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Highway 1 Widening – 264th Street to Whatcom Road (Segment 2)

MT. LEHMAN UNDERPASS (STRUCTURE NO. 1562)

GEOTECHNICAL RECOMMENDATIONS (REVISION 2) 100% DETAILED DESIGN

Client Name: Associated Engineering (B.C.) Ltd.
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1. INTRODUCTION

At the request of Associated Engineering (B.C.) Ltd., Thurber has prepared this report summarizing our draft geotechnical design recommendations for 100% Detailed Design of Mt. Lehman Underpass, BC Ministry of Transportation and Infrastructure (MoTI) Structure No. 1562 (Bridge) in Abbotsford, B.C. This revision of the report provides recommendations for the underpass that are based on progress prints of the 100% Detailed Design drawings provided to us on December 11, 2023 (Current Drawings). Geotechnical recommendations related to the highway widening below the underpass will be provided in the Highway Grading Report.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Limitations and Conditions.

1.1 Existing Conditions

The existing underpass is an approximately 77 m long, four-span structure with concrete box stringers that crosses Highway 1 at about Sta. 2028+70. The general arrangement of the existing underpass is shown in Figure 1.1 (as-built Dwg. 1562-102-Rev. A dated August 22, 2007) in Appendix A. The span lengths ranged from 15.8 m to 22.9 m. It was designed and constructed between about 2004 and 2007 to replace an original bridge that was constructed circa 1959 located to the east. The approximate footing locations of the 1959 bridge are shown in Figure 1.7 in Appendix A.

The existing underpass is supported on conventional spread footings as shown on as-built drawings presented in Figures 1.1 to 1.3 in Appendix A. According to as-built Dwg.1562-103-Rev. A dated August 22, 2007 (Figure 2), the abutment footings with a typical width of 1.4 m are perched on top of the mechanically stabilized earth (MSE) retaining walls which were designed and supplied by Reinforced Earth Company (RECO). For the piers, the footing width was typically 2.5 m. Based on our interpretation of as-built Dwgs. 1562-101-Rev. A and 1562-103-Rev. A dated August 22, 2007 (Figures 1.3 and 1.2, respectively), the footing embedment depths are estimated to be about 1.9 m for Pier 1 and Pier 2 and about 1.6 m for Pier 3.

As-built Dwg. 1562-102-A dated August 22, 2007 (Figure 1.1) and Trow Associates Inc. (Trow)'s geotechnical report dated September 23, 2004 indicate that the existing underpass was designed in accordance with the Canadian Highway Bridge Design Code (CHBDC) S6-00. The design seismic event for the CHBDC S6-00 had a 10% probability of exceedance in 50 years (A475). Trow's geotechnical report recommended a peak ground acceleration (PGA) of 0.2g and a site coefficient of 1.2 corresponding to Soil Profile Type II for seismic design.

As-built Dwg. 1562-102-A (Figure 1.1) indicated that a horizontal ground acceleration coefficient of 0.20 and a soil amplification factor of 1.2 were used to design the existing underpass.

MoTI provided us with RECO's as-built drawings of the existing MSE walls during the 100% detailed design phase. RECO's as-built drawings are provided in Figures 1.4 to 1.6 in Appendix A. Figure 1.4 indicates that the existing MSE walls were designed using a PGA of 0.2g.

1.2 Proposed Upgrades

The existing underpass will be widened to the east by about 8.3 m to accommodate a new 3.6 m wide southbound lane and a new 3.5 m wide multi-use path (MUP) as shown in Figure 1.7 in Appendix A. In general, the existing pier and abutment footings will be extended to the east to support the widened section. According to AE, the dimensions of the new pier and abutment footings will be the same as those for the existing footings to develop a consistent structural response under static and seismic loading conditions.

At the abutments, placement of about 6.5 m high fill will be required to facilitate the proposed widening. New MSE walls parallel to the highway alignment will be used to retain the new abutment fills.

To retrofit the existing piers, AE proposes micropiles be installed between pier columns to provide axial compressive and tensile resistances under seismic loading conditions. Following the micropile installation, the opening between the columns will be infilled with concrete. Given that the new piers will have the same arrangement as the existing piers, micropiles will also be installed between the new pier columns in a similar manner.

2. SITE CONDITIONS

2.1 General

Construction History

This section of Highway 1 was constructed in a cut and the original bridge was constructed in the 1960's. Available historical drawings suggest that the original ground level was about 4 m to 5 m above the existing highway grades as shown in Figure 2.1. The design embankment slopes were at inclinations of 2H:1V to 1.5H:1V.

Historical and Current Investigations

Our design has been based on results from both historical and current investigations. A summary of relevant historical and current geotechnical investigations completed at or in the proximity of the existing underpass are provided in Appendix B and summarized below.

- Between 2022 and 2023, a site-specific geotechnical investigation was completed by Thurber to support detailed design of the proposed widening. The investigated locations are shown on Dwg. 32079-SEG 2-3 and the results of the investigation are provided in Appendix B. In summary, the investigation program included the following:
    - A cone penetration test (CPT) profile with shear wave velocity measurements to a depth of 11.9 m and a solid-stem auger test hole to a depth of 18.3 m at SCPT22-SEG 2-01 (termination due to practical CPT and auger refusal).
    - A sonic test hole to a depth of 30.5 m at SH22-SEG 2-01.
    - A mud-rotary test hole with SPT measurements and a vibrating wire piezometer (VWP) to a depth of 30.5 m at MRH22-SEG 2-06.
    - Downhole seismic testing (DHST) at MRH22-SEG 2-06.
    - A CPT profile to a depth of 15.3 m and a solid-stem auger test hole to a depth of 15.2 m at CPT22-SEG 2-15.
    - Three solid-stem auger test holes with dynamic cone penetration test (DCPT) profiles to a typical depth of 9.1 m at TH22-SEG 2-71, -72 and -73.

- In 1959, four test holes designated TH1 to TH4 were drilled by the Department of Highways along the original underpass alignment. The test holes were advanced to depths of about 15 m to 25 m below the original grades in conjunction with standard penetration test (SPT) measurements.

- In 1994, two test holes, designated TH94-15 and TH94-16, were drilled by the Ministry of Transportation and Highways in the close proximity to the original underpass alignment. The test holes were advanced to a depth of about 15.5 m below the site grade with SPT measurements.

There were historical geotechnical investigations completed by others near the existing underpass. However, these investigations were either located further away from the existing underpass or completed using test pits only. Hence, we have not referred to the results of these investigations in our current assessment.

Sacrificial Micropile Testing

A sacrificial micropile testing program was completed near the north abutment of the existing bridge during the 100% detailed design phase. The intent was to determine the soil-grout bond resistance for design of micropiles. Details and results of the sacrificial micropile testing are provided in Section 2.5 below.

2.2 Surficial Geology

Geology Survey of Canada (GSC)'s surficial map 1485A suggests that the site is underlain by glaciomarine deposits as part of the Fort Langley Formation (FLc). Typically, glaciomarine deposits comprise stony silt to loamy clay.

2.3 Soil Conditions

In general, the results from the current and historical investigations suggest that the site is underlain by fill over native silty clay (FLc). From the recent investigation, fill comprising sand and gravel to gravelly sand was encountered to depths of 0.6 m to 1.4 m below the highway grades. The fill thickness was about 1.1 m at the north abutment and about 2 m at the south abutment from the Mt. Lehman Road level. Where SPT or DCPT blow counts were measured, the fill appeared to be compact to dense.

Below the fill, generally stiff to very stiff silty clay was encountered to the depth investigated in the current and historical test holes completed along the Bridge alignment. Discontinuous silt layers were encountered in SH22-SEG 2-01, SCPT22-SEG 2-01 and TH22-SEG 2-73, as well as in the CPT profiles. The CPT and SCPT results from SCPT22-SEG 2-01 and CPT22-SEG 2-15 suggest that the silty clay is highly over-consolidated. Locally at MRH22-SEG 2-06, a layer of compact to dense sand and silt was encountered below the fill to a depth of about 7 m with uncorrected SPT values greater than 27. Further, lenses of sand and gravel may be present within the silty clay layer where SCPT22-SEG 2-01 was terminated. An approximately 0.8 m thick layer of organic clay was encountered at the north abutment (TH22-SEG 2-71) at a depth of about 3.8 m (El. 94.3 m).

A generalized soil profile along the underpass in the longitudinal direction is shown in Dwg. 32079-SEG 2-15 in Appendix B.

2.4 Groundwater and Surficial Drainage

From the recent investigation, groundwater was encountered at depths of 7.5 m to 9.2 m (El. 81.3 m to El. 89.9 m) in the open holes of TH22-SEG 2-71 to -73 during drilling. These measurements may not represent the stabilized groundwater level.

Groundwater levels were monitored at MRH22-SEG 2-06 between August 31, 2022 and March 22, 2023. Two readings were taken per day during the monitoring period. In general, an average groundwater depth below the highway grades of about 8 m (El. 83.8 m) was recorded between September 2022 and mid-October 2022. The average groundwater depth below the highway grades was about 6 m (El. 85.8 m) between mid-October 2022 and March 2023.

For reference, historical test holes indicate that groundwater was encountered at depths of 1 m to 2 m or lower from the highway grades. Groundwater levels expected to vary seasonally with infiltration and surface drainage conditions and groundwater may be perched at the top of the silty clay layer.

2.5 Sacrificial Micropile Testing

The sacrificial micropile testing program was carried out by Kani Foundation Technologies Inc. (Kani) under subcontract to Thurber. Three sacrificial micropiles were installed between July 5 and 7, 2023. They were tested on July 10 and 11, 2023. Prior to initiation of ground disturbance activities, a BC One Call notification was completed and Western U.T.S. Utility & Technical Services Ltd. (Western) was retained by Thurber to complete a field utility locate on June 30, 2023. A field engineer from Thurber was on site full-time to coordinate the field work and witness and log the sacrificial micropile installation and testing.

The test location was situated within the northwest quadrant of the interchange, approximately 120 m west of the piers, as shown in Figure 2.2 in Appendix A. The test location was selected with approval from MoTI due to ease of construction access. A second test location in the median beside the pier was also provided for MoTI consideration as an option but it was not selected due to access challenges and headroom restrictions. TH22-SEG 2-70 was drilled in the vicinity of the test pile location to confirm that the soil conditions at the test location comprising silty clay are consistent with soil conditions encountered from test holes completed at the piers. The drill holes advanced to install the micropiles also encountered silty clay. The general soil conditions at the test location comprise topsoil, variable fill and soft to firm silty clay to a depth of about 3 m, below which firm to very stiff silty clay was encountered to the full depth of the anchor holes at the test site.

Each micropile comprised a #10 (32 mm nominal diameter) steel threadbar (517 MPa) installed in a 150 mm diameter cased hole. The bond length for each sacrificial micropile was 5 m in the native silty clay. Each micropile was installed at various depths below the surface. The sacrificial piles were loaded in approximately 20 kN increments. Each load increment was held for 1 minute.

The test results and our interpretation are summarized as follows:

- Test pile #1 included 1 m of free length and a bond length between depths of 1 m and 6 m. The test pile was stressed to a maximum load of about 305 kN where slippage was observed. Our interpreted maximum load on the pile is about 214 kN. Based on the results, the ultimate (unfactored) bond strength is estimated to be between about 90 kPa and 130 kPa in this zone with variable soil conditions.

- Test pile #2 included 5 m of free length with a bond length between depths of 5 m and 10 m.

- Test pile #3 included 10 m of free length with a bond length between depths of 10 m and 15 m.

- Both test piles were stressed to a maximum load of about 360 kN without observed slippage. Based on the results, the ultimate (unfactored) bond strength is estimated to be at least 150 kPa in the very stiff silty clay.

  According to Table 20.10 in the 5th Edition Canadian Foundation Engineering Manual (CFEM), the estimated ultimate load transfers for soil anchors typically range from 30 kN/m to 60 kN/m for stiff to hard silt and clay. Assuming a drill hole diameter of 150 mm, the corresponding unfactored bond strengths are estimated to be 64 kPa to 127 kPa. Hence, the ultimate unfactored bond strength of 150 kPa obtained from Test piles #2 and #3 is in general agreement with the suggested range in the CFEM.

3. ENGINEERING ASSESSMENT AND RECOMMENDATIONS

3.1 General

From a geotechnical perspective, we consider that conventional footings can be used to support the new pier and abutment footings. Further, the proposed use of micropiles to retrofit the pier footings is considered feasible. Our recommendations for design of footings and micropiles, as well as the new MSE walls, are provided in Sections 3.2 to 3.11.

The MoTI Supplement to S6-19 (Supplement) has been used to design this structure. Two key differences between the CHBDC S6-19 and the Supplement that affect design considerations for this bridge are discussed below.

3.1.1 Seismic Geotechnical Resistance Factor

In CHBDC S6-19, Clause 6.14.4.1 allows the use of a seismic geotechnical resistance factor (GRF) of 1.0 for capacity-protected elements or for performance-based design. However, the Supplement only allows a seismic GRF equal to the static GRF plus 0.2 unless a sensitivity analysis is completed. This requirement had some effect on design of shallow foundations and nominally increased the minimum required bond length for the proposed micropiles.

3.1.2 Rigorous Dynamic Analyses

The height of the proposed MSE walls will be greater than 6 m and the seismic performance category (SPC) for the underpass is 3. Under these circumstances, Clause 6.14.4.2 in the Supplement requires the walls to be assessed using rigorous dynamic analysis using finite element or finite difference methods. To meet this requirement, we have completed dynamic analyses using Plaxis2D during the 100% detailed design phase. Preliminary details are provided in Section 3.4.

3.2 Design Criteria

Geotechnical design criteria for new structures in Segments 1 and 2 were documented in Thurber's letter dated May 8, 2023 to AE. However, this bridge also includes seismic retrofit of the existing structure. Hence, we have excerpted key geotechnical design criteria related to new and retrofitting of the bridge below.

According to AE, the seismic performance criteria for the existing and new structures are summarized below.

- Service level for the new structure:
  - Service disruptions at A2475
  - Immediate at A475
- Damage level for the new structure:
  - Extensive at A2475
  - Minimal at A475

- Service level for the existing structure:
    - Life safety at A2475
    - Service limited at A475
- Damage level for the existing structure:
    - Probable replacement at A2475
    - Repairable at A475

The following has been assumed in the 100% detailed design:

- Importance Category = Major-Route Bridge (Clause 4.4.2 CHBDC S6-19)
- Seismic Performance Category = 3 (Clause 4.4.4 CHBDC S6-19)
- Degree of Understanding = Typical (Clause 6.5.3.2 CHBDC S6-19)
- Consequence Factor = 1.0 (Table 6.1 CHBDC S6-19)
- GRF for bearing resistance of shallow foundations = 0.5 (non-seismic, Table 6.2 CHBDC S6-19)
- GRF for bearing resistance of shallow foundations = 0.7 (seismic, Clause 6.14.4.1 Supplement)
- GRF for sliding resistance of shallow foundations = 0.8 (frictional, non-seismic, Table 6.2 CHBDC S6-19)
- GRF for sliding resistance of shallow foundations = 0.6 (cohesive, non-seismic, Table 6.2 CHBDC S6-19)
- GRF for sliding resistance of shallow foundations = 1.0 (frictional, seismic, Clause 6.14.4.1 Supplement)
- GRF for sliding resistance of shallow foundations = 0.8 (cohesive, seismic, Clause 6.14.4.1 Supplement)
- GRF for passive resistance of shallow foundations = 0.5 (non-seismic, Table 6.2 CHBDC S6-19)
- GRF for passive resistance of shallow foundations = 0.7 (seismic, Clause 6.14.4.1 Supplement)
- Factor of safety for global stability (permanent) = 1.54 (Table 6.2b Supplement)
- Factor of safety for global stability (seismic) = 1.18 (Clause 6.14.4.1 Supplement)

3.3 Seismic Design

3.3.1 Seismic Hazard Values

According to the DHST results at MRH22-SEG 2-06, the time-averaged shear wave velocity in the upper 30 m (V_{s30}) is about 305 m/s, which is within a Site Class D classification in accordance with Table 4.1 in the CHBDC S6-19. However, according to Table 6.14.8.13 in the Supplement, routine analysis based on 1D dynamic site response analysis with equivalent linear models using non-liquefied soil parameters should be carried out for evaluation of liquefaction potential. Hence, a site-specific response analysis (SSRA) was completed.

The results of the SSRA, including design response spectra, are summarized in Appendix C. For completeness, the 2020 National Building Code of Canada (NBCC) seismic hazards corresponding to Site Class D obtained from the Natural Resources Canada (NRC) website are also provided in Appendix C. In general, the design response spectra are governed by the results of the SSRA for a period of vibration shorter than about 0.5 seconds. Beyond that, the spectra generally follow 80% of the Site Class D values in accordance with Clause 4.4.3.1 in the Supplement.

3.3.2 Liquefaction Potential

The liquefaction potential of the underlying soils was assessed in general accordance with Clause 6.14.8.1.3 in the Supplement. For the stress-based approach, the soil cyclic resistance ratio (CRR) profiles were estimated based on methods outlined by Boulanger and Idriss (2014) using the CPT, SCPT and SPT data from the recent investigation. The cyclic stress ratio (CSR) profiles were estimated based on the SSRA results. Based on the groundwater monitoring data at MRH22-SEG 2-06, a groundwater level at a depth of 6 m below the highway grades (about El. 86 m) was assumed in the assessment. The liquefaction triggering analyses were completed using the software programs CLiq by Geologismiki and the results are presented in Appendix D.

The liquefaction potential of the fine-grained soils was also assessed using the plasticity approach in accordance with the Bray and Sancio (2006) method. Atterberg limit tests were completed on sixteen selected samples from SCPT22-SEG 2-01, CPT22-SEG 2-15, TH22-SEG 2-71, TH22-SEG 2-72 and TH22-SEG 2-73. The summary plot is provided in Appendix D.

Our comments on liquefaction potential for the underlying soils under the design A2475 seismic event are summarized as follows:

- The fill layer is expected to be situated above groundwater. Hence, the liquefaction potential was not assessed.

- The native silty clay layer is not expected to experience liquefaction or strain-softening in A2475 given the plasticity index and over-consolidation.

- The sand and silt layer encountered locally at MRH22-SEG 2-06 to a depth of about 7 m is not expected to liquefy based on SPT measurements and most of the layer is expected to be situated above groundwater.

- Potentially liquefaction-susceptible lenses were identified in SCPT22-SEG 2-01 and CPT22-SEG 2-15 from the stress-based method. In our opinion, this is likely related to the "thin lens effect" from the CPT and SCPT data. Results from the plasticity approach suggest that these lenses are not susceptible to liquefaction.

- Layers of sand and silt were encountered within the native silty clay at SH22-SEG 2-01 at a depth of about 24 m, TH22-SEG 2-72 at depths of about 2 m and 8 m and TH22-SEG 2-73 at a depth of about 5 m. Atterberg limit tests were completed on two samples. Even though the plasticity approach suggests that these sand and silt layers may be susceptible to liquefaction or strain-softening, we consider the potential for liquefaction or strain-softening of these layer to be relatively low because the material was deposited thousands of years ago. Literature suggests that it is highly unlikely for aged deposits to liquefy in a seismic event. Furthermore, these layers appear to be relatively thin and discontinuous. Therefore, we do not anticipate that these potentially liquefiable layers will affect the seismic performance of the abutment and pier foundations, as well as the proposed micropiles.

In summary, we consider the liquefaction potential of the soils underlying the existing and proposed structure to be relatively low. Additional information related to seismic deformations is provided in Section 3.4.

3.4 Rigorous Dynamic Analysis

3.4.1 Modelling Approach

In accordance with Clause 6.14.4.2 in the Supplement, a two-dimensional (2D) seismic numerical deformation assessment was completed using the software program Plaxis2D. Plaxis2D is an advanced finite element modelling program that allows for complex modelling of cyclic soil behaviour. The deformation assessment incorporated complex cyclic soil behaviour using the HSsmall and PM4Sand soil models. HSsmall and PM4Sand are both capable of modelling the small-strain stiffness degradation associated with seismic loading. PM4Sand is also capable of modelling pore pressure build-up, liquefaction triggering and post-triggering displacements.

Development of pore pressures is tracked using the excess pore pressure ratio, R_u, defined as the ratio between a soil's excess pore-water pressure (i.e., pressure above hydrostatic) and effective vertical stress. Liquefaction occurs when build-up of pore-water pressure causes soil to rapidly lose shear strength and stiffness. The onset of liquefaction in the soil is generally defined when the R_u exceeds 0.7, but any increase in R_u will induce some strength loss.

The Plaxis2D model geometry and material zoning is shown in Appendix E: Figure E1 for the longitudinal section and Figure E6 for the transverse section. The ground profile used in the models was based on AE's 50% Detailed Design Drawings. The ground water table was assumed to be level across the site at El. 86 m.

HSsmall and PM4Sand soil models were assigned to non-liquefiable and potentially liquefiable soils, respectively. Both HSsmall and PM4Sand are stress-dependent and are implemented in Plaxis with normalized input parameters. Median soil parameters were derived for each soil unit from the available nearby CPTs, SPTs, and shear wave velocity measurements as input for the HSsmall and PM4Sand models. We allowed the soil models to populate soil stiffnesses from the normalized median soil parameters and model stress field.

To understand the potential effects associated with liquefaction, two, 1 m thick silty sand layers centred at El. 85 m and El. 81 m were modelled beneath the south abutment using the PM4Sand soil model. Based on interpretation of the SCPTs, CPTs, and SPTs, blow count values of $(N_1)_{60}$ = 29 and $(N_1)_{60}$ = 17 were assigned to the upper and lower silty sand layers, respectively. We selected higher, mid-range blow count estimates for the soils because the layers are being modeled as both thicker and more continuous than suggested by the soil data. Triggering of the silty sand layers were calibrated using Plaxis SoilTest to match cyclic resistance values from Boulanger and Idriss (2014). Calibration was set on achieving 3% shear strain after 15 uniform cycles. The number of cycles to liquefaction were not adjusted for MSF so the CRR values are based on Magnitude 7.5. Separate calibrations of the PM4Sand parameters were completed for the difference initial stress conditions below the existing abutments and below the highway.

In general, the modelling details are summarized as follows:

- HSsmall was assigned first to all soil layers to establish static stress conditions.
- PM4Sand was assigned to the two silty sand layers for dynamic phases.
- Free-field boundary conditions were applied as lateral boundaries in the models.
- All dynamic runs were completed with groundwater flow on which allows Plaxis2D to calculate pore pressure redistribution during the dynamic phase.

- This typically results in upward seepage and a more realistic distribution of predicted R_u values.
- A hydraulic conductivity of 1 x 10^{-6} m/s was assigned to the two silty sand layers.
- The remaining soils were assigned hydraulic conductivities consistent with published typical value estimates (Freeze and Cherry, 1979).

3.4.2 Seismic Deformations

For this submission, we have analyzed the full suite of 60 ground motions provided by MoTI comprising the following:

- 475-year return period level: 10 Crustal, 10 Inslab, and 10 Subduction ground motions
- 2475-year return period level: 10 Crustal, 10 Inslab, and 10 Subduction ground motions

Input ground motions for our analyses can be found in Golder Associates Ltd. (Golder)'s technical memorandum entitled "Earthquake Scenario Spectra and Acceleration Time Histories for 1/475, 1/975 and 1/2,475 Annual Exceedance Probabilities for Trans Canada – Fraser Valley Project, Abbotsford, British Columbia, Canada" dated February 22, 2022 (Golder's Reference No. 21498748-001-TM-Rev0).

Comparison of the horizontal and vertical displacements estimated using the full suite of motions for each return period are included in Figures E2 to E5 and Figures E7 to E10 in Appendix E. The comparisons show that Inslab motions largely govern the displacement estimates. The average displacement profiles generated from the full earthquake suite are highlighted. We recommend that the average profiles are used to generate deformation estimates for structural evaluation.

Generally, R_u values in the upper silty sand layer layer did not exceed 0.7 for either earthquake return periods. R_u values in the lower silty sand layer were generally higher than in the upper silty sand layer. The R_u values in the lower silty sand layer exceeded 0.7 for some of the inslab and crustal ground motions consistent with the 2475-year return period, but only in free-field conditions away from the abutments. The R_u did not exceed 0.7 beneath the abutment embankments in either the transverse or longitudinal model.

Post-seismic displacement estimates for the average profiles are also provided in tables within the figures in Appendix E. The maximum displacement estimates contained in the average profiles are summarized in Table 3-1. The profiles should be referenced to develop differential lateral and horizontal movements for use in structural evaluation.

3.4.3 Summary of Rigorous Dynamic Analyses

Overall, the rigorous dynamic modelling completed for the Mt. Lehman Underpass indicates that the structure is not anticipated to be subject to significant post-seismic displacements with maximum design values below about 25 mm for the 1 in 475-year return period ground motions and below about 50 mm for the 1 in 2475-year return period ground motions. It should be noted that the Plaxis models did not include any contributions from lateral resistance by the bridge superstructure during dynamic analyses. These effects (e.g., a bridge ‘strutting’ force) are not possible to accurately model without a significantly more complex geostructural model but would be expected to reduce the deformation from those provided in our estimates.

Table 3.1: Maximum Average Post-Seismic Displacements for Mt. Lehman Abutment Models

Condition	**1 in 475-year Earthquake**		**1 in 2,475-year Earthquake**	
	Transverse Model (mm)	**Longitudinal Model (mm)**	**Transverse Model (mm)**	**Longitudinal Model (mm)**
Maximum Average Horizontal Displacement	5 to 10	20 to 25	20 to 30	30 to 40
Maximum Average Vertical Displacement	-10 to -15	-10 to -15	-35 to -40	-25 to -35

Note: Negative vertical displacement = settlement

3.5 Micropile Design

3.5.1 General

AE has proposed the use of micropiles to retrofit the existing pier footings and reinforce the new pier footings under seismic loading conditions. The typical general arrangement for piers is shown in Figure 3.1. In general, micropiles will be installed between the existing and new pier columns through the footings. According to AE, a construction sequence has been developed such that the new structure including the footings, micropiles and the superstructure will first be constructed without infilling the areas between new pier columns, i.e. the new structure at this stage will be the same as the existing structure. This allows the new pier footings to support all superstructure loads under service conditions first. Following a waiting period and immediately prior to pouring concrete for the infill walls, the micropiles will be grouted and the heads will be installed to engage the pier footings.

According to AE, micropiles will primarily support seismic loads. Under service loading conditions, some live loads may be transferred to the micropiles but the demands are expected to be below the design seismic loads as outlined below.

For permanent applications, the micropiles should comprise double corrosion protected (DCP) solid threadbars. In general, the micropiles will be divided into two sections, free-stressing and bonded lengths. Geotechnical inputs for design of the micropiles are provided below. Structural design of the micropiles and the required number of micropiles will be completed by AE based on our recommendations provided below.

3.5.2 Free-Stressing (Unbonded) Length

We recommend a minimum free-stressing length of 1 m be provided below the underside of the pier footings. The free length should be developed by placing prefabricated smooth plastic sheathing over the DCP threadbars.

3.5.3 Permanent Casing

AE indicated that a 200 mm diameter permanent casing will be required below the underside of the pier footings due to bending moments induced by eccentric footing loads and that the length of the permanent casing will be 3 m. For design purposes, axial compressive and tensile resistances of the micropiles where the permanent casing is present have been ignored.

3.5.4 Bonded Length

AE indicated that an ultimate limit state (ULS) load of 726 kN per micropile can be used for design under seismic loading conditions. With the consideration of the footing depths of 1.6 m to 1.9 m and the permanent casing length of 3 m, the bonded length of the micropiles will start at a depth of about 5 m below existing ground surface. Hence, we consider the ultimate (unfactored) bond strength of 150 kPa in the firm to very stiff silty clay obtained from the sacrificial micropile testing to be applicable for design of the permanent micropiles.

For a drilled hole diameter of 150 mm and a GRF of 0.8 for seismic design, the minimum required bonded length of a micropile is estimated to be 13 m below the permanent casing.

3.5.5 Testing

All micropiles should be subjected to proof-testing in tension to 100% of the ULS loads. The testing procedures and acceptance criteria should be in accordance with the Post Tensioning Institute manual entitled “Recommendations for Prestressed Rock and Soil Anchors” (document no. PTI DC35.1-14).

3.5.6 Other Design Considerations for Micropiles

Based on the assumed non-liquefiable crust of about 6 m at the pier locations, we do not envisage that the micropiles will be subject to kinematic loading.

3.6 Shallow Foundation Design for Pier and Abutment Footings

3.6.1 Bearing and Sliding Resistances

Shallow foundations are expected for the new piers and abutments. The estimated factored bearing resistances and coefficients of friction for sliding resistances are summarized in Table 3.2 below.

Table 3.2: Estimated Bearing and Sliding Resistances for Shallow Foundations

Location	Piers	Abutments
Subgrade Condition	Native Soils	Bridge End Fill
Factored ULS Bearing Resistance (Non-Seismic)	375	320
Factored ULS Bearing Resistance (Seismic)	525	450
SLS Bearing Resistance	250	240
Factored Coefficient of Friction (Sliding Resistance: Non-Seismic)	0.27	0.48
Factored Coefficient of Friction (Sliding Resistance: Seismic)	0.36	0.6

The factored coefficients of friction for piers in Table 3.2 are provided for force-based design checks. The factored coefficients applied GRF values of 0.6 and 0.8 for non-seismic and seismic conditions, respectively, in accordance with Table 6.2 of CHBDC S6-19 and Clause 6.14.4.1 of the Supplement assuming that the native soils are cohesive.

As discussed in Section 3.1.1, a seismic GRF of 1.0 is allowed in CHBDC, Clause 6.14.4.1 but the Supplement indicates that a seismic GRF of 1.0 may be used if a sensitivity analysis is completed. Using a seismic GRF of 1.0, the unfactored sliding resistance would be 0.45 under seismic loading conditions.

According to AE, the unfactored sliding resistance would be equal to the factored sliding force demand under ULS loading conditions if a coefficient of friction of 0.45 is used. This indicates that footing sliding deformations will be minimal.

3.6.2 Minimum Embedment

In general, a minimum footing depth of 600 mm should be provided for frost protection. According to the available as-built drawings, the embedment depths for the existing footings are estimated to be about 1.9 m, 1.8 m and 1.6 at Piers 1, 2 and 3, respectively. We anticipate that the new pier footings will be constructed to match the existing pier footings.

3.6.3 Load-Deflection for Pier Footings

We have developed an estimated vertical load-deflection curve for structural assessment of the pier footing based on a simplified Plaxis2D analysis. The results are shown in Figure 3.2. The results represent static loading conditions only. Bending moments were not considered in the model.

3.6.4 Linear Compliance Springs

We have estimated the linear compliance springs for the pier footings in general accordance with S6-19 Commentary Section C6.14.5. The results are provided in Figure 3.3. As shown, the compliance springs include upper and lower bound values with the consideration of soil stiffness ranging from 20% to 50% of the peak value. The compliance springs also vary with structural loading. In particular, the compliance springs with a load eccentricity ratio of between 0.17 and 0.4 would be applicable for foundation racking.

It should be noted that the compliance springs provided do not consider the presence of the micropiles.

3.6.5 Non-Linear Compliance Springs for Design of Pier Footings

AE indicated that non-linear compliance soil springs with the consideration of the micropiles are required to aid the structural pushover analysis. Accordingly, Thurber completed a pushover analysis in Plaxis2D to determine the translational and rotational behaviors of a pier footing in conjunction with a micropile. The force-lateral displacement (V-U) and moment-rotation (M-θ) relationships of the footing with micropile, as well as the axial load-vertical displacement relationship for a micropile, are shown in Figures 3.4 to 3.6.

