APPENDIX 'G'

REVISED RECOMMENDATIONS FOR DRIVEN STEEL H-PILES

TREK GEOTECHNICAL Quality Engineering | Valued Relationships

September 27, 2018 File No. 0035-037-00

Beth Phillips, P. Eng., C.I.M.
Morrison Hershfield
59 Scurfield Blvd
Winnipeg, Manitoba
R3Y 1G4

RE Empress Pedestrian Ramp and St. Matthews Retaining Wall
Revised Recommendations for Driven Steel H-Piles

Further to TREK's geotechnical report for structural pedestrian ramps issued on March 13, 2018 to Morrison Hershfield (MH), this letter provides revised recommendations for driven steel H-piles.

As summarized in our previous report for the pedestrian ramps, subsurface conditions encountered in our February 2018 investigation consisted of compact till containing boulders and sand layers overlying bedrock at a depth of approximately 15.0 m. Recommendations were provided for driven steel piles, end-bearing and rock-socketed caissons. Poor till conditions and conflicts between required pile caps and adjacent utilities precluded the use of precast prestressed concrete hexagonal (PPCH) piles or belled piles. The detailed design proceeded with rock-socketed caissons to provide adequate axial and lateral capacity, while avoiding conflicts to nearby utilities and limiting installation vibrations (as shown on the attached preliminary structural drawings).

Proof-coring of the bedrock to confirm adequate rock quality at the proposed socket depth was conducted in September 2018 at three accessible pier locations (SU-1, SU-3 and SU-4) as summarized on the attached proof-core logs; our previous test hole (TH18-01) was located within 2 m of SU-2. The bedrock conditions encountered in the proof-coring holes was significantly different from that observed in TH18-01, consisting of lower strength and poorer quality rock, as well as the presence of what is suspected to be an infilled solution cavity within the bedrock. Core photos for all proof-core holes as well as TH18-01 are attached. The conditions encountered in the proof-core holes consisted of a layer of limestone bedrock 9.3 m, 0.3 m and 1.6 m thick at SU-1, SU-3 and SU-4, respectively, below which suspected sand and clay extended to the depth of exploration in all holes (23.9 m in PC18-01, 20.7 m in PC18-03, 35.7 m in PC18-04). Due to the drilling method used for proof-coring, very little information could be gained on the strength and consistency of suspected sand and clay layers below the upper layers of bedrock. Based on these conditions, TREK does not consider that rock-socketed caissons are feasible at SU-1, SU-3 and SU-4 and driven steel H-Piles are recommended for all piers.

In addition, steel H-piles are now required to support a retaining wall extending south along the cycle track and sidewalk from St. Matthews Avenue. Test holes at this location from our previous investigations did not extend down to bedrock.

Driven Steel H-Piles

Steel H-piles driven to refusal on bedrock are considered suitable to support the proposed ramp structures. However, observed variability of bedrock conditions at this site and the presence of a possible infilled solution cavity within the bedrock may require piles be driven to greater depths prior to reaching adequate capacity or refusal. If refusal is reached on bedrock, H-Piles will derive a majority of their resistance in end bearing with a relatively small contribution from shaft friction. If refusal is not reached, piles driven deeper are expected to derive most of their resistance in shaft friction.

It is our understanding that the desired factored Ultimate Limit State (ULS) capacity for driven steel H-piles (HP310x110 sections) at the pedestrian ramp structure is on the order of 500 kN. We anticipate that piles driven at SU-1 and SU-2 will reach refusal on bedrock at depths of approximately 13 to 15 m below existing ground to achieve a factored ULS capacity of 1,000 kN based on a resistance factor of ϕ=0.4 (Nominal capacity of 2,500 kN). At SU-3 and SU-4, practical refusal may be observed at depths ranging from 17 to 22 m, however the strength and quality of rock at these locations is relatively poor and piles may penetrate through to greater depths. For this reason, we recommend that all steel piles at SU3 to SU-6 be driven with dynamic monitoring using the Pile Driving Analyser (PDA) throughout initial drive and on restrike such that driving can be terminated once a nominal capacity of 1,000 kN is achieved (based on a resistance factor of ϕ=0.5), thus reducing the pile length required.

Piles for the St. Matthews retaining wall can be expected to reach refusal on bedrock, however the depth and quality of bedrock is not known at this location. Geological maps of the area indicate bedrock is likely present at 15 to 20 m below ground surface, however the depth of refusal may vary significantly without site specific information. Steel H-piles (HP310x110 sections) at this location can be designed for a factored ULS Capacity of 1,000 kN, provided they are driven to refusal on bedrock.

