CITY OF MORDEN

GEOTECHNICAL INVESTIGATION

MORDEN BRIDGES REPLACEMENT

ALVEY STREET

DECEMBER 2022

WSP
1600 BUFFALO PLACE
WINNIPEG, MANITOBA
R3T 6B8 CANADA

TEL: +1 204 915-8087
WSP.COM

WSP PROJECT NO. 221-07930-00

SIGNATURES

PREPARED BY

W.
GAO
Member
30874
2022.12.20
14:42:25-06'00'

ENGINEERS
GEOSCIENTISTS
MANITOBA
Certificate of Authorization
WSP Canada Inc.
No. 5750 Date: December, 20, 2022

Wei Gao, P.Eng.
Geotechnical Engineer, Earth and Environment

REVIEWED BY

Jerry

Jerry Leung, P.Eng.
Geotechnical Engineer, Earth and Environment

PREPARED FOR:

City of Morden
133 7th Street
Morden, MB R6M 1S3

WSP prepared this report solely for the use of the intended recipient, City of Morden, in accordance with the professional services agreement between the parties. In the event a contract has not been executed, the parties agree that the WSP General Terms for Consultant shall govern their business relationship which was provided to you prior to the preparation of this report.

The report is intended to be used in its entirety. No excerpts may be taken to be representative of the findings in the assessment.

The conclusions presented in this report are based on work performed by trained, professional and technical staff, in accordance with their reasonable interpretation of current and accepted engineering and scientific practices at the time the work was performed.

The content and opinions contained in the present report are based on the observations and/or information available to WSP at the time of preparation, using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by WSP and other engineering/scientific practitioners working under similar conditions, and subject to the same time, financial and physical constraints applicable to this project.

WSP disclaims any obligation to update this report if, after the date of this report, any conditions appear to differ significantly from those presented in this report; however, WSP reserves the right to amend or supplement this report based on additional information, documentation or evidence.

WSP makes no other representations whatsoever concerning the legal significance of its findings.

The intended recipient is solely responsible for the disclosure of any information contained in this report. If a third party makes use of, relies on, or makes decisions in accordance with this report, said third party is solely responsible for such use, reliance or decisions. WSP does not accept responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken by said third party based on this report.

WSP has provided services to the intended recipient in accordance with the professional services agreement between the parties and in a manner consistent with that degree of care, skill and diligence normally provided by members of the same profession performing the same or comparable services in respect of projects of a similar nature in similar circumstances. It is understood and agreed by WSP and the recipient of this report that WSP provides no warranty, express or implied, of any kind. Without limiting the generality of the foregoing, it is agreed and understood by WSP and the recipient of this report that WSP makes no representation or warranty whatsoever as to the sufficiency of its scope of work for the purpose sought by the recipient of this report.

In preparing this report, WSP has relied in good faith on information provided by others, as noted in the report. WSP has reasonably assumed that the information provided is correct and WSP is not responsible for the accuracy or completeness of such information.

Benchmark and elevations used in this report are primarily to establish relative elevation differences between the specific testing and/or sampling locations and should not be used for other purposes, such as grading, excavating, construction, planning, development, etc.

WSP disclaims any responsibility for consequential financial effects on transactions or property values, or requirements for follow-up actions /or costs.

recommendations given in this report are applicable only to the project and areas as described in the text and then only if constructed in accordance with the details stated in this report. The comments made in this report on potential construction issues and possible methods are intended only for the guidance of the designer. The number of testing and/or sampling locations may not be sufficient to determine all the factors that may affect construction methods and costs. We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

Overall conditions can only be extrapolated to an undefined limited area around these testing and sampling locations. The conditions that WSP interprets to exist between testing and sampling points may differ from those that actually exist. The accuracy of any extrapolation and interpretation beyond the sampling locations will depend on natural conditions, the history of Site development and changes through construction and other activities. In addition, analysis has been carried out for the identified chemical and physical parameters only, and it should not be inferred that other chemical species or physical conditions are not present. WSP cannot warrant against undiscovered environmental liabilities or adverse impacts off-Site.

The original of this digital file will be kept by WSP for a period of not less than 11 years. As the digital file transmitted to the intended recipient is no longer under the control of WSP, its integrity cannot be assured. As such, WSP does not guarantee any modifications made to this digital file subsequent to its transmission to the intended recipient.

This limitations statement is considered an integral part of this report.

TABLE OF CONTENTS

1 INTRODUCTION 1

2 SITE AND PROJECT DESCRIPTION 1

2.1 Published Geological Information 1

3 METHODOLOGY 1

3.1 Field Investigation 1

3.2 Laboratory Testing 2

4 SUBSURFACE CONDITIONS 2

4.1 Asphalt 2

4.2 Sand Fill 3

4.3 Native Clay 3

4.4 Clay Shale 3

4.5 Groundwater and Sloughing Observations 4

5 GEOTECHNICAL CONSIDERATIONS 4

5.1 Frost Penetration Depth and Frost Heave 5

5.2 Deformable Native Clay 5

6 FOUNDATION RECOMMENDATIONS 5

6.1 Driven Steel Piles 6

6.2 Pile Group 7

6.3 Pile Downdrag 8

6.4 Pile Settlement 8

6.5 Lateral Loads on Piles 8

6.5.1 Lateral Pile Capacity 8

6.5.2 Lateral Pile Deflection 9

7 LATERAL EARTH PRESSURE 9

8 TEMPORARY EXCAVATIONS 10

9 EMBANKMENT SLOPE STABILITY 10

9.1 Design Criteria 10

9.2 Proposed Configuration and Methodology 11

9.3 Soil Parameters 11

9.4 Groundwater Conditions and Creek Water Levels 11

9.5 Slope Stability Results 12

10 COCLUSIONS 13

11 REFERENCES 13

TABLES

Table 1 Test Hole Details 2

Table 2 Laboratory Test Results for Native Clay 3

Table 3 Consolidation Parameters 3

Table 4 Laboratory Test Results for Clay Shale 4

Table 5 Bult Unit Weight of Clay Shale 4

Table 6 Groundwater and Sloughing Observations 4

Table 7 Resistance Factors for Deep Foundations (CHBDC) 5

Table 8 Unfactored Skin Friction and End-Bearing for Driven H Piles 6

Table 9 Soil Properties for Analysis of Laterally Loaded Piles 9

Table 10 Earth Pressure Coefficients 9

Table 11 Soil Parameters Used in SLOPE/W Analysis 11

Table 12 Creek Water Levels Under Normal and Extreme Conditions 12

Table 13 Slope Stability Results 12

APPENDICES

A Site Plan & Typical Cross Section Profiles

B Soil Logs

C Laboratory Testing Sheet

D Computer Modelling Output

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 for the existing structure at the location of Alvey Street and Parkhill Drive in the City of Morden, Manitoba. This report mainly focuses on the subject site at Alvey Street; the Parkhill Drive 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 Alvey Street is located southwest quadrant of the City of Morden. The existing bridge is culvert crossing structure and required to be replaced due to the culvert deformation and displacement, including the collapsed asphalt wearing surface resulting from material being washed away 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 at the subject site 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., TH22-01) to auger refusal of 15.1 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
TH22-01	Alvey Street	July 29/22	15.1	5448703	564266	Southeast 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 (12)
- 2 Atterberg limits tests and 1 Grain size analysis tests (sieve and hydrometer)
- 1 Consolidation test (1D)
- 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 thick 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 approximately 6.4 mbgs. The clay was described as sandy with some silt to silty, trace gravel, dark brown, moist, medium plastic. Based on pocket pen readings obtained during drilling, shear strength values varied throughout the soil strata and ranged from 72 kPa at upper 2 mbgs to 24 kPa to 48 kPa near 5.0 m depth, with an average value of 36 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 21 percent to 39 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 to high 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 (%)
TH22-01	3.1	21	47	28	CI	0.1	24.2	38.8	36.8

Notes: CI – medium plastic clay

In addition, a 1-D consolidation test was also conducted from the obtained Shelby tube in TH22-01 (between 4.6 m and 5.2 m) in order to estimate the consolidation design parameters. Table 3 below outlines the test results.

Table 3 Consolidation Parameters

Test Hole #	Sample Depth (m)	Soil Type	Initial Void Ratio e_o	Compression Index C_c	Recompression Index C_r	Coefficient of Vertical Consolidation C_v
TH22-01	4.6 to 5.2	Clay	0.75	0.22	0.05	6.3 m^2/year

4.4 Clay Shale

Clay shale was encountered at the depth of 6.4 mbgs during drilling and extended to 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 168 and 192 kPa were obtained throughout the clay shale layer, with SPT 'N' values varying from 33 to 84, indicated very stiff to hard. The moisture content results obtained from the tested samples ranged from 21 percent to 42 percent.

One particle size analysis test was conducted on the selected samples; the result is outlined in Table 4.

Table 4 Laboratory Test Results for Clay Shale

Test Hole #	Sample Depth (mbgs)	Atterberg Limits			Sieve and Hydrometer			
		Plastic Limit (%)	Liquid Limit (%)	Plasticity Symbol	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
TH22-01	9.1	-	--	-	0.0	3.0	31.7	65.4

In addition, a Shelby tube sample was collected at depth from 9.1 m to 9.7 mbgs from TH22-01 in order to determine the Unconfined Compressive Strength (UCS) value of the clay shale. However, due to hard drilling and lack of sample retrieved from the tube, the UCS test cannot be carried out.

Unit weight tests were also carried out on the selected samples, the results of which are summarized in Table 5.

Table 5 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)
TH22-01	7.5	Clay Shale	26.0	1961	1556

Auger refusal was encountered at depth of 15.1 m, with SPT ‘N’ values of >> 50.

4.5 Groundwater and Sloughing Observations

No slough was observed during or after the drilling. However, water seepage was observed and recorded during field drilling. The detailed information related to water seepage and slough are summarized in Table 6 below.

Table 6 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)
TH22-01	15.1	No Slough	2.4	7.0

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 structural elements to prevent damage due to uplift pressures from 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 requirement at the bridge abutment is greater than 1.0 m. In addition, a slope stability review should also be conducted in order to examine whether the additional fill may have an 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 sites, 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 7 below.

Table 7 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 location, 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 8 below.

Table 8 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 +/- 15.1	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 7.

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 8.
- All piles must be driven continuously once driving is initiated. Where steel H piles are driven to the 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 8 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 anticipated depths from 10 m to 12 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 location 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 load induced by the additional fill placement that could cause movements of soil during pile driving that is transferred to the pile itself. Based on the soil conditions encountered at the test hole locations, it is anticipated that the drowndrag 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 7.

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 the 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 the 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. The lateral resistance of the upper 1.0 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 9.

Table 9 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	35	8	0.009	2.3/d
Clay (Shale)	6.0 to 15.0	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 the buildup of hydrostatic pressure. The coefficient for the active, at-rest and passive earth pressure for different soil types can be referenced in Table 10 below.

Table 10 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 the 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 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 10 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, the 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 water ponding 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 relatively 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-01 & XS-02) 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 Alvey Street is unknown at the time of this report writing. However, the additional fill thickness is not anticipated to be greater than 0.36 m on both side 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 the 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 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 11 below.

Table 11 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 - 15.0 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) for both culvert and bridge options at the 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 the 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 Alvey Street are summarized in Table 12.

Table 12 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)
Alvey Street	305.48	305.47	305.50	304.59	306.80	306.79	306.80	+/- 304.00

9.5 Slope Stability Results

The results of the slope stability analysis is outlined in Table 13 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 they 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 13 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)
Alvey Street	XS1	West Abutment	4.5H:1V followed by 0.5H:1V	1.42	1.47	1.34	>2.0	1.54	1.35
		East Abutment	2H:1V followed by a flatter Toe	1.63	1.63	1.42	>2.0	1.81	1.41
	XS2	West Abutment	1.2H:1V followed by a flatter Toe	1.58	1.55	1.54	1.94	1.58	1.52
		East Abutment	2H:1V	1.62	1.75	1.38	>2.0	1.53	1.36

(1) Based on normal groundwater and creek water level at 305.5 m (3dQ10) and additional fill at Alvey St
(2) Based on normal groundwater level at 305.5 m (3dQ10), empty channel and additional fill at Alvey St

Table 13 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 are 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 angles steeper than 2H:1V require stabilization measures, 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.

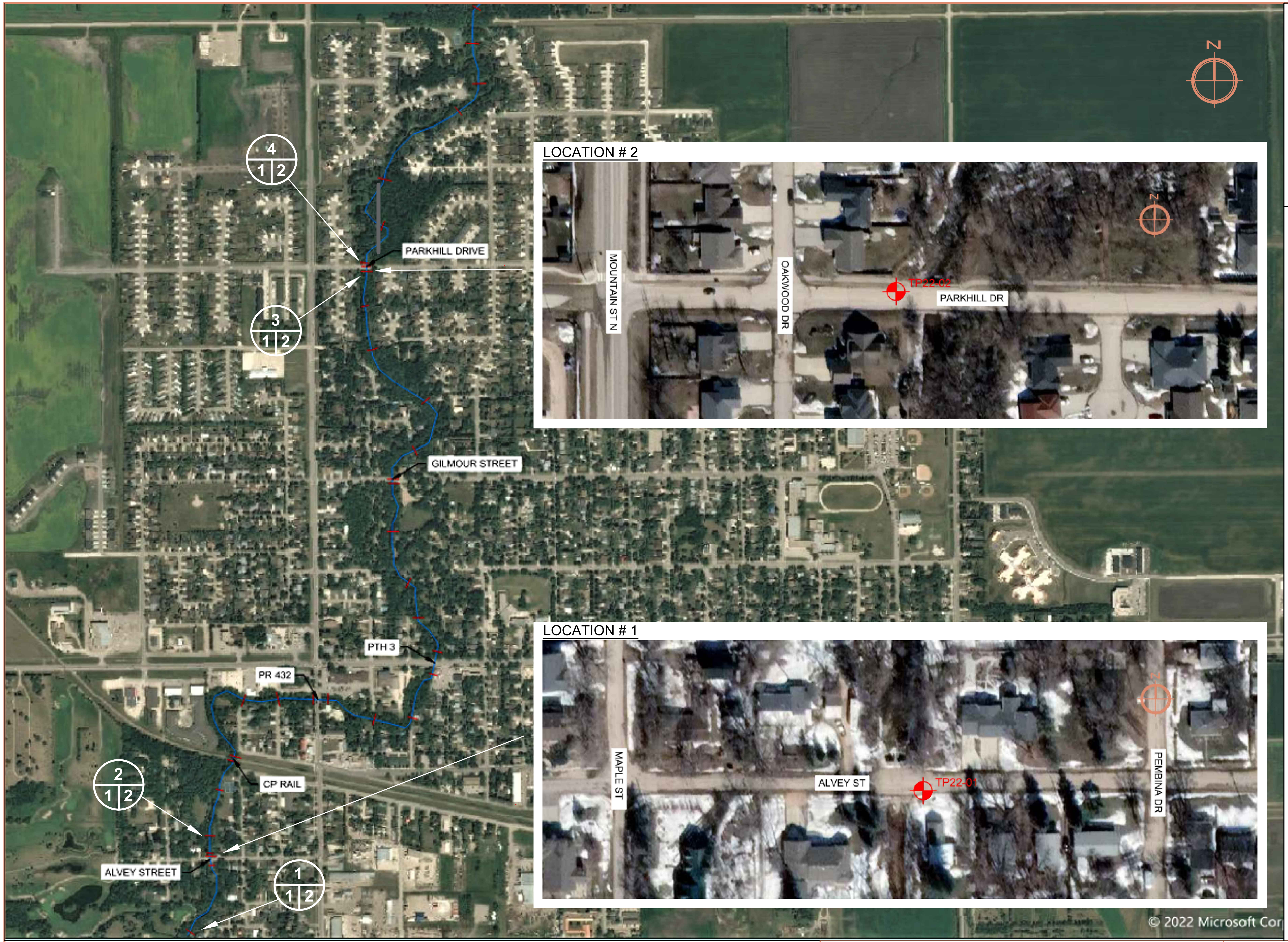
Canadian Geotechnical Society. (2006). Canadian Foundation Engineering Manual. Calgary, Alberta.

