CITY OF MORDEN

GEOTECHNICAL INVESTIGATION

MORDEN BRIDGES REPLACEMENT

PARKHILL DRIVE

DECEMBER 2022

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1 INTRODUCTION

WSP Canada Inc. (WSP) was retained by City of Morden to complete a geotechnical investigation for the preliminary design of two bridges replacement at the location of Alvey Street and Parkhill Drive in the City of Morden, Manitoba. This report mainly focuses on the subject site at Parkhill drive; the Alvey Street will be prepared in a separate report. The scope of work for the geotechnical investigation included the following:

- One geotechnical test hole
- Laboratory testing
- Geotechnical report

The objective of this geotechnical investigation was to assess the subsurface soil and groundwater conditions at the site in order to provide geotechnical recommendations for future detailed design and construction.

The following report summarizes the field and laboratory testing program, outlines the subsurface conditions encountered at the test hole location, and presents recommendations for: bridge foundation; frost design considerations; abutment backfill and lateral earth pressure and riverbank/channel slope stabilities.

2 SITE AND PROJECT DESCRIPTION

The bridge replacement at the Parkhill Drive is located northwest quadrant of the City of Morden. The existing bridge at the Parkhill Drive is culvert crossing structure and is required to be replaced due to the loss of structural elements (i.e., W-beam guardrails and pedestrian handrail) and separation of the concrete slope paving, cracking, sinking and undermining of the wearing surface and concrete spalls during the flood event.

The bridge opening and span configuration are not available at the time of this report writing, which will be based on the completion of a hydraulic analysis at the subject location. It is anticipated that the proposed bridge will be a single span structure supported by driven steel piles (H-Pile). It is anticipated that the proposed design slope configuration at the bridge location will be relatively consistent with the bank geometry upstream and downstream of the creek to achieve continuity in some aspects of the design.

2.1 Published Geological Information

Based on a review of available surficial geology mapping (Matile and Keller, 2004), alluvial sediments up to 20 m thick are present on site. The sediments reworked by existing rivers and deposited primarily as bars and consist of sand and gravel, sand, silt, clay and organic detritus.

3 METHODOLOGY

3.1 Field Investigation

Prior to the field investigation, WSP completed a Manitoba One-Call and obtained clearances from public utility providers (i.e., MB Hydro, MTS, the City, etc.) for the drilling locations.

WSP oversaw the drilling of one (1) geotechnical test hole (i.e., TH22-02) to the auger refusal of 11.3 m below grade surface (mbgs) that was completed on July 29, 2022. The test hole was drilled using a truck-mounted B40 drill rig equipped with 125 mm diameter solid stem augers and a Standard Penetration Test (SPT) auto hammer, owned and operated by Maple Leaf Drilling Ltd. The test hole was backfilled to grade using the auger cuttings and bentonite upon completion of drilling. Test hole details are provided in Table 1. A site plan showing the test hole locations is provided in Appendix A.

Table 1 Test Hole Details

Test Hole #	Site Location	Completion Date	Auger Refusal Depth (mbgs)	*Northing	*Easting	Approximate Location
TH22-02	Parkhill Drive	July 29/22	11.3	5450228	564639	Northwest of the existing structure

Note: *UTM Coordinates are in NAD 83 Zone 14U

WSP field personnel visually classified the observed soils according to the modified Unified Soil Classification System (USCS) during site drilling. Disturbed soil samples were retrieved from auger flight at a continuous depth at the test hole location. A pocket penetrometer reading was also taken on the cohesive auger samples. In addition, SPTs were performed, split-spoon samples were collected at selected depth using the auto hammer of weight 624 N and drop height 760 mm as per ASTM D1586. The collected samples were labelled with the project name and number, test hole number, date of sampling, sample number and depth of the sample and submitted to WSP in Winnipeg Laboratory for soil testing.

3.2 Laboratory Testing

The following laboratory tests were completed on soil samples collected on site:

- Moisture content tests for all collected samples (10)
- 1 Atterberg limit test and 1 Grain size analysis test (sieve and hydrometer)
- 1 Unconfined compressive strength test
- 2 Bulk unit weight tests

The laboratory test results are discussed in Section 4. The test results are shown on the test hole logs in Appendix B, and the laboratory test sheets are included in Appendix C.

4 SUBSURFACE CONDITIONS

The soil profile encountered at the test hole locations generally consisted of a 80 mm asphalt pavement structure followed by sand fill then native clay overlying clay shale to the depth terminated. A description of the subsurface soil strata is provided in the following sub-sections.

4.1 Asphalt

Asphalt pavement was encountered at the surface with a thickness of approximately 80 mm. The asphalt was described as black, compact, gravelly, and contained some sand.

4.2 Sand Fill

Sand fill was encountered immediately below the asphalt pavement and had a thickness of approximately 220 mm. When encountered, the sand fill was described as brown to dark brown, moist, compact, fine grained, and contained trace gravel, trace silt and trace clay.

4.3 Native Clay

Native clay was encountered below the sand fill and extended to depth of approximately 5.6 mbgs. The clay was described as sandy with some silt to silty, trace gravel, dark brown, moist, medium plastic. Based on pocket pen readings and unconfined compressive strength tests, shear strength values varied throughout the soil strata and ranged from 72 kPa at upper 2.0 mbgs to 24 kPa at 4.0 m to 5.0 m depth, with an average value of 48 kPa. Sand inclusions were encountered at depths between 2.5 m and 5.0 mbgs with water seepage. The moisture content results obtained from the tested samples ranged from 27 percent to 44 percent.

Atterberg limit and particle size analysis tests were conducted on the selected samples and indicated the native clay at the upper surface was considered a medium plastic clay with medium swelling potential. The test results on selected samples are summarized in Table 2 below.

Table 2 Laboratory Test Results for Native Clay

Test Hole #	Sample Depth (mbgs)	Atterberg Limits				Sieve and Hydrometer			
		Plastic Limit (%)	Liquid Limit(%)	Plasticity Index (%)	Plasticity Symbol	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
TH22-02	4.6	20	41	21	SM*	0.7	45.4	27.7	26.3

Notes: *SM – silty sand (400 mm thick) inclusion encountered within the native clay

Unconfined Compressive Strength (UCS) tests were completed on collected Shelby tube samples, which was relatively consistent with pocket pen readings; the results of which are summarized in Table 3.

Table 3 Summary of Unconfined Compressive Strength (UCS) Test Results

Test Hole #	Sample Depth (mbgs)	Soil Type	UCS (kPa)	Undrained Shear Strength Su (kPa)	Moisture Content (%)	Wet Density (kg/m^3)
TH22-02	4.6-5.2	Native Clay	42	21	33.1	1,833

4.4 Clay Shale

Clay shale was encountered at the depth of 5.6 m and extended to the auger refusal. The clay shale was described as silty, trace sand, dark grey to brown, moist. Based on the pocket pen readings, shear strength values between 96 and 192 kPa were obtained throughout the clay shale layer, with SPT 'N' values varying from 33 to 100, indicated very stiff to hard. The moisture content results obtained from the tested samples ranged from 26 percent to 42 percent.

Unit weight tests were also carried out on the selected samples, the results of which are summarized in Table 4.

Table 4 Bult Unit Weight of Clay Shale

Test Hole #	Sample Depth (mbgs)	Soil Type	Moisture Content (%)	Wet Density (kg/m^3)	Dry Density (kg/m^3)
TH22-02	7.5	Clay Shale	28.0	1936	1512

Auger refusal was encountered at depth of 11.3 mbgs, with SPT ‘N’ values of >> 50.

4.5 Groundwater and Sloughing Observations

The water and sloughing conditions were observed and recorded during field drilling. The depth to slough and accumulated water level within each of the test holes are summarized in Table 5 below.

Table 5 Groundwater and Sloughing Observations

Test Hole #	Test Hole Depth (m)	Depth to Slough (m)	Depth of Water Observed at Drilling (m)	Depth to Groundwater upon Completion of Drilling (m)
TH22-02	11.3	10.8	4.6	3.1

Groundwater levels are prone to fluctuations and may be affected by seasonal fluctuations, recent rainfall, surface drainage, and infiltration, etc.

5 GEOTECHNICAL CONSIDERATIONS

This section provides geotechnical recommendations based on WSP’s interpretation of the field and laboratory testing information. The recommendations provided are intended as guidance for planning and design by qualified engineers and architects. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the implementation of the project. Parties requiring information beyond the scope or purpose of this report must contact WSP or make their own interpretation of the information provided.

Where the subsurface conditions encountered during construction are different from stated and/or assumed in this report, WSP should be provided with the opportunity to revise the geotechnical recommendations contained in this report.

Based on the information obtained during our geotechnical investigation, it is prudent to note that there are some geotechnical concerns related to the design and construction of the proposed development, as discussed below.

5.1 Frost Penetration Depth and Frost Heave

The near surface soils on site are considered frost susceptible. The maximum seasonal frost penetration depth was calculated for the near-surface soils using the procedure described in the Canadian Foundation Engineering Manual (CFEM). A mean freezing index of 2000 °C days. The maximum seasonal frost penetration depth is estimated to be 2.4 mbgs. The estimated frost penetration depth assumes a uniform soil type without snow cover.

Based on the encountered soil conditions at the test hole location, the upper clay soil is considered to be medium to high frost susceptible. Therefore, it has a medium to high potential for frost heave in the presence of water and freezing temperatures.

Piles should have a minimum embedment depth of 8.0 m to provide frost heave resistance. In addition, a minimum void space of 150 mm or compressible void form should be applied under all non-bearing surfaces of the pile caps and other structure elements to prevent damage due to uplift pressures and potential swelling of the underlying soils.

5.2 Deformable Native Clay

Long term settlement could be expected and the consolidation settlement analysis should be carried out if the fill thickness requirements at the bridge abutment is greater than 1.0 m. In addition, slope stability review should also be conducted in order to examine the additional fill may have impact on the overall embankment stability. WSP could provide a consolidation assessment after the final site configuration is available to review.