The pile head settlement under unfactored service loads can be calculated based on 10 mm or less of pile tip displacement plus elastic shortening of the pile.

Steel H-piles will derive their uplift resistance in skin friction within overburden deposits. An average ULS skin friction of 10 kPa should be used for soils above bedrock for the purposes of uplift resistance calculations.

Design Recommendations

1. The weight of the embedded portion of the pile should be neglected in design.
2. Pile spacing should not be less than 2.5 pile diameters, measured centre to centre. If a closer spacing is required, TREK should be contacted to review the pile layout.
3. The piles must be structurally designed to withstand the design loads, handling stresses, and driving stresses.
4. All piles should be fitted with driving tips to help protect the pile tip during installation. The driving tip must be designed to withstand driving stresses and long-term design load cases.

Installation Recommendations

1. A pile driving system (*i.e.* pile-driving hammer) capable of developing at least 350 J/cm^2 (open-ended diesel hammers) or 250 J/cm^2 (hydraulic hammers) should be specified for driving steel piles. The minimum developed energy for the hammer can be calculated by multiplying this value by the cross-sectional area of the pile in cross-section. For example, an HP310x110 steel H-pile has a cross-sectional area of 141 cm^2 and therefore should be driven with at least 49 kJ of developed energy for a diesel hammer. Developed energy is the potential energy of the ram and can be estimated by measuring the blow rate of the hammer (single-acting diesel hammers), ram velocity or ram drop height. The pile-driving hammer should have the capability of adjusting the fuel setting or stroke to deliver higher energy to the pile during driving if the energy is not sufficient to drive the pile to the required tip elevation. The driving system should also have the capability of adjusting the fuel setting or stroke to deliver lower energy to prevent pile damage upon sudden pile refusal.
2. Piles at the St. Matthews retaining wall and at piers SU-1 and SU-2 of the pedestrian ramps should be driven to refusal on bedrock. Pile installation should be completed carefully near refusal to avoid overdriving of the piles, which could lead to pile damage or misalignment. Refusal is generally considered to be reached when three consecutive sets of 12 blows of the hammer produce 25 mm (1”) or less of pile penetration (per set), provided that a driving system capable of producing the required delivered energy to the pile per blow is used.
3. Piles at SU-3 to SU-6 of the pedestrian ramps should be driven to a nominal capacity of 1,000 kN (or greater) based on dynamic measurements using the PDA and CAPWAP signal-matching analysis at the beginning of restrike (BOR). Restrike testing should be conducted a minimum of 12 hours after the end of initial drive. PDA testing should also be conducted throughout initial drive to monitor driving stresses and field capacities, such that driving can be terminated as soon as the required nominal capacity is anticipated to be reached. If piles are allowed to penetrate through shallower dense materials, they may penetrate to depths of 30 m or greater prior to reaching capacity.
4. Driving stresses in the pile should not exceed 90% of the yield stress of the pile material.
5. The Contractor should be required to submit a proposed driving system for approval a minimum of 7 days prior to the start of pile driving. The pile driving system should be capable of installing the piles to the required tip elevation within specified allowable driving stresses.
6. All piles driven within 5 pile diameters of one another should be monitored for pile heave and where heave is observed, all piles should be checked and piles exhibiting heave should be re-driven to one set of the specified refusal criteria.
7. Pile verticality (plumbness) should be measured on all piles after practical refusal has been achieved to check if verticality is within the limits of the structural design. It is common local practice to specify a maximum acceptable percentage that the pile can be out of vertical plumbness (e.g. 2% out of plumb) or out of the specified batter.
8. Inspection of all driven H-piles should be performed by TREK personnel to confirm that the refusal criteria have been met and to record that pile installation has been completed according to the design.
9. Any piles damaged, out of plumb an excessive amount or reaching premature refusal may need to be replaced. The structural designer will have to assess non-conforming piles to determine if they are acceptable. PDA testing with CAPWAP analysis is recommended for any piles that are suspected to not meet the design capacity or to be damaged if a structural solution is not possible.

10. PDA testing of driven steel piles is considered good practice to verify end-bearing capacity, that piles have been installed without exceeding the permissible driving stresses such that no pile damage occurs and to verify the relationship between driving resistance and capacity. PDA testing is therefore recommended for all piles.

Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practice (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 Morrison Hershfield (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.

If you have any questions, please contact the undersigned.

