TREK GEOTECHNICAL

Quality Engineering | Valued Relationships

Falcon River Diversion and Shoal Lake Aqueduct Bridges – Geotechnical Report

Prepared for:

Dillon Consulting Ltd.
1558 Wilson Place
Winnipeg, Manitoba
R3T 0Y4

Distribution:

Mr. Graeme Loeppky, P. Eng.

Project Number:
0022 005 01

Date:
March 25, 2013

www.trekgeotechnical.ca
1712 St. James Street | Winnipeg, Manitoba R3H 0L3 | Tel 1.204.975.9433 | Fax 1.204.975.9435

TREK GEOTECHNICAL Quality Engineering | Valued Relationships

March 26, 2013 Our File No. 0022 005 01

Mr. Graeme Loeppky, P.Eng.
1558 Wilson Place
Winnipeg, Manitoba
R3T 0Y4

RE: Falcon River Diversion and Shoal Lake Aqueduct Bridges – Geotechnical Report

TREK Geotechnical Inc. is pleased to submit our Final Report for the geotechnical investigation for the Falcon River Diversion and Shoal Lake Aqueduct (Mile 93) bridge crossings.

Please contact Ken Skaftfeld if you have any questions.

Sincerely,

TREK Geotechnical Inc.
Per:

Ken Skaftfeld FOR KEN SKAFTFELD
Senior Geotechnical Engineer
Tel: 204.975.9433

KS:bh
Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	BH	March 26, 2013	Final Report

Authorization Signatures

Prepared By:

Brent Hay, P.Eng.
Geotechnical Engineer

PROVINCE OF MANITOBA
K. M.
SKAFTFELD
March 26/13
REGISTERED ENGINEER

Reviewed By:

Ken Skaftfeld
Senior Geotechnical Engineer

APEGM
Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: March 26/13

Table of Contents

List of Drawings

List of Appendices

1.0 Introduction and Background

TREK Geotechnical Inc. (TREK) was retained by Dillon Consulting Ltd. (Dillon) to complete a geotechnical investigation and provide recommendations related to the preliminary design of proposed bridge crossings at Mile 93 along the Shoal Lake Aqueduct and at the Falcon River Diversion. The terms of reference for the work are included in our proposal addressed to Mr. Graeme Loeppky, P.Eng., dated February 13, 2012.

The crossing locations are based on the proximity of the winter road under construction and the location of a reinforced section of the Aqueduct at a road allowance at Mile 93 as shown on Figure 01. The winter road presently crossing the Aqueduct at Mile 91.69 will be abandoned once the new bridge at Mile 93 has been completed. It is our understanding that the Falcon River Diversion crossing, originally planned to coincide with the all season road constructed on the Shoal Lake Band No. 40 will be shifted approximately 100 m southeast to allow for the horizontal curve necessary on the east side of the diversion channel.

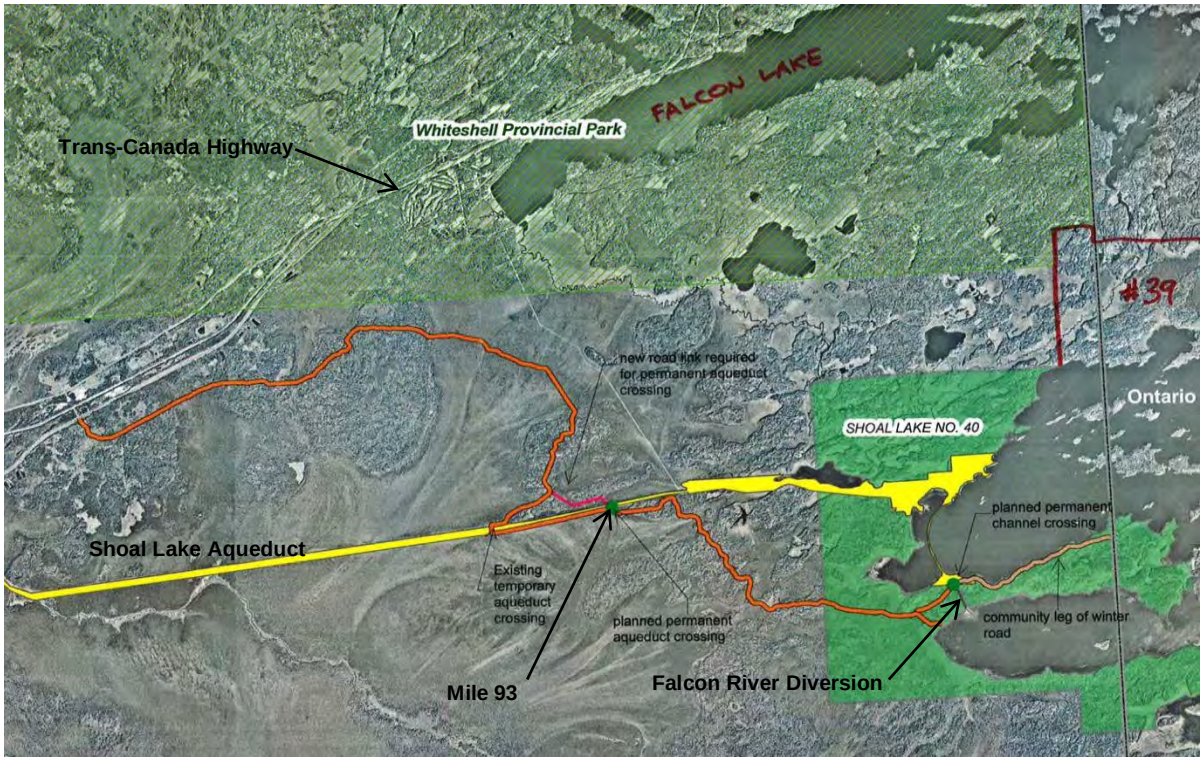


Figure 01 Proposed Bridge Crossings

The proposed bridges consist of prefabricated steel box truss ACROW panel structures, 33.5 m in length. Concrete abutments (on piles) will provide support at either end of the bridge deck. Approach embankments will be required to provide the necessary bridge deck clearance for hydraulic and navigation considerations (Falcon River Diversion) and necessary vertical and horizontal alignments accommodate proposed road alignments and Greater Winnipeg Water District (GWWD) Railway grades (Mile 93). As part of our assignment, TREK evaluated the stability of the slopes at the Mile 93

Aqueduct crossing and Falcon River Diversion crossing based on the proposed bridge configuration. Stress-deformation analysis was also carried out at the Mile 93 crossing to quantify any associated stress changes and/or settlement at the Aqueduct structure.

A site visit was completed by Mr. Ken Skaftfeld of TREK Geotechnical in the accompaniment of Mr. Graeme Loeppky and Mr. Mark Doucet of Dillon Consulting and Mr. Richard Song of the City of Winnipeg on February 17, 2012. Photographs from the site visit, along with a photograph index are included in Appendix B. Select photos are referenced throughout this report.

2.0 Existing Information

Available information pertinent to the geotechnical investigation and preliminary design was reviewed and includes the following:

Assessment and Rehabilitation of the Shoal Lake Aqueduct – Work Area 2.9 Falcon River Diversion (UMA Engineering Ltd., April 2000). This report provides an assessment of the site conditions and survey information of several cross sections of the Falcon River Diversion Area.

Assessment and Rehabilitation of the Shoal Lake Aqueduct – Buoyancy Assessment Program Geotechnical Investigation Mile 84 to Mile 95 (UMA Engineering Ltd., January 2000). Includes survey and subsurface information along the Aqueduct channel from Mile 84 to 95.

Borehole Logs from Mile 92. 1992 (UMA Engineering, 1994).

Draft Report to City of Winnipeg: Shoal lake Aqueduct – Winter Road Crossing Near Mile 93 (AECOM, May 2010).

3.0 Subsurface Investigation

The drilling program was originally planned for March 2012 when access to both crossing locations was possible via the winter road from the Trans-Canada Highway. The drill and support equipment was loaded onto a flat car at East Braintree (Mile 77.6) and transported to the Mile 93 site by the GWWD Railway. Drilling was carried out as planned at Mile 93, however, warm weather prevented drill rig access from Mile 93 along the winter road to the Falcon River Diversion crossing site. Drilling at the Falcon River Diversion was therefore deferred to the fall of 2012 when a barge was used to transport the drill rig to Shoal Lake No.40 and then along the existing all weather road to the east side of the crossing location.

Test hole (TH) 12-01 was drilled on March 27, 2012 at Mile 93 using a track mounted drill operated by Paddock Drilling Ltd. The hole was located on the south side of the Aqueduct ditch. Access to the north side of the Aqueduct was not possible. Test hole TH12-02 was drilled on October 24, 2012 using a track mounted drill rig operated by Maple Leaf Drilling Ltd. The test hole was located on the east side of the Falcon River Diversion channel at the end of the road from the Shoal Lake No. 40 community. TH 12-03 was completed on the west side of the diversion channel by TREK personnel using a 50 mm hand auger.

TH 12-01 and 02 were drilled with 125 mm diameter solid stem augers to refusal and completed with NQ coring into bedrock. Drilling was completed under the supervision of TREK personnel and test holes were visually logged based on the Unified Soil Classification System (USCS). Disturbed (auger cutting), relatively undisturbed (Shelby tube) and bedrock core samples were recovered during the drilling. Test holes TH12-01 and 12-02 were advanced into the underlying bedrock approximately 3.0 m. TH12-03 was advanced 3.0 m below ground to evaluate near surface conditions. TH 12-01 was surveyed by Dillon in October 2012 and the ground elevation was determined based on relative elevations from the top of a standpipe piezometer installed in 1994 at Mile 92.992. Approximate locations for THs 12-02 and 12-03 were determined using a hand held GPS. Ground elevations at THs 12-02 and 12-03 were determined based on a relative channel water level as surveyed on October 12, 2012. Borehole logs for the test holes are attached in Appendix A.

Soil samples were transported to TREK's soils laboratory in Winnipeg for further classification and testing. Testing included the determination of moisture content, Atterberg limits, grain size (hydrometer method), unit weight, undrained shear strengths and rock quality density (RQD) on select samples. The results from the laboratory testing are shown on the individual test hole logs in Appendix A.

4.0 Mile 93 – Shoal Lake Aqueduct

4.1 Site Conditions

The proposed bridge crossing is located at Mile 93.03 of the City of Winnipeg's Shoal Lake Aqueduct (SLA), in an undeveloped area south of the Trans-Canada Highway. A location plan of the crossing is shown on Drawing 01 along with the existing ground profile. The crossing is at an existing road allowance where the Aqueduct structure has been reinforced. The SLA traverses flat and poorly drained organic terrain bounded by a tamarack forest to the north and the GWWD Railway to the south (Figure 02). The Aqueduct was constructed using a cut and cover technique resulting in a ditch on either side of the structure and spoil material on the north side of the north ditch. The ditches are water-filled year round and the top of the berm above the Aqueduct is partially exposed.

The ground elevation on the north and south sides of the Aqueduct are at approximately Elev. 325.5 and 324.5 m respectively with the higher north side elevation resulting in part from spoil material (Figures 03 and 04). The ditch invert is at approximately Elev. 322.5 m and 2 to 3 m wide. The berm above the Aqueduct slopes at about 2H:1V and cresting at about Elev. 323.5 m. A cross section of the proposed crossing at Mile 93 is shown on Drawing 02.

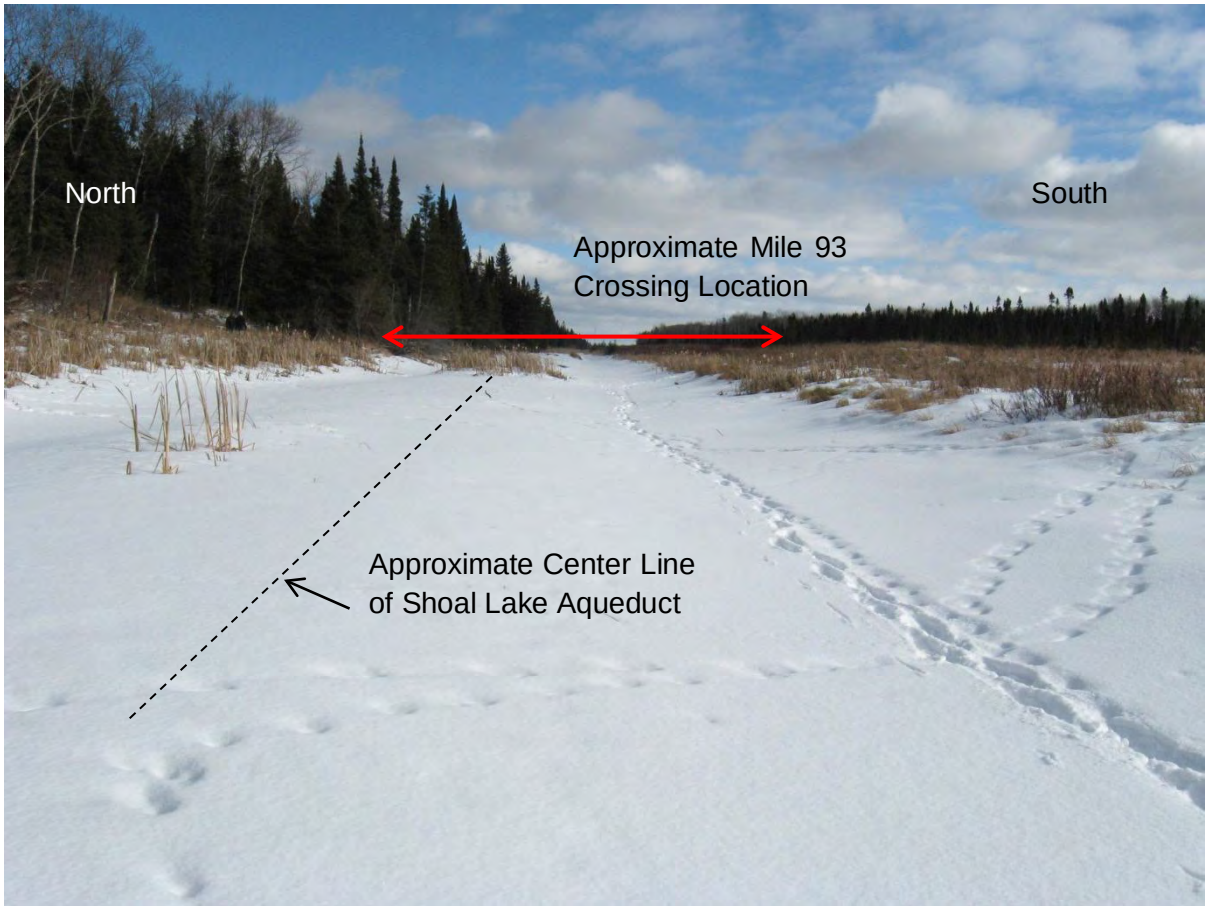


Figure 02 View East Along Aqueduct ROW Taken From WP 341 at From TH5 at Mile 92.992

Figure 03 View N at N Abutment Location

Figure 04 View SW at S Abutment Location

4.2 Soil and Groundwater Conditions

4.2.1 ***Soil Profile***

The soil profile at the Mile 93 crossing generally consists of peat overlying low to high plasticity alluvial silts and clays overlying highly plastic lacustrine clay, till and bedrock. A brief description of the soil units encountered at the test hole locations is provided below. All interpretations of soil stratigraphy for the purposes of design should refer to the detailed test hole logs in Appendix A. A cross section at Mile 93 showing the soil profile from TH 12-01 is shown on Drawing 01.

Peat

A 0.5 m thick peat layer was encountered at surface in TH12-01. The peat is fibrous, fine, dark brown and wet. The peat is an H3 in degree of humification based on the Von Post peat classification system. The moisture content of one sample of the peat was 342 %.

Alluvial Silts and Clays

Interlaid alluvial silts and clays were encountered below the peat in TH12-01 to 7.6 m below ground surface (bgs). The alluvial soils contain trace sand, are light brown to grey and of low to intermediate plasticity. Moisture contents range from 19 to 41 % with an average of 26%. Bulk unit weights range from 17.3 to 22.0 kN/m^3 with an average of 20.4 kN/m^3. Based on unconfined compression tests, undrained shear strengths range from 11 to 53 kPa with an average of 36 kPa. The plastic limits from two samples of the clay are both 13 % with liquid limits of 23% and 40 %.

Lacustrine Clay

Lacustrine clay was found underlying the alluvial soils to a depth of 24.4 m bgs. The clay is silty, contains trace gravel, is grey, moist and of high plasticity. Moisture contents range from 23 to 35% with an average of 32%. Bulk unit weights range from 18.6 to 19.2 kN/m^3 with an average of 19.0 kN/m^3. Based on unconfined compression tests, undrained shear strengths range from 28 to 51 kPa with an average of 42 kPa. The plastic and liquid limits from one sample of the clay are 15 and 51 % respectively.

Sand

A sand layer was found underlying the lacustrine clay to from 24.4 to 26.8 m bgs. The moisture content of one sample from the sand and gravel was 14 %. Within the sand layer, a cobble, approximately 150 mm in diameter, was cored through at 26.4 m bgs.

Silt (Till)

Silt (till) was encountered below the sand from 26.8 to 28.3 m bgs. The silt (till) is dense and contains some sand, some gravel, and is grey. The moisture content of one sample was 10%.

Bedrock

Bedrock was encountered at 28.3 m bgs (Elev. 296.7). The drilling was advanced 3.8 m into the bedrock. The bedrock is amphibolite, greenish grey in color, strong to very strong (R4 to R5) and homogenous. The upper 1.0 m of the bedrock is strongly fractured and weathered with a rock quality designation (RQD) of 55 %. The lower 2.8 m of bedrock is intact with an average RQD of 97 %.

4.2.2 *Groundwater Conditions*

A groundwater level of 0.9 m bgs was measured immediately after drilling TH 12-01 on the south side of the crossing. Sloughing was observed at 3.1 m bgs during drilling. It is important to recognize that the measured groundwater levels should be considered short-term and may vary seasonally, after heavy precipitation events or as a result of construction activities. Seepage from the near surface organic soils can also be expected. Groundwater levels on the north side of the Aqueduct may be different than observed on the south side at TH 12-01 and should be confirmed prior to detailed design (Section 8.0).

4.3 Slope Stability Analysis

Slope stability analysis was completed for the proposed Mile 93 bridge geometry provided by Dillon. The preliminary assumptions included an earth fill approach embankment and concrete abutments (pile supported). The stability analysis was conducted using a limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2007 software package (Geo-Slope International Inc.). Slip surfaces were specified with the grid and radius method, with factors of safety calculated using the Morgenstern-Price method of slices. Groundwater conditions were modelled using piezometric lines.