3.7 Estimated Settlement

A settlement analysis has been completed based on the proposed abutments and retaining wall geometry and piers under service loading. The total settlements at the new pier footings and the new abutments are estimated to be about 85 mm and 100 mm, respectively. We estimate that at least 50% of the total settlements will take place in the three to six months following application of the service loads and that the remaining settlements will take place gradually in the next 25 years.

Differential settlements can be estimated as follows:

- 25 mm (i.e. 50% of the total) between new pier footings,
- 50 mm between new abutment and the nearest new pier footings,
- 100% of the total between new and existing pier footings.

3.8 Lateral Earth Pressures and Resistances

3.8.1 Lateral Earth Pressures

Recommended lateral earth pressure coefficients for design of the abutment walls are summarized in Table 3.3. The values provided assume backfill will comprise bridge end fill per MoTI's Standard Specifications Section 202.

Table 3.3: Summary of Lateral Earth Pressure Coefficients for Static and Seismic Conditions

Backfill	Unit Weight	Friction Angle	At-Rest (K_o)	Active (K_a)	Seismic (ΔK_{ae}, 1:475)	Seismic (ΔK_{ae}, 1:2,475)
Bridge End Fill	22 kN/m^3	38°	0.38	0.22	0.15	0.35

The calculated values assume an interface friction coefficient of 0.5 between the wall face and backfill. Seismic lateral pressure on the abutment wall can be estimated using K_{ae}, which is equal to the sum ΔK_{ae} and K_a. The value of ΔK_{ae} assumes full horizontal acceleration applied to the backfill. Further refinement of this value will be conducted during detailed design.

Lateral earth pressures acting on the abutment wall under static loading should assume at-rest conditions. A 12 kPa compaction surcharge should be applied behind the wall varying linearly from ground surface to zero at 2 m below surface per CHBDC S6-19 Section 6.12.3. A live load surcharge of 16 kPa may be assumed per CHBDC S6-19 Section 6.12.5. Live load and compaction surcharge are not addictive.

3.8.2 Passive Resistances

For the existing and future pier footings, an unfactored passive soil resistance can be estimated using a K_p of 3.5 and a soil unit weight of 19 kN/m^3. For ULS design, geotechnical resistance factors provided in Section 3.2 are considered applicable.

To develop a lateral soil spring for the pier footings, the required displacement or rotation to develop the unfactored passive resistance is provided in Table C6.12 in the Commentary to the CHBDC S6-19. An excerpt is shown in Figure 3.7. It should be noted that we have ignored the lateral resistance from the micropiles as the lateral pile resistance is expected to be relatively small. Additional information, if required, can be provided in the next revision of this report.

3.8.3 Abutment Soil Springs

The near-field lateral spring for abutments estimated using Caltrans (2013) is considered applicable given that the abutment height is generally lower than 1.7 m. Additional information can be found in the Commentary to CHBDC S6-19 in Section C6.14.7.

It should be noted that Table 4.4.5.4-1, Item 12 in the Supplement indicates that passive abutment resistance should be based on 70% of the ultimate value as determined in accordance with CHBDC S6-19 Clause 6.14.7.1.

3.9 Existing MSE Walls at Abutments

3.9.1 General

Shop drawings of the existing MSE walls at the abutments were provided to us during the 100% detailed design phase. Thurber completed a global stability analysis for the existing MSE walls. Thurber also engaged RECO to assess the internal stability of the existing MSE walls using the latest seismic hazard values provided in Section 3 above. Details of the assessment completed for the existing MSE walls at the abutments are provided below.

3.9.2 Global Stability

Global stability of the existing MSE walls was checked using the limit equilibrium software Slide2 Version 9, published by Rocscience. The MSE walls were analyzed under pseudo-static conditions. Horizontal seismic coefficients of 50% of the PGAs for A475 and A2475 as outlined in Section 3.3.1 were used in the analysis. The dimensions of the wall and the reinforcement length were estimated from RECO's shop drawings. The reinforced zone of the wall was modelled as a cohesive block. The results are shown in Figures 3.8 to 3.11 in Appendix A and are summarized in Table 3.4 below.

Table 3.4: Summary of Global Stability Analysis for Existing MSE Walls

Abutment	Factor of Safety for Global Stability	
	A475	A2475
North	1.36	1.19
South	1.54	1.36

According to Table 6.2b and Clause 6.14.4.1 in the Supplement, the minimum required factor of safety for global stability (FS) is 1.18 under pseudo-static loading conditions. Based on the above results, this requirement is met for the existing MSE walls at the abutments.

3.9.3 Internal Stability

RECO completed an internal stability assessment of the existing MSE walls at the abutments using the foundation loads provided by AE in conjunction with the seismic hazard values provided in Section 3 above. Based on preliminary discussions with RECO, our understanding the existing structures outlined in Section 3.2 can meet the current seismic performance requirements with remining design life of the wall of about 75 years. Additional information can be found in the draft report by RECO in Appendix F.

3.10 New MSE Walls at Abutments

3.10.1 General

MSE walls should be designed in general accordance with the Supplement followed by CHBDC S6-19. AASHTO LRFD Design Specifications and FHWA-NHI-10-024 and -025 should take lower precedence compared to S6-19.

The new MSE walls will tie in to the existing MSE walls at the abutments. Design and construction of the new MSE walls must consider the presence of the existing retaining walls supporting the existing Mt. Lehman approach embankments.

3.10.2 MSE Wall Type, Minimum Reinforcement Length and Wall Embedment

Consistent with the MSE walls at the existing abutment and common MoTI practice, the MSE walls at the new abutments should comprise a vertical segmental concrete-faced panel wall with inextensible (steel) reinforcement. The reinforcement should be a minimum length of 0.6 times the wall height plus 2 m (0.6H + 2 m) or 2.4 m, whichever is greater. The wall height should be measured from the underside of the levelling pad to the top of the finished road grade.

The recommended minimum embedment depth for walls from adjoining finished grade to the top of the levelling pad is 600 mm for frost protection in accordance with Clause 6.19.3.3 of CHBDC S6-19.

3.10.3 Bearing Resistances

The recommended factored bearing resistances at the base of the MSE walls are 350 kPa and 250 kPa under ULS and SLS conditions, respectively, provided that the wall foundation subgrade comprises well compacted granular fill or native very stiff silty clay. The ULS bearing resistance included a geotechnical resistance factor of 0.5.

3.10.4 Settlement

The MSE wall should be designed to tolerate up to 100 mm of total settlement. The short-term settlement is estimated to be on the order of 50 mm.

3.10.5 Wall Backfill

We recommend the MSE wall backfill comprise Bridge End Fill (BEF) in accordance with SS 202.04 and 202.05 of the 2020 MoTI Standard Specifications. The wall backfill should also meet the electrochemical requirements for the reinforcement to be determined by the wall supplier.

In general, the wall backfill should be placed and compacted following BEF requirements in the 2020 MoTI Standard Specifications. For areas immediately behind the wall face, light, hand operated compaction equipment should be used and the lift thickness should be less than 200 mm. Quality control compaction testing must be explicitly completed in this zone in additional to other fill zones.

3.10.6 Global Stability

The global stability of the new MSE walls was checked using the limit equilibrium method for completeness given that a rigorous dynamic analysis has been completed for the abutments. In this case, the new MSE walls were analysed under static and pseudo-static conditions using the minimum required reinforcement length provided in Section 3.10.2 and groundwater below the bottom of the wall. Under static conditions, we assume that a 16 kPa traffic surcharge is applied on top of the wall and that peak soil strength parameters apply. Under pseudo-static loading, a horizontal seismic coefficient of 50% of the amplified peak ground acceleration, in conjunction with peak soil strength parameters, was used for the analysis.

The results of the limit equilibrium analysis are attached in Figures 3.12 to 3.17 in Appendix A and are summarized in Table 3.5.

Table 3.5: Summary of Global Stability Analysis for New MSE Walls

Abutment	Factor of Safety for Global Stability		
	Static	A475	A2475
North	1.97	1.35	1.21
South	2.14	1.53	1.35

Table 6.2b in the Supplement indicate that the minimum required FS values are 1.54 under static loading conditions and 1.18 under pseudo-static loading conditions with a typical degree of understanding and a typical consequence. The results from our assessment indicate that the requirements are met. Seismic deformations for the abutments based on rigorous dynamic analyses can be found in Table 3-1.

3.10.7 Wall Drainage

Wall sub-drains should comprise a continuous perforated 150 mm PVC pipe immediately behind the facing and at the rear of the reinforced zone. The PVC pipe should be encased in 150 mm of clear crush and wrapped in a non-woven geotextile filter fabric with properties in Table 3.6.

Front and rear drainage should be connected with 150 mm solid PVC pipe at regular intervals. The drainage system should drain positively away from the backfill zone (typically at 2% grade) and should be connected to the nearby storm sewer system.

Table 3.6: Properties for Non-Woven Geotextile

Property	Test Method	Unit	Value
Grab Tensile Strength	**ASTM-D4632**	**N**	**712**
Grab Elongation	**ASTM-D4632**	**%**	**50 – 105**
Tear Resistance	**ASTM-D4533**	**N**	**267**
Puncture CBR	**ASTM-D6241**	**N**	**1820**
Permeability	**ASTM-D4491**	**sec^{-1}**	**1.50**
Water Flow	**ASTM-D4491**	**$l/min/m^2$**	**4.480**

Property	Test Method	Unit	Value
Apparent Opening Size (A.O.S.)	**ASTM-D4751**	**mm**	**0.212**
U.V. Resistance	**ASTM-D4355**	**% @ 500 h**	**70**

3.10.8 Proprietary Wall Design Parameters

We expect that internal and external wall design will be completed by a proprietary wall supplier/designer. Global and compound wall stability should be checked by Thurber after the proprietary wall design has been completed. Shop drawings and design reports of the walls should be provided to Thurber for review.

We recommend the following parameters be used for design of MSE walls:

- Reinforced Fill (Bridge End Fill)
  - Effective angle of friction: 35° (Maximum per CHBDC S6-19)
  - Unit weight: 22 kN/m^3
- Retained Fill
  - Effective angle of friction: 30°
  - Unit weight: 19.5 kN/m^3
- Foundation Soil (for sliding)
  - Effective angle of friction: 30°
- Seismic
  - PGA (A475) = 0.22g
  - PGA (A2475) = 0.4g
- Bearing Resistances
  - ULS: 350 kPa
  - SLS: 250 kPa
- Settlement for design
  - Total – 100 mm
  - Short term – 50 mm
- Surcharge
  - Abutment footing: See Table 3.2 and AE’s drawings
  - Traffic surcharge: 16 kPa
  - Sloping backslope: See AE’s drawings

3.11 Construction Considerations

3.11.1 Site Preparation

Abutments

The proposed abutment locations are occupied by existing slopes adjacent to the existing abutments. The existing slopes are covered with vegetation and trees. As part of the site preparation, the vegetation and tress should be removed and excavation into the existing slopes will be required to facilitate construction of the new MSE walls, including the inextensible reinforcement and wall drainage.

MSE walls supporting the existing approach embankments are present. The existing walls may interfere with construction of the new MSE walls. If they are to remain, temporary shoring will likely be required to underpin the existing MSE walls and facilitate construction of the new walls.

Piers

Pier 1 has been integrated into an existing barrier separating the current eastbound lanes. Site preparation at this location will generally include removal of existing asphalt and concrete barriers, as well as unsuitable soil where present.

Piers 2 and 3 are located within the existing highway median. A soil berm is present along the median. Beyond the footprint of the existing underpass, low vegetation is also present. Site preparation at Piers 2 and 3 will include removal of the soil berm, low vegetation and existing asphalt and concrete barriers, as well unsuitable soil, where present.

Potential Conflicts with Old Foundations

The new underpass will extend to the east of the existing structure. Figure 1.7 shows the approximate locations of the original bridge foundations. It is uncertain if the old foundations have been completely or partially removed.

Figure 1.7 suggests that the new MSE wall footprint at the south abutment and the new footing at Pier 1 may be partially within the old foundation footprints. If the old foundations have not been completely removed, then it may be necessary to remove the old foundations partially or completely at these locations to facilitate construction of the new wall foundation and pier footing. The contractor should be made aware of the potential for these obstructions.

3.11.2 Shallow Foundation Subgrade Preparation

If the soil conditions at the foundation subgrade level for the MSE walls and pier footings comprise granular fill, the exposed subgrade should be compacted to a dense and unyielding condition. Light compaction equipment should be used in the proximity of existing structures to avoid potential adverse effects on the existing foundations due to the compaction operations. Any soft, wet or unsuitable materials encountered at the bearing surface should be subexcavated and replaced with compacted granular fill.

If native silty clay is encountered at the exposed subgrade, no compaction will be required. Further, a smooth-edge cleanout bucket must be used to prepare the subgrade. It should be noted that the silty clay will be sensitive to changes in moisture content and susceptible to disturbance, especially in freezing or wet weather conditions. Accordingly, foot and equipment traffic on the native silty clay should be limited unless it is covered by a skim coat of lean-mix concrete concreate or a nominal thickness (± 50 mm) of road base compacted to 100% standard proctor maximum dry density using a light compaction equipment.

In general, we assume that the depths of new MSE foundations and pier footings will match the existing ones. Caution should be applied not to undermine the existing foundations and footings during foundation subgrade preparation.

As mentioned above, the new MSE wall footprint at the south abutment and the new footing at Pier 1 may be partially within the old foundation footprints. If the old footings were previously removed, then fill material should be anticipated in these areas. If the fill is loose, subexcavating the existing fill and replacing it with well compacted road base or approved equivalent will be required.

3.11.3 Temporary Excavation

For planning purposes, temporary excavation using conventional cut slopes, trench boxes or a combination of both will likely be required to facilitate construction of the new pier footings. For the new MSE wall foundations, conventional cut slopes in conjunction with temporary shoring such as shotcrete and soil anchors will likely be required. The design of temporary excavation is the responsibility of the contractor. If temporary shoring such as trench boxes, shotcrete and soil anchors, or similar is required, the contractor should provide a work plan and supporting design documents for review and approval. Regardless, the contractor must maintain integrity and stability of the existing and new structures during construction.

3.11.4 Temporary Dewatering

The design of temporary dewatering is the responsibility of the contractor. In general, the depths of excavation appear to be above the groundwater depth observed in the recent geotechnical investigation. However, groundwater levels are expected to vary seasonally with infiltration and surface drainage conditions. For planning purposes, we envisage that groundwater, if encountered in foundation excavations, can be managed by conventional sumps and pumps.

3.11.5 Potential Impacts to Existing Utilities and Infrastructure

Temporary excavations, fill placement or compaction operations will induce vibrations or settlements that could affect existing structures in the vicinity of these operations. Hence, we recommend that a settlement and vibration monitoring program be developed to confirm that there are no negative effects on key existing structures. The contractor should engage a qualified professional engineer to develop the monitoring program prior to construction and execute it during construction. The monitoring program should be submitted for review prior to construction.

AE should identify monitoring locations and establish tolerances for existing infrastructure where applicable.

3.11.6 Monitoring Requirements for New Structure

To help determine the waiting period prior to engaging the micropiles to the pier footings, we recommend a monitoring program be established for the new pier footings. At least one monitoring point should be installed near the bottom of the new pier column. The monitoring points should be above grade for ease of survey. Two sets of baseline readings should be taken shortly after the pier column construction. One set of readings should be taken immediately after construction of the superstructure, followed by weekly readings for at least four weeks. The results should be sent to the design team for review within 24 hours after the readings are taken. Survey data should include vertical displacements to an accuracy of ± 2 mm.

4. SIGNATURES/CLOSURE

This report was issued before any final design or construction details had been prepared or issued. Therefore, differences may exist between the report recommendations and the final design, the contract documents, or conditions during construction. In such instances, Thurber Engineering Ltd. should be contacted immediately to address these differences. Designers and contractors undertaking or bidding the work should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for design and construction, and make their own interpretation of the data as it may affect their proposed scope of work, cost, schedules, safety, and equipment capabilities.

We trust this information meets your present needs. If you have any questions, please contact the undersigned at your convenience.

PROFESSIONAL PROVINCE OF C. K. L. NG # 31250 BRITISH COLUMBIA ENGINEER

Charles Ng, M.Eng., P. Eng.	Denny Ma, M.Eng., P. Eng.
Senior Associate, Project Engineer	Associate, Review Engineer

Thurber Engineering Ltd.
Permit to Practice #1001319

Date: ***February 23, 2024***
File: ***32079***

Attachment

- Statement of Limitations and Conditions

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client, the BC Ministry of Transportation and Infrastructure (MoTI) and Authorized Users as defined in the MoTI Special Conditions Form H0461d. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT THURBER'S WRITTEN CONSENT AND SUCH USE SHALL BE ON SUCH TERMS AND CONDITIONS AS THURBER MAY EXPRESSLY APPROVE. Any use which an unauthorized third party makes of the Report, is the sole responsibility of such third party. Thurber accepts no responsibility whatsoever for damages suffered by any unauthorized third party resulting from use of the Report without Thurber's express written permission.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.

d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

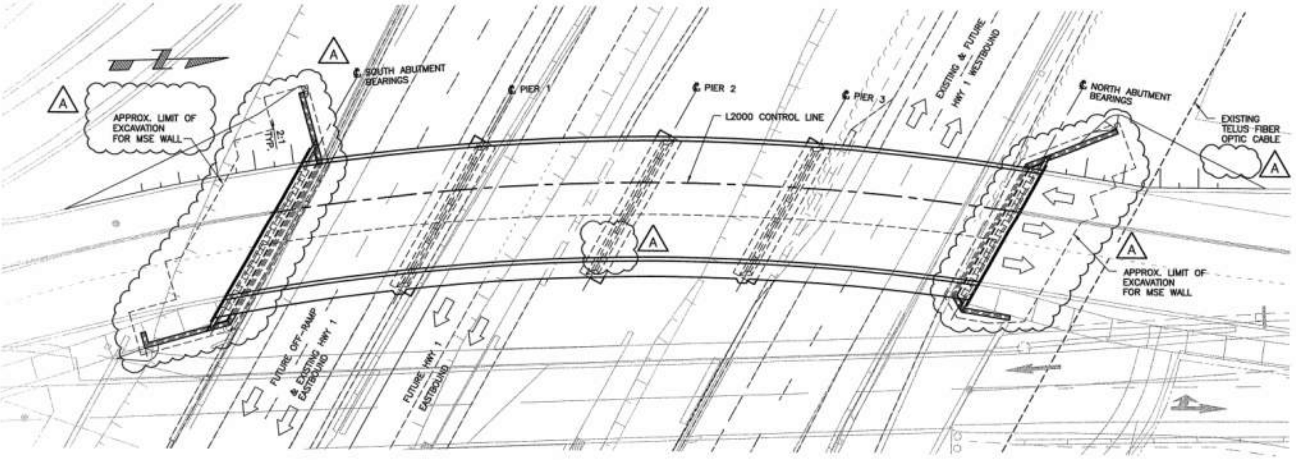
6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

7. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes but is not limited to decisions made to develop, purchase or sell land.

APPENDIX A FIGURES



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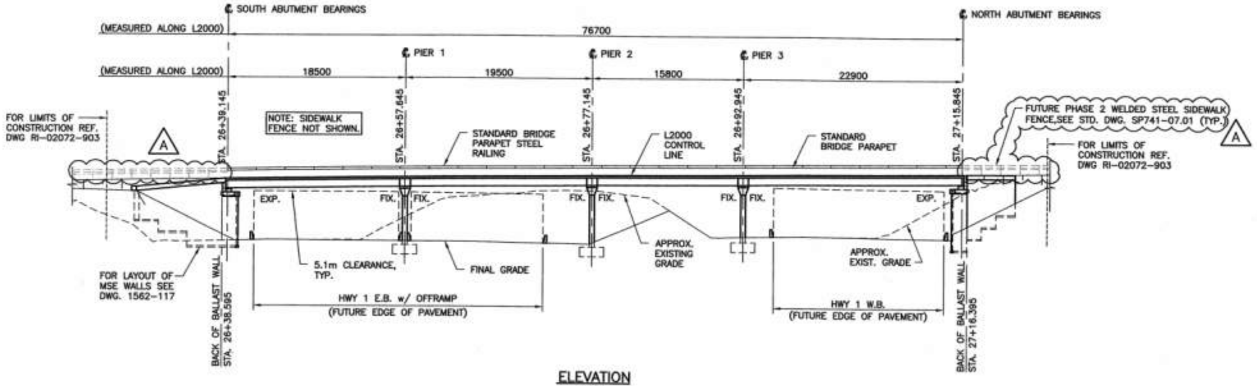

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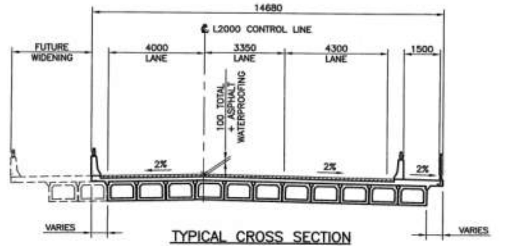

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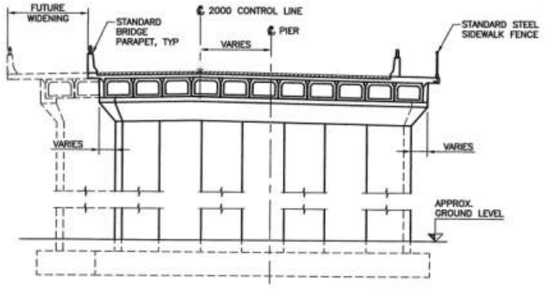


TYPICAL CROSS SECTION AT PIERS
1:100

GENERAL NOTES:

1. DESIGN SPECIFICATION: CSA STANDARD CAN/CSA-S6-00.
2. SEISMIC DESIGN SPECIFICATIONS:
   BRIDGE:
   CSA STANDARD CAN/CSA-S6-00
   MSE WALLS:
   PER AASHTO STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 16th ED., 1996.
3. DESIGN LOADS:
   (a) LIVE LOAD: CS-625
   (b) WIND LOAD:
   10 YEAR RETURN: 0.415 kPa
   30 YEAR RETURN: 0.550 kPa
   100 YEAR RETURN: 0.710 kPa
   (c) SEISMIC:
   ACCELERATION COEFICIENT A: 0.20
   (10% PROBABILITY OF EXCEEDANCE IN 50 YEARS)
   SOIL AMPLIFICATION A: 1.2
   SEISMIC PERFORMANCE CATEGORY D
4. CLIMATIC:
   (a) MAXIMUM DAILY MEAN TEMPERATURE: +28° C
   (b) MINIMUM DAILY MEAN TEMPERATURE: -14° C
   (c) DESIGN RAINFALL: 10mm IN 15 MINUTES
5. DIMENSIONS AND ELEVATIONS SHOWN ON DRAWINGS ARE FOR A REFERENCE TEMPERATURE OF 15° C.

I. L. WALTERS

Rev	Date	Description	Initial
J			
I			
H			
G			
F			
E			
D			
C			
B			
A	07/08/22	RECORD DRAWINGS	ILW
		REVISIONS	

SCALE
AS SHOWN

DESIGNED GAN DATE 04/09/30
CHECKED ILW DATE 04/09/30
DRAWN CTL DATE 04/09/30

Date	ISSUE RECORD	Initial
04/12/14	ISSUED FOR CONSTRUCTION	ILW
04/11/03	ISSUED FOR TENDER	ILW
04/09/30	100% ISSUE FOR REVIEW	ILW
04/08/26	REVISED 90% ISSUE	ILW
04/07/20	REVISED 50% ISSUE	ILW
04/06/22	50% ISSUE FOR REVIEW	ILW

City of Abbotsford

MINISTRY OF TRANSPORTATION
SOUTH COAST REGION

1562 MT. LEHMAN UNDERPASS
GENERAL ARRANGEMENT
601897

FILE No.
BUCKLAND & TAYLOR LTD. Bridge Engineering
REGION 1
DRAWING No. 1562-102 A

LEGEND / NOTES

T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
As-Built Drawing 1562-102

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
-

DESIGNED BY
-

APPROVED BY
-

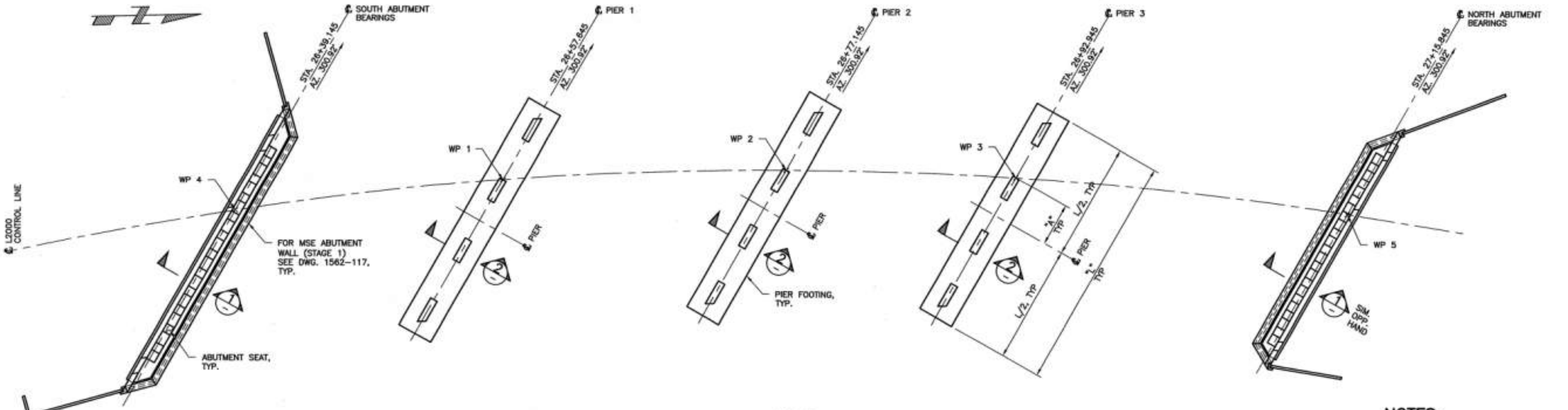
DATE
2023-12-12

SCALE
-

PROJECT No.
32079

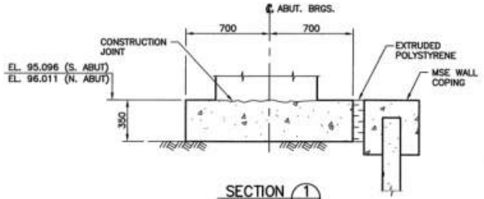
DRAWING / FIGURE No.
1.1

REV.
0

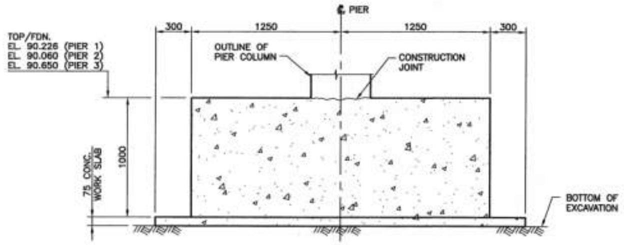


PLAN

1:150



SECTION ①
SCALE 1:20



SECTION ②
SCALE 1:20

PIER	"A"	"L"
1	3266	18000
2	3092	17100
3	2979	16300

WP	STATION	COORDINATES	
		NORTHING	EASTING
1	26+57.645	35901.366	45487.014
2	26+77.145	35920.859	45486.727
3	26+92.945	35936.632	45487.609
4	26+39.145	35882.947	45488.690
5	27+15.845	35959.321	45490.650

NOTES:

1. FOR GENERAL NOTES SEE DRAWING 1562-102.

SUBSTRUCTURE NOTES:

1. CONCRETE SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS AS FOLLOWS:

   SUBSTRUCTURE CONCRETE: 30 MPa
   WORK SLAB: 20 MPa

2. ALL EXPOSED EDGES OF CONCRETE SHALL BE CHAMFERED 20mm UNLESS NOTED OTHERWISE.
3. FOOTINGS AND WORK SLABS SHALL BE AT DEPTH SHOWN OR TO LOWER ELEVATIONS AS MAY BE ORDERED BY THE OWNER'S REPRESENTATIVE.
4. ALL REINFORCING STEEL SHALL CONFORM TO CAN/CSA G30.18-M GRADE 400R. MAXIMUM YIELD STRENGTH 525 MPa.
5. CLEAR COVER TO REINFORCING STEEL:

   CAST AGAINST SOIL: 100mm
   ELSEWHERE: 70mm

6. BARS DENOTED "ME" SHALL BE EPOXY COATED.
7. ALL LAPS OF REINFORCING BARS FOR SPLICES SHALL BE AS FOLLOWS UNLESS NOTED OTHERWISE:

BAR SIZE	UNCOATED	EPOXY COATED (B*)	EPOXY COATED (T**)
10M	400	500	600
15M	500	700	850
20M	700	800	1050
25M	1100	1300	1700
30M	1300	1550	2000

(B*) DENOTES VERTICAL BARS WITH LESS THAN 300mm CONCRETE BELOW BARS

(T**) DENOTES HORIZONTAL BARS WITH MORE THAN 300mm OF CONCRETE BELOW BARS

L.L. WALTERS

Rev	Date	Description	Initial
J			
I			
H			
G			
F			
E			
D			
C			
B			
A	07/08/22	RECORD DRAWINGS	ILW

REVISIONS

SCALE
AS SHOWN

DESIGNED GAH DATE 04/09/30
CHECKED ILW DATE 04/09/30
DRAWN CTL DATE 04/09/30

Date	ISSUE RECORD	Initial
04/12/14	ISSUED FOR CONSTRUCTION	ILW
04/11/03	ISSUED FOR TENDER	ILW
04/09/30	100% ISSUE FOR REVIEW	ILW
04/08/26	90% ISSUE FOR REVIEW	ILW
04/06/22	50% ISSUE FOR REVIEW	ILW

City of Abbotsford

BRITISH COLUMBIA
MINISTRY OF TRANSPORTATION
SOUTH COAST REGION

1562 MT. LEHMAN UNDERPASS
FOUNDATION LAYOUT

601898

FILE No.
BUCKLAND & TAYLOR Ltd. Bridge Engineering
REGION 1
DRAWING No. 1562-103
A

LEGEND / NOTES

T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
As-Built Drawing 1562-103

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
-

DATE
2023-12-12

DESIGNED BY
-

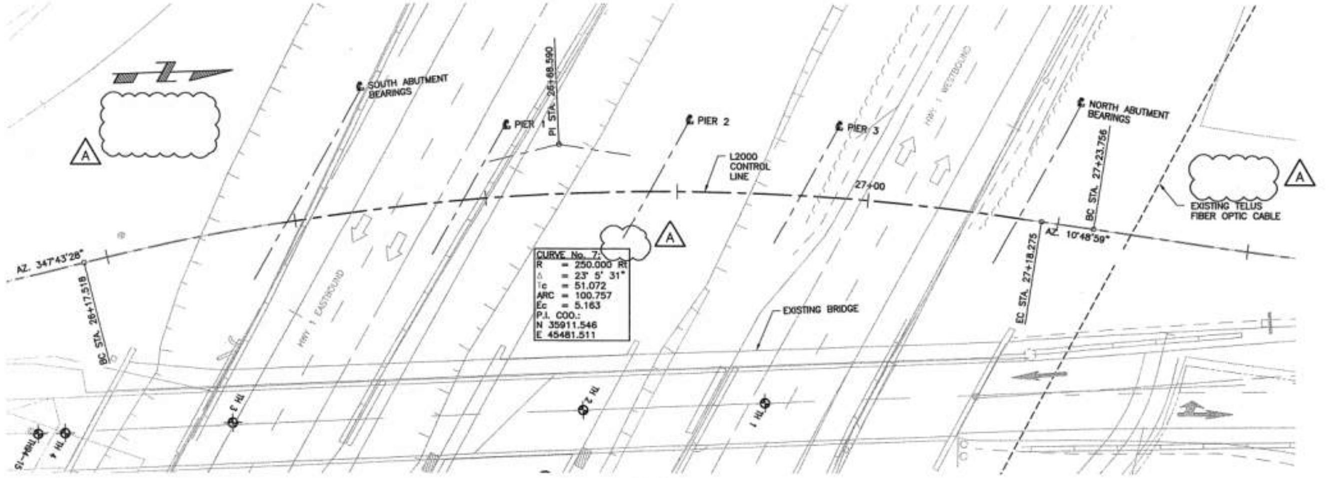
SCALE
-

APPROVED BY
-

PROJECT No.
32079

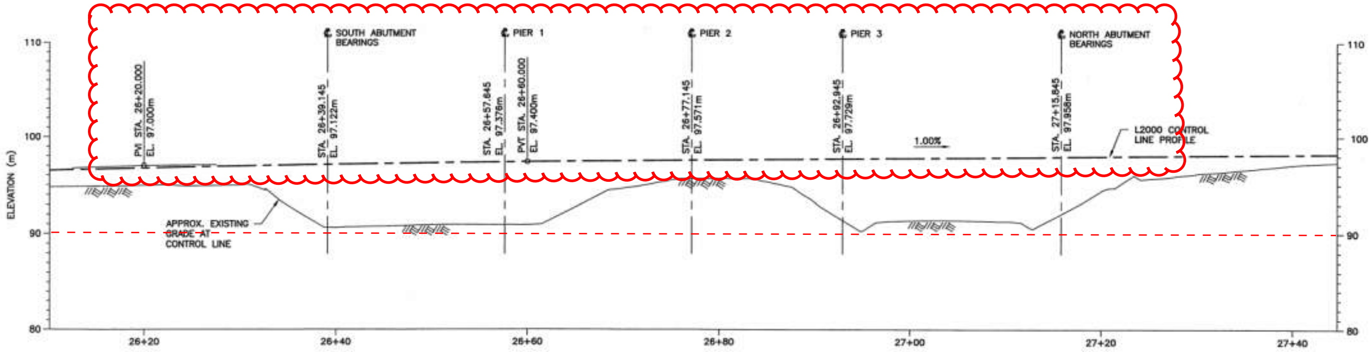
DRAWING / FIGURE No.
1.2

REV.
0

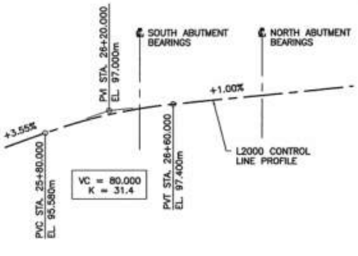


CURVE No. 7:
R = 250.000 Rt
Δ = 23° 5' 31"
Tc = 51.072
ARC = 100.757
Ec = 5.163
P.I. COO.:
N 35911.546
E 45481.511

PLAN
1:250



PROFILE ALONG L2000 CONTROL LINE
1:250



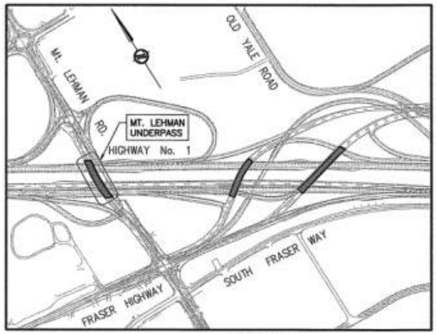
L2000 VERTICAL ALIGNMENT
NTS

NOTES:

1. FOR GENERAL NOTES SEE DRAWING 1562–102.
2. SURVEY BY: BC MINISTRY OF HIGHWAYS
3. DATUM: GEODETIC.
4. ALL STATIONS AND ELEVATIONS IN METRES.