Kind Regards,

TREK Geotechnical
Per:

2018/09/27
M. J. VAN HELDEN
Member
24729

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

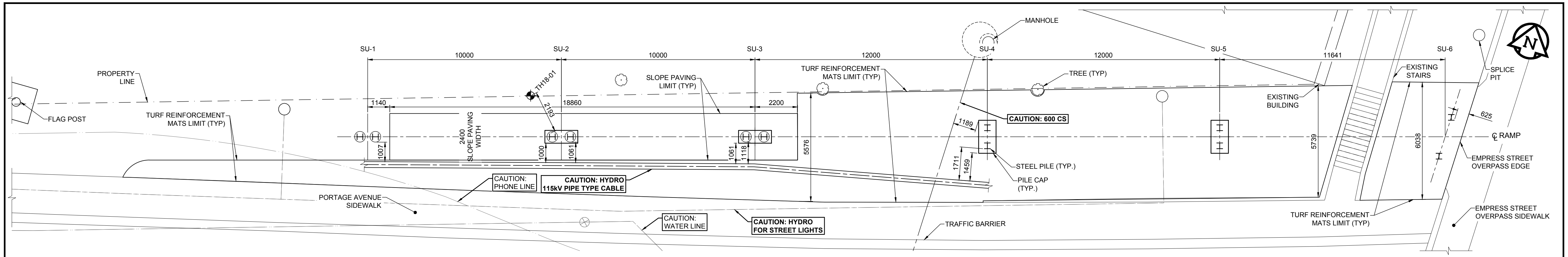
Reviewed By:

For:

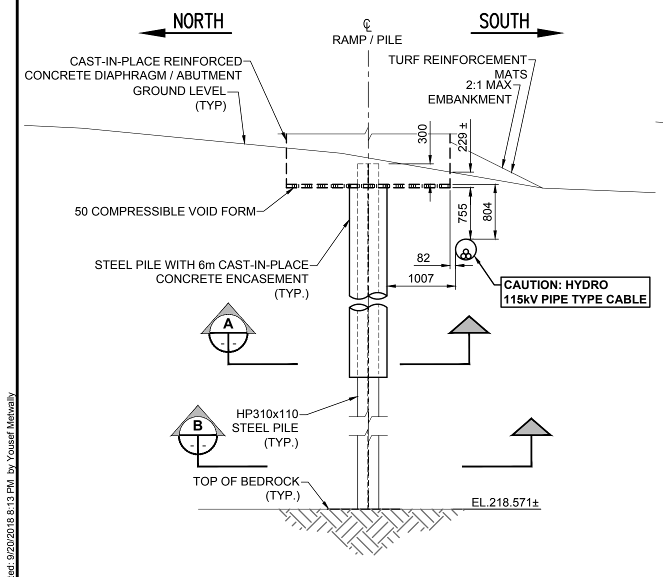
Ken Skaftfeld, M.Sc., P.Eng.
Senior Geotechnical Engineer

ENGINEERS
GEOSCIENTISTS
MANITOBA
Certificate of Authorization
TREK GEOTECHNICAL INC.
No. 4877 Date: Sept. 27/2018

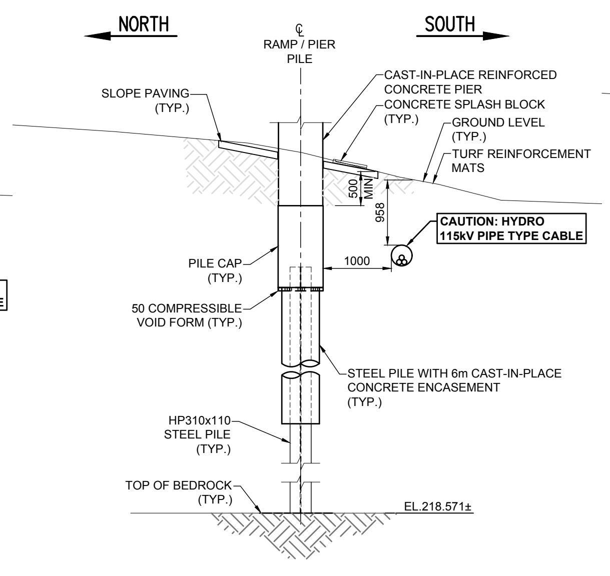
MVH.