4.3.1 *Model Geometry*

The model geometry is based upon the topographic survey information collected by Dillon on October 12, 2012 supplemented with ditch inverts from ice auger soundings carried out during the initial site reconnaissance. The water level in the ditch of the SLA crossing is based on the top of ice level obtained in the Dillon October 12, 2012 survey. The cross section is taken just outside of the abutment where the fill is at a maximum height. The preferred layout has the middle of the bridge shifted to the south of the Aqueduct centerline and as a result, the north abutment is about 6 m closer to the Aqueduct than the south abutment.

4.3.2 *Soil Properties*

The soil parameters used in the slope stability analysis are based on the field and laboratory testing, the results of hand auger test holes in the backfill for the Aqueduct from previous studies (AECOM, 2010) and typical values for the nature of soils encountered. It was assumed that soil conditions are the same on the north side of the Aqueduct (in the vicinity of the north abutment) as determined on the south side during the sub surface investigation, in particular the near-surface soil unit (alluvial silt and clay). Table 4.1 presents the engineering properties for the soil units used in the analysis.

Table 4.1 – Soil Unit Engineering Properties (Mile 93)

Soil Unit	Location	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle (degrees)
Peat	Aqueduct Backfill	14.0	5	14
Alluvial Silts and Clays	South Bank	20.5	2	20
Lacustrine Silty Clay	South Bank	19.0	5	17
Clay Fill	Aqueduct Backfill	18.5	5	17
Embankment Fill	Approach Fill For Proposed Bridge	21.0	0	40
Silt Till	South Bank	20.0	0	45

4.3.3 *Groundwater Conditions*

In the vicinity of the proposed abutments, groundwater levels were assumed to be approximately at the base of the embankment fill, sloping towards the surveyed ice level in the ditches. Although this ground water level is higher than measured during drilling, it is considered representative of potential ground saturation due to seasonal changes and environmental effects.

4.3.4 ***Modelling Results***

The factors of safety (FS) for potential slip surfaces (PSS) through the approach fill immediately adjacent to the abutment on both sides of the Aqueduct were determined for the original proposed bridge geometry. Any structural support provided by the piles and/or abutment was neglected in the analysis. Three key slip surfaces were examined, the slip surface with the minimum FS at the crossing (critical) which could negatively impact the bridge abutment, a slip surface that extends to the top of the Aqueduct, and a slips surface that extends below the Aqueduct. The latter two are considered potential slip surfaces that could impact the integrity of the Aqueduct.

The potential slip surfaces are identified in the analysis as PSS-1 (critical), PSS-2 (above the Aqueduct) and PSS-3 (beneath the Aqueduct). To differentiate between the north and south sides of the crossing, a notation of N or S has been added to the description e.g. PSS-N1. A minimum FS of 1.5 was targeted for the critical slip surface. As a consequence, it can be expected that FS for theoretical slip surfaces extending to the Aqueduct will be higher than 1.5.

Modelling of the originally proposed bridge geometry resulted in an estimated FS for the critical slip surfaces on the north and south sides of the crossing of 1.41 and 1.45 respectively. The following modifications were then incorporated into the model to achieve the target FS:

- Increase the depth of granular fill around the abutments to improve soil strength and reduce groundwater levels in the vicinity of the abutment;
- Construct wing walls behind the abutments to offset fill loading away from the top of riverbank. This was accomplished by analyzing wing wall lengths which are considered practical of 3, 4 and 5 m.

Table 4.2 presents the results of the modelling with the proposed modifications and with a 4 m long wing wall on the north side of the crossing. The locations of the PSS in relation to the proposed bridge and the Aqueduct are shown on Drawing 02.

Table 4.2 – Factor of Safety with Stability Improvements

Slip Surface	PSS-1 (Critical)		PSS-2		PSS-3	
Location	North Side PSS-N1	South Side PSS-S1	North Side PSS-N2	South Side PSS-S2	North Side PSS-N3	South Side PSS-S3
Factor of Safety	1.50	1.51	1.61	1.59	1.66	1.59

To account for potential variability in soil conditions on the north side of the Aqueduct, further analysis was carried out assuming clay (rather than silt) in the upper soil horizon (top 7.5 m). This assumption lowers the FS below the target of 1.5 by about 10%. This result reinforces the need to confirm near surface soil and groundwater conditions at the north abutment prior to detailed design (Section 8.0).

4.4 Stress and Settlement Analysis

A stress-deformation analysis was completed to evaluate the stresses that may be imposed on the Aqueduct structure and associated settlements as a result of bridge construction. The cross section geometry used in the analysis was taken through the centre of the approach fill on both the north and south sides. The stress analysis was completed using a stress-deformation finite element model (Sigma/W) from the GeoStudio 2007 software package (Geo-Slope International Inc.). Deformations were modelled using linear elastic constitutive soil models. Soil properties used in the analysis were based off measured values or were assumed based on typical values used for similar soil types. Table 4.3 presents the representative soil units and the parameters used in the stress-deformation analysis.

Table 4.3 – Soil Properties for Shoal Lake Aqueduct Stress-Deformation Analysis

Soil Unit	Unit Weight (kN/m^3)	Young`s Modulus (kPa)	Poisson`s Ratio
Peat	14	200	0.4
Alluvial Silts and Clays	20.5	15,000	0.4
Lacustrine Silty Clay	19	5,000	0.4
Clay Fill	19	5,000	0.4
Embankment Fill	21	40,000	0.3
Silt Till	19	100,000	0.3

4.4.1 ***Stress Analysis***

The Aqueduct structure was modelled both as a rigid member (no displacement allowed) and as a free moving member. The model assumes 1.2 m of clay backfill at the Aqueduct base with peat backfill to surface based on previous investigations at Mile 92.99 (UMA, 1994). The estimated increase in stress in both the horizontal (x-direction) and vertical (y-direction) direction were then determined at various locations (nodes) along the outside surface of the structure as shown on Figures 05 and summarized in Table 4.4. Detailed model outputs are included in Appendix C.

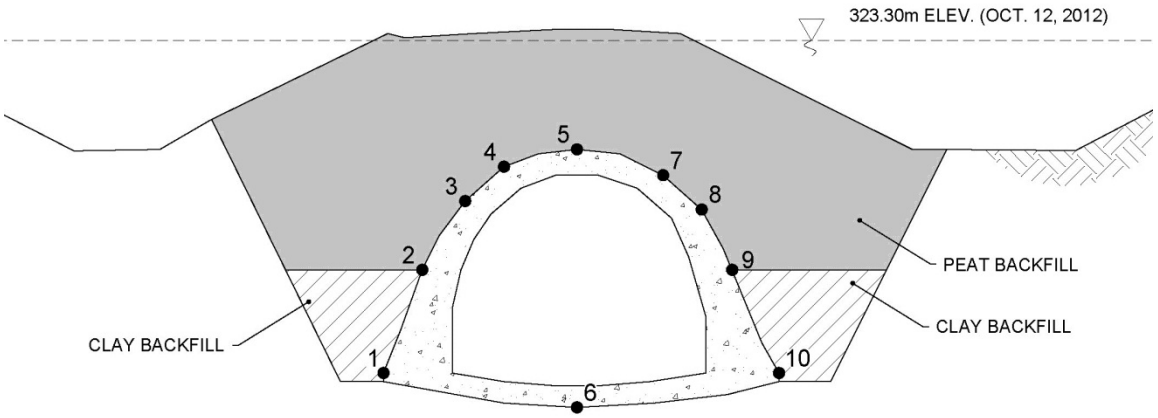


Figure 05 Nodes on Aqueduct For Stress Change Determination

Table 4.4 – Results of Stress Analysis

Node	Aqueduct Boundary Condition			
	No Displacement Allowed		Free Moving	
	Change in X-Effective Stress (kPa)	Change in Y-Effective Stress (kPa)	Change in X-Effective Stress (kPa)	Change in Y-Effective Stress (kPa)
1	9	3	1	<1
2	2	<1	<1	<1
3	<1	<1	<1	<1
4	<1	<1	<1	<1
5	<1	<1	<1	<1
6	<1	<1	7	<1
7	<1	<1	<1	<1
8	<1	<1	<1	<1
9	<1	<1	<1	<1
10	2	1	1	<1

From Figure 05, the maximum stresses in the horizontal and vertical directions are 9 and 3 kPa respectively. For comparison, maximum stress changes in the order of 23 (horizontal) and 14 kPa (vertical) were estimated by TREK in 2010 as part of a conceptual evaluation of a bridge crossing carried out for AECOM in May 2010. The 2010 modelling however, assumed a shorter bridge span with approach fills in closer proximity to the Aqueduct. In both the 2010 and 2013 modelling, the maximum horizontal and vertical stress increases occur at the outside edges of the base of the structure (invert). If the estimated stresses are greater than what can be tolerated by the structure, a more rigorous analysis should be carried out during detailed design. Additionally, options to reduce the loading from proposed fills, such as lightweight fill or increasing the setback distance of the abutments could be investigated.

4.4.2 ***Settlement Analysis***

Consolidation settlement of the soils beneath the approach fills can be expected although it will take a number of years for the settlement to occur due the fine grained nature of the soils on site. The largest settlement magnitudes will be immediately beneath the maximum fill heights and will dissipate with increasing distance away from the fill. Settlement of the approach fills can likely be accommodated in the bridge design, however, any associated settlement of the soil beneath the Aqueduct must be within an acceptable range for the structure. In this regard, a 1-dimensional (1-D) analysis was carried out to predict consolidation settlements under the north and south abutments and under the centre of the Aqueduct using the maximum vertical stresses estimated from the finite element model. The results of the analysis are summarized in Table 4.5.

Table 4.5 – Estimated Settlements at Aqueduct and Abutment

Location	Estimated Settlement (mm)
Under Aqueduct	10
North Abutment	66
South Abutment	51

In the event that this magnitudes of settlement at the abutment locations cannot be accommodated by regular maintenance (e.g. asphalt overlays at the bridge approaches) techniques to accelerate consolidation settlement such as preloading or the installation of vertical drains may be considered. If the estimated settlements of the Aqueduct are greater than what can be tolerated by the structure, options to reduce the loading from proposed fills, such as lightweight fill or increasing the setback distance of the abutments should be investigated.

5.0 Falcon River Diversion

5.1 Site Conditions

The proposed bridge will cross the Falcon River Diversion (FRD) channel approximately 3.5 km west of the community at Shoal Lake No. 40. A location plan of the crossing is shown on Drawing 03. The Falcon River Diversion channel was excavated across the peninsula separating Indian Bay and Snowshoe Bay during construction of the Shoal lake Aqueduct between 1914 and 1915 to divert Falcon River water away from the Intake. The originally proposed channel crossing is situated in a relatively flat, heavily treed area immediately northwest of a horizontal bend in the channel where a large area on the west side of the channel has been cleared at the termination of a recently constructed winter road (Figure 06). On the east side, a road connects the Community to the channel crossing location (Figure 07). It is our understanding that the proposed crossing is to be relocated about 100 m farther southeast, along a straight portion of the channel; however, our investigation was carried out at the originally proposed location.

The surveyed water level of the channel was Elev. 322.22 m on October 12, 2012. Historical records obtained from Dillon show that the lowest water level to date in the FRD channel was Elev. 321.87 m recorded on March 28, 1930. Between 2000 and 2011, the lowest water Elev. was 322.26 m. At ice level, the channel is about 17 m wide with banks in the order of 2 m high. Spoil material from the original excavation has been piled on the tops of both banks ranging in height from about 2 m on the west side to about 1.5 m on the east bank. The channel spoil banks show evidence of undercutting although no instabilities of the banks were visible. A cross section and site plan of the existing ground profile is presented on Drawing 03. A cross section of the proposed crossing at the Falcon River Diversion is shown on Drawing 04.

Figure 06 View NW at W Side of Crossing (IMG_4399&4400)

Figure 07 View NE at E Side of Crossing (IMG_4414)

5.2 Soil and Groundwater Conditions

5.2.1 ***Soil Profile***

5.2.1.1 Alluvial Silt

Alluvial silt was encountered at surface in TH12-02 to 4.6 m bgs and to the end of hole in TH12-03. The silt is clayey, contains trace fine sand and trace gravel, is light brown, moist and of intermediate plasticity. Based on measured undrained shear strengths the silt is firm to stiff with a trend of decreasing shear strength with depth. Moisture contents range from 17 % to 42 % with an average of 25 %. Average bulk unit weights are 19.6 kN/m^3. Based on unconfined compression tests, undrained shear strengths range from 34 to 52 kPa with an average of 46 kPa.

Lacustrine Clay

Lacustrine clay was found underlying the silt to a depth of 12.8 m below surface. The clay is silty, contains trace fine sand and trace gravel, is grey, moist, soft to firm and of intermediate to high plasticity. Moisture contents range from 27 % to 69 % with an average of 41 %. Bulk unit weights range from 15.3 to 18.8 kN/m^3 with an average of 17.5 kN/m^3. Based on unconfined compression tests, undrained shear strengths range from 18 to 46 kPa with an average of 32 kPa with a trend of decreasing shear strength with depth. The plastic and liquid limits from one sample of the clay were 12 % and 41 %, respectively.

Silt Till

Silt (till) was encountered below the clay to 15.1 m below surface. The silt (till) is clayey, contains trace sand, trace gravel, is grey, moist, soft and of intermediate plasticity. The moisture content of one sample from the silt (till) was 27 %. Sample recovery from the lower portion of the silt till was not possible due to the drilling method (NQ coring).

Bedrock

Bedrock was encountered at 15.1 m below surface (Elev. 309.0). The drilling was advanced 2.6 m into the bedrock. The bedrock is amphibolite, greenish grey in color, strong to very strong (R4 to R5) and homogenous. The bedrock is intact with an average RQD of 95 %.

5.2.2 *Groundwater Conditions*

The groundwater level was at 2.9 m after completion of drilling in TH12-02, respectively. No seepage was observed in TH12-03. Minor sloughing was observed in the silt (till) in TH12-02 but the hole remained open to 14.5 m bgs after completion. It is important to recognize that the measured groundwater levels should be considered short-term and may vary seasonally, after heavy precipitation events or as a result of construction activities. Groundwater levels on the north side of the channel may also vary and should be confirmed prior to detailed design (Section 8.0).

5.3 Slope Stability Analysis

Slope stability analysis was completed for the proposed bridge geometry provided by Dillon. The preliminary assumptions included an earth fill approach embankment and concrete abutments (pile supported). The stability analysis was conducted using a limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2007 software package (Geo-Slope International Inc.). Slip surfaces were specified with the grid and radius method, with factors of safety calculated using the Morgenstern-Price method of slices. Groundwater conditions were modelled using piezometric lines.

5.3.1 *Model Geometry*

The model geometry is based upon the topographic survey information collected by Dillon on October 12, 2012. The cross section is taken just outside of the abutment where the fill is at a maximum height. The water level in the channel is based on the top of ice level obtained in the Dillon October 12, 2012 survey. The preferred layout has the 33.4 m long bridge centered on the channel with east and west abutments set back about 3 m from the existing top of bank.

5.3.2 ***Soil Properties***

The soil parameters used in the analysis are based on the field and laboratory testing and the nature of the soils encountered. It was assumed that soil conditions are the same on the west side of the channel as determined on the east side during the sub surface investigation, in particular the near-surface soil unit (alluvial silt and clay). A friction angle of 22^{o} was assumed based on the appreciable silt and sand content. Table 5.1 presents the representative soil units and the strength properties used in the stability analysis.

Table 5.1 – Soil Properties for Falcon River Diversion Stability Analysis

Soil Unit	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle (deg)
Alluvial Silt	19.5	2	22
Lacustrine Silty Clay	18.0	5	17
Embankment Fill	21.0	0	40
Silt Till	20.0	0	45
Rip Rap	21.0	0	45

5.3.3 ***Groundwater Conditions***

In the vicinity of the proposed abutments, groundwater levels were assumed to be approximately at the base of the embankment fill, sloping towards the surveyed water level in the channel. These levels are higher than those observed during drilling however they reflect the potential for the ground to be saturated. Groundwater levels on both sides of the channel at the new crossing location should be verified prior to detailed design (Section 8.0).

5.4 Modelling Results

The FS for potential slip surfaces through the approach fill immediately adjacent to the abutment on both sides of the channel were determined. The critical slip surface is representative of one that potentially could affect the bridge abutments which is also the slip surface with the minimum FS for the cross-section analyzed. These potential slip surfaces (PSS) have been identified in the analysis as PSS-W1 and PSS-E1 representing the west and east abutments. A minimum FS of 1.5 was targeted for the critical slip surface. Modelling of the originally proposed bridge geometry resulted in calculated FS for the critical slip surfaces on the west and east sides of the channel of 1.22 and 1.24 respectively. The following modifications were then incorporated into the model to achieve the target FS of 1.5:

- Increase the depth of granular fill around the abutments to improve soil strength and lower groundwater levels in the vicinity of the abutments;
- Construct wing walls behind the abutments to offset fill loading away from the top of riverbank. In this regard, wing walls wall lengths of 3, 4 and 5 m were considered practical.
- Extend the proposed rip rap and adjust the thickness of the blanket for additional toe support and scour protection.

Table 5.2 presents the results of the modelling with the proposed modifications and with a 4 m long wing wall on both sides of the channel. The locations of the PSS in relation to the proposed bridge are shown on Drawing 04. The concept for Drawing 04 is based on the information at the original channel crossing location. Once a final crossing location has been determined, the design should be optimized based on site specific geometry, and soil and groundwater conditions. For example, it may be possible to reduce the rip rap blanket thickness by incorporating a 5 m long (rather than 4 m) wing wall.

Table 5.2 – Factor of Safety with Stability Improvements

Slip Surface	Critical	
Location	West Side PSS-W1	East Side PSS-E1
Factor of Safety	1.50	1.50

6.0 Foundation Recommendations

The soil conditions encountered at both crossing locations make cast-in-place concrete friction piles and driven steel piles end bearing on the bedrock viable foundation options. If cast-in-place concrete friction piles do not provide sufficient resistance for the anticipated loads, driven steel end bearing piles should be used. Due to the sloughing and groundwater conditions encountered during drilling, it is likely that cast-in-place concrete piles end bearing in the till or bedrock are not a viable option as full length sleeving would be required to maintain an open hole.

