)	1	81.26
	2	82.15
	3	82.38
	4	81.87
Average Length (m)	-	0.082
Diam. (mm)	1	68.55
	2	71.32
	3	72.98
	4	72.41
Average Diameter (m)	-	0.071
Volume (m³)	-	3.27E-04
Bulk Unit Weight (kN/m³)	-	19.1
Bulk Unit Weight (pcf)	-	121.8
Dry Unit Weight (kN/m³)	-	15.5
Dry Unit Weight (pcf)	-	98.5



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-03
Sample # T09
Depth (m) 6.1 - 6.7
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Tube Extraction

Recovery (mm)	540		
Bottom	A	B	Top
6.64 m	6.58 m	6.50 m	6.10 m
Moisture Content PP/TV Visual	Moisture Content Visual	Bulk/Keep	
60 mm (A)	80 mm (B)	400 mm (B)	

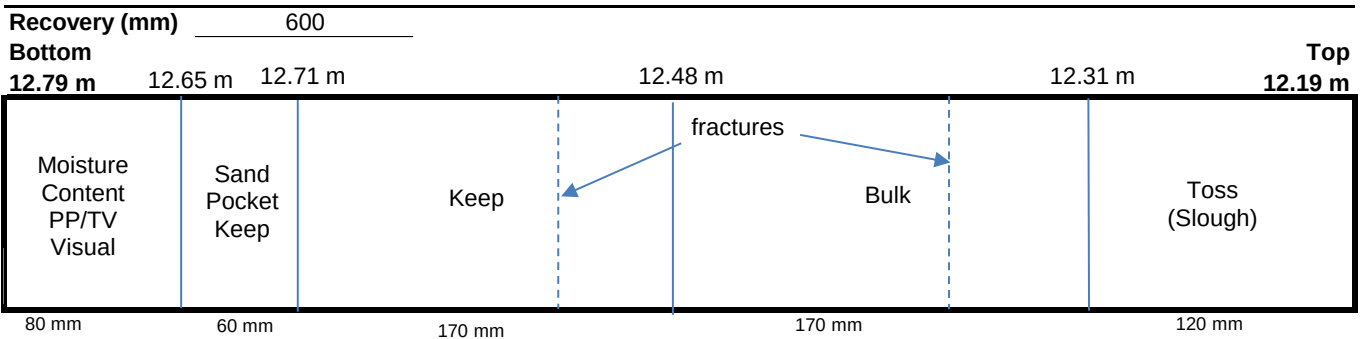
Visual Classification	A	B
Material	CLAY	SAND
Composition	silty	trace clay
	trace fine sand	trace silt
	trace oxidation	trace oxidation
Color	grey	beige
Moisture	moist	moist
Consistency	firm to stiff	compact
Plasticity	intermediate plasticity	no to low plasticity
Structure		
Gradation		
Torvane	A	B
Reading	0.5	-
Vane Size (s,m,l)	m	m
Undrained Shear Strength	49.0	-
		(kPa)
Pocket Penetrometer	A	B
Reading	1	2
	1.00	-
	1.20	-
	1.20	-
Average	1.13	-
Undrained Shear Strength	55.6	-
		(kPa)

Moisture Content	A	B
Tare ID	E04	AA7
Mass tare (g)	6.8	6.6
Mass wet + tare (g)	292.4	538.0
Mass dry + tare (g)	226.4	441.0
Moisture %	30.1%	22.3%
Unit Weight		
Bulk Weight (g)	-	1127.20
Length (mm)	1	2
	-	143.11
	-	143.11
	3	4
	-	143.21
	-	143.00
Average Length (m)	-	0.143
Diam. (mm)	1	2
	-	71.40
	-	72.20
	-	72.40
	-	70.79
Average Diameter (m)	-	0.072
Volume (m³)	-	5.78E-04
Bulk Unit Weight (kN/m³)	-	19.1
Bulk Unit Weight (pcf)	-	121.8
Dry Unit Weight (kN/m³)	-	15.6
Dry Unit Weight (pcf)	-	99.6

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-03
Sample # T15
Depth (m) 12.2 - 12.8
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Tube Extraction



Visual Classification

Material	CLAY
Composition	silty
	trace to some fine sand
	trace organics
Color	dark grey
Moisture	moist
Consistency	firm to stiff
Plasticity	intermediate to high plasticity
Structure	friable
Gradation	-

Torvane

Reading	0.50
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	49.0

Pocket Penetrometer

Reading	1	1.00
	2	1.20
	3	1.20
	Average	1.13
Undrained Shear Strength (kPa)		55.6

Moisture Content

Tare ID	QT76
Mass tare (g)	8.2
Mass wet + tare (g)	340.2
Mass dry + tare (g)	256.8
Moisture %	33.5%

Unit Weight

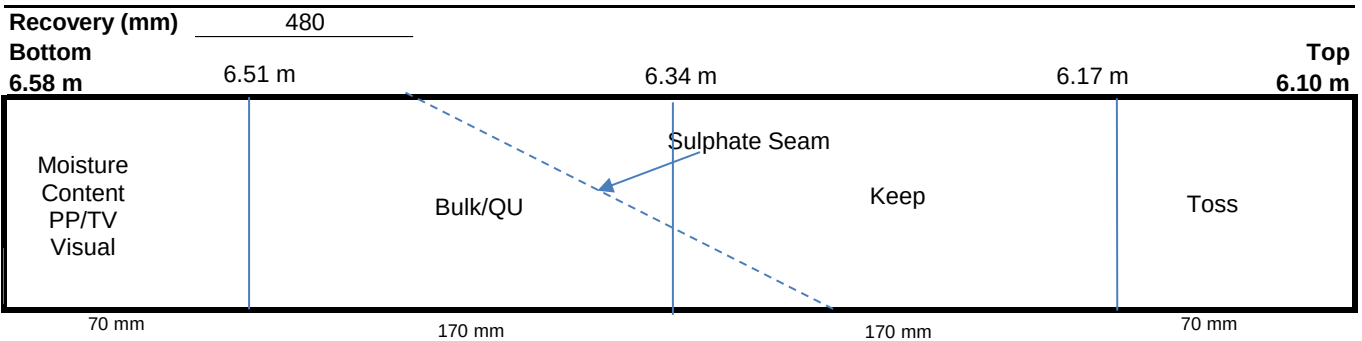
Bulk Weight (g)		966.0
Length (mm)	1	122.92
	2	122.68
	3	122.55
	4	122.79
Average Length (m)		0.123
Diam. (mm)	1	72.58
	2	72.46
	3	72.29
	4	72.44
Average Diameter (m)		0.072

Volume (m³)	5.06E-04
Bulk Unit Weight (kN/m³)	18.7
Bulk Unit Weight (pcf)	119.2
Dry Unit Weight (kN/m³)	14.0
Dry Unit Weight (pcf)	89.3

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T34
Depth (m) 6.1 - 6.7
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Tube Extraction



Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<3mm diam.)	
trace precipitates (sulphate seam, <5mm thick)	
trace rootlets	
Color	mottled grand and brown
Moisture	moist
Consistency	stiff
Plasticity	high plasticity
Structure	laminated (brown and grey clay, <1mm thick)
Gradation	-

Torvane

Reading	0.85
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	83.4

Pocket Penetrometer

Reading	1	1.70
	2	1.80
	3	1.70
	Average	1.73
Undrained Shear Strength (kPa)		85.0

Moisture Content

Tare ID	P14
Mass tare (g)	8.6
Mass wet + tare (g)	333.4
Mass dry + tare (g)	228.2
Moisture %	47.9%

Unit Weight

Bulk Weight (g)		1061.0
Length (mm)	1	149.07
	2	149.02
	3	149.31
	4	149.18
Average Length (m)		0.149
Diam. (mm)	1	72.57
	2	72.45
	3	72.30
	4	72.06
Average Diameter (m)		0.072

Volume (m³)	6.13E-04
Bulk Unit Weight (kN/m³)	17.0
Bulk Unit Weight (pcf)	108.0
Dry Unit Weight (kN/m³)	11.5
Dry Unit Weight (pcf)	73.0

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T34
Depth (m) 6.1 - 6.7
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Unconfined Strength

	kPa	ksf
Max q_u	101.3	2.1
Max S_u	50.7	1.1

Specimen Data

Description CLAY - silty, trace silt inclusions (<3mm diam.), trace precipitates (sulphate seam, <5mm thick), trace rootlets, mottled grand and brown, moist, stiff, high plasticity, laminated (brown and grey clay, <1mm thick)

Length	149.1	(mm)	Moisture %	48%
Diameter	72.3	(mm)	Bulk Unit Wt.	17.0 (kN/m ³)
L/D Ratio	2.1		Dry Unit Wt.	11.5 (kN/m ³)
Initial Area	0.00411	(m ²)	Liquid Limit	-
Load Rate	1.00	(%/min)	Plastic Limit	-
			Plasticity Index	-

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.85	83.4	1.74
Vane Size		
m		

Pocket Penetrometer

Reading	Undrained Shear Strength	
tsf	kPa	ksf
1.70	83.4	1.74
1.80	88.3	1.84
1.70	83.4	1.74
Average	1.73	85.0
		1.8

Failure Geometry

Sketch:

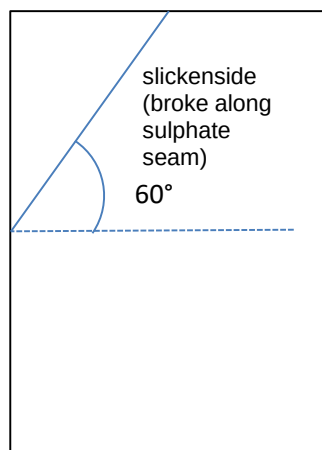
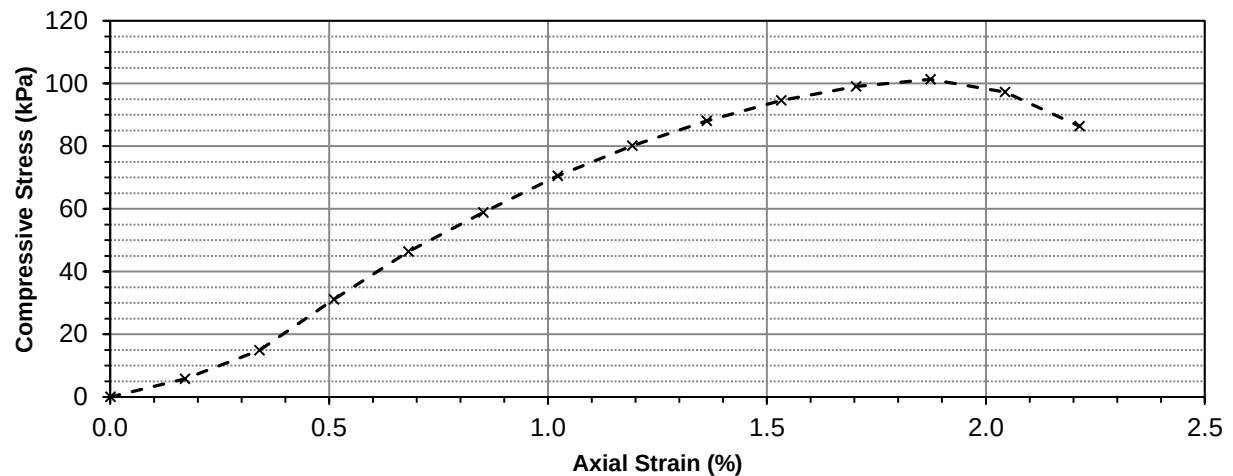


Photo:



Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	1.02	0.0000	0.00	0.004111	0.0	0.00	0.00
10	1.49	0.2540	0.17	0.004118	23.7	5.75	2.88
20	2.24	0.5080	0.34	0.004125	61.5	14.91	7.45
30	3.57	0.7620	0.51	0.004132	128.5	31.11	15.55
40	4.83	1.0160	0.68	0.004139	192.0	46.40	23.20
50	5.86	1.2700	0.85	0.004146	244.0	58.84	29.42
60	6.83	1.5240	1.02	0.004153	292.8	70.51	35.26
70	7.63	1.7780	1.19	0.004160	333.2	80.08	40.04
80	8.30	2.0320	1.36	0.004167	366.9	88.05	44.02
90	8.85	2.2860	1.53	0.004175	394.7	94.54	47.27
100	9.24	2.5400	1.70	0.004182	414.3	99.07	49.54
110	9.44	2.7940	1.87	0.004189	424.4	101.31	50.65
120	9.12	3.0480	2.04	0.004196	408.3	97.29	48.64
130	8.22	3.3020	2.21	0.004204	362.9	86.33	43.16



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T36
Depth (m) 9.1 - 9.8
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Tube Extraction

Recovery (mm)	510			
Bottom				Top
9.65 m	9.49 m	9.42 m	9.24 m	9.14 m
Keep	Moisture Content PP/TV Visual	Bulk/QU	Toss	
160 mm	70 mm	180 mm	100 mm	

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<10mm diam.)	
trace oxidation	
Color	grey
Moisture	moist
Consistency	very stiff
Plasticity	high plasticity
Structure	friable
Gradation	-

Torvane

Reading	0.50
Vane Size (s,m,l)	s
Undrained Shear Strength (kPa)	122.6

Pocket Penetrometer

Reading	1	2.50
	2	2.30
	3	2.20
	Average	2.33
Undrained Shear Strength (kPa)		114.4

Moisture Content

Tare ID	D203
Mass tare (g)	6.8
Mass wet + tare (g)	342.4
Mass dry + tare (g)	248.7
Moisture %	38.7%

Unit Weight

Bulk Weight (g)		1144.4
Length (mm)	1	151.94
	2	152.02
	3	152.16
	4	152.13
Average Length (m)		0.152
Diam. (mm)	1	72.30
	2	72.39
	3	72.28
	4	72.38
Average Diameter (m)		0.072

Volume (m³)	6.25E-04
Bulk Unit Weight (kN/m³)	18.0
Bulk Unit Weight (pcf)	114.3
Dry Unit Weight (kN/m³)	12.9
Dry Unit Weight (pcf)	82.4

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T36
Depth (m) 9.1 - 9.8
Sample Date 24-Mar-25
Test Date 17-Apr-25
Technician A. Dustmamatov

Unconfined Strength

	kPa	ksf
Max q_u	141.2	2.9
Max S_u	70.6	1.5

Specimen Data

Description CLAY - silty, trace silt inclusions (<10mm diam.), trace oxidation, grey, moist, very stiff, high plasticity, friable

Length	152.1	(mm)	Moisture %	39%
Diameter	72.3	(mm)	Bulk Unit Wt.	18.0 (kN/m ³)
L/D Ratio	2.1		Dry Unit Wt.	12.9 (kN/m ³)
Initial Area	0.00411	(m ²)	Liquid Limit	-
Load Rate	1.00	(%/min)	Plastic Limit	-
			Plasticity Index	-

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.50	122.6	2.56
Vane Size		
s		

Pocket Penetrometer

Reading	Undrained Shear Strength	
tsf	kPa	ksf
2.50	122.6	2.56
2.30	112.8	2.36
2.20	107.9	2.25
Average	2.33	114.5
		2.4

Failure Geometry

Sketch:

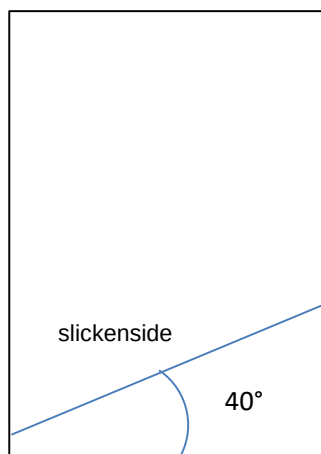
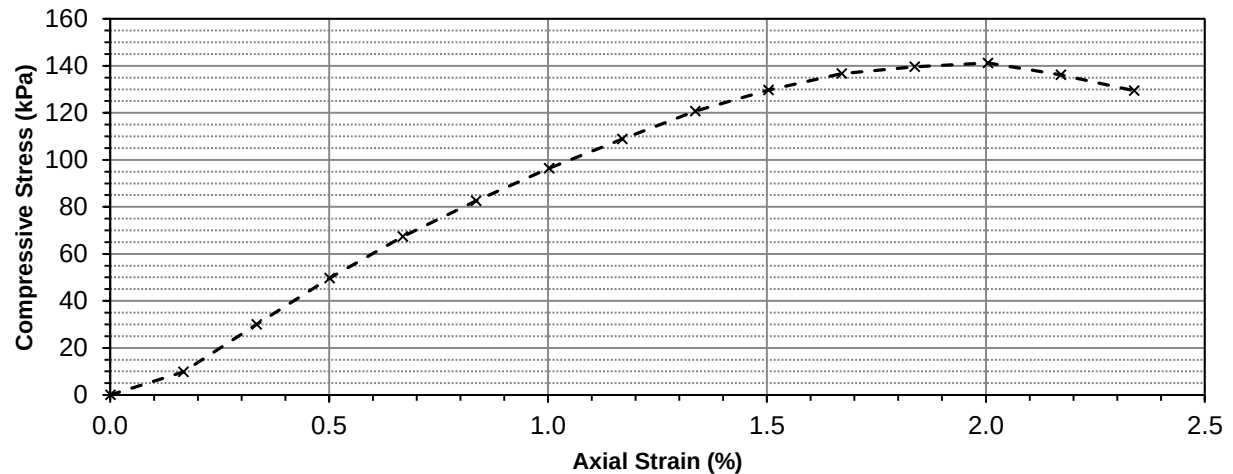


Photo:



Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	1.13	0.0000	0.00	0.004110	0.0	0.00	0.00
10	1.93	0.2540	0.17	0.004117	40.3	9.79	4.90
20	3.58	0.5080	0.33	0.004124	123.5	29.95	14.97
30	5.20	0.7620	0.50	0.004130	205.1	49.67	24.83
40	6.66	1.0160	0.67	0.004137	278.7	67.37	33.68
50	7.92	1.2700	0.84	0.004144	342.2	82.58	41.29
60	9.07	1.5240	1.00	0.004151	400.2	96.40	48.20
70	10.11	1.7780	1.17	0.004158	452.6	108.84	54.42
80	11.10	2.0320	1.34	0.004165	502.5	120.64	60.32
90	11.86	2.2860	1.50	0.004172	540.8	129.62	64.81
100	12.46	2.5400	1.67	0.004180	571.1	136.63	68.32
110	12.72	2.7940	1.84	0.004187	584.2	139.53	69.77
120	12.88	3.0480	2.00	0.004194	592.2	141.22	70.61
130	12.48	3.3020	2.17	0.004201	572.1	136.18	68.09
140	11.93	3.5560	2.34	0.004208	544.4	129.36	64.68



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T38
Depth (m) 12.2 - 12.8
Sample Date 24-Mar-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Tube Extraction

Recovery (mm)	540			
Bottom				Top
12.73 m	12.65 m	12.47 m	12.30 m	12.19 m
Moisture Content PP/TV Visual	Bulk/QU	Keep	Toss	
80 mm	180 mm	170 mm	110 mm	

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<10mm diam.)	
trace sand	
Color	grey
Moisture	moist
Consistency	stiff
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.75
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	73.6

Pocket Penetrometer

Reading	1	1.50
	2	1.60
	3	1.60
	Average	1.57
Undrained Shear Strength (kPa)		76.8

Moisture Content

Tare ID	QT40
Mass tare (g)	8.4
Mass wet + tare (g)	381.0
Mass dry + tare (g)	257.8
Moisture %	49.4%

Unit Weight

Bulk Weight (g)		1046.6
Length (mm)	1	148.45
	2	148.59
	3	148.45
	4	148.43
Average Length (m)		0.148
Diam. (mm)	1	72.09
	2	72.39
	3	71.90
	4	72.74
Average Diameter (m)		0.072

Volume (m³)	6.09E-04
Bulk Unit Weight (kN/m³)	16.8
Bulk Unit Weight (pcf)	107.2
Dry Unit Weight (kN/m³)	11.3
Dry Unit Weight (pcf)	71.8

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T38
Depth (m) 12.2 - 12.8
Sample Date 24-Mar-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Unconfined Strength

	kPa	ksf
Max q_u	129.7	2.7
Max S_u	64.9	1.4

Specimen Data

Description CLAY - silty, trace silt inclusions (<10mm diam.), trace sand, grey, moist, stiff, high plasticity

Length 148.5 (mm)
Diameter 72.3 (mm)
L/D Ratio 2.1
Initial Area 0.00410 (m²)
Load Rate 1.00 (%/min)

Moisture % 49%
Bulk Unit Wt. 16.8 (kN/m³)
Dry Unit Wt. 11.3 (kN/m³)
Liquid Limit -
Plastic Limit -
Plasticity Index -

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.75	73.6	1.54
Vane Size		
m		

Pocket Penetrometer

Reading	Undrained Shear Strength	
tsf	kPa	ksf
1.50	73.6	1.54
1.60	78.5	1.64
1.60	78.5	1.64
Average	1.57	76.8
		1.6

Failure Geometry

Sketch:

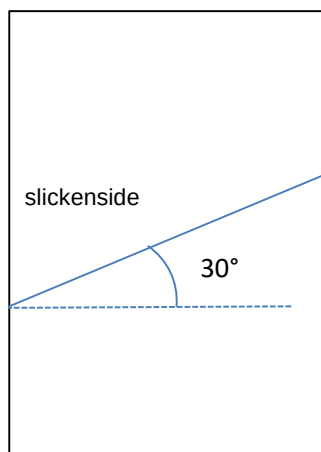
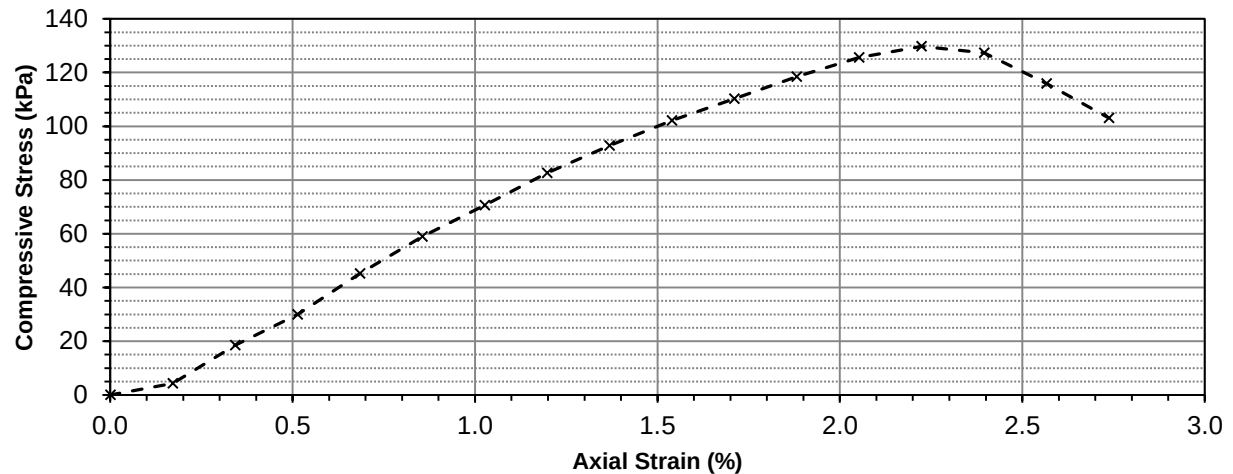


Photo:



Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	1.09	0.0000	0.00	0.004103	0.0	0.00	0.00
10	1.44	0.2540	0.17	0.004110	17.6	4.29	2.15
20	2.60	0.5080	0.34	0.004117	76.1	18.48	9.24
30	3.54	0.7620	0.51	0.004124	123.5	29.94	14.97
40	4.79	1.0160	0.68	0.004132	186.5	45.14	22.57
50	5.93	1.2700	0.86	0.004139	244.0	58.94	29.47
60	6.90	1.5240	1.03	0.004146	292.8	70.64	35.32
70	7.90	1.7780	1.20	0.004153	343.2	82.65	41.33
80	8.75	2.0320	1.37	0.004160	386.1	92.81	46.40
90	9.54	2.2860	1.54	0.004167	425.9	102.20	51.10
100	10.22	2.5400	1.71	0.004175	460.2	110.23	55.12
110	10.92	2.7940	1.88	0.004182	495.5	118.48	59.24
120	11.53	3.0480	2.05	0.004189	526.2	125.61	62.80
130	11.89	3.3020	2.22	0.004197	544.4	129.71	64.86
140	11.71	3.5560	2.39	0.004204	535.3	127.33	63.66
150	10.77	3.8100	2.57	0.004211	487.9	115.86	57.93
160	9.72	4.0640	2.74	0.004219	435.0	103.11	51.55



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T41
Depth (m) 15.2 - 15.8
Sample Date 24-Mar-25
Test Date 15-Apr-25
Technician A. Dustmamatov

Tube Extraction

Recovery (mm)	610			
Bottom				
15.85 m	15.68 m	15.51 m	15.41 m	Top 15.24 m
Keep	Bulk/QU	Moisture Content PP/TV Visual	Toss	
170 mm	170 mm	100 mm	170 mm	

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<5mm diam.)	
trace fine sand	
trace silt seam (5mm thick)	

Color	grey
Moisture	moist
Consistency	stiff
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.55
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	53.9

Pocket Penetrometer

Reading	1	1.10
	2	1.00
	3	1.10
	Average	1.07
Undrained Shear Strength (kPa)		52.3

Moisture Content

Tare ID	QT02
Mass tare (g)	8.2
Mass wet + tare (g)	458.2
Mass dry + tare (g)	294.8
Moisture %	57.0%

Unit Weight

Bulk Weight (g)	1015.4
------------------------	--------

Length (mm)	1	148.44
	2	148.99
	3	148.55
	4	148.44

Average Length (m)	0.149
---------------------------	-------

Diam. (mm)	1	71.95
	2	72.71
	3	72.46
	4	72.74

Average Diameter (m)	0.072
-----------------------------	-------

Volume (m³)	6.13E-04
Bulk Unit Weight (kN/m³)	16.2
Bulk Unit Weight (pcf)	103.4
Dry Unit Weight (kN/m³)	10.3
Dry Unit Weight (pcf)	65.9

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T41
Depth (m) 15.2 - 15.8
Sample Date 24-Mar-25
Test Date 15-Apr-25
Technician A. Dustmamatov

Unconfined Strength

	kPa	ksf
Max q_u	83.7	1.7
Max S_u	41.8	0.9

Specimen Data

Description CLAY - silty, trace silt inclusions (<5mm diam.), trace fine sand, trace silt seam (5mm thick), grey, moist, stiff, high plasticity

Length 148.6 (mm)
Diameter 72.5 (mm)
L/D Ratio 2.1
Initial Area 0.00412 (m²)
Load Rate 1.00 (%/min)

Moisture % 57%
Bulk Unit Wt. 16.2 (kN/m³)
Dry Unit Wt. 10.3 (kN/m³)
Liquid Limit 93
Plastic Limit 26
Plasticity Index 67

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.55	53.9	1.13
Vane Size		
m		

Pocket Penetrometer

Reading	Undrained Shear Strength	
tsf	kPa	ksf
1.10	54.0	1.13
1.00	49.1	1.02
1.10	54.0	1.13
Average	1.07	52.3
		1.1

Failure Geometry

Sketch:

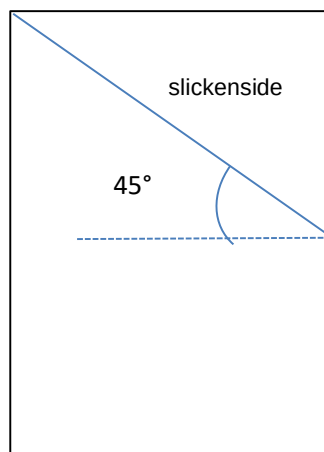
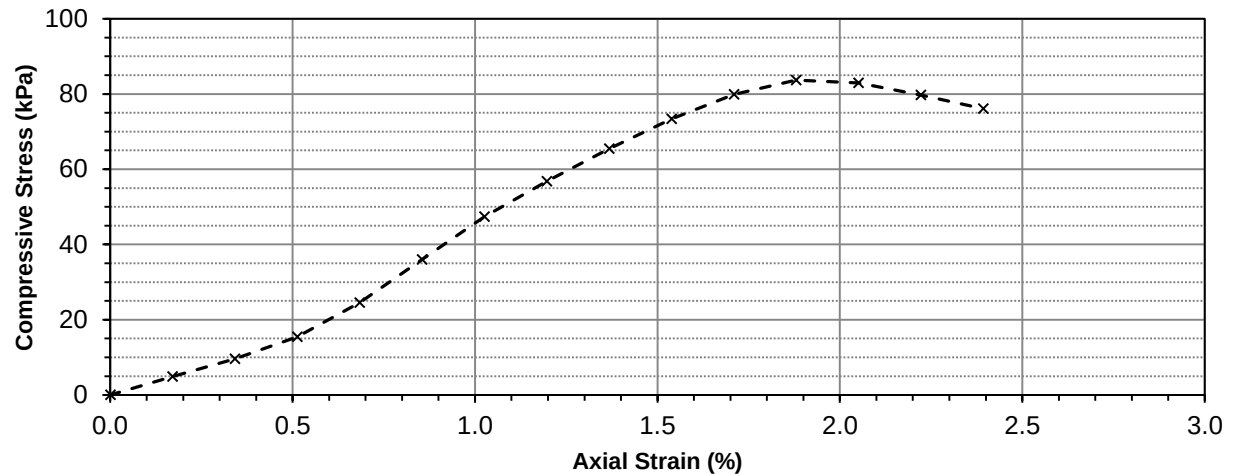


Photo:



Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	1.09	0.0000	0.00	0.004124	0.0	0.00	0.00
10	1.49	0.2540	0.17	0.004131	20.2	4.88	2.44
20	1.88	0.5080	0.34	0.004138	39.8	9.62	4.81
30	2.36	0.7620	0.51	0.004146	64.0	15.44	7.72
40	3.11	1.0160	0.68	0.004153	101.8	24.52	12.26
50	4.06	1.2700	0.85	0.004160	149.7	35.99	17.99
60	5.01	1.5240	1.03	0.004167	197.6	47.42	23.71
70	5.79	1.7780	1.20	0.004174	236.9	56.75	28.38
80	6.52	2.0320	1.37	0.004181	273.7	65.45	32.73
90	7.19	2.2860	1.54	0.004189	307.5	73.40	36.70
100	7.74	2.5400	1.71	0.004196	335.2	79.88	39.94
110	8.07	2.7940	1.88	0.004203	351.8	83.70	41.85
120	8.02	3.0480	2.05	0.004211	349.3	82.96	41.48
130	7.76	3.3020	2.22	0.004218	336.2	79.70	39.85
140	7.47	3.5560	2.39	0.004225	321.6	76.10	38.05



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T42
Depth (m) 18.3 - 18.9
Sample Date 02-Apr-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Tube Extraction

Recovery (mm)	510			
Bottom				
18.80 m	18.70 m	18.53 m	18.36 m	Top
				18.29 m
Moisture Content PP/TV Visual	Bulk/QU		Keep	Toss
100 mm	170 mm	170 mm	70 mm	

Visual Classification

Material	CLAY
Composition	silty
trace silt inclusions (<5mm diam.)	
Color	grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.30
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	29.4

Pocket Penetrometer

Reading	1	0.60
	2	0.70
	3	0.60
	Average	0.63
Undrained Shear Strength (kPa)		31.1

Moisture Content

Tare ID	D237
Mass tare (g)	7.2
Mass wet + tare (g)	376.0
Mass dry + tare (g)	222.4
Moisture %	71.4%

Unit Weight

Bulk Weight (g)		988.8
Length (mm)	1	150.30
	2	150.29
	3	150.13
	4	150.09
Average Length (m)		0.150
Diam. (mm)	1	72.02
	2	71.97
	3	72.23
	4	72.67
Average Diameter (m)		0.072

Volume (m³)	6.15E-04
Bulk Unit Weight (kN/m³)	15.8
Bulk Unit Weight (pcf)	100.3
Dry Unit Weight (kN/m³)	9.2
Dry Unit Weight (pcf)	58.5

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-02
Sample # T42
Depth (m) 18.3 - 18.9
Sample Date 02-Apr-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Unconfined Strength

	kPa	ksf
Max q_u	42.2	0.9
Max S_u	21.1	0.4

Specimen Data

Description CLAY - silty, trace silt inclusions (<5mm diam.), grey, moist, firm, high plasticity

Length	150.2	(mm)	Moisture %	71%
Diameter	72.2	(mm)	Bulk Unit Wt.	15.8 (kN/m ³)
L/D Ratio	2.1		Dry Unit Wt.	9.2 (kN/m ³)
Initial Area	0.00410	(m ²)	Liquid Limit	-
Load Rate	1.00	(%/min)	Plastic Limit	-
			Plasticity Index	-

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.30	29.4	0.61
Vane Size		
m		

Pocket Penetrometer

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.60	29.4	0.61
0.70	34.3	0.72
0.60	29.4	0.61
Average	0.63	31.1
		0.6

Failure Geometry

Sketch:

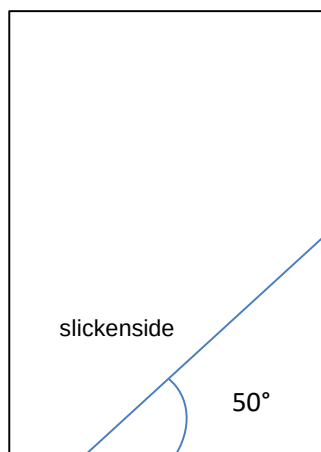
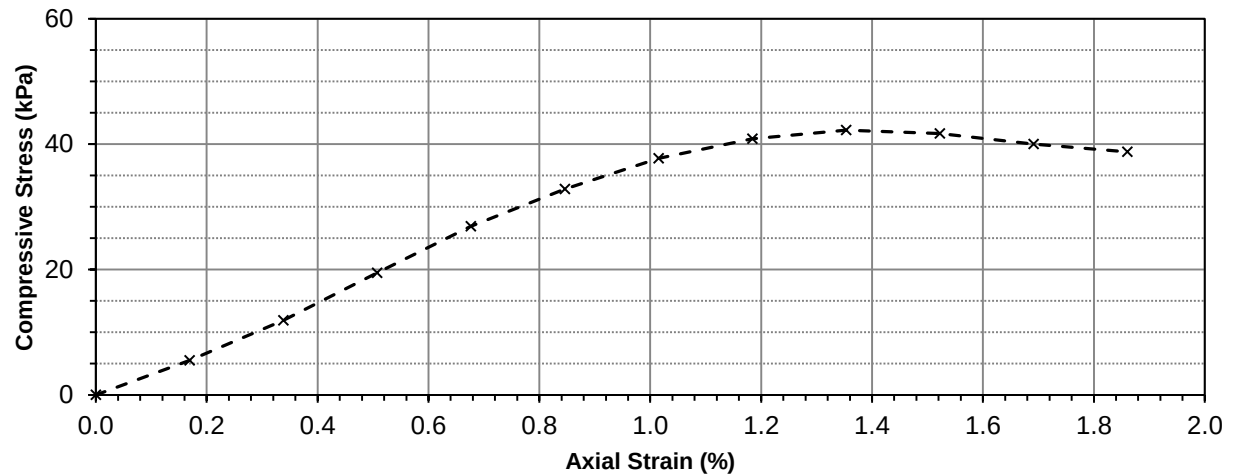


Photo:



Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	1.08	0.0000	0.00	0.004097	0.0	0.00	0.00
10	1.53	0.2540	0.17	0.004104	22.7	5.53	2.76
20	2.05	0.5080	0.34	0.004111	48.9	11.89	5.95
30	2.67	0.7620	0.51	0.004118	80.1	19.46	9.73
40	3.28	1.0160	0.68	0.004125	110.9	26.88	13.44
50	3.77	1.2700	0.85	0.004132	135.6	32.82	16.41
60	4.18	1.5240	1.01	0.004139	156.2	37.75	18.88
70	4.44	1.7780	1.18	0.004146	169.4	40.85	20.42
80	4.56	2.0320	1.35	0.004153	175.4	42.24	21.12
90	4.52	2.2860	1.52	0.004160	173.4	41.68	20.84
100	4.39	2.5400	1.69	0.004167	166.8	40.04	20.02
110	4.29	2.7940	1.86	0.004174	161.8	38.76	19.38



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-01
Sample # T57
Depth (m) 4.6 - 5.2
Sample Date 03-Apr-25
Test Date 14-Apr-25
Technician A. Bhullar

Tube Extraction

Recovery (mm)	535			
Bottom				
5.11 m	5.05 m	4.87 m	4.64 m	Top 4.57 m
Moisture Content PP/TV Visual	Bulk/QU		Keep	
60 mm	180 mm		230 mm	
			65 mm	

Visual Classification

Material	CLAY
Composition	silty
trace sand	
trace gravel (<10mm diam.)	
trace silt inclusions (<10mm diam.)	
trace oxidation	
Color	greyish brown
Moisture	moist
Consistency	stiff
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.75
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	73.6

Pocket Penetrometer

Reading	1	1.50
	2	1.50
	3	1.60
	Average	1.53
Undrained Shear Strength (kPa)		75.2

Moisture Content

Tare ID	AB61
Mass tare (g)	6.8
Mass wet + tare (g)	426.4
Mass dry + tare (g)	300.8
Moisture %	42.7%

Unit Weight

Bulk Weight (g)		1131.6
Length (mm)	1	151.63
	2	152.20
	3	151.90
	4	151.80
Average Length (m)		0.152
Diam. (mm)	1	72.20
	2	72.50
	3	72.50
	4	72.06
Average Diameter (m)		0.072

Volume (m³)	6.24E-04
Bulk Unit Weight (kN/m³)	17.8
Bulk Unit Weight (pcf)	113.2
Dry Unit Weight (kN/m³)	12.5
Dry Unit Weight (pcf)	79.3

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-01
Sample # T57
Depth (m) 4.6 - 5.2
Sample Date 03-Apr-25
Test Date 14-Apr-25
Technician A. Bhullar

Unconfined Strength

	kPa	ksf
Max q_u	158.0	3.3
Max S_u	79.0	1.6

Specimen Data

Description CLAY - silty, trace sand, trace gravel (<10mm diam.), trace silt inclusions (<10mm diam.), trace oxidation, greyish brown, moist, stiff, high plasticity

Length	151.9	(mm)	Moisture %	43%
Diameter	72.3	(mm)	Bulk Unit Wt.	17.8 (kN/m ³)
L/D Ratio	2.1		Dry Unit Wt.	12.5 (kN/m ³)
Initial Area	0.00411	(m ²)	Liquid Limit	-
Load Rate	1.00	(%/min)	Plastic Limit	-
			Plasticity Index	-

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.75	73.6	1.54
Vane Size		
m		

Pocket Penetrometer

Reading	Undrained Shear Strength	
tsf	kPa	ksf
1.50	73.6	1.54
1.50	73.6	1.54
1.60	78.5	1.64
Average	1.53	75.2
		1.6

Failure Geometry

Sketch:

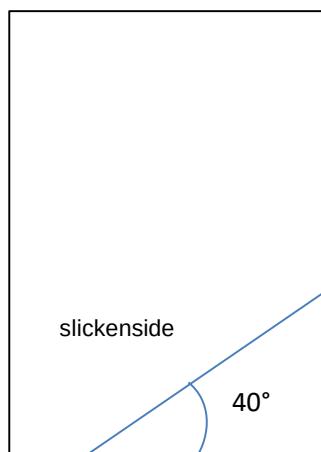
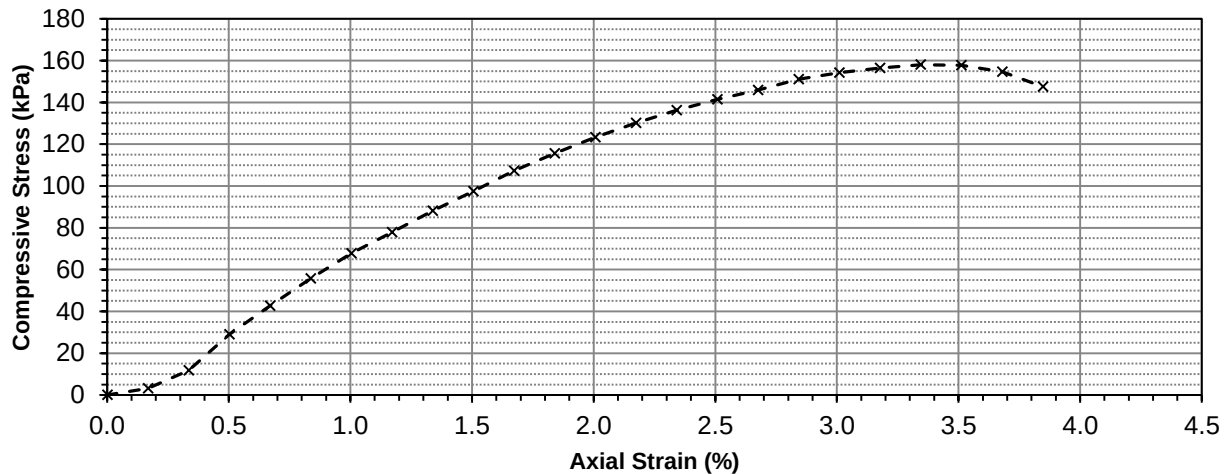


Photo:



Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	1.02	0.0000	0.00	0.004107	0.0	0.00	0.00
10	1.28	0.2540	0.17	0.004114	13.1	3.19	1.59
20	1.98	0.5080	0.33	0.004121	48.4	11.74	5.87
30	3.39	0.7620	0.50	0.004128	119.5	28.94	14.47
40	4.52	1.0160	0.67	0.004135	176.4	42.66	21.33
50	5.60	1.2700	0.84	0.004142	230.8	55.74	27.87
60	6.60	1.5240	1.00	0.004149	281.2	67.79	33.89
70	7.44	1.7780	1.17	0.004156	323.6	77.86	38.93
80	8.30	2.0320	1.34	0.004163	366.9	88.14	44.07
90	9.08	2.2860	1.51	0.004170	406.2	97.42	48.71
100	9.91	2.5400	1.67	0.004177	448.1	107.27	53.64
110	10.61	2.7940	1.84	0.004184	483.4	115.52	57.76
120	11.27	3.0480	2.01	0.004191	516.6	123.26	61.63
130	11.86	3.3020	2.17	0.004198	546.4	130.13	65.07
140	12.39	3.5560	2.34	0.004206	573.1	136.26	68.13
150	12.84	3.8100	2.51	0.004213	595.8	141.41	70.71
160	13.23	4.0640	2.68	0.004220	615.4	145.83	72.91
170	13.69	4.3180	2.84	0.004227	638.6	151.06	75.53
180	13.97	4.5720	3.01	0.004235	652.7	154.14	77.07
190	14.19	4.8260	3.18	0.004242	663.8	156.48	78.24
200	14.34	5.0800	3.34	0.004249	671.4	157.99	79.00
210	14.34	5.3340	3.51	0.004257	671.4	157.72	78.86
220	14.10	5.5880	3.68	0.004264	659.3	154.61	77.31
230	13.52	5.8420	3.85	0.004272	630.0	147.50	73.75



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-01
Sample # T60
Depth (m) 7.6 - 8.2
Sample Date 03-Apr-25
Test Date 14-Apr-25
Technician A. Dustmamatov

Tube Extraction

Recovery (mm)	520			Top					
Bottom				7.62 m					
8.14 m	7.97 m	7.90 m	7.73 m						
<table><tr><td>Bulk/QU</td><td>Moisture Content PP/TV Visual</td><td>Keep</td><td>Toss</td><td></td></tr></table>					Bulk/QU	Moisture Content PP/TV Visual	Keep	Toss	
Bulk/QU	Moisture Content PP/TV Visual	Keep	Toss						
170 mm	70 mm	170 mm	110 mm						

Visual Classification

Material	CLAY
Composition	silty
trace gravel (<10mm diam.)	
trace silt inclusions (<5mm diam.)	
Color	grey
Moisture	moist
Consistency	stiff
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.65
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	63.8

Pocket Penetrometer

Reading	1	1.20
	2	1.30
	3	1.20
	Average	1.23
Undrained Shear Strength (kPa)		60.5

Moisture Content

Tare ID	D227
Mass tare (g)	6.8
Mass wet + tare (g)	391.0
Mass dry + tare (g)	250.8
Moisture %	57.5%

Unit Weight

Bulk Weight (g)		1062.2
Length (mm)	1	150.68
	2	151.09
	3	151.01
	4	150.75
Average Length (m)		0.151
Diam. (mm)	1	72.10
	2	72.48
	3	72.02
	4	72.48
Average Diameter (m)		0.072

Volume (m³)	6.19E-04
Bulk Unit Weight (kN/m³)	16.8
Bulk Unit Weight (pcf)	107.1
Dry Unit Weight (kN/m³)	10.7
Dry Unit Weight (pcf)	68.0

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-01
Sample # T60
Depth (m) 7.6 - 8.2
Sample Date 03-Apr-25
Test Date 14-Apr-25
Technician A. Dustmamatov