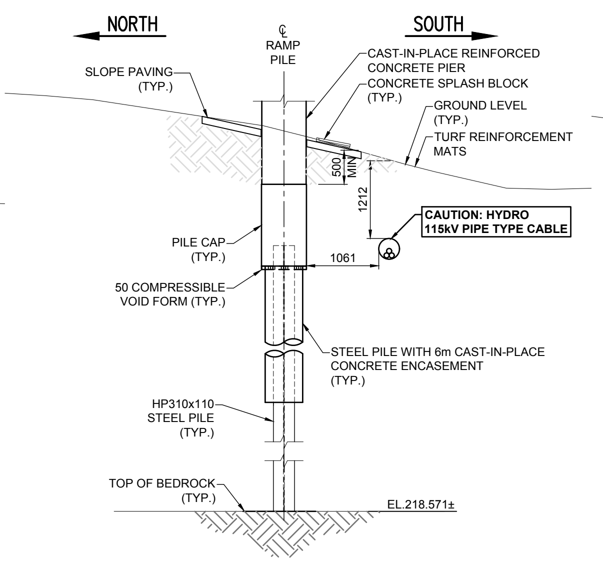
FOUNDATION PLAN
SCALE 1:100



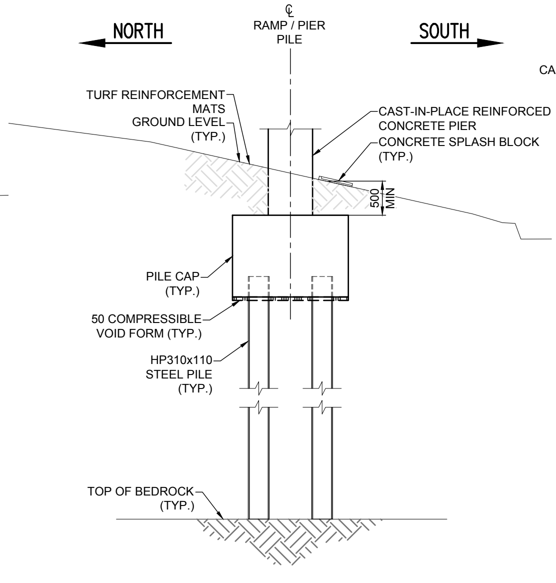
WEST ELEVATION
PILE AT SU-1
SCALE: 1:50



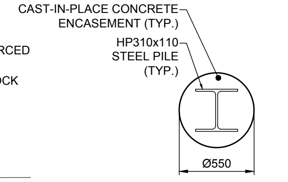
WEST ELEVATION
PILE AT SU-2
SCALE: 1:50



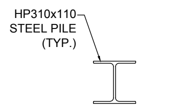
WEST ELEVATION
PILE AT SU-3
SCALE: 1:50



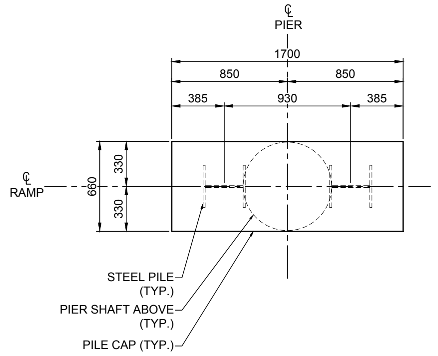
WEST ELEVATION
PILE AT SU-4, SU-5
SCALE: 1:50



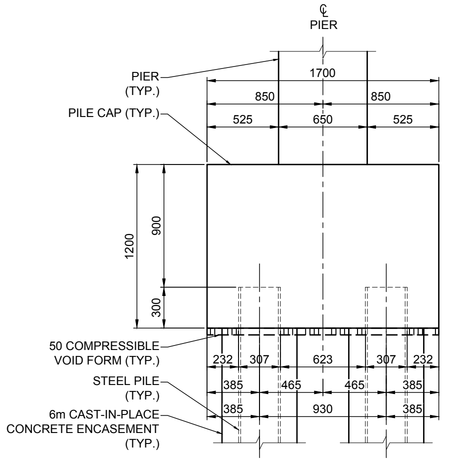
A **SECTION**
SCALE: 1:25



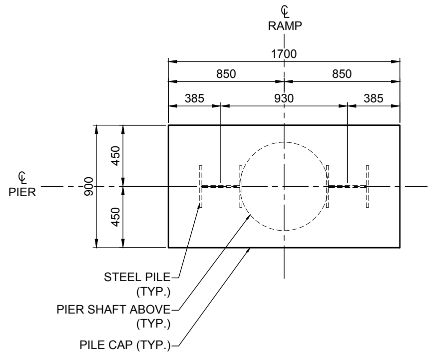
B **SECTION**
SCALE: 1:25



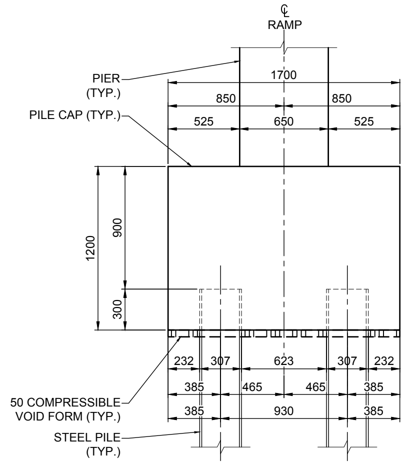
PILE CAP PLAN
PILE AT SU-2, SU-3
SCALE: 1:25