6.1 Limit States Design

Limit state design recommendations according to the 2010 Canadian Highway and Bridge Design Code (CHBDC) are provided below. Limit States design requires consideration of distinct loading scenarios and prescribes resistance factors (reduction factors) that are based upon the method used to evaluate pile capacity.

The ultimate bearing capacity values for the soils at the site need to be factored using resistance factors as defined in the 2010 CHBDC. Table 1 indicates the resistance factors for deep foundations. The ultimate pile capacities are to be multiplied by the appropriate resistance factor in Table 6.1 to establish the Ultimate Limit State (ULS) pile capacity, which can be compared against the ULS (factored) load combinations defined for the structure. The Service Limit State (SLS) is concerned with limiting the deformation or settlement of the foundation under static loading conditions such that the integrity of the structure will not be impacted by comparing SLS (unfactored) structural loads to the SLS pile capacity.

Table 6.1 – ULS Resistance Factors for Deep Foundations

Case	Resistance Factor
Deep Foundation with member in compression using static analysis	0.4
Deep Foundation with member in compression using static load testing	0.6

6.2 Cast-in-Place Friction Piles

ULS and SLS geotechnical resistances are provided in Table 6.2 and 6.3 for cast-in-place friction piles at the SLA and FRD crossing locations, respectively. Based on the use of measured undrained shear strengths, a resistance factor of 0.4 has been applied to the nominal skin friction value to arrive at the recommended values provided in the tables. A resistance factor of 0.6 may only be used if a static pile load test is carried out at the site. The SLS pile capacities presented below are provided to limit settlement amounts to less than 25 mm.

Table 6.2 – ULS and SLS Skin Friction Values

Crossing Location	Soil	Depth (m)		ULS Skin Friction Value (kPa)	SLS Skin Friction Value (kPa)
		From	To		
Shoal Lake Aqueduct (Mile 93)	Alluvial Soils	0	2.5	0	0
	Alluvial Soils	2.5	7.5	18	15
	Silty Clay	7.5	24.5	16	13

[1] ULS skin friction value – A Resistance Factor of 0.4 is applied

Table 6.3 – ULS and SLS Skin Friction Values

Crossing Location	Soil	Depth (m)		ULS Skin Friction Value (kPa)	SLS Skin Friction Value (kPa)
		From	To		
Falcon River Diversion	Silt	0	2.5	0	0
	Silt	2.5	4.5	18	15
	Silty Clay	4.5	12.8	12	10

[1] ULS skin friction value – A Resistance Factor of 0.4 is applied

Adhesion within the upper 2.5 m of the pile should be ignored to take into consideration potential shrinkage and environmental effects such as frost action over that depth. Shaft support within any fill materials should also be ignored. A minimum pile length of 8 m below ground surface is recommended for straight shaft piles to protect against frost jacking.

Additional Design and Construction Recommendations

Additional design and construction recommendations for cast-in-place concrete piles are provided below:

1. The weight of the embedded portion of the pile may be neglected.
2. The contribution from end bearing should be ignored.
3. Based on observed conditions sleeving of pile holes may be necessary. If seepage and sloughing conditions are observed during shaft drilling the holes should be sleeved.
4. Drilling and concrete placement for the piles should be inspected by geotechnical personnel to verify the soil conditions and proper installation of the piles.
5. Prior to casting the pile, any groundwater within the shaft should be removed or controlled.
6. Pile spacing should not be less than 2.5 pile diameters, measured centre to centre.
7. Once the pile spacing, length and layout of pile groups are known, the foundation system should be evaluated to determine if pile group effects are applicable.
8. All cast-in-place piles require reinforcement design by a qualified structural engineer for the anticipated axial, lateral and bending loads from the structure.

6.3 Driven Steel Piles

Piles driven to refusal on the bedrock are considered a viable option for support of bridge abutments at the proposed Falcon River Diversion channel and Mile 93 crossings. It is anticipated that piles can be driven through the clays and tills to the underlying bedrock at each crossing location. At the Mile 93 crossing, the presence of cobbles within the sand layer above the bedrock may create some installation difficulties; there is a risk of reduced capacity resulting from shallow refusal or the need for a replacement pile(s). The ULS design criteria outlined in the CHBDC (Clause C10.22.2) present three resistance factors that should be considered when driving steel piles. Table 6.4 presents the three resistance factors.

Table 6.4 – ULS Resistance Factors for Steel Piles

Case	Resistance Factor
To account for the unintended eccentricity of the applied load about the pile axis	0.8
For the variation of material and geometric properties of the pile	0.9
To account for pile damage during driving	0.75

Due to the nature of driving steel piles to refusal on bedrock, all three resistance factors should be used for the ULS design case. The product of all three results in a resistance factor of 0.5 (rounded). At the FRD crossing location, steel piles driven to refusal on bedrock may be designed with an ULS capacity of 50% of the yield stress of the steel, multiplied by the cross sectional area of the steel. Steel piles driven to refusal on bedrock may be designed with a SLS capacity of 30% of the yield stress of the steel, multiplied by the cross sectional area of the steel.

Refusal criteria and load capacity for specific piles should be established by TREK once the pile sizes and driving method are known in order to verify that the geotechnical and structural capacity has been adequately addressed to minimize the potential for pile damage during driving. Driving should proceed under careful observation near bedrock to avoid overdriving the pile, which could lead to pile damage or misalignment.

It is common for bedrock in these areas to slope significantly. In the event that it appears that piles are sliding on bedrock during construction, misalignment and pile damage could occur. Where this occurs, driving should be discontinued to avoid further misalignment of the pile, and an assessment made of the pile capacity and anticipated performance. Where the pile capacity is found to be insufficient to support the design loads, additional piles may be required.

The following additional recommendations regarding steel piles are provided.

1. The allowable capacities noted pertain to geotechnical resistance only. The pile cross sections must be designed to withstand the design loads, handling stresses and the driving forces during installation.
2. The weight of the embedded portion of the pile may be neglected in design.
3. If drop hammers are used, the drop hammer should have a minimum mass equivalent to three times the mass of the pile.
4. The driving of all piles should be documented and approved by qualified geotechnical personnel.
5. Pile spacing should be a minimum of 2.5 pile diameters measured centre to centre.
6. All piles driven within 5 pile diameters of one another should be monitored for heave and where heave is observed the piles should be re-driven to the specified refusal criteria.
7. All piles should be fitted with rock points (driving shoes) to reduce potential damage to the toe of the pile when driving through cobbles or boulders onto bedrock.
8. Driven steel piles should extend a minimum of 8 m below grade to resist adfreezing forces.
9. During the final set, piles should be driven continuously once driving is initiated to the required refusal criteria.
10. A steel follower should not be used for driving of steel piles.

6.4 Lateral Pile Capacity

TREK understands that the lateral loads for the bridges will be accommodated by using battered piles. Additional recommendations or detailed lateral pile analysis can be provided if lateral pile capacity needs to be assessed at either bridge location.

7.0 Excavations and Shoring

All excavations must be carried out in compliance with the appropriate regulation(s) under the Manitoba Workplace Safety and Health Act. Flattening of open excavation side slopes may be required, in particular if saturated soils are encountered. Gravel buttresses could be used to prevent wet silts from flowing into excavations, in conjunction with sump pits used to dewater the excavation.

8.0 Other Considerations

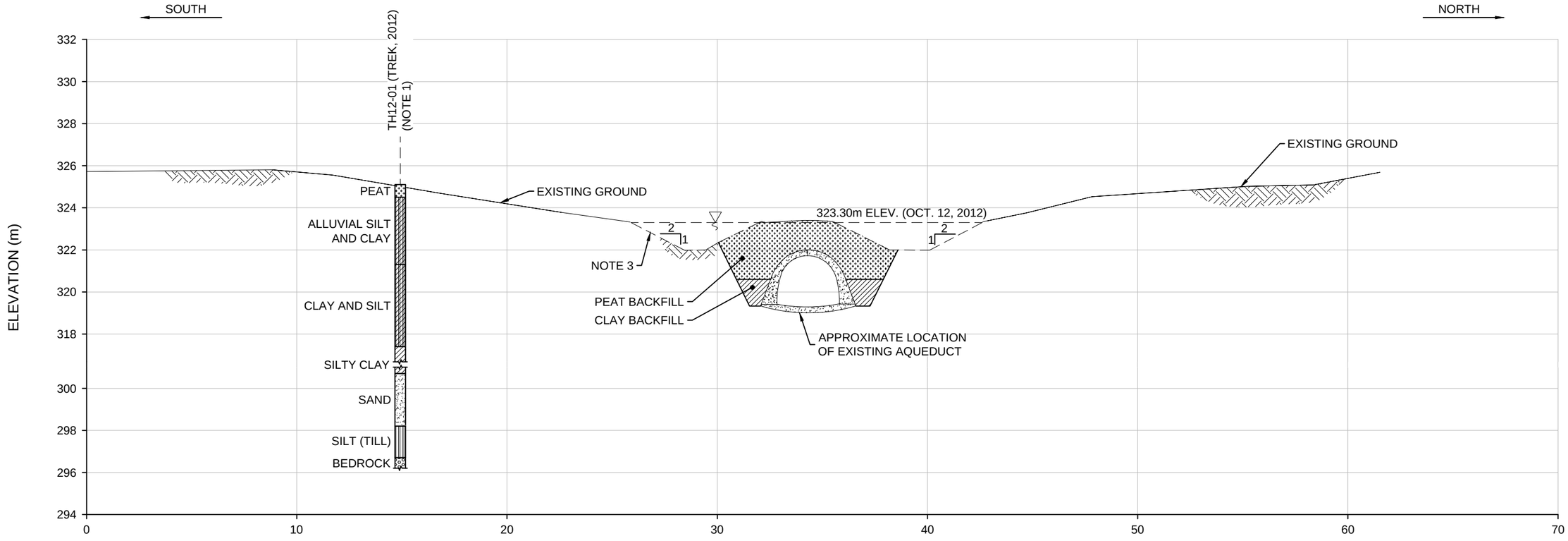
The following items should be considered in addition to the recommendations provided above:

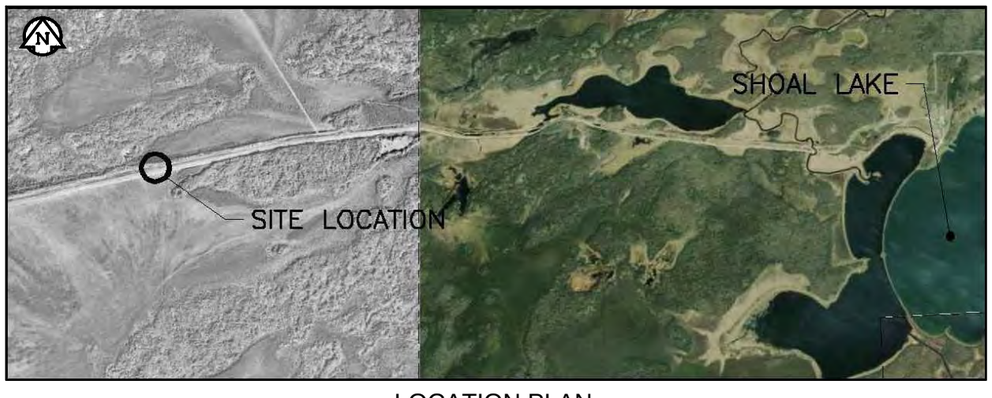
- A hand auger test hole should be completed on the north side of the SLA crossing to confirm the presence of alluvial silts and clays and to establish the alluvial soils/lacustrine clay contact elevation. A piezometer should also be installed in the hand augured test hole to confirm the groundwater levels used in the stability analysis. Should it be considered necessary to confirm the depth to bedrock at the north abutment, it may be preferable to mobilize a drill rig once the road on the north side of the Aqueduct ROW has been cleared.
- Once the crossing location is established for the FRD crossing, an additional topographic and bathymetric survey should be completed to confirm the geometry used in the stability analysis.
- Two hand auger test holes should be completed at the new FRD crossing to confirm the presence of alluvial silts and clays and to establish the alluvial soils/lacustrine clay contact elevation. Piezometers should also be installed in the hand augured test holes to confirm the groundwater levels used in the stability analysis.
- A deep (drill rig) test hole should be completed on one side of the new FRD crossing location to establish the till and bedrock contact elevations.
- The hydraulic and environmental impacts of the proposed rip rap at the FRD crossing should be considered in the detailed design.

9.0 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 identified (field investigation, laboratory testing, geometries). 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 herein, TREK Geotechnical should be notified to adjust the findings outlined in this report as necessary.

Drawings





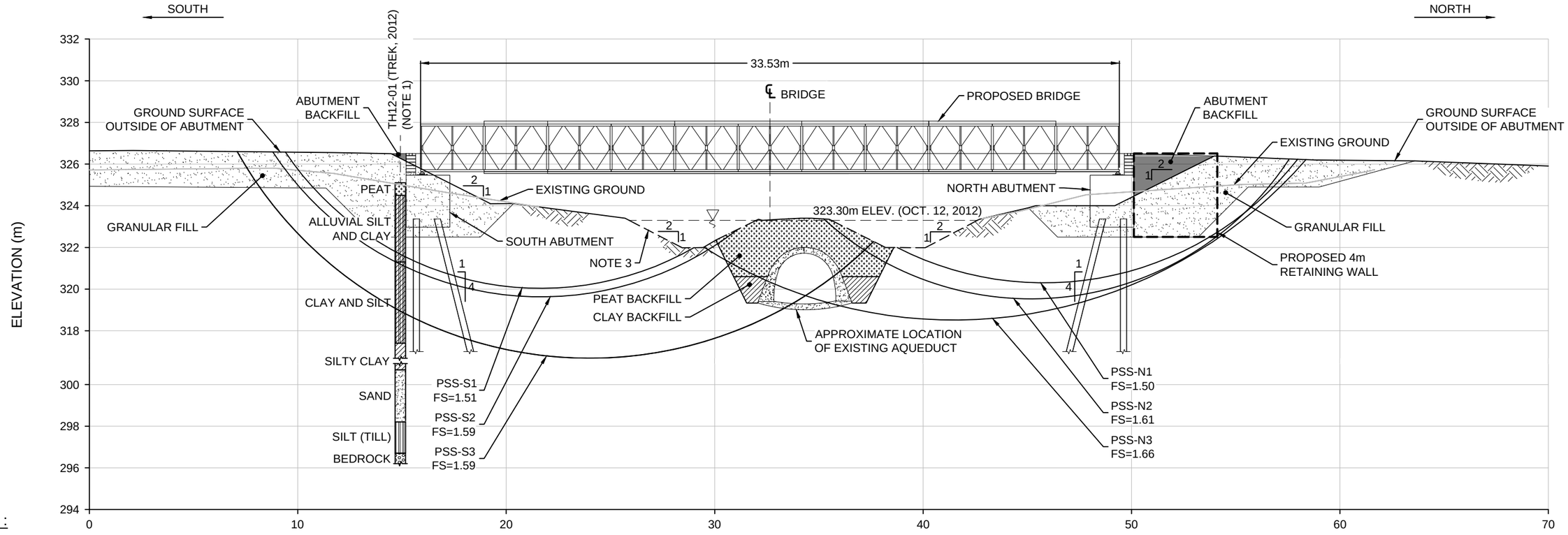
LOCATION PLAN
SCALE : N.T.S.

NOTES :

1. TH12-01 OFFSET 4.3m EAST OF CROSS-SECTION.
2. CROSS-SECTION GEOMETRY BASED OFF OF DILLON OCTOBER 12, 2012 SURVEY.
3. CHANNEL GEOMETRY BELOW WATER LEVEL NOT SURVEYED.

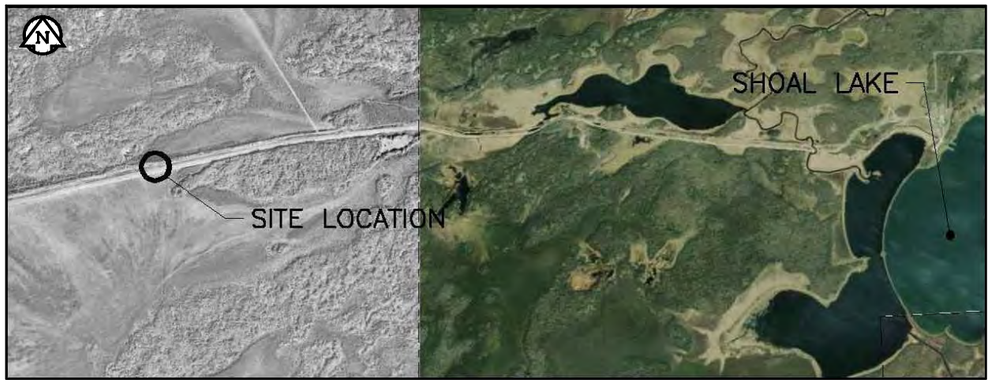
0 2 4 6 8m
SCALE : 1:200 (279mm x 432mm)

Drawing 01
Existing Cross-Section at Mile 93 on Shoal Lake Aqueduct

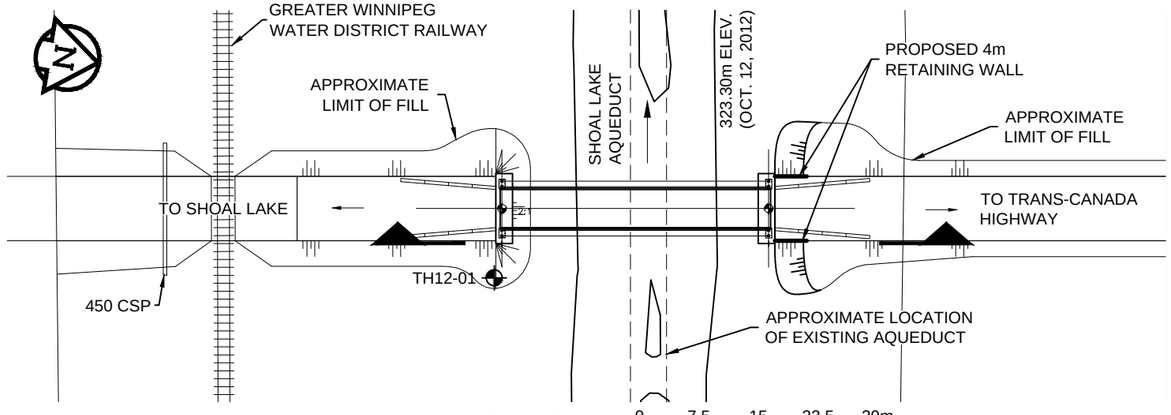


NOTES :

1. TH12-01 OFFSET 4.3m EAST OF CROSS-SECTION.
2. CROSS-SECTION GEOMETRY BASED OFF OF DILLON OCTOBER 12, 2012 SURVEY.
3. CHANNEL GEOMETRY BELOW WATER LEVEL NOT SURVEYED.