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

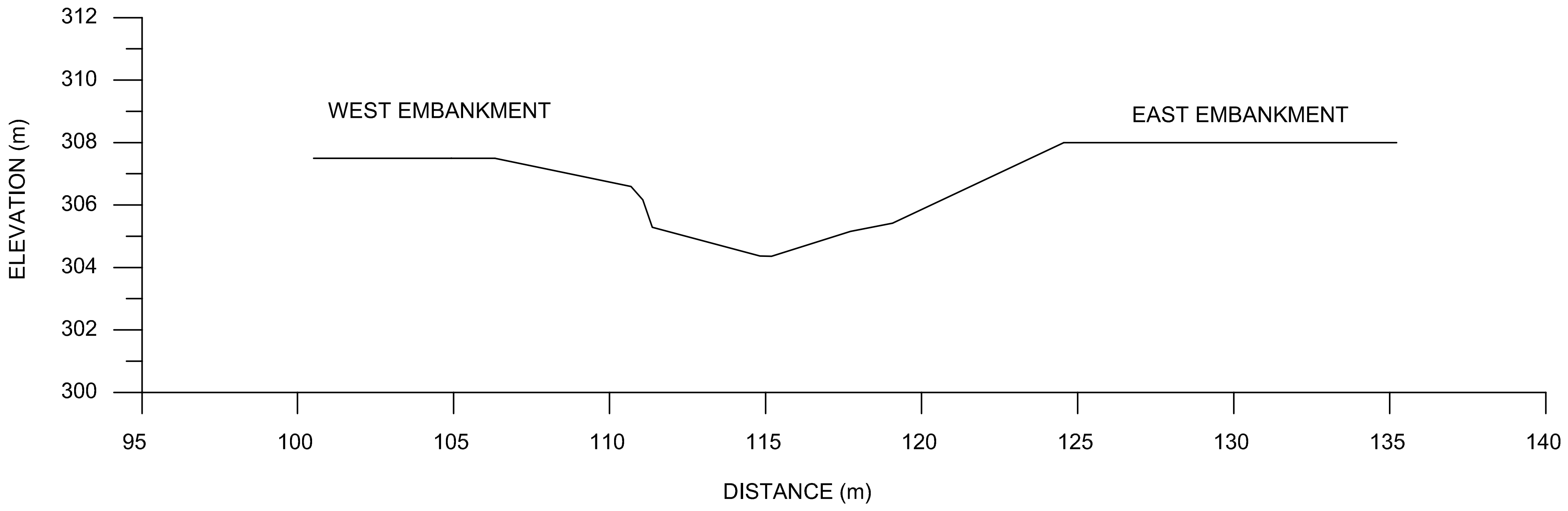
- ALL TEST HOLES PRESENT ON SITE ARE APPROXIMATE LOCATION.

WSP Canada Group Limited
1600 Buffalo Place
Winnipeg, MB R3T 6B8
t. 204.477.6650

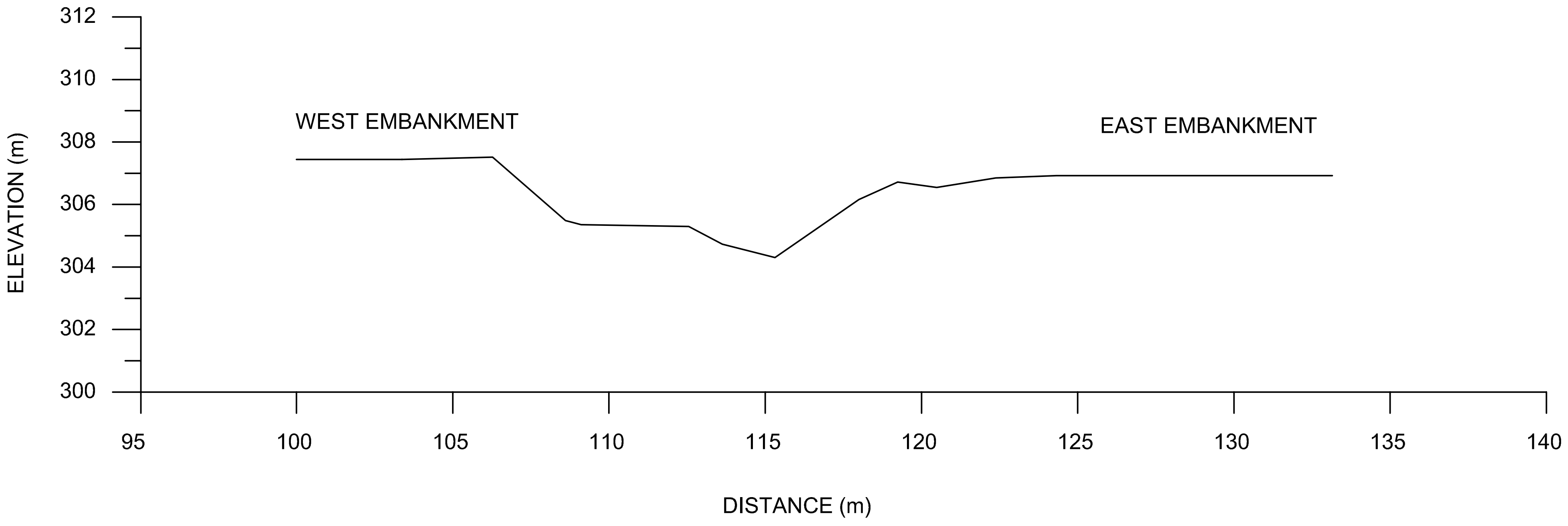
www.wsp.com

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



1 | 1|2 CROSS SECTION 1



2 | 1|2 CROSS SECTION 2

WSP Canada Group Limited
1600 Buffalo Place
Winnipeg, MB R3T 6B8
t. 204.477.6650

www.wsp.com

GEOTECHNICAL INVESTIGATION
MORDEN BRIDGES - ALVEY STEET - MORDEN, MANITOBA
CROSS SECTION DETAILS

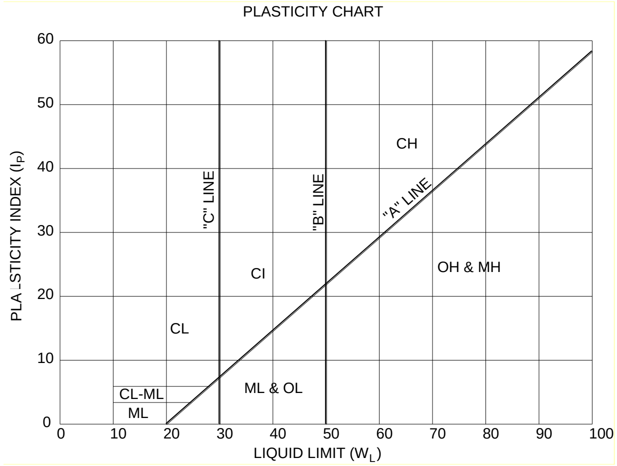
SCALE:	DATE:	DWG. No.
1:200	NOV. 18, 2022	FIG. 2 OF 2

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	 (NOTE: For soils passing No. 400 sieve & sieve size in μm)		
		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			
	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		
Oversize Material	Cobbles	76 mm to 300 mm
	Boulders	> 300 mm

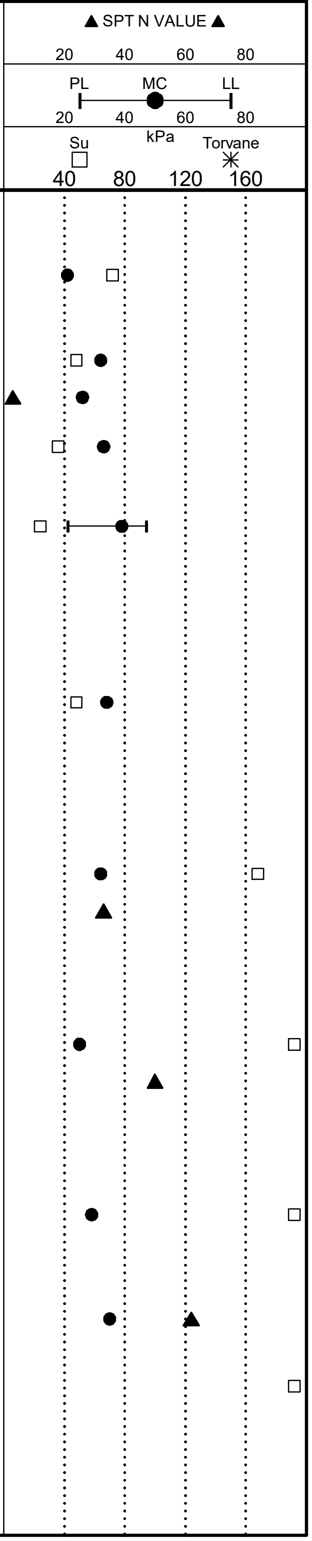
RELATIVE DENSITY AND CONSISTENCY

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

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 Alvey Street (5448703 N; 564266 E)
AFTER DRILLING ---

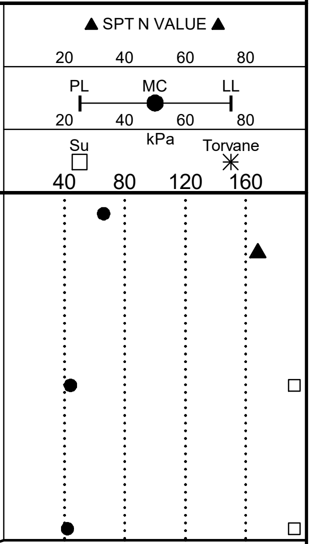
DEPTH (m)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	Shear Strength Su (kPa)	TORVANE (kPa)	MOISTURE CONTENT (%)
	ASPHALT PAVEMENT (80 mm) - Black, compact, gravelly, some sand					
	SAND FILL (220 mm) - Dark brown, moist, compact, fine grained, trace gravel, trace silt, trace clay	GB S1		72		21
	CLAY (CI) - Sandy, some silt, trace gravel, dark brown, moist, medium plastic - At 1.5 m depth, soft	GB S2		48		32
2	- At 1.8 m depth, medium brown to grey, silty, trace oxidized	SPT S2A	1-1-2 (3)			26
	- At 2.4 m depth, dark grey, moist to wet - From 2.4 m to 3.1 m, wet, trace sand inclusions, water seepage observed	GB S3		36		33
		GB S4		24		39
4	- Below 3.1 m, moist, silty, some sand, coarse grained - Particle size analysis obtained on S4: Gravel (0.1%); Sand (24.2%); Silt (38.8%); Clay (36.8%)					
		GB S5		48		34
	- Below 5.0 m, dark grey, moist, very stiff	ST ST1				
6		GB S6		168		32
	CLAY (SHALE) - Dark grey to dark brown, moist, very stiff, silty, trace sand	SPT S6A	4-12-21 (33)			
		GB S7		192		25
8	- Bulk unit weight obtained on S7A at 8.0 mbgs is 1961 kg/m3	SPT S7A	10-20-30 (50)			
	- Below 9.0 m, hard - Particle size analysis obtained on S8: Gravel (0.0%); Sand (3.0%); Silt (31.7%); Clay (65.4%)	GB S8		192		29
		ST ST2				
10		SPT S8A	19-30-32 (62)			35
		GB S9		192		
12						



(Continued Next Page)

CLIENT City of Morden
PROJECT NUMBER 221-07930-00 & 221-07931-00
PROJECT NAME Geotech Investigation - Bridge Replacement
PROJECT LOCATION City of Morden, MB

DEPTH (m)	GRAPHIC LOG	ELEV. (m)	WATER LEVEL	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	Shear Strength Su (kPa)	TORVANE (kPa)	MOISTURE CONTENT (%)
					GB S10				33
					SPT S10A	20-29-55 (84)			
					GB S11		192		22
14									
					GB S12		192		21



END OF TESTHOLE
- Auger refusal encountered in the clay shale layer at 15.1 mbgs.
- No sloughing observed upon completion of drilling.
- Water seepage first observed at 2.4 mbgs from the sand inclusions area in the clay layer and measured at 7.0 mbgs upon completion of drilling.
- Testhole backfilled with auger cuttings and bentonite, then patched with cold mix asphalt upon completion of drilling.

APPENDIX

C

LABORATORY TESTING SHEET

MOISTURE CONTENT OF SOIL AND ROCK
(ASTM D2216)

Client:	WSP Canada Inc.	**Lab No.:**	22-001-018-S152
Project:	Morden Bridges - Geotechnical Investigation	**Project No.:**	221-07930-00 (Alvey
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 (%)
TH01	S1	2.5	20.8
TH01	S2	5.0	31.8
TH01	S2A	5-6.5	26.2
TH01	S3	7.5	32.6
TH01	S4	10.0	38.7
TH01	S5	15.0	33.5
TH01	S6	20.0	31.9
TH01	S7	25.0	24.6
TH01	S8	30.0	28.7
TH01	S9	35.0	35.1
TH01	S10	40.0	32.6
TH01	S11	45.0	22.0
TH01	S12	49.5	21.3

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)

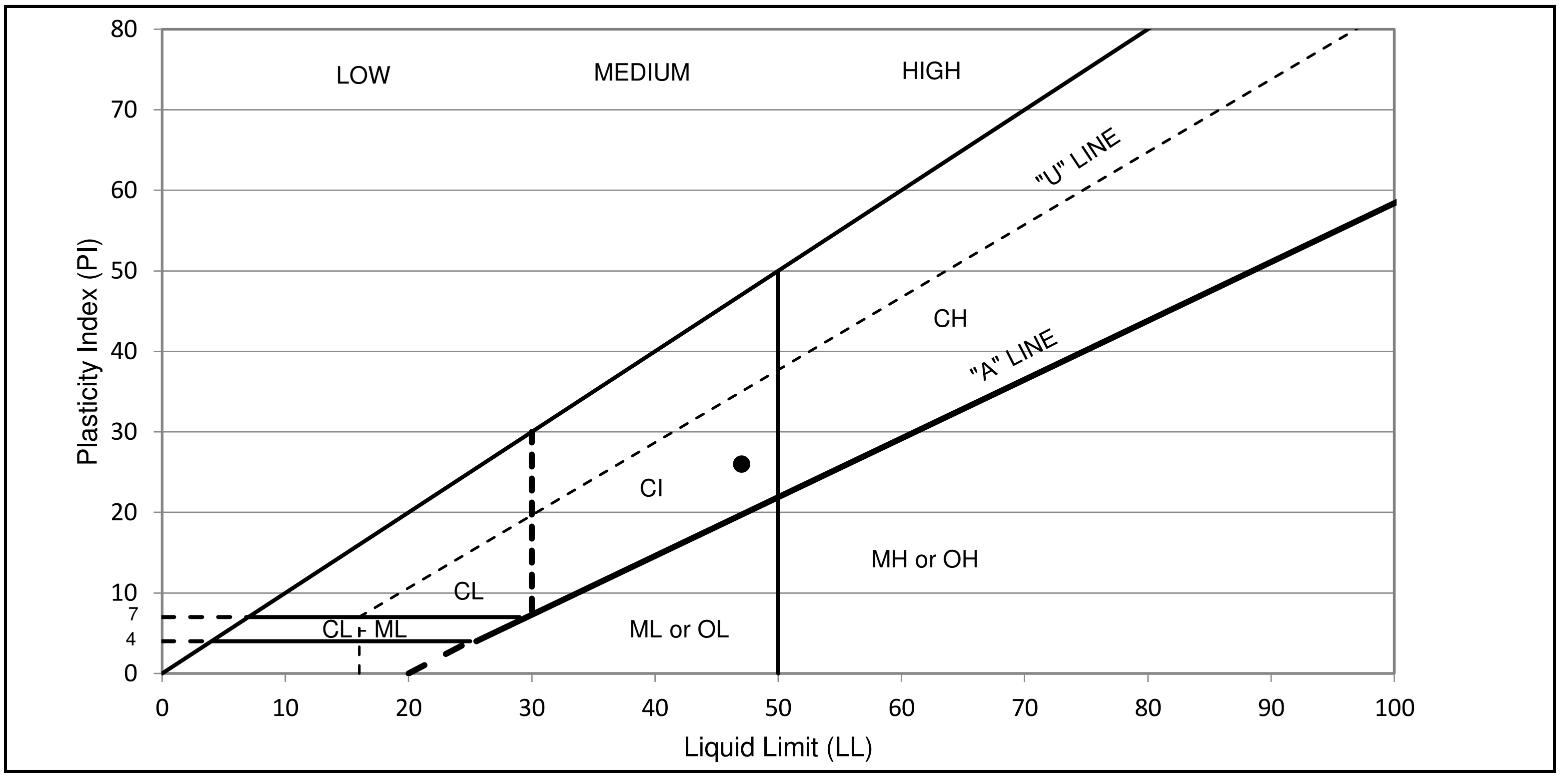
Client:	WSP Canada Inc.			**Lab No.:**	22-001-018-S152
Project:	Mordern Bridges - Geotechnical Investigation			**Project No.:**	221-07930-00 (Alve
Site Location:	Project Site			**Report Date:**	Aug 12, 2022
Date Sampled:	Jul 29, 2022			**Date Tested:**	Aug 11, 2022
Sampled By:	PD	**Date Received:**	Aug 02, 2022	**Tested By:**	TL
Testhole No.:	TH01	**Sample No.:**	S4	**Depth (ft):**	10
Drying Method:	Oven	**Method:**	Multi-Point		

Liquid Limit Test
(Manual, Plastic Grooving tool)

Trial	**A**	**B**	**C**
No. of Blows	16	26	31
Moisture Content (%)	49.0	46.7	46.0

Plastic Limit Test
(Hand rolled)

Trial	**A**	**B**
Moisture Content (%)	21.4	21.1



USCS Symbol CI

LL, Liquid Limit (%) 47

PL, Plastic Limit (%) 21

PI, Plasticity Index 26

Soil Description: Medium Plastic Clay

Reviewed by: Bryan Hiebert

Bryan Hiebert, CET

Comment: As received moisture content is 38.7%.