6 FOUNDATION RECOMMENDATIONS

It is understood that driven steel H-Piles are preferred to support the proposed bridge structure. Based on the soil conditions encountered at the subject site, it is considered suitable provided that the driving steel piles would be driven to underlying hard shale or ‘practical refusal’. In this regard, only driven steel H piles will be provided in this report.

The Canadian Highway Bridge Design Code (CHBDC) is referenced for the bridge foundation design. Resistance factors to be used for the determination of factored geotechnical resistance (ULS) following the CHBDC for the deep foundation are summarized in Table 6 below.

Table 6 Resistance Factors for Deep Foundations (CHBDC)

Application		Resistance Factor
Static Analysis	Compression	0.4
	Tension	0.3
Static Test	Compression	0.6
	Tension	0.4
Dynamic Analysis	Compression	0.4
Dynamic Test	Compression (field measurement and analysis)	0.5
Horizontal Passive Resistance		0.5

6.1 Driven Steel Piles

Based on the subsurface conditions encountered at the test hole locations, driven steel H piles can be used to support the proposed bridge structures.

The soil strength contributions in the upper 2.4 m of the subsoil should be ignored due to the effects of soil desiccation and frost heave. The piles should be driven a minimum of 8.0 mbgs to resist the effect of frost uplift. The ad-freeze acting along the pile shaft within the frost zone can be considered as 100 kPa (for steel). The unfactored ultimate limit state (ULS) skin friction and end-bearing resistances for driven piles are summarized in the Table 7 below.

Table 7 Unfactored Skin Friction and End-Bearing for Driven H Piles

Depth (mbgs)	Material	Skin Friction (kPa; Unfactored)	End-Bearing (kPa; Unfactored)
0 to 2.4	Fill and Upper Clay	-	-
2.4 to +/- 6.0	Native Clay (CH)	35	-
+/- 6.0 to +/- 11.3	Clay Shale	70	2,250

The ultimate (unfactored) geotechnical resistance of driven pile can be estimated using the following equation:

$$Q_u = q_s * P_s * L + q_t * A_t$$

Where:

Q_u = unfactored ultimate geotechnical resistance of pile (kN);
q_s = unfactored skin friction (kPa);
P_s = external perimeter of the pile section (m);
L = effective pile embedment length (m);
q_t = unfactored end-bearing (kPa); and,
A_t = cross-sectional area of the steel pile, the full cross sectional rectangular area at the pile toe (m^2).

A resistance factor should be used for compression loading to obtain the factored ULS pile capacity, the geotechnical resistance factor for compression application is outlined in Table 6.

Additional recommendations for driven steel H piles are as follow:

- The recommended minimum pile spacing is three times the pile diameter (3D) as measured from center to center.
- All pile cross-sections must be structurally designed to withstand the design loads and the driving forces during installation.
- The ultimate uplift resistance due to shaft friction can be determined using the unfactored unit shaft rection values outlined in Table 7.
- All piles must be driven continuously once driving is initiated. Where steel H piles are driven to practical refusal in hard shale, the ultimate pile capacity of the steel piles can be designed on the basis of the structural column capacity of the steel pile section rather than the geotechnical design parameters provided in Table 9 above. The ultimate pile capacity may be taken as **$0.60*Fy*A_t$**, where Fy is the yield strength of the steel (typically 350 MPa), and A_t is the full cross-sectional steel area at the toe.
- Practical refusal can be defined as 12 blows per 25 mm penetration using a well-maintained hammer with rated energy of not less than 50 kJ. This could be experienced in the hard clay shale at an anticipated depths from 8.5 m to 11.0 mbgs based on the SPT ‘N’ values encountered at the test hole location. However, the subsurface conditions could vary across the site and may differ from the test hole locations encountered at the site.
- Cobbles and boulders are likely expected during pile installation, and the toe of all driven piles should be equipped with cutting shoe to reinforce the toe of the pile during driving.
- The maximum driving stress must not exceed 90% of the yield strength of the steel for driven piles in order to reduce the potential for structural damage to the pile.
- If pre-drilling is used, pre-boring up to one-third (1/3) of the total pile length within the hard stratum is acceptable, and the pre-bored diameter should be slightly less than the pile size to ensure maximum shaft resistance.
- The elevation of the tops of driven piles should be recorded immediately after driving. This will allow checks for heave due to driving of adjacent piles. If uplift occurs during driving of the adjacent piles, the displaced pile should be re-driven to at least its original embedment depth and final set. Piles should be checked during installation to ensure the vertical piles are within 2% of plumb.

- Full-time inspection by a qualified geotechnical engineer is recommended in order to verify, confirm and record acceptable pile installation.

6.2 Pile Group

Piles may be installed in groups to accommodate heavier loads and a pile cap is placed over the group of piles. The pile cap can be founded on the ground or floated above the ground. Axially-loaded pile groups can act as a block, which may lead to the development of a shaft resistance around the perimeter of the pile group and end resistance at the bottom of the pile-soil block. Thus, a rational approach to estimating the pile group capacity involves the use of the minimum between:

- The sum of individual pile capacities
- The equivalent pile-soil block capacity

The following equations summarize the statement above and can be used to check the pile group capacity for the refined layout.

$$Q_{uG} = \sum_{1}^{n} P_i$$
$$Q_{uB} = q_a A_B + 2q_s(L_B + W_B)H_{eB}$$

Where:

Q_{uG} is the sum of individual pile capacity in the group
P_i is the individual capacity of pile "i"
Q_{uB} is the total capacity of the pile-soil block
q_a is the factored bearing capacity of the pile
A_B is the base area of the pile group
q_s is the factored skin friction of the pile group
L_B and W_B are the block length and widths of the pile group
H_{eB} is the effective embedment depth of the pile group

The factors that influence the pile group response include the method of installation, geometry of the pile group, relative stiffness of pile and the soil, mode of load transfer in the pile, etc. It is desirable to space piles in a group to ensure that the load-bearing capacity of the pile group is not less than the sum of bearing capacity of each pile in the group. Thus, the group efficiency of the pile must be taken into consideration. The group efficiency (η) is defined as:

$$\eta = \frac{Nominal\ load\ bearing\ capacity\ of\ the\ pile\ group}{Sum\ of\ individual\ load\ bearing\ capacity\ of\ each\ pile}$$

As mentioned above, the recommended minimum pile spacing is 3D.

6.3 Pile Downdrag

The downdrag load may be considered due to the additional loading induced by the fill placement and cause movements of soil during pile driving that transferred to the pile itself. Based on the soil conditions encountered at the test hole locations, it is anticipated that the drowndrag have impact on the underlying hard shale is minimal when the steel H piles are driven to hard clay shale or 'practical refusal'. Instead, the upper native clay will be susceptible to downdrag induced by negative shaft friction and the overlying embankment fill, where present. In this regard, the drag load may be determined using a negative unit shaft friction of 35 kPa over the length of the pile in contact with soil at upper 6.0 m from the grade.

6.4 Pile Settlement

The settlement of a single pile depends on so many factors including applied load, strength-deformation properties of the foundation soils, load distribution over the embedded pile length, relative proportions of the loads carried by shaft friction and end bearing, and construction workmanship. For steel piles driven to the hard shale and/or practical refusal, the total pile head settlement is typically governed by 1) toe mobilization settlement and 2) elastic shortening due to compressive load acting on the pile.

The full toe resistance is typically mobilized at pile displacements in the range of 1 to 2 percent of the pile toe diameter and this may be used to estimate the toe mobilization settlement. The elastic settlement due to compression load can be estimated using the equation QL/AE, where, Q = sum of all unfactored applied load (kN); L = pile length (m); A = cross-sectional area of pile (m^2); and E = elastic modulus of pile material (kPa).

6.5 Lateral Loads on Piles

The lateral load carrying capacity and deflection of a pile subjected to a lateral load is dependent on the stiffness of the pile and soil strength. The stiffness of a pile can be calculated using well defined properties of steel; however, the response of soil under loading is subject to some variability.

6.5.1 Lateral Pile Capacity

The lateral load capacities of piles can be estimated using Broms static analysis approach outlined in Section 18.4.1 of CFEM (CFEM, 2006). The Broms solutions for ultimate lateral pile capacity are presented in graphical form in Figures 18.9 and 18.10. The factored ultimate lateral pile capacity can be calculated using the geotechnical resistance factor outlined in Table 6.

It is recommended to consider the use of lateral pile load tests to verify the lateral pile capacity analyses depending on the amount of piles subject to lateral loads, lateral load magnitudes, and importance of structures.

Given that the lateral resistance of a pile is usually developed within the upper 4 m to 6 m of the pile below ground surface, it is important any gaps that may develop during pile installation between the ground and pile be filled to ensure contact between the ground and pile. If not, the lateral resistance of a vertical pile will be reduced significantly. Lateral resistance of the upper 1 m soil layer is recommended to be discounted.

6.5.2 Lateral Pile Deflection

The LPILE program computes deflection, shear, bending moment, and soil response with respect to depth in a nonlinear soil. Soil behaviour is modelled with p-y curves that are generated by the software following published recommendations for various types of soils. These relationships consider the relationship between undrained shear strength and soil modulus, as well as strain at 50% of the maximum stress.

The pile lateral deflection may be analyzed using the software application LPILE. Soil properties that can be used for analyses are summarized in Table 8.

Table 8 **Soil Properties for Analysis of Laterally Loaded Piles**

Soil Strata	Depth (mbgs)	Average Undrained Shear Strength (kPa)	Effective Unit Weight (kN/m^3)	Strain Factor, E_{50}	k_s(Static) MN/m^3
Clay (Firm)	1.0 to 6.0	45	8	0.009	3/d
Clay (Shale)	6.0 to 11.0	145	9	0.005	10/d

d – pile diameter or width

7 LATERAL EARTH PRESSURE

Abutment wing walls and other substructures may be required to resist lateral pressures from the surrounding soils. The lateral earth pressure transferred to bridge abutment and other substructures will be a function of backfill soil type, the degree of compaction of the backfill against the structure, surcharge loading, soil and groundwater conditions. It is recommended that a free draining course granular fill be used as backfill material against the structure to mitigate groundwater accumulation and frost action on the vertical wall within the frost penetration depth. In addition, a perforated drainage pipe connected to a suitable discharge point or weep holes may also be considered if the wall is in excess of 1.5 m to protect against buildup of hydrostatic pressure. The coefficient for the active, at-rest and passive earth pressure for different soil type can be referenced in Table 9 below.