LOCATION PLAN
SCALE : N.T.S.



SITE PLAN
SCALE : 1:750

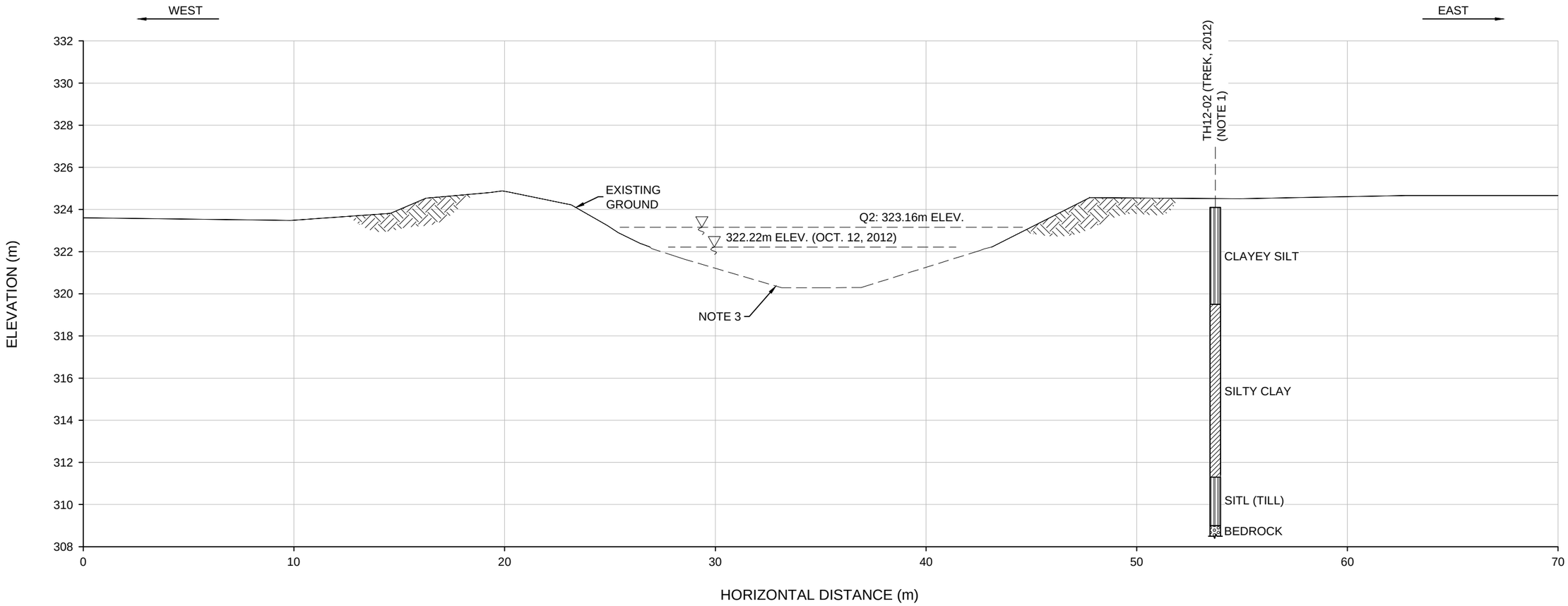
0 7.5 15 22.5 30m
SCALE : 1:750 (279mm x 432mm)

0 2 4 6 8m
SCALE : 1:200 (279mm x 432mm)

LEGEND :

TEST HOLE (TREK, 2012)

Drawing 02
Proposed Mile 93 Bridge Crossing on Shoal Lake Aqueduct



LOCATION PLAN
SCALE : N.T.S.

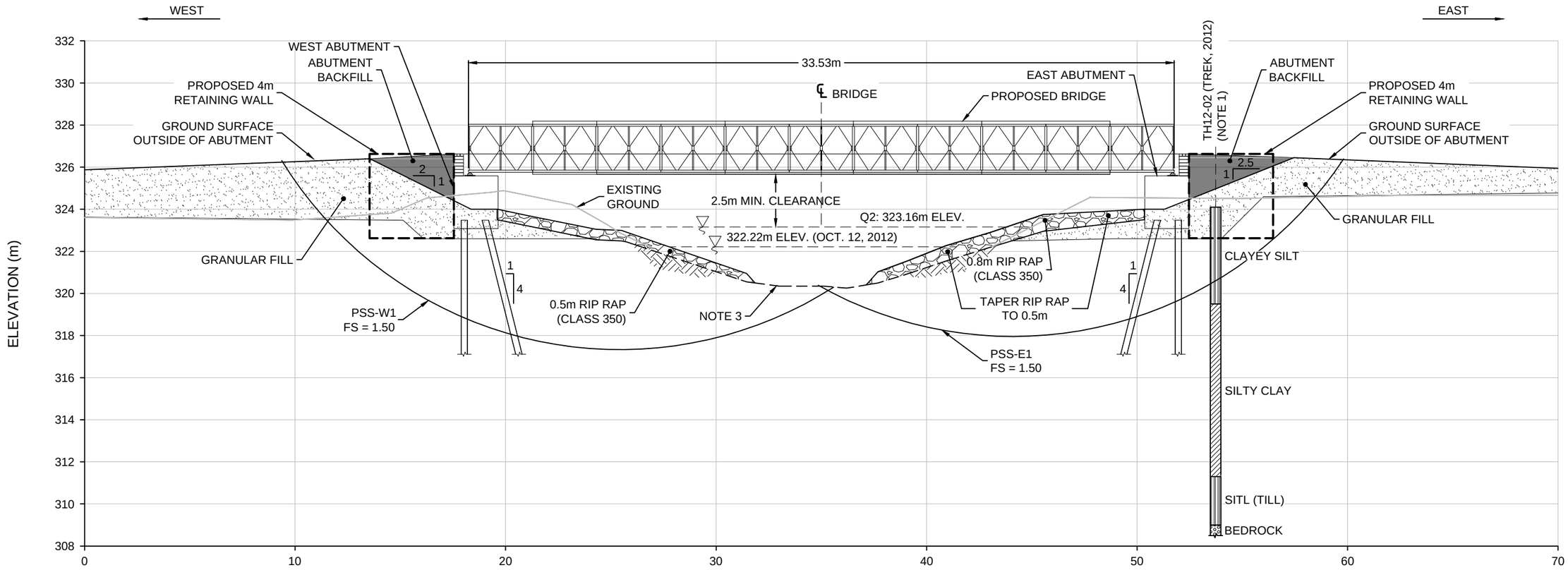
NOTES :

1. TH12-02 OFFSET 3.0m SOUTH OF CROSS-SECTION.
2. CROSS-SECTION GEOMETRY BASED OFF OF DILLON OCTOBER 12, 2012 SURVEY OF ORIGINAL PROPOSED CROSSING LOCATION. NEW PROPOSED CROSSING LOCATION IS APPROXIMATELY 100m SOUTH EAST OF ORIGINAL PROPOSED CROSSING LOCATION.
3. CHANNEL GEOMETRY BELOW WATER LEVEL NOT SURVEYED. CHANNEL GEOMETRY TAKEN FROM UMA ENGINEERING LTD. 2000 SURVEY CROSS SECTION D, APPROXIMATELY 110m SOUTH EAST OF ORIGINAL PROPOSED CROSSING LOCATION.

0 2 4 6 8m
SCALE : 1:200 (279mm x 432mm)

Drawing 03
Existing Cross Section at Falcon River Diversion Channel

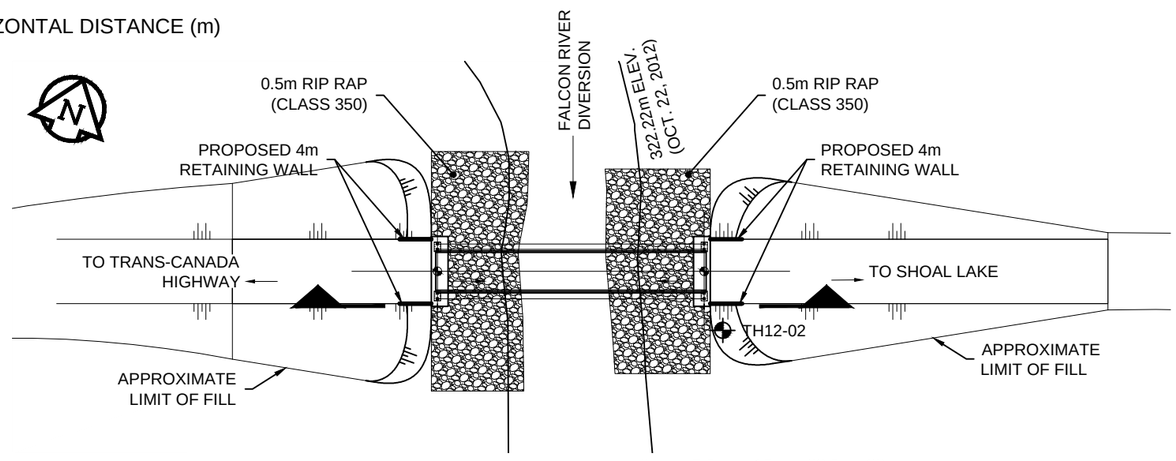
TREK GEOTECHNICAL



LOCATION PLAN
SCALE : N.T.S.

NOTES :

1. TH12-02 OFFSET 3.0m SOUTH OF CROSS-SECTION.
2. CROSS-SECTION GEOMETRY BASED OFF OF DILLON OCTOBER 12, 2012 SURVEY OF ORIGINAL PROPOSED CROSSING LOCATION. NEW PROPOSED CROSSING LOCATION IS APPROXIMATELY 100m SOUTH EAST OF ORIGINAL PROPOSED CROSSING LOCATION.
3. CHANNEL GEOMETRY BELOW WATER LEVEL NOT SURVEYED. CHANNEL GEOMETRY TAKEN FROM UMA ENGINEERING LTD. 2000 SURVEY CROSS SECTION D, APPROXIMATELY 110m SOUTH EAST OF ORIGINAL PROPOSED CROSSING LOCATION.



SITE PLAN
SCALE : 1:750

0 7.5 15 22.5 30m
SCALE : 1:750 (279mm x 432mm)

0 2 4 6 8m
SCALE : 1:200 (279mm x 432mm)

LEGEND :

TEST HOLE (TREK, 2012)

Drawing 04
Proposed Falcon River Diversion Channel Bridge Crossing

Appendix A

Test Hole Logs

Sub-Surface Log

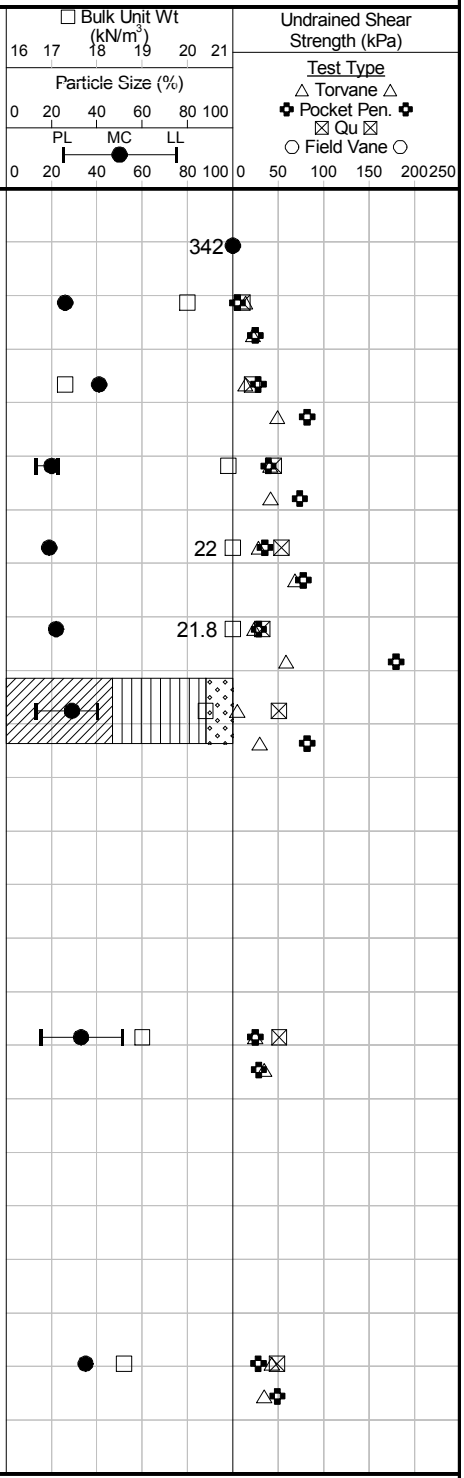
Test Hole TH12-01

Client:	Dillon Consulting	**Project Number:**	0022 005 01
Project Name:	Falcon River Diversion and Shoal Lake Aqueduct Bridges	**Location:**	UTM 15 N-5499351, E-334334 (SLA-Mile 93)
Contractor:	Paddock Drilling Ltd.	**Ground Elevation:**	325.06 m
Method:	170 mm Hollow Stem Auger, Acker SS3 Track Mount	**Date Drilled:**	27 March 2012 - 28 March 2012

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
324.5	0		PEAT - fibrous, fine, dark brown, wet, H3		G01	
	1		SILT - trace clay, trace sand - light grey - moist, very soft - intermediate plasticity		T02	
323.4	2		CLAY - silty - grey, moist, soft, intermediate plasticity		T03	
322.8	3		SILT - some clay, trace oxidation - light brown - moist to wet, soft to firm - low plasticity		T04	
					T05	
321.3	4		CLAY and SILT - some sand, trace gravel, trace oxidation - light brown - moist - firm to stiff, intermediate plasticity		T06	
	5				T07	
317.4	8		CLAY - silty, trace gravel (<10 mm diam.), trace fine to medium sand - grey - moist, firm to stiff - high plasticity		T08	
	11				T09	



Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

SUB-SURFACE LOG AQUEDUCT TEST HOLE LOGS.GPJ TREK GEOTECHNICAL.GDT 25/3/13

Sub-Surface Log

Test Hole TH12-01

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
	13					
	14		-soft to firm, trace silt inclusions (<5mm diam.) below 10.7m		T10	
	15					
	16					
	17				T11	
	18					
	19					
	20				T12	
	21					
	22				T13	
	23				T14	
	24					
300.7			SAND - some gravel, trace to some cobbles - grey - poorly graded, coarse sand to coarse gravel - subrounded gravel		SB15	
	25					
	26		-brown below 26.2 m -cored through 150 mm cobble below from 26.4 m		G16	

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21

Particle Size (%): 0 20 40 60 80 100

PL MC LL: 0 20 40 60 80 100

Undrained Shear Strength (kPa)

Test Type: △ Torvane △ ✿ Pocket Pen. ✿ ☒ Qu ☒ ○ Field Vane ○

0 50 100 150 200 250

Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

SUB-SURFACE LOG AQUEDUCT TEST HOLE LOGS.GPJ TREK GEOTECHNICAL.GDT 25/3/13

Sub-Surface Log

Test Hole TH12-01

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
298.2	27		SILT (Till) - some sand, some gravel - grey - moist - poorly graded, coarse sand to coarse gravel - subrounded gravel		G17	
	28				C18	55
296.7			AMPHIBOLITE (Bedrock) - grey green, fine grained - strong to very strong (R4-R5) - homogenous			
	29				C19	97
	30					
	31				C20	98
292.9	32					

END OF HOLE AT 32.2 m IN AMPHIBOLITE

Notes:

1) Water level was 0.9 m below ground surface during drilling.
2) Sloughing observed at 3.1 m below ground surface during drilling.
3) Drilling method switched to NQ coring below 26.4 m.
4) Upper contact with bedrock is strongly weathered, fractured, and crumbly.

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21

Particle Size (%): 0 20 40 60 80 100

PL MC LL

0 20 40 60 80 100

Undrained Shear Strength (kPa)

Test Type

△ Torvane △

✤ Pocket Pen. ✤

☒ Qu ☒

○ Field Vane ○

0 50 100 150 200 250

Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

Sub-Surface Log

Test Hole TH12-02

Client:	Dillon Consulting	**Project Number:**	0022 005 01
Project Name:	Falcon River Diversion and Shoal Lake Aqueduct Bridges	**Location:**	UTM 15 N-5497152, E-341230 (FRD)
Contractor:	Maple Leaf Drilling	**Ground Elevation:**	324.06 m
Method:	125 mm Solid Stem Auger, Acker Renegade Track Mount	**Date Drilled:**	24 October 2012 - 25 October 2012

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)	Bulk Unit Wt (kN/m³) 16 17 18 19 20 21 / Particle Size (%) 0 20 40 60 80 100 / PL MC LL 0 20 40 60 80 100	Undrained Shear Strength (kPa) Test Type: Torvane, Pocket Pen., Qu, Field Vane 0 50 100 150 200 250
	0		SILT - clayey, trace fine sand, trace gravel, trace organics (rootlets <5 mm diam.), trace oxidation, trace fine grained sand laminations (<1 mm thick) - light brown - moist, firm to very stiff - low to intermediate plasticity		G04			
	1				T05			
	2		-5 mm thick fine sand seam at 1.8 m		T06			
	3				T07			
					T08			
	4				G09			
319.5			CLAY - silty, trace fine to medium sand, trace gravel, trace oxidation, trace organics (rootlets), trace fine grained sand laminations (<1 mm thick) - grey - moist, firm to stiff - intermediate plasticity, laminated		T10			
	5							
	6		-firm and high plasticity below 6.1 m		T11			
	7							
	8		-soft to firm below 7.6 m		T12			
	9							
	10				T13			
	11		-homogenous below 10.7 m		T14	15.3		

Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

SUB-SURFACE LOG AQUEDUCT TEST HOLE LOGS.GPJ TREK GEOTECHNICAL.GDT 25/3/13

Sub-Surface Log

Test Hole TH12-02

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
311.3					T15	
	13		SILT (Till) - clayey, trace coarse sand, trace gravel (<15 mm diam.) - grey - moist, soft - intermediate plasticity			
	14				T16	
309.0	15		AMPHIBOLITE (Bedrock) - grey green, fine grained - strong to very strong (R4-R5) - homogenous		C17	92
	16					
	17				C18	98
306.4						

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21

Particle Size (%): 0 20 40 60 80 100 — PL MC LL — 0 20 40 60 80 100

Undrained Shear Strength (kPa): 0 50 100 150 200 250

Test Type: △ Torvane △ ✿ Pocket Pen. ✿ ☒ Qu ☒ ○ Field Vane ○

END OF HOLE AT 17.7 m IN AMPHIBOLITE

Notes:
1) Water level was 2.9 m below ground surface during drilling.
2) Test hole stayed open to 14.5 m.
3) Drilling method switched to NQ coring below 13.7 m.
4) Could not obtain sample of SILT (Till) below 13.7 m due to drilling method.

Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

Sub-Surface Log

Test Hole TH12-03

Client:	Dillon Consulting	**Project Number:**	0022 005 01
Project Name:	Falcon River Diversion and Shoal Lake Aqueduct Bridges	**Location:**	UTM 15 N-5497133, E-341195 (FRD)
Contractor:	TREK Geotechnical	**Ground Elevation:**	325.85 m
Method:	50 mm Hand Auger	**Date Drilled:**	24 October 2012 - 24 October 2012

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)	Bulk Unit Wt (kN/m³) 16 17 18 19 20 21 / Particle Size (%) 0 20 40 60 80 100 / PL MC LL	Undrained Shear Strength (kPa) 0 50 100 150 200 250 / Test Type: △ Torvane △, ✪ Pocket Pen. ✪, ☒ Qu ☒, ○ Field Vane ○
322.9	0		SILT (Fill) - clayey, trace fine sand, trace gravel, trace organics (rootlets <5 mm diam.), trace oxidation, trace fine grained sand laminations (<1 mm thick) - light brown - moist, firm to stiff - low plasticity		G01			
					G02			
	1				G03			
					G03a			
	2				G03b			
					G03c			
					G03d			

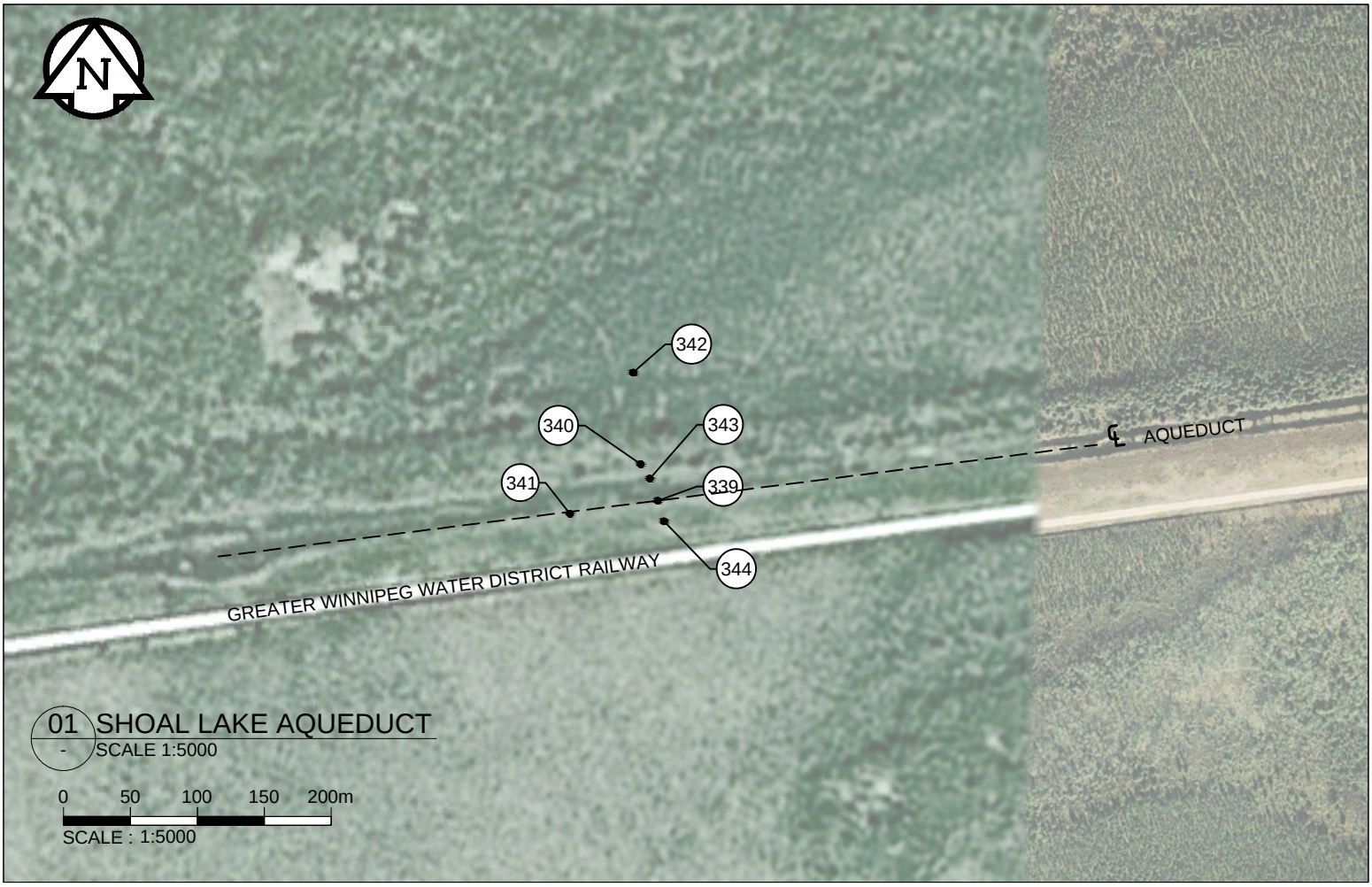
END OF HOLE AT 3.0 m BELOW GROUND SURFACE

Notes:
1) Test hole was dry one day after hand augering
2) Test hole stayed open to 2.6 m one day after hand augering.

Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

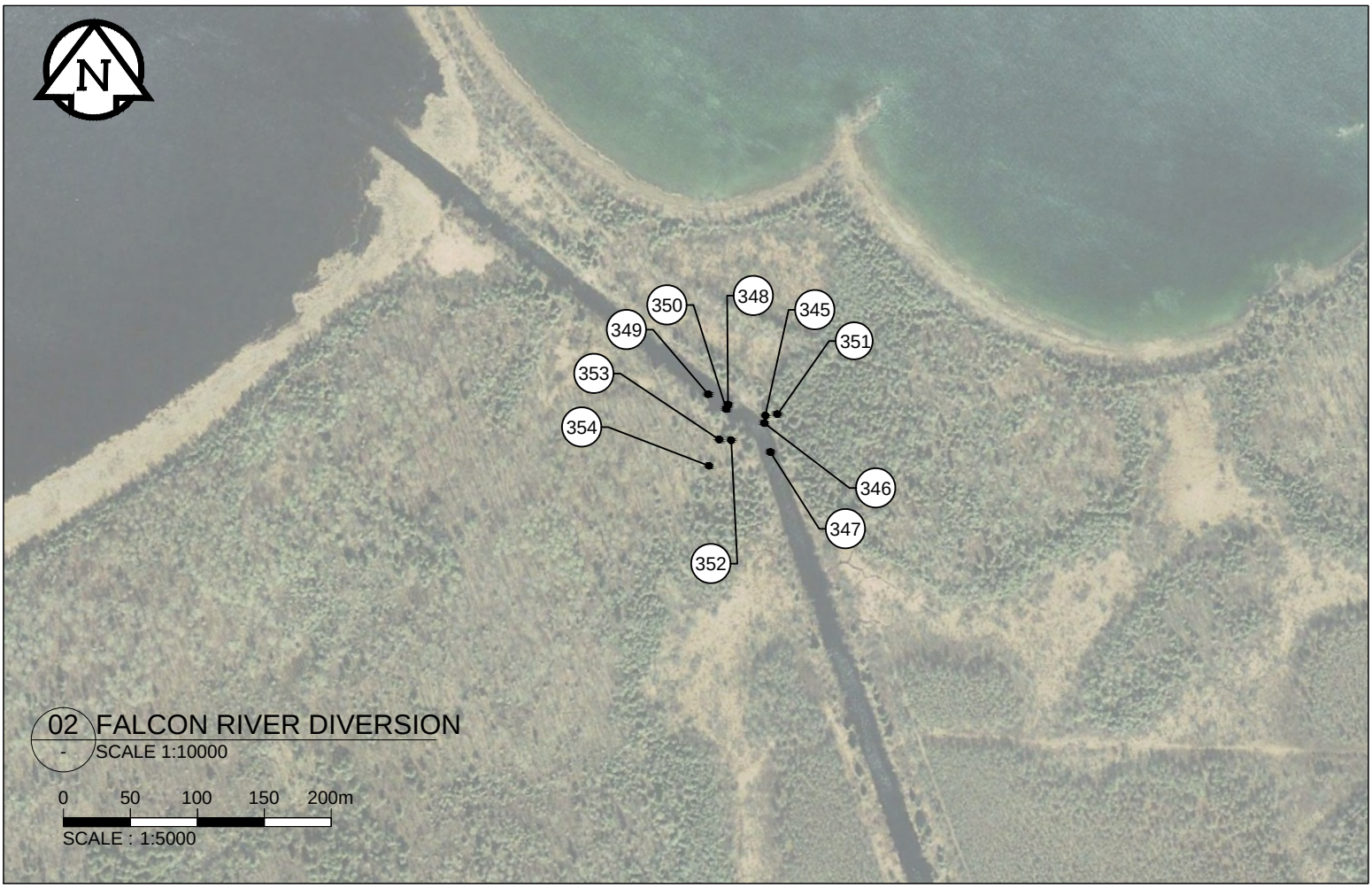
Appendix B

Site Photographs and Photograph Index



01 SHOAL LAKE AQUEDUCT
- SCALE 1:5000

0 50 100 150 200m
SCALE : 1:5000



02 FALCON RIVER DIVERSION
- SCALE 1:10000

0 50 100 150 200m
SCALE : 1:5000

0022 005 01

Shoal Lake Aqueduct (Mile 93) and Falcon RIver Diversion Crossings

Digital Photograph Index

Photo Number	Date Taken	Way Point	UTM (Zone 14U)		Description
			Northing (m)	Easting (m)	
SHOAL LAKE AQUEDUCT (MILE 93)					
IMG_4379	Feb 17 2012	WP339	5503440.1	767633.0	Looking W from center of channel from proposed bridge crossing location
IMG_4380	Feb 17 2012	WP339	5503440.1	767633.0	Looking N from center of channel from proposed bridge crossing location
IMG_4381	Feb 17 2012	WP339	5503440.1	767633.0	Looking NE from center of cahnnel from proposed bridge crossing location
IMG_4382	Feb 17 2012	WP339	5503440.1	767633.0	Looking E from center of channel from proposed bridge crossing location
IMG_4383	Feb 17 2012	--	--	--	Looking S from S bank at proposed bridge crossing location
IMG_4384	Feb 17 2012	--	--	--	Looking S from center of channel from proposed channel crossing location
IMG_4385	Feb 17 2012	WP340	5503466.2	767618.1	Looking S from N bank at proposed bridge crossing location
IMG_4386	Feb 17 2012	WP340	5503466.2	767618.1	Looking SE from N bank at proposed bridge crossing location
IMG_4387	Feb 17 2012	WP340	5503466.2	767618.1	Looking SW from N bank at proposed bridge crossing location
IMG_4388	Feb 17 2012	WP341	5503425.1	767568.3	Looking SE from center of channel from proposed bridge crossing location
IMG_4389	Feb 17 2012	WP341	5503425.1	767568.3	Looking E from center of channel from proposed bridge crossing location
IMG_4390	Feb 17 2012	WP341	5503425.1	767568.3	Looking NE from center of channel from proposed bridge crossing location
IMG_4391	Feb 17 2012	WP344	5503425.1	767638.9	Looking W from center of channel from proposed bridge crossing location
IMG_4392	Feb 17 2012	WP344	5503425.1	767638.9	Looking N from center of channel from proposed bridge crossing location
IMG_4393	Feb 17 2012	WP344	5503425.1	767638.9	Looking E from center of channel from proposed bridge crossing location
FALCON RIVER DIVERSION					
IMG_4394	Feb 17 2012	WP345	5501775.9	774689.1	Looking E towards channel crossing location from E of channel on ATV trail/future road alignment
IMG_4395	Feb 17 2012	WP346	5501770.2	774688.9	Looking S from W bank at proposed crossing location
IMG_4396	Feb 17 2012	WP346	5501770.2	774688.9	Looking at E bank from center of channel from proposed bridge crossing location
IMG_4397	Feb 17 2012	WP346	5501770.2	774688.9	Looking N down channel to Falcon Bay from proposed channel crossing location

0022 005 01

Shoal Lake Aqueduct (Mile 93) and Falcon RIver Diversion Crossings

Digital Photograph Index

Photo Number	Date Taken	Way Point	UTM (Zone 14U)		Description
			Northing (m)	Easting (m)	
IMG_4398	Feb 17 2012	WP346	5501770.2	774688.9	Looking S down channel (downstream) to Snowshoe Bay from proposed channel crossing location
IMG_4399	Feb 17 2012	WP346	5501770.2	774688.9	Looking W from E bank at from proposed channel crossing location
IMG_4400	Feb 17 2012	WP346	5501770.2	774688.9	Looking N down channel to Falcon Bay from proposed channel crossing location
IMG_4401	Feb 17 2012	--	--	--	Looking at E bank from center of channel from proposed bridge crossing location
IMG_4402	Feb 17 2012	--	--	--	Looking NW on E bank from proposed bridge crossing location
IMG_4403	Feb 17 2012	WP347	5501749.1	774695.1	Looking at undercut on E bank from E bank from proposed bridge crossing location
IMG_4404	Feb 17 2012	WP348	5501781.9	774660.7	Looking S from center of channel from proposed channel crossing location
IMG_4405	Feb 17 2012	WP349	5501788.4	774645.3	Looking S from center of channel from proposed channel crossing location
IMG_4406	Feb 17 2012	WP349	5501788.4	774645.3	Looking at E bank from center of channel from proposed channel crossing location
IMG_4407	Feb 17 2012	WP349	5501788.4	774645.3	Looking at W bank from center of channel from proposed channel crossing location
IMG_4408	Feb 17 2012	WP350	5501778.5	774659.6	Looking S from center of channel from proposed channel crossing location
IMG_4409	Feb 17 2012	WP350	5501778.5	774659.6	Looking at W bank from center of channel from proposed channel crossing location
IMG_4410	Feb 17 2012	WP350	5501778.5	774659.6	Looking at E bank from center of channel from proposed channel crossing location
IMG_4411	Feb 17 2012	WP350	5501778.5	774659.6	Looking at E bank from center of channel from proposed channel crossing location
IMG_4412	Feb 17 2012	WP350	5501778.5	774659.6	Looking S from center of channel from proposed channel crossing location
IMG_4413	Feb 17 2012	WP350	5501778.5	774659.6	Looking S from center of channel from proposed channel crossing location
IMG_4414	Feb 17 2012	WP350	5501778.5	774659.6	Looking at E bank from center of channel from proposed bridge crossing location
IMG_4415	Feb 17 2012	WP350	5501778.5	774659.6	Looking at E bank from center of channel from proposed bridge crossing location
IMG_4416	Feb 17 2012	WP350	5501778.5	774659.6	Looking at E bank from center of channel from proposed bridge crossing location
IMG_4417	Feb 17 2012	WP350	5501778.5	774659.6	Looking at E bank from center of channel from proposed bridge crossing location
IMG_4418	Feb 17 2012	WP351	5501777.7	774698.0	Looking at location of proposed TH12-02 on E bank

0022 005 01

Shoal Lake Aqueduct (Mile 93) and Falcon RIver Diversion Crossings

Digital Photograph Index

Photo Number	Date Taken	Way Point	UTM (Zone 14U)		Description
			Northing (m)	Easting (m)	
IMG_4419	Feb 17 2012	WP353	5501755.4	774656.2	Looking at location of proposed TH12-03 on W bank
IMG_4420	Feb 17 2012	WP353	5501755.4	774656.2	Looking E at winter road on W side
IMG_4421	Feb 17 2012	WP353	5501755.4	774656.2	Looking S at winter road on W side
IMG_4422	Feb 17 2012	WP353	5501755.4	774656.2	Looking W at winter road on W side
IMG_4423	Feb 17 2012	WP354	5501735.1	774650.1	Looking E at winter road on W side
IMG_4424	Feb 17 2012	--	--	--	Looking E at winter road entering Horseshoe Bay
IMG_4425	Feb 17 2012	--	--	--	Looking SE at winter road enterting Horseshoe Bay
IMG_4426	Feb 17 2012	--	--	--	Looking W at winter road entering Horseshoe Bay
img_01	Oct 24 2012				Transporting drill rig over the channel on the barge
img_02	Oct 24 2012				TH12-02 looking S on E bank at proposed channel crossing location
img_03	Oct 24 2012				TH12-02 looking S on E bank at proposed channel crossing location
img_04	Oct 24 2012				TH12-02 looking S on E bank at proposed channel crossing location
img_05	Oct 24 2012				TH12-02 looking W on E bank at proposed channel crossing location
img_06	Oct 24 2012				TH12-03 looking E on W bank at proposed channel crossing location
img_07	Oct 24 2012				TH12-03 looking E on W bank at proposed channel crossing location
img_08	Oct 24 2012				TH12-03 looking S on W bank at proposed channel crossing location
img_09	Oct 24 2012				TH12-03 looking S on W bank at proposed channel crossing location