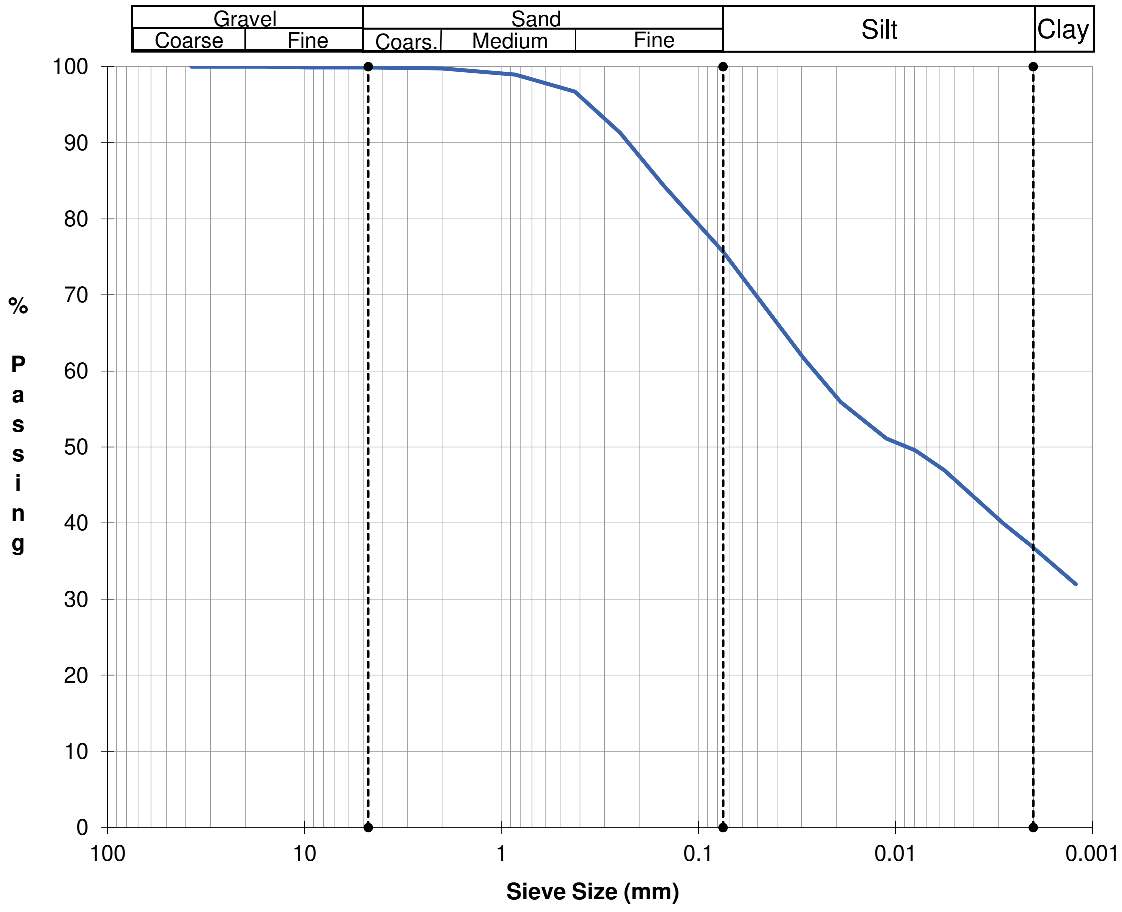
The test data given herein pertain to the sample provided. Reporting of these data constitutes a testing service. Engineering review and interpretation may be provided upon written request.

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

Client: WSP Canada Inc.
Lab No.: 22-001-018-S152
Project: Mordern Bridges - Geotechnical Investigation
Project No.: 21-07930-00 (Alvey St)

Testhole No.: TH01
Sampled by: PD
Sample Source: Project Site
Sample No.: S4
Date Sampled: Jul 29, 2022
Date Received: Aug 02, 2022
Depth (ft): 10.0
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	99.9
4.75	99.9
2.00	99.8
0.850	98.9
0.425	96.7
0.250	91.3
0.150	84.3
0.075	75.7
0.029	61.7
0.019	55.9
0.011	51.1
0.008	49.5
0.006	47.0
0.0028	39.9
0.0021	37.1
0.0012	32.0

Percent of: Gravel = 0.1% Sand = 24.2% Silt = 38.8% Clay = 36.8%

Sample Description: Silt and clay, sandy, 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.

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

Client: WSP Canada Inc.
Lab No.: 22-001-018-S152

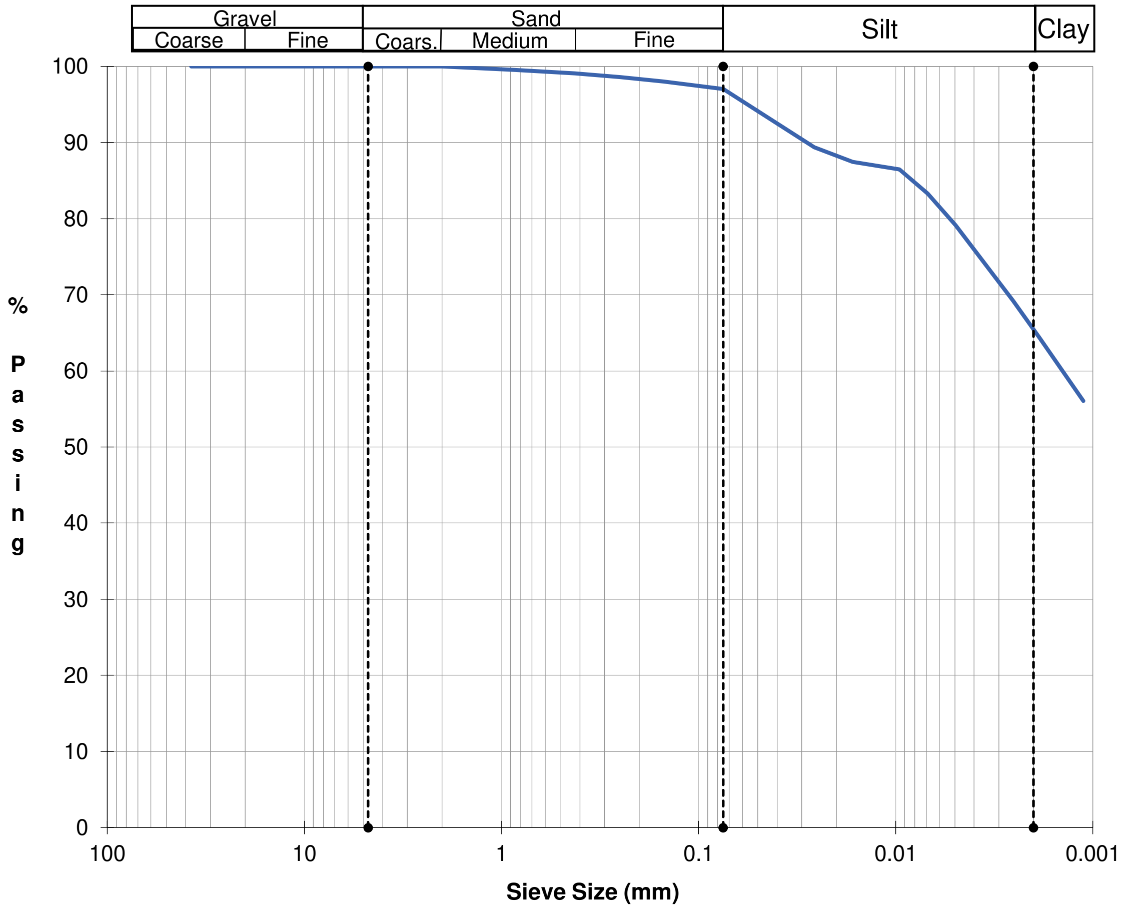
Project: Mordern Bridges - Geotechnical Investigation
Project No.: 1-07930-00 (Alvey St)

Testhole No.: TH01
Sampled by: PD
Sample Source: Project Site

Sample No.: S8
Date Sampled: Jul 29, 2022
Date Received: Aug 02, 2022

Depth (ft): 30.0
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	100.0
2.00	100.0
0.850	99.5
0.425	99.1
0.250	98.6
0.150	98.0
0.075	97.0
0.026	89.4
0.016	87.4
0.010	86.5
0.007	83.3
0.005	79.1
0.0025	69.2
0.0019	64.4
0.0011	56.0

Percent of: Gravel = 0.0% Sand = 3.0% Silt = 31.7% Clay = 65.4%

Sample Description: Silty clay, trace sand

Remarks: Separation made on No 10 sieve (2.0 mm).

Reviewed by: Bryan Hiebert
Bryan Hiebert, CET

Unconfined Compressive Strength of Cohesive Soils

ASTM D2166

Client:	WSP Canada Inc	Sampled By:	WG
Project:	Morden Bridges (Alvey)	Tested By:	BMH
Job No.:	221-07930-00	Sample Date:	2022-07-29
Report Date:	2022-08-08	Test Date:	2022-08-08
Test Hole No.	TH01	Sample No.:	ST2
Depth (ft)	30' - 32'		

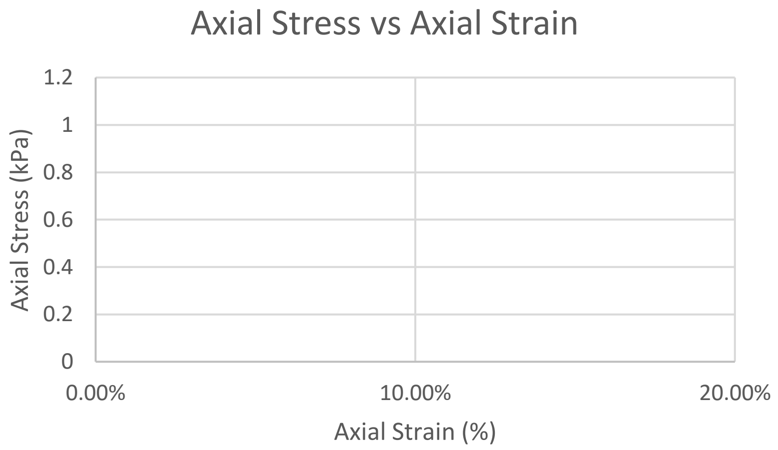
Specimen Info

Soil Description: CLAY, silty, trace sand, moist, dark grey, laminated(<5 mm thick), very stiff, medium to high plasticity

Sample Type:	Intact	Specimen Wet Density (kg/m^3):	1954.2
Length to Diam. Ratio:	0.58	Specimen Dry Density (kg/m^3):	1518.5
Average Rate of Strain:	0.00%	Moisture Content:	28.7%

Atterberg Information:

Test not performed



Specimen Strength Properties

Unconfined Compressive Strength (Qu)	Sample was not suitable for testing
Undrained Shear Strength (Su)	Sample was not suitable for testing

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.

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.:	221-07930-00 (Alvey St)	**Test Date:**	Aug 11, 2022
Lab No.:	22-001-018-S152	**Report Date:**	Aug 12, 2022

Sample Information	
Test Hole No.	TH01
Sample No.	S7A
Soil Description	Clay
Specimen Shape	Cylindrical
Specimen Type	Intact

Test Data	
Mass of Soil (g)	180.80
Mass of Soil + Wax (g)	183.1
Submerged Mass (g)	88.6
Temp. of Water (°C)	25.0
Density of Water	0.9971

Moisture Content	
Mass of Wet + Tare (g)	145.8
Mass of Dry + Tare (g)	122.3
Mass of Tare (g)	31.8
Moisture Content (%)	26.0%

Density	
Wet Density of Soil (kg/m3)	1961
Dry Density of Soil (kg/m3)	1556

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.

SOLUM™
CONSULTANTS LTD.
GEOTECHNICAL & MATERIAL
TESTING LABORATORY
#9-3620 29 St. NE, Calgary, AB T1Y 5Z8
Phone: (403)250-3035 Cell: (403)619-7250
Prime email: solum@mymts.net
2nd email: solumconsultantsltd111@outlook.com
www.solumconsultantsltd.com

Geo-Lab Report

Revision # 0

Report Date:	*August 29, 2022*
Client:	*WSP Canada Inc.*
Address:	*Suite 3300, 237-4th Ave. SW, Calgary, AB T2P 4K3*
Attn:	*Wei Gao*
Project No:	*221-07930-00*
Project Name:	*N/A*
Solum Job No.:	*07101220805(200)*

Sample Received Date:	August 5, 2022
Sample Quantity:	1 ST

Test	Quantity	Destination
1-D CONSOLIDATION	1	D2435

President: Saad Farag

CONSOLIDATION TEST DATA

2022-08-29

Client: WSP
Project: N/A
Project Number: 221-07930-00
Depth: 15-17'
Sample Number: ST1

Test Specimen Data

NATURAL MOISTURE

Wet w+t = 159.47 g.
Dry w+t = 134.63 g.
Tare Wt. = 37.12 g.
Moisture = 25.5 %

UNIT WEIGHT

Height = 0.787 in.
Diameter = 2.433 in.
Weight = 115.73 g.
Dry Dens. = 1538 kg/m^3

VOID RATIO

Spec. Gr. = 2.69
Est. Ht. Solids = 1.143 cm.
Init. V.R. = 0.749
Init. Sat. = 91.5 %

TEST START

Height = 0.787 in.
Diameter = 2.433 in.

AFTER TEST

Wet w+t = 155.99 g.
Dry w+t = 135.22 g.
Tare Wt. = 43.00 g.
Moisture = 22.5 %

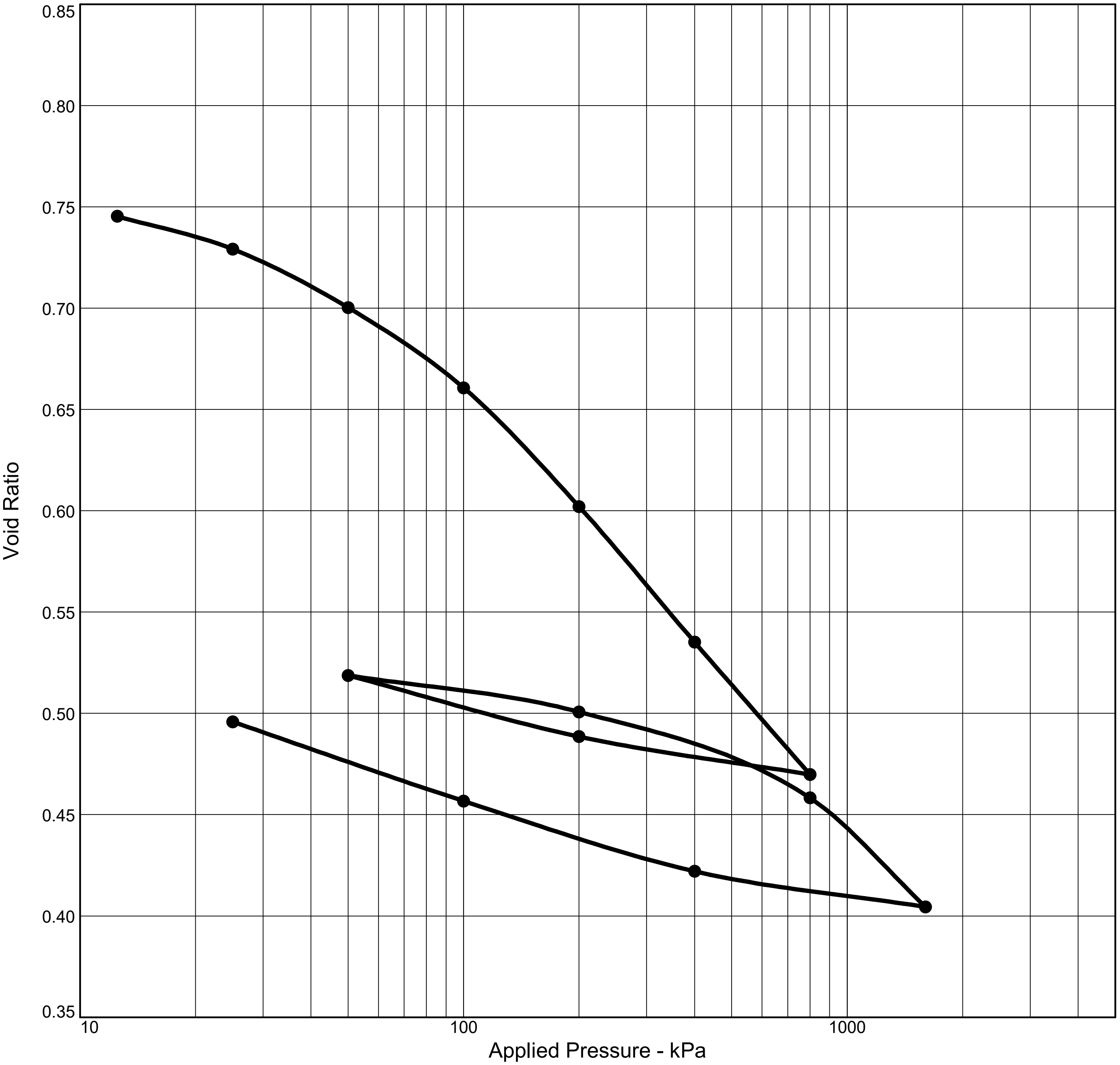
Dry Wt. = 92.22 g.