LEGEND:

TH	TESTHOLE SEE MOT TESTHOLE LOGS (REF: FACTUAL GEOTECHNICAL REPORT, SEPT. 5, 2000, APPENDIX C)
TP	TEST PIT SEE TEST PIT LOGS BY TROW ASSOCIATES INK.



KEY PLAN
N.T.S.

DRAWING LIST:

DRAWING No.	TITLE
1562–101A	SITE PLAN
1562–102A	GENERAL ARRANGEMENT
1562–103A	FOUNDATION LAYOUT
1562–104A	FOUNDATION REINFORCEMENT
1562–105A	SOUTH ABUTMENT
1562–106A	NORTH ABUTMENT
1562–107A	ABUTMENT – REINFORCEMENT
1562–108A	PIERS
1562–109A	PIER REINFORCEMENT
1562–110A	PRECAST STRINGER LAYOUT
1562–111A	CONCRETE BOX STRINGERS MK1
1562–112A	CONCRETE BOX STRINGERS MK2
1562–113A	CONCRETE BOX STRINGERS MK3
1562–114A	CONCRETE BOX STRINGERS MK4
1562–115A	DECK
1562–116A	DECK REINFORCEMENT
1562–117B	MSE WALLS

REFERENCE DRAWINGS:

DRAWING No.	REV.	TITLE
2784–1	Q	STANDARD BRIDGE PARAPET – 810mm HIGH
SP941–03.04.01, 02		STANDARD BRIDGE PARAPET – 810mm HIGH TRANSITION
2785–2	B	STANDARD BRIDGE PARAPET STEEL RAILING (1995)
2891–1	L	STANDARD STEEL SIDEWALK FENCE
SP741–07.01		SIDEWALK FENCE – WELDED OR SLIP–ON
SP941–02.01.05–07		PRECAST CONCRETE PIER BARRIER – 810mm CPB

I. L. WALTERS

Rev	Date	Description	Initial
J			
I			
H			
G			
F			
E			
D			
C			
B			
A	07/08/22	RECORD DRAWINGS	ILW
REVISIONS			

SCALE
AS SHOWN

DESIGNED GAW DATE 04/09/30
CHECKED ILW DATE 04/09/30
DRAWN CTL DATE 04/09/30

Date	ISSUE RECORD	Initial
04/12/14	ISSUED FOR CONSTRUCTION	ILW
04/11/03	ISSUED FOR TENDER	ILW
04/09/30	100% ISSUE FOR REVIEW	ILW
04/08/26	90% ISSUE FOR REVIEW	ILW
04/06/22	50% ISSUE FOR REVIEW	ILW

City of Abbotsford

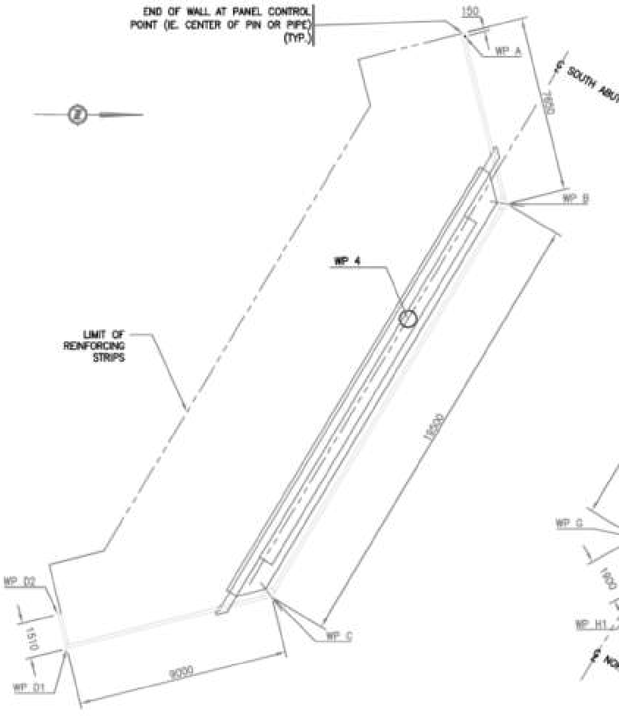
MINISTRY OF TRANSPORTATION
SOUTH COAST REGION

1562 MT. LEHMAN UNDERPASS
SITE PLAN

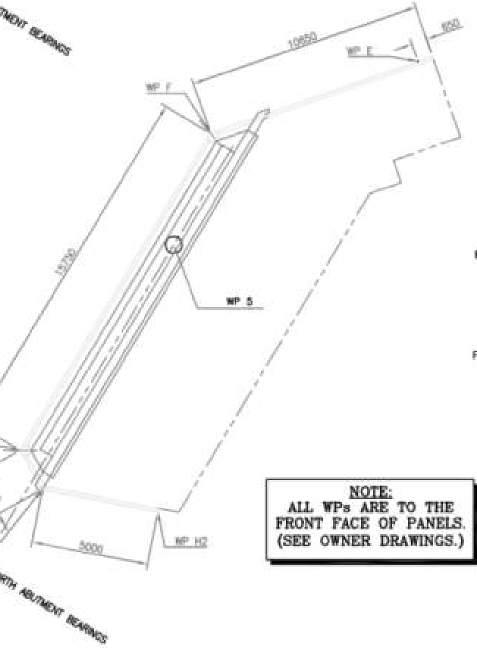
601896

FILE No.		REGION	DRAWING No.
	BUCKLAND & TAYLOR LTD. Bridge Engineering	1	1562–101 A

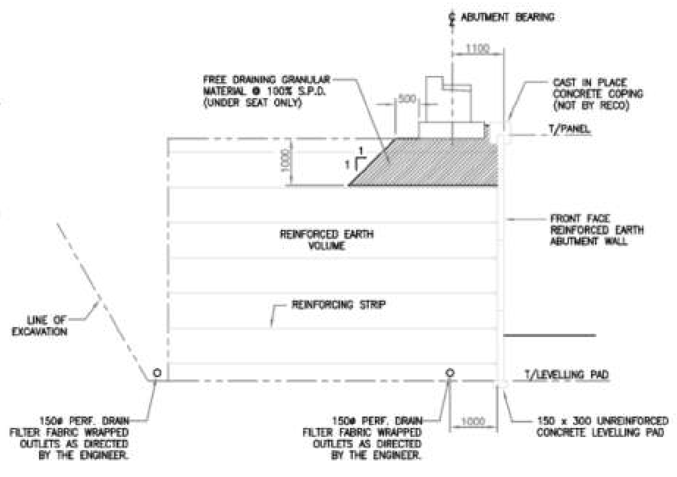
LEGEND / NOTES		
CLIENT NAME BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE	DRAWN BY -	DATE 2023-12-12
DRAWING TITLE As-Built Drawing 1562-101	DESIGNED BY -	SCALE -
PROJECT NAME AND LOCATION HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2) ABBOTSFORD, BC	APPROVED BY -	PROJECT No. 32079
	DRAWING / FIGURE No. 1.3	REV. 0



PLAN – SOUTH ABUTMENT
1:100



PLAN – NORTH ABUTMENT
1:100

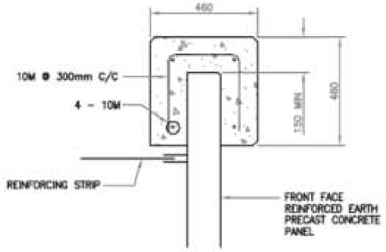


TYPICAL SECTION
1:50

NOTE:
ALL WPs ARE TO THE FRONT FACE OF PANELS. (SEE OWNER DRAWINGS.)

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES; ELEVATIONS ARE IN METRES.
2. ITEMS SUPPLIED BY 'REINFORCED EARTH COMPANY LTD.':
   i) PRECAST CONCRETE PANELS AND ALL EMBEDDED HARDWARE.
   ii) GALVANIZED REINFORCING STRIPS AND BOLTS.
   iii) FILTER FABRIC AND RUBBER PADS.
   iv) PRECAST CORNER ELEMENTS.
3. DESIGN PARAMETERS:
   i) DESIGN CODE: AASHTO 96
   ii) SURCHARGE: 17.6 kPa
   iii) BACKFILL: PHI(re) = 38°, GAMMA(re)= 21 kN/m^3
   PHI(random) = 30°, GAMMA(random)= 20 kN/m^3
   PHI(foundation) = 30°.
   iv) DESIGN LOADS: (UNFACTORED AT POINT OF GIRDER BEARING)
   VERTICAL DEAD LOAD: 187 kN/m
   VERTICAL LIVE LOAD: 97 kN/m
   HORIZONTAL BRAKING: 9 kN/m
   TRANSVERSE SEISMIC: 27 kN/m
   LONGITUDINAL SEISMIC: 69 kN/m
   v) SERVICE LIFE: 100 YEARS (e_s = 2.0)
   vi) SEISMIC: a/g = 0.2
4. THE APPLIED BEARING PRESSURE UNDER THE R/E VOLUME IS SHOWN ON THE ELEVATIONS. THE SUITABILITY OF THE FOUNDATION TO CARRY THIS LOAD AND THE OVERALL GLOBAL STABILITY SHALL BE REVIEWED BY THE GEOTECHNICAL ENGINEER.
5. ARCHITECHTURAL FINISH: GRAVE



CAST IN PLACE COPING DETAIL
1:50

JOINTS IN COPING ARE TO LINE UP WITH PANEL JOINTS. FILTER FABRIC TO BE CONTINUED OVER TOP OF PANEL AND DOWN FRONT FACE AT ALL CONSTRUCTION JOINTS IN COPING. CONSTRUCTION JOINT SPACING TO BE 3000mm

No.	Date	Issue or Revision
4	23DEC05	AS-BUILT
3	2FEB05	ISSUED FOR CONSTRUCTION
2	25JAN05	RE-ISSUED FOR APPROVAL
1	24DEC04	ISSUED FOR APPROVAL
0	16NOV04	ISSUED FOR TENDER

Reinforced Earth Company Ltd.

Project: MT. LEHMAN UNDERPASS

Title: REINFORCED EARTH ABUTMENT WALLS
PLAN, SECTION AND DETAILS

Drawn: DC
Date: NOV'04
Scale: AS SHOWN
Project No. 2872
Drawing No. 2872-1
Rev. 4

LEGEND / NOTES

T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Mt Lehman Underpass - Existing MSE Wall Plan, Section and Details

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
-

DESIGNED BY
-

APPROVED BY
-

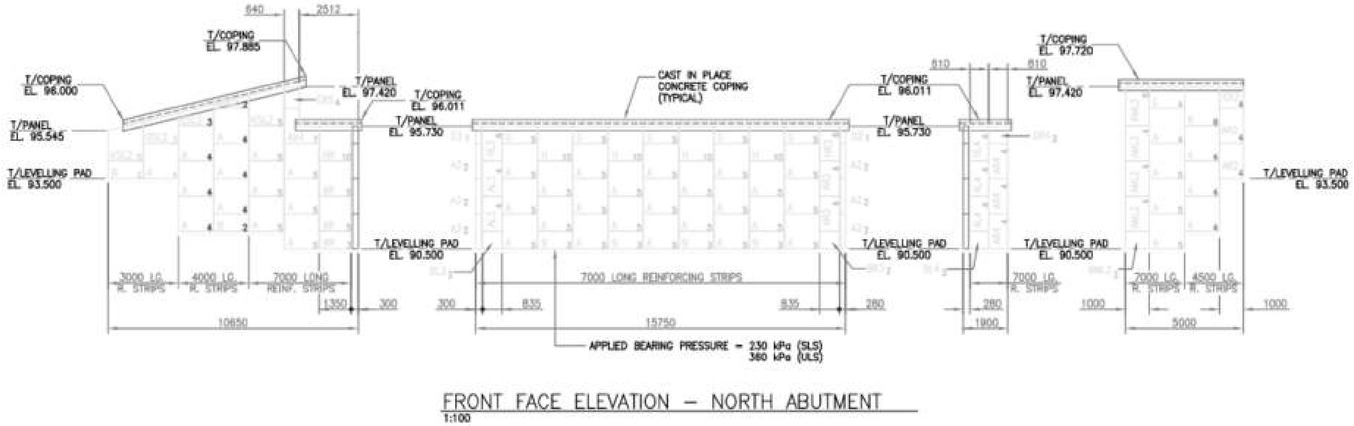
DATE
2023-12-12

SCALE
N/A

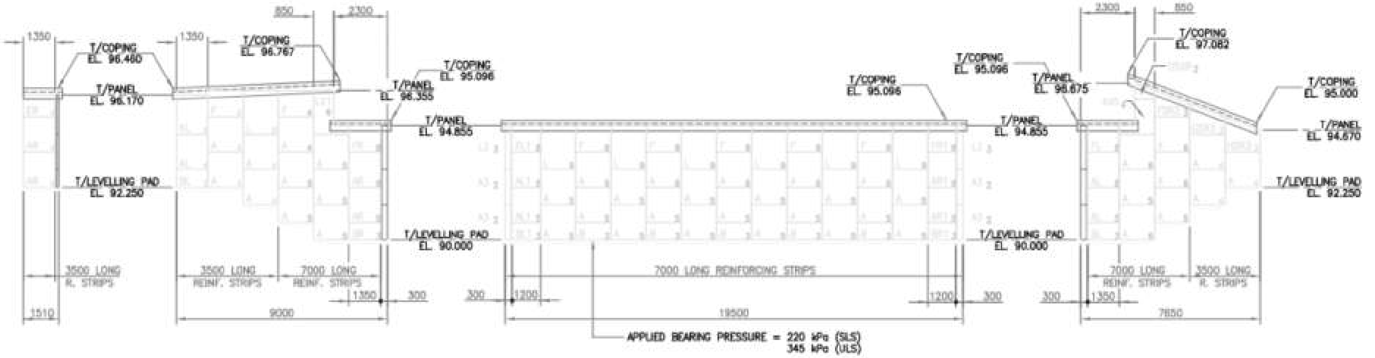
PROJECT No.
32079

DRAWING / FIGURE No.
1.4

REV.
0



FRONT FACE ELEVATION — NORTH ABUTMENT
1:100



FRONT FACE ELEVATION — SOUTH ABUTMENT
1:100

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No.	Date	Issue or Revision
4	23DEC05	AS-BUILT
3	2FEB05	ISSUED FOR CONSTRUCTION
2	25JAN05	RE-ISSUED FOR APPROVAL
1	24DEC04	ISSUED FOR APPROVAL
0	16NOV04	ISSUED FOR TENDER

Reinforced Earth Company Ltd.
1550 Enterprise Road, Suite 229
Mississauga, Ontario
L4W 4P4
Tel: (905) 564-0896
Fax: (905) 564-2609

Project: MT. LEHMAN UNDERPASS

Title: REINFORCED EARTH ABUTMENT WALLS
FRONT FACE ELEVATIONS

Drawn:	DC	Checked:		Approved:		Scale:	1:100	Project No:	2872
Date:	NOV'04	Date:		Date:		Drawing No:	2872-2	Rev:	4

LEGEND / NOTES

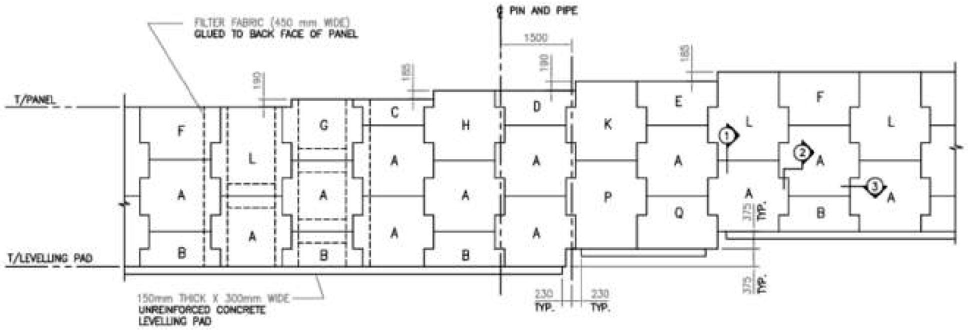
T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Mt Lehman Underpass - Existing MSE Wall Front Face Elevations

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

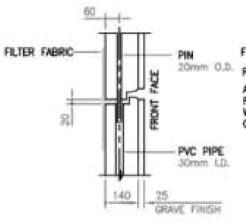
DRAWN BY: -
DESIGNED BY: -
APPROVED BY: -
DATE: 2023-12-12
SCALE: N/A
PROJECT No.: 32079
DRAWING / FIGURE No.: 1.5
REV.: 0



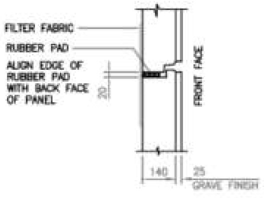
PANEL TYPE	A	B	C	D	E	F	G	H	K	L	P	Q
EFFECTIVE HEIGHT (mm)	1500	750	545	730	920	1105	1295	1480	1670	1855	1875	1125

TYPICAL FRONT FACE ELEVATION

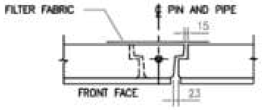
NOT TO SCALE



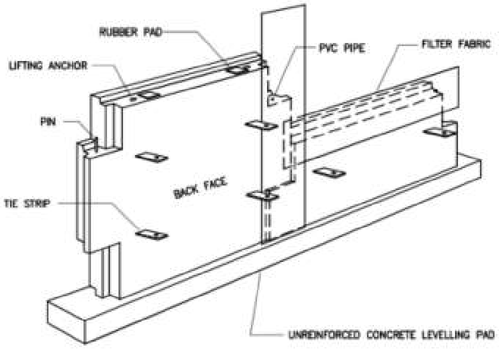
SECTION 2



SECTION 1

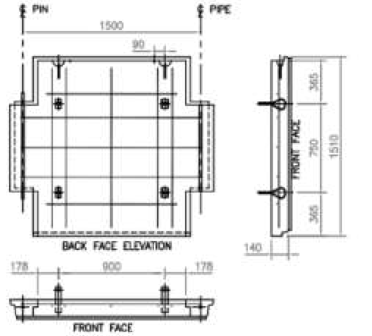


SECTION 3



TYPICAL GENERAL ARRANGEMENT

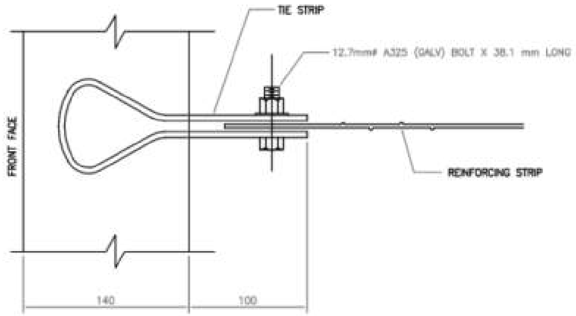
NOT TO SCALE



NOTE:
1. CONCRETE COMPRESSIVE STRENGTH (f'c) SHALL BE 30 MPa AT 28 DAYS.
2. COVER TO REINFORCEMENT SHALL BE 40mm ±10 MIN.
3. ALL REINFORCING BARS SHOWN ARE 10M, fy = 400 MPa, CONFORMING TO CSA G30.18-M92
4. CLEARANCE BETWEEN REINFORCING BARS AND GALVANIZED TIE STRIPS SHALL BE 30mm MIN.
5. ARCHITECHTURAL FINISH: GRAVE

TYPICAL PANEL

NOT TO SCALE



TIE STRIP/REINFORCING STRIP CONNECTION DETAIL

NOT TO SCALE

No.	Date	Issue or Revision
4	23DEC05	AS-BUILT
3	2FEB05	ISSUED FOR CONSTRUCTION
2	25JAN05	RE-ISSUED FOR APPROVAL
1	24DEC04	ISSUED FOR APPROVAL
0	16NOV04	ISSUED FOR TENDER

Reinforced Earth Company Ltd.
1550 Enterprise Road, Suite 324
Mississauga, Ontario
L4W 4P4
Tel: (905) 564-0896
Fax: (905) 564-2609

Project: MT. LEHMAN UNDERPASS

Title: REINFORCED EARTH ABUTMENT WALLS
TYPICAL DETAILS

Drawn: DC
Checked:
Approved:
Scale: AS SHOWN
Project No: 2872
Date: NOV'04
Date:
Date:
Drawing No: 2872-3
Rev: 4

LEGEND / NOTES

T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Mt Lehman Underpass - Existing MSE Wall Typical Details

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
-

DESIGNED BY
-

APPROVED BY
-

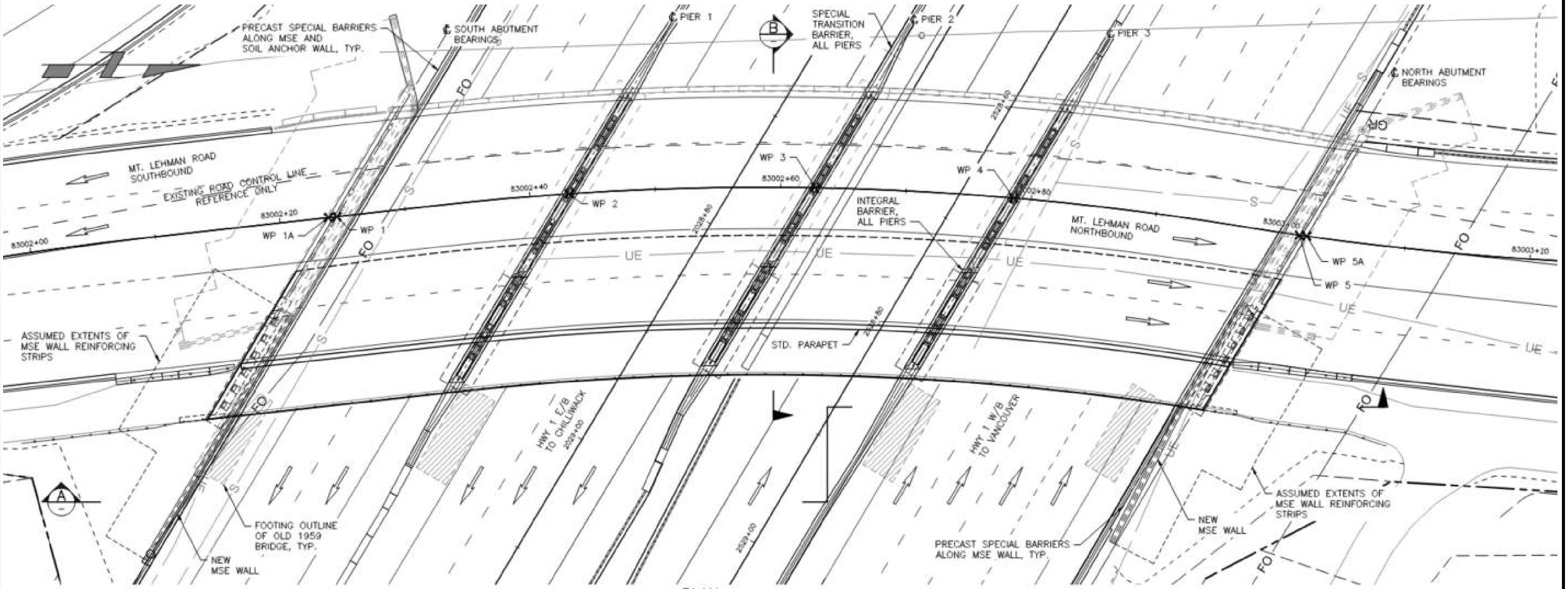
DATE
2023-12-12

SCALE
N/A

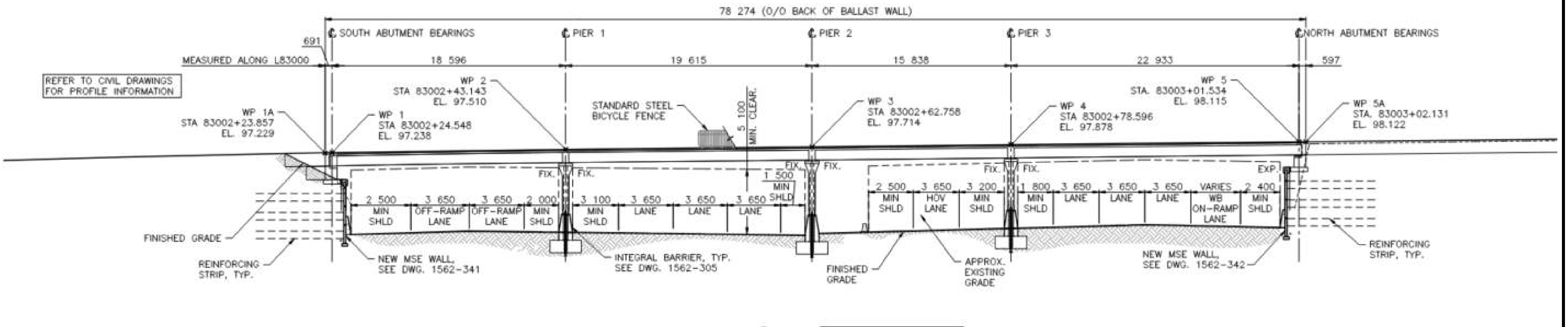
PROJECT No.
32079

DRAWING / FIGURE No.
1.6

REV.
0



PLAN
SCALE 1:200



SECTION A
SCALE 1:200

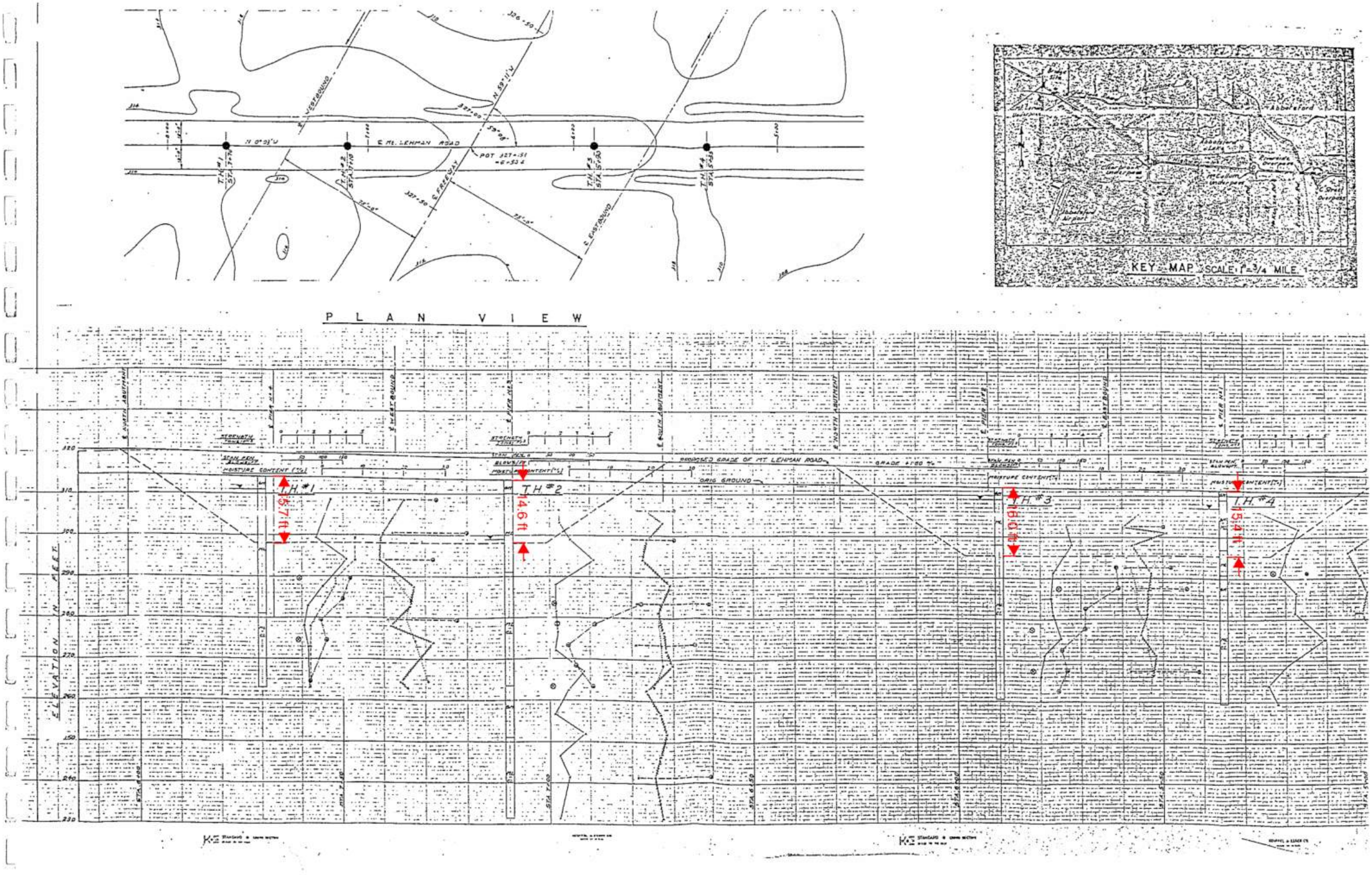
NOTE: ALL HWY 1 LANING DIMENSIONS ARE SQ.