IMG_4379

IMG_4380

IMG_4381

IMG_4382

Mile 93 Shoal Lake Aqueduct - February 17, 2012

IMG_4383

IMG_4384

IMG_4385

IMG_4386

Mile 93 Shoal Lake Aqueduct - February 17, 2012

IMG_4387

IMG_4388

IMG_4389

IMG_4390

Mile 93 Shoal Lake Aqueduct - February 17, 2012

IMG_4391

IMG_4392

IMG_4393

Mile 93 Shoal Lake Aqueduct - February 17, 2012

IMG_4394

IMG_4395

IMG_4396

IMG_4397

Falcon River Diversion - February 17, 2012

IMG_4398

IMG_4399

IMG_4400

IMG_4401

Falcon River Diversion - February 17, 2012

IMG_4402

IMG_4403

IMG_4404

IMG_4405

Falcon River Diversion - February 17, 2012

IMG_4406

IMG_4407

IMG_4408

IMG_4409

Falcon River Diversion - February 17, 2012

IMG_4410

IMG_4411

IMG_4412

IMG_4413

Falcon River Diversion - February 17, 2012

IMG_4414

IMG_4415

IMG_4416

IMG_4417

Falcon River Diversion - February 17, 2012

IMG_4418

IMG_4419

IMG_4420

IMG_4421

Falcon River Diversion - February 17, 2012

IMG_4422

IMG_4423

IMG_4424

IMG_4425

Falcon River Diversion - February 17, 2012

IMG_4426

img_01

img_02

img_03

img_04

Falcon River Diversion - October 24, 2012

img_05

img_06

img_07

img_08

Falcon River Diversion - October 24, 2012

img_09

Appendix C

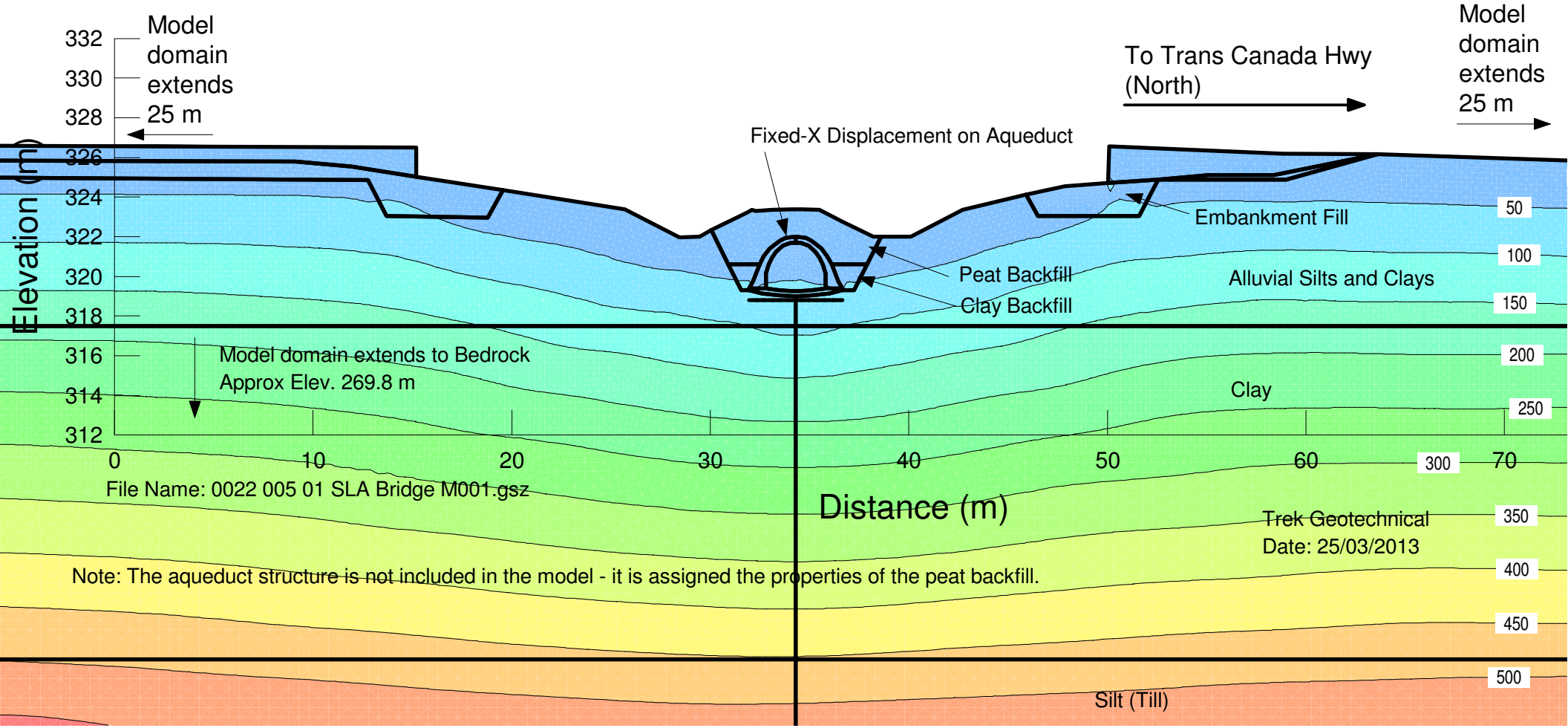
Stress-Deformation Model Outputs

0022 005 01
Dillon Consulting Ltd.
City of Winnipeg Shoal Lake Aqueduct
Proposed Bridge Crossing at Mile 93
Load/Deformation

Method: Load/Deformation

Name: Clay Effective Young's Modulus (E'): 5000 kPa Poisson's Ratio: 0.4 Unit Weight: 19 kN/m³
Name: Peat Backfill Effective Young's Modulus (E'): 200 kPa Poisson's Ratio: 0.4 Unit Weight: 14 kN/m³
Name: Clay Backfill Effective Young's Modulus (E'): 5000 kPa Poisson's Ratio: 0.4 Unit Weight: 19 kN/m³
Name: Embankment Fill Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.3 Unit Weight: 21 kN/m³
Name: Alluvial Silts and Clays Effective Young's Modulus (E'): 15000 kPa Poisson's Ratio: 0.4 Unit Weight: 20.5 kN/m³
Name: Silt (Till) Effective Young's Modulus (E'): 100000 kPa Poisson's Ratio: 0.4 Unit Weight: 19 kN/m³

Type: Y-Effective Stress kPa
Starting Contour Value: 0kPa
Increment by: 50kPa



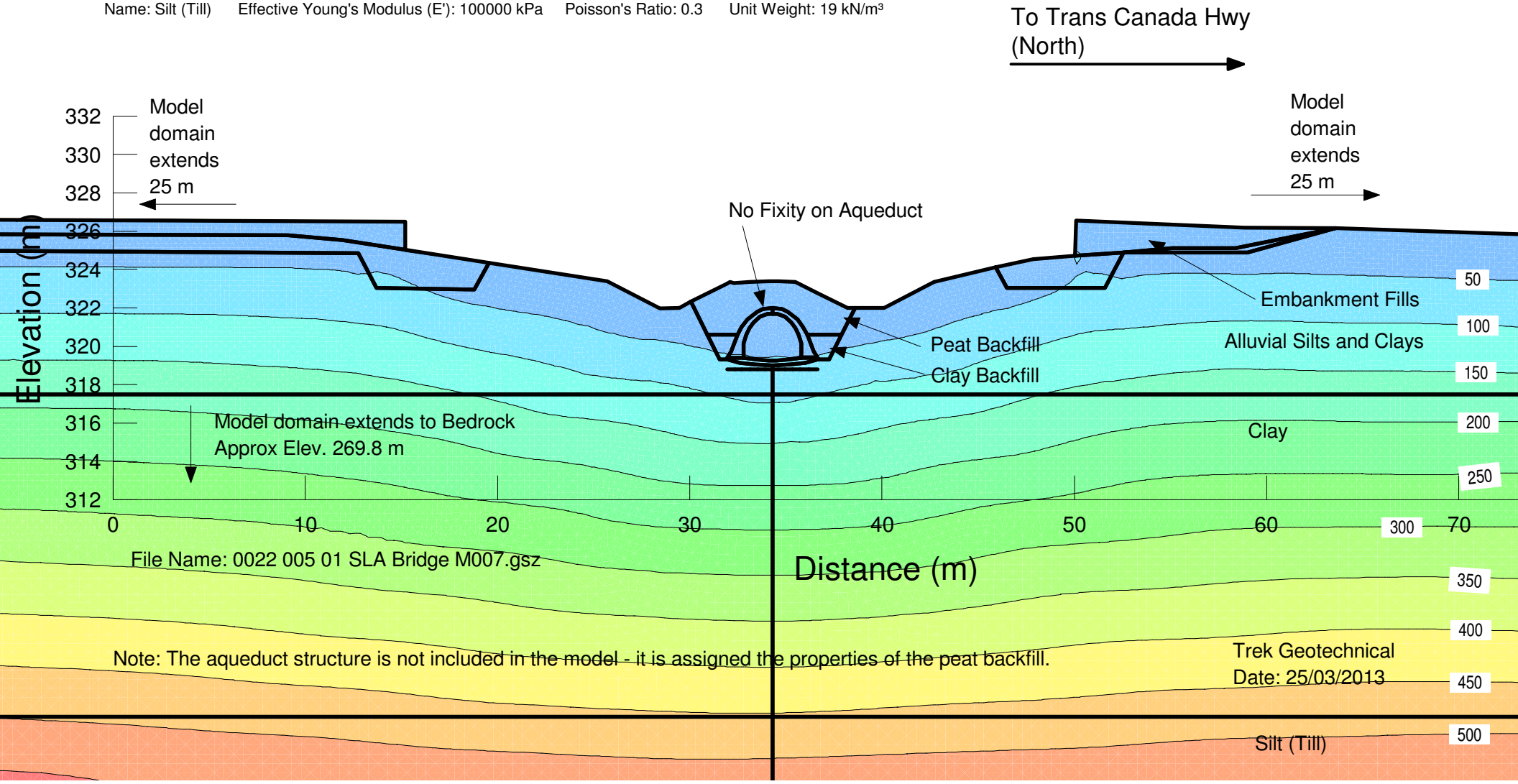
Note: The aqueduct structure is not included in the model - it is assigned the properties of the peat backfill.

0022 005 01
Dillon Consulting Ltd.
City of Winnipeg Shoal Lake Aqueduct
Proposed Bridge Crossing at Mile 93
Load/Deformation

Method: Load/Deformation

Name: Clay Effective Young's Modulus (E'): 5000 kPa Poisson's Ratio: 0.4 Unit Weight: 19 kN/m³
Name: Peat Backfill Effective Young's Modulus (E'): 200 kPa Poisson's Ratio: 0.4 Unit Weight: 14 kN/m³
Name: Clay Backfill Effective Young's Modulus (E'): 5000 kPa Poisson's Ratio: 0.4 Unit Weight: 19 kN/m³
Name: Embankment Fill Effective Young's Modulus (E'): 40000 kPa Poisson's Ratio: 0.3 Unit Weight: 21 kN/m³
Name: Alluvial Silts and Clays Effective Young's Modulus (E'): 15000 kPa Poisson's Ratio: 0.4 Unit Weight: 20.5 kN/m³
Name: Silt (Till) Effective Young's Modulus (E'): 100000 kPa Poisson's Ratio: 0.3 Unit Weight: 19 kN/m³

Type: Y-Effective Stress kPa
Starting Contour Value: 0kPa
Increment by: 50kPa



TREK GEOTECHNICAL Quality Engineering | Valued Relationships

Dillon Consulting Ltd.

Falcon River Diversion Bridge

Prepared for:

Mr. Graeme Loeppky
Dillon Consulting Ltd.
1558 Willson Place
Winnipeg, MB R3T 0Y4

Project Number: 0022 009 00

Date: October 17, 2014

www.trekgeotechnical.ca
1712 St. James Street | Winnipeg, Manitoba R3H 0L3 | Tel 1.204.975.9433 | Fax 1.204.975.9435

TREK GEOTECHNICAL Quality Engineering | Valued Relationships

October 17, 2014 Our File No. 0022 009 00

Mr. Graeme Loeppky
Dillon Consulting Ltd.
1558 Willson Place
Winnipeg, MB R3T 0Y4

RE: Falcon River Diversion Bridge

TREK Geotechnical Inc. is pleased to submit our Final Report for the geotechnical investigation for the above noted project.

Please contact the undersigned should you have any questions.

Sincerely,

TREK Geotechnical Inc.
Per:

Ken Skaftfeld, P.Eng
Senior Geotechnical Engineer
Tel: 204.975.9433 ext. 106

KS: jh
Encl.

Revision History

Revision No.	Author	Issue Date	Description
0	KMS	October 17, 2014	Final Report

Authorization Signatures

Prepared By:

Ken Skaftfeld, P.Eng.
Senior Geotechnical Engineer, Associate

PROVINCE OF MANITOBA
K. M. SKAFTFELD
OCT 17/14
REGISTERED ENGINEER

Reviewed By:

Kent Bannister, P.Eng.
Geotechnical Engineer, Associate

APEGM
Certificate of Authorization
Trek Geotechnical Inc.
No. 4877 Date: Oct 17/14

Table of Contents

List of Drawings

List of Appendices

1.0 Introduction and Background

TREK Geotechnical Inc. (TREK) was retained by Dillon Consulting Ltd. (Dillon) to complete a geotechnical investigation and provide recommendations related to the detailed design of the bridge crossing on the Falcon River Diversion (FRD). The terms of reference for the work are included in our proposal to Mr. Graeme Loeppky, P.Eng., and dated July 12, 2013. TREK previously prepared a preliminary design report for bridges proposed at Mile 93 on the Shoal Lake Aqueduct and the Falcon River Diversion (March 26, 2013). The location of the Falcon River Diversion crossing, originally planned to coincide with the all season road constructed on the Shoal Lake Band No. 40 has since been shifted approximately 300 m southeast to allow for the horizontal curve necessary on the east side of the diversion channel. The general location is shown on Figure 01. This report provides information relative to the detailed design of the FRD Bridge at the revised location; readers are referred to TREK's Preliminary Design report for additional information.

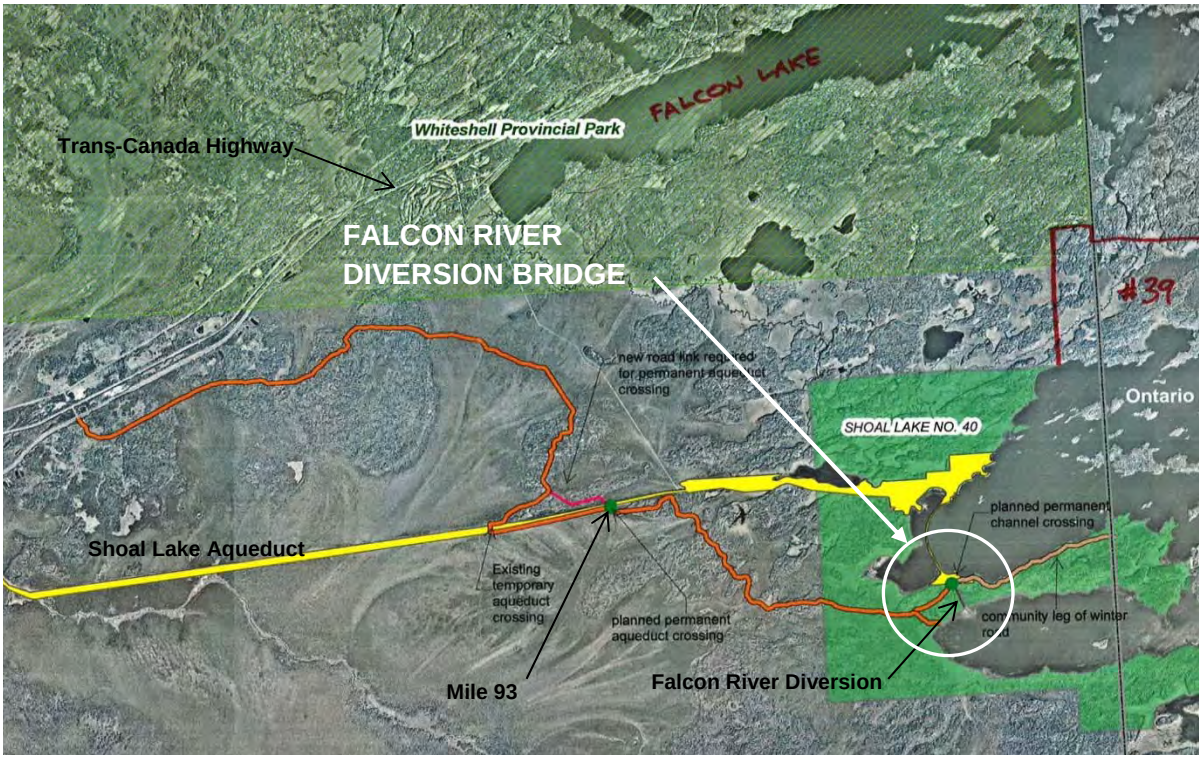


Figure 01 Falcon River Diversion Bridge Crossing

The proposed bridge consists of prefabricated steel box truss ACROW panel structures, 36.5 m in length. Concrete abutments (on piles) will provide support at either end of the bridge deck. Approach embankments will be required to provide the necessary bridge deck clearance for hydraulic and navigation considerations.

An initial site visit was completed by Mr. Ken Skaftfeld of TREK in the accompaniment of Mr. Graeme Loeppky and Mr. Mark Doucet of Dillon and Mr. Richard Song of the City of Winnipeg on

February 17, 2012. Subsequent to the decision to move the crossing location, a second site visit was made by Mr. Ken Skaftfeld in the accompaniment of Mr. Martial Lemoine of TREK and Mr. Bruce Harding of Bruce Harding Consulting Limited on September 4, 2013. Select photographs from the site visit are referenced throughout this report.

2.0 Subsurface Investigation

2.1 2012 Investigation (Preliminary Design)

THs 12-02 and 12-03 are located about 25 m northwest of the crossing location selected for detailed design. Test hole (TH) 12-02 was drilled on October 24, 2012 using a track mounted drill rig operated by Maple Leaf Drilling Ltd. The test hole was located on the east side of the Falcon River Diversion at the end of the road from the Shoal Lake No. 40 community. TH 12-03 was completed on the west side of the diversion channel by TREK personnel using a 50 mm hand auger. Logs for these test holes are provided in TREK's March 2013 report and are attached for reference in Appendix A-1.

2.2 2013 Investigation (Detailed Design)

THs 13-01, 13-02, and 13-03 were drilled in proximity to the revised crossing location to confirm soil conditions and depth to bedrock at the location selected for detailed design. TH 13-01 was drilled with a 50 mm diameter hand auger on the west side of the channel through the spoil bank. TH 13-02 was drilled on the east side of the channel in proximity to the east abutment with 125 mm diameter solid stem augers to refusal and completed with NQ coring into bedrock. Test hole 13-03 was drilled with a 50 mm diameter hand auger adjacent to TH 13-02 to install a standpipe piezometer. Test hole locations are shown on Drawing 01.

Drilling was completed under the supervision of TREK personnel and test holes were visually logged based on the Unified Soil Classification System (USCS). Disturbed (auger cuttings), relatively undisturbed (Shelby tube) and bedrock core samples were recovered during the drilling. A standpipe piezometer was installed in TH 13-01 and 13-03 to measure short term groundwater levels. Test hole locations were surveyed by Dillon or by TREK using a hand held GPS. Test hole elevations were established by rod and level using a local benchmark.