Unconfined Strength

	kPa	ksf
Max q_u	83.7	1.7
Max S_u	41.9	0.9

Specimen Data

Description CLAY - silty, trace gravel (<10mm diam.), trace silt inclusions (<5mm diam.), grey, moist, stiff, high plasticity

Length	150.9	(mm)	Moisture %	57%
Diameter	72.3	(mm)	Bulk Unit Wt.	16.8 (kN/m ³)
L/D Ratio	2.1		Dry Unit Wt.	10.7 (kN/m ³)
Initial Area	0.00410	(m ²)	Liquid Limit	-
Load Rate	1.00	(%/min)	Plastic Limit	-
			Plasticity Index	-

Undrained Shear Strength Tests

Torvane

Reading	Undrained Shear Strength	
tsf	kPa	ksf
0.65	63.8	1.33
Vane Size		
m		

Pocket Penetrometer

Reading	Undrained Shear Strength	
tsf	kPa	ksf
1.20	58.9	1.23
1.30	63.8	1.33
1.20	58.9	1.23
Average	1.23	60.5
		1.3

Failure Geometry

Sketch:

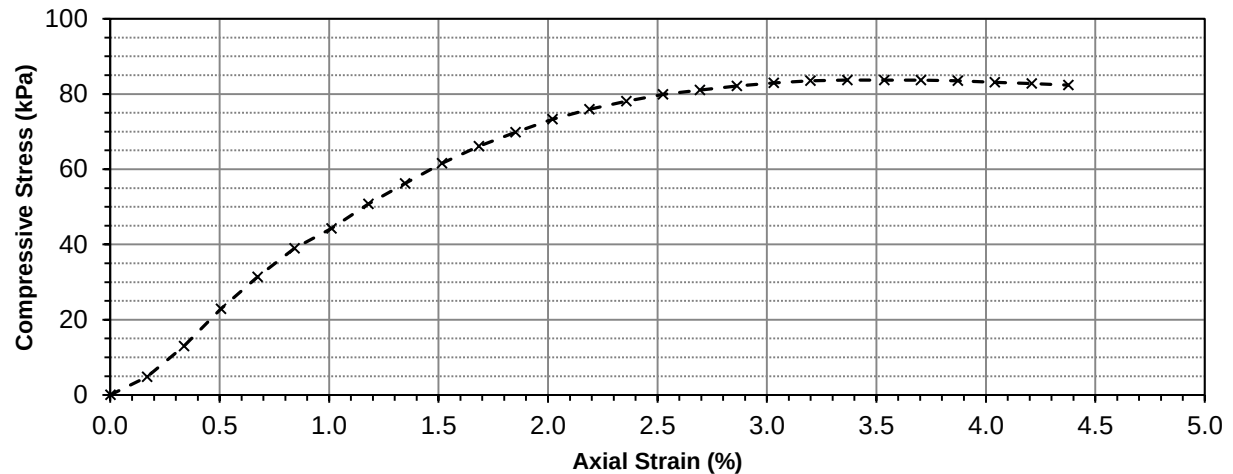


Photo:



Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	1.05	0.0000	0.00	0.004102	0.0	0.00	0.00
10	1.44	0.2540	0.17	0.004109	19.7	4.78	2.39
20	2.11	0.5080	0.34	0.004116	53.4	12.98	6.49
30	2.92	0.7620	0.51	0.004123	94.3	22.86	11.43
40	3.62	1.0160	0.67	0.004130	129.5	31.37	15.68
50	4.25	1.2700	0.84	0.004137	161.3	38.99	19.49
60	4.69	1.5240	1.01	0.004144	183.5	44.27	22.14
70	5.23	1.7780	1.18	0.004151	210.7	50.75	25.38
80	5.69	2.0320	1.35	0.004158	233.9	56.24	28.12
90	6.14	2.2860	1.52	0.004165	256.6	61.59	30.80
100	6.52	2.5400	1.68	0.004172	275.7	66.08	33.04
110	6.84	2.7940	1.85	0.004179	291.8	69.83	34.91
120	7.14	3.0480	2.02	0.004187	307.0	73.32	36.66
130	7.37	3.3020	2.19	0.004194	318.5	75.96	37.98
140	7.56	3.5560	2.36	0.004201	328.1	78.10	39.05
150	7.72	3.8100	2.53	0.004208	336.2	79.89	39.94
160	7.83	4.0640	2.69	0.004216	341.7	81.06	40.53
170	7.93	4.3180	2.86	0.004223	346.8	82.12	41.06
180	8.01	4.5720	3.03	0.004230	350.8	82.93	41.46
190	8.07	4.8260	3.20	0.004238	353.8	83.50	41.75
200	8.10	5.0800	3.37	0.004245	355.3	83.71	41.85
210	8.11	5.3340	3.54	0.004252	355.8	83.68	41.84
220	8.12	5.5880	3.70	0.004260	356.3	83.65	41.83
230	8.12	5.8420	3.87	0.004267	356.3	83.51	41.75



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Unconfined Compressive Strength ASTM D2166

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Data (cont'd)

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
240	8.10	6.0960	4.04	0.004275	355.3	83.12	41.56
250	8.08	6.3500	4.21	0.004282	354.3	82.74	41.37
260	8.06	6.6040	4.38	0.004290	353.3	82.36	41.18



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Shelby Tube Visual

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-01
Sample # T62
Depth (m) 10.7 - 11.3
Sample Date 03-Apr-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Tube Extraction

Recovery (mm)	680			
Bottom				
11.35 m	11.25 m	11.07 m	10.89 m	Top
				10.67 m
Moisture Content PP/TV Visual	Bulk/QU	Keep	Toss	
100 mm	180 mm	180 mm	220 mm	

Visual Classification

Material	CLAY
Composition	silty
trace sand	
trace gravel (<25mm diam.)	
trace silt inclusions (<15mm diam.)	

Color	grey
Moisture	moist
Consistency	firm
Plasticity	high plasticity
Structure	-
Gradation	-

Torvane

Reading	0.40
Vane Size (s,m,l)	m
Undrained Shear Strength (kPa)	39.2

Pocket Penetrometer

Reading	1	0.90
	2	0.80
	3	0.80
	Average	0.83
Undrained Shear Strength (kPa)		40.9

Moisture Content

Tare ID	M28
Mass tare (g)	7.0
Mass wet + tare (g)	334.8
Mass dry + tare (g)	220.0
Moisture %	53.9%

Unit Weight

Bulk Weight (g)		1037.2
Length (mm)	1	147.48
	2	147.58
	3	147.63
	4	147.85
Average Length (m)		0.148
Diam. (mm)	1	72.42
	2	72.39
	3	72.43
	4	72.39
Average Diameter (m)		0.072

Volume (m³)	6.08E-04
Bulk Unit Weight (kN/m³)	16.7
Bulk Unit Weight (pcf)	106.5
Dry Unit Weight (kN/m³)	10.9
Dry Unit Weight (pcf)	69.2

Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Test Hole TH25-01
Sample # T62
Depth (m) 10.7 - 11.3
Sample Date 03-Apr-25
Test Date 16-Apr-25
Technician A. Dustmamatov

Unconfined Strength

	kPa	ksf
Max q_u	82.0	1.7
Max S_u	41.0	0.9

Specimen Data

Description CLAY - silty, trace sand, trace gravel (<25mm diam.), trace silt inclusions (<15mm diam.), grey, moist, firm, high plasticity

Length 147.6 (mm)
Diameter 72.4 (mm)
L/D Ratio 2.0
Initial Area 0.00412 (m²)
Load Rate 1.00 (%/min)

Moisture % 54%
Bulk Unit Wt. 16.7 (kN/m³)
Dry Unit Wt. 10.9 (kN/m³)
Liquid Limit -
Plastic Limit -
Plasticity Index -

Undrained Shear Strength Tests

Torvane

Reading
tsf
0.40
Vane Size
m

Undrained Shear Strength
kPa
39.2
ksf
0.82

Pocket Penetrometer

Reading
tsf
0.90
0.80
0.80
Average
0.83

Undrained Shear Strength
kPa
44.1
39.2
39.2
40.9
ksf
0.92
0.82
0.82
0.9

Failure Geometry

Sketch:

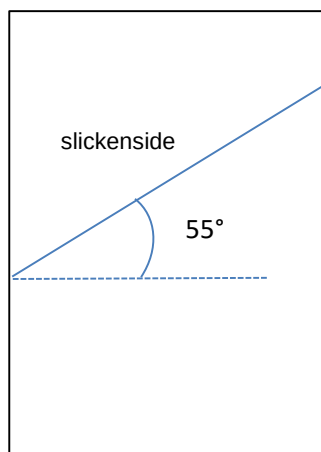
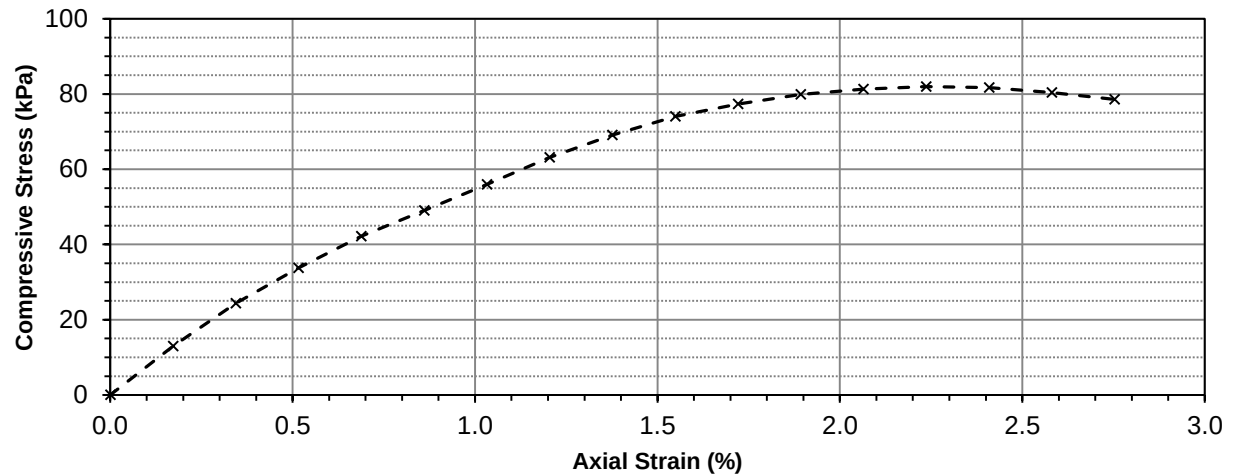


Photo:



Project No. 0002-171-00
Client Tetra Tech Inc.
Project Louise Bridge Rehabilitation

Unconfined Compression Test Graph



Unconfined Compression Test Data

Deformation Dial Reading	Load Ring Dial Reading	Deflection (mm)	Axial Strain (%)	Corrected Area (m ²)	Axial Load (N)	Compressive Stress, q_u (kPa)	Shear Stress, S_u (kPa)
0	1.03	0.0000	0.00	0.004118	0.0	0.00	0.00
10	2.09	0.2540	0.17	0.004125	53.4	12.95	6.48
20	3.03	0.5080	0.34	0.004132	100.8	24.40	12.20
30	3.80	0.7620	0.52	0.004139	139.6	33.73	16.87
40	4.50	1.0160	0.69	0.004146	174.9	42.18	21.09
50	5.07	1.2700	0.86	0.004153	203.6	49.03	24.51
60	5.65	1.5240	1.03	0.004161	232.9	55.97	27.98
70	6.25	1.7780	1.20	0.004168	263.1	63.13	31.56
80	6.75	2.0320	1.38	0.004175	288.3	69.05	34.53
90	7.17	2.2860	1.55	0.004182	309.5	73.99	37.00
100	7.46	2.5400	1.72	0.004190	324.1	77.35	38.68
110	7.68	2.7940	1.89	0.004197	335.2	79.86	39.93
120	7.81	3.0480	2.06	0.004205	341.7	81.28	40.64
130	7.88	3.3020	2.24	0.004212	345.3	81.97	40.99
140	7.87	3.5560	2.41	0.004219	344.8	81.71	40.85
150	7.77	3.8100	2.58	0.004227	339.7	80.37	40.19
160	7.63	4.0640	2.75	0.004234	332.7	78.56	39.28



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MEMORANDUM

Date	May 13, 2025
To	Nuno Mendonca, Micheal Van Helden TREK Geotechnical
From	Angela Fidler-Kliewer, TREK Geotechnical
Project No.	0002-171-00
Project	Louise Bridge
Subject	Laboratory Testing Results – Lab Req. R25-142

Distribution	David Clark
---------------------	-------------

Attached are the laboratory testing results for the above noted project. The testing included unconfined compressive strength on Rock Core samples.

Regards,

Angela Fidler-Kliewer, C.Tech.

Attach.

Review Control:

<i>Prepared By: KM</i>	<i>Reviewed By: AFK</i>	<i>Checked By: NJF</i>
------------------------	-------------------------	------------------------



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Rock Core Unconfined Compressive Strength Report

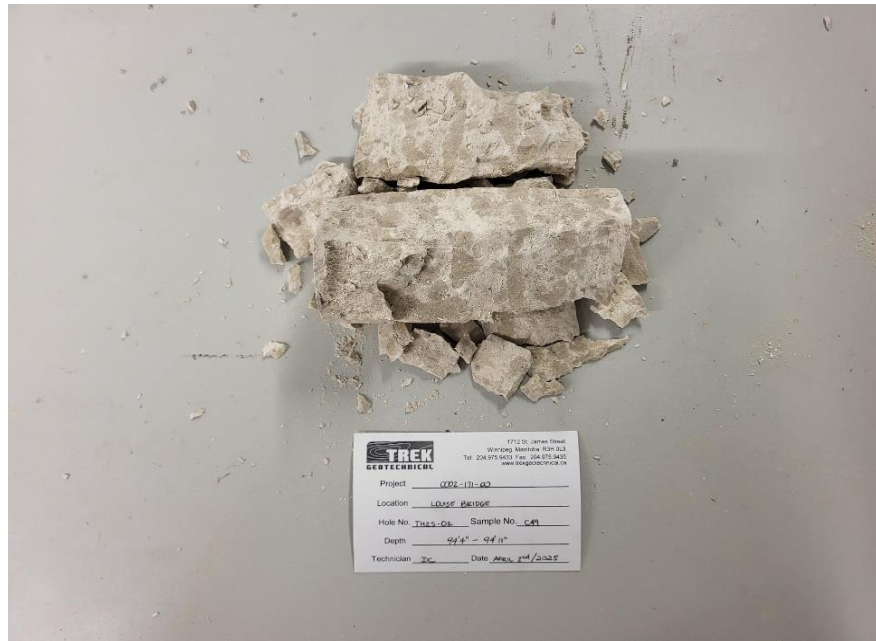
STANDARD TEST METHOD FOR UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN (ASTM D 7012)