LEGEND / NOTES

- Originated from AE's Dwg. 1562-303-B

T

CLIENT NAME BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE	DRAWN BY -	DATE 2023-12-12	
DRAWING TITLE Mt Lehman Underpass General Arrangement	DESIGNED BY -	SCALE -	
PROJECT NAME AND LOCATION HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2) ABBOTSFORD, BC	APPROVED BY -	PROJECT No. 32079	
	DRAWING / FIGURE No. 1.7		REV. 0



LEGEND / NOTES

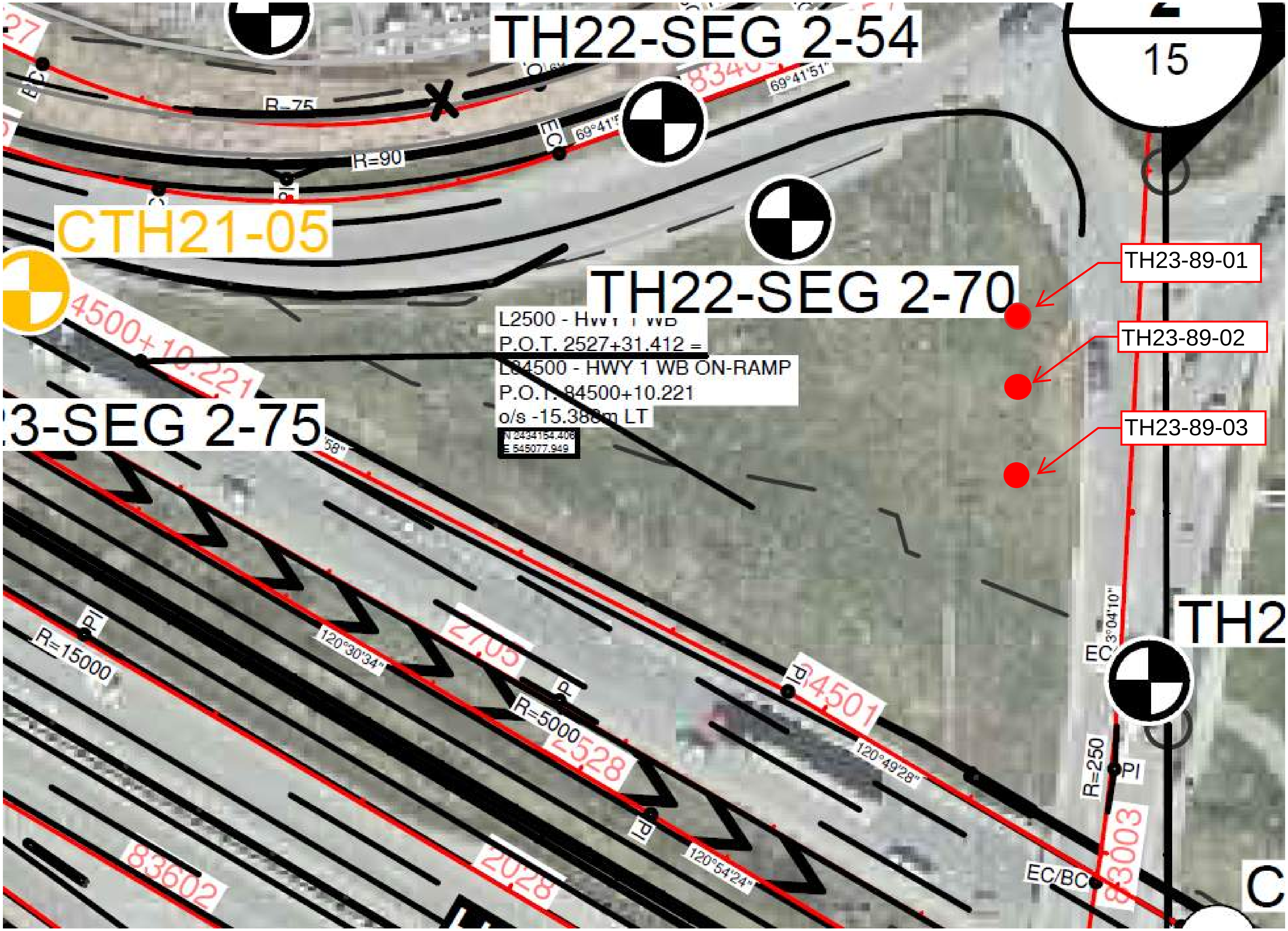
T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
1960's Mt Lehman Bridge Plan and Section

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY: -
DATE: 2023-12-12
DESIGNED BY: -
SCALE: -
APPROVED BY: -
PROJECT No.: 32079
DRAWING / FIGURE No.: 2.1
REV.: 0



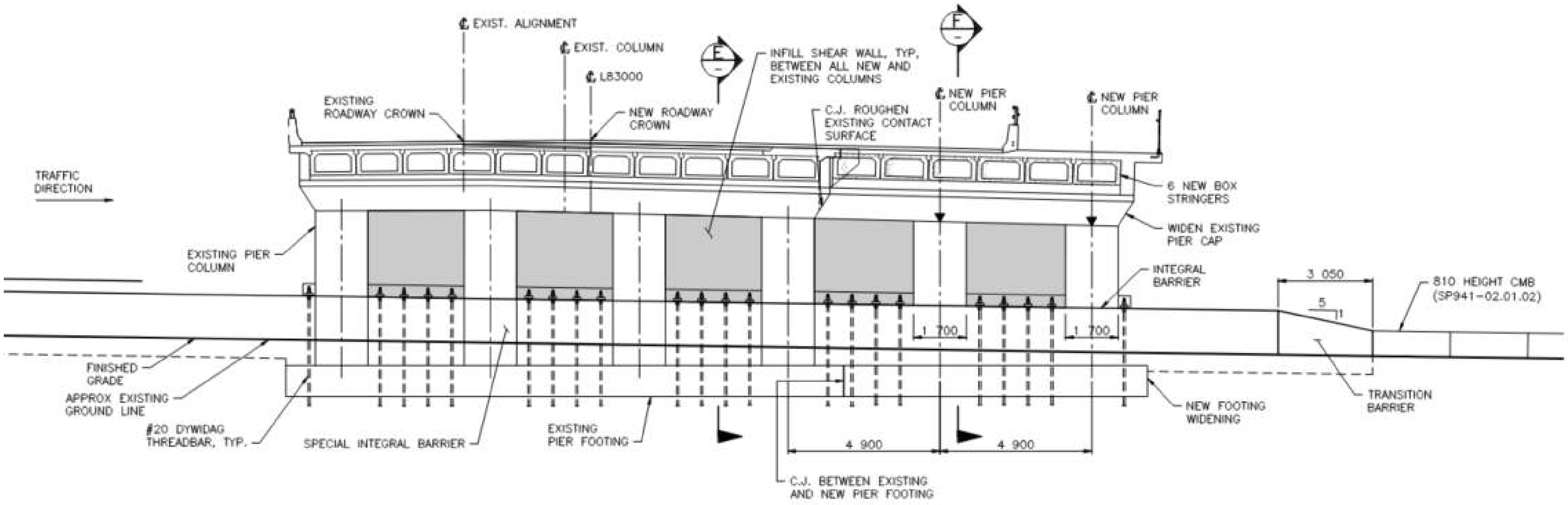
LEGEND / NOTES

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

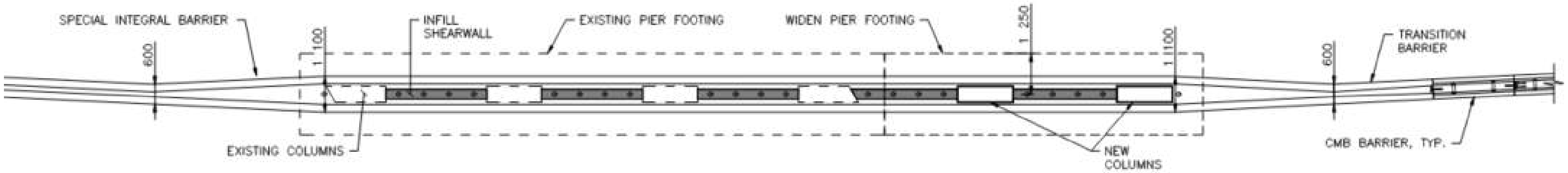
DRAWING TITLE
Approximate Location of Sacrificial Micropiles

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY -
DATE 2023-12-12
DESIGNED BY -
SCALE -
APPROVED BY -
PROJECT No. 32079
DRAWING / FIGURE No. 2.2
REV. 0



TYPICAL PIER ELEVATION
SCALE 1:100
PIER 1 SHOWN, PIER 2 & PIER 3 SIMILAR



TYPICAL PIER FOUNDATION
SCALE 1:100
PIER 1 SHOWN, PIER 2 & PIER 3 SIMILAR

LEGEND / NOTES
- Originated from AE's Dwg. 1562-305-B

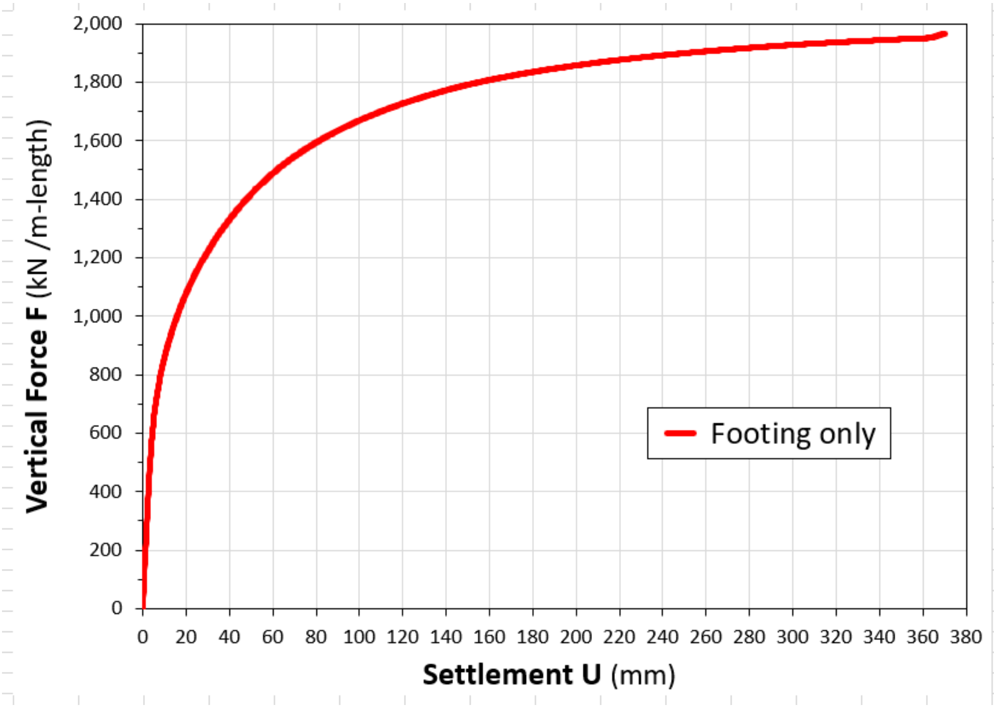
T

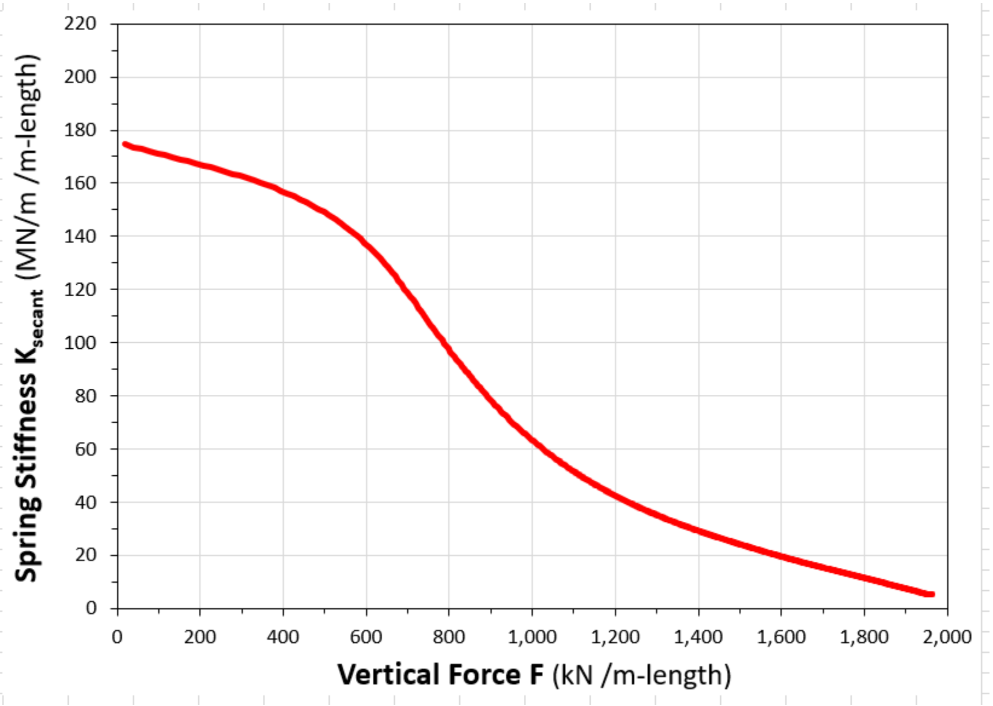
CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Typical New Widening and Existing Footing Retrofit with Micropiles at Piers

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY -	DATE 2023-12-12
DESIGNED BY CKN	SCALE NTS
APPROVED BY -	PROJECT No. 32079
DRAWING / FIGURE No. 3.1	REV. 0





LEGEND / NOTES

- Bending moments were not considered in development of the load-deflection and stiffness relationships.
- The results represent static loading conditions only.

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Vertical Load-Deflection & Stiffness Relationship for Static Design of Mt. Lehman Pier Footings

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY JCC	DATE 2023-12-12
DESIGNED BY CKN	SCALE -
APPROVED BY -	PROJECT No. 32079
DRAWING / FIGURE No. 3.2	REV. 0

LUMPED SOIL SPRINGS FOR SHALLOW FOOTINGS

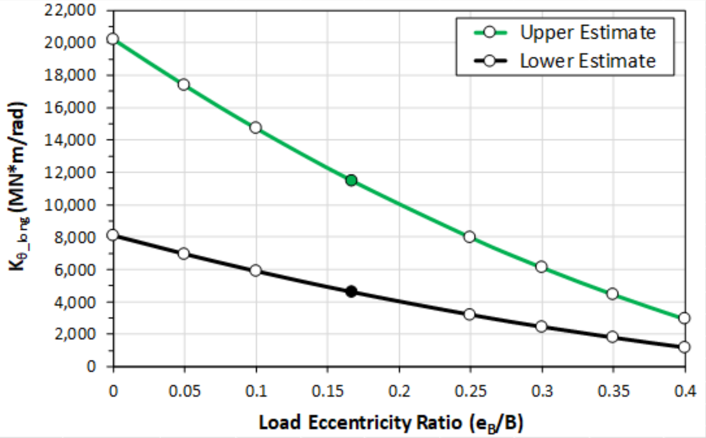
LOWER Estimate: V_s = 300 m/s, G/G_{max} = 0.20

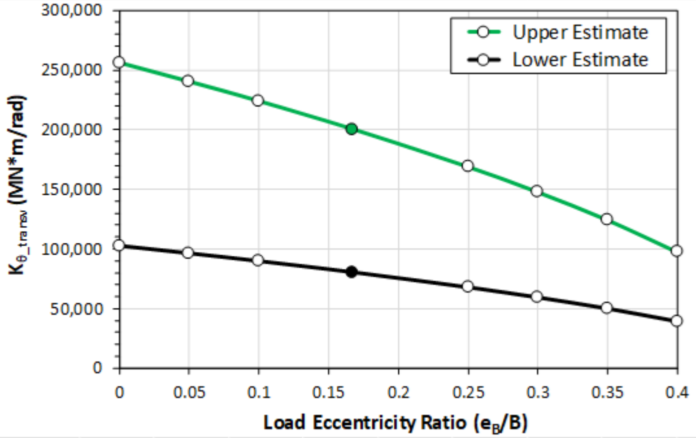
e_B/B		Translation K_u (MN/m)			Rotation K_θ (MN*m/rad)		
		vert	transv	long	vert	long	transv
≤0.17	No uplift	1,322	1,551	1,282	159,304	see table	
0.17 to 0.30	Uplift	1,252	1,472	1,162	155,143		
0.30 to 0.40	Uplift	1,250	1,433	1,067	160,017		

UPPER Estimate: V_s = 300 m/s, G/G_{max} = 0.50

e_B/B		Translation K_u (MN/m)			Rotation K_θ (MN*m/rad)		
		vert	transv	long	vert	long	transv
≤0.17	No uplift	3,305	3,878	3,205	398,259	see table	
0.17 to 0.30	Uplift	3,129	3,679	2,904	387,859		
0.30 to 0.40	Uplift	3,126	3,581	2,667	400,041		

		LOWER Estimate		UPPER Estimate	
e_B/B		K_θ (MN*m/rad)		K_θ (MN*m/rad)	
		long	transv	long	transv
0	No Uplift	8,084	102,395	20,209	255,986
0.05	"	6,943	96,110	17,358	240,276
0.10	"	5,885	89,543	14,713	223,858
0.17	"	4,600	80,256	11,501	200,639
0.25	Uplift	3,193	67,525	7,982	168,814
0.30	"	2,450	59,061	6,125	147,653
0.35	"	1,780	49,697	4,450	124,241
0.40	"	1,177	38,964	2,943	97,410





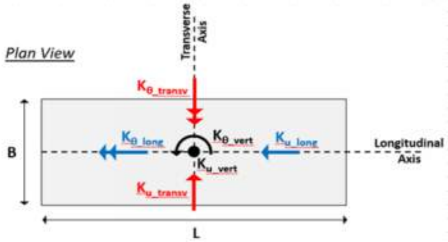
LUMPED SOIL SPRINGS FOR SHALLOW FOOTINGS

Local Axes: Forces

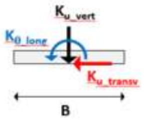
Vert: vertical
Long: longitudinal
Transv: transverse

u: translational component
θ: rotational component

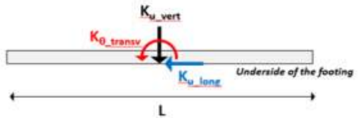
Plan View



Transverse Cross Section



Longitudinal Cross Section

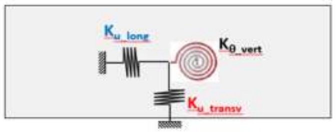


Local Axes: Springs

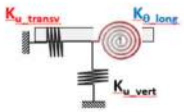
Vert: vertical
Long: longitudinal
Transv: transverse

u: translational component
θ: rotational component

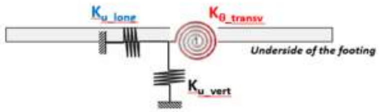
Plan View



Transverse Cross Section



Longitudinal Cross Section



LEGEND / NOTES

- The compliance spring do not consider the presence of the micorpiles.

T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Upper and Lower Bound Linear Compliance Springs for Mt. Lehman Pier Footings

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
JCC

DATE
2023-12-12

DESIGNED BY
CKN

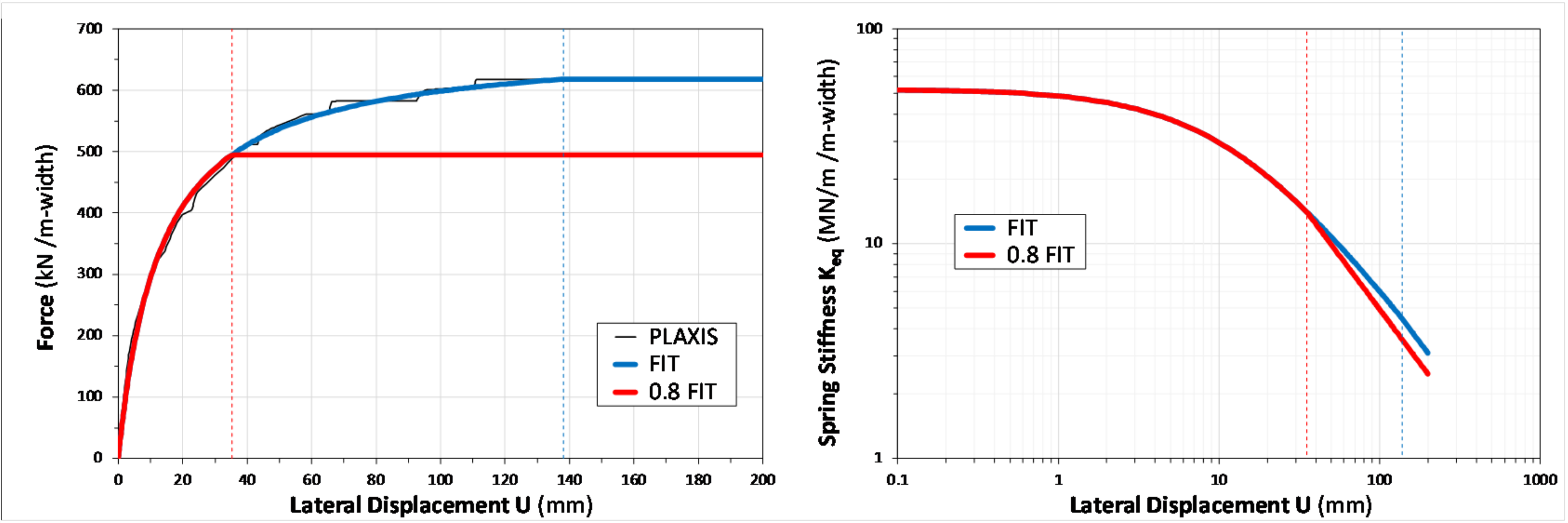
SCALE
-

APPROVED BY
-

PROJECT No.
32079

DRAWING / FIGURE No.
3.3

REV.
0



LEGEND / NOTES

- The results represent the footing with a micropile.

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Non-Linear Compliance Springs (Translational) for Mt. Lehman Pier Footings

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
JCC

DATE
2023-12-12

DESIGNED BY
CKN

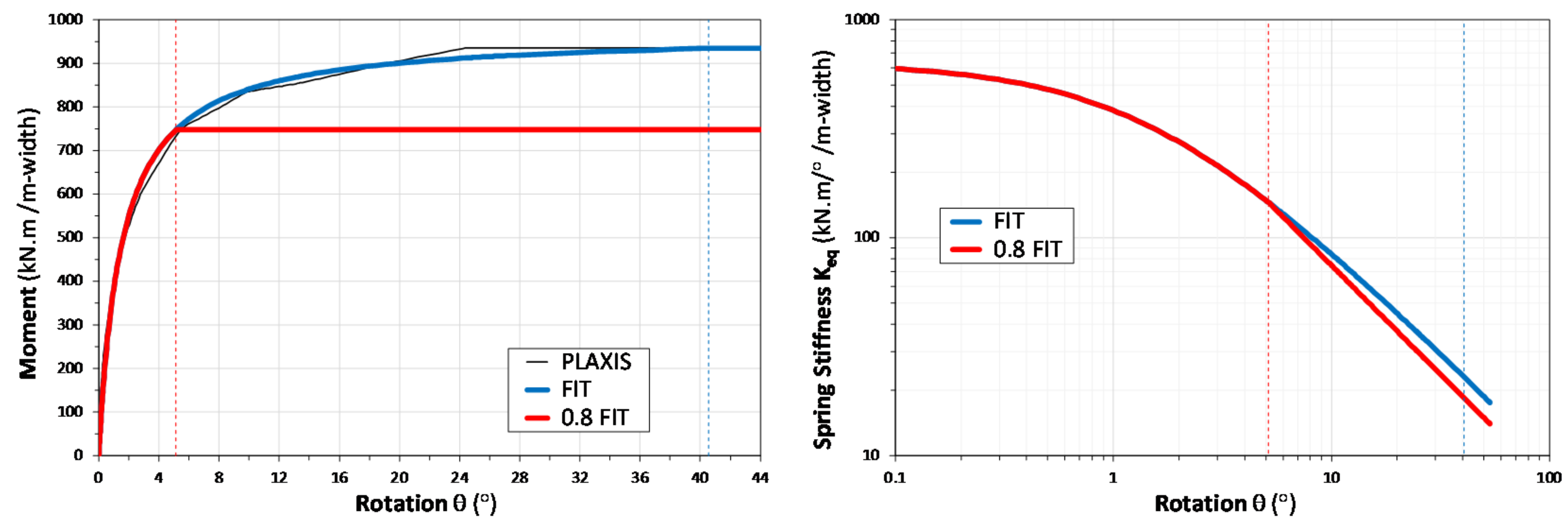
SCALE
-

APPROVED BY
-

PROJECT No.
32079

DRAWING / FIGURE No.
3.4

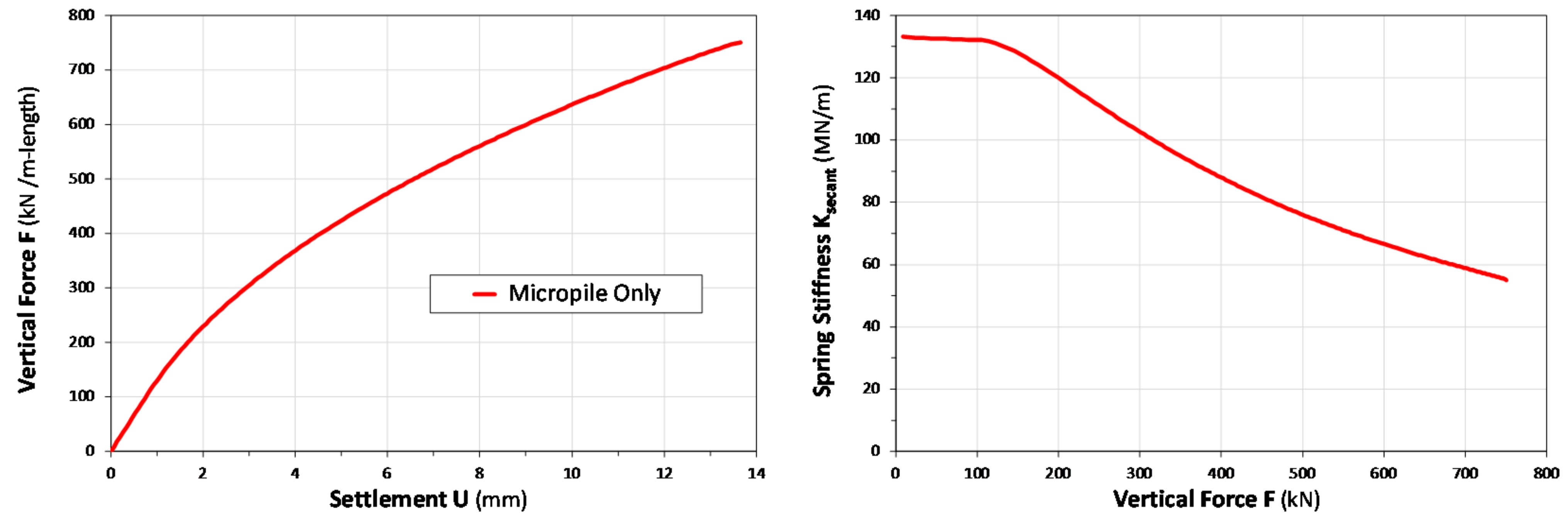
REV.
0



LEGEND / NOTES

- The results represent the footing with a micropile.

CLIENT NAME	BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE
DRAWING TITLE	Non-Linear Compliance Springs (Rotational) for Mt. Lehman Pier Footings
PROJECT NAME AND LOCATION	HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2) ABBOTSFORD, BC
DRAWN BY	JCC
DATE	2023-12-12
DESIGNED BY	CKN
SCALE	-
APPROVED BY	-
PROJECT No.	32079
DRAWING / FIGURE No.	3.5
REV.	0



LEGEND / NOTES

- The results represent a micropile only.

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Non-Linear Compliance Springs for Micropiles at the Pier Footings

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
JCC

DATE
2023-12-12

DESIGNED BY
CKN

SCALE
-

APPROVED BY
-

PROJECT No.
32079

DRAWING / FIGURE No.
3.6

REV.
0

Figure C6.27
Various earth pressures

(See Clauses C6.12.1, C6.14.6.4, and C6.14.7.2.)

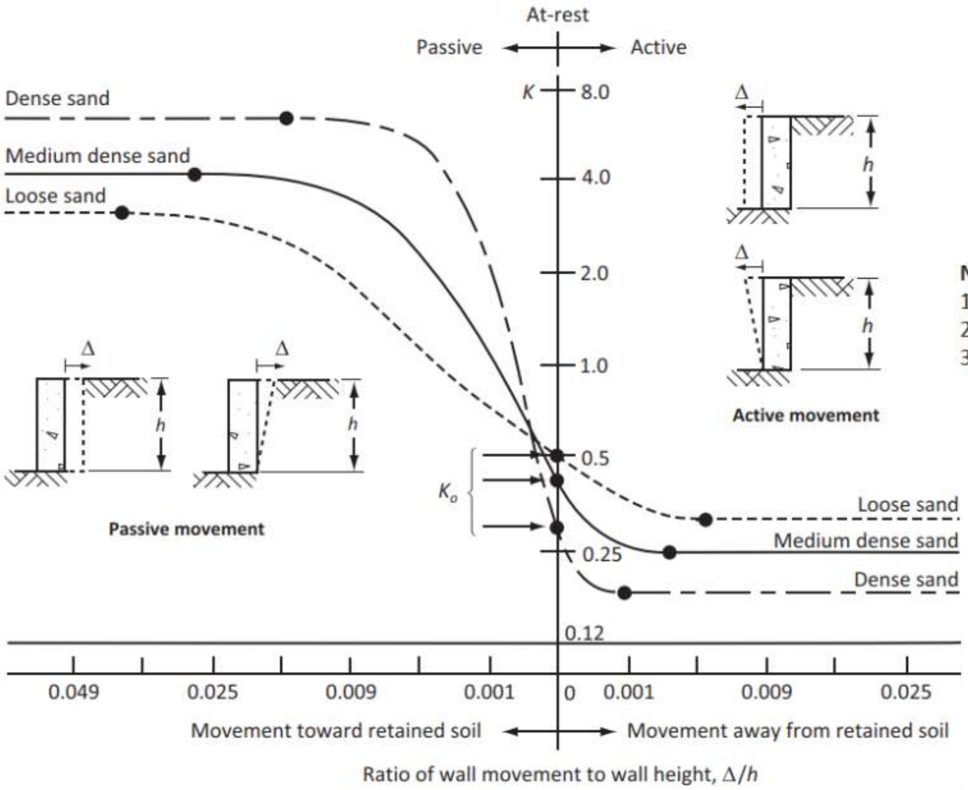


Table C6.12
Movements required to mobilize various conditions

(See Clauses C6.12.1 and C6.12.2.2.)