End-Of-Load Summary

Pressure (kPa)	Final Dial (in.)	Deformation (in.)	C_v (cm.^{2}/min.)	C_α	Void Ratio	% Strain
start	0.00000	0.00000			0.749	
12.5	-0.00150	0.00150	0.147		0.745	0.2 Comprs.
25.0	-0.00880	0.00880	0.180		0.729	1.1 Comprs.
50.0	-0.02180	0.02180	0.189		0.700	2.8 Comprs.
100.0	-0.03960	0.03960	0.141		0.661	5.0 Comprs.
200.0	-0.06600	0.06600	0.099		0.602	8.4 Comprs.
400.0	-0.09610	0.09610	0.048		0.535	12.2 Comprs.
800.0	-0.12550	0.12550	0.036		0.470	15.9 Comprs.
200.0	-0.11710	0.11710	0.052		0.488	14.9 Comprs.
50.0	-0.10350	0.10350	0.018		0.519	13.2 Comprs.
200.0	-0.11160	0.11160	0.043		0.501	14.2 Comprs.
800.0	-0.13070	0.13070	0.055		0.458	16.6 Comprs.
1600.0	-0.15490	0.15490	0.026		0.405	19.7 Comprs.
400.0	-0.14700	0.14700	0.039		0.422	18.7 Comprs.
100.0	-0.13140	0.13140	1.442		0.457	16.7 Comprs.
25.0	-0.11380	0.11380	0.003		0.496	14.5 Comprs.

Compression index (C_c), kPa = 0.22 **Preconsolidation pressure (P_p), kPa =** 58 **Void ratio at P_p (e_m) =** 0.693
Recompression index (C_r) = 0.05

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (kg/m3)	LL	PI	Sp. Gr.	Overburden (kPa)	P_c (kPa)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
91.5 %	25.5 %	1538			2.69		58	0.22	0.05	0.749

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 221-07930- **Client:** WSP

Project: N/A

Depth: 15-17' **Sample Number:** ST1

SOLUM™
CONSULTANTS LTD
GEOTECHNICAL & MATERIAL
TESTING LABORATORY

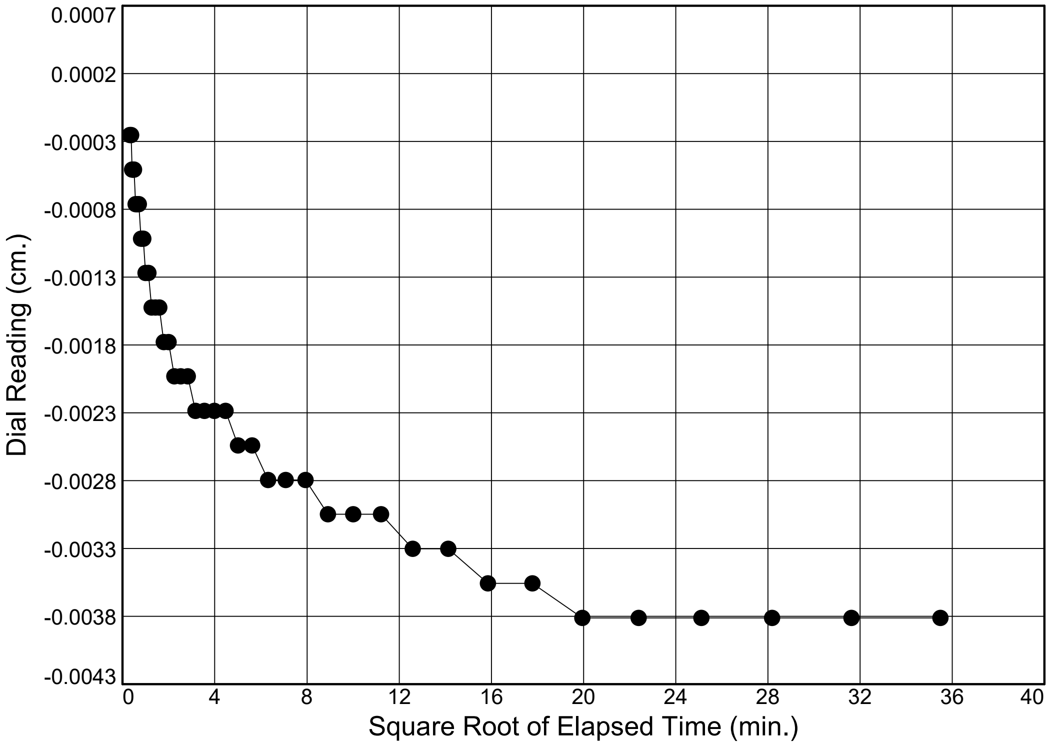
Remarks:

Figure

Dial Reading vs. Time

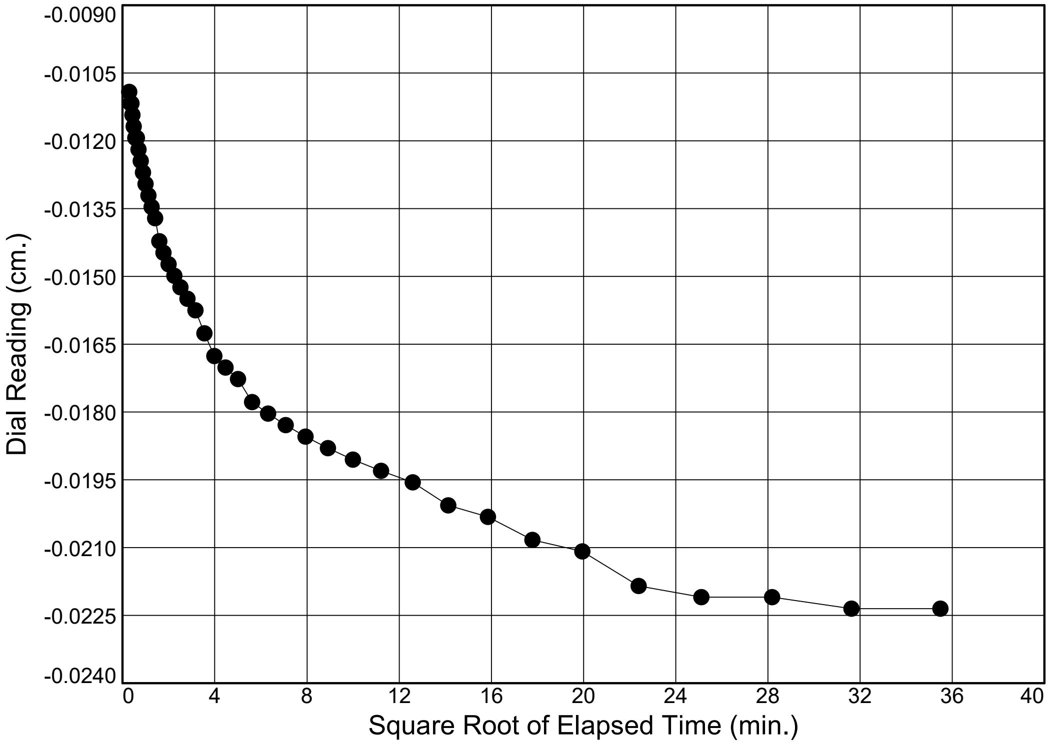
Project No.: 221-07930-00
Project: N/A

Depth: 15-17'   Sample Number: ST1



Load No.= 1
Load= 12.5 kPa
D_0 = -0.0004
D_{90} = -0.0052
D_{100} = -0.0057
T_{90} = 5.75 min.

C_v @ T_{90}
0.147 cm.^{2}/min.



Load No.= 2
Load= 25.0 kPa
D_0 = -0.0266
D_{90} = -0.0378
D_{100} = -0.0391
T_{90} = 4.62 min.

C_v @ T_{90}
0.180 cm.^{2}/min.

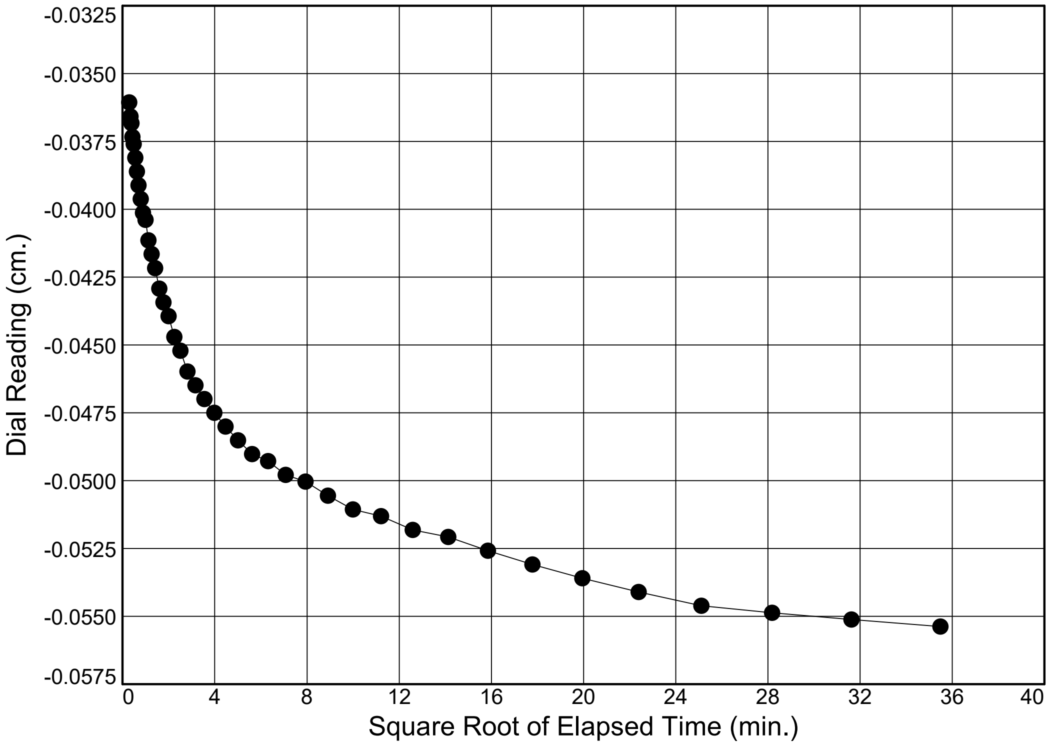
Figure

Solum Consultants Ltd.

Dial Reading vs. Time

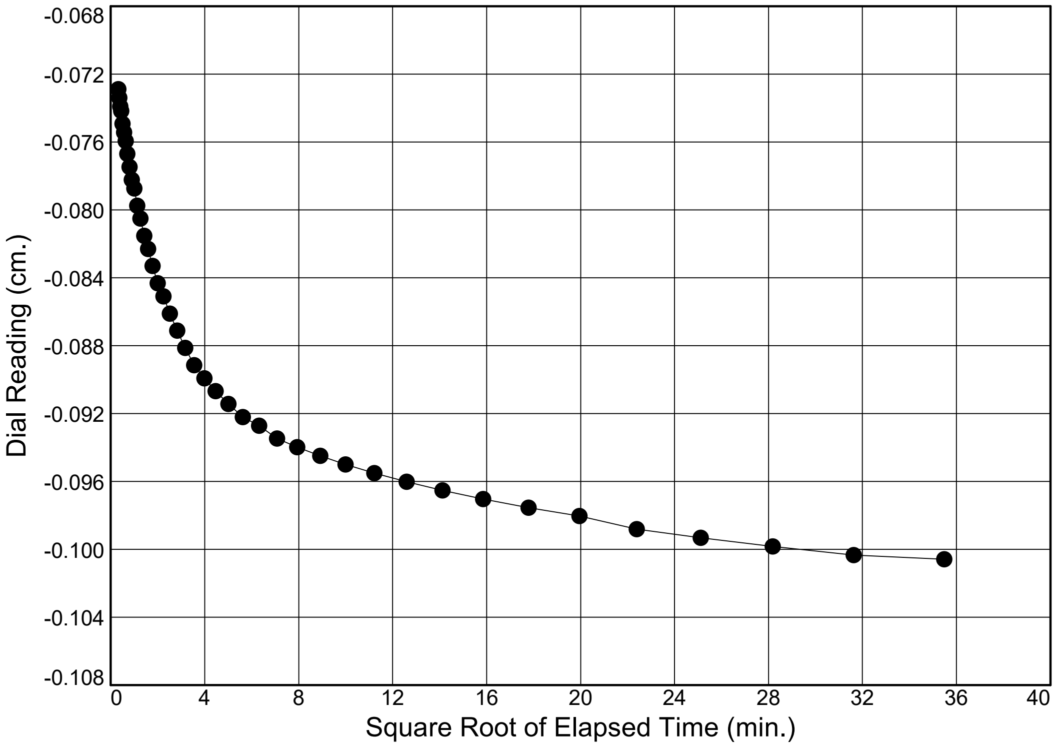
Project No.: 221-07930-00
Project: N/A

Depth: 15-17' Sample Number: ST1



Load No.= 3
Load= 50.0 kPa
D_0 = -0.0896
D_{90} = -0.1122
D_{100} = -0.1147
T_{90} = 4.29 min.

C_v @ T_{90}
0.189 cm.^{2}/min.



Load No.= 4
Load= 100.0 kPa
D_0 = -0.1816
D_{90} = -0.2171
D_{100} = -0.2211
T_{90} = 5.52 min.

C_v @ T_{90}
0.141 cm.^{2}/min.