Soil samples were transported to TREK's soils laboratory in Winnipeg for further classification and testing. Testing included the determination of moisture content, Atterberg limits, grain size (hydrometer method), unit weight, and undrained shear strengths. The results from the laboratory testing are shown on the individual test hole logs in Appendix A-2.

2.3 Site Conditions

The proposed bridge will cross the Falcon River Diversion approximately 3.5 km west of the community at Shoal Lake No. 40. The channel was excavated across the peninsula separating Indian Bay and Snowshoe Bay during construction of the Shoal lake Aqueduct between 1914 and 1915 to divert Falcon River water away from the Intake. The originally proposed channel crossing was situated in a relatively flat, heavily treed area immediately northwest of a horizontal bend in the channel where a large area on the west side of the channel had been cleared at the termination of a recently constructed winter road. The bridge crossing has since been relocated about 30 m southeast, along a straight portion of the channel. A temporary bridge was erected about 50 m northwest of the revised channel crossing in the winter of 2012/13 (Figure 02).

Figure 02 View North at Temporary Bridge

The surveyed channel water level was Elev. 322.22 m on October 12, 2012. Historical records obtained from Dillon show that the lowest water level to date in the FRD channel was Elev. 321.87 m recorded on March 28, 1930. Between 2000 and 2011, the lowest water Elev. was 322.26 m. The ordinary high water mark (Q_2)[1] is estimated to be at Elev. 323.16 m. The channel is about 15 m wide with banks in the order of 2 m high (Figure 03). Spoil material from the original excavation has been piled on the top of the west bank to a height of about 1.5 to 2 m. The channel spoil banks show evidence of undercutting although no instabilities of the banks were visible. A site plan and cross section at the crossing location is presented on Drawing 01.

[1] Level that will be equaled or exceeded, on average, once every 2 years

Figure 03 View SE at Proposed Crossing Location

3.0 Soil and Groundwater Conditions

3.1 Soil Profile

The subsurface stratigraphy in descending order from ground surface generally consists of silt fill (west bank) overlying native deposits of alluvial silt, lacustrine clay, silt till, and bedrock. More detailed descriptions of soil units are provided in TREK's preliminary design report. All interpretations of soil stratigraphy for the purposes of design and construction should refer to the detailed test hole logs attached in Appendix A-1 and A-2. Of note is that bedrock was encountered at 10.7 m below ground surface (Elev. 317.7 m) in TH 13-02, compared with 15.1 m below ground surface (Elev. 309.0) in TH 12-02 at the original crossing location. These holes are approximately 40 m apart.

3.2 Groundwater Conditions

Groundwater conditions observed over the short term (during and shortly after drilling) are summarized below. More detailed information is available on the test hole logs. It is important to recognize that the measured groundwater levels should be considered short-term and may vary seasonally, after heavy precipitation events or as a result of construction activities or changes in the water level in the diversion channel.

3.2.1 *2012 Investigation (Preliminary Design)*

Seepage was observed in TH12-02. Groundwater levels in TH 12-02 appeared to be at about 2.9 m below ground surface (Elev. 321.16 m). No seepage was observed in TH12-03.

3.2.2 *2013 Investigation (Detailed Design)*

Seepage was observed at a depth of 2.9 m below the top of the spoil pile in TH 13-01 during drilling (approximately Elev. 322.1 m.). Seepage was also observed during drilling of TH 13-02 at a depth of about 4.3 m below ground surface (approximately Elev. 320.3 m). A groundwater elevation of 322.49 m was measured in TH 13-01 (SP 13-01) one to two weeks after piezometer installation. A groundwater elevation of 320.1 m was measured in TH 13-03 (SP 13-03) within one hour after drilling.

3.3 Slope Stability Analysis

Slope stability analysis was completed for the proposed bridge and abutment geometry provided by Dillon. The preliminary assumptions included an earth fill approach embankment and concrete abutments (pile supported). The stability analysis was conducted using a limit-equilibrium slope stability model (Slope/W) from the GeoStudio 2007 software package (Geo-Slope International Inc.). Slip surfaces were specified with the grid and radius method, with factors of safety calculated using the Morgenstern-Price method of slices.

3.3.1 *Model Geometry*

The model geometry is based upon the topographic survey information provided by Dillon and the cross section used for the analysis was taken just outside of the abutment where the fill is at a maximum height. The water level in the channel is based on the top of ice level obtained in the Dillon October 12, 2012 survey. The bridge deck is 36.4 m long and approximately centered on the channel.

3.3.2 *Soil Properties*

The soil parameters used in the analysis are identical to those assumed for the slope stability analysis carried out for preliminary design. Table 3.1 presents the representative soil units and the strength properties used in the stability analysis.

Table 3.1 – Soil Properties for Falcon River Diversion Stability Analysis

Soil Unit	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle (deg)
Alluvial Silt	19.5	2	22
Lacustrine Silty Clay	18.0	5	17
Embankment Fill	21.0	0	40
Silt Till	20.0	0	45
Rip Rap	21.0	0	45

3.3.3 ***Groundwater Conditions***

Groundwater conditions were modelled using piezometric lines. In the vicinity of the proposed abutments, groundwater levels were assumed to be approximately at the base of the embankment fill, sloping towards the surveyed water level in the channel. These levels are higher than those observed during drilling however they reflect the potential for the ground to be saturated.

3.4 Modelling Results

The factor of safety (FS) for potential slip surfaces through the approach fill immediately adjacent to the abutment on both sides of the channel were determined. The critical slip surface is representative of one that potentially could affect the bridge abutments which is also the slip surface with the minimum FS for the cross-section analyzed. A minimum FS of 1.5 (rounded) was targeted for potential slip surfaces extending behind the abutments. A minimum FS of 1.3 (rounded) was targeted for potential slip surfaces in front of the abutments.

Modelling of the originally proposed bridge geometry resulted in calculated factors of safety lower than the targets and the following modifications were then incorporated into the model to achieve the target FS values. The output shown in Appendix B presents the results of the modelling with the proposed modifications that satisfy the target FS values.

- Increase the depth of granular fill around the abutments to improve soil strength and lower groundwater levels in the vicinity of the abutments;
- Construct 4 m long wing walls behind the east and west abutments to offset fill loading away from the top of bank.
- Extend the proposed rip rap and adjust the thickness of the blanket for additional toe support and scour protection, but leaving 300 mm between the end of the riprap and edge of the Q_2 water level.

4.0 Foundation Recommendations

In consideration of the anticipated loading and installation difficulties associated with cast-in-place concrete piles, driven steel piles were selected as the preferred foundation type.

4.1 Limit States Design

Limit state design recommendations according to the 2010 Canadian Highway and Bridge Design Code (CHBDC) are provided below. Limit States design requires consideration of distinct loading scenarios and prescribes resistance factors (reduction factors) that are based upon the method used to evaluate pile capacity.

The ultimate bearing capacity values for the soils and rock at the site need to be factored using resistance factors as defined in the 2010 CHBDC to establish the Ultimate Limit State (ULS) pile capacity, which can be compared against the ULS (factored) load combinations defined for the structure. The Service Limit State (SLS) is concerned with limiting the deformation or settlement of

the foundation under static loading conditions such that the integrity of the structure will not be impacted by comparing SLS (unfactored) structural loads to the SLS pile capacity.

4.2 Driven Steel Piles

Piles driven to refusal on the bedrock are considered a viable option for support of bridge abutments at the proposed Falcon River Diversion. It is anticipated that piles can be driven through the clays and tills to the underlying bedrock. The ULS design criteria outlined in the CHBDC (Clause C10.22.2) present three resistance factors that should be combined when driving steel piles. Table 4.1 presents the three resistance factors.

Table 4.1 – ULS Resistance Factors for Driven Steel Piles

Case	Resistance Factor
To account for the unintended eccentricity of the applied load about the pile axis	0.8
For the variation of material and geometric properties of the pile	0.9
To account for pile damage during driving	0.75

Due to the nature of driving steel piles to refusal on bedrock, all three resistance factors should be used for the ULS design case. The product of all three results in a resistance factor of 0.5 (rounded). Steel piles driven to refusal on bedrock may be designed with an ULS capacity of 50% of the yield stress of the steel, multiplied by the cross sectional area of the steel. Steel piles driven to refusal on bedrock may be designed with a SLS capacity of 30% of the yield stress of the steel, multiplied by the cross sectional area of the steel.

Refusal criteria and load capacity for specific piles should be established by TREK once the pile sizes and driving method are known in order to verify that the geotechnical and structural capacity has been adequately addressed to minimize the potential for pile damage during driving. Driving should proceed under careful observation near bedrock to avoid overdriving the pile, which could lead to pile damage or misalignment.

It is common for bedrock in these areas to slope significantly. In the event that it appears that piles are sliding on bedrock during construction, misalignment and pile damage could occur. Where this occurs, driving should be discontinued to avoid further misalignment of the pile, and an assessment made of the pile capacity and anticipated performance. Where the pile capacity is found to be insufficient to support the design loads, additional piles may be required. The type of driving shoe selected for the piles should consider the potential for sloping bedrock.

The following additional recommendations regarding steel piles are provided.

1. The allowable capacities noted pertain to geotechnical resistance only. The pile cross sections must be designed to withstand the design loads, handling stresses and the driving forces during installation.
2. The weight of the embedded portion of the pile may be neglected in design.
3. If drop hammers are used, the drop hammer should have a minimum mass equivalent to three times the mass of the pile.
4. The driving of all piles should be documented and approved by TREK.
5. Pile spacing should be a minimum of 2.5 pile diameters measured centre to centre.
6. All piles driven within 5 pile diameters of one another should be monitored for heave and where heave is observed the piles should be re-driven to the specified refusal criteria.
7. All piles should be fitted with rock points (driving shoes) to reduce potential damage to the toe of the pile when driving through cobbles or boulders onto bedrock.
8. Driven steel piles should extend a minimum of 8 m below grade to resist ad-freezing forces.
9. During the final set, piles should be driven continuously once driving is initiated to the required refusal criteria.
10. A steel follower should not be used for driving of steel piles.

4.3 Lateral Pile Capacity

TREK understands that the lateral loads for the bridges will be accommodated by using battered piles. Additional recommendations or detailed lateral pile analysis can be provided if lateral pile capacity needs to be assessed at either bridge location.

5.0 Excavations and Shoring

All excavations must be carried out in compliance with the appropriate regulation(s) under the Manitoba Workplace Safety and Health Act. Flattening of open excavation side slopes may be required, in particular if saturated soils are encountered; such conditions may be expected in excavations for the granular fill beneath the abutments. Gravel buttresses or blankets could be used to prevent wet silts from flowing into excavations, in conjunction with sump pits used to dewater the excavation.

6.0 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 subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

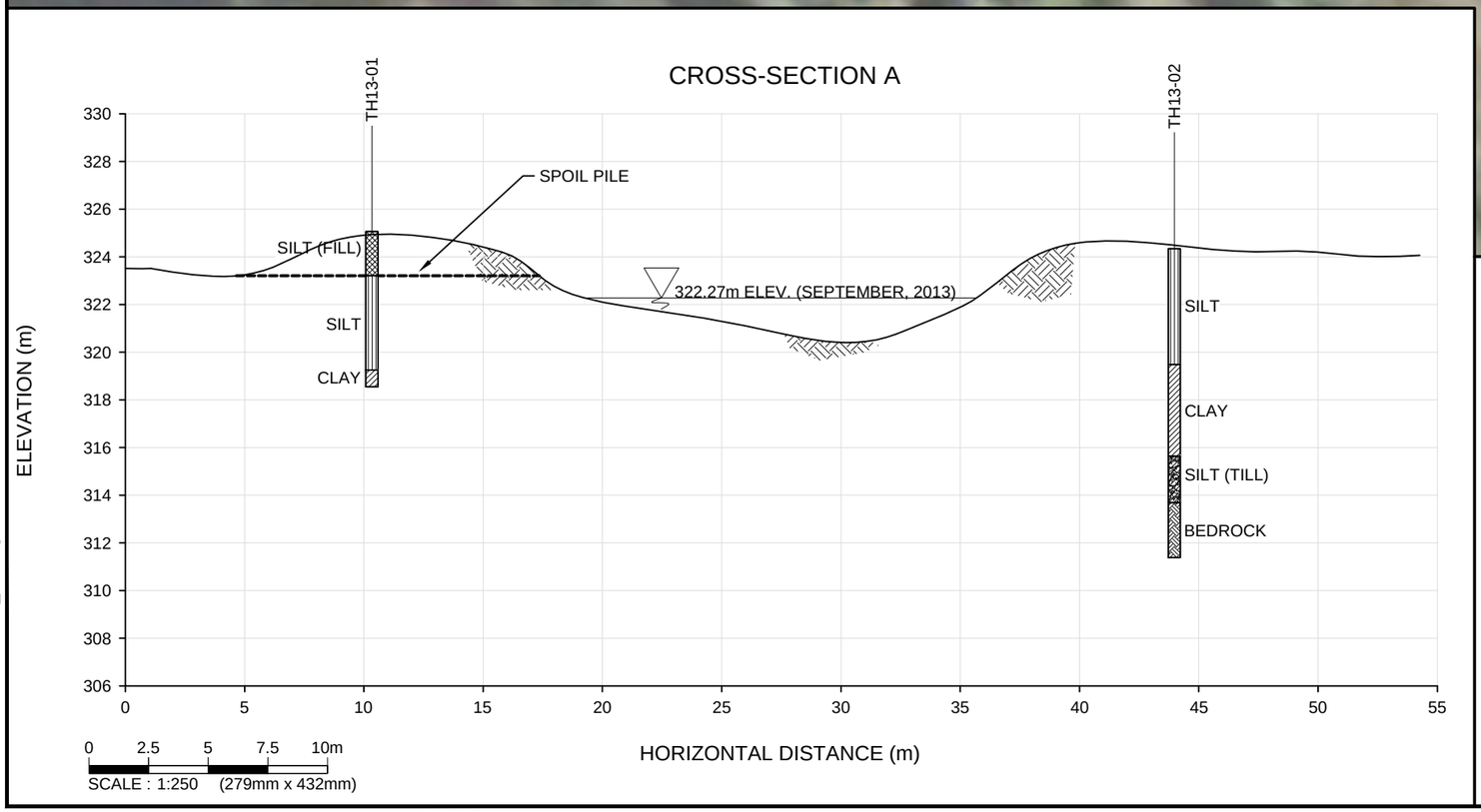
All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already

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 Geotechnical Inc. (the Consultant) for the exclusive use of the Dillon (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.

Drawings

0 4 8 12 16m
SCALE : 1:400 (279mm x 432mm)



0 2.5 5 7.5 10m
SCALE : 1:250 (279mm x 432mm)

LEGEND :

TH12-02 — TEST HOLE (TREK, 2012).

TH13-01 — TEST HOLE (TREK, 2013).

NOTES :

1. CROSS SECTION PROVIDED BY DILLON.

KEY PLAN
SCALE: N.T.S.

Drawing 01
Site Plan

Appendix A-1

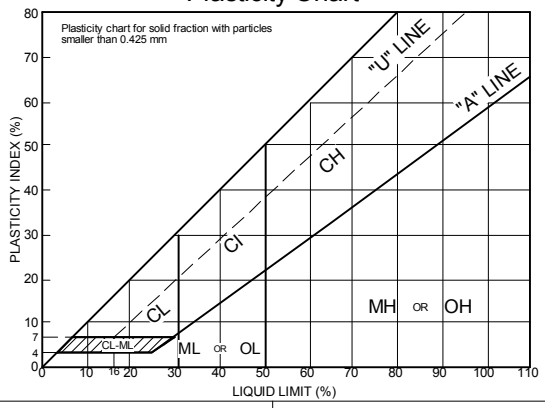
Test Hole Logs (2012 Sub-surface Investigation)

EXPLANATION OF FIELD AND LABORATORY TESTING

GENERAL NOTES

1. Classifications are based on the United Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
2. 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.
3. 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
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 case4s 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
			GP		Poorly-graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW
		Gravel with fines (Appreciable amount of fines)	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
			SP		Poorly-graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW
		Sands with fines (Appreciable amount of fines)	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)	Silts 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
			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	
	Silts 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



Particle Size			Material
ASTM Sieve sizes	mm		
#10 to #4	2.00 to 4.75		Sand
#40 to #10	0.425 to 2.00		Coarse
#200 to #40	0.075 to 0.425		Medium
< #200	< 0.075		Fine
			Silt or Clay

Particle Size			Material
ASTM Sieve Sizes	mm		
> 12 in.	> 300		Boulders
3 in. to 12 in.	75 to 300		Cobbles
3/4 in. to 3 in.	19 to 75		Gravel
#4 to 3/4 in.	4.75 to 19		Coarse
			Fine

* 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

EXPLANATION OF FIELD AND LABORATORY TESTING

LEGEND OF ABBREVIATIONS AND SYMBOLS

LL - Liquid Limit (%)
PL - Plastic Limit (%)
PI - Plasticity Index (%)
MC - Moisture Content (%)
SPT - Standard Penetration Test
RQD - Rock Quality Designation
Qu - Unconfined Compression
Su - Undrained Shear Strength
VW - Vibrating Wire Piezometer
SI - Slope Inclinometer

▽ Water Level at Time of Drilling

▼ Water Level at End of Drilling

▼ Water Level After Drilling as Indicated on Test Hole Logs

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

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:

Descriptive Terms	SPT (N) (Blows/300 mm)
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:

Descriptive Terms	SPT (N) (Blows/300 mm)
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:

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

Sub-Surface Log

Test Hole TH12-02

Client:	Dillon Consulting	**Project Number:**	0022 005 01
Project Name:	Falcon River Diversion and Shoal Lake Aqueduct Bridges	**Location:**	UTM 15 N-5497152, E-341230 (FRD)
Contractor:	Maple Leaf Drilling	**Ground Elevation:**	324.06 m
Method:	125 mm Solid Stem Auger, Acker Renegade Track Mount	**Date Drilled:**	24 October 2012 - 25 October 2012

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)	Bulk Unit Wt (kN/m³) 16 17 18 19 20 21 / Particle Size (%) 0 20 40 60 80 100 / PL MC LL	Undrained Shear Strength (kPa) 0 50 100 150 200 250 / Test Type: Torvane, Pocket Pen., Qu, Field Vane
	0		SILT - clayey, trace fine sand, trace gravel, trace organics (rootlets <5 mm diam.), trace oxidation, trace fine grained sand laminations (<1 mm thick) - light brown - moist, firm to very stiff - low to intermediate plasticity		G04			
	1				T05			
	2		-5 mm thick fine sand seam at 1.8 m		T06			
	3				T07			
					T08			
	4				G09			
319.5			CLAY - silty, trace fine to medium sand, trace gravel, trace oxidation, trace organics (rootlets), trace fine grained sand laminations (<1 mm thick) - grey - moist, firm to stiff - intermediate plasticity, laminated		T10			
	5							
	6		-firm and high plasticity below 6.1 m		T11			
	7							
	8		-soft to firm below 7.6 m		T12			
	9							
					T13			
	10							
	11		-homogenous below 10.7 m		T14	15.3		

Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

SUB-SURFACE LOG AQUEDUCT TEST HOLE LOGS.GPJ TREK GEOTECHNICAL.GDT 25/3/13

Sub-Surface Log

Test Hole TH12-02

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
311.3					T15	
	13		SILT (Till) - clayey, trace coarse sand, trace gravel (<15 mm diam.) - grey - moist, soft - intermediate plasticity			
	14				T16	
309.0	15		AMPHIBOLITE (Bedrock) - grey green, fine grained - strong to very strong (R4-R5) - homogenous		C17	92
	16					
	17				C18	98
306.4						

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21

Particle Size (%): 0 20 40 60 80 100 — PL MC LL — 0 20 40 60 80 100

Undrained Shear Strength (kPa)

Test Type: △ Torvane △ ✿ Pocket Pen. ✿ ☒ Qu ☒ ○ Field Vane ○

0 50 100 150 200 250

END OF HOLE AT 17.7 m IN AMPHIBOLITE

Notes:

1) Water level was 2.9 m below ground surface during drilling.
2) Test hole stayed open to 14.5 m.
3) Drilling method switched to NQ coring below 13.7 m.
4) Could not obtain sample of SILT (Till) below 13.7 m due to drilling method.

Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

Sub-Surface Log

Test Hole TH12-03

Client:	Dillon Consulting	**Project Number:**	0022 005 01
Project Name:	Falcon River Diversion and Shoal Lake Aqueduct Bridges	**Location:**	UTM 15 N-5497133, E-341195 (FRD)
Contractor:	TREK Geotechnical	**Ground Elevation:**	325.85 m
Method:	50 mm Hand Auger	**Date Drilled:**	24 October 2012 - 24 October 2012

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)	Bulk Unit Wt (kN/m³) 16 17 18 19 20 21 / Particle Size (%) 0 20 40 60 80 100 / PL MC LL	Undrained Shear Strength (kPa) 0 50 100 150 200 250 / Test Type: △ Torvane △, ✪ Pocket Pen. ✪, ☒ Qu ☒, ○ Field Vane ○
325.9	0		SILT (Fill) - clayey, trace fine sand, trace gravel, trace organics (rootlets <5 mm diam.), trace oxidation, trace fine grained sand laminations (<1 mm thick) - light brown - moist, firm to stiff - low plasticity		G01			
					G02			
	1				G03			
					G03a			
	2				G03b			
					G03c			
322.9					G03d			

END OF HOLE AT 3.0 m BELOW GROUND SURFACE

Notes:
1) Test hole was dry one day after hand augering
2) Test hole stayed open to 2.6 m one day after hand augering.

Logged By: Tom Hildahl **Reviewed By:** Brent Hay **Project Engineer:** Ken Skaftfeld

SUB-SURFACE LOG AQUEDUCT TEST HOLE LOGS.GPJ TREK GEOTECHNICAL.GDT 25/3/13

Appendix A-2

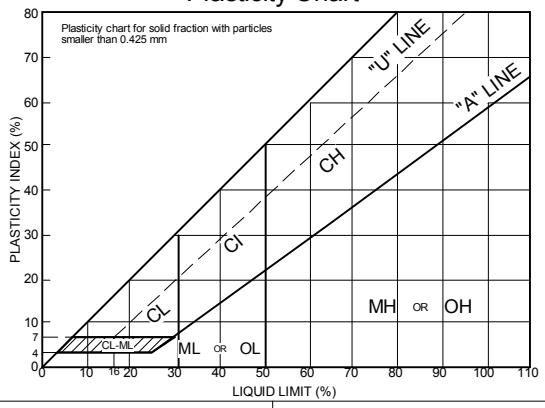
Test Hole Logs (2013 Sub-surface Investigation)

EXPLANATION OF FIELD AND LABORATORY TESTING

GENERAL NOTES

1. Classifications are based on the United Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
2. 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.
3. 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
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 case4s 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
			GP		Poorly-graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW
		Gravel with fines (Appreciable amount of fines)	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
			SP		Poorly-graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW
		Sands with fines (Appreciable amount of fines)	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)	Silts 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
			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	
	Silts 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



Particle Size			Material
ASTM Sieve sizes	mm		
#10 to #4	2.00 to 4.75		Sand
#40 to #10	0.425 to 2.00		Coarse
#200 to #40	0.075 to 0.425		Medium
< #200	< 0.075		Fine
			Silt or Clay

Particle Size			Material
ASTM Sieve Sizes	mm		
> 12 in.	> 300		Boulders
3 in. to 12 in.	75 to 300		Cobbles
3/4 in. to 3 in.	19 to 75		Gravel
#4 to 3/4 in.	4.75 to 19		Coarse
			Fine

* 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

EXPLANATION OF FIELD AND LABORATORY TESTING

LEGEND OF ABBREVIATIONS AND SYMBOLS

LL - Liquid Limit (%)
PL - Plastic Limit (%)
PI - Plasticity Index (%)
MC - Moisture Content (%)
SPT - Standard Penetration Test
RQD - Rock Quality Designation
Qu - Unconfined Compression
Su - Undrained Shear Strength
VW - Vibrating Wire Piezometer
SI - Slope Inclinometer

▽ Water Level at Time of Drilling

▼ Water Level at End of Drilling

▼ Water Level After Drilling as Indicated on Test Hole Logs

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

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:

Descriptive Terms	SPT (N) (Blows/300 mm)
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:

Descriptive Terms	SPT (N) (Blows/300 mm)
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:

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

Sub-Surface Log

Test Hole TH13-01
1 of 1

Client:	Dillon Consulting Ltd	**Project Number:**	0022 009 00
Project Name:	Detailed Design For Falcon River Diversion Channel Bridge	**Location:**	5497101.25 m N, 341207.50 m E, Zone 15 U
Contractor:	TREK Geotechnical Inc.	**Ground Elevation:**	325.05 m
Method:	50 mm Hand Auger	**Date Drilled:**	September 4, 2013

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

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

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
	0.5	SILT (FILL) - trace to some clay, trace fine sand, trace organics (roots), trace oxidation - light brown to brown - moist to wet - soft	G	G01	
	1.0	- trace to some fine sand below 0.8 m			
	1.5	- compact to dense below 1.2 m	G	G02	
323.2	2.0	SILT - trace to some clay, trace fine sand, trace organics (rootlets), trace oxidation - light brown to brown - moist to wet, firm to soft	G	G03	
	2.5	- soft with increasing moisture below 2.5 m	G	G04	
322.0	3.0	- firm and free water (pat test) below 2.9 m SILT - clayey, trace fine sand, trace organics (rootlets), trace oxidation - brown - moist to wet - stiff - trace seepage	G	G05	
	3.5		G	G06	
	4.0				
	4.5	- increasing stiffness with depth below 4.5 m	G	G07	
	5.0				
	5.5		G	G08	
319.3	6.0	CLAY - silty, trace fine sand, trace oxidation - grey - moist to wet - firm to stiff, intermediate plasticity	G	G09	
318.5	6.5		G	G10	

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21
Particle Size (%): 0 20 40 60 80 100
PL MC LL: 0 20 40 60 80 100
Undrained Shear Strength (kPa): 0 50 100 150 200 250
Test Type: △ Torvane △ ✤ Pocket Pen. ✤ ☒ Qu ☒ ○ Field Vane ○

END OF HOLE AT 6.5 m IN CLAY

Notes:
1) Seepage observed below 2.9 m.
2) No sloughing observed.
3) Water level at 2.6 m below ground surface 1/2 hr after drilling.
4) Standpipe piezometer SP13-01 installed upon completion.
5) Water level in standpipe piezometer at 2.56 m below ground surface 1-2 weeks after installation.

Logged By: Martial Lemoine **Reviewed By:** Ken Skaftfeld **Project Engineer:** Ken Skaftfeld

SUB-SURFACE LOG LOGS 2014-10-09 FALCON RIVER BRIDGE 0_A_ML_02 0022 009 00.GPJ TREK GEOTECHNICAL.GDT 10/17/14

Sub-Surface Log

Test Hole TH13-02

Client:	Dillon Consulting Ltd	**Project Number:**	0022 009 00
Project Name:	Detailed Design For Falcon River Diversion Channel Bridge	**Location:**	5497118 m N 0341237 m E, Zone 15 U
Contractor:	Maple Leaf Drilling	**Ground Elevation:**	324.33 m
Method:	125 mm Solid Stem Auger, Acker Renegade Track Mount	**Date Drilled:**	November 2, 2013

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
	0		SILT - some clay, trace coarse sand, trace organics (woody), trace oxidation - light brown - moist, firm - low to non plastic		G11	
	0.5					
	1.0					
	1.5				G12	
	2.0					
321.9	2.5		- clayey below 2.4 m			
	3.0				G13	
	3.5					
	4.0		-seepage and soft below 4.0 m			
	4.5				G14	
319.5	5.0		CLAY - silty, trace fine to medium sand, trace gravel, trace oxidation, trace organics (rootlets), trace fine grained sand laminations (<1 mm thick) - grey - moist, firm to stiff - intermediate plasticity, laminated			
	5.5					
	6.0				G15	
	6.5		-dark grey, firm and high plasticity below 6.4 m			
	7.0					
	7.5				G16	

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21

Particle Size (%): 0 20 40 60 80 100

PL MC LL: 0 20 40 60 80 100

Undrained Shear Strength (kPa): 0 50 100 150 200 250

Test Type: △ Torvane △ | ✤ Pocket Pen. ✤ | ☒ Qu ☒ | ○ Field Vane ○

Logged By: Martial Lemoine **Reviewed By:** Ken Skaftfeld **Project Engineer:** Ken Skaftfeld

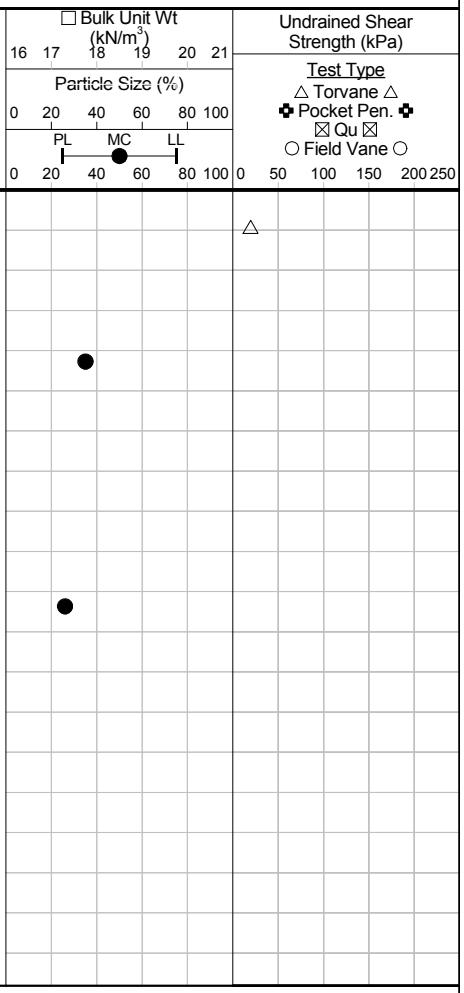
Sub-Surface Log

Test Hole TH13-02

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
315.6	8.5					
	9.0		SILT (Till) - clayey, trace coarse sand, trace gravel (<15 mm diam.), trace cobble or boulders - light grey - wet, soft - low plasticity		G17	
	9.5					
	10.0					
313.7	10.5				G18	
	11.0		AMPHIBOLITE (Bedrock) - grey green, fine grained - strong to very strong (R4-R5) - homogenous			
	11.5					
	12.0					
311.4	12.5					

END OF HOLE AT 13.0 m IN AMPHIBOLITE

Notes:
1) Water level was 4.3 m below ground surface during drilling.
2) Sloughing below 5.8 m.
3) Drilling method switched to NQ coring below 10.0 m.



Logged By: Martial Lemoine **Reviewed By:** Ken Skaftfeld **Project Engineer:** Ken Skaftfeld

Sub-Surface Log

Test Hole TH13-03

Client:	Dillon Consulting Ltd	**Project Number:**	0022 009 00
Project Name:	Detailed Design For Falcon River Diversion Channel Bridge	**Location:**	5497116 m N 0341239 m E, Zone 15 U
Contractor:	Maple Leaf Drilling	**Ground Elevation:**	324.33 m
Method:	125 mm Solid Stem Auger, Acker Renegade Track Mount	**Date Drilled:**	November 2, 2013

Sample Type: Grab (G) | Shelby Tube (T) | Split Spoon (SS) | Split Barrel (SB) | Core (C)

Particle Size Legend: Fines | Clay | Silt | Sand | Gravel | Cobbles | Boulders

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

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION
	0	SILT - some clay, trace coarse sand, trace organics (woody), trace oxidation - light brown - moist, firm - low to non plastic
321.9	2.5	- clayey below 2.4 m
	3.0	-seepage and soft below 4.0 m
319.5	5.0	CLAY - silty, trace fine to medium sand, trace gravel, trace oxidation, trace organics (rootlets), trace fine grained sand laminations (<1 mm thick) - grey - moist, firm to stiff - intermediate plasticity, laminated
318.2	6.0	END OF HOLE AT 6.1 m IN SILTY CLAY

Notes:
1) Water level was 4.3 m below ground surface during drilling.
2) Standpipe (SP13-02) was installed at 4.9 m depth with a stickup of 0.9 m.

Bulk Unit Wt (kN/m³): 16 17 18 19 20 21

Particle Size (%): 0 20 40 60 80 100

PL MC LL: 0 20 40 60 80 100

Undrained Shear Strength (kPa): 0 50 100 150 200 250

Test Type: △ Torvane △ | ✤ Pocket Pen. ✤ | ☒ Qu ☒ | ○ Field Vane ○

Logged By: Martial Lemoine **Reviewed By:** Ken Skaftfeld **Project Engineer:** Ken Skaftfeld

Appendix B

Slope Stability Model Outputs

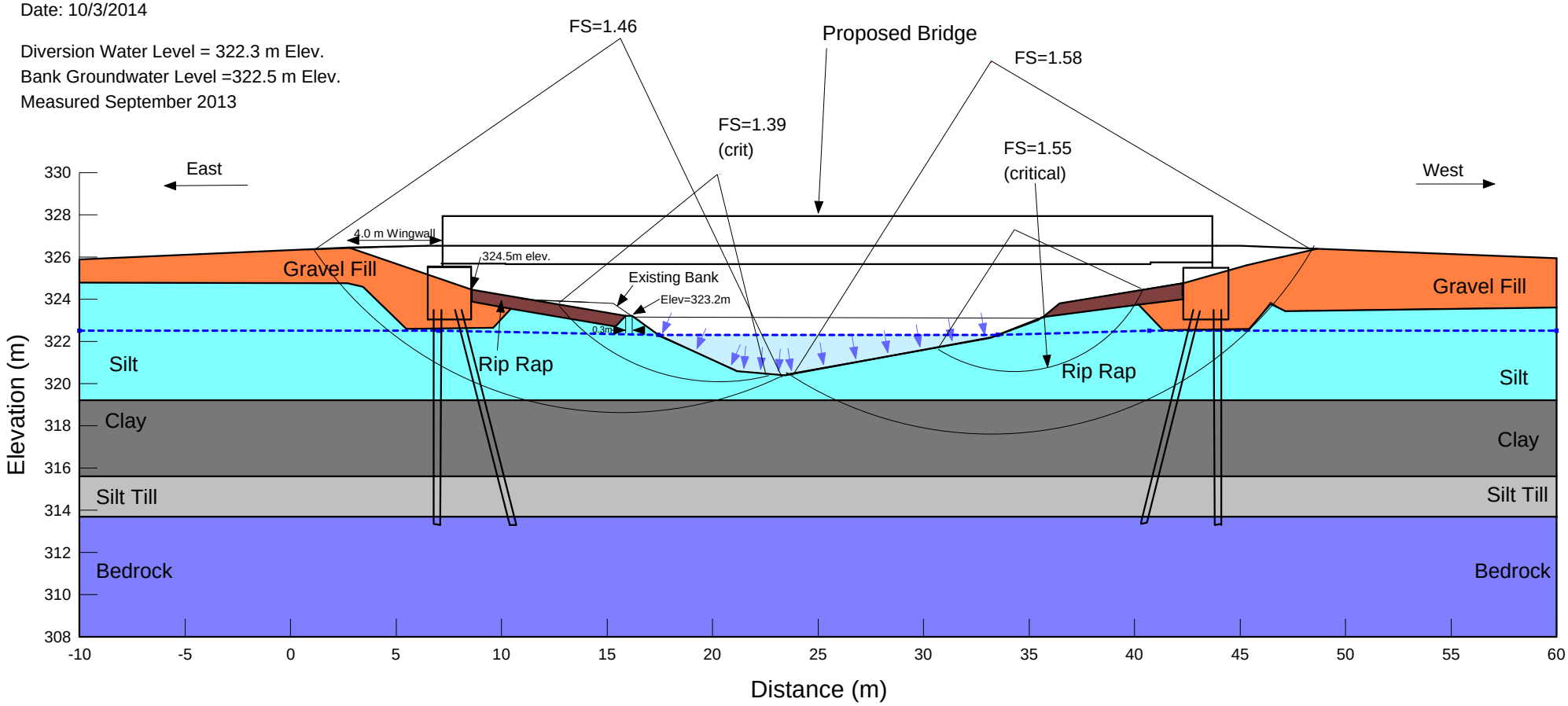
0022 009 00
Dillon Consulting Ltd.
Detailed Design for Falcon River Bridge
Proposed Crossing

120' Bridge Crossing with
Adjusted Rip Rap Geometry

Trek Geotechnical
Date: 10/3/2014

Diversion Water Level = 322.3 m Elev.
Bank Groundwater Level =322.5 m Elev.
Measured September 2013

Name: Clay Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 17 °
Name: Gravel Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 40 °
Name: Silt Unit Weight: 19.5 kN/m³ Cohesion: 2 kPa Phi: 22 °
Name: Bedrock
Name: Silt Till Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 45 °
Name: Rip Rap Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 45 °



File Name: 0022 009 00 FRD Bridge M009.gsz