Project No. 0002-171-00
Project Louise Bridge
Client Tetra Tech

Date Received 12-May-25
Sampled by D. Clark
Requested by N. Mendonca

Test Date 13-May-25
Report No. R25-142
Technician K. Manchur

Core No.	Core Length as Received (mm)	Core Diameter (mm)	Core Length (mm)	Core Weight (g)	Density (g/mm ³)	Area (sq.mm)	Core Load (kN)	Core Strength (Mpa)	Notes
TH25-02, C49	230	63.00	128.00	977.2	2.449E-03	3117.245	118.68	38.1	28.8 - 28.9m



Comments:



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Rock Core Unconfined Compressive Strength Report

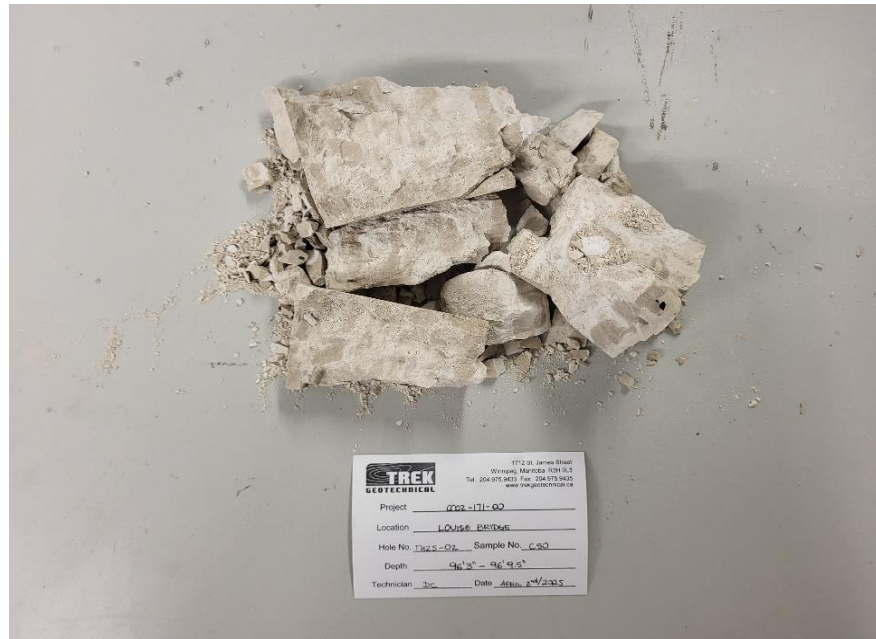
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Project No. 0002-171-00
Project Louise Bridge
Client Tetra Tech

Date Received 12-May-25
Sampled by D. Clark
Requested by N. Mendonca

Test Date 13-May-25
Report No. R25-142
Technician K. Manchur

Core No.	Core Length as Received (mm)	Core Diameter (mm)	Core Length (mm)	Core Weight (g)	Density (g/mm ³)	Area (sq.mm)	Core Load (kN)	Core Strength (Mpa)	Notes
TH25-02, C50	190	63.00	135.00	992.8	2.359E-03	3117.245	112.86	36.2	29.3 - 29.5m



Comments:



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

Rock Core Unconfined Compressive Strength Report

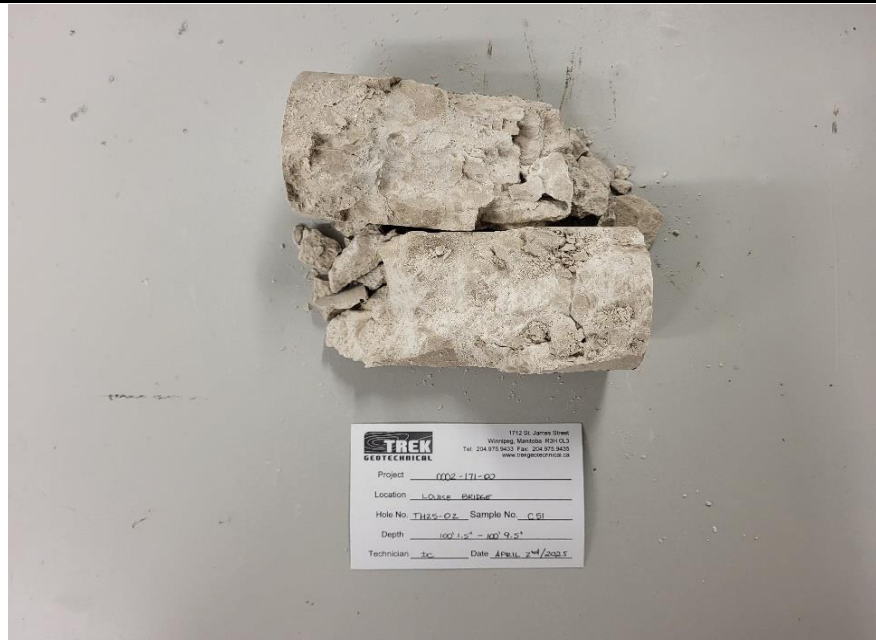
STANDARD TEST METHOD FOR UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN (ASTM D 7012)

Project No. 0002-171-00
Project Louise Bridge
Client Tetra Tech

Date Received 12-May-25
Sampled by D. Clark
Requested by N. Mendonca

Test Date 13-May-25
Report No. R25-142
Technician K. Manchur

Core No.	Core Length as Received (mm)	Core Diameter (mm)	Core Length (mm)	Core Weight (g)	Density (g/mm ³)	Area (sq.mm)	Core Load (kN)	Core Strength (Mpa)	Notes
TH25-02, C51	224	63.00	135.00	1012	2.405E-03	3117.245	119.76	38.4	30.5 - 30.7m



Comments:



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Rock Core Unconfined Compressive Strength Report

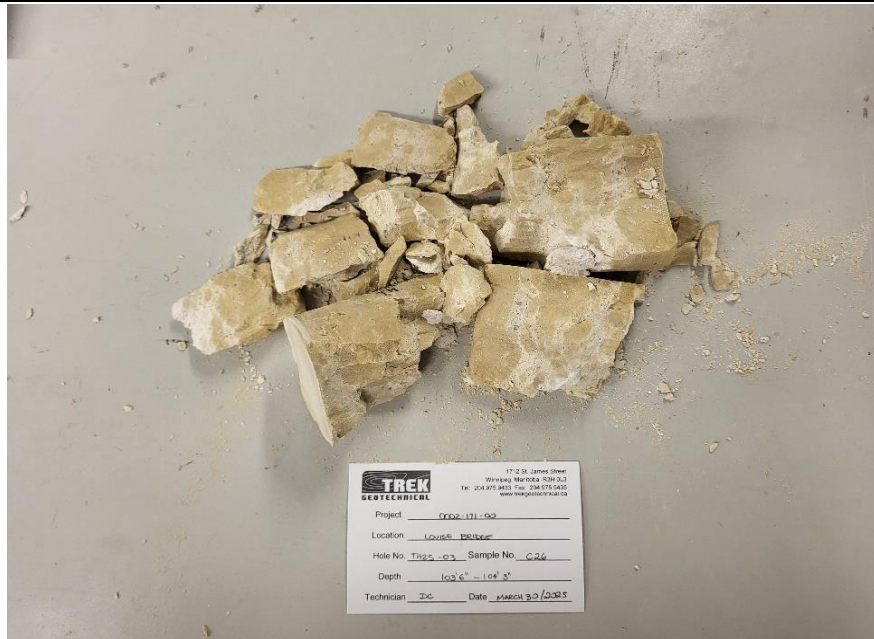
STANDARD TEST METHOD FOR UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN (ASTM D 7012)

Project No. 0002-171-00
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Requested by N. Mendonca

Test Date 13-May-25
Report No. R25-142
Technician K. Manchur

Core No.	Core Length as Received (mm)	Core Diameter (mm)	Core Length (mm)	Core Weight (g)	Density (g/mm ³)	Area (sq.mm)	Core Load (kN)	Core Strength (Mpa)	Notes
TH25-03, C26	235	63.00	135.00	1014.2	2.410E-03	3117.245	117.33	37.6	31.5 - 31.8m



Comments:



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Rock Core Unconfined Compressive Strength Report

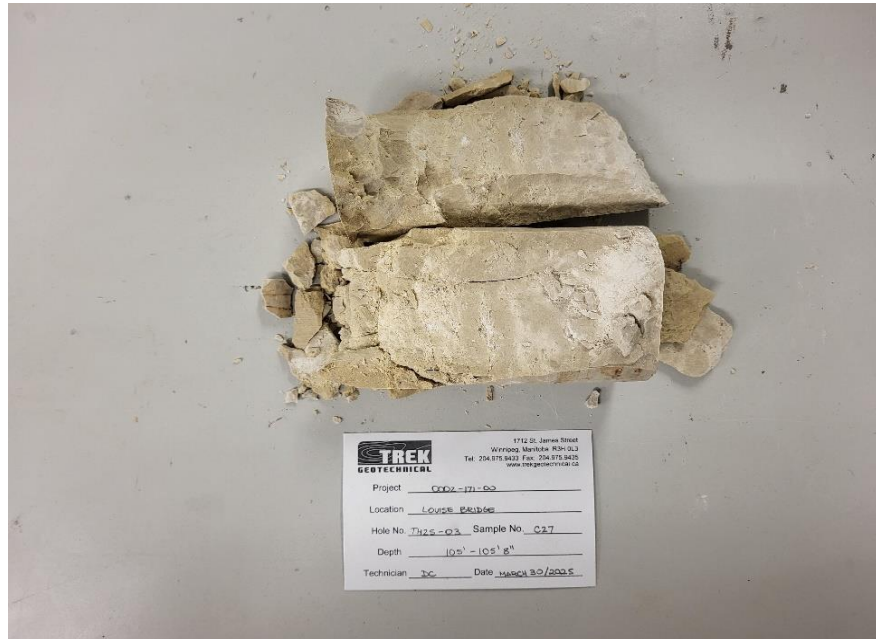
STANDARD TEST METHOD FOR UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN (ASTM D 7012)

Project No. 0002-171-00
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Date Received 12-May-25
Sampled by D. Clark
Requested by N. Mendonca

Test Date 13-May-25
Report No. R25-142
Technician K. Manchur

Core No.	Core Length as Received (mm)	Core Diameter (mm)	Core Length (mm)	Core Weight (g)	Density (g/mm ³)	Area (sq.mm)	Core Load (kN)	Core Strength (Mpa)	Notes
TH25-03, C27	208	63.00	134.00	1015.4	2.431E-03	3117.245	145.34	46.6	32.0 - 32.2m

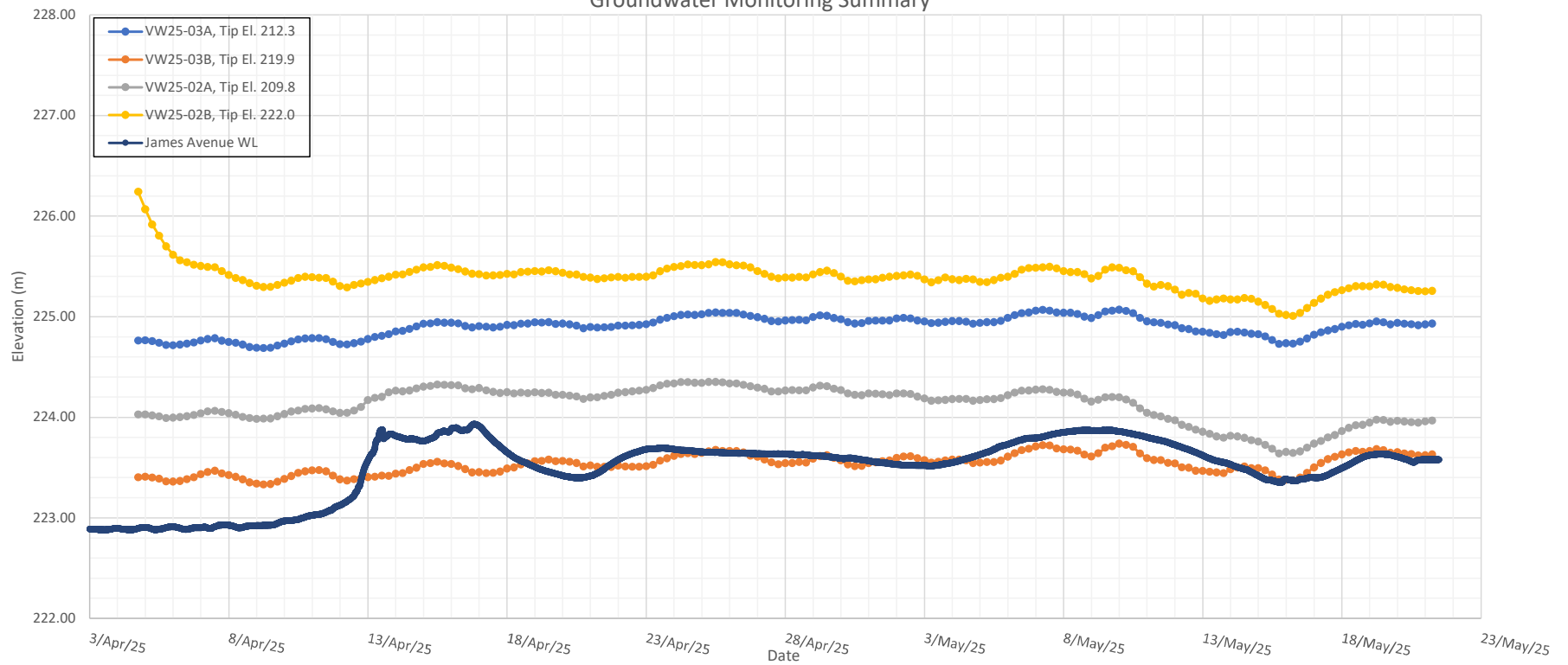


Comments:

Appendix C

Groundwater monitoring data

Louise Bridge Rehabilitation Groundwater Monitoring Summary



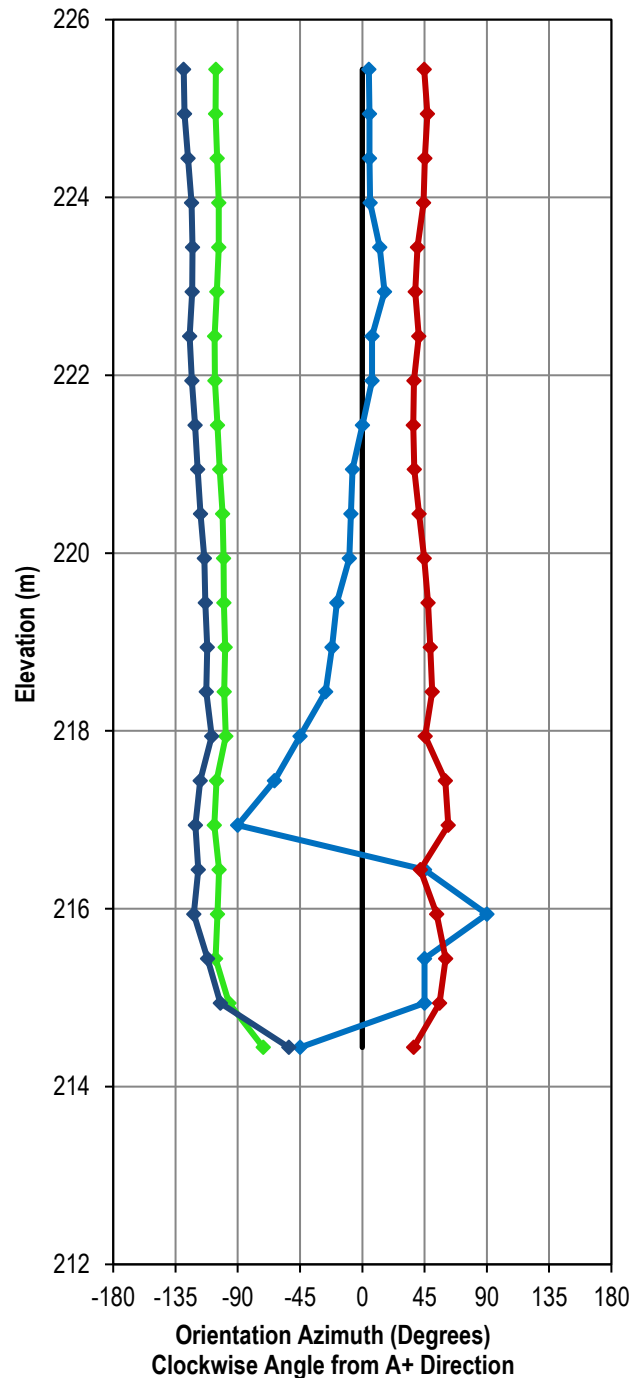
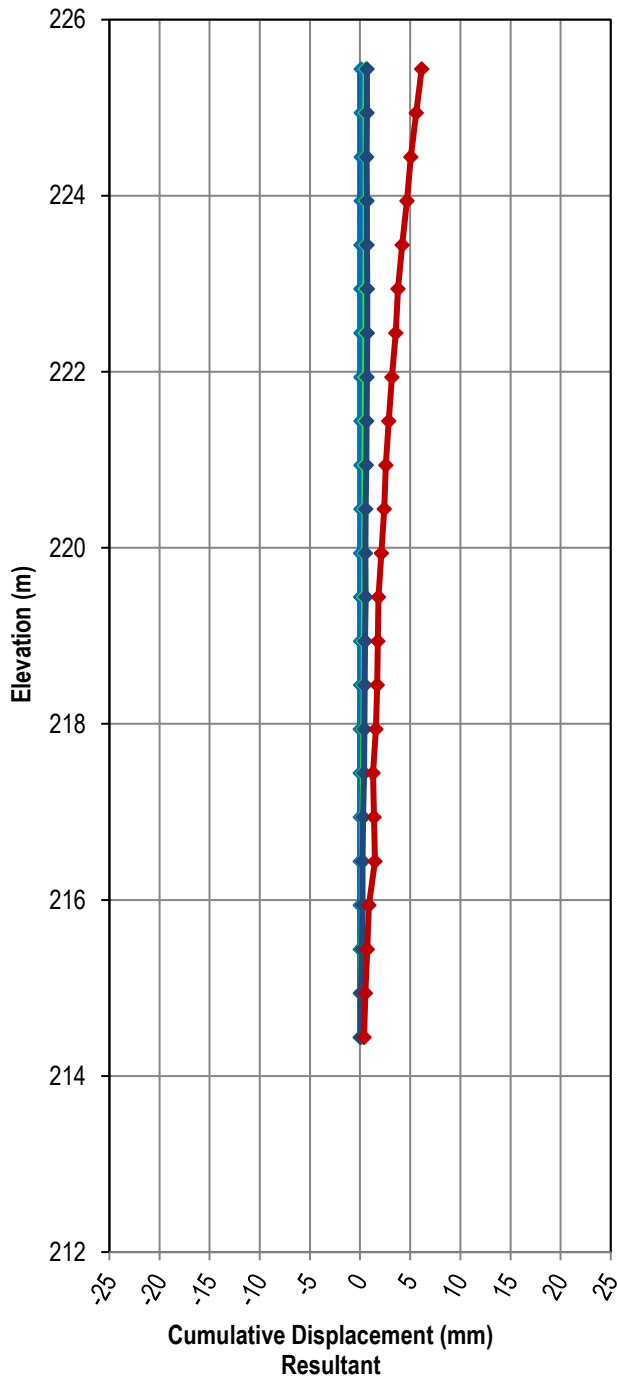
Appendix D

Slope Inclinometer Plots

SLOPE INCLINOMETER DATA PLOTS

KGS TH20-01

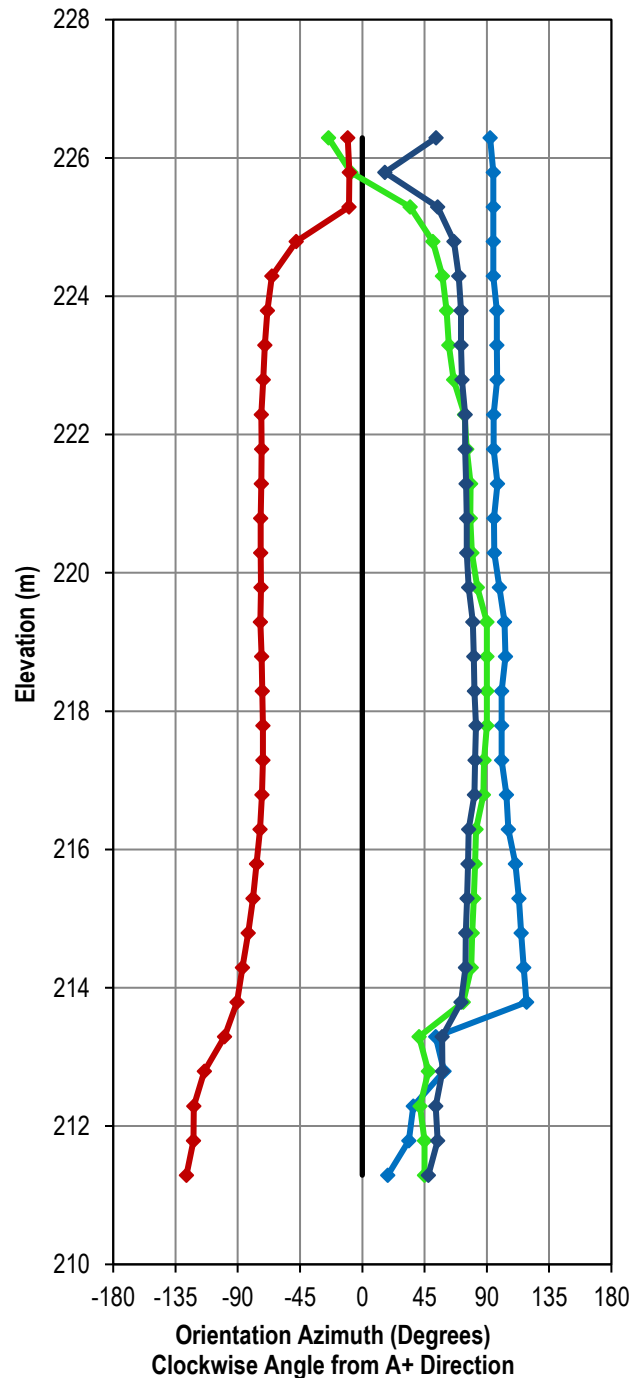
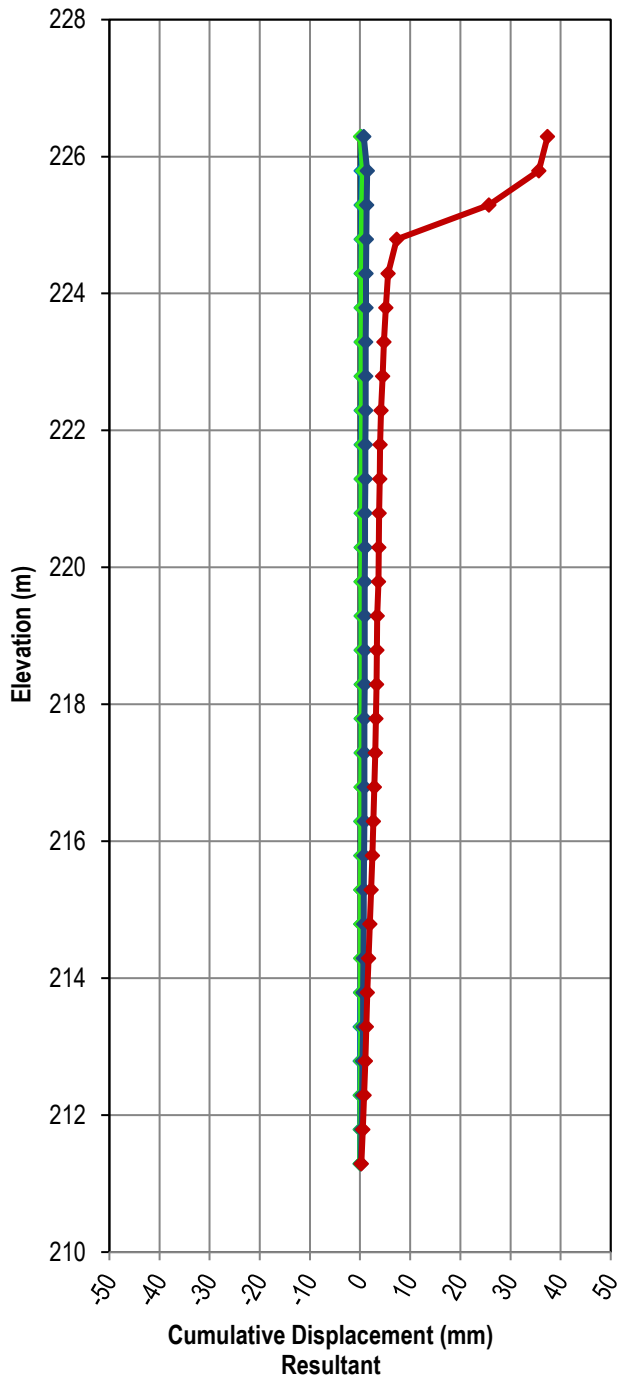
Baseline 2020-08-18 2020-09-04 2020-09-23 2024-12-19



SLOPE INCLINOMETER DATA PLOTS

KGS TH20-02

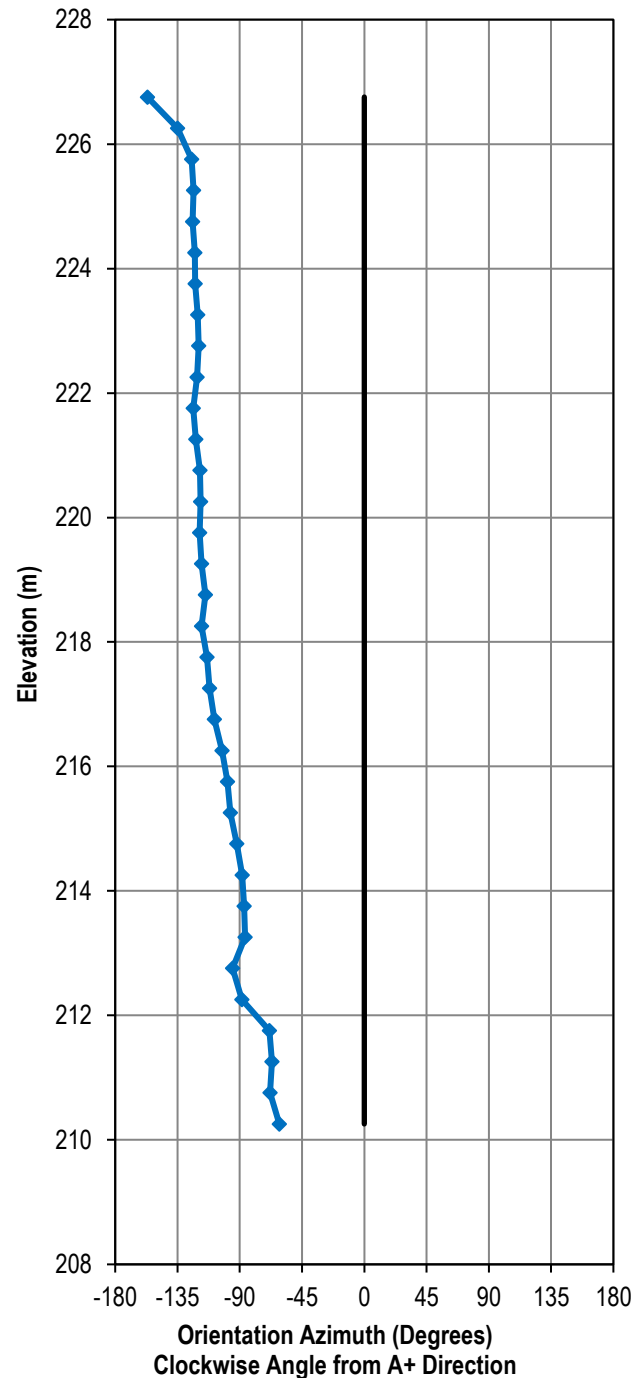
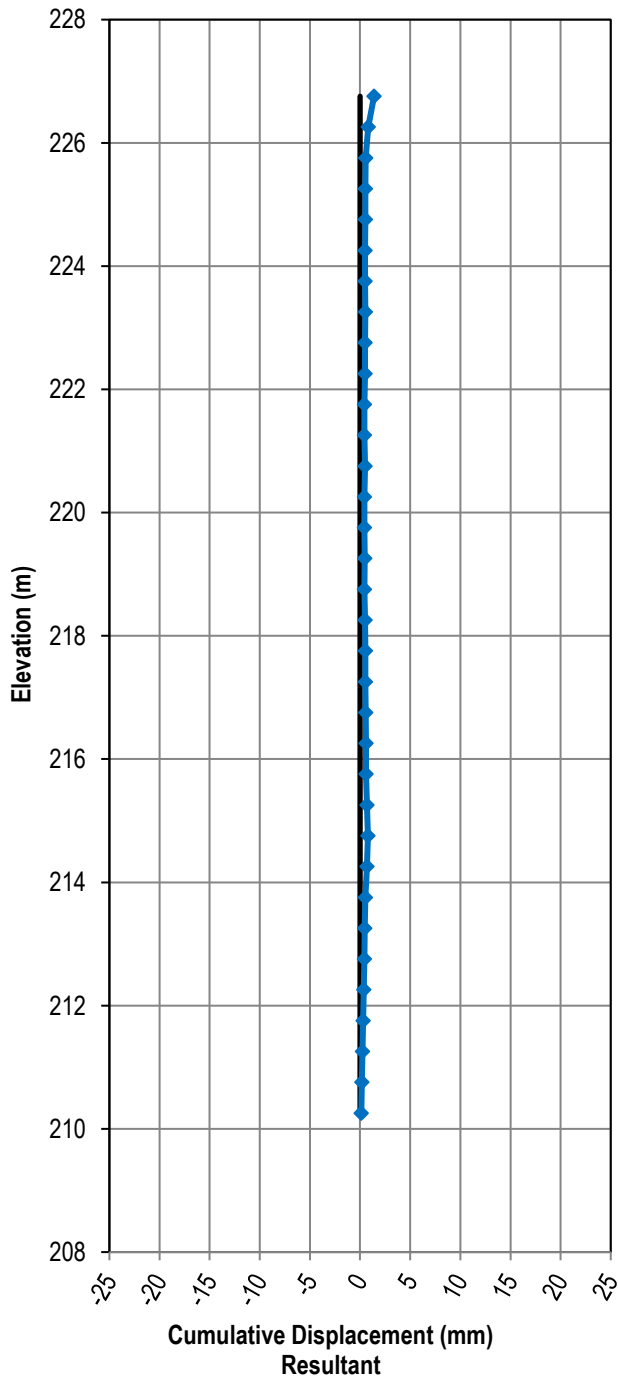
Baseline 2020-08-18 2020-09-04 2020-09-23 2024-12-19