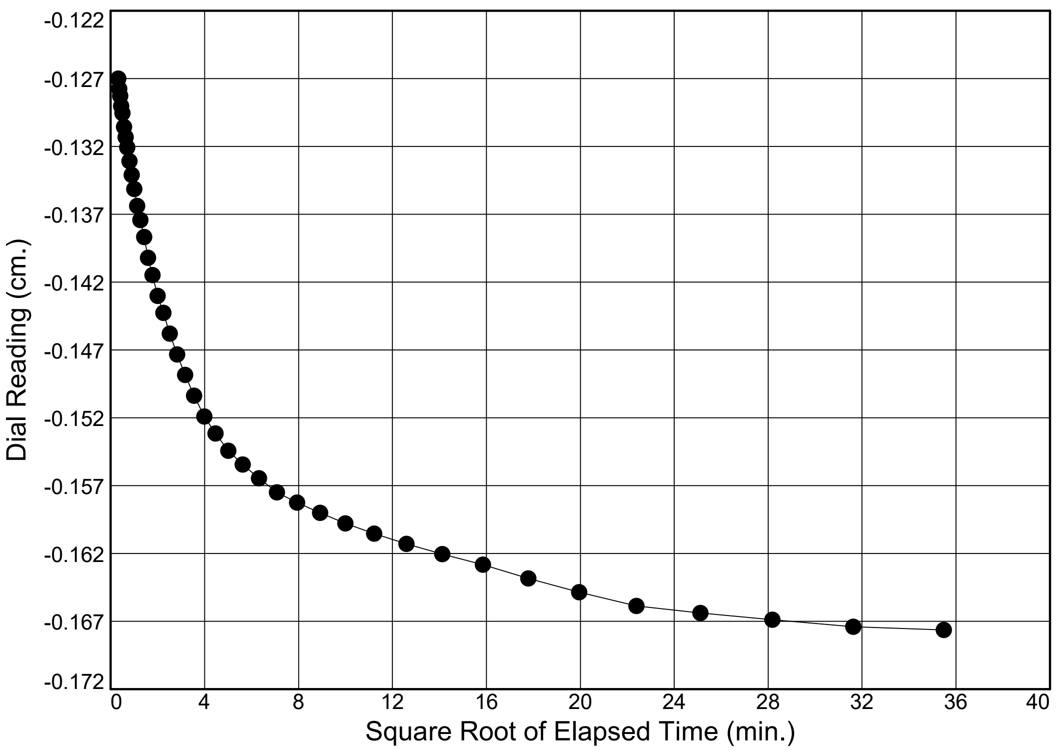
Figure

Solum Consultants Ltd.

Dial Reading vs. Time

Project No.: 221-07930-00
Project: N/A

Depth: 15-17'   Sample Number: ST1



Load No.= 5

Load= 200.0 kPa

D_0 = -0.3182

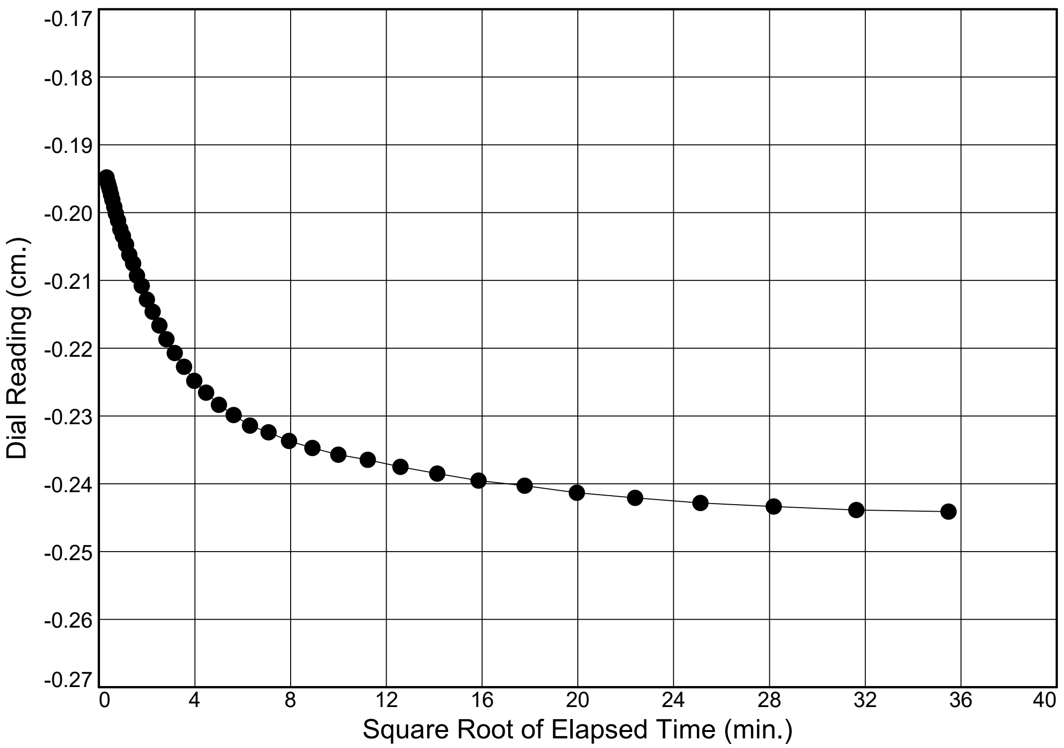
D_{90} = -0.3729

D_{100} = -0.3789

T_{90} = 7.38 min.

C_v @ T_{90}

0.099 cm.^{2}/min.



Load No.= 6

Load= 400.0 kPa

D_0 = -0.4909

D_{90} = -0.5680

D_{100} = -0.5766

T_{90} = 13.95 min.

C_v @ T_{90}

0.048 cm.^{2}/min.

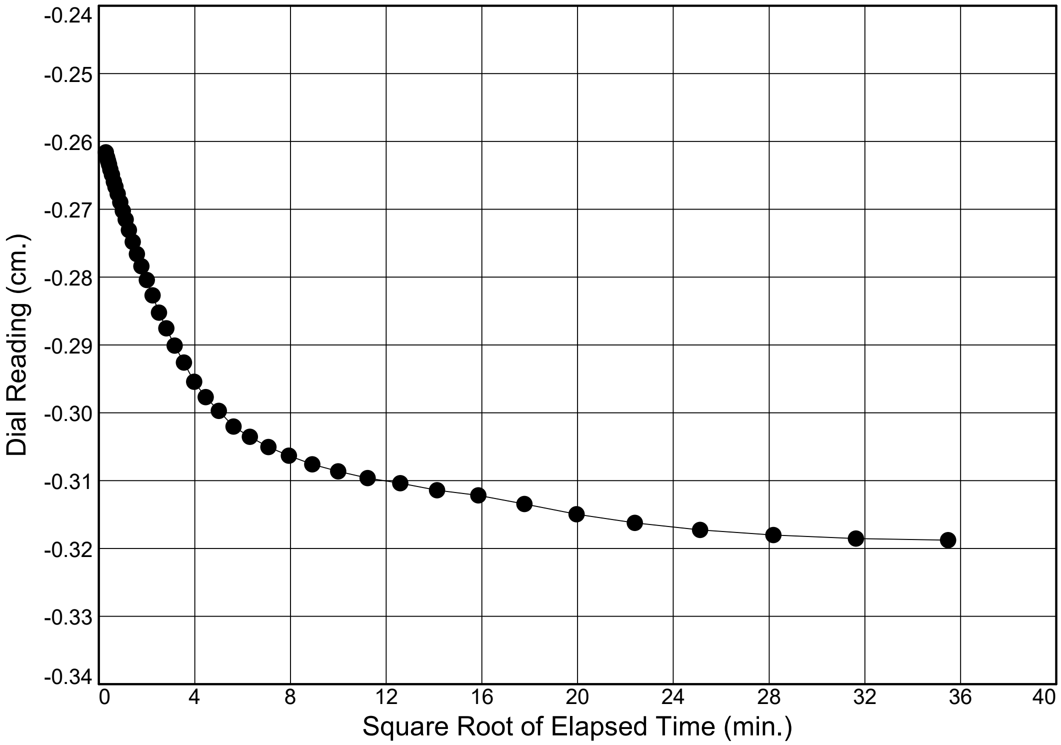
Figure

Solum Consultants Ltd.

Dial Reading vs. Time

Project No.: 221-07930-00
Project: N/A

Depth: 15-17' Sample Number: ST1



Load No.= 7

Load= 800.0 kPa

D_0 = -0.6594

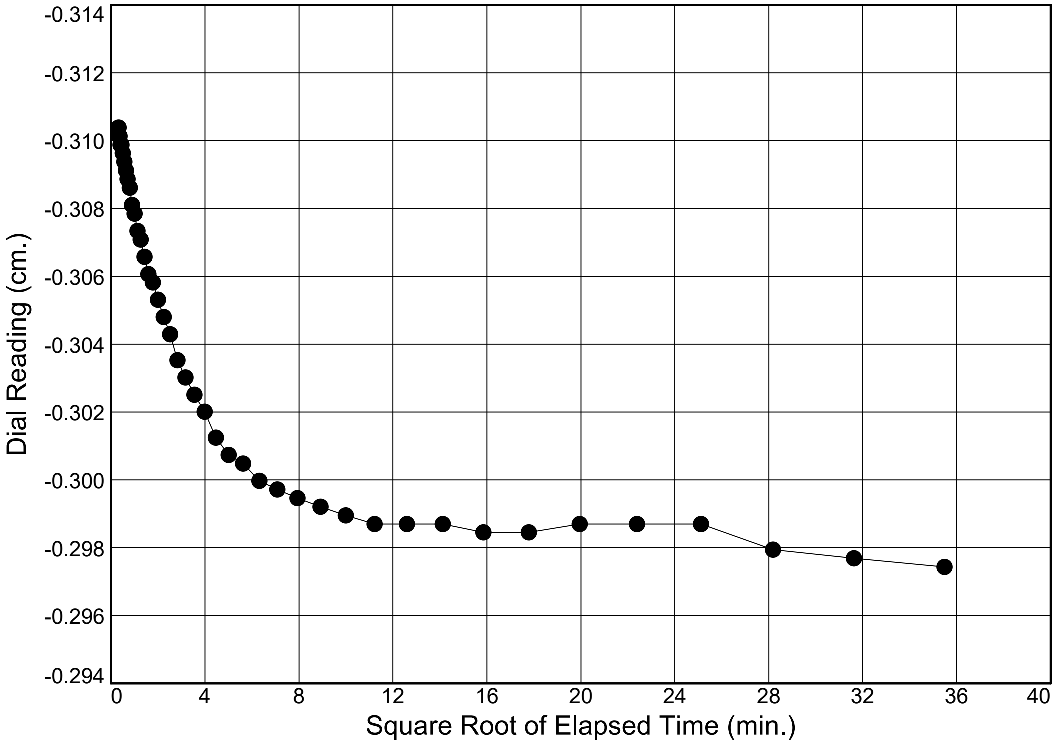
D_{90} = -0.7522

D_{100} = -0.7625

T_{90} = 17.12 min.

C_v @ T_{90}

0.036 cm.^{2}/min.



Load No.= 8

Load= 200.0 kPa

D_0 = -0.7895

D_{90} = -0.7688

D_{100} = -0.7665

T_{90} = 11.70 min.

C_v @ T_{90}

0.052 cm.^{2}/min.

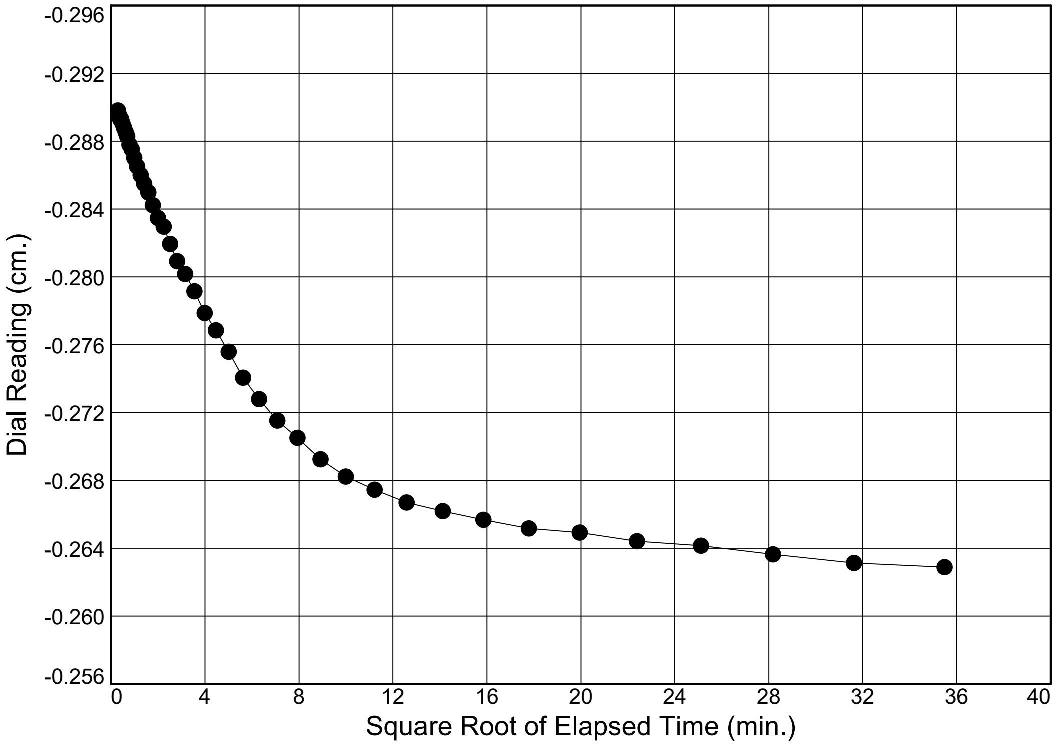
Figure

Solum Consultants Ltd.

Dial Reading vs. Time

Project No.: 221-07930-00
Project: N/A

Depth: 15-17'   Sample Number: ST1



Load No.= 9

Load= 50.0 kPa

D_0 = -0.7377

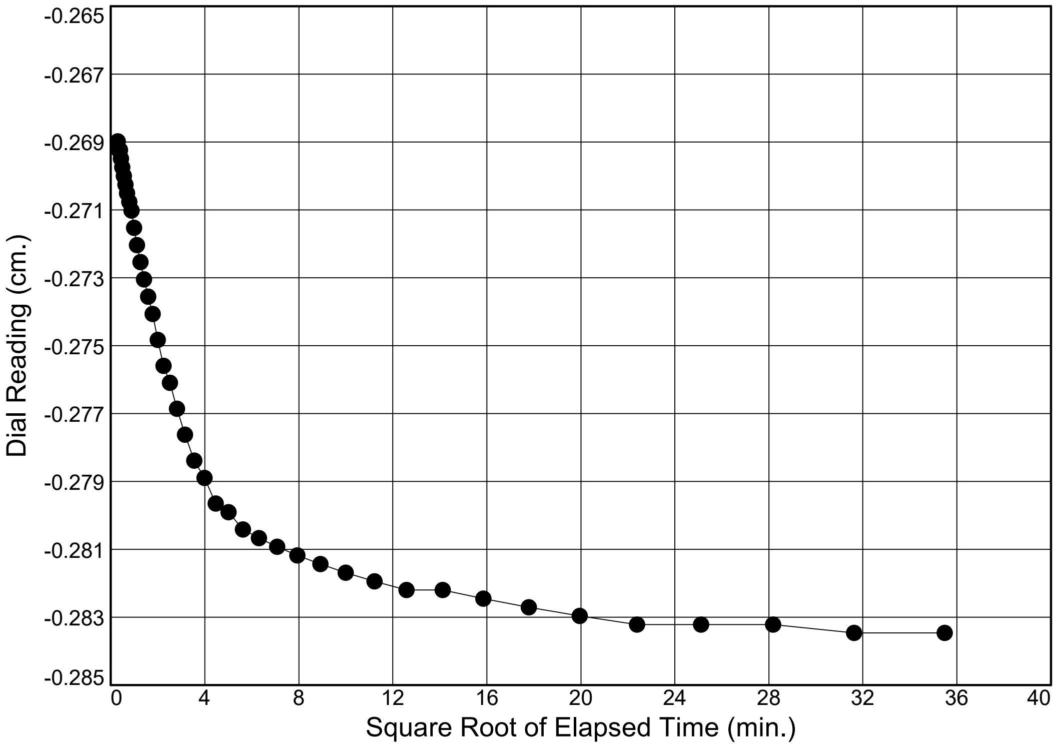
D_{90} = -0.6948

D_{100} = -0.6900

T_{90} = 34.84 min.

C_v @ T_{90}

0.018 cm.^{2}/min.



Load No.= 10

Load= 200.0 kPa

D_0 = -0.6816

D_{90} = -0.7079

D_{100} = -0.7109

T_{90} = 14.69 min.