Movement to mobilize			
Active pressure		**Passive pressure**	
Displacement, *Δ*	**Rotation, *Δ/h***	**Displacement, *Δ***	**Rotation, *Δ/h***
0.001h	0.002 (about bottom of wall)	0.050h	0.100 (about bottom of wall) 0.020 (about top of wall)

Notes:

1) *Displacements take place in the absence of rotation.*
2) *h is the height of the retaining wall.*
3) *Rotation is assumed to take place about a fixed point at either the top or the bottom of the wall.*

LEGEND / NOTES

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
CHBDC S6-19 Commentary - Movements Required to Mobilize Passive Pressures

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
JCC

DATE
2023-12-12

DESIGNED BY
CKN

SCALE
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APPROVED BY
-

PROJECT No.
32079

DRAWING / FIGURE No.
3.7

REV.
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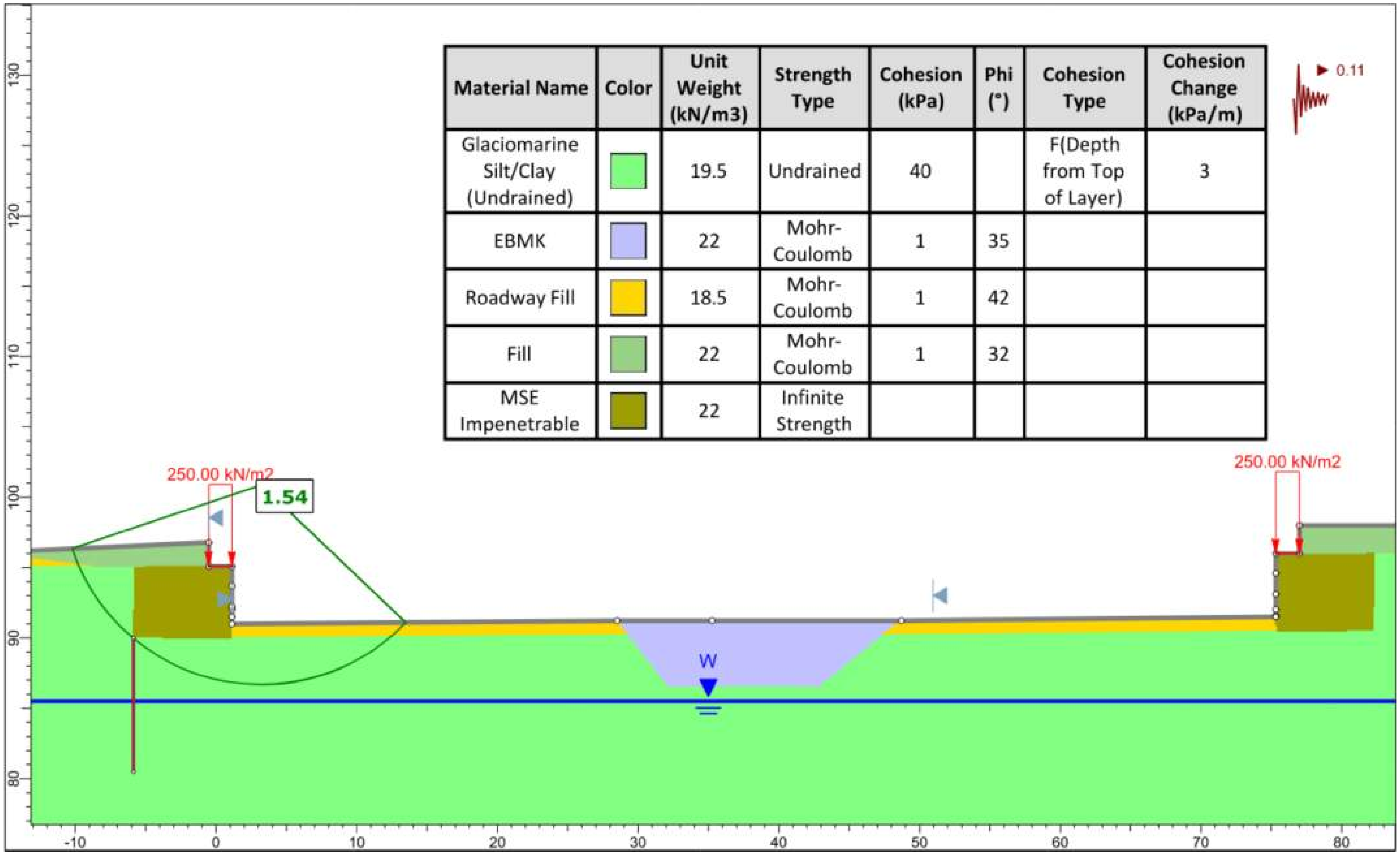

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Cohesion Change (kPa/m)
Glaciomarine Silt/Clay (Undrained)	🟩	19.5	Undrained	40		F(Depth from Top of Layer)	3
EBMK	🟪	22	Mohr-Coulomb	1	35		
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42		
Fill	🟩	22	Mohr-Coulomb	1	32		
MSE Impenetrable	🟫	22	Infinite Strength				

LEGEND / NOTES

T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Global Stability Results for Existing MSE Wall at South Abutment (A475)

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
CKN

DATE
2023-12-12

DESIGNED BY
DM

SCALE
-

APPROVED BY
-

PROJECT No.
32079

DRAWING / FIGURE No.
3.8

REV.
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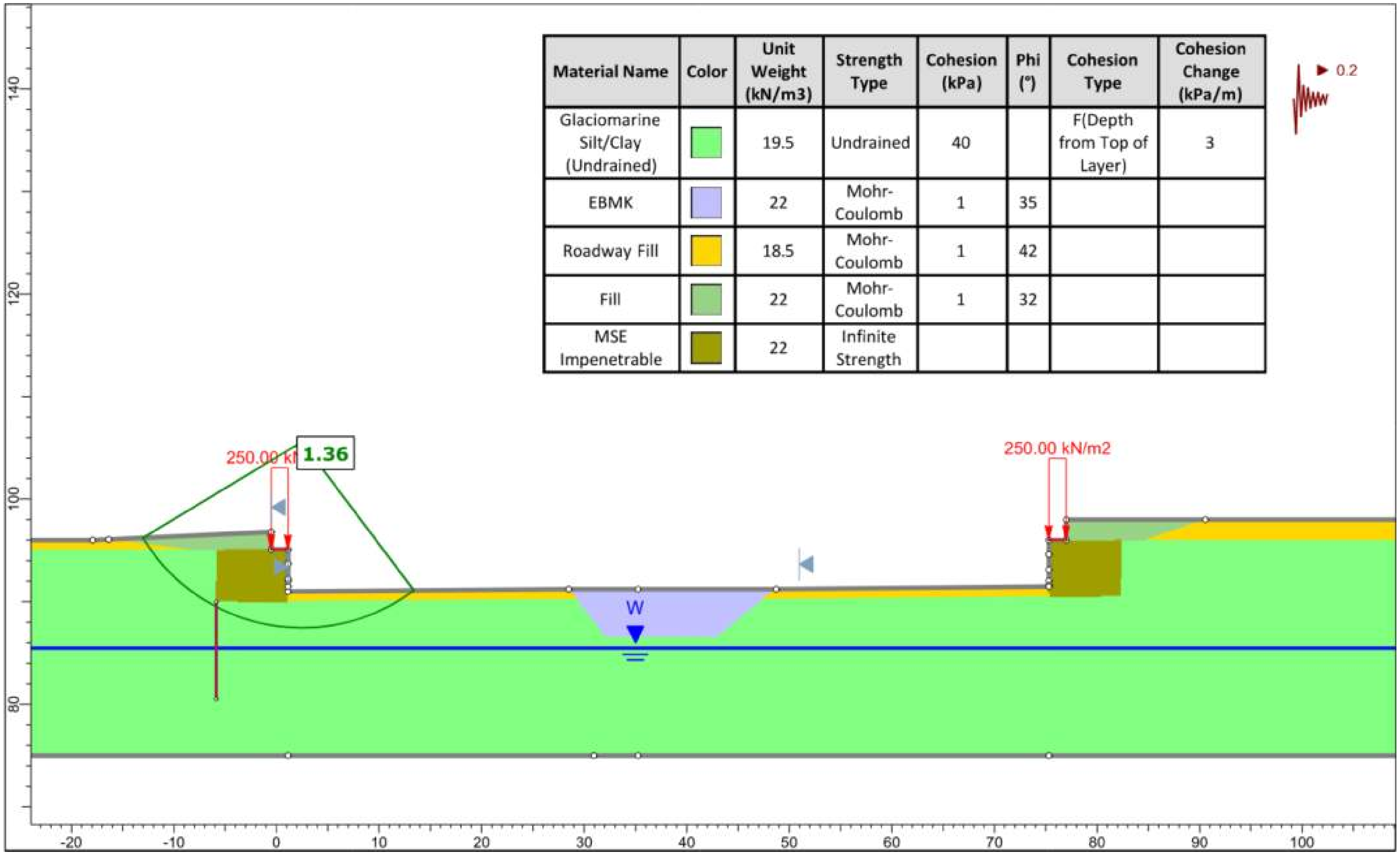

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Cohesion Change (kPa/m)
Glaciomarine Silt/Clay (Undrained)	🟩	19.5	Undrained	40		F(Depth from Top of Layer)	3
EBMK	🟪	22	Mohr-Coulomb	1	35		
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42		
Fill	🟩	22	Mohr-Coulomb	1	32		
MSE Impenetrable	🟫	22	Infinite Strength				

LEGEND / NOTES

CLIENT NAME BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE	DRAWN BY CKN	DATE 2023-12-12	
DRAWING TITLE Global Stability Results for Existing MSE Wall at South Abutment (A2475)	DESIGNED BY DM	SCALE -	
PROJECT NAME AND LOCATION HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2) ABBOTSFORD, BC	APPROVED BY -	PROJECT No. 32079	
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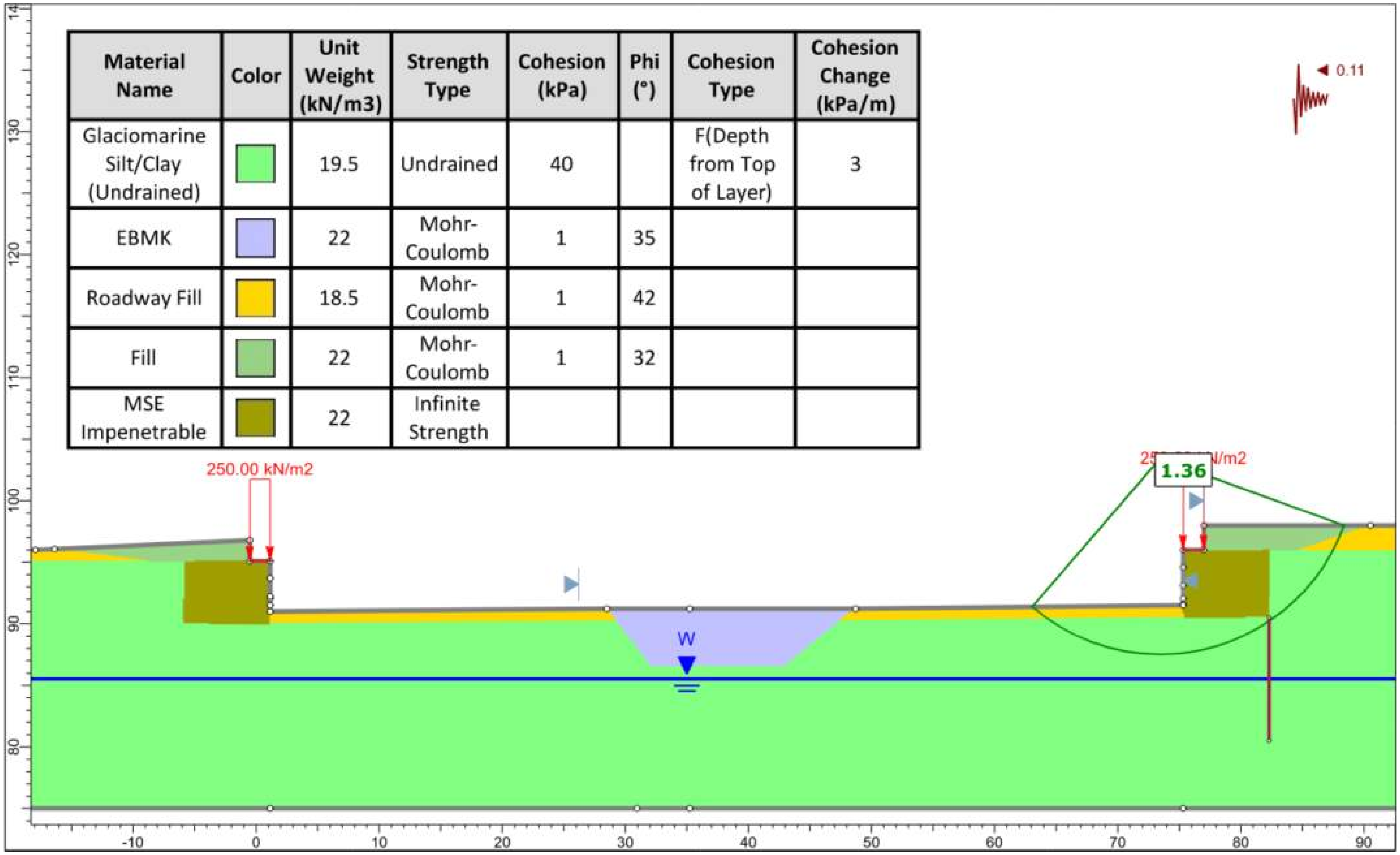


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Cohesion Change (kPa/m)
Glaciomarine Silt/Clay (Undrained)	🟩	19.5	Undrained	40		F(Depth from Top of Layer)	3
EBMK	🟦	22	Mohr-Coulomb	1	35		
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42		
Fill	🟩	22	Mohr-Coulomb	1	32		
MSE Impenetrable	🟫	22	Infinite Strength				

LEGEND / NOTES

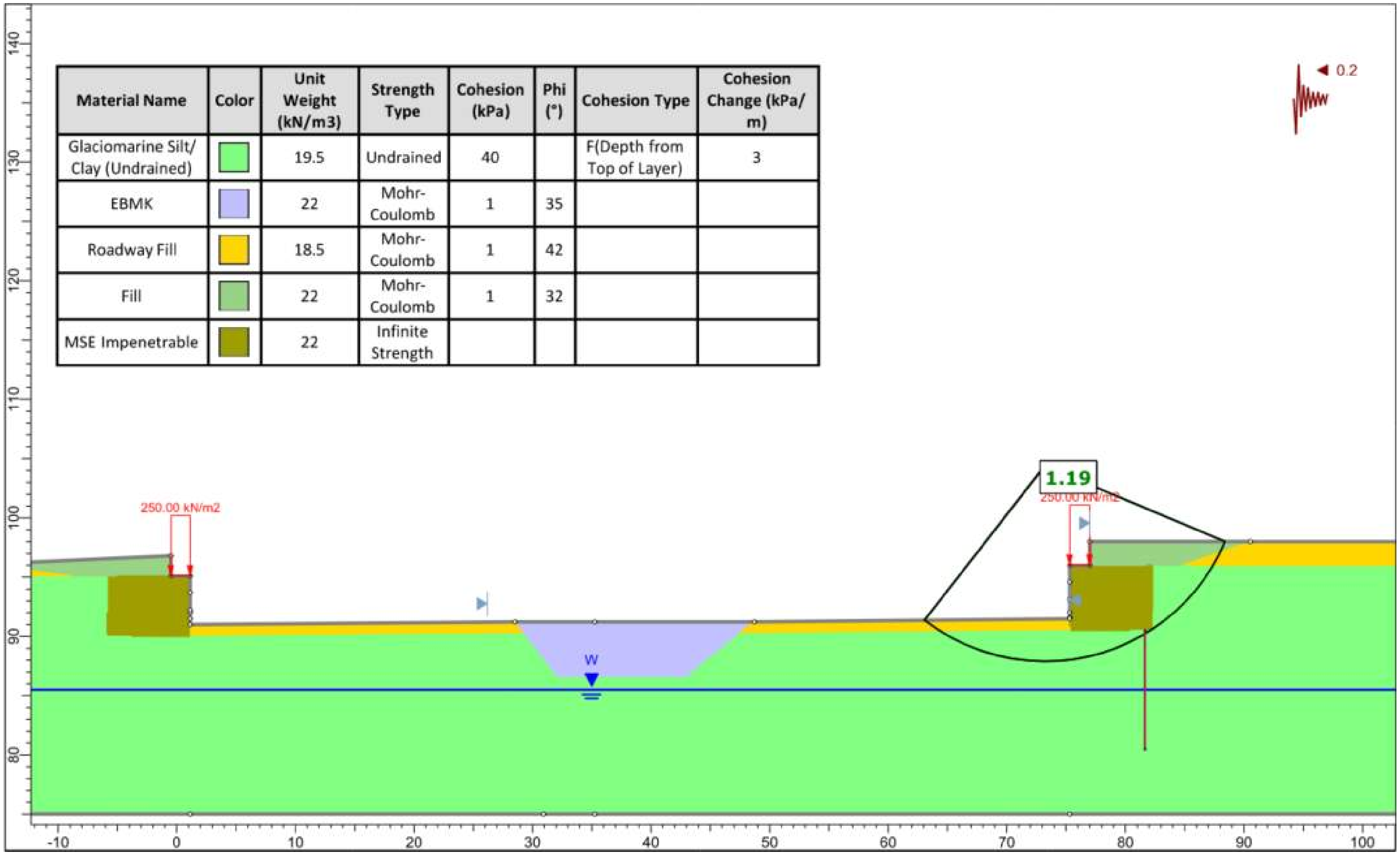
T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Global Stability Results for Existing MSE Wall at North Abutment (A475)

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY CKN	DATE 2023-12-12
DESIGNED BY DM	SCALE -
APPROVED BY -	PROJECT No. 32079
DRAWING / FIGURE No. 3.10	REV. 0



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Cohesion Change (kPa/m)
Glaciomarine Silt/ Clay (Undrained)	🟩	19.5	Undrained	40		F(Depth from Top of Layer)	3
EBMK	🟪	22	Mohr-Coulomb	1	35		
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42		
Fill	🟩	22	Mohr-Coulomb	1	32		
MSE Impenetrable	🟫	22	Infinite Strength				

LEGEND / NOTES

T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Global Stability Results for Existing MSE Wall at North Abutment (A2475)

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
CKN

DATE
2023-12-12

DESIGNED BY
DM

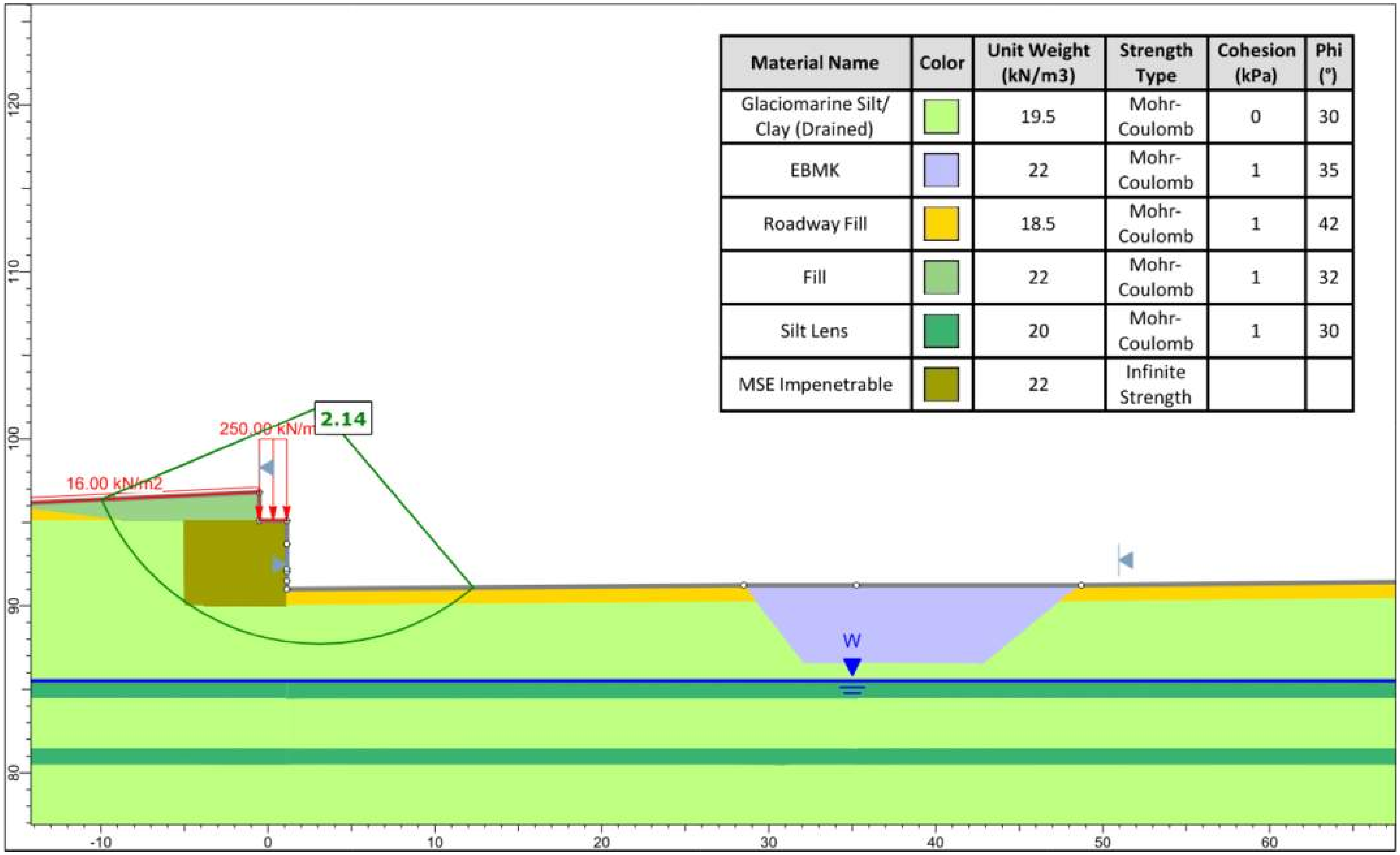
SCALE
-

APPROVED BY
-

PROJECT No.
32079

DRAWING / FIGURE No.
3.11

REV.
0



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Glaciomarine Silt/ Clay (Drained)	🟩	19.5	Mohr-Coulomb	0	30
EBMK	🟪	22	Mohr-Coulomb	1	35
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42
Fill	🟩	22	Mohr-Coulomb	1	32
Silt Lens	🟩	20	Mohr-Coulomb	1	30
MSE Impenetrable	🟫	22	Infinite Strength		

LEGEND / NOTES

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWN BY
CKN

DATE
2023-12-12

DRAWING TITLE
Global Stability Results for New MSE Wall at South Abutment (Static)

DESIGNED BY
DM

SCALE
-

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

APPROVED BY
-

PROJECT No.
32079

DRAWING / FIGURE No.
3.12

REV.
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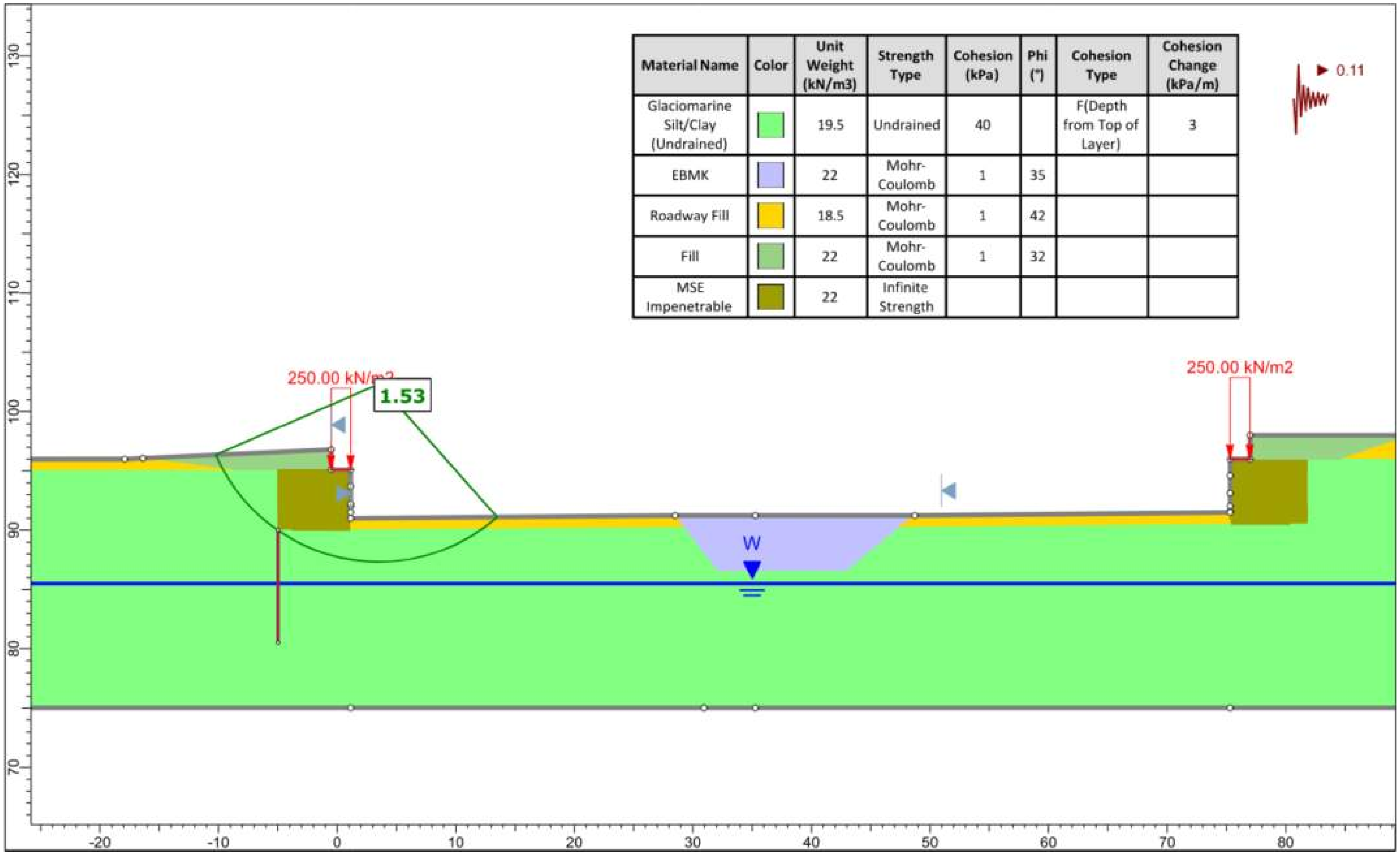


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Cohesion Change (kPa/m)
Glaciomarine Silt/Clay (Undrained)	🟩	19.5	Undrained	40		F(Depth from Top of Layer)	3
EBMK	🟪	22	Mohr-Coulomb	1	35		
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42		
Fill	🟩	22	Mohr-Coulomb	1	32		
MSE Impenetrable	🟫	22	Infinite Strength				

LEGEND / NOTES

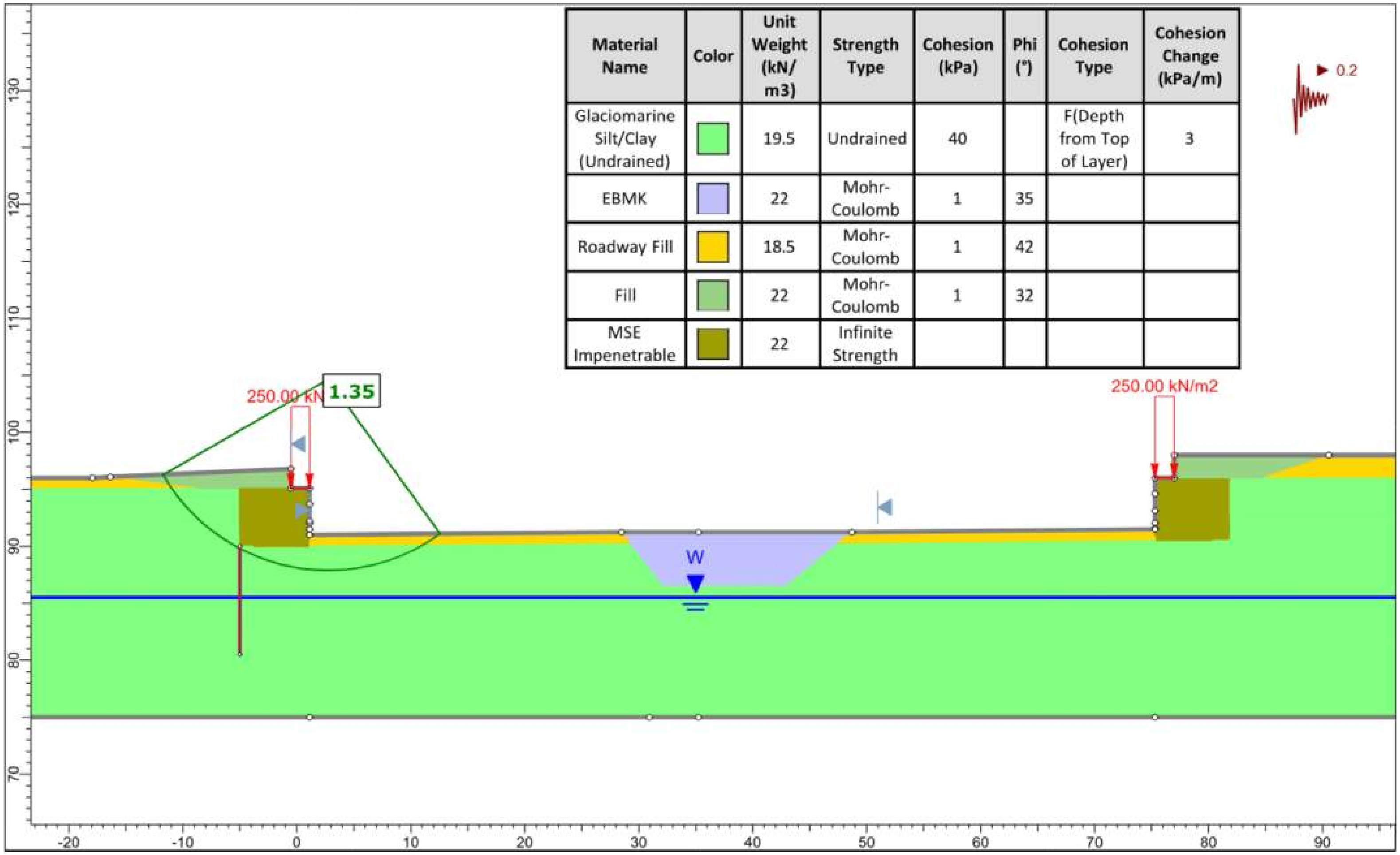
T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Global Stability Results for New MSE Wall at South Abutment (A475)

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY CKN	DATE 2023-12-12
DESIGNED BY DM	SCALE -
APPROVED BY -	PROJECT No. 32079
DRAWING / FIGURE No. 3.13	REV. 0



Material Name	Color	Unit Weight (kN/ m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Cohesion Change (kPa/m)
Glaciomarine Silt/Clay (Undrained)	🟩	19.5	Undrained	40		F(Depth from Top of Layer)	3
EBMK	🟪	22	Mohr-Coulomb	1	35		
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42		
Fill	🟩	22	Mohr-Coulomb	1	32		
MSE Impenetrable	🟫	22	Infinite Strength				