PILE CAP ELEVASION
PILE AT SU-2, SU-3
SCALE: 1:25



PILE CAP PLAN
PILE AT SU-4, SU-5
SCALE: 1:25



PILE CAP ELEVASION
PILE AT SU-4, SU-5
SCALE: 1:25

NOTES:

1. TH REFERS TO BOREHOLE TEST LOG. TEST HOLE TH18-01 WAS TAKEN BY TREK GEOTECHNICAL ON FEBRUARY 23, 2018.
2. CONTRACTOR IS RESPONSIBLE TO CONFIRM THE LOCATION OF ALL UTILITIES / SERVICES IN THE FIELD PRIOR TO CONSTRUCTION AND IS RESPONSIBLE FOR NOTIFYING THE APPROPRIATE COMPANY, DEPARTMENT OR PERSON(S) OF INTENTION TO CARRY OUT ITS OPERATIONS
3. TOP OF BEDROCK ARE BASED ON TEST HOLE TH18-01
4. CONTRACTOR TO CONFIRM THE LEVEL OF TOP OF BEDROCK PRIOR TO CONSTRUCTION AT SU-5 AND SU-6 WITH PROOF-CORE AT PILES LOCATION.
5. TEMPORARY REMOVAL OF EXISTING STAIR HANDRAIL TO ACCESS SU-6 IS ALLOWED.THE CONTRACTOR SHALL SUBMIT HIS COMPLETE CONSTRUCTION STAGING PLAN AND SCHEDULE TO THE CONTRACT ADMINISTRATOR FOR REVIEW AND ACCEPTANCE PRIOR TO PROCEEDING WITH THE WORK.
6. MINIMUM CLEAR DISTANCE BETWEEN SURFACE OF CONCRETE PILE CAP AND 115KV LINE IS 1.0m.

BID OPPORTUNITY No. 602-2018

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
MORRISON HERSHFIELD
No. 1736

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE
DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

BM
ELEV

No.	REVISIONS	YY/MM/DD	BY
C	ISSUED FOR TENDER	18/09/04	BAP
B	ISSUED FOR 95% DESIGN REVIEW	18/08/07	BAP
A	ISSUED FOR 50% DESIGN REVIEW	18/06/28	BAP

MORRISON HERSHFIELD

DESIGNED BY YM
CHECKED BY AF
DRAWN BY ALP
APPROVED BY BAP
HOR SCALE AS SHOWN
VERT SCALE AS SHOWN
DATE
RELEASED FOR CONSTRUCTION N/A
DATE

PROFESSIONAL'S SEAL
A. FATHI
Member
41159

CONSULTANT FILE NAME
W160034 - DD NR H-STEEL PILE.DWG

THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
ENGINEERING DIVISION

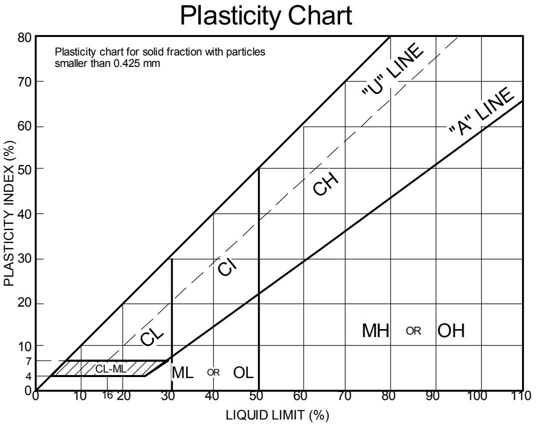
EMPRESS STREET PROJECT
PEDESTRIAN ACCESSIBILITY RAMPS
NORTH RAMP
FOUNDATION DETAILS

CITY DRAWING NUMBER P-3494-140
SHEET 140 OF 168
DRAWING No. 140 REV C

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 Classi-fication	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 border-line 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 border-line 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			
			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	

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

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

* 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

EXPLANATION OF ROCK CLASSIFICATION

(Canadian Foundation Engineering Manual, 4th Edition, 2006)

Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field Estimate of Strength	Examples
R6	Extremely strong	>250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100-250	4-10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite, rhyolite, tuff
R4	Strong	50-100	2-4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium Strong	25-50	1-2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concrete, phyllite, schist, siltstone
R2	Weak	5-25	***	Can be peeled with a pocket knife with difficulty, shallow indentation made by a firm blow with the point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rocksalt
R1	Very weak	1-5	***	Crumbles under firm blows with point of a geological hammer, can be peeled with a pocket knife	Highly weathered or altered rock, shale
R0	Extremely weak	0.25-1	***	Indented by thumbnail	Stiff fault gouge

* Grade according to ISRM (1981).