C_v @ T_{90}

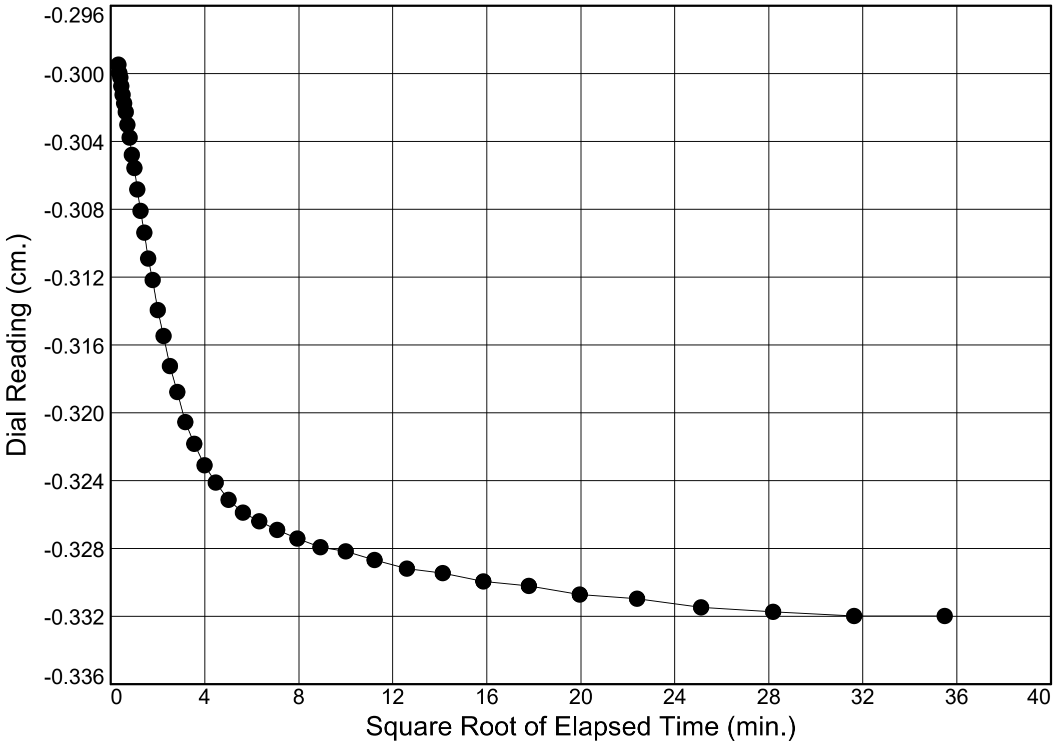
0.043 cm.^{2}/min.

Figure

Dial Reading vs. Time

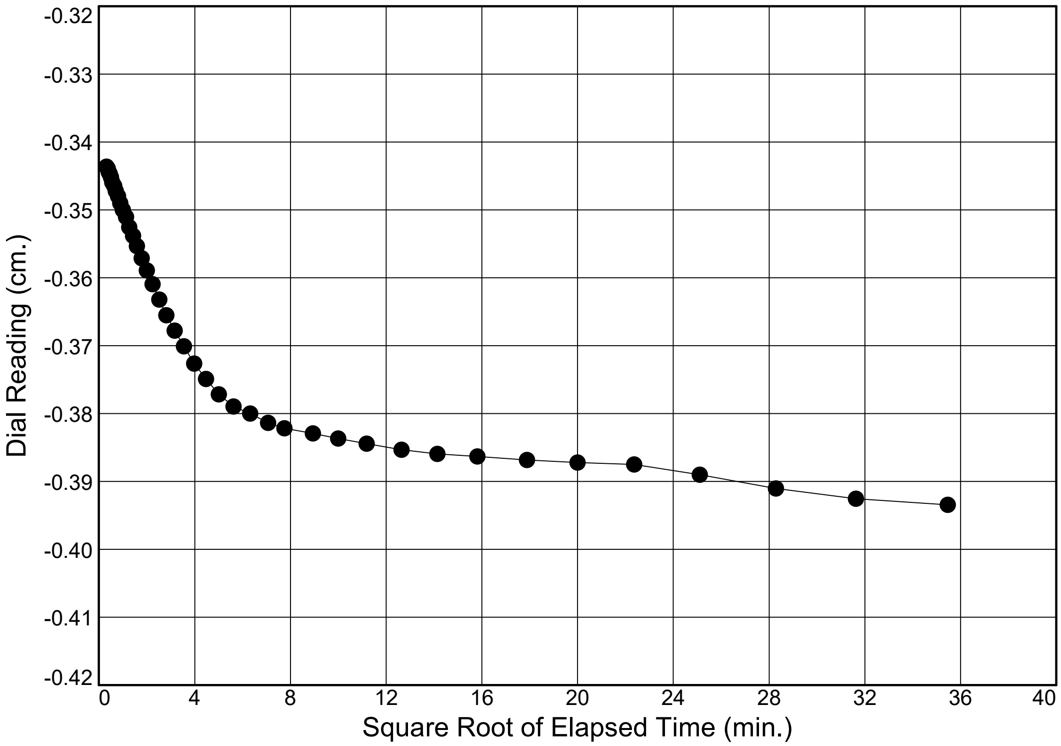
Project No.: 221-07930-00
Project: N/A

Depth: 15-17' Sample Number: ST1



Load No.= 11
Load= 800.0 kPa
D_0 = -0.7548
D_{90} = -0.8153
D_{100} = -0.8220
T_{90} = 10.88 min.

C_v @ T_{90}
0.055 cm.^{2}/min.



Load No.= 12
Load= 1600.0 kPa
D_0 = -0.8668
D_{90} = -0.9539
D_{100} = -0.9636
T_{90} = 21.40 min.

C_v @ T_{90}
0.026 cm.^{2}/min.

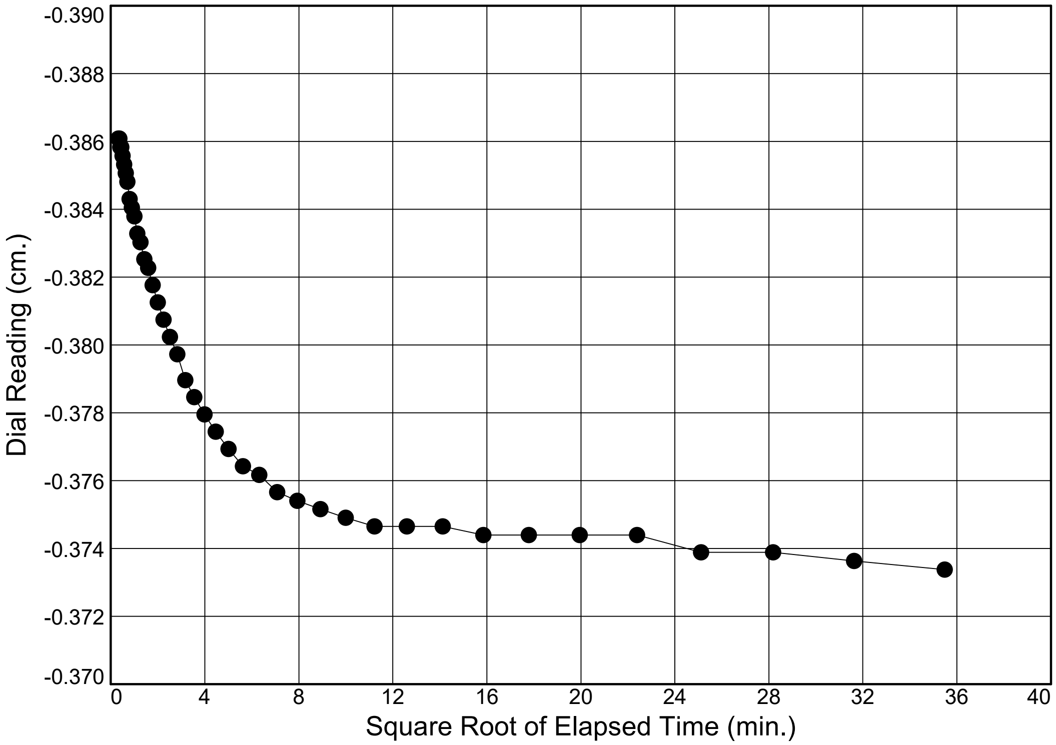
Figure

Solum Consultants Ltd.

Dial Reading vs. Time

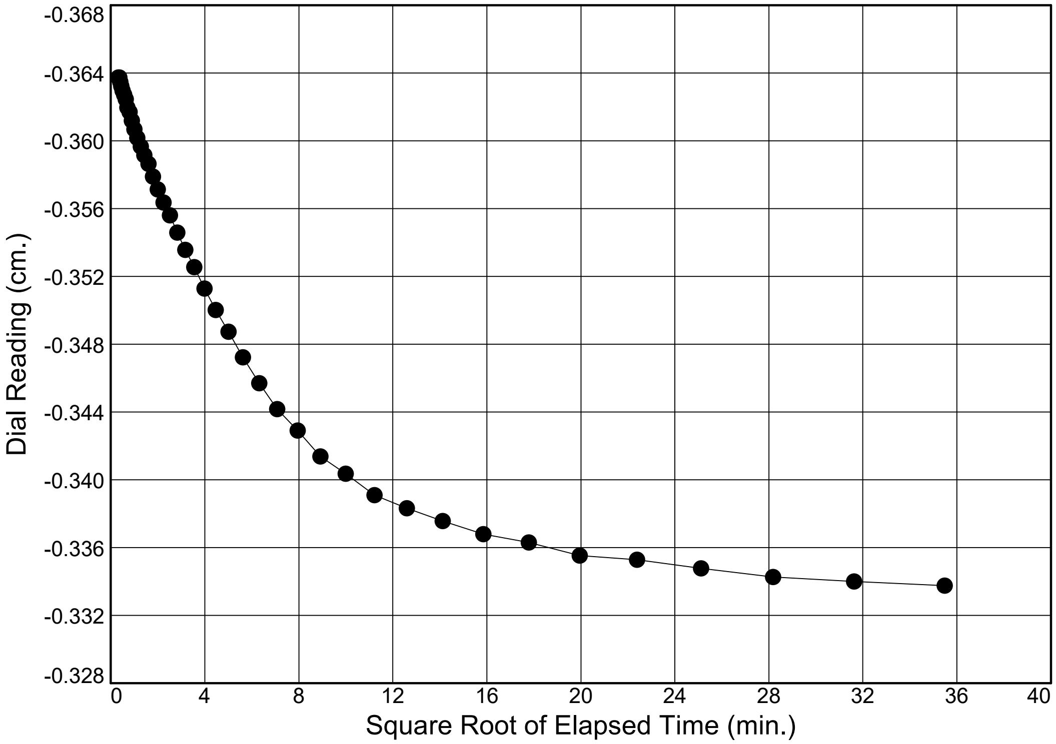
Project No.: 221-07930-00
Project: N/A

Depth: 15-17' Sample Number: ST1



Load No.= 13
Load= 400.0 kPa
D_0 = -0.9820
D_{90} = -0.9607
D_{100} = -0.9583
T_{90} = 14.02 min.

C_v @ T_{90}
0.039 cm.^{2}/min.



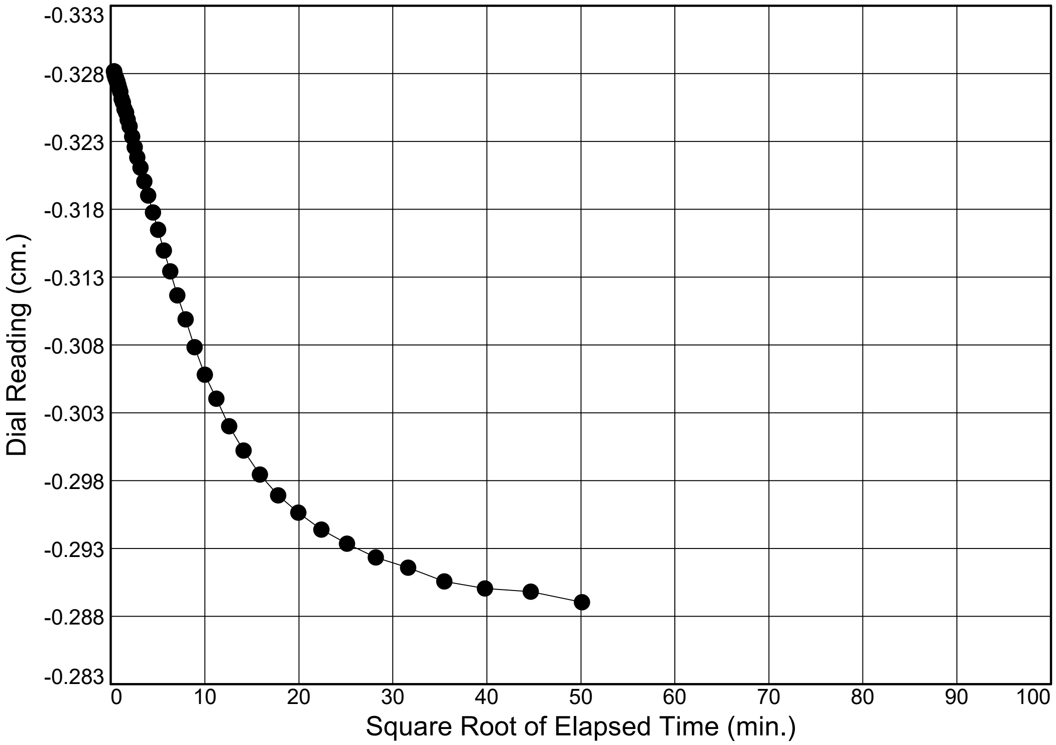
Load No.= 14
Load= 100.0 kPa
D_0 = -0.9252
D_{90} = -0.9206
D_{100} = -0.9201
T_{90} = 0.40 min.

C_v @ T_{90}
1.442 cm.^{2}/min.

Dial Reading vs. Time

Project No.: 221-07930-00
Project: N/A

Depth: 15-17'   Sample Number: ST1



Load No.= 15

Load= 25.0 kPa

D_0 = -0.8353

D_{90} = -0.7651

D_{100} = -0.7573

T_{90} = 176.46 min.

C_v @ T_{90}

0.003 cm.^{2}/min.

Figure

Solum Consultants Ltd.

SOLUM™
CONSULTANTS LTD.
GEOTECHNICAL & MATERIAL
TESTING LABORATORY

Standard Laboratory Terms and Conditions

1.0 Description of Services to be Performed by Solum Consultants Ltd. (Solum)

Solum shall provide geotechnical and material laboratory testing services on samples in general conformance with these terms and conditions and excuted Laboratory Testing Requested Forms. Solum shall perform its work in accordance with accepted laboratory standards and accepted satndard operating procedures as well in-house developed procedures. Solum reserves the right to modify methods as necessary based upon experience and/or current scientific literature. If the Client requests a manner of analysis that varies from standard operating or recommanded procedures, the Client shall not hold Solum responsible for the results. Solum reserves the right to subcontract laboratory testing (especially chemical related testing) if a particular test cannot be performed by Solum after liason with the Client.

2.0 Reports, Confidentiality and Third Parties

Laboratory reports provided by Solum will be composed of a cover page, tables and figures if applicable. Reports will be emailed in PDF format to the individual(s) specified on the Laboratory Testing Request Forms. Laboratory reports may also be faxed or mailed to the Client upon request. Except as required by law, Solum shall not disclose testing results or reports to any party other than the Client, unless the Client, in writing, requests information to be provided to a third party. Solum shall abide by any additional confidentiality requirements requested by the Client provided that such requirements are provided to Solum at or before execution of the testing.

Indormation provided by Solum is inteded for Client use only. Any use by a third party, of reports or documents authored by Solum, or any reliance on or decisions made by a third party based on the findings described in said documents, are the sole responsibility of such third parties, and Solum accepts no responsibility of damages suffered by any third party as a result of decisions made or actions conducted.

3.0 Laboratory Testing Request Form (Chain of Custody)

The laboratory testing request form must be completed by the Client and be accompanied with the samples. Other form of COC may be accepted; however, the condition of Solum COC is still applied. Testing will not commence until the laboratory testing request formhas been completed. If requested by the Client, Solum shall provide a copy of the laboratory testing request form with the report.

No persons other than the designated representitives for each Laboratory Testing Request Form are authorized to act regarding changes to the testing request form. Any changes or amendments of the laboratory testing request form must be in writing and be completed by the originator.

4.0 Acceptance, Contamination and Disposal of Samples

Loss or damages to samples remains the responsibility of the Client until Solum representitives acceptance of samples by notation on the laboratory testing request form.

As to any samples that are suspected of containing hazardous substances, the Client will specify the suspected or known substance and level of contamination. This information is to be stated on the laboratory testing request form and be accompanied with the samples before testing can commence.

Solum may refuse acceptance of samples if it determines they present a risk to health and safety.

Samples accepted by Solum shall remain the property and liability of the Client while in the custody of Solum. Solum will discard all non-contaiminated samples after two weeks of submitting lab report or a month from the date of receiving the samples without additional retention period at a fixed disposal charge, or if requested by the Client, samples may be returned to the Client at no cost to Solum. If requested by Client, Solum will store samples provided the Client agrees to pay for the storage charge. Contaminated material may be returned/shipped to the Client at the Client's expense or Solum will discard samples with disposal rates varying for samples containing higher levels of contamination, refer to price list.