Table 9 **Earth Pressure Coefficients**

Soil Type	Total Unit Weight (kN/m^3)	Active Pressure Coefficient (K_a)	At-Rest Pressure Coefficient (K_o)	Passive Pressure Coefficient (K_p)	Soil Friction Angle (ϕ', Deg)
Granular Fill	20	0.27	0.43	3.69	35
Native Clay	18	0.53	0.69	1.89	18

Note: Earth pressure coefficients provided in the table above assume horizontal grades and a vertical wall with light to moderate compaction

Cohesive soils are not recommended for backfill behind retaining structures. In addition, backfill material against the retaining structure should be conducted with a light, hand operated vibrating plate compactor. Over compacting the backfill material may result in earth pressures that are considerably higher than those predicted in design. Backfilling procedures should be reviewed to confirm the earth pressure coefficients provided in the table above during detailed design.

For soils below the groundwater levels or sub-drainage is not provided behind a wall, effective soil unit weights should be used; this can be determined by subtracting the unit weight of water (10 kN/m^3) from the provided total unit weights.

In addition to the earth pressures, the surcharge loads resulting from point or line loads could also generate lateral stresses, which also need to be considered in the design. The lateral pressure on a wall due to point and line load surcharges can be calculated using the graph presented in Figure 24.8 from CFEM (CFEM, 2006). Where uniformly surcharge loads applied on the retained soils behind the wall, the inducted lateral pressure could be calculated by multiplying the surcharge load by the appropriate earth pressure coefficients as shown in Table 9 above.

8 TEMPORARY EXCAVATIONS

Temporary excavations at the site should be sloped or shored for worker and foundation protection as per Manitoba regulations (Safe Work Manitoba, 2011). According to Manitoba's Guide for Excavation Work, the site soil is to be classified as Category 1; therefore, excavation walls must be sloped 1 (horizontal) to 1 (vertical) from the base of the excavation.

Excavations must be protected from rain, snow, or any ingress of free water. Prolonged exposure of excavated areas should be avoided to prevent deterioration of exposed soil with resultant slope instability. Similarly, excavated materials should be stockpiled away from the excavations to avoid any slope instability and to prevent materials from falling into excavations. Temporary surcharge loads, such as stock of material or heavy equipment, should be kept back from excavation faces a distance equal to at least one-half the excavation depth.

It is anticipated that the depth of the excavation should be no greater than 2.0 mbgs. In this regard, water seepage should not be encountered within the proposed excavations on site. However, groundwater level will be dependent upon weather conditions and the time of year of construction. If seepage is encountered during construction, groundwater may be controlled by sump and pumping methods. During construction, the prepared subgrade surface should be shaped to prevent ponding of water on the site. Excess water should not be allowed to pond and should be drained or pumped from within the construction areas as quickly as possible.

9 EMBANKMENT SLOPE STABILITY

Slope stability analyses were completed to ensure the stability of the new proposed configuration meets the design criteria. The slope stability assessment was completed using the computer program SLOPE/W, a limit-equilibrium slope stability model developed by Geoslope International Ltd. This analysis method compared forces resisting instability against those driving instability and expressed this as a ratio referred to as Factor of Safety (FS).

9.1 Design Criteria

For embankment stability assessment, a FS value of 1.5 is typically considered adequate for long term stability under the normal condition; whereas a minimum FS value of 1.3 is required for short term under extreme condition (i.e., low water levels or empty channel and/or rapid drawdown where water level from spring flood to normal creek water level under short term). The water level and groundwater level are discussed in Section 9.4.

9.2 Proposed Configuration and Methodology

It is understood that the channel slope at the subject location should be relative consistent with the configurations upstream and downstream of the creek to provide continuity for the water flow. In this regard, the slope stability assessment consists of 1) the examination of the existing bank stability based on the slope geometry obtained near the bridge location and 2) with evaluations, a recommended channel slope is to be provided in order to satisfy the design criteria mentioned in Section 9.1.

Based on the survey data provided by KGS Group, two (2) cross sections (i.e., XS-03 & XS-04) at both sides of the creek were evaluated at the upstream and downstream near the subject bridge location to evaluate the slope stability of the existing channel slope. The locations at each cross section taken for the slope stability assessment are outlined in Figure 1, including the cross section profiles shown in Figure 2, outlined in Appendix A.

It is understood that the proposed top of road elevation at the Parkhill Drive is unknown at the time of this report writing. However, the additional fill thickness is anticipated to be no greater than 0.54 m on both sides of the Creek. In this regard, the consolidation settlement assessment is not likely to be required due to the minimal fill placed on the existing grade. If the proposed grade has a significant change (i.e. more than 1.0 m from original proposed grade), WSP should be notified and review both consolidation and slope stability assessment as necessary.

Slope stability assessment was completed using Morgensten-Price circular slip surfaces to estimate the critical FS of Potential Slip Surfaces (PSSs) for each cross section. The model evaluated both normal and extreme conditions to determine the slope stability of the existing channel under the steady-state analysis. The transient analysis is not likely to be required since the creek is considered as a small channel, the impact of the changing water levels on the bank stability is considered minimal.

9.3 Soil Parameters

Post peak shear strength values were used for the native clay soil since there were no signs of cracks or tension failures observed other than soil washed away due to flooding event. The bulk density of all the soil layers was evaluated based on typical density values that we experienced in the slope stability assessment. All soil shear strength parameter values in the model are outlined in Table 10 below.

Table 10 Soil Parameters Used in SLOPE/W Analysis

Soil Layers	Depth (m)	Unit Weight (kN/m^3)	Cohesion C' (kPa)	Friction ϕ' (Degree)
Granular Fill	Upper 0.5 m from Surface	20	0	35
Native Clay (Alluvial)	0.5 - 6.0 m below grade	17	5	18
Clay (Shale)	6.0 - 11.0 m below grade	Impenetrable		

9.4 Groundwater Conditions and Creek Water Levels

The groundwater level is assumed relatively consistent with the creek water level and is assigned to the clay stratum as a groundwater boundary condition in the model.

With respect to the creek water levels, the hydrotechnical assessment report conducted by KGS Group provided the Q2% water level (2% frequency return period flood event) and 3dQ10 water level (10% occurrence of the 3 day delay flow) at the existing bridge location. Since the bridge replacement option is preferred at the site, the water level of Q2% and 3dQ10 under the bridge option is considered in this model setup and used for the 'Flood' condition and 'Normal' condition, respectively. The existing water level obtained at the time of survey is considered as 'Existing' condition. A dewatered creek (Empty Creek Channel) is considered as 'Extreme' condition. The elevations related to the creek water levels at the Parkhill Drive are summarized in Table 11.

Table 11 Creek Water Levels Under Normal and Extreme Conditions

Site Location	Normal Condition (3dQ10)			Existing Condition	Flood Condition (Q2%)			Extreme Condition
	Headwater (m)	Tailwater (m)	Average (m)	Surveyed Creek Water Level (m)	Headwater (m)	Tailwater (m)	Average (m)	Empty Creek Channel (m)
Parkhill Drive	299.32	299.31	299.30	298.70	300.68	300.67	300.70	+/- 297.88

9.5 Slope Stability Results

The results of the slope stability analysis is outlined in Table 12 below. It should be noted that the model only considers the soil weight based on the creek slope geometry, and does not include any foundation loadings since the abutment is to be supported on pile foundations and most of the loads should be transferred to the hard clay shale through the pile. Shallow foundations such as footings are not recommended as it could produce pressures near the upper bank and therefore impact the overall bank stability. If shallow foundation is selected, WSP should be notified and the slope analysis should be reviewed and modified as necessary.

Table 12 Slope Stability Results

Site Locations	Cross Sections		Existing Slope Profile	Critical Factor of Safety (FS)					
				Existing Condition	Normal Condition (3dQ10)	Extreme Condition (Empty Creek)	Flood Condition (Q2%)	Recommended (2.5H:1V)	
								Normal(1)	Extreme(2)
Parkhill Drive	XS3	West Abutment	1.5:1V	1.27	1.32	1.08	1.68	1.51	1.30
		East Abutment	1.7:1V	1.43	1.49	1.21	1.97	1.60	1.34
	XS4	West Abutment	3.5H:1V	1.78	1.85	1.56	>2.0	Not Required	
		East Abutment	5H:1V	1.90	1.95	1.67	>2.0	Not Required	

(1) Based on normal groundwater and creek water level at 299.3 m (3dQ10) and additional fill at Parkhill Drive

(2) Based on normal groundwater levels of 299.3 m (3dQ10), empty channel and additional fill at Parkhill Drive

Table 12 captures the slope configurations encountered near the subject site, and it indicates the existing bank configuration having slope angle of 2H:1V or steeper could necessitate additional slope improvement through bank reshape and/or stabilization measures (i.e., rip-rap placement). The abutment slope should be constructed to a minimum slope angle of 2.5H:1V or flatter if no stabilization works required and meet the stability requirement.

Rip-rap could be placed along the shoreline as per the recommendations provided in the KGS Hydrotechnical report. The use of limestone rip-rap could slightly improve the bank stability, but the majority of its purpose is to minimize shoreline erosion along the shoreline.

The computer modelling output for the recommended slope configuration (i.e., 2.5H:1V) is outlined in Appendix D.

10 COCLUSIONS

Based on the above, WSP concludes the following,

- Driven steel piles are considered suitable to support the proposed bridge structure, provided that the driven steel piles are installed to underlying hard shale or ‘practical refusal’ based on the criteria described in the report herein.
- The existing slope configurations having slope angle steeper than 2H:1V will require some stabilization works and a minimum of 2.5H:1V or flatter is required for the proposed conceptual design without any bank stabilization measures. If shallow foundation is to be selected for any structures support or additional surcharge loading resulting from the additional fill placement WSP should be notified and review the slope assessment as necessary.
- Consolidation settlement should be conducted if the thickness of the additional fill near the abutment approach is greater than 1.0 m, including the slope stability review due to the impact from the additional fill near the abutment headslope.