** All rock types exhibit a broad range of uniaxial comprehensive strengths reflecting heterogeneity in composition and anisotropy in structure. Strong rocks are characterized by well-interlocked crystal fabric and few voids.

*** Rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results under point load testing.

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole PC18-01

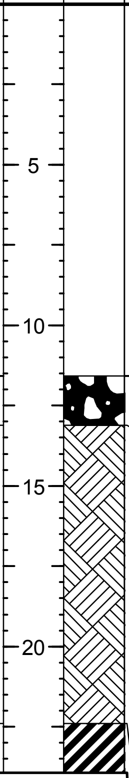
1 of 1

Client: Morrison Hershfield
Project Name: Empress Pedestrian Ramp Proof Coring
Contractor: Rodren Drilling Ltd.
Method: Mobile EF-75 HQ Coring

Project Number: 0035 037 00
Location: 1.6 m north of the centre of SU1
Ground Elevation: 100.00 m Not Measured
Date Drilled: 12 September 2018 - 13 September 2018

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)	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
	OVERBURDEN - soils not logged due to drilling methods used			
88.4	COBBLES AND BOULDERS (suspected granitic and limestone) - 80% core recovery from 11.6 m to 13.1 m	C	C07	
86.9	DOLOMITIC LIMESTONE - cherty, Red River Formation, Upper Fort Garry Member - light brown to cream - vuggy throughout - hard, calcareous, R4 - 100% core recovery from 13.1 m to 14.6 m - clay seams at 15.4 m, 16.8 m and 19.8 m - brecciated from 13.7 to 16.8 m, 18.8 to 19.8 m and 20.9 to 22.3 m - 97% core recovery from 14.6 m to 16.2 m - 100% core recovery from 16.2 m to 17.7 m - 100% core recovery from 17.7 m to 19.2 m - 100% core recovery from 19.2 m to 20.7 m - 73 % core recovery from 20.7 m to 22.3 m - poor recovery and suspected clay seams within bedrock below 20.9 m	C C C C C C	C08 C09 C10 C11 C12 C13	38 10 47 67 22 32
77.6	ARGILLITE- with cherty dolomitic layers, Red River Formation, Upper Fort Garry Member - red, soft, calcareous, R2 - 6% core recovery from 22.3 to 23.8 m - suspected clay below 22.6 m	C	C14	
76.1				

☐ Bulk Unit Wt (kN/m^3): 16 17 18 19 20 21
Particle Size (%): 0 20 40 60 80 100
PL MC LL: 0 20 40 60 80 100

END OF CORING AT 23.9 m IN SUSPECTED CLAY
Notes:
1) Backfilled with cement/bentonite grout mix to surface.

Logged By: Jenna Roadley
Reviewed By: Ken Skaftfeld
Project Engineer: Michael Van Helden

Sub-Surface Log

Test Hole PC18-03

1 of 1

Client:	Morrison Hershfield	**Project Number:**	0035 037 00
Project Name:	Empress Pedestrian Ramp Proof Coring	**Location:**	1.5 m north of the centre of SU3
Contractor:	Rodren Drilling Ltd.	**Ground Elevation:**	100.00 m Not Measured
Method:	Mobile EF-75 HQ Coring	**Date Drilled:**	13 September 2018

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/m^3) 16 17 18 19 20 21 Particle Size (%) 0 20 40 60 80 100 PL MC LL 0 20 40 60 80 100
			OVERBURDEN - soils not logged due to drilling method used				
	5						
	10						
87.8		■	BOULDERS AND COBBLES - 100% core recovery from 12.2 m to 13.1 m - 50% core recovery from 13.1 m to 14.6 m	▯	C16		
				▯	C17		
86.0	15	▨	CLAY - 17% core recovery from 14.6 m to 17.7 m				
82.3							
82.0			DOLOMITIC LIMESTONE - Red River Formation, Upper Fort Garry Member, cherty - light brown to cream, vuggy throughout, hard, R4	▯	C18	20	
	20		SUSPECTED SAND - poor recovery, trace cobbles	▯	C19		
79.3							

END OF CORING AT 20.7 m IN SAND

Notes:

1) Backfilled with cement/bentonite grout mix to surface.

Logged By: Jenna Roadley **Reviewed By:** Ken Skaftfeld **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole PC18-04

1 of 1

Client: Morrison Hershfield

Project Name: Empress Pedestrian Ramp Proof Coring

Contractor: Rodren Drilling Ltd.