Soil samples will be discarded upon the expiration date of the storage period unless the Client requests either extending storage period or return samples back to client at no cost to Solum.

5.0 Indemnification / Hold Harmless

Solum shall protect, indemnify and save harmless Client, and its directors, officers, employees, agents, represensitives, invitees and subcontractors, and at Client's request, investigate and defend such entities form and against all claims, demands and causes of action, of every kind and character, without limitation, arising in favor of or made by third parties, on account of bodily injury, death or damage to or loss of their property resulting from any negligent act or wilful misconduct of Solum.

The client shall protect, indemnify and save harmness Solum, and its directors, officers, employees, agents, represensitives, invitees and subcontractors, and at Solum's request, investigate and defend such entities form and against all claims, demands and causes of action, of every kind and character, without limitation, arising in favor of or made by third parties, on account of bodily injury, death or damage to or loss of their property resulting from any negligent act or wilful misconduct of Client.

6.0 Limitation of Liability

The total liability of Solum or its staff whether based in contract or tort, will be limited to the lesser of the fees paid or actual damages incurred by the Client.

Solum will not be responsible for any consequencial or indirect damages even if caused by negligence of Solum. Solum will only be liable for damages resulting from negligence of Solum. All claims by the client shall be deemed relinquished if not made within three months after lab report submittal date. No warranty is either expressed or implied, or intended by any agreement or by funishing oral or written reports or findings.

7.0 Termination of Testing Work Order

The client may order work suspended or terminated upon seven days advance written notice. If work suspended, Solum shall receive, upon resumption, and adjustment in the cost of services to compensate for additional costs incurred due to the interruption of services. Upon suspension or termination, Solum shall preserve samples provided that the Client agrees to pay the sample storage charge.

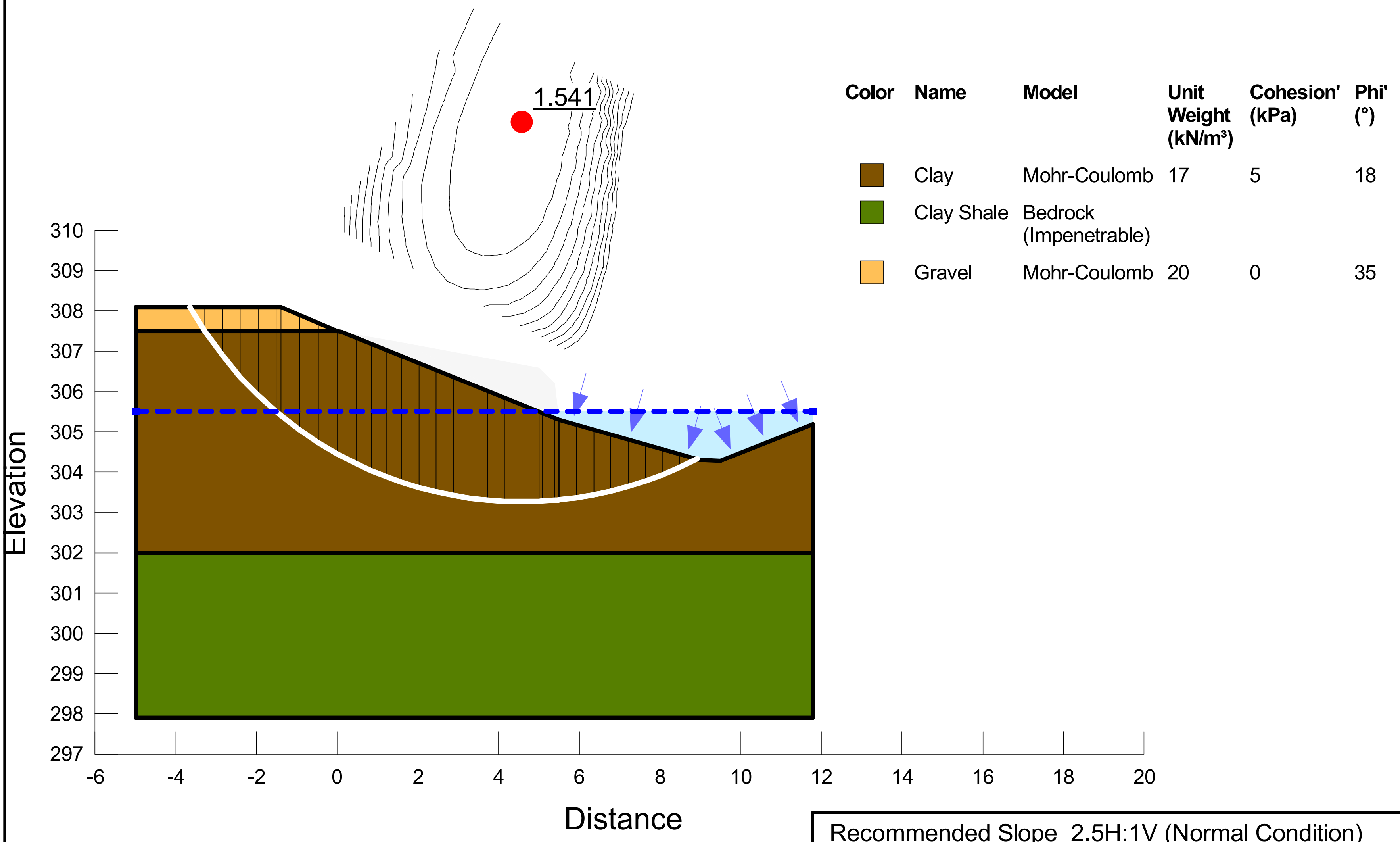
8.0 Pricing, Payments and Invoicing

Invoices will be based on most current Solum laboratory testing rates; rates may change without notice. Solum invoices shall be paid within thirty(30) days of receipt of the invoice. Amounts not paid when due shall bear interest at the rate of 18% per annual from the date due until the date of payment.

APPENDIX

D

COMPUTER MODELLING OUTPUT

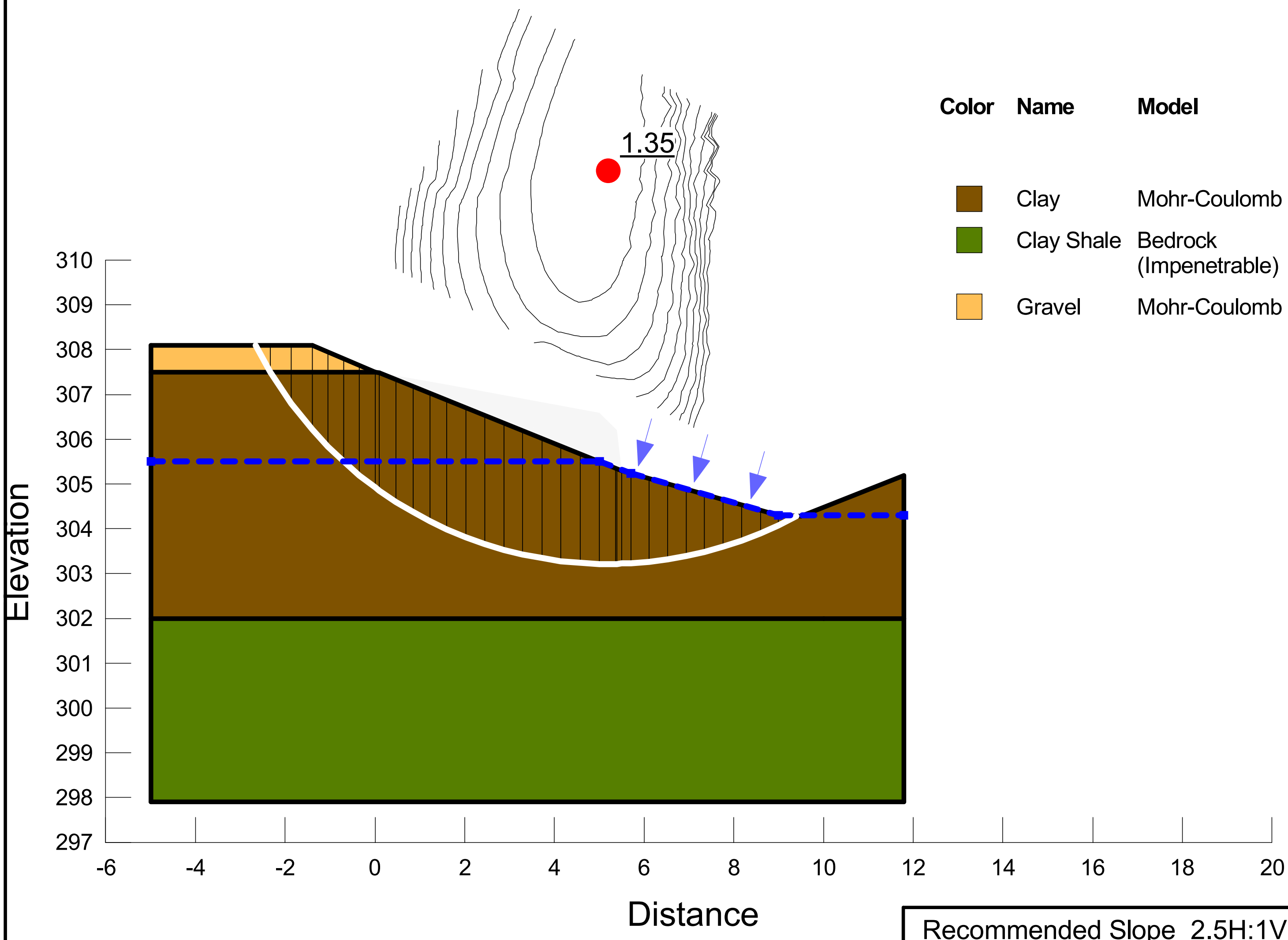


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟧	Gravel	Mohr-Coulomb	20	0	35

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

Cross Section 1_West Embankment

221-07930-00_Alvey St.

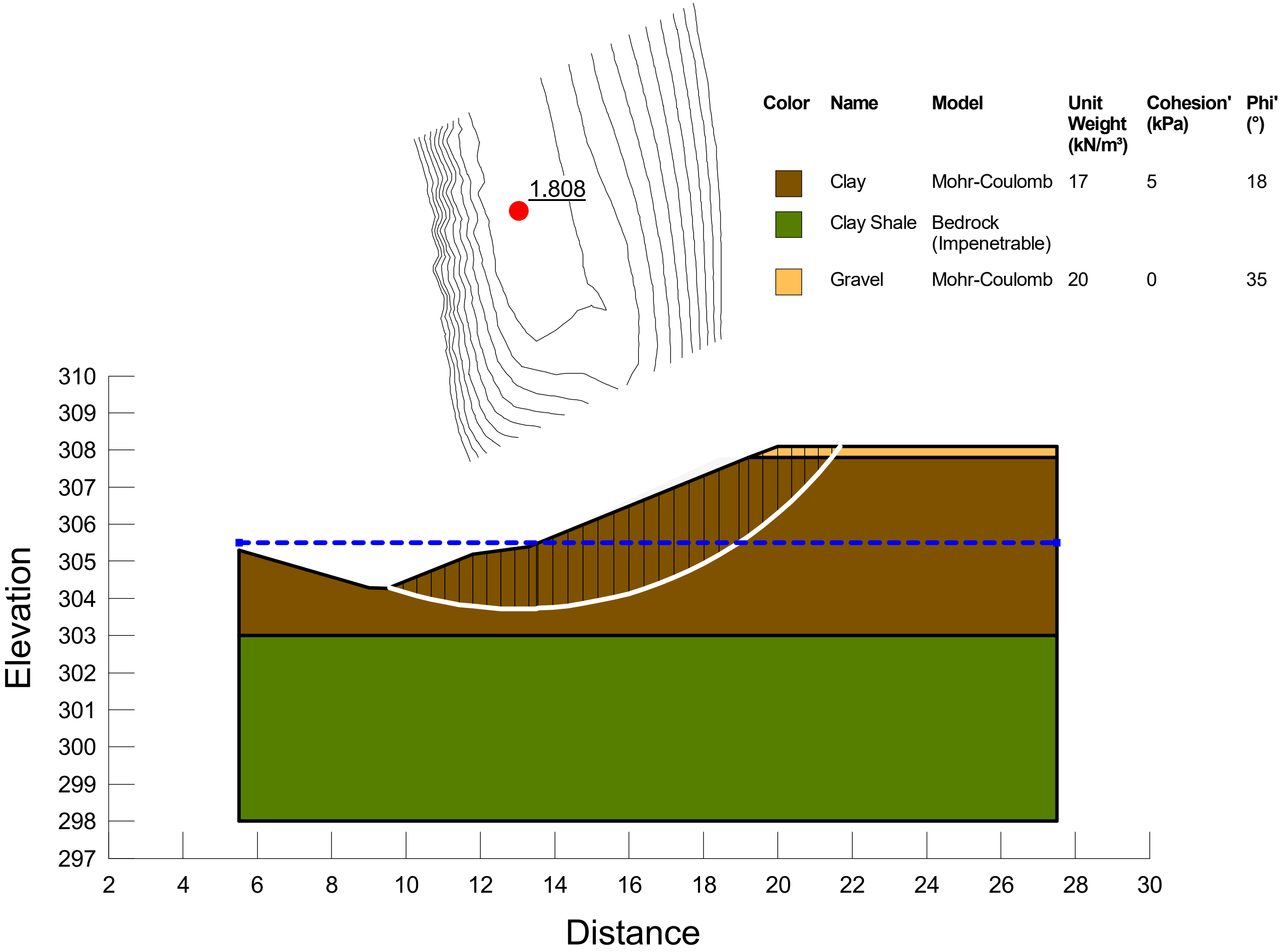


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟧	Gravel	Mohr-Coulomb	20	0	35

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

Cross Section 1_West Embankment

221-07930-00_Alvey St.

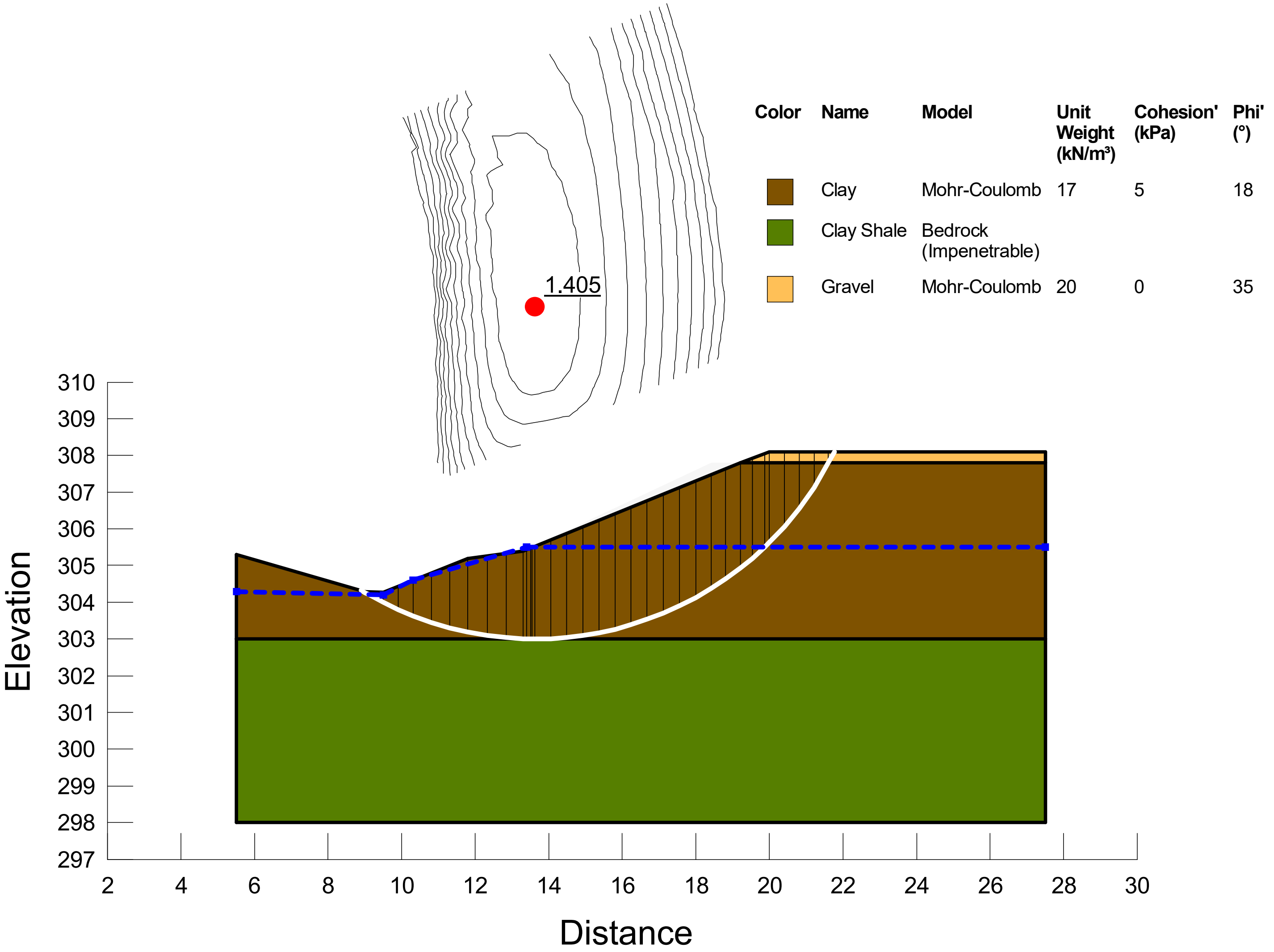


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟨	Gravel	Mohr-Coulomb	20	0	35

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

Cross Section 1_East Embankment

221-07930-00_Alvey St.