11 REFERENCES

Matile, G.L.D. and Keller, G.R. 2004: Surficial geology of the Brandon map sheet (NTS 62G), Manitoba; Manitoba Industry, Economic Development and Mines, Manitoba Geological Survey, Surficial Geology Compilation Map Series, SG-62G, scale 1:250000.

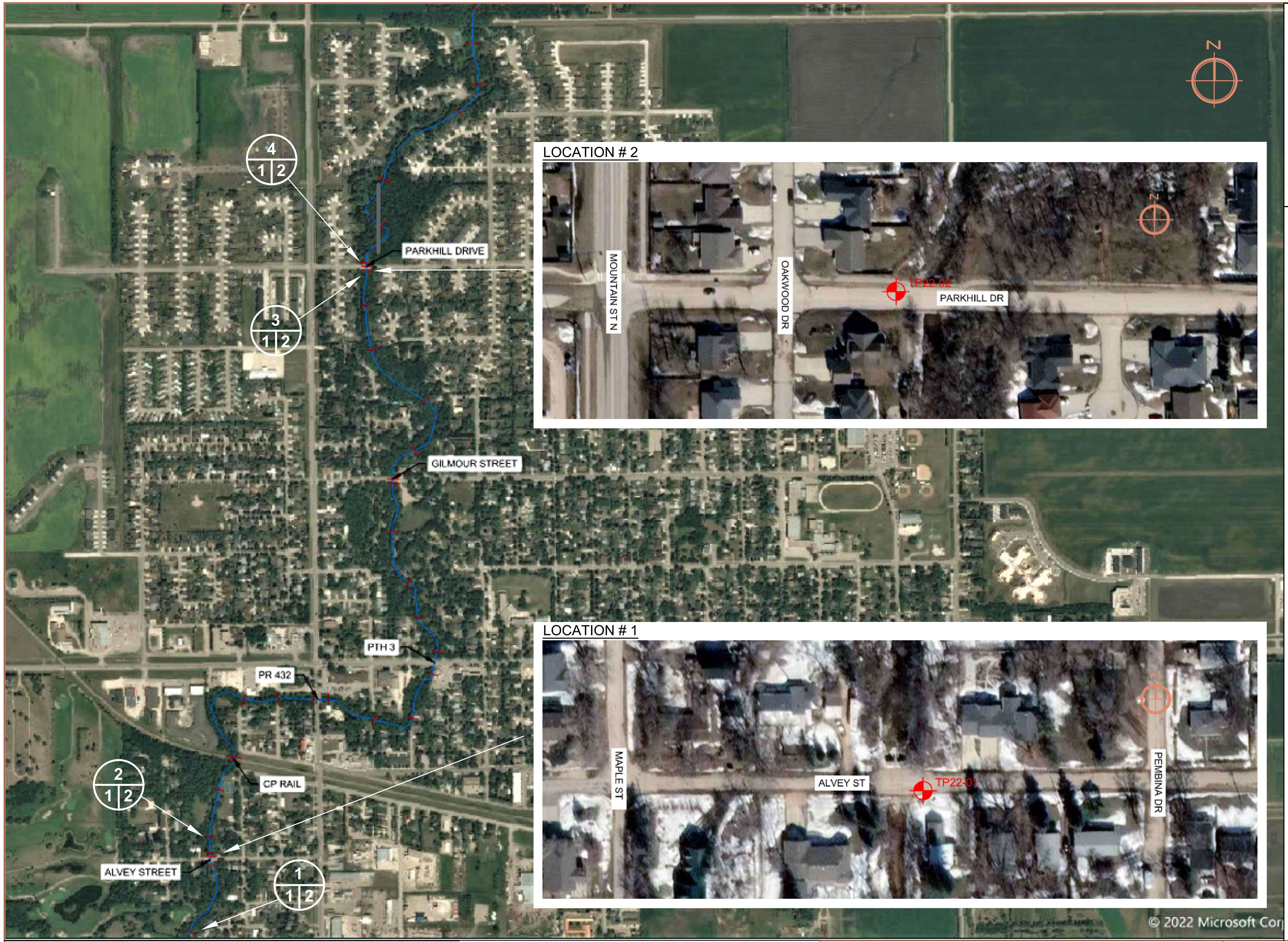
Canadian Geotechnical Society. (2006). Canadian Foundation Engineering Manual. Calgary, Alberta.

Safe Work Manitoba. (2011, May). Retrieved from Excavation Work Guideline: https://www.safemanitoba.com/Page%20Related%20Documents/uploads/guidelines/excavation_guide_updated_2011_web.pdf

APPENDIX

A

SITE PLAN & TYPICAL CROSS SECTION PROFILES



LEGEND

⌖ GEOTECHNICAL TEST HOLES

NOTE

- ALL TEST HOLES PRESENT ON SITE ARE APPROXIMATE LOCATION.

NOTE:
These design documents are prepared solely for the use by the party with whom the design professional has entered into a contract and there are no representations of any kind made by the design professional to any party with whom the design professional has not entered into a contract.

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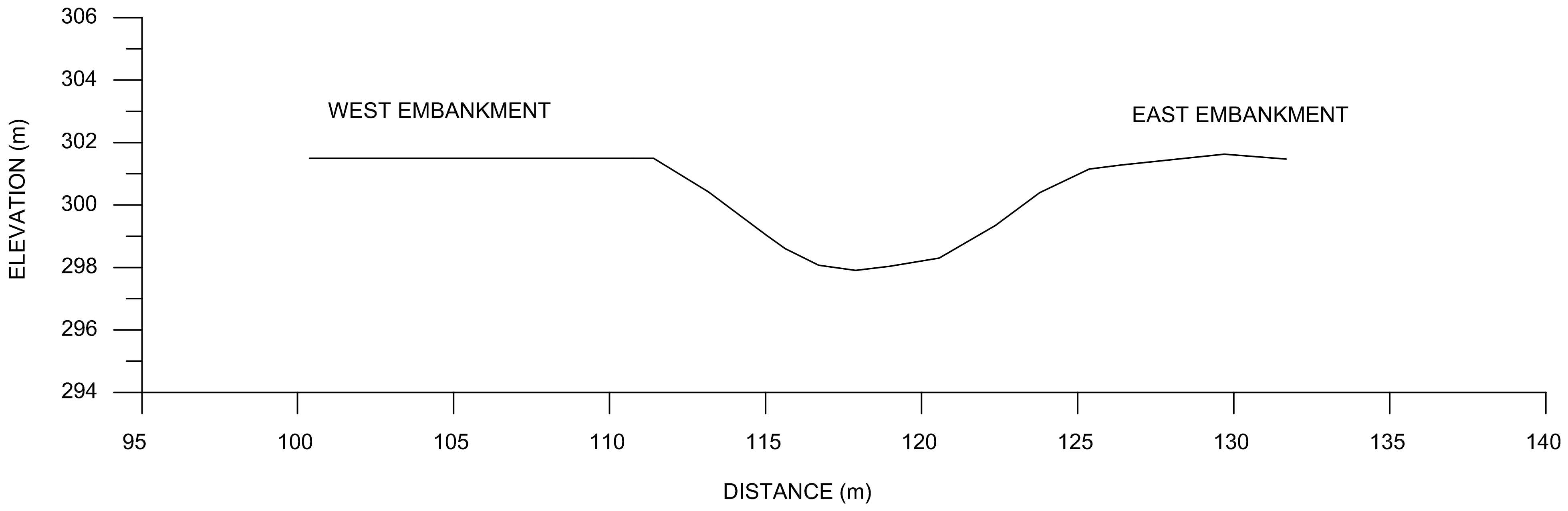
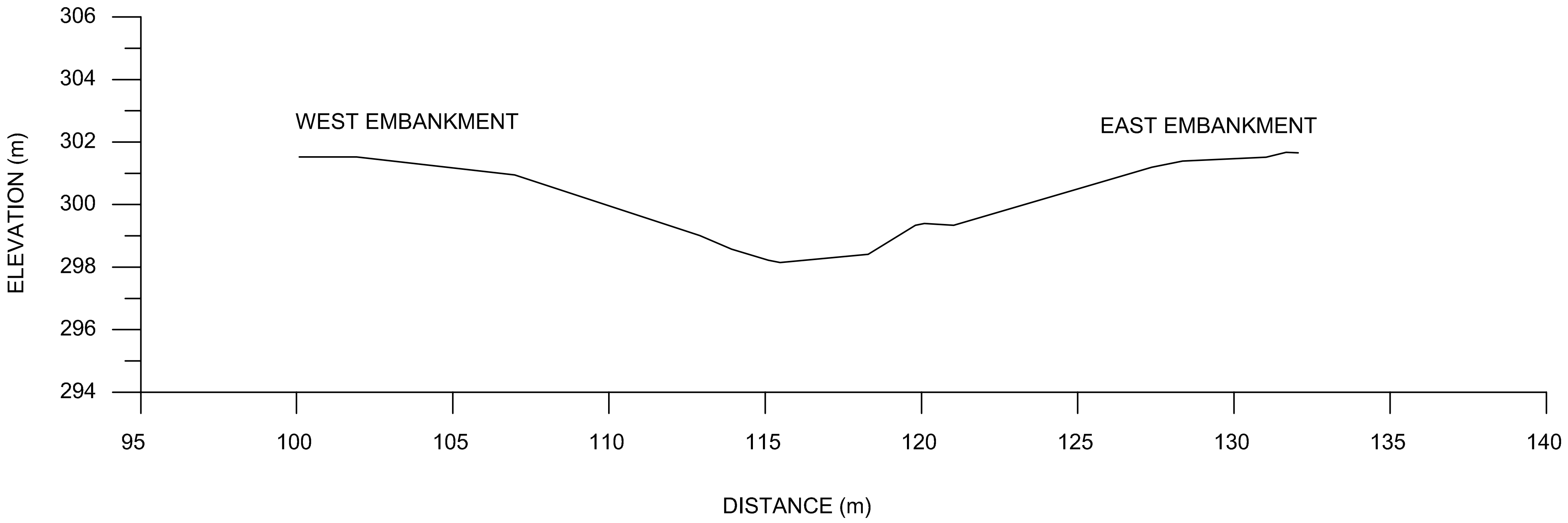
www.wsp.com