SLOPE INCLINOMETER DATA PLOTS

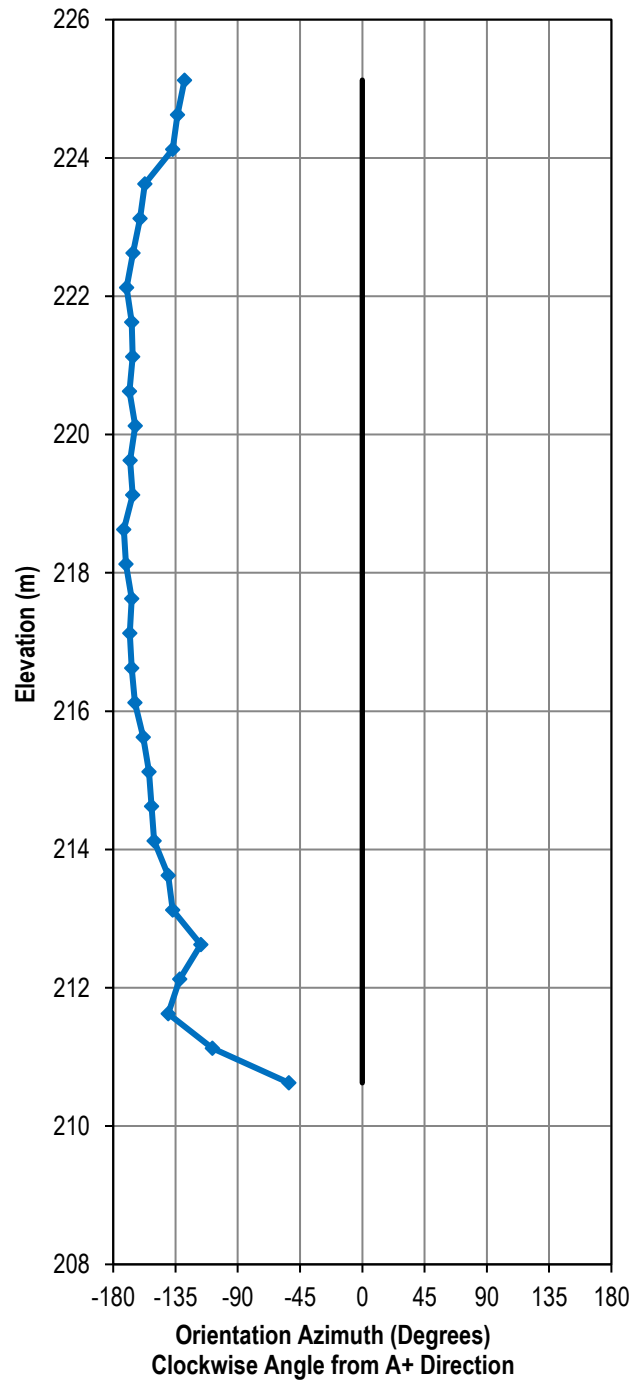
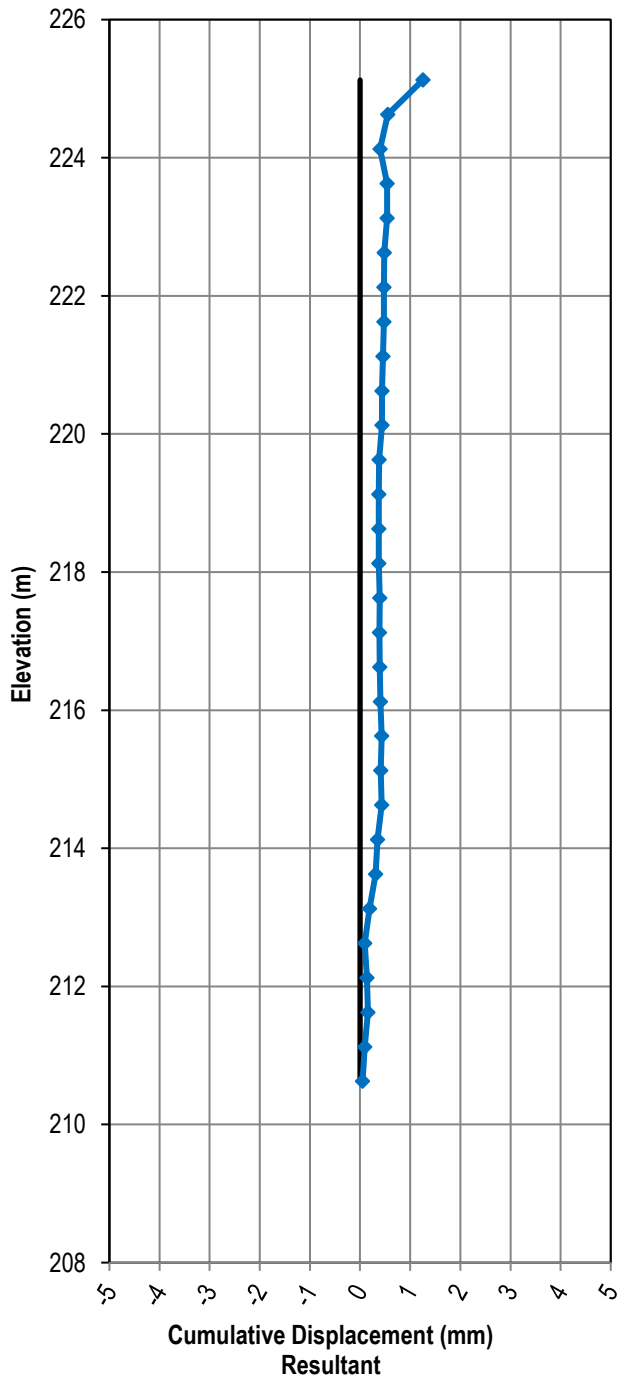
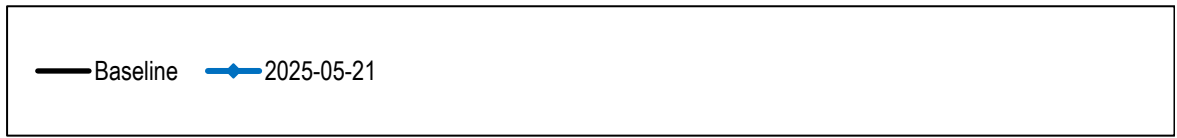
TH25-03A

— Baseline — 2025-05-21



SLOPE INCLINOMETER DATA PLOTS

TH25-01



Appendix E

Slope Stability Analysis Output

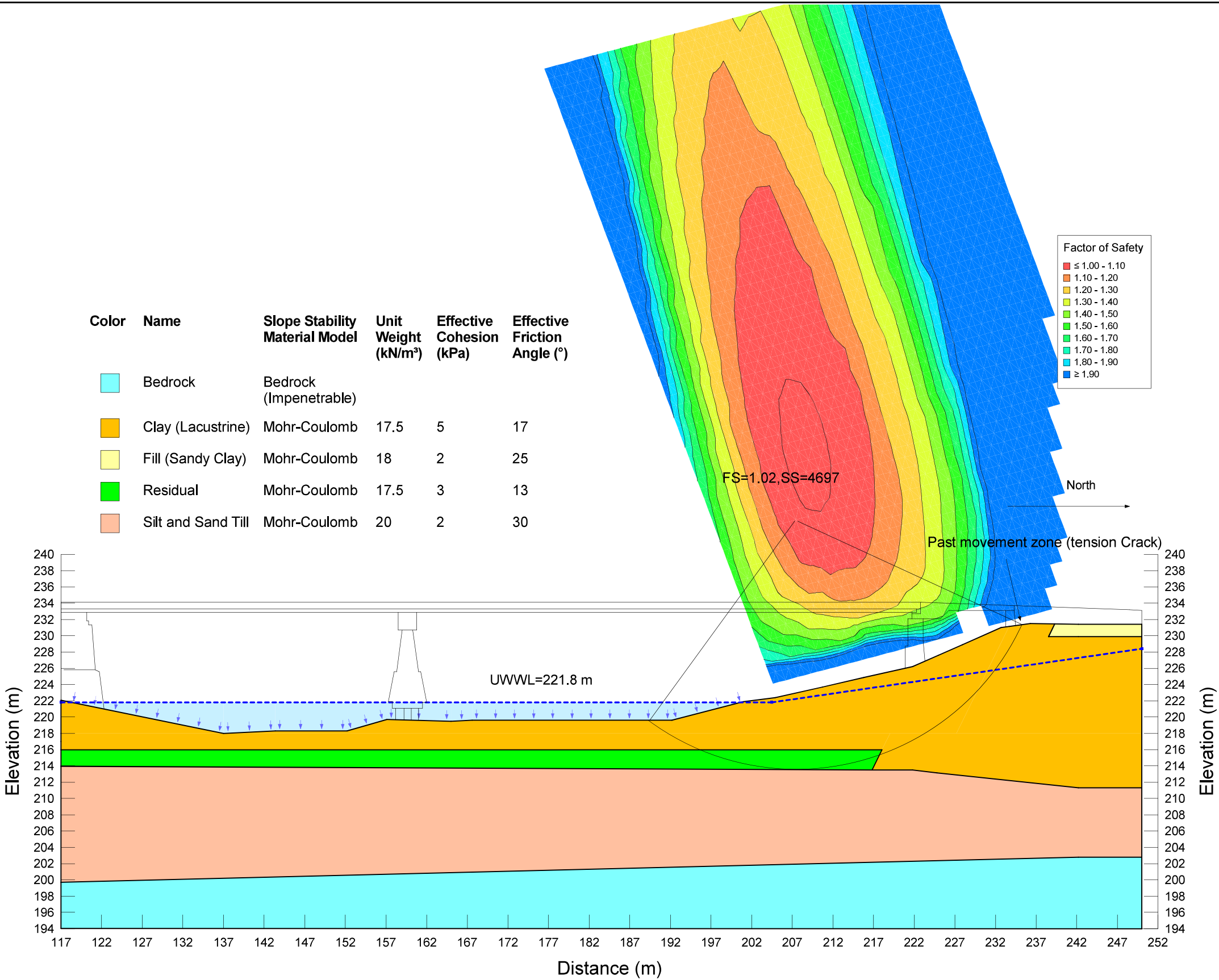


Figure E-1

Limit-Equilibrium Slope Stability Analysis
Short-Term Conditions - North Abutment - Back Analysis

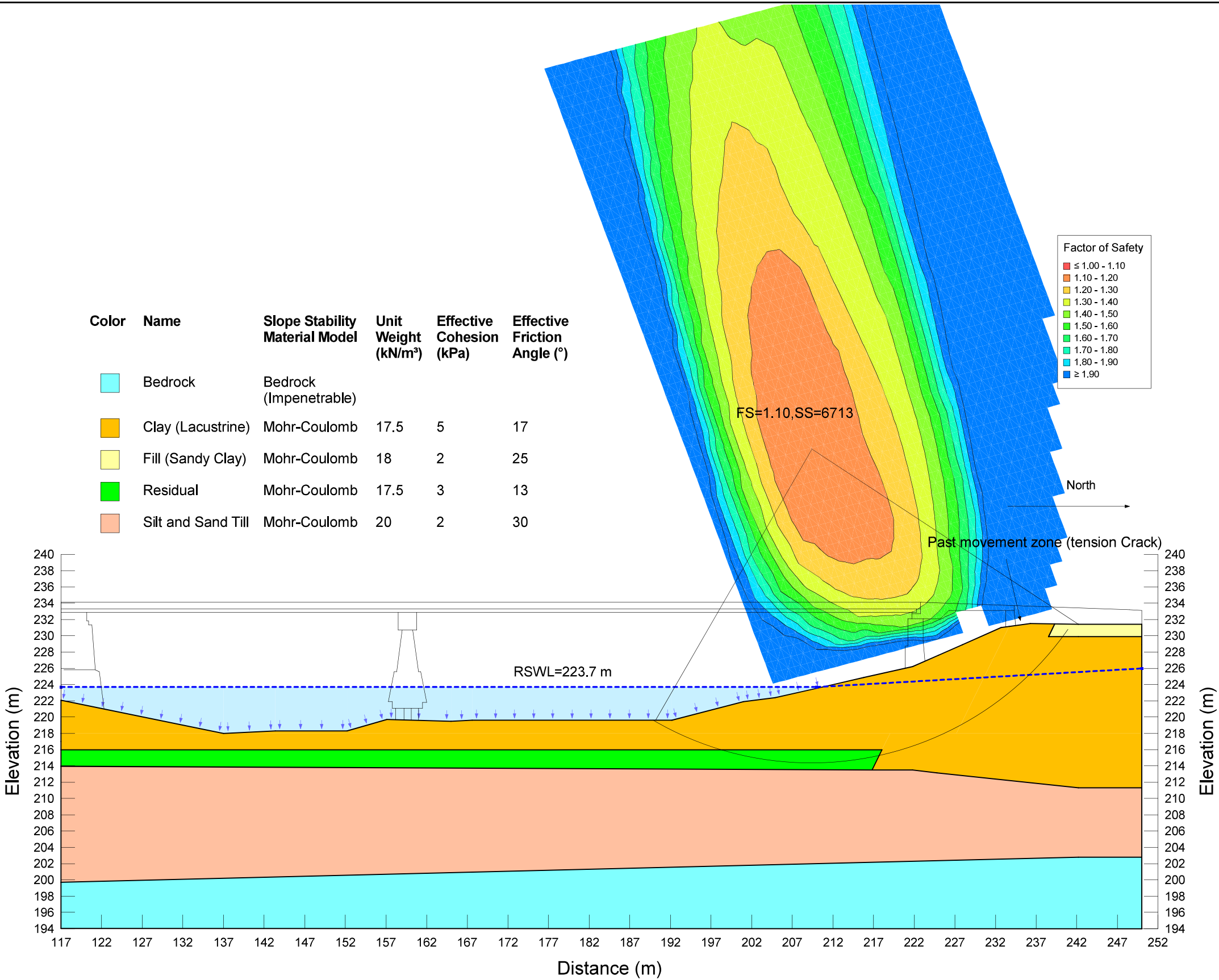


Figure E-2

Limit-Equilibrium Slope Stability Analysis
Long-Term Conditions - North Abutment - Existing Site Conditions