LEGEND / NOTES

T

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Global Stability Results for New MSE Wall at South Abutment (A2475)

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY CKN	DATE 2023-12-12
DESIGNED BY DM	SCALE -
APPROVED BY -	PROJECT No. 32079
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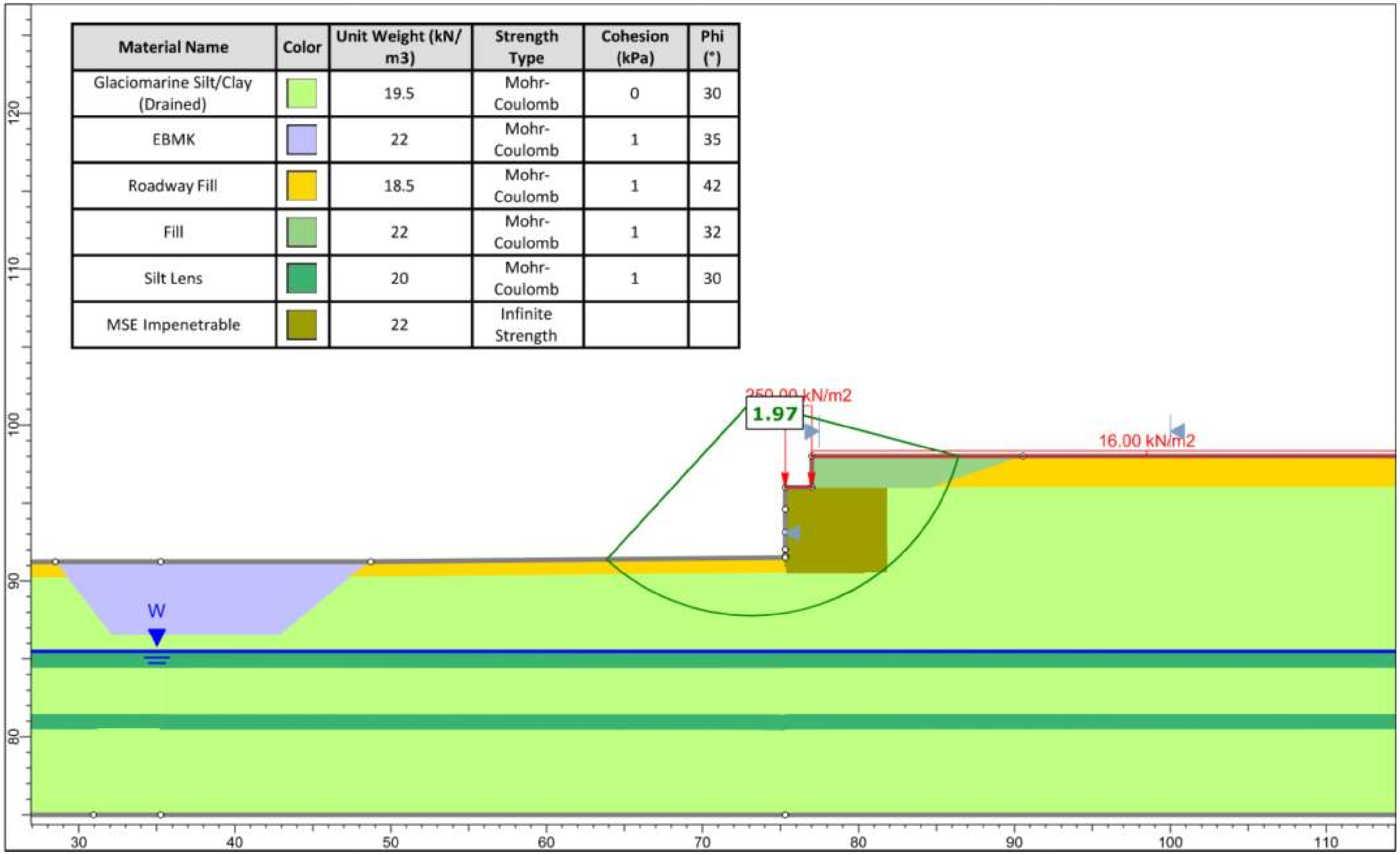


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)
Glaciomarine Silt/Clay (Drained)	🟩	19.5	Mohr-Coulomb	0	30
EBMK	🟪	22	Mohr-Coulomb	1	35
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42
Fill	🟩	22	Mohr-Coulomb	1	32
Silt Lens	🟩	20	Mohr-Coulomb	1	30
MSE Impenetrable	🟫	22	Infinite Strength		

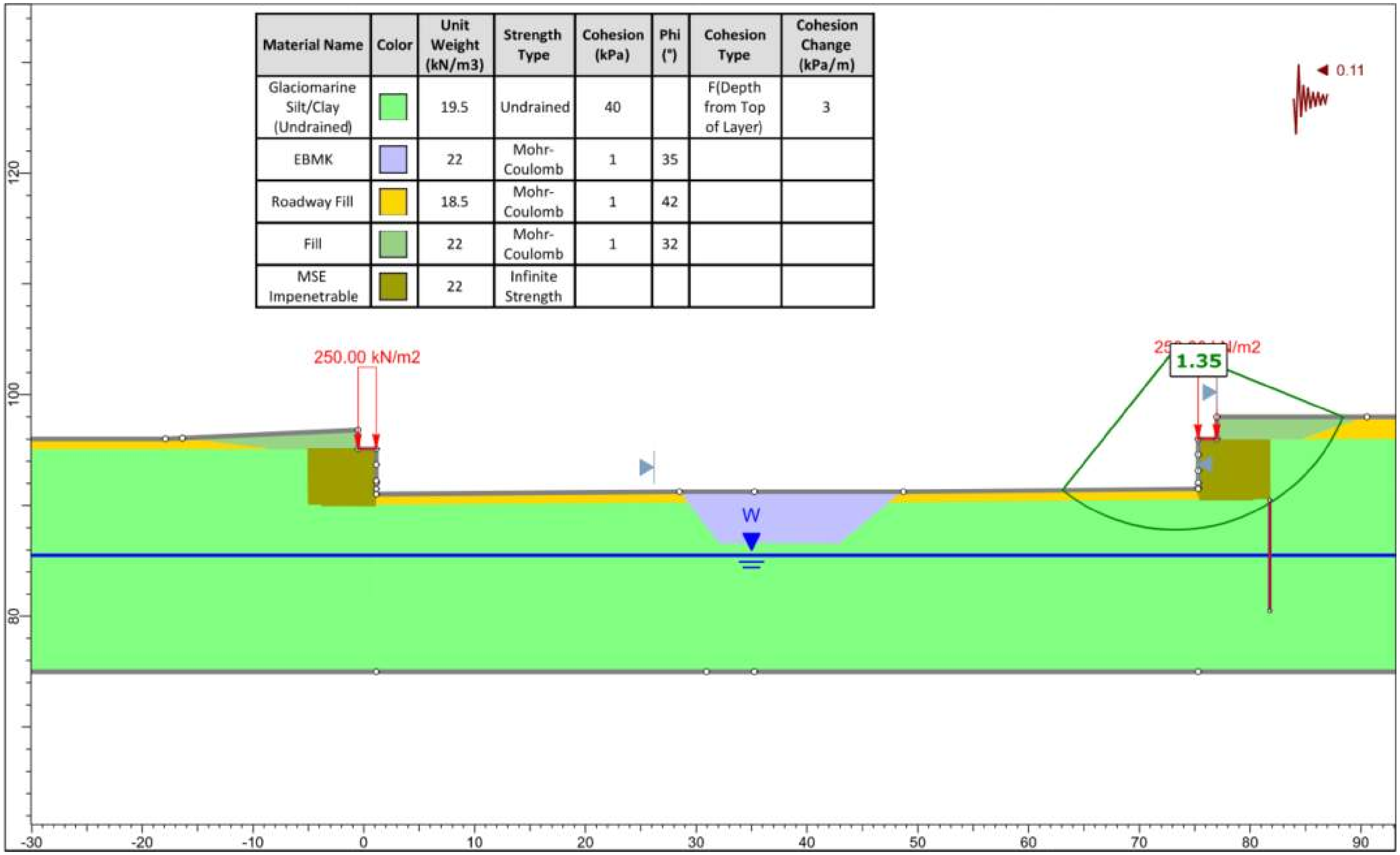
LEGEND / NOTES

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Global Stability Results for New MSE Wall at North Abutment (Static)

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY CKN	DATE 2023-12-12
DESIGNED BY DM	SCALE -
APPROVED BY -	PROJECT No. 32079
DRAWING / FIGURE No. 3.15	REV. 0



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Cohesion Change (kPa/m)
Glaciomarine Silt/Clay (Undrained)	🟩	19.5	Undrained	40		F(Depth from Top of Layer)	3
EBMK	🟪	22	Mohr-Coulomb	1	35		
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42		
Fill	🟩	22	Mohr-Coulomb	1	32		
MSE Impenetrable	🟫	22	Infinite Strength				

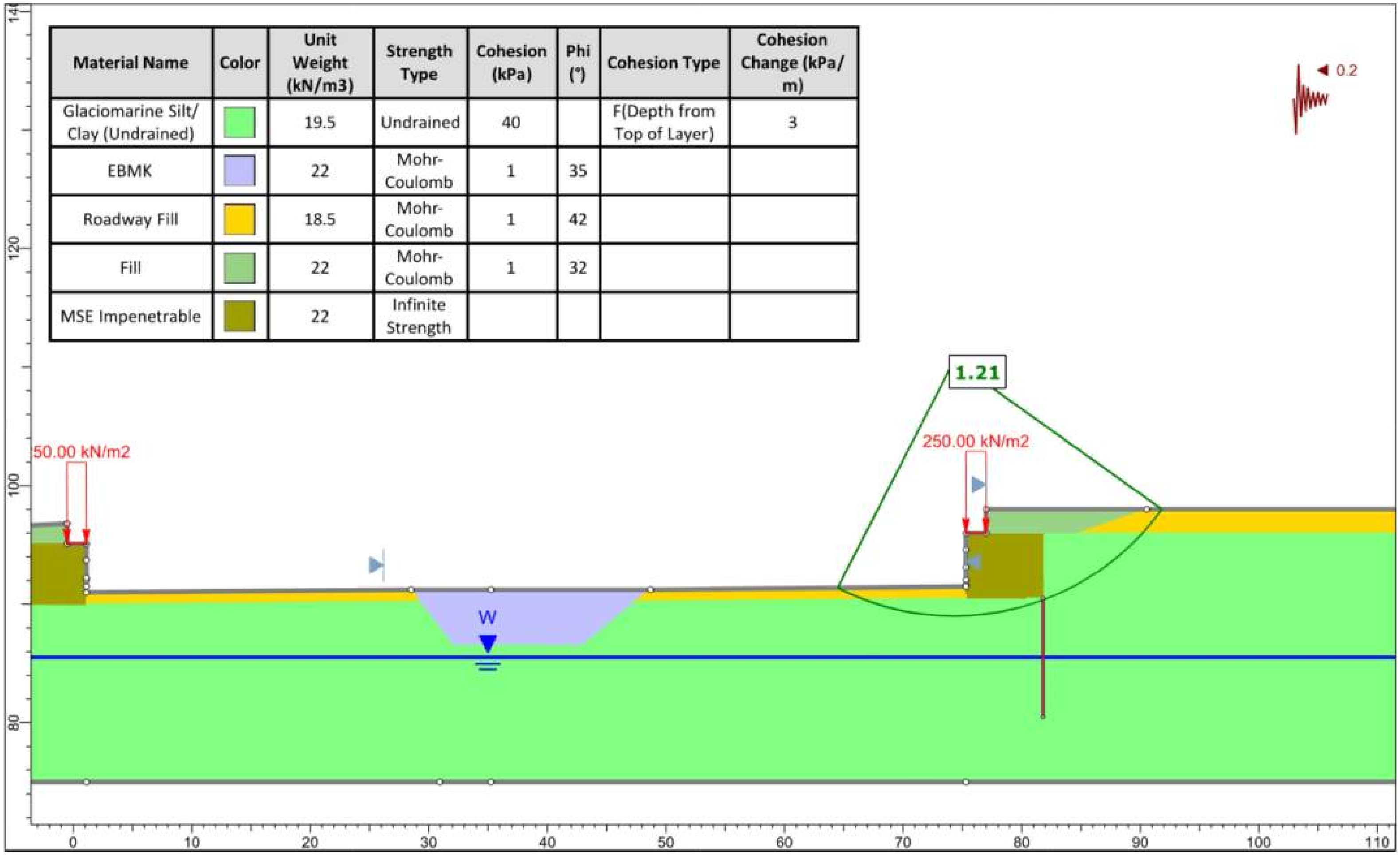
LEGEND / NOTES

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Global Stability Results for New MSE Wall at North Abutment (A475)

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY CKN	DATE 2023-12-12
DESIGNED BY DM	SCALE -
APPROVED BY -	PROJECT No. 32079
DRAWING / FIGURE No. 3.16	REV. 0



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (°)	Cohesion Type	Cohesion Change (kPa/m)
Glaciomarine Silt/ Clay (Undrained)	🟩	19.5	Undrained	40		F(Depth from Top of Layer)	3
EBMK	🟪	22	Mohr-Coulomb	1	35		
Roadway Fill	🟨	18.5	Mohr-Coulomb	1	42		
Fill	🟩	22	Mohr-Coulomb	1	32		
MSE Impenetrable	🟫	22	Infinite Strength				

LEGEND / NOTES

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Global Stability Results for New MSE Wall at North Abutment (A2475)

PROJECT NAME AND LOCATION
HIGHWAY 1 WIDENING - 264th STREET TO WHATCOM ROAD (SEGMENT 2)
ABBOTSFORD, BC

DRAWN BY
CKN

DATE
2023-12-12

DESIGNED BY
DM

SCALE
-

APPROVED BY
-

PROJECT No.
32079

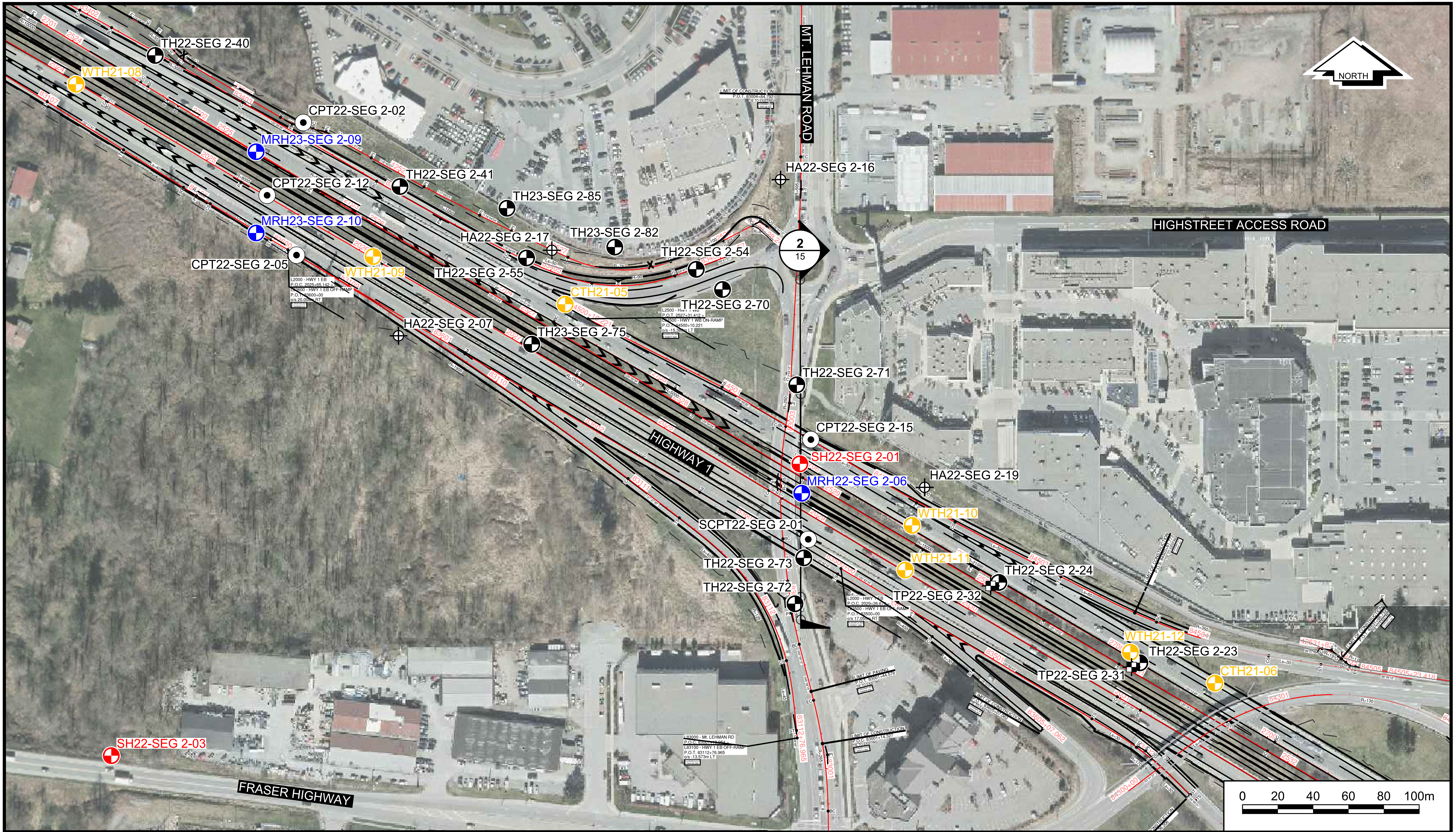
DRAWING / FIGURE No.
3.17

REV.
0

APPENDIX B CURRENT AND HISTORIC INVESTIGATION INFORMATION

Dwg. 32079-SEG 2-3 Investigation Location Plan near Mt Lehman Underpass
Dwg. 32079-SEG 2-15 Segment 2 Stratigraphic Cross-Section, Section 2

Thurber Investigation (Draft Logs for Test Holes, CPT and SCPT, DHST Table and Plot)
2004 Investigation Information by Trow Associates Inc.
1994 BC Ministry of Transportation Test Hole Logs
1959 BC Department of Highways Test Hole Logs



LEGEND:

◐	TEST HOLE (SOLID STEM AUGER)
◐	TEST HOLE (MUD ROTARY)
◐	TEST HOLE (SONIC)
⊕	TEST HOLE (HAND AUGER)
⌖	TEST PIT
⊙	CPT / SCPT
◐	TEST HOLE (TETRA-TECH)
△	INFILTRATION TEST

NOTES:

1. AERIAL MAGE TAKEN FROM THE CITY OF ABBOTSFORD MAPPING SITE.
2. INVESTIGATION LOCATIONS ARE APPROXIMATE .
3. BASE PLAN TAKEN FROM "ACAD-GEOMLANE-12947-HWY1_20230313.DWG" FOR PROVIDED BY ASSOCIATED ENGINEERING.

THURBER ENGINEERING LTD.

MINISTRY OF TRANSPORTATION AND INFRASTUCTURE

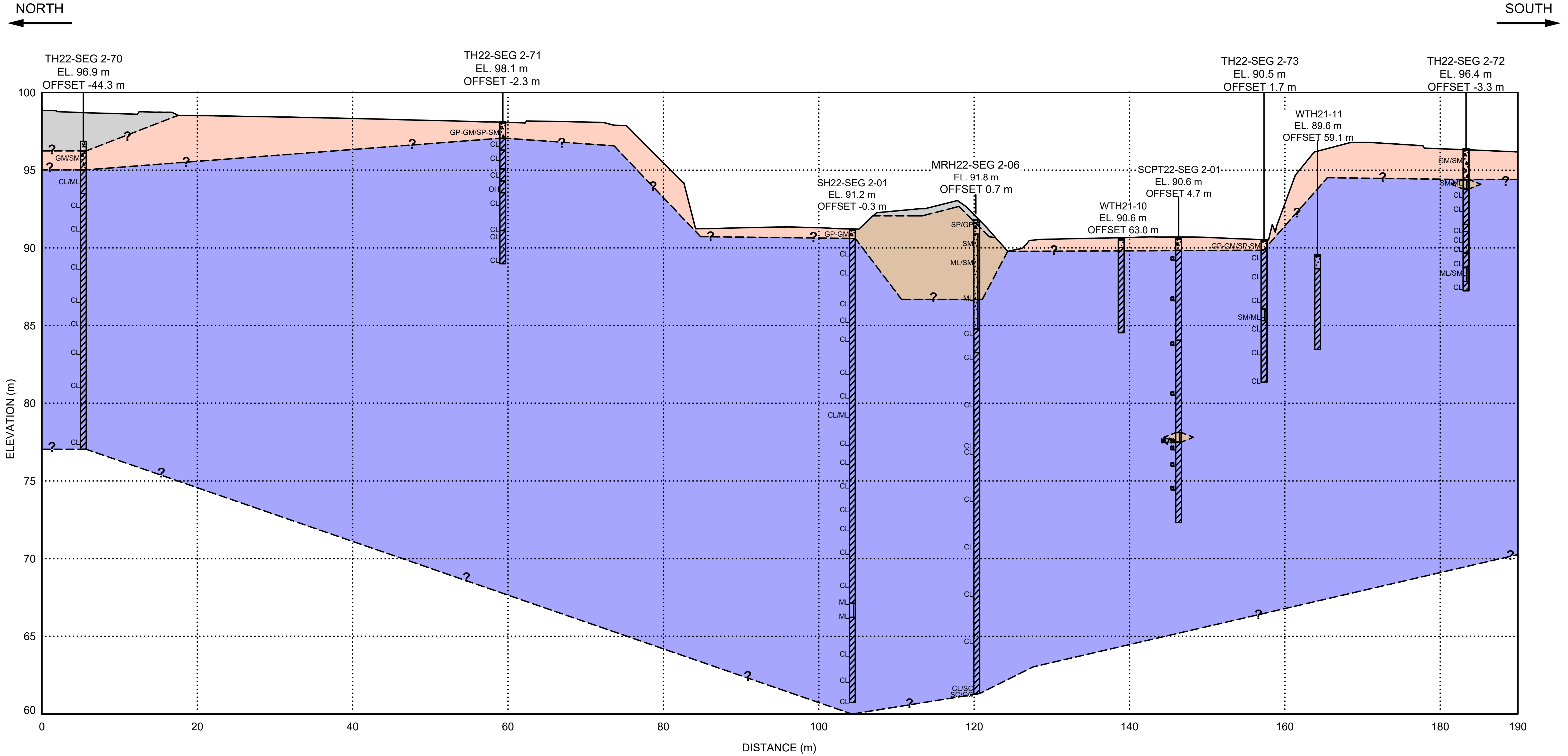
INVESTIGATION LOCATION PLAN

FRASER VALLEY HIGHWAY 1
CORRIDOR IMPROVEMENT

LANGLEY AND ABBOTSFORD, BC

DESIGNED	DRAWN	APPROVED	DATE	SCALE	PROJECT No.	DWG. NO.	REV.
ANR	MOM		MAR. 16, 2023	1:2000	32079 -	SEG 2-3	0

Plotted: March 16, 2023



NOTES:
1. STRATIGRAPHY IS SHOWN FOR ILLUSTRATIVE PURPOSES ONLY AND IS BASED ON INTERPRETED CONDITIONS BETWEEN BOREHOLES. ACTUAL STRATIGRAPHY MAY BE DIFFERENT THAN WHAT IS SHOWN.
2. GROUND SURFACE PROFILE IS BASED ON THE 100% FUNCTIONAL DESIGN DRAWINGS PROVIDED BY ASSOCIATED ENGINEERING LTD. ON JULY 18, 2022. SEE DWG. 32079-SEG 2-3 FOR SECTION LOCATION.

H= 0 5 10 15 20 25m

V= 0 2.5 5 7.5 10 12.5m

LEGEND:

—?— STRATIGRAPHY LINE

▨ ASPHALT, ROADBASE AND GRANULAR FILL MATERIAL

▨ ORGANIC SILT / TOPSOIL

▨ FINE GRAINED FILL

▨ SANDY GRAVEL TO SAND AND GRAVEL

▨ COMPACT TO DENSE SAND WITH TRACE TO SOME SOME SILT TRACE GRAVEL

▨ STIFF CLAY - SOME THIN BEDS OF SILTY SAND - GLACIOMARINE DEPOSITS

▨ DENSE SAND TO SILTY SAND, SOME GRAVEL TO GRAVELLY.

▨ SILT TO SILTY SAND (LIQUEFIABLE)

THURBER ENGINEERING LTD.

MINISTRY OF TRANSPORTATION AND INFRASTUCTURE

SEGMENT 2 STRATIGRAPHIC CROSS-SECTION
SECTION 2

FRASER VALLEY HIGHWAY 1
CORRIDOR IMPROVEMENT

LANGLEY AND ABBOTSFORD, BC

DESIGNED	DRAWN	APPROVED	DATE	SCALE	PROJECT No.	DWG. NO.	REV.
ANR	MOM		MAR. 16, 2023	H=1:500, V=1:250	32079 -	SEG 2-15	0

S:\Data\BST Projects\32xxx\32079\06_Drafting\03_Working\SECTIONS\SEG 2_SECTION 2 (March 2023).dwg

BRITISH COLUMBIA
Ministry of Transportation and Infrastructure

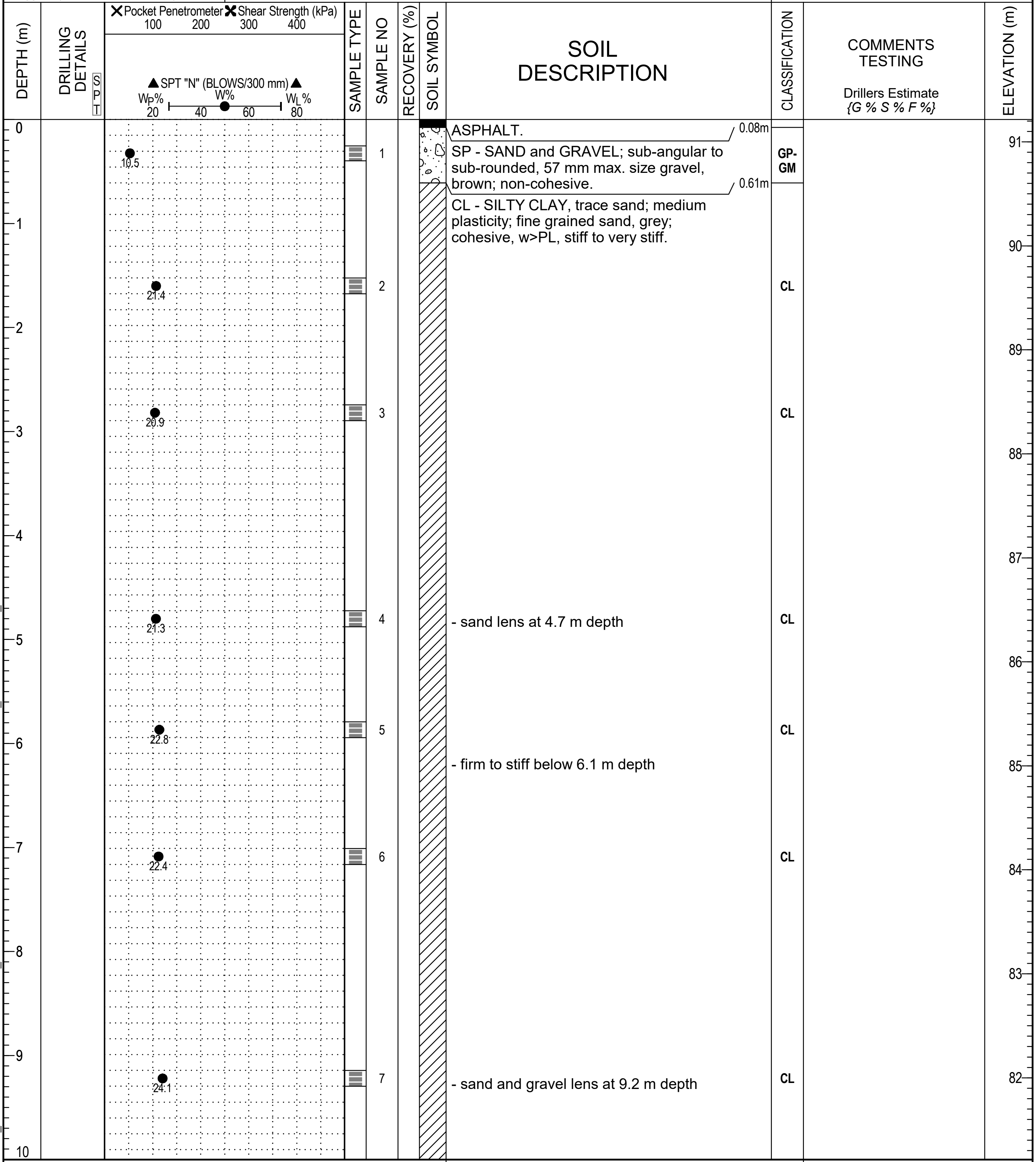
SUMMARY LOG

Drill Hole #: SH22-SEG 2-01

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5434072 , 545198
Elevation: 91.2 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Prepared by: 32079 Thurber Engineering Ltd.
Logged by: RJT
Reviewed by: ANR

Date(s) Drilled: 2022-06-25
Company: Mud Bay
Driller: Stephen
Drill Make/Model: Terra Sonic TSCC-05
Drilling Method: Sonic



SOIL DESCRIPTION	CLASSIFICATION
ASPHALT. (0.08m)	
SP - SAND and GRAVEL; sub-angular to sub-rounded, 57 mm max. size gravel, brown; non-cohesive. (0.61m)	GP-GM
CL - SILTY CLAY, trace sand; medium plasticity; fine grained sand, grey; cohesive, w>PL, stiff to very stiff.	
- sand lens at 4.7 m depth	
- firm to stiff below 6.1 m depth	
- sand and gravel lens at 9.2 m depth	

SAMPLE NO	W%	CLASSIFICATION
1	10.5	GP-GM
2	21.4	CL
3	20.9	CL
4	21.3	CL
5	22.8	CL
6	22.4	CL
7	24.1	CL

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 30.5 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

Drill Hole #: **SH22-SEG 2-01**

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC

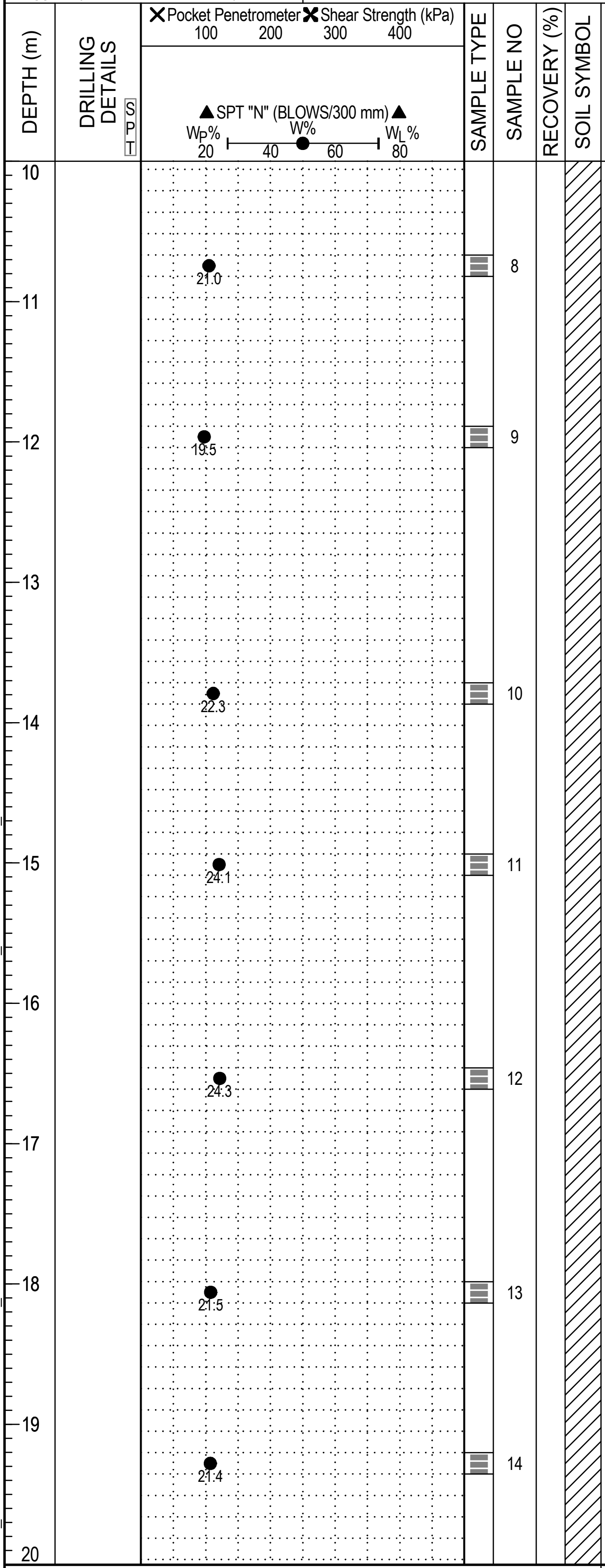
Prepared by: 32079
Thurber Engineering Ltd.