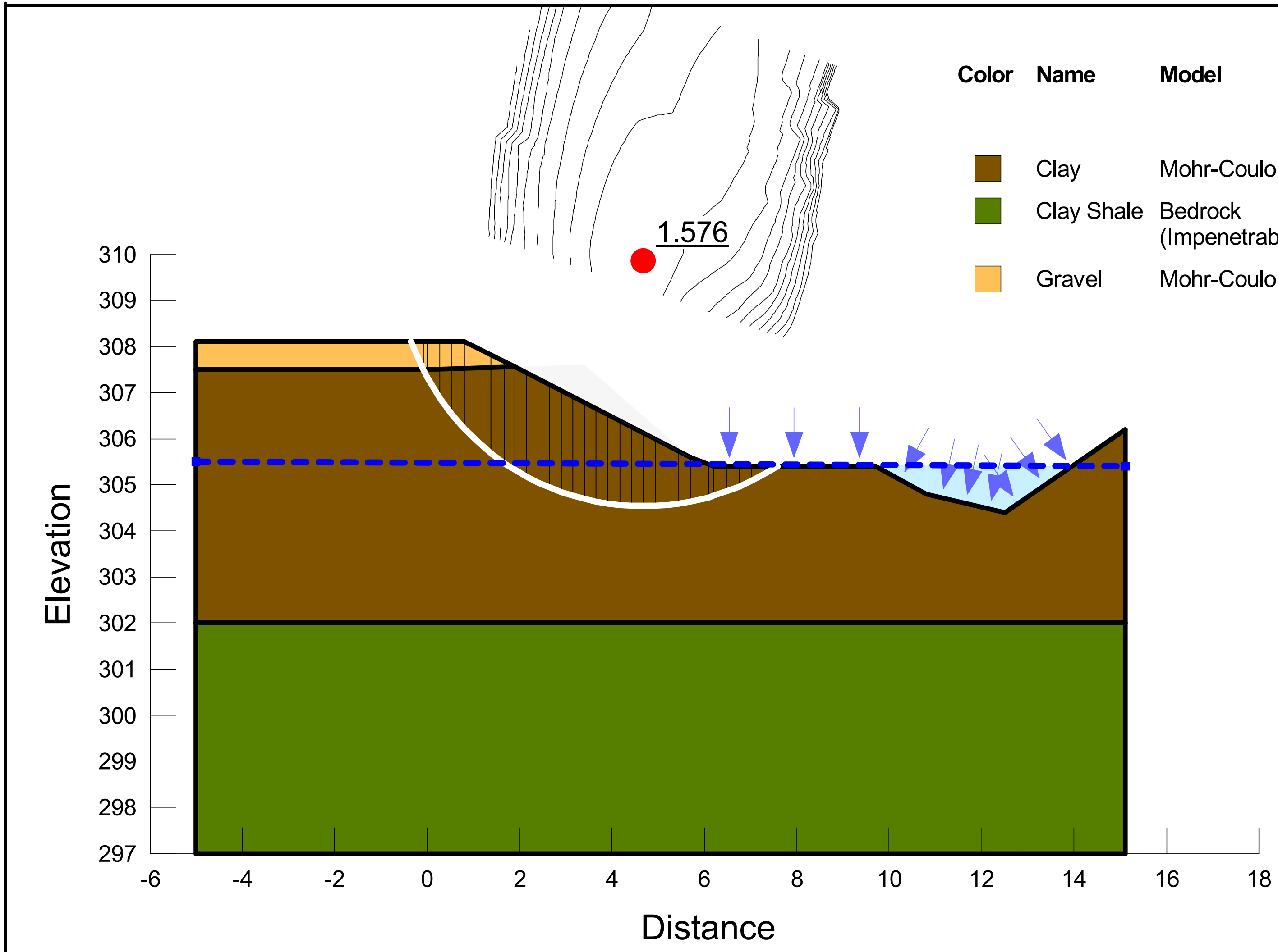


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟧	Gravel	Mohr-Coulomb	20	0	35

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

Cross Section 1_East Embankment

221-07930-00_Alvey St.

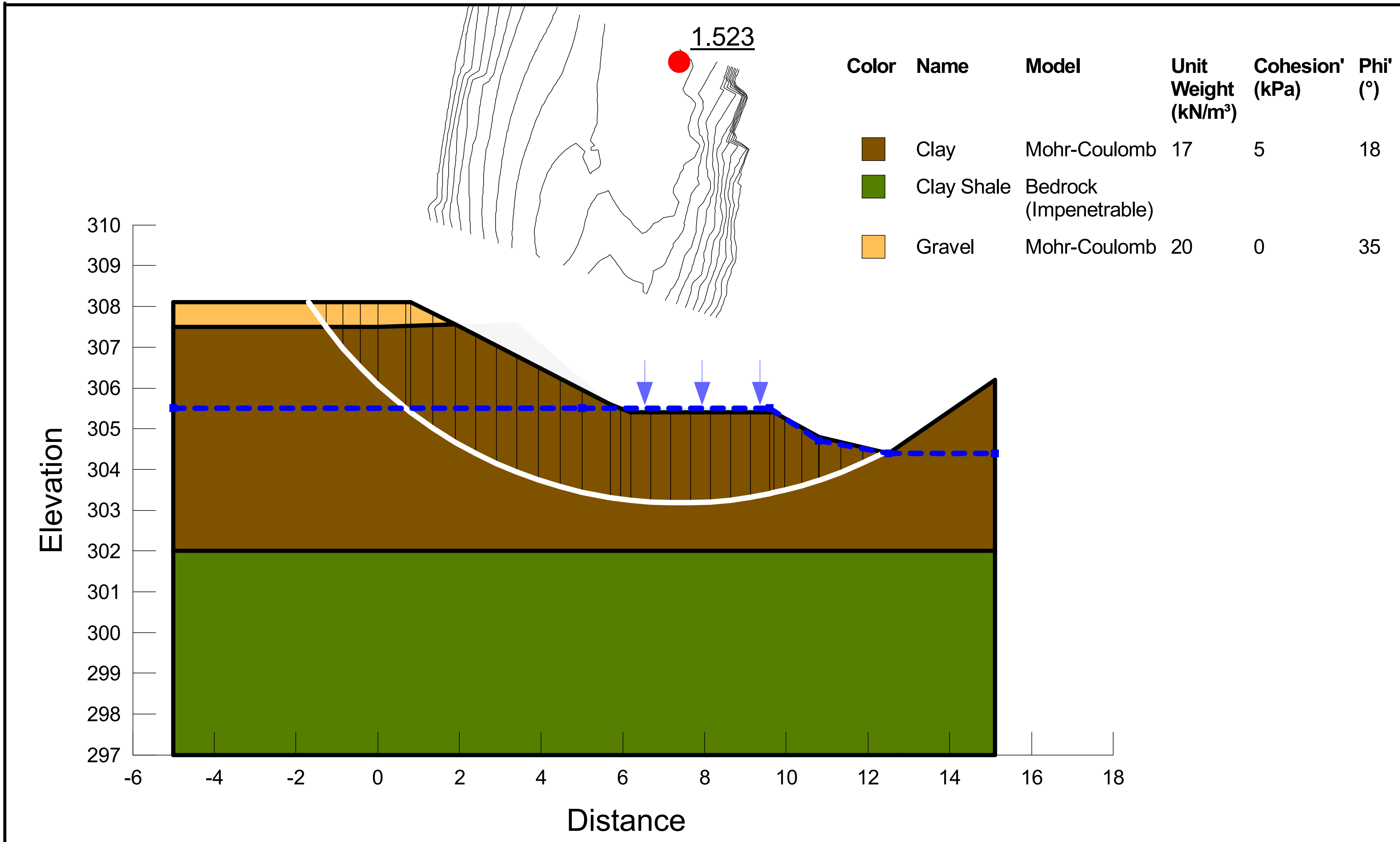


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟧	Gravel	Mohr-Coulomb	20	0	35

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

Cross Section 2_West Embankment

221-07930-00_Alvey St.

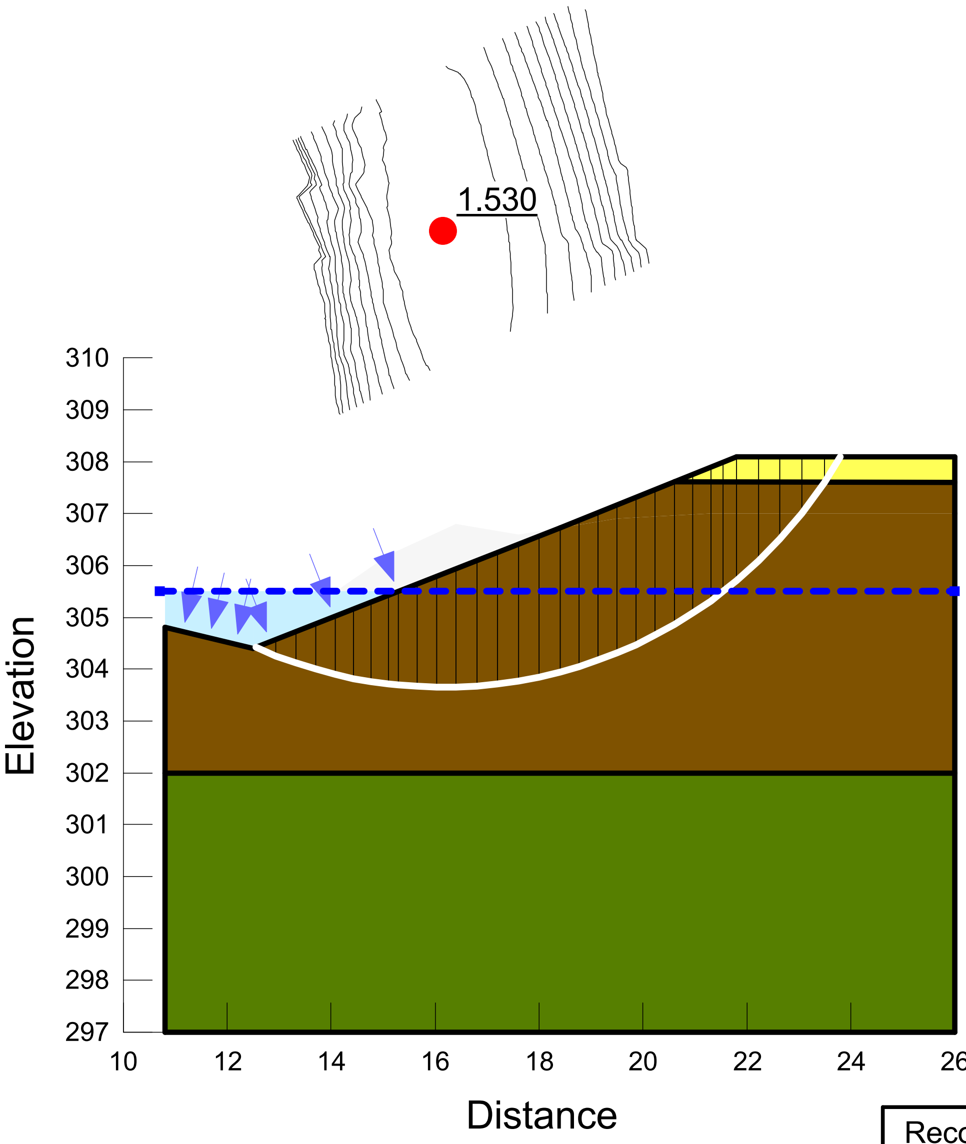


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟧	Gravel	Mohr-Coulomb	20	0	35

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

Cross Section 2_West Embankment

221-07930-00_Alvey St.

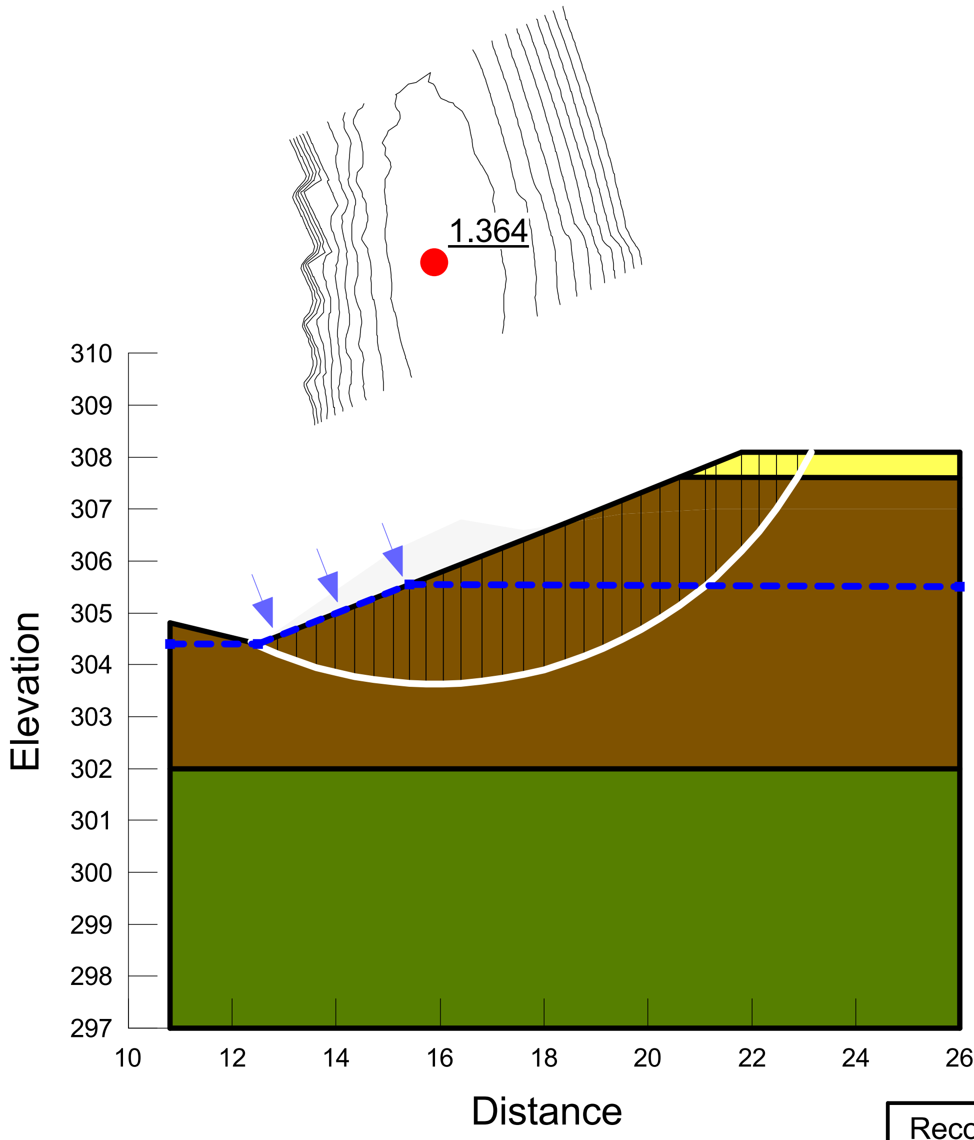


Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
🟫	Clay	Mohr-Coulomb	17	5	18
🟩	Clay Shale	Bedrock (Impenetrable)			
🟨	Granular	Mohr-Coulomb	20	0	35

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

Cross Section 2_East Embankment

221-07930-00_Alvey St.



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 2_East Embankment

221-07930-00_Alvey St.