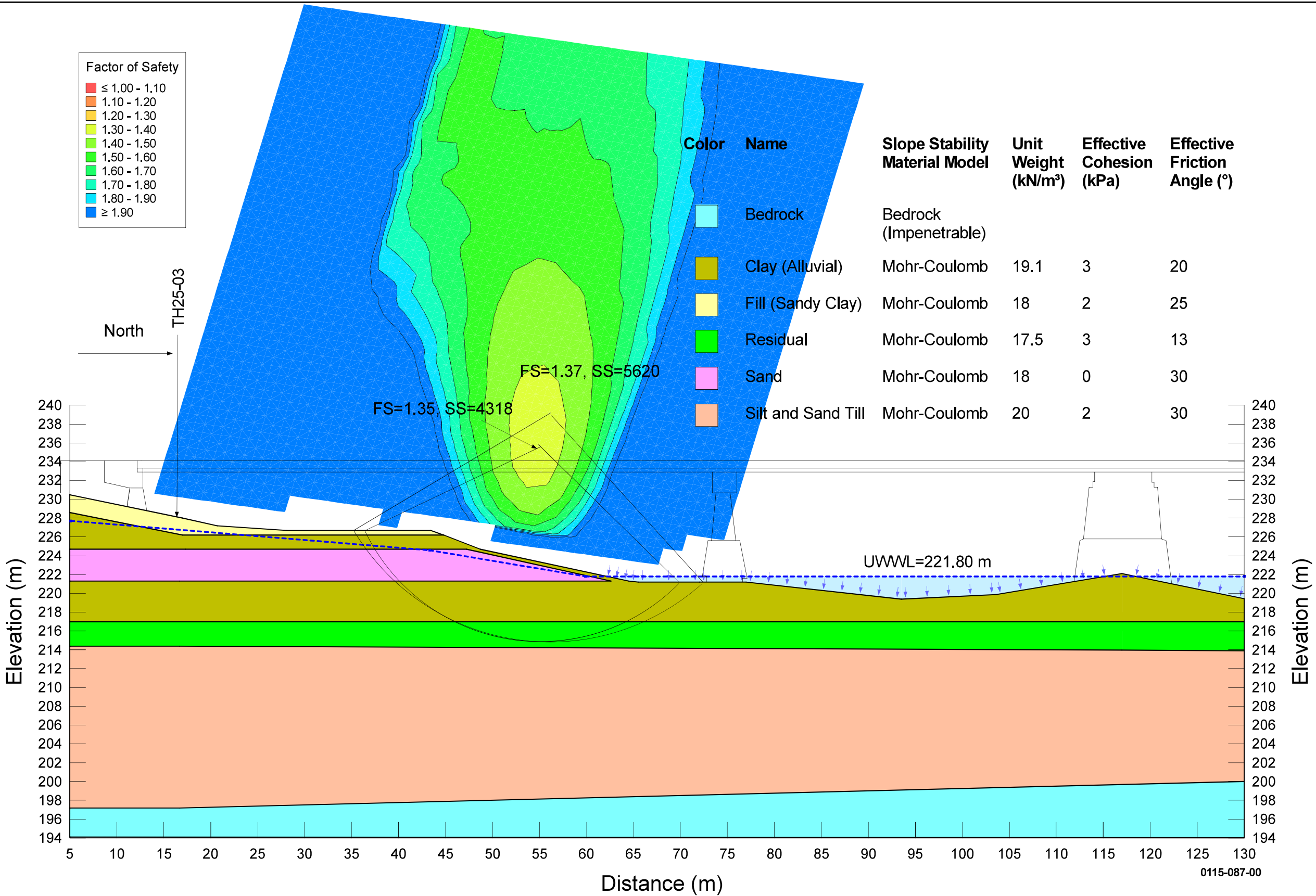


Figure E-3

Limit-Equilibrium Slope Stability Analysis
Short-Term Conditions - South Abutment - Existing Site Conditions

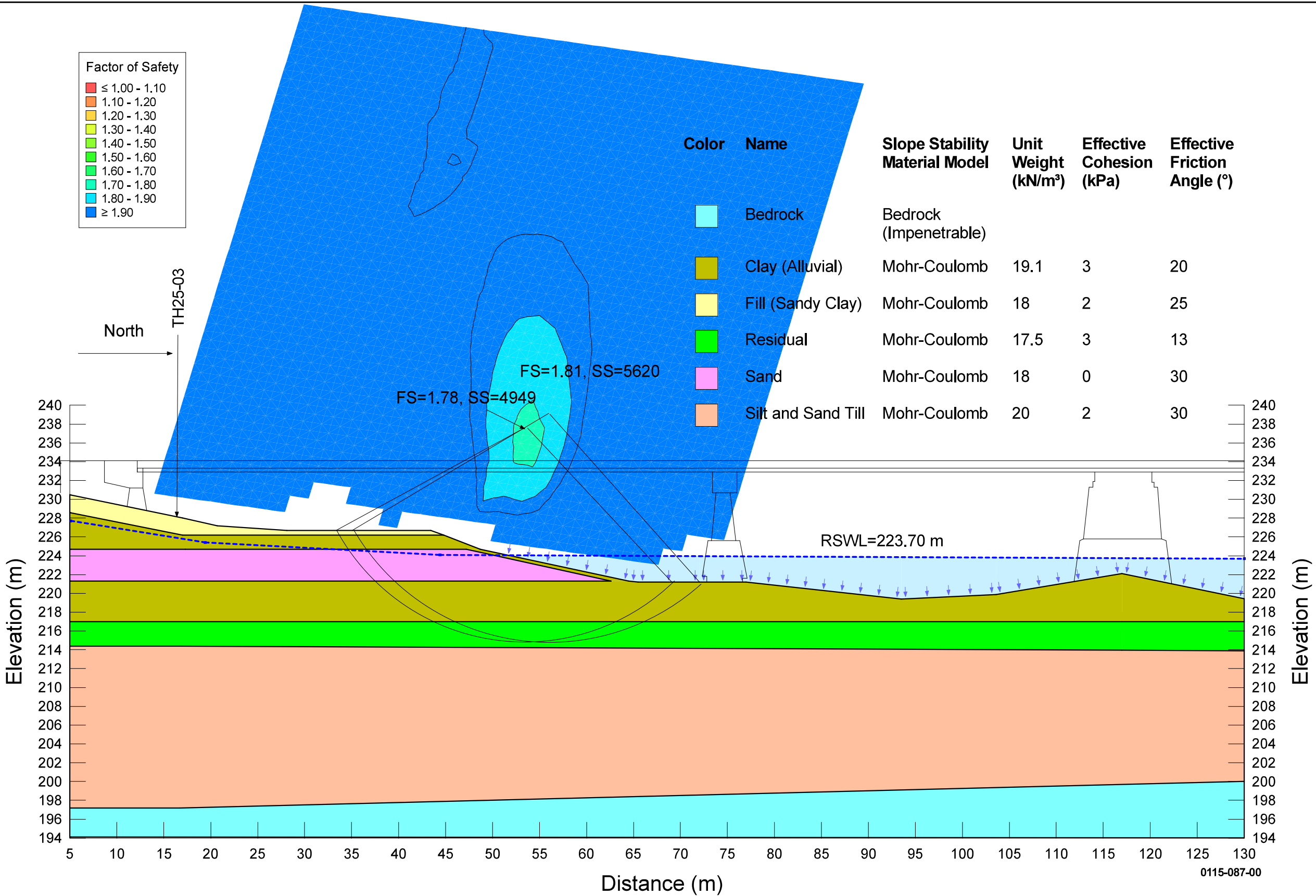


Figure E-4

Limit-Equilibrium Slope Stability Analysis
Long-Term Conditions - South Abutment - Existing Site Conditions

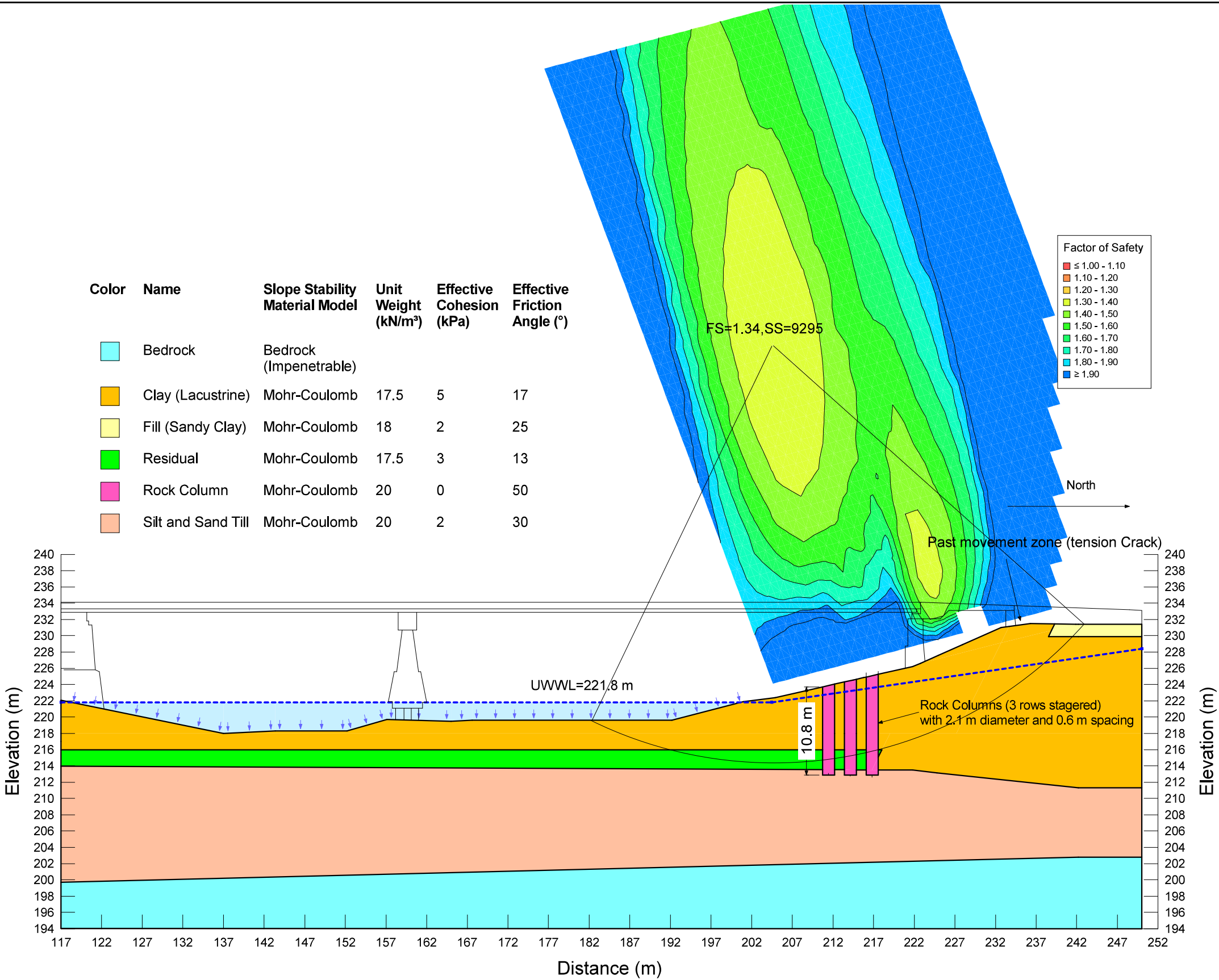


Figure E-5

Limit-Equilibrium Slope Stability Analysis
Short-Term Conditions - North Abutment - Rock Columns (3 Rows)

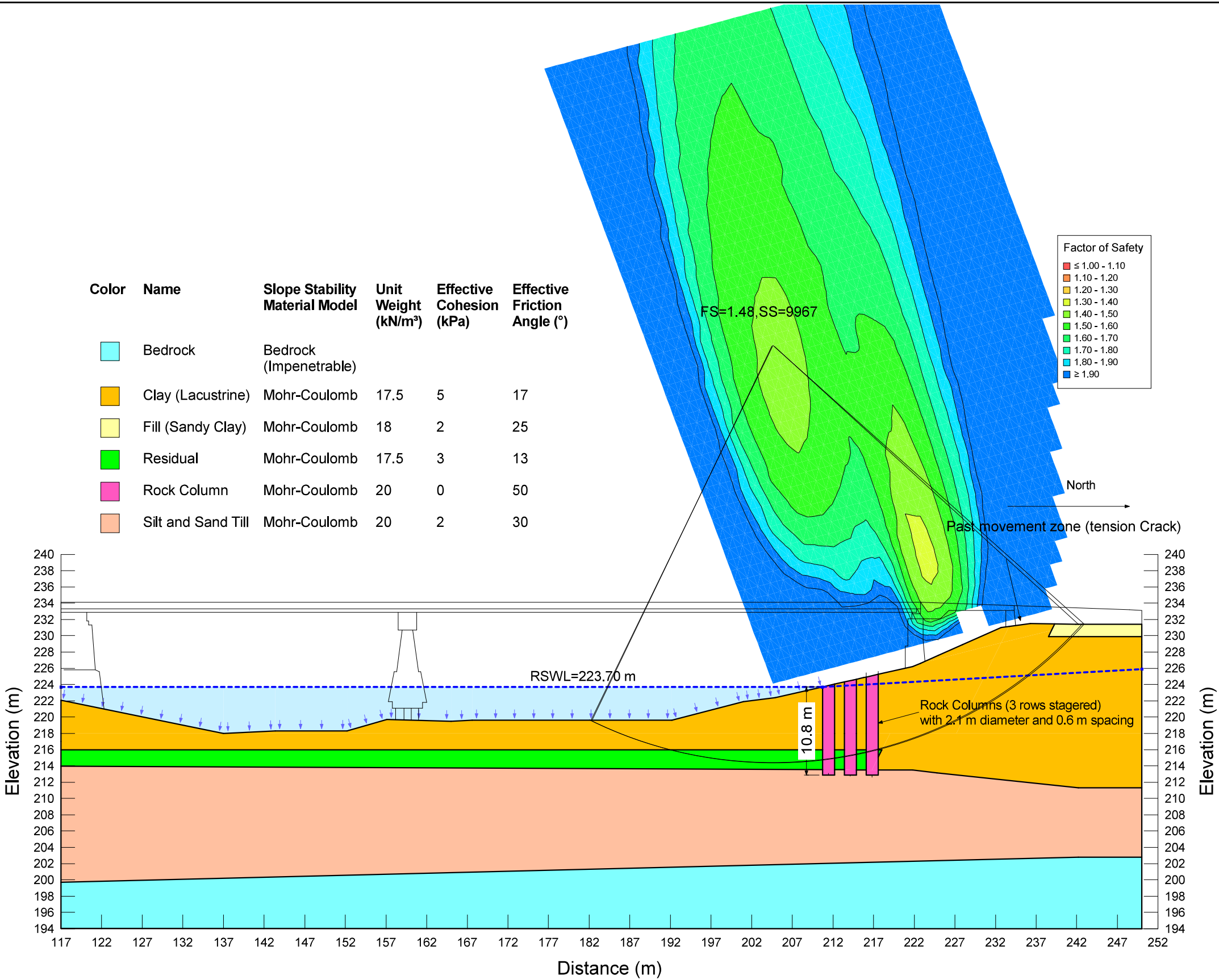


Figure E-6

Limit-Equilibrium Slope Stability Analysis
Long-Term Conditions - North Abutment - Rock Columns (3 Rows)