Logged by: RJT Reviewed by: ANR

Datum: UTM-Nad83
Northing/Easting: 5434072 , 545198
Elevation: 91.2 m

Alignment:
Station/Offset:
Coordinates taken with GPS

Date(s) Drilled: 2022-06-25
Company: Mud Bay
Driller: Stephen
Drill Make/Model: Terra Sonic TSCC-05
Drilling Method: Sonic



SOIL DESCRIPTION

CL - SILTY CLAY, trace sand; medium plasticity; fine grained sand, grey; cohesive, w>PL, stiff to very stiff.
(continued)

CLASSIFICATION	SAMPLE NO
CL	8
CL	9
CL	10
CL	11
CL	12
CL	13
CL	14

COMMENTS TESTING

Drillers Estimate
{G % S % F %}

ELEVATION (m)
81
80
79
78
77
76
75
74
73
72

Legend
Sample Type: A-Auger B-Becker C-Core G-Grab V-Vane L#-Lab Sample S-Split Spoon O-Odex (air rotary) W-Wash (mud return) T-Shelby Tube

Legend
Installation: Sand Grout Cement Bentonite Drill Cuttings Slotted Slough Piezometer

Final Depth of Hole: 30.5 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

Drill Hole #: SH22-SEG 2-01

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5434072 , 545198
Elevation: 91.2 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Prepared by: 32079
Thurber Engineering Ltd.
Logged by: RJT Reviewed by: ANR

Date(s) Drilled: 2022-06-25
Company: Mud Bay
Driller: Stephen
Drill Make/Model: Terra Sonic TSCC-05
Drilling Method: Sonic

✕ Pocket Penetrometer ✕ Shear Strength (kPa) 100 200 300 400
▲ SPT "N" (BLOWS/300 mm) ▲
W_P% W% W_L% 20 40 60 80

Depth (m): 20 – 30; Elevation (m): 71 – 62

Sample No	W%	Classification
15	23.2	CL
16	23.1	CL
17	26.5	ML
18	18.2	ML
19	14.1	CL
20	17.3	CL

SOIL DESCRIPTION

CL - SILTY CLAY, trace sand; medium plasticity; fine grained sand, grey; cohesive, w>PL, stiff to very stiff.
(continued)

24.08m

ML - SILT, trace sand, trace gravel; low plasticity; fine to coarse grained sand, grey; cohesive, w>PL, firm.
- sand and gravel lens at 24.4 m depth

24.99m

CL - SILTY CLAY, trace to some sand, trace gravel; medium plasticity, grey; cohesive, w>PL, very stiff.

COMMENTS TESTING
Drillers Estimate
{G % S % F %}

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 30.5 m
Depth to Top of Rock:

BRITISH COLUMBIA
Ministry of Transportation and Infrastructure

SUMMARY LOG

Drill Hole #: SH22-SEG 2-01

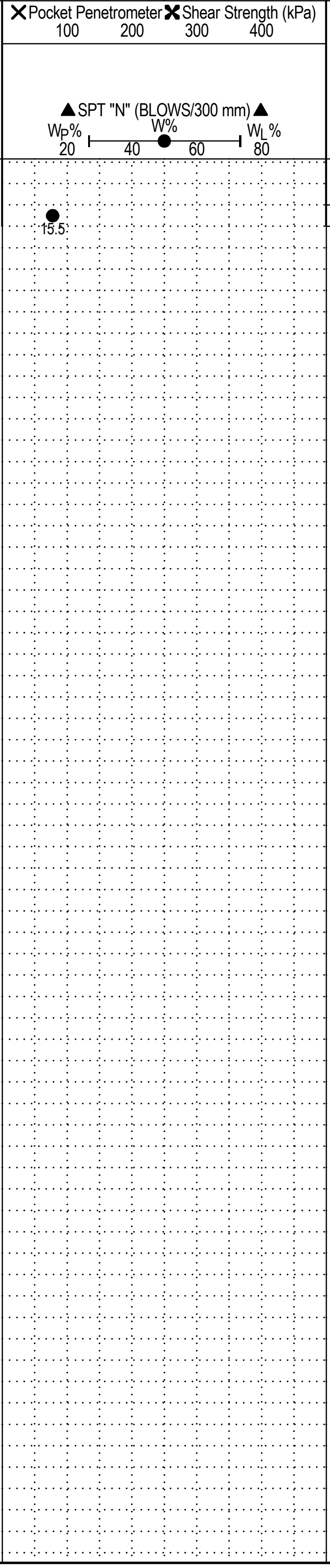
Prepared by: 32079
Thurber Engineering Ltd.

Logged by: RJT Reviewed by: ANR

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5434072 , 545198
Elevation: 91.2 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Date(s) Drilled: 2022-06-25
Company: Mud Bay
Driller: Stephen
Drill Make/Model: Terra Sonic TSCC-05
Drilling Method: Sonic

DEPTH (m)	DRILLING DETAILS	SPT	SAMPLE TYPE	SAMPLE NO	RECOVERY (%)	SOIL SYMBOL	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*	ELEVATION (m)
30			▤	21		▨	30.48m	CL		61
							End of hole at 30.5 m depth.			
31										60
32										59
33										58
34										57
35										56
36										55
37										54
38										53
39										52
40										



Legend Sample Type:
- A-Auger
- B-Becker
- C-Core
- G-Grab
- V-Vane
- L#-Lab Sample
- S-Split Spoon
- O-Odex (air rotary)
- W-Wash (mud return)
- T-Shelby Tube

Legend Installation:
- Sand
- Grout
- Cement
- Bentonite
- Drill Cuttings
- Slotted
- Slough
- Piezometer

Final Depth of Hole: 30.5 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

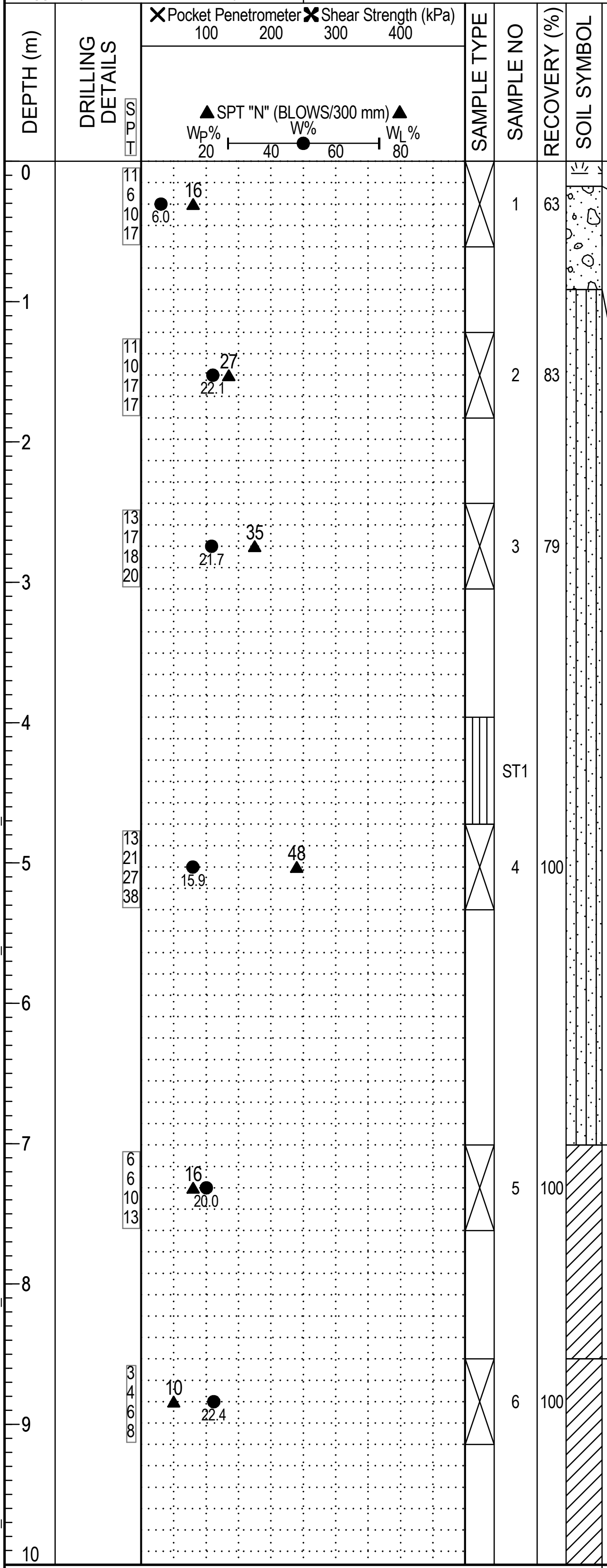
SUMMARY LOG Drill Hole #: **MRH22-SEG 2-06**

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC

Prepared by: 32079
Thurber Engineering Ltd.
Logged by: SY Reviewed by: ANR

Datum: UTM-Nad83
Northing/Easting: 5434056 , 545199
Elevation: 91.8 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Date(s) Drilled: 2022-05-25
Company: Mud Bay
Driller: Brendan
Drill Make/Model: Fraste XL -03
Drilling Method: Mud Rotary



SPT	SPT "N"	W%	SAMPLE NO	RECOVERY (%)	CLASSIFICATION
11 6 10 17	16	6.0	1	63	SP/GP
11 10 17 17	27	22.1	2	83	SM
13 17 18 20	35	21.7	3	79	SM
			ST1		
13 21 27 38	48	15.9	4	100	ML
6 6 10 13	16	20.0	5	100	CL
3 4 6 8	10	22.4	6	100	CL

SOIL DESCRIPTION

TOPSOIL.

0.18m

SP - SAND and GRAVEL, trace partially decomposed organics; fine to coarse grained, poorly graded sand; fine to coarse grained, sub-rounded, 30 mm max. size gravel, grey; non-cohesive (FILL), dry, compact.

0.91m

SM - SAND and SILT to silty; fine grained, well graded sand; low plasticity silt, grey; cohesive, w>PL, compact.

7.01m

CL - SILTY CLAY, trace to some sand; low to medium plasticity; fine grained sand, grey; cohesive, w>PL, stiff.

8.53m

CL - SILTY CLAY, trace sand; medium plasticity; fine grained sand, grey; cohesive, w>PL, stiff.

COMMENTS TESTING
Drillers Estimate
{G % S % F %}

ELEVATION (m)
91
90
89
88
87
86
85
84
83
82

Legend Sample Type:
A-Auger
B-Becker
C-Core
G-Grab
V-Vane
L#-Lab Sample
S-Split Spoon
O-Odex (air rotary)
W-Wash (mud return)
T-Shelby Tube

Legend Installation:
Sand
Grout
Cement
Bentonite
Drill Cuttings
Slotted
Slough
Piezometer

Final Depth of Hole: 30.5 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

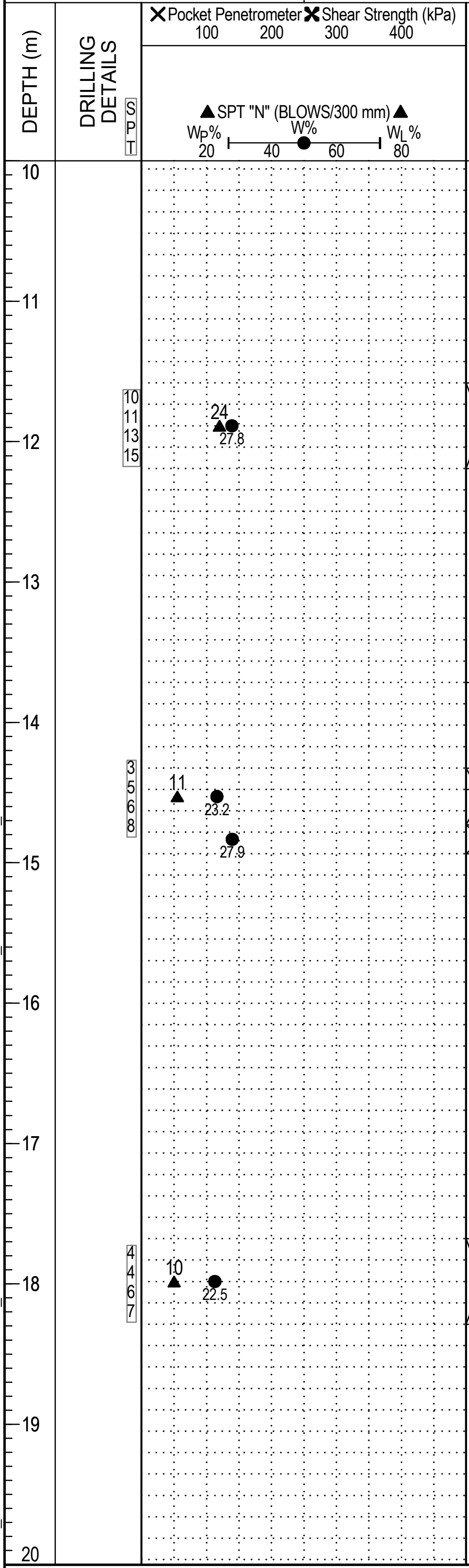
SUMMARY LOG

Drill Hole #: MRH22-SEG 2-06

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5434056 , 545199
Elevation: 91.8 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Prepared by: 32079
Thurber Engineering Ltd.
Logged by: SY
Reviewed by: ANR

Date(s) Drilled: 2022-05-25
Company: Mud Bay
Driller: Brendan
Drill Make/Model: Fraste XL -03
Drilling Method: Mud Rotary



SPT	SAMPLE TYPE	SAMPLE NO	RECOVERY (%)	CLASSIFICATION
10 11 13 15	S-Split Spoon	7	54	CL
	T-Shelby Tube	ST2		
3 5 6 8	S-Split Spoon	8	100	CL
	S-Split Spoon	9		CL
4 4 6 7	S-Split Spoon	10	100	CL

SOIL SYMBOL: ▨

SOIL DESCRIPTION

CL - SILTY CLAY, trace sand; medium plasticity; fine grained sand, grey; cohesive, w>PL, stiff. *(continued)*

COMMENTS TESTING
Drillers Estimate
{G % S % F %}

ELEVATION (m): 81, 80, 79, 78, 77, 76, 75, 74, 73, 72

Legend
Sample Type:
A-Auger
B-Becker
C-Core
G-Grab
V-Vane
L#-Lab Sample
S-Split Spoon
O-Odex (air rotary)
W-Wash (mud return)
T-Shelby Tube

Legend
Installation:
Sand
Grout
Cement
Bentonite
Drill Cuttings
Slotted
Slough
Piezometer

Final Depth of Hole: 30.5 m
Depth to Top of Rock:

British Columbia Ministry of Transportation and Infrastructure

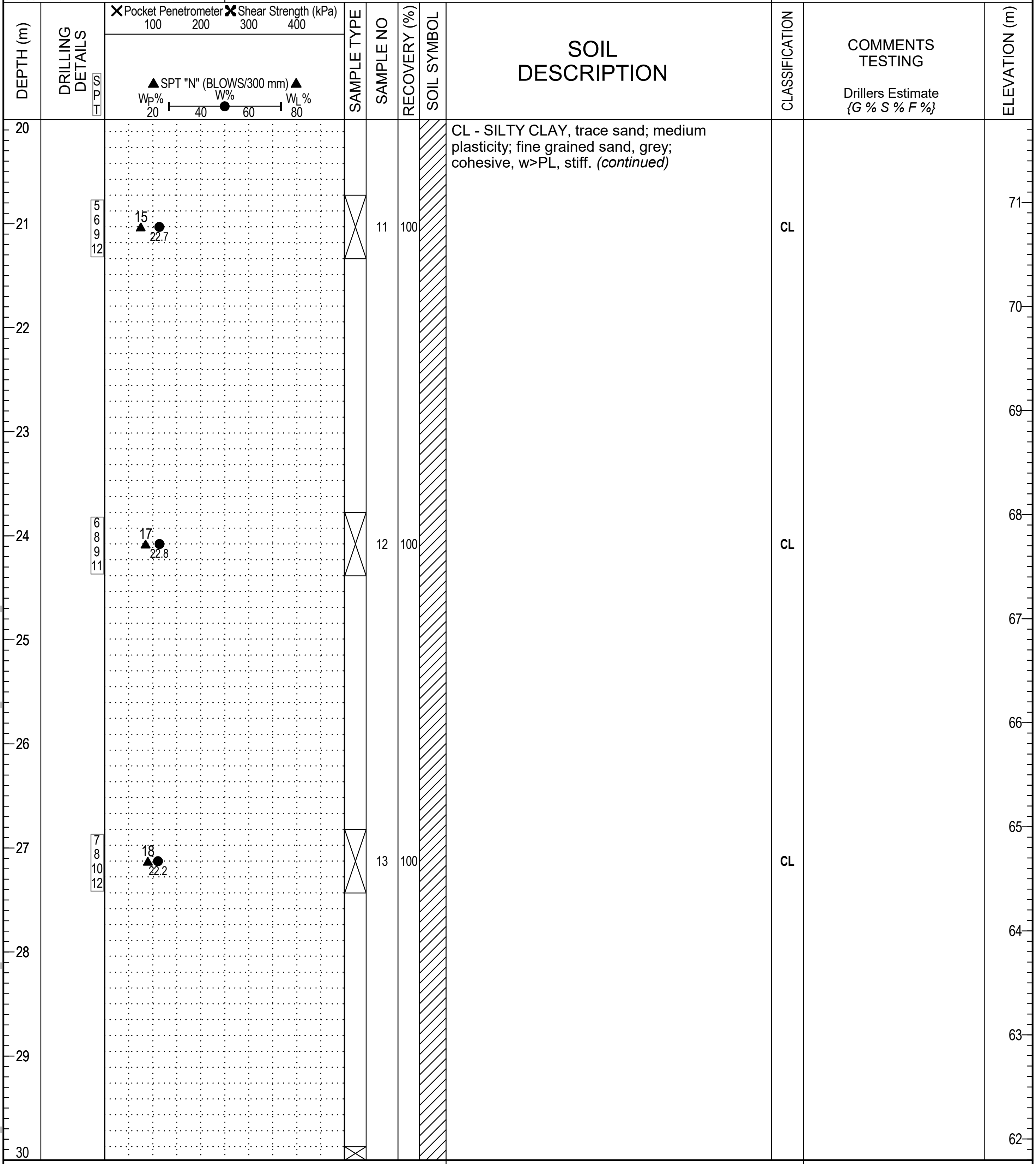
SUMMARY LOG

Drill Hole #: MRH22-SEG 2-06

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5434056 , 545199
Elevation: 91.8 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Prepared by: 32079
Thurber Engineering Ltd.
Logged by: SY Reviewed by: ANR

Date(s) Drilled: 2022-05-25
Company: Mud Bay
Driller: Brendan
Drill Make/Model: Fraste XL -03
Drilling Method: Mud Rotary



DEPTH (m)	SPT	SPT "N"	W%	SAMPLE TYPE	SAMPLE NO	RECOVERY (%)	CLASSIFICATION
21	5 6 9 12	15	22.7	☒	11	100	CL
24	6 8 9 11	17	22.8	☒	12	100	CL
27	7 8 10 12	18	22.2	☒	13	100	CL

SOIL DESCRIPTION

CL - SILTY CLAY, trace sand; medium plasticity; fine grained sand, grey; cohesive, w>PL, stiff. *(continued)*

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 30.5 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

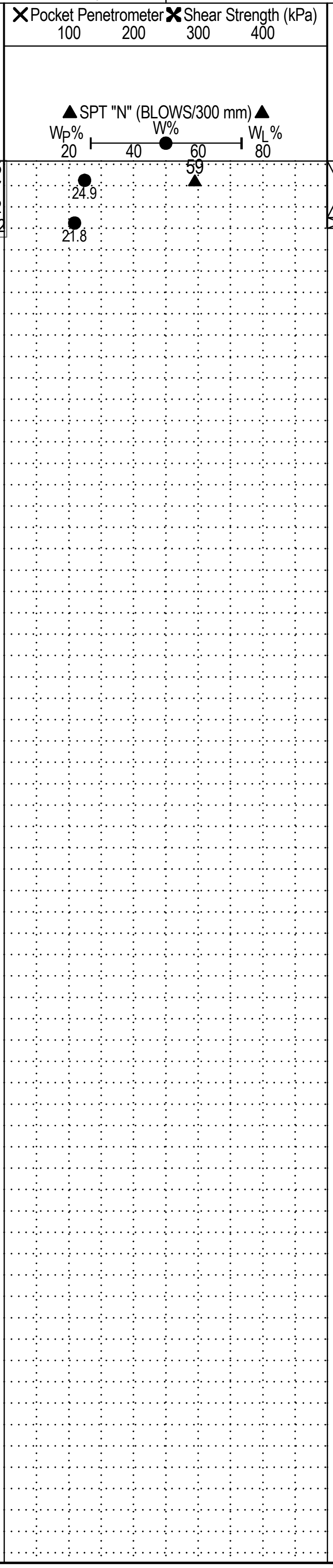
Drill Hole #: MRH22-SEG 2-06

Prepared by: 32079
Thurber Engineering Ltd.

Logged by: SY Reviewed by: ANR

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5434056 , 545199
Elevation: 91.8 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Date(s) Drilled: 2022-05-25
Company: Mud Bay
Driller: Brendan
Drill Make/Model: Fraste XL -03
Drilling Method: Mud Rotary



DEPTH (m)	DRILLING DETAILS	SPT	SAMPLE TYPE	SAMPLE NO	RECOVERY (%)	SOIL SYMBOL	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*	ELEVATION (m)
30		25 27 32	S	14	29	▨		CL		
		112	S	15			30.48m	SC/GC		
							End of hole at 30.5 m depth.			61
31										60
32										59
33										58
34										57
35										56
36										55
37										54
38										53
39										52
40										

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 30.5 m
Depth to Top of Rock:

THURBER

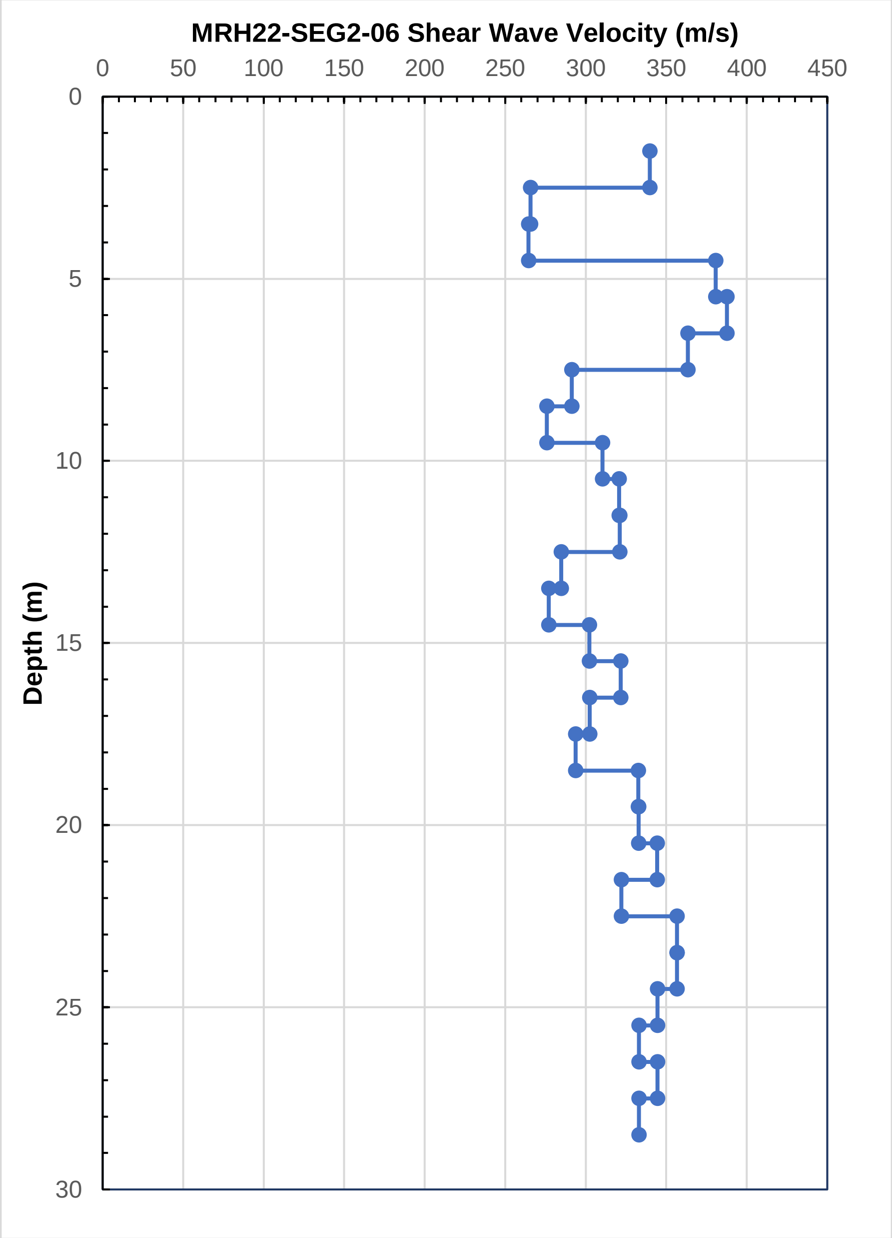
DOWNHOLE SEISMIC TEST DATA

Client:	**MoTI**	**Date:**	**31-Aug-22**
Test Hole ID	**MRH22-SEG2-06**		
Site:	**Highway 1, Abbotsford**	**Source Offset:**	**0.98 m**
Location:	**264th Street to Whatcom Road - Segment 2**	**Source:**	**Wood 2.4 m Beam**

Geophone Depth (m)	Measured Travel Time from Source (ms)	Vertical Component of Travel Time (ms)	Incremental Shear Wave Velocity (m/s)
1.50	13.5	11.3	
2.50	15.3	14.2	340
3.50	18.7	18.0	266
4.50	22.3	21.8	264
5.50	24.8	24.4	381
6.50	27.3	27.0	388
7.50	30.0	29.7	363
8.50	33.4	33.2	291
9.50	37.0	36.8	276
10.50	40.2	40.0	310
11.50	43.3	43.1	321
12.50	46.4	46.3	321
13.50	49.9	49.8	285
14.50	53.5	53.4	277
15.50	56.8	56.7	302
16.50	59.9	59.8	322
17.50	63.2	63.1	302
18.50	66.6	66.5	294
19.50	69.6	69.5	333
20.50	72.6	72.5	333
21.50	75.5	75.4	344
22.50	78.6	78.5	322
23.50	81.4	81.3	357
24.50	84.2	84.1	357
25.50	87.1	87.0	344
26.50	90.1	90.0	333
27.50	93.0	92.9	345
28.50	96.0	95.9	333

VELOCITY PROFILE

Client:	**MoTI**	**Date:**	**31-Aug-22**
Test ID:	**MRH22-SEG2-06**		
Site:	**Highway 1, Abbotsford**	**Source Offset:**	**0.98 m**
Location:	**264th Street to Whatcom Road - Segment 2**	**Source:**	**Wood 2.4 m Beam**



Shear wave velocity measurements by Thurber Engineering Ltd.

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

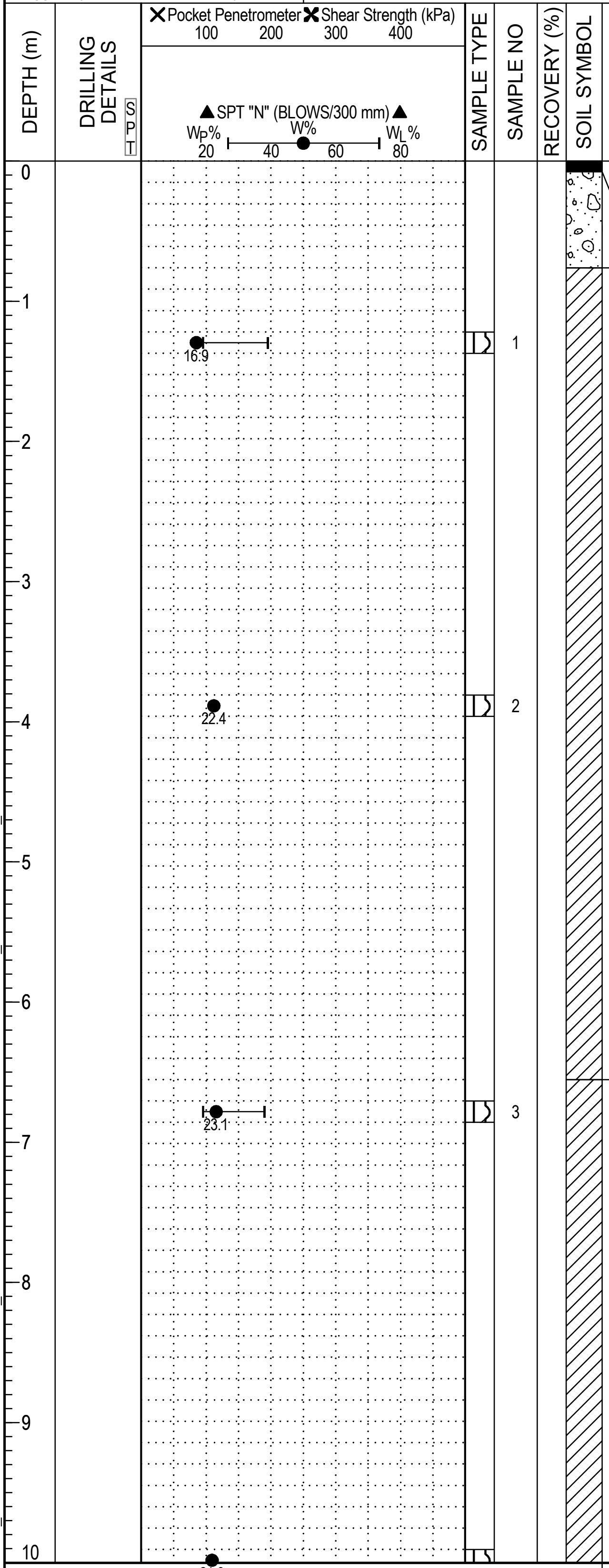
Drill Hole #: SCPT22-SEG 2-01

Prepared by: 32079
Thurber Engineering Ltd.