Method: Mobile EF-75 HQ Coring

Project Number: 0035 037 00

Location: 1.2 m north of the centre of SU4

Ground Elevation: 100.00 m Not Measured

Date Drilled: 7 September 2018 - 12 September 2018

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

☐ Bulk Unit Wt (kN/m^3): 16 17 18 19 20 21

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

PL MC LL: 0 20 40 60 80 100

Elevation (m)	Depth (m)	Soil Symbol	MATERIAL DESCRIPTION	Sample Type	Sample Number	RQD (%)
	0		OVERBURDEN - soils not logged due to drilling method used - 0.5 m of core recovered from surface to 17.4 m - concrete encountered at 1.5 m (<0.15 m thick)	C	C01	
82.6	17.4		COBBLES AND BOULDERS (Granitic and Limestone) - 50% core recovery from 17.4 m to 18.9 m - 90% core recovery from 18.9 m to 20.4 m - 26% core recovery from 20.4 m to 22.0 m	C	C02 C03 C04	
78.4			DOLOMITIC LIMESTONE - Red River Formation, Upper Fort Garry Member - light brown to cream - vuggy to open cavities - hard, calcareous, R4	C	C05 C06	70
76.8			SUSPECTED CLAY - poor recovery, trace boulders			
68.9			SUSPECTED SAND - poor recovery, trace boulders			
64.3	35					

Depth scale (m): 5, 10, 15, 20, 25, 30, 35

END OF CORING AT 35.7 m IN SUSPECTED SAND

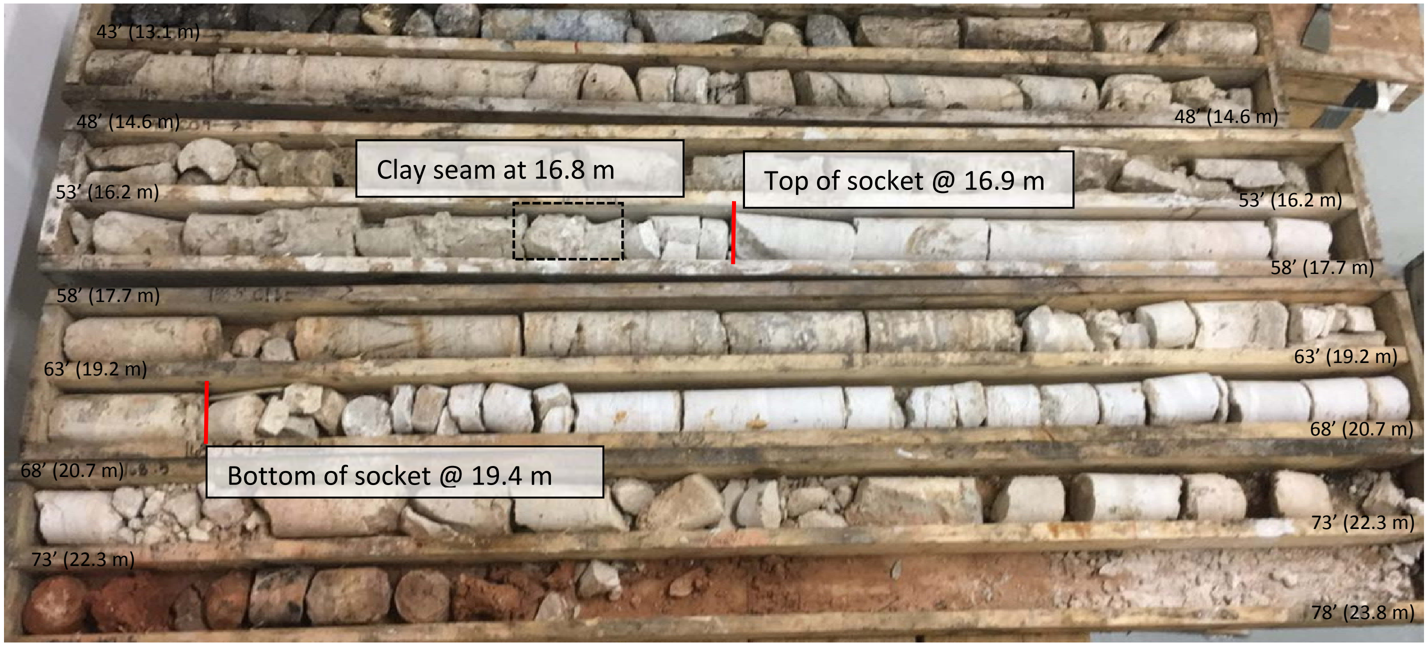
Notes:

1) Casing advanced to 42.7 m on suspected bedrock.
2) Core barrel and two rods abandoned in hole.
3) Backfilled with benotonite to ground surface.