GEOTECHNICAL INVESTIGATION
MORDEN BRIDGES - ALVEY ST & PARKHILL DR - MORDEN, MANITOBA
SITE LOCATION PLAN

SCALE:	DATE:	DWG. No.
NTS	OCT 27, 2022	FIG. 1 OF 2



3
1 | 2
CROSS SECTION 3



4
1 | 2
CROSS SECTION 4

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GEOTECHNICAL INVESTIGATION
MORDEN BRIDGES - PARKHILL DRIVE - MORDEN, MANITOBA
CROSS SECTION DETAILS

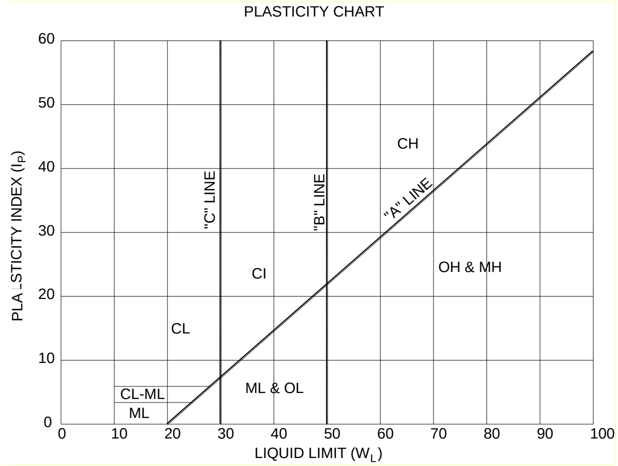
SCALE: 1:200	DATE: NOV. 18, 2022	DWG. No. FIG. 2 OF 2

APPENDIX

B SOIL LOGS

SOIL DESCRIPTION CHART

MODIFIED UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES	LABORATORY CLASSIFICATION CRITERIA		
COARSE GRAINED SOILS (More than half of material is retained in No.200 Sieve (0.075 mm))	GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size (4.75 mm))	CLEAN GRAVELS (<5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine amount of sand and gravel from grained size curve Depending on percent of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: < 5%..............GW, GP, SW, SP > 12%....................GM, GC, SM, SC 5 -12%..................Borderline cases requiring dual symbols**	$C_u = D_{60}/D_{10}$: $C_u \geq 4$ $C_c = (D_{30})^2/(D_{10}xD_{60})$: $1 < C_c > 3$	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradations requirements for GW	
		GRAVELS WITH FINES (> 12% fines)	GM	Silty gravels, gravel-sand-silt mixtures		Atterberg Limits below "A" Line or P.I. < 4	Above "A" Line with P.I. Between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg Limits above "A" Line with P.I. > 7	
	SANDS (More than half of coarse fraction is smaller than No. 4 sieve size (4.75	CLEAN SANDS (< 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = D_{60}/D_{10}$: $C_u \geq 6$ $C_c = (D_{30})^2/(D_{10}xD_{60})$: $1 < Cc > 3$	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradations requirements for SW	
		DIRTY SANDS (>12% fines)	SM	Silty Sands; sand-silt mixtures		Atterberg Limits below "A" Line or P.I. < 4	Limits plotting in hatched zone 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 with P.I. > 7	
FINE GRAINED SOILS (More than half of material pass the No. 200 sieve size (0.075 mm))	CLAYS (Above "A" Line on PLASTICITY CHART: negligible organic content)	W_L<30%	CL	Inorganic clays of low plasticity, gravelly clays, sandy clays, silty clays, lean clays			
		30<W_L <50%	CI	Inorganic clays of medium plasticity, gravelly clays, sandy clays, silty clays			
		W_L>50%	CH	Inorganic clays of high plasticity, fat clays			
	SILTS (Below "A" Line; negligible organic content)	W_L<50%	ML	Inorganic silts and very fine sands, silty or clayey fine sands, clayey silts with slight plasticity			
		W_L>50%	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
	ORGANIC SILTS AND CLAYS (Below "A" Line)	W_L<50%	OL	Organic silts and organic silty clays of low plasticity			
		W_L>50%	OH	Organic clays of medium to high plasticity, organic silts	(NOTE: For soils passing No. 400 sieve & sieve size in µm)		
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils	Strong colour or odor and fibrous textures		

SOIL COMPONENTS

Fraction	U.S. Standard Sieve Size		Percentage (by weight)	Description
	Passing	Retained		
Gravel				
Coarse	76 mm	19 mm	35-50	AND
Fine	19 mm	4.75 mm		
Sand			20-35	Y
Coarse	4.75 mm	2.00 mm		
Medium	2.00 mm	0.425 mm	10-20	SOME
Fine	0.425 mm	0.075 mm		
Fines (Silt or Clay)	0.075 mm or less		0-10	TRACE

Oversize Material	Cobbles	76 mm to 300 mm
	Boulders	> 300 mm

RELATIVE DENSITY AND CONSISTENCY

Cohesionless Soils		Cohesive Soils	
Relative Density	SPT (N) Value	Consistency	Undrained Shear Strength (kPa)
Very Loose	0-4	Very Soft	<12
Loose	4-10	Soft	12-25
Compact	10-30	Firm	25-50
Dense	30-50	Stiff	50-100
Very Dense	>50	Very Stiff	100-200
		Hard	>200

Updated - January 2022

WSP Canada Inc

TH22-02

CLIENT City of Morden

PROJECT NAME Geotech Investigation - Bridge Replacement

PROJECT NUMBER 221-07930-00 & 221-07931-00

PROJECT LOCATION City of Morden, MB

DATE STARTED 7/29/22 **COMPLETED** 7/29/22

GROUND ELEVATION

HOLE SIZE 125 mm

DRILLING CONTRACTOR Maple Leaf Drilling

GROUND WATER LEVELS:

DRILLING METHOD Solid Stem Auger - B40 Truck Rig

AT TIME OF DRILLING ---

LOGGED BY Pan Ding **CHECKED BY** Wei Gao

AT END OF DRILLING ---

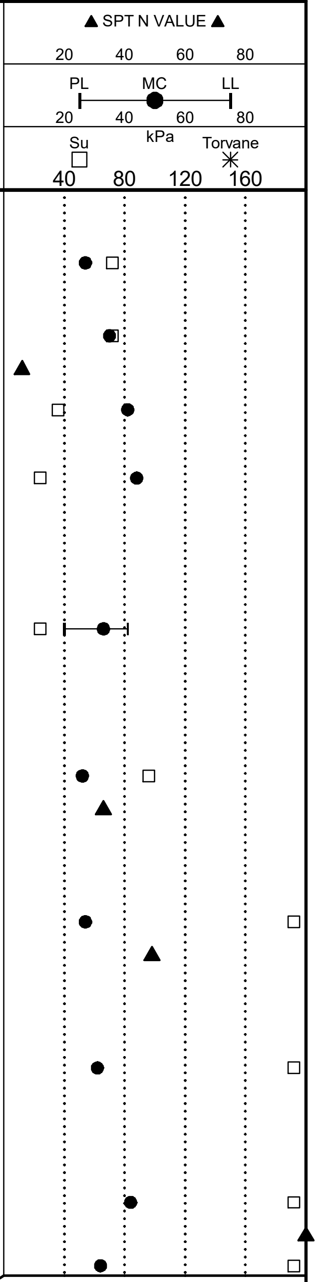
NOTES Parkhill Drive (5450228 N; 564639 E)

AFTER DRILLING ---

DEPTH (m)	GRAPHIC LOG	ELEV. (m)	WATER LEVEL	MATERIAL DESCRIPTION
0				**ASPHALT PAVEMENT (80 mm)** - Black, compact, gravelly, some sand
				SAND FILL (220 mm) - Dark brown, moist, compact, fine grained, trace gravel, trace silt, trace clay
2				**CLAY (CI)** - Sandy, some silt to silty, trace gravel, dark brown, moist, medium plastic - Below 1.5 m, dark grey to dark brown, trace sand pockets, trace wood pieces, trace rootlets - Below 2.3 m, soft, trace oxidized, trace sand pockets
4				- From 4.6 m to 5.0 m, sand inclusions, loose, some clay to clayey, silty, water seepage observed - Particle size analysis results on S5: Gravel (0.7%); Sand (45.4%); Silt (27.7%); Clay (26.3%) - Unconfined Compressive Strength (UCS) obtained on ST1 is 43 kPa
6				**CLAY (SHALE)** - Dark grey to dark brown, moist, silty, trace sand, stiff to very stiff
8				- Bulk unit weight obtained on S7A at 8.0 mbgs is 1936 kg/m3
10				- At 10.7 m, hard

END OF TESTHOLE
- Auger refusal encountered in the clay shale layer at 11.3 mbgs.
- Sloughing observed at 10.8 mbgs upon completion of drilling.
- Water seepage first observed at 4.6 mbgs from the sand inclusions area in the clay layer and measured at 3.1 mbgs upon completion of drilling.
- Testhole backfilled with auger cuttings and bentonite, then patched with cold mix asphalt upon completion of drilling.

SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	Shear Strength Su (kPa)	TORVANE (kPa)	MOISTURE CONTENT (%)
GB S1		72		27
GB S2		72		35
SPT S2A	3-3-3 (6)			
GB S3		36		41
GB S4		24		44
GB S5		24		33
ST ST1				
GB S6		96		26
SPT S6A	9-14-19 (33)			
GB S7		192		27
SPT S7A	14-20-29 (49)			
GB S8		192		31
GB S9		192		42
SPT S9A	30-50-50 (100)			
GB S10		192		32



APPENDIX

C

LABORATORY TESTING SHEET

MOISTURE CONTENT OF SOIL AND ROCK
(ASTM D2216)

Client:	WSP Canada Inc.	**Lab No.:**	22-001-019-S153
Project:	Morden Bridges - Geotechnical Investigation	**Project No.:**	221-07931-00
Site Location:	Project Site	**Report Date:**	Aug 12, 2022
Date Sampled:	Jul 29, 2022	**Date Tested:**	Aug 08, 2022
Sampled By:	PD	**Tested By:**	TL

Test Hole No.	Sample No.	Depth (ft)	Moisture Content (%)
TH02	S1	2.5	27.2
TH02	S2	5.0	34.6
TH02	S3	7.5	40.5
TH02	S4	10.0	43.5
TH02	S5	15.0	33.1
TH02	S6	20.0	25.8
TH02	S7	25.0	27.1
TH02	S8	30.0	30.9
TH02	S9	35.0	42.3
TH02	S10	37.0	32.4

Reviewed by: Bryan Hiebert

Bryan Hiebert, CET

Notice: The test data given herein pertain to the sample provided. Reporting of these data constitutes a testing service. Engineering review and interpretation may be provided upon written request.

ATTERBERG LIMITS
(ASTM D4318)

Client: WSP Canada Inc. **Lab No.:** 22-001-095-S153

Project: Morden Bridges - Geotechnical Investigation **Project No.:** 221-07931-00 (Park

Site Location: Project Site **Report Date:** Aug 12, 2022

Date Sampled: Jul 29, 2022 **Date Tested:** Aug 11, 2022

Sampled By: PD **Date Received:** Aug 02, 2022 **Tested By:** TL

Testhole No.: TH02 **Sample No.:** S5 **Depth (ft):** 15

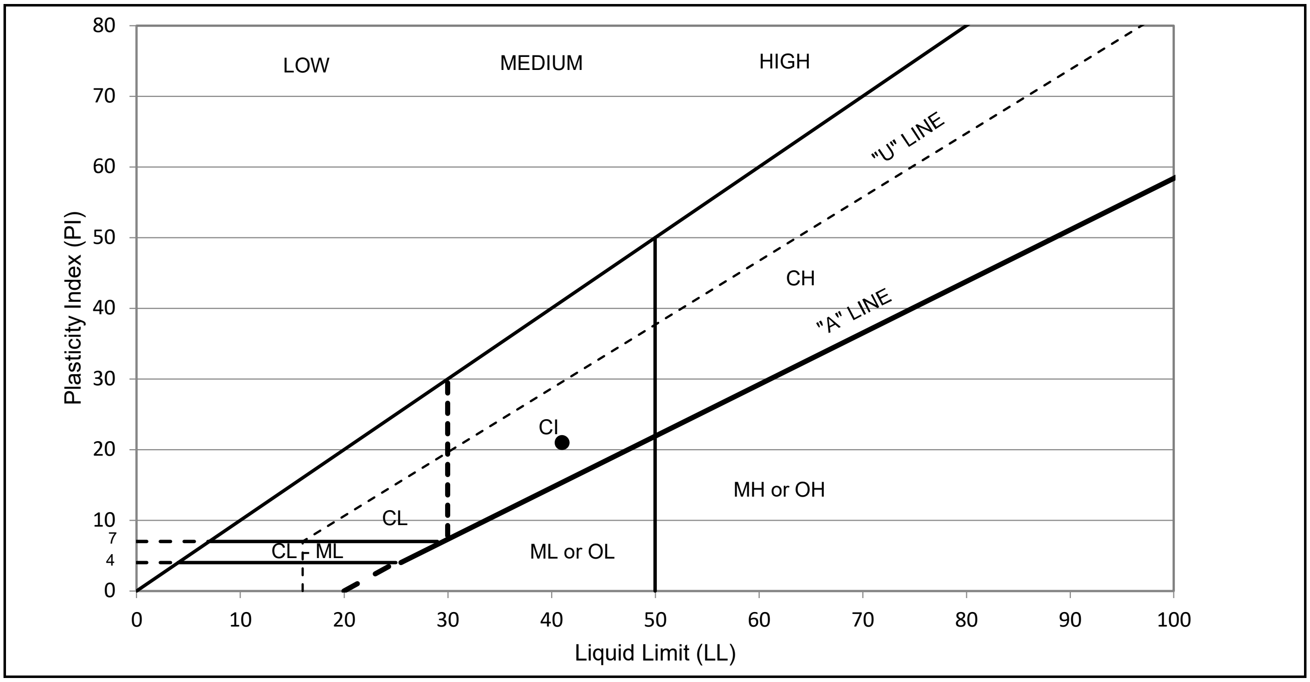
Drying Method: Oven **Method:** Multi-Point

Liquid Limit Test
(Manual, Plastic Grooving tool)

Trial	**A**	**B**	**C**
No. of Blows	16	26	30
Moisture Content (%)	43.6	40.9	40.2

Plastic Limit Test
(Hand rolled)

Trial	**A**	**B**
Moisture Content (%)	20.3	20.2



USCS Symbol CI

LL, Liquid Limit (%) 41

PL, Plastic Limit (%) 20

PI, Plasticity Index 21

Soil Description: Medium Plastic Clay

Reviewed by: Bryan Hiebert

Bryan Hiebert, CET

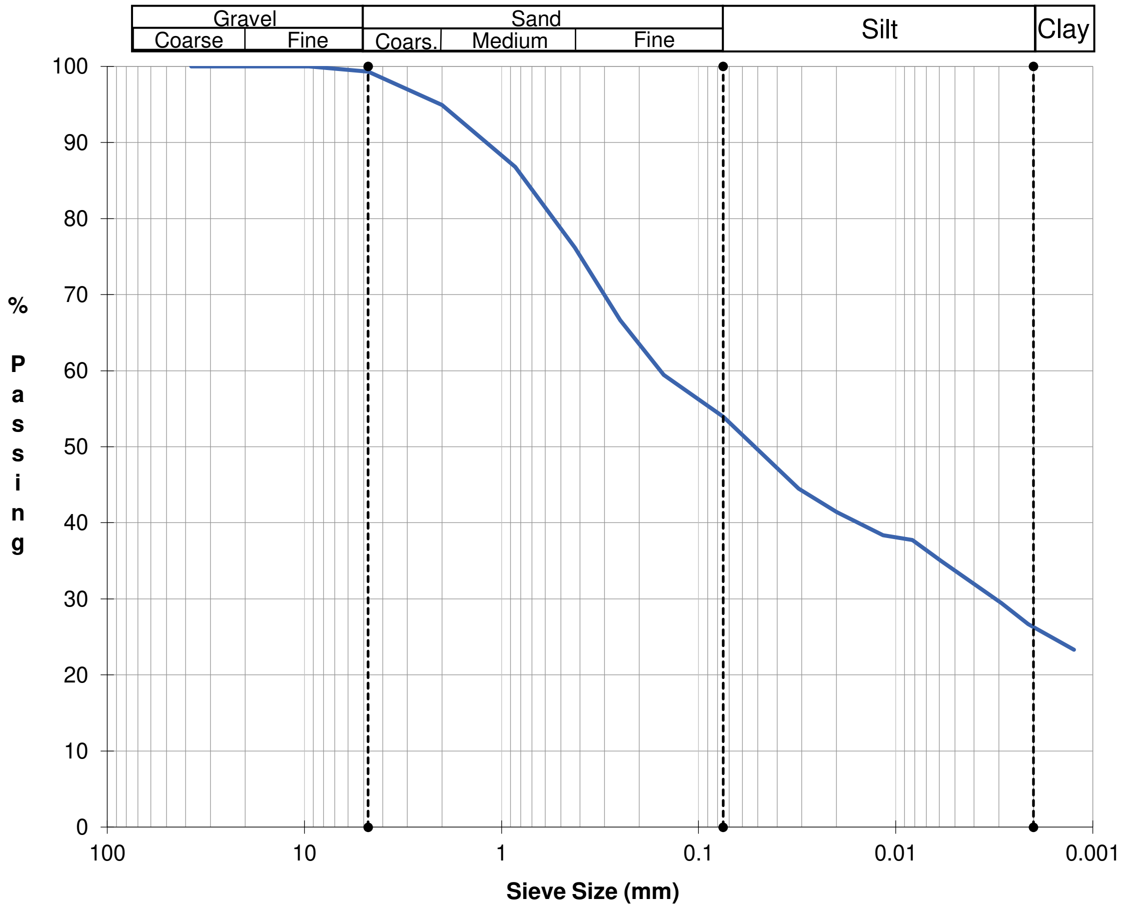
Comment: As received moisture content is 33.1%.

PARTICLE-SIZE DISTRIBUTION OF SOILS USING SIEVE AND HYDROMETER ANALYSIS
(ASTM D6913 and D7928)

Client: WSP Canada Inc.
Lab No.: 22-001-019-S153
Project: Morden Bridges - Geotechnical Investigation
Project No.: 221-07931-00

Testhole No.: TH02
Sampled by: PD
Sample Source: Project Site
Sample No.: S5
Date Sampled: Jul 29, 2022
Date Received: Aug 02, 2022
Depth (ft): 15.0
Sampling Method: Grab
Tested By: PD/TL

Dispersion Method: Stirring
Dispersion Period (min): 1
S.G. (assumed): 2.65



Sieve Size (mm)	Percent Passing (%)
37.5	100.0
25.0	100.0
19.0	100.0
16.0	100.0
9.5	100.0
4.75	99.3
2.00	94.9
0.850	86.8
0.425	76.2
0.250	66.6
0.150	59.4
0.075	53.9
0.031	44.5
0.020	41.4
0.012	38.3
0.008	37.7
0.006	35.0
0.0029	29.5
0.0021	26.7
0.0012	23.3

Percent of: Gravel = 0.7% Sand = 45.4% Silt = 27.7% Clay = 26.3%

Sample Description: Sand, silty, clayey, trace gravel
Remarks: Separation made on No 10 sieve (2.0 mm).