Logged By: Jenna Roadley **Reviewed By:** Ken Skaftfeld **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Empress Pedestrian Ramp
Proof Holes for Caissons – Bedrock Core Photograph
SU1



Clay seam at 16.8 m

Top of socket @ 16.9 m

Bottom of socket @ 19.4 m

Socket: 762 mm diam. X 2.44 m length socket

Note: Top and bottom of rock socket *estimated* based on bedrock core recovery.

Project Number:	**Date:**	**Location:** Structure: **SU1**	**Created By:**	**Reviewed By:**
0035-037-00-104	**September 18, 2018**	Elevation Relative to existing grade.	JR	

Empress Pedestrian Ramp
Proof Holes for Caissons – Bedrock Core Photograph
SU2

Top of socket @ 14.6 m

Core used for compressive strength testing

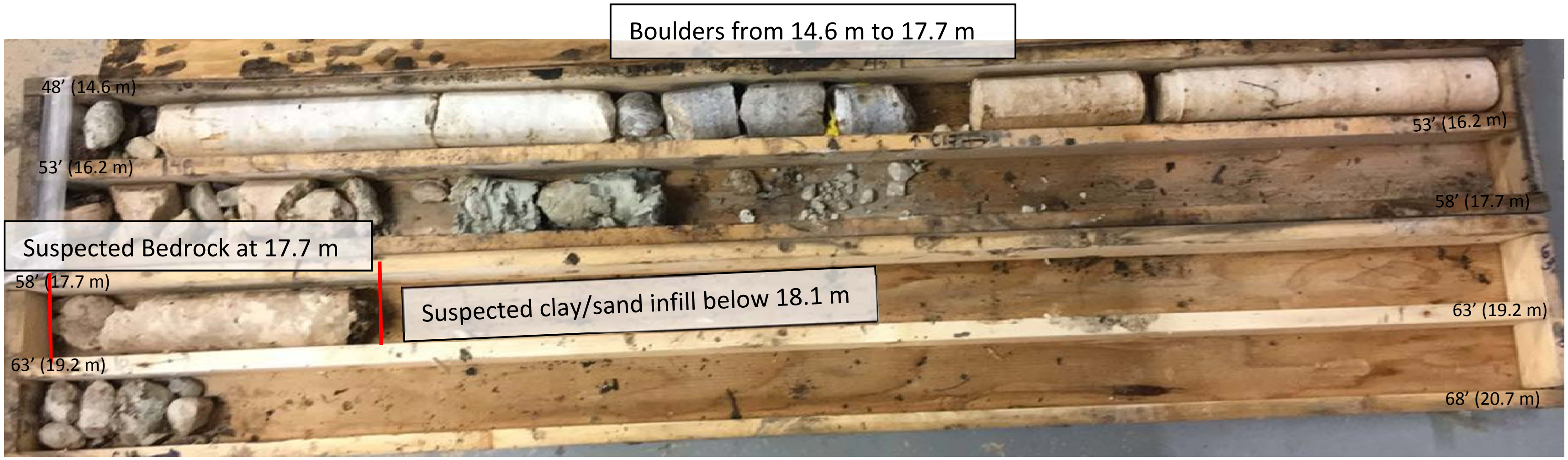
Bottom of socket @ 17.1 m

Socket: 762 mm diam. X 2.44 m length socket

Note: Top and bottom of rock socket *estimated* based on bedrock core recovery.

Project Number: 0035-037-00-104	**Date:** **September 18, 2018**	**Location:** Structure: **SU2** Elevation Relative to existing grade.	**Created By:** JR	**Reviewed By:**

Empress Pedestrian Ramp
Proof Holes for Caissons – Bedrock Core Photograph
SU3



Note: Proof core photos for reference only; suitable bedrock for rock sockets was not identified within depth of proofcore (20.7 m).

Project Number: 0035-037-00-104	**Date:** **September 18, 2018**	**Location:** Structure: **SU3** Elevation Relative to existing grade.	**Created By:** JR	**Reviewed By:**

Empress Pedestrian Ramp
Proof Holes for Caissons – Bedrock Core Photograph
SU4

Note: Proof core photos for reference only; suitable bedrock for rock sockets was not identified within depth of proofcore (24.4 m).

dProject Number:	**Date:**	**Location:** Structure: **SU4**	**Created By:**	**Reviewed By:**
0035-037-00-104	**September 18, 2018**	Elevation Relative to existing grade.	JR	