Reviewed by: Bryan Hiebert
Bryan Hiebert, CET

The test data given herein pertain to the sample provided. Reporting of these data constitutes a testing service. Engineering review and interpretation may be provided upon written request.

Unconfined Compressive Strength of Cohesive Soils

ASTM D2166

Client:	WSP Canada Inc	Sampled By:	WG
Project:	Morden Bridges (Parkhill Dr.)	Tested By:	BMH
Job No.:	211-07930-00	Sample Date:	2022-07-29
Report Date:	2022-08-08	Test Date:	2022-08-08
Test Hole No.	TH02	Sample No.:	ST1
Depth (ft)	15' - 17'		

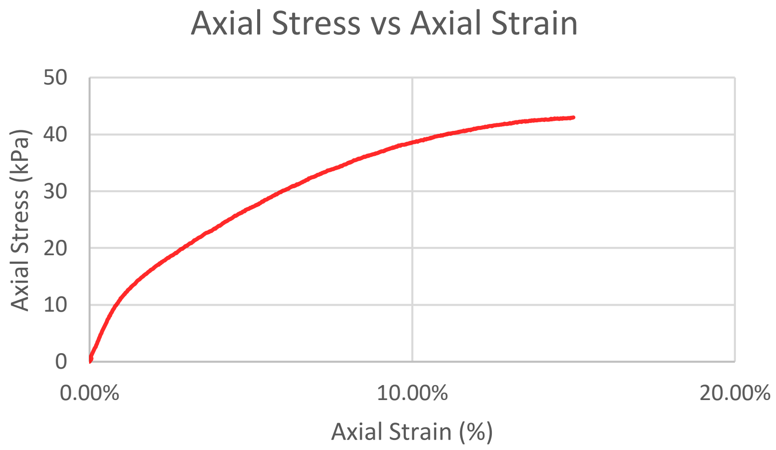
Specimen Info

Soil Description: SAND, silty, clayey, trace organics, moist, mottled grey and black, unsorted, medium plasticity

Sample Type:	Intact	Specimen Wet Density (kg/m^3):	1833.1
Length to Diam. Ratio:	2.32	Specimen Dry Density (kg/m^3):	1377.6
Average Rate of Strain:	1.84%	Moisture Content:	33.1%

Atterberg Information:

Test not performed



Specimen Strength Properties

Unconfined Compressive Strength (Qu)	42.99 kPa @ 15.00% Strain
Undrained Shear Strength (Su)	21.49 kPa

Reviewed by: Bryan Hiebert

Bryan Hiebert, CET

BULK UNIT WEIGHT OF SOIL SPECIMENS (METHOD A)

(ASTM D7263)

Client:	WSP Canada Inc.	**Sampled By:**	PD
Project:	Morden Bridges - Geotechnical Investigation	**Tested By:**	TL
Project No.:	221-07931-00 (Parkhill Dr)	**Test Date:**	Aug 11, 2022
Lab No.:	22-001-019-S153	**Report Date:**	Aug 12, 2022

Sample Information	
Test Hole No.	TH02
Sample No.	S7A
Soil Description	Clay
Specimen Shape	Cylindrical
Specimen Type	Intact

Test Data	
Mass of Soil (g)	182.90
Mass of Soil + Wax (g)	184.6
Submerged Mass (g)	88.5
Temp. of Water (°C)	25.0
Density of Water	0.9971

Moisture Content	
Mass of Wet + Tare (g)	142.8
Mass of Dry + Tare (g)	118.5
Mass of Tare (g)	31.7
Moisture Content (%)	28.0%

Density	
Wet Density of Soil (kg/m3)	1936
Dry Density of Soil (kg/m3)	1512

Reviewed by: Bryan Hiebert

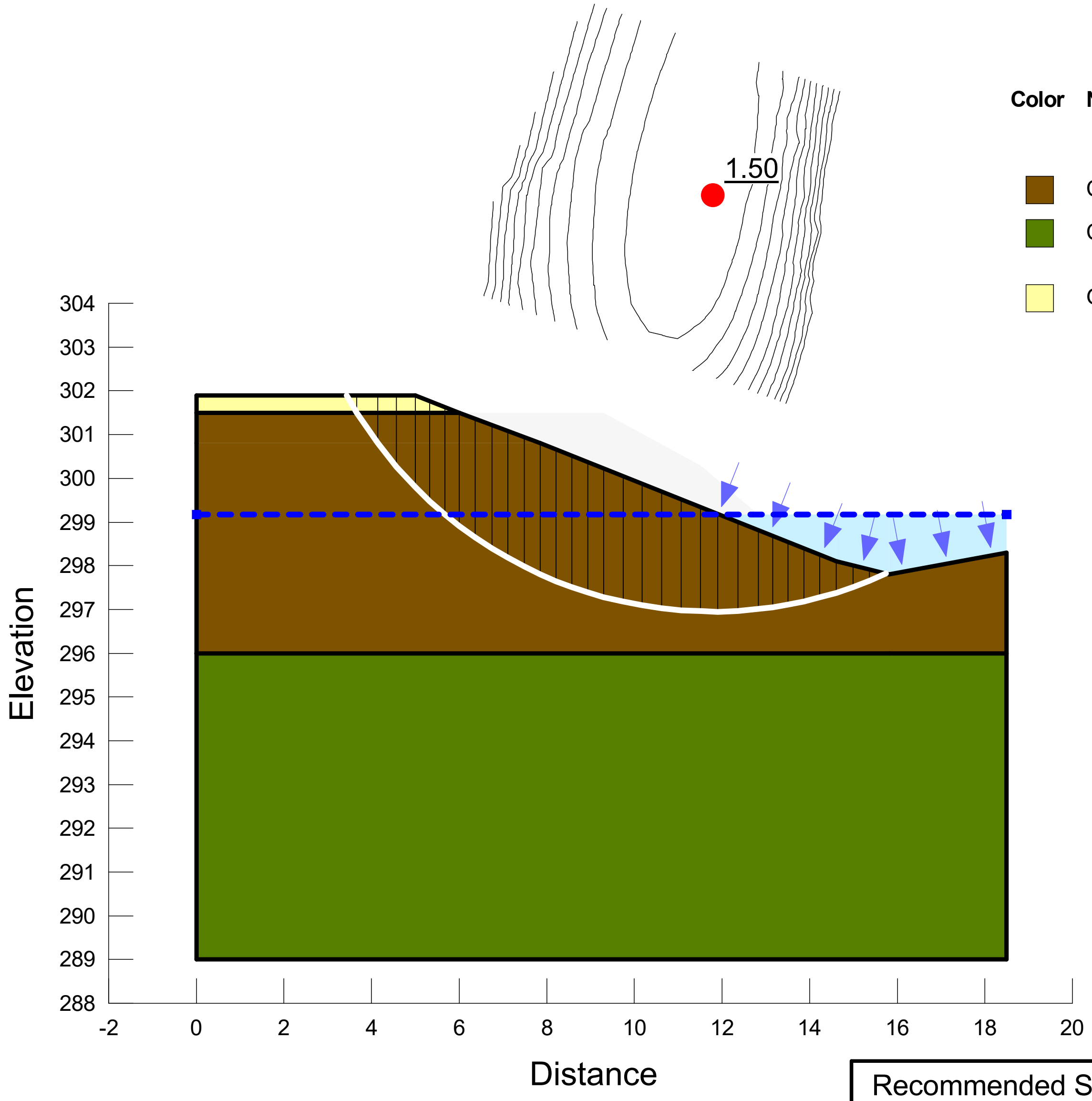
Bryan Hiebert, CET

The test data given herein pertain to the sample provided. Reporting of these data constitutes a testing service. Engineering review and interpretation may be provided upon written request.

APPENDIX

D

COMPUTER MODELLING OUTPUT

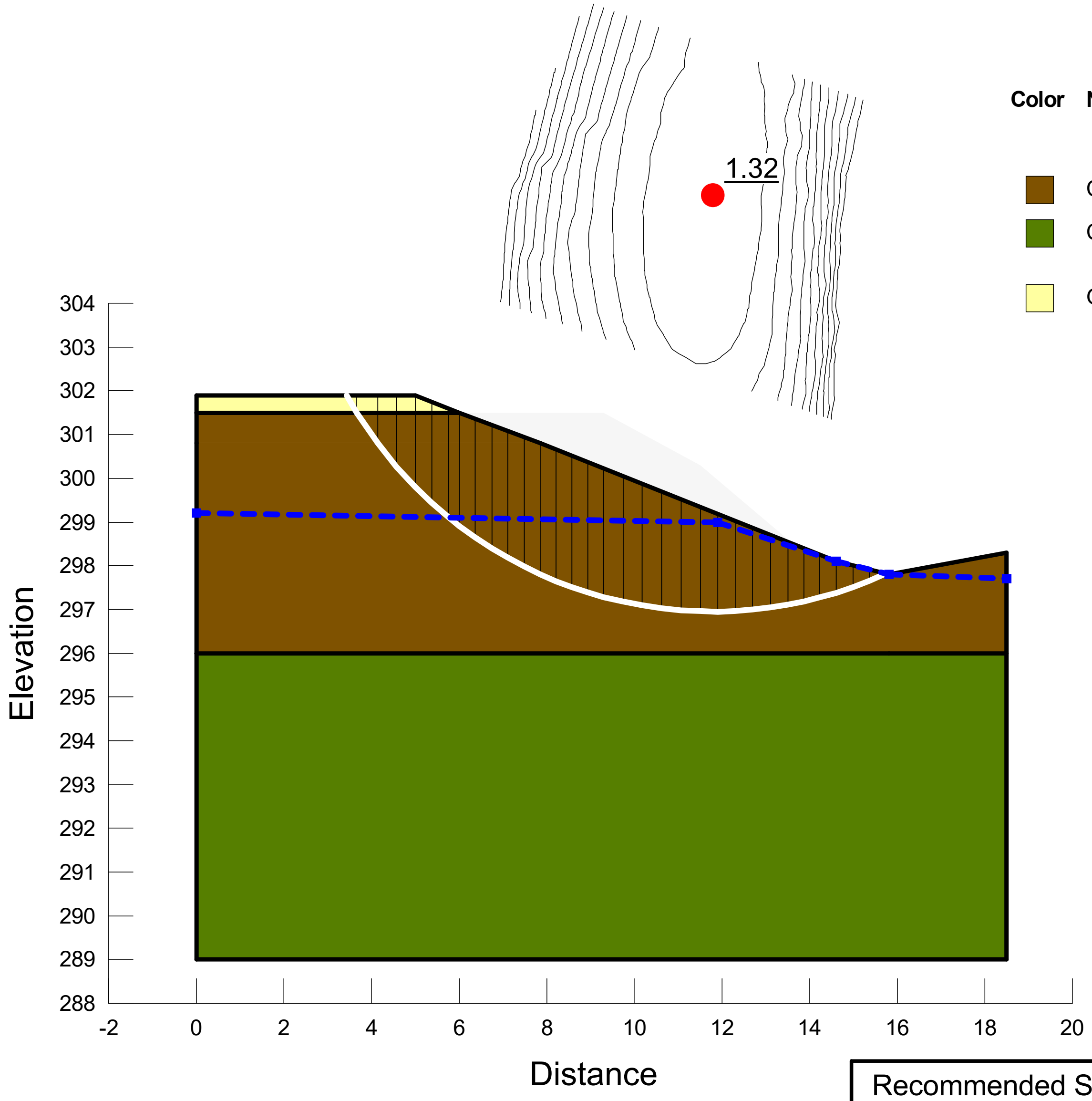


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟨	Granular	Mohr-Coulomb	20	0	35

Recommended Slope_2.5H:1V (Normal Condition)

Cross Section 3_West Embankment

221-07931-00_Parkhill Dr.

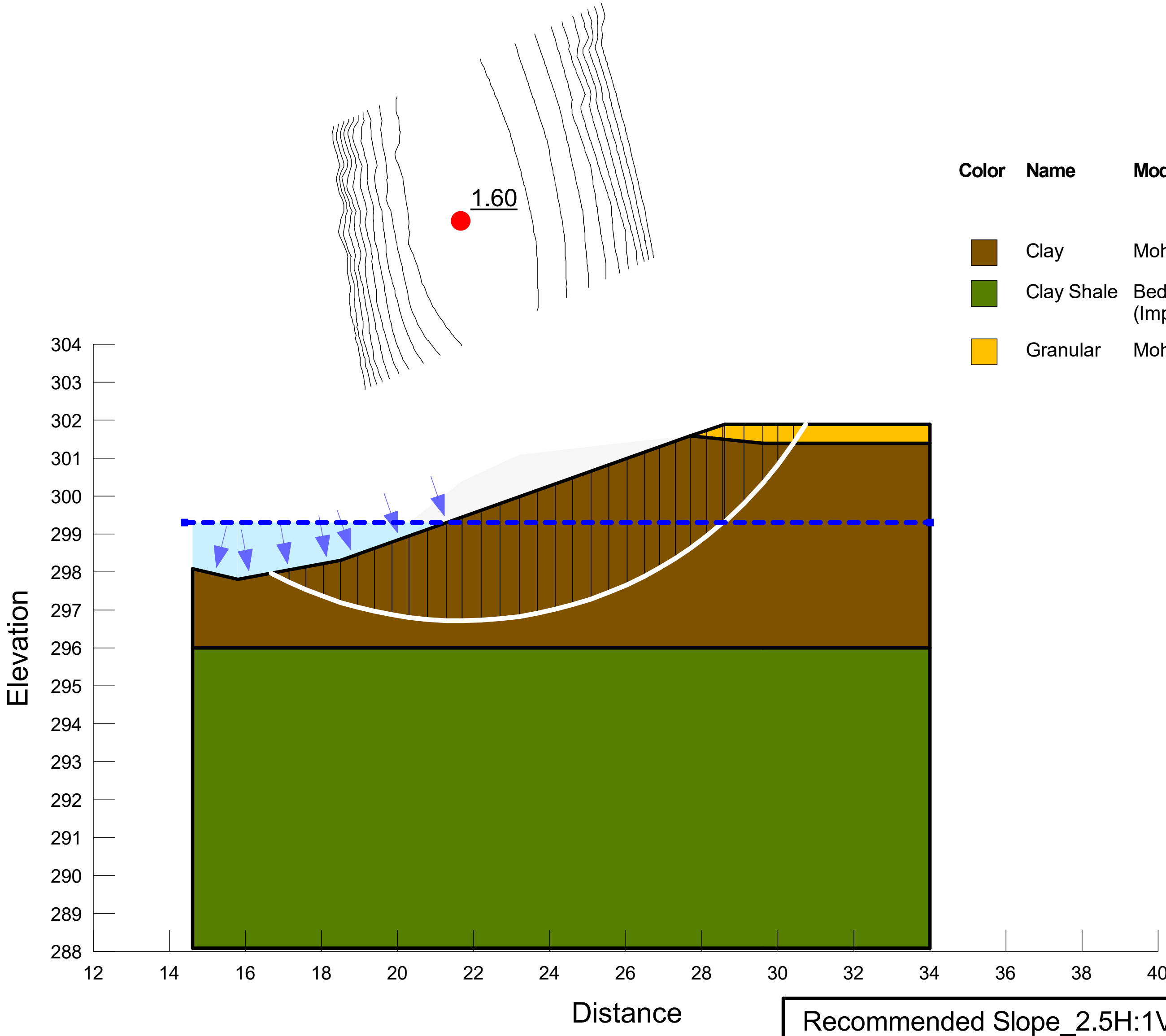


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟨	Granular	Mohr-Coulomb	20	0	35

Recommended Slope_2.5H:1V (Extreme Condition)

Cross Section 3_West Embankment

221-07931-00_Parkhill Dr.

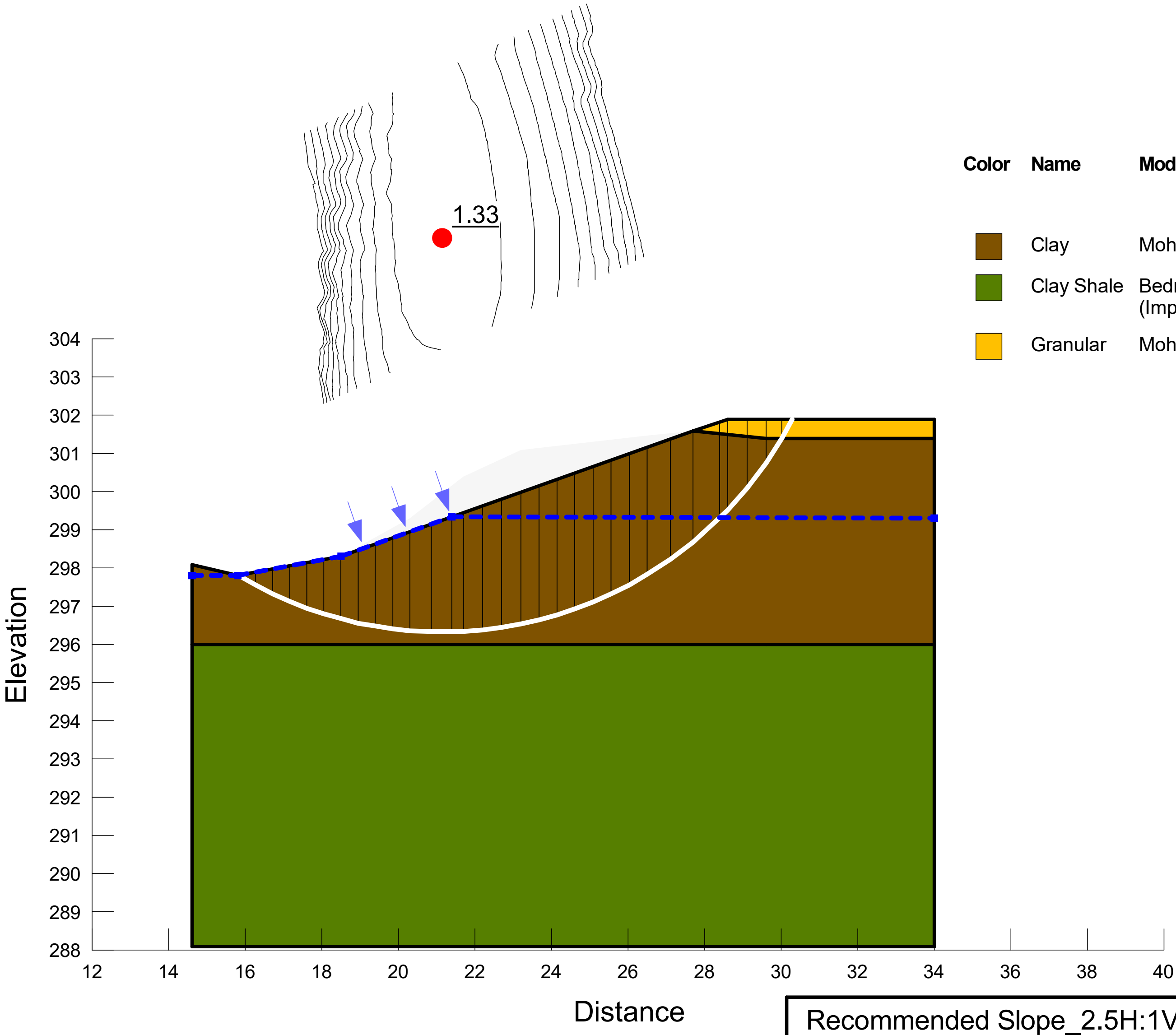


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟨	Granular	Mohr-Coulomb	20	0	35

Recommended Slope_2.5H:1V (Normal Condition)

Cross Secton 3_East Embankment

221-07931-00_Parkhill Dr.



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟨	Granular	Mohr-Coulomb	20	0	35

Recommended Slope_2.5H:1V (Extreme Condition)

Cross Secton 3_East Embankment

221-07931-00_Parkhill Dr.