Logged by: SY Reviewed by: ANR

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5434030 , 545203
Elevation: 90.6 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Date(s) Drilled: 2022-04-21
Company: OnTrack
Driller: Andrew
Drill Make/Model: MPP Geoteck 60
Drilling Method: SCPT/Solid Stem Auger



SOIL DESCRIPTION	Depth	CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*
ASPHALT.	0.08m		
SP - SAND and GRAVEL; fine to coarse grained, well graded, brown (FILL); non-cohesive, moist.	0.76m		
CL - SILTY CLAY, some to trace sand, trace gravel; medium plasticity; fine to medium grained sand, grey; cohesive, w>PL, firm to stiff.		CL	**Atterberg** (Sa#1): PL:19% LL:39%
		CL	
CL - SILTY CLAY, trace sand; medium plasticity; fine grained sand, grey; cohesive, w<PL, stiff to very stiff.	6.55m	CL	**Atterberg** (Sa#3): PL:19% LL:38%

ELEVATION (m): 90, 89, 88, 87, 86, 85, 84, 83, 82, 81

Legend Sample Type: **A**-Auger **B**-Becker **C**-Core **G**-Grab **V**-Vane **L#**-Lab Sample **S**-Split Spoon **O**-Odex (air rotary) **W**-Wash (mud return) **T**-Shelby Tube

Legend Installation: Sand Grout Cement Bentonite Drill Cuttings Slotted Slough Piezometer

Final Depth of Hole: 18.3 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

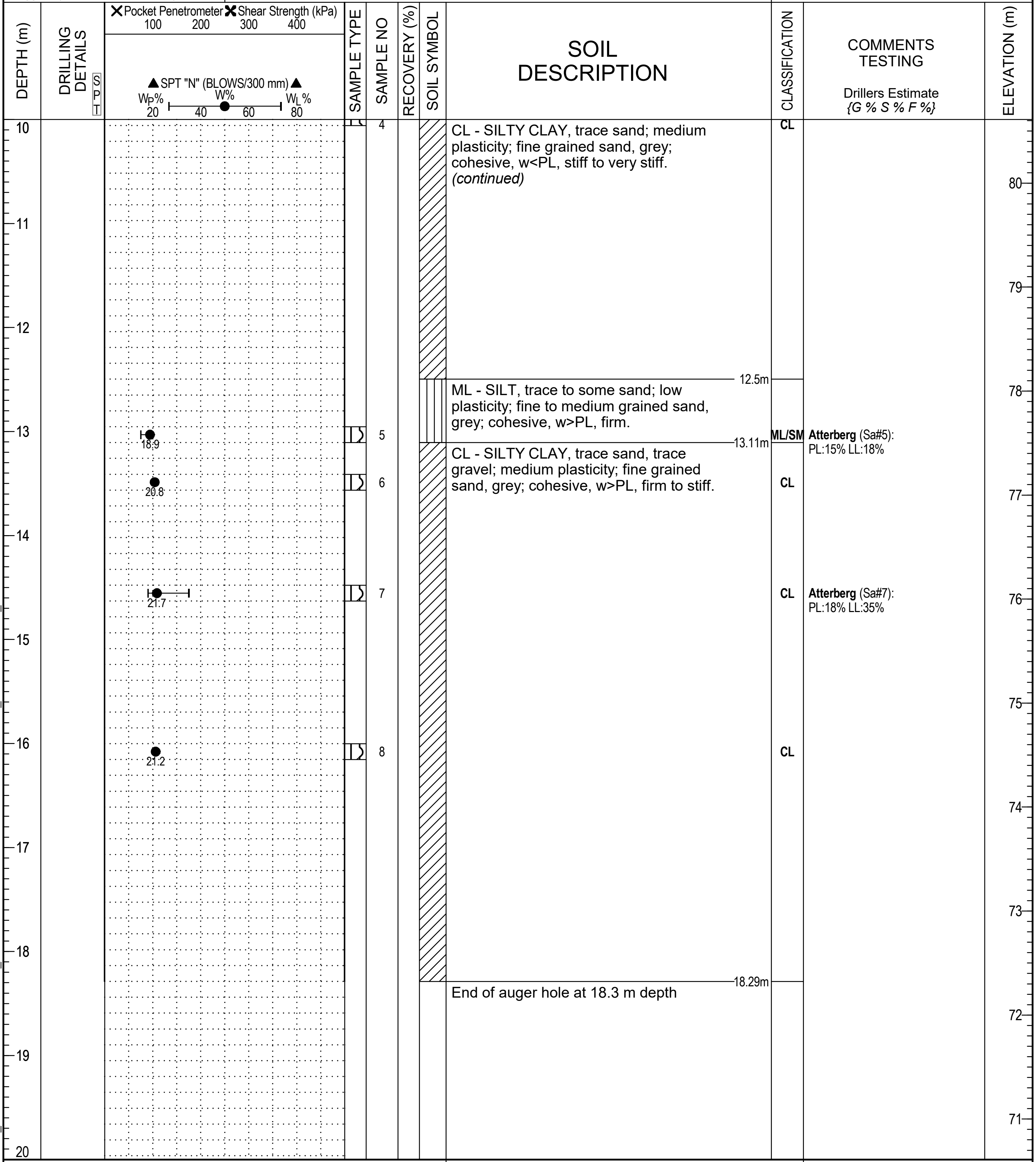
SUMMARY LOG

Drill Hole #: SCPT22-SEG 2-01

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Alignment:
Northing/Easting: 5434030 , 545203
Station/Offset:
Elevation: 90.6 m
Coordinates taken with GPS

Prepared by: 32079
Thurber Engineering Ltd.
Logged by: SY Reviewed by: ANR

Date(s) Drilled: 2022-04-21
Company: OnTrack
Driller: Andrew
Drill Make/Model: MPP Geoteck 60
Drilling Method: SCPT/Solid Stem Auger



Depth	Soil Description	Classification	Sample No	Comments / Testing (Drillers Estimate {G % S % F %})
10	CL - SILTY CLAY, trace sand; medium plasticity; fine grained sand, grey; cohesive, w<PL, stiff to very stiff. *(continued)*	CL	4	
12.5m	ML - SILT, trace to some sand; low plasticity; fine to medium grained sand, grey; cohesive, w>PL, firm.	ML/SM	5	**Atterberg** (Sa#5): PL:15% LL:18%
13.11m	CL - SILTY CLAY, trace sand, trace gravel; medium plasticity; fine grained sand, grey; cohesive, w>PL, firm to stiff.	CL	6	
		CL	7	**Atterberg** (Sa#7): PL:18% LL:35%
		CL	8	
18.29m	End of auger hole at 18.3 m depth			

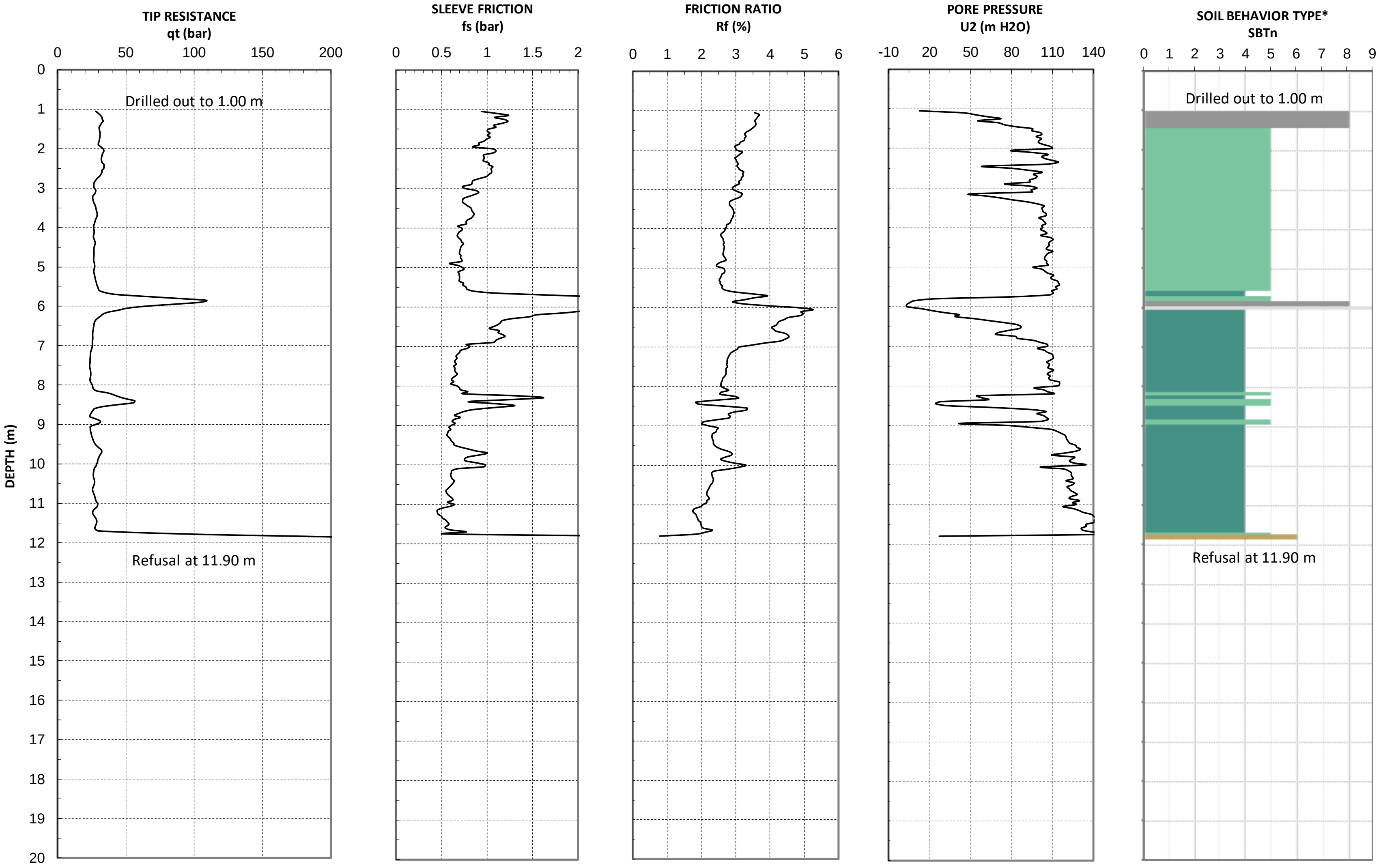
Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 18.3 m
Depth to Top of Rock:

ON TRACK
An earth DRILLING Company
SAFETY IS OUR CORE

Sounding: SCPT22-SEG 2-01	Client: Thurber Engineering Ltd.
21-Apr-2022	Project: Trans-Canada Fraser Valley - Highway 1 Improvements



* Based on Robertson et. al 1990

🟥 1. Sensitive Fine Grained
🟫 2. Organic Material
🟦 3. Clay to Silty Clay
🟩 4. Clayey Silt to Silty Clay
🟩 5. Silty Sand to Sandy Silt
🟨 6. Clean Sand to Silty Sand
🟧 7. Gravely Sand to Sand
⬛ 8. Very Stiff Sand to Clayey Sand
⬜ 9. Very Stiff Fine Grained

Depth Increment: 0.05 m
Geodetic Elevation: N/A
Maximum Depth: 11.90 m

Cone ID: DDG1522
Operator: CS

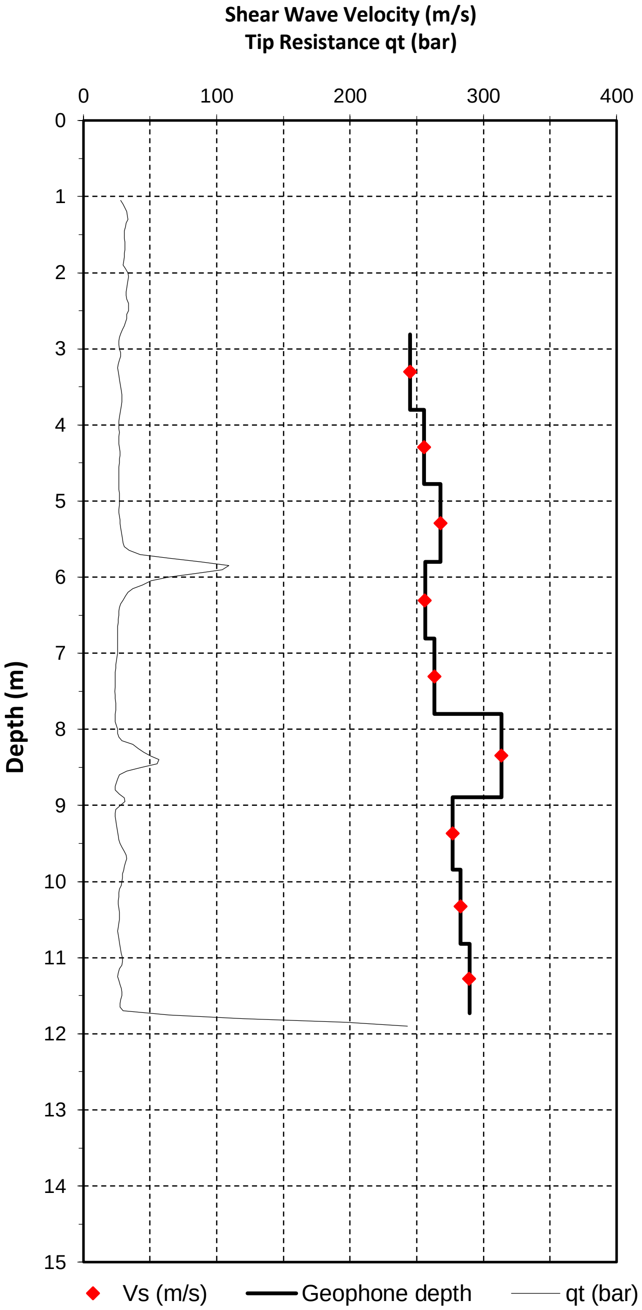
Sounding: SCPT22-SEG 2-01	Client: Thurber Engineering Ltd.
21-Apr-22	Project: Trans-Canada Fraser Valley - Highway 1 Improvements

Seismic Source: Beam
Source to cone (m): 1.2

Geodetic Elevation: N/A
Cone ID: DDG1522
Operator: CS

Shear Wave Velocity Data (Vs)

Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Time Difference (ms)	Shear Wave Velocity Vs (m/s)
3.01	2.81	3.06			
4.00	3.80	3.98	0.93	3.79	**245**
4.98	4.78	4.93	0.94	3.69	**256**
6.00	5.80	5.92	0.99	3.71	**268**
7.01	6.81	6.91	0.99	3.87	**256**
8.00	7.80	7.89	0.98	3.71	**263**
9.09	8.89	8.97	1.08	3.44	**314**
10.04	9.84	9.91	0.94	3.40	**277**
11.02	10.82	10.89	0.97	3.44	**283**
11.93	11.73	11.79	0.90	3.13	**290**



Shear Wave Velocity (m/s)
Tip Resistance qt (bar)
0
100
200
300
400
Depth (m)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Vs (m/s)
Geophone depth
qt (bar)

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

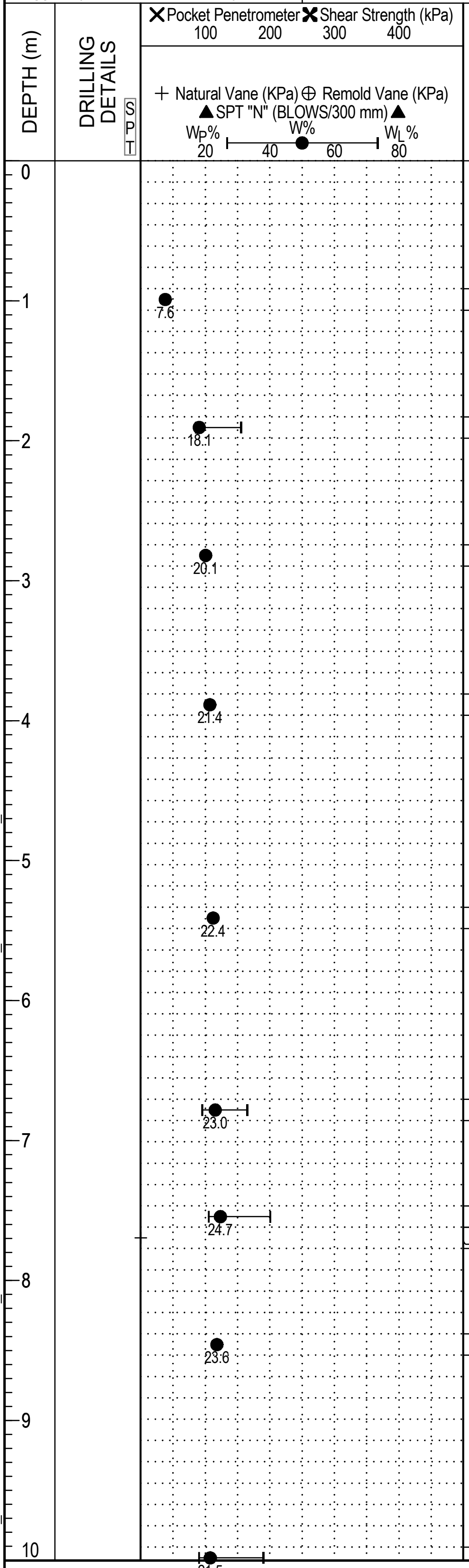
SUMMARY LOG

Drill Hole #: **CPT22-SEG 2-15**

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Alignment:
Northing/Easting: 5434086 , 545204
Station/Offset:
Elevation: 91.2 m
Coordinates taken with GPS

Prepared by: 32079
Thurber Engineering Ltd.
Logged by: RJT
Reviewed by: ANR

Date(s) Drilled: 2022-12-15
Company: OnTrack
Driller: Andrew
Drill Make/Model: MPP Geoteck 60
Drilling Method: CPT/Solid Stem Auger



SAMPLE TYPE | SAMPLE NO | RECOVERY (%) | SOIL SYMBOL | SOIL DESCRIPTION | CLASSIFICATION | COMMENTS TESTING Drillers Estimate *{G % S % F %}* | ELEVATION (m)

ASPHALT. — 0.1m

SP - SAND, gravelly, trace to some silt, brown (FILL), moist. — Sample 1 — GW-GM/SW-SM

1.37m — CL - SILTY CLAY, trace sand; none to low plasticity, brown; cohesive, w>PL, very stiff.
- Sample 2 — CL — **Atterberg** (Sa#2): PL:17% LL:31%
- Sample 3 — CL

3.05m — CL - SILTY CLAY, trace sand; medium plasticity, grey; cohesive, w>PL, very stiff to stiff.
- Sample 4 — CL
- Sample 5 — CL

5.79m — CL - SILTY CLAY, trace sand; low plastic, grey; cohesive, w>PL, stiff.
- Sample 6 — CL — **Atterberg** (Sa#6): PL:19% LL:33%

7.32m — CL - SILTY CLAY, trace sand; medium plasticity, grey; cohesive, w>PL, stiff to very stiff.
- Sample 7 — CL — **Atterberg** (Sa#7): PL:21% LL:40%
- V1
- Sample 8 — CL

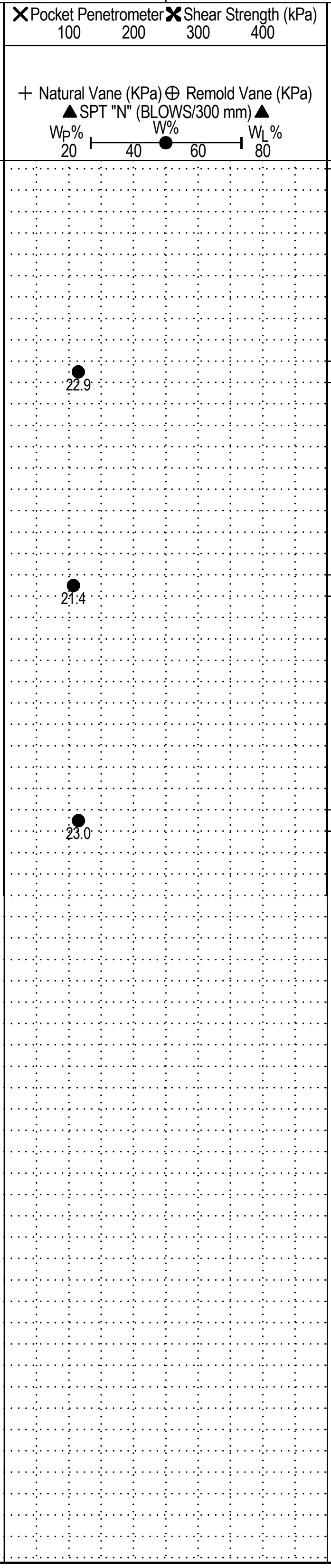
Elevation: 91, 90, 89, 88, 87, 86, 85, 84, 83, 82

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, ■ Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 15.2 m
Depth to Top of Rock:

BRITISH COLUMBIA Ministry of Transportation and Infrastructure	**SUMMARY LOG**	Drill Hole #: **CPT22-SEG 2-15**	
Prepared by: 32079 Thurber Engineering Ltd.	Project: **Fraser Valley Highway 1 Corridor Improvement** Location: Abbotsford, BC		Date(s) Drilled: 2022-12-15 Company: OnTrack
Logged by: RJT Reviewed by: ANR	Datum: UTM-Nad83 Northing/Easting: 5434086 , 545204 Elevation: 91.2 m	Alignment: Station/Offset: *Coordinates taken with GPS*	Driller: Andrew Drill Make/Model: MPP Geoteck 60 Drilling Method: CPT/Solid Stem Auger



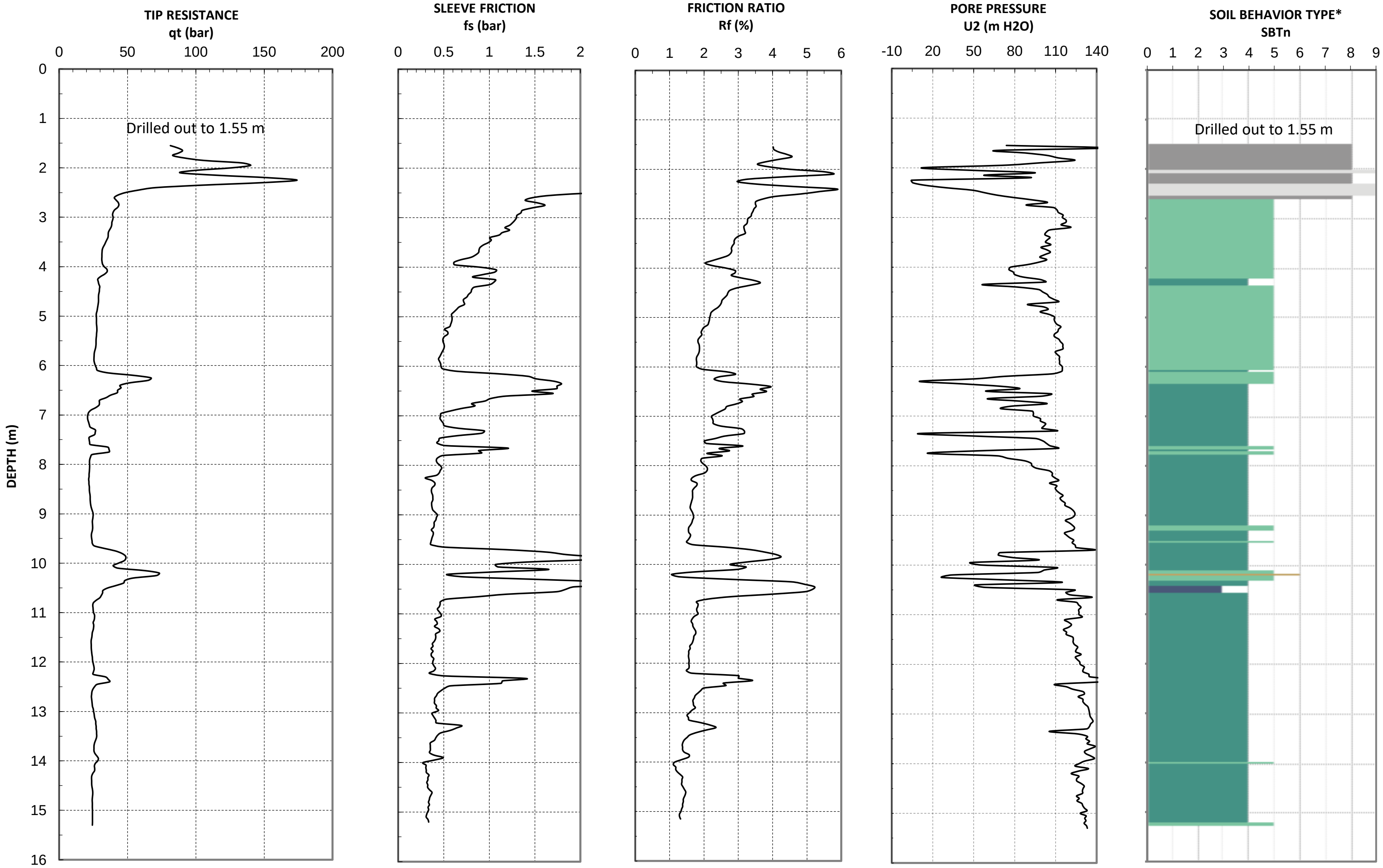
DEPTH (m)	SAMPLE TYPE	SAMPLE NO	SOIL SYMBOL	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*	ELEVATION (m)
10	A-Auger	9	▨	CL - SILTY CLAY, trace sand; medium plasticity, grey; cohesive, w>PL, stiff to very stiff. *(continued)*	CL	**Atterberg** (Sa#9): PL:18% LL:38%	81
11							80
	A-Auger	10			CL		
12							79
13	A-Auger	11			CL		78
14							77
	A-Auger	12			CL		
15				15.24m			76
				End of hole at 15.2 m depth. Hole open to 14.5 m depth. No water observed.			
16							75
17							74
18							73
19							72
20							

Legend Sample Type: A-Auger; B-Becker; C-Core; G-Grab; V-Vane; L#-Lab Sample; S-Split Spoon; O-Odex (air rotary); W-Wash (mud return); T-Shelby Tube

Legend Installation: Sand; Grout; Cement; Bentonite; Drill Cuttings; Slotted; Slough; Piezometer

Final Depth of Hole: 15.2 m
Depth to Top of Rock:

Sounding: **CPT22-SEG 2-15**	Client: Thurber Engineering Ltd.
15-Dec-2022	Project: Trans-Canada Fraser Valley - Highway 1 Improvements



* Based on Robertson et. al 1990

- 🟥 1. Sensitive Fine Grained
- 🟫 2. Organic Material
- 🟦 3. Clay to Silty Clay
- 🟩 4. Clayey Silt to Silty Clay
- 🟩 5. Silty Sand to Sandy Silt
- 🟨 6. Clean Sand to Silty Sand
- 🟧 7. Gravely Sand to Sand
- ⬛ 8. Very Stiff Sand to Clayey Sand
- ⬜ 9. Very Stiff Fine Grained

Depth Increment: 0.05 m
Geodetic Elevation: N/A
Maximum Depth: 15.30 m

Cone ID: DDG1521
Operator: RS

SUMMARY LOG

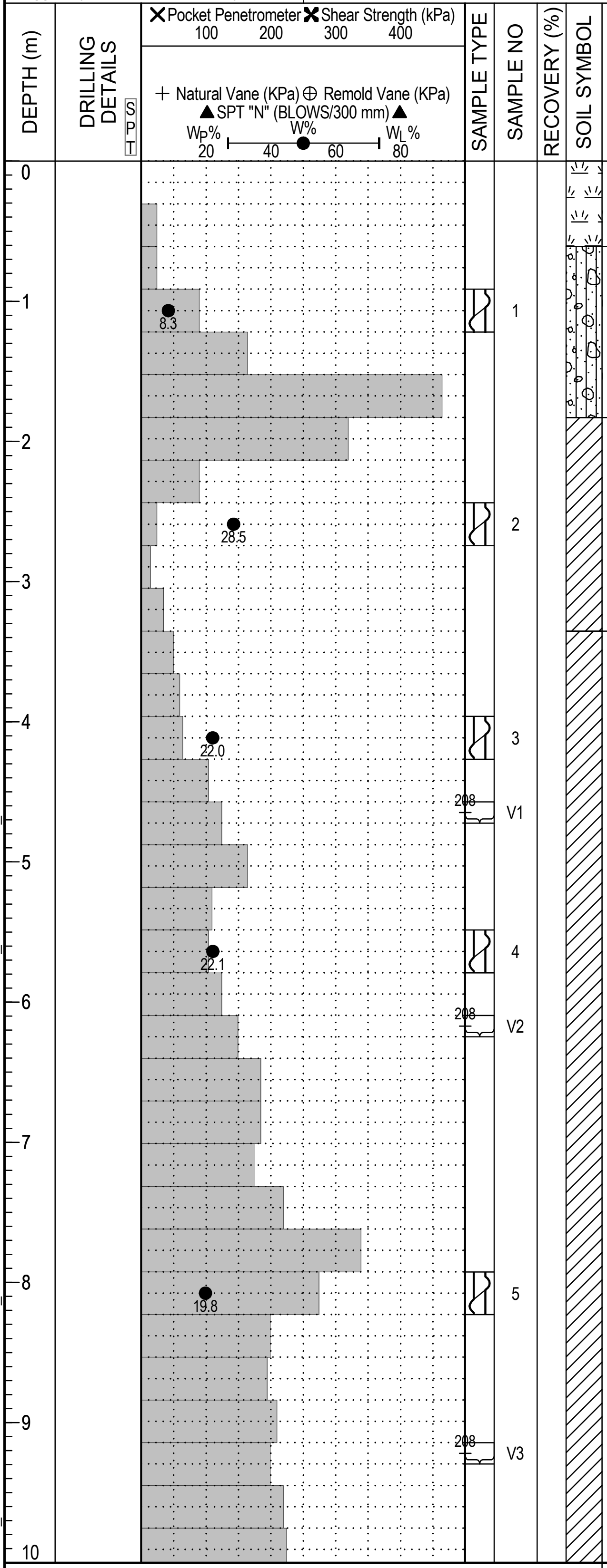
Drill Hole #: TH22-SEG 2-70

Prepared by: 32079
Thurber Engineering Ltd.

Logged by: HG Reviewed by: ANR

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5434171 , 545154
Elevation: 96.9 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Date(s) Drilled: 2022-12-12
Company: OnTrack
Driller: Andrew
Drill Make/Model: Diedrick D-120
Drilling Method: DCPT/Solid Stem Auger



SOIL DESCRIPTION	Depth	CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*
TS - SAND AND ORGANICS, some gravel, brown; non-cohesive.	0.61m		
GM/SM - SAND AND GRAVEL, silty to some silt, trace organics; sub-angular to sub-rounded, 7 mm max. size gravel; fine to medium grained sand, brown-grey; non-cohesive, moist, compact.	1.83m	GM/SM	
CL - SILTY CLAY, sandy to some sand, some gravel, trace organics; sub-angular, 19 mm max. size gravel; fine grained sand, brown-grey; cohesive, w>PL, very soft to firm.	3.35m	CL	
CL - SILTY CLAY, some to trace sand, trace organics; medium plasticity, brown-grey; cohesive, firm to very stiff.		CL CL	
- trace gravel at 8.0 m depth		CL	

ELEVATION (m): 96 95 94 93 92 91 90 89 88 87

Legend Sample Type: A-Auger | B-Becker | C-Core | G-Grab | V-Vane | L#-Lab Sample | S-Split Spoon | O-Odex (air rotary) | W-Wash (mud return) | T-Shelby Tube

Legend Installation: Sand | Grout | Cement | Bentonite | Drill Cuttings | Slotted | Slough | Piezometer

Final Depth of Hole: 19.8 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

Drill Hole #: TH22-SEG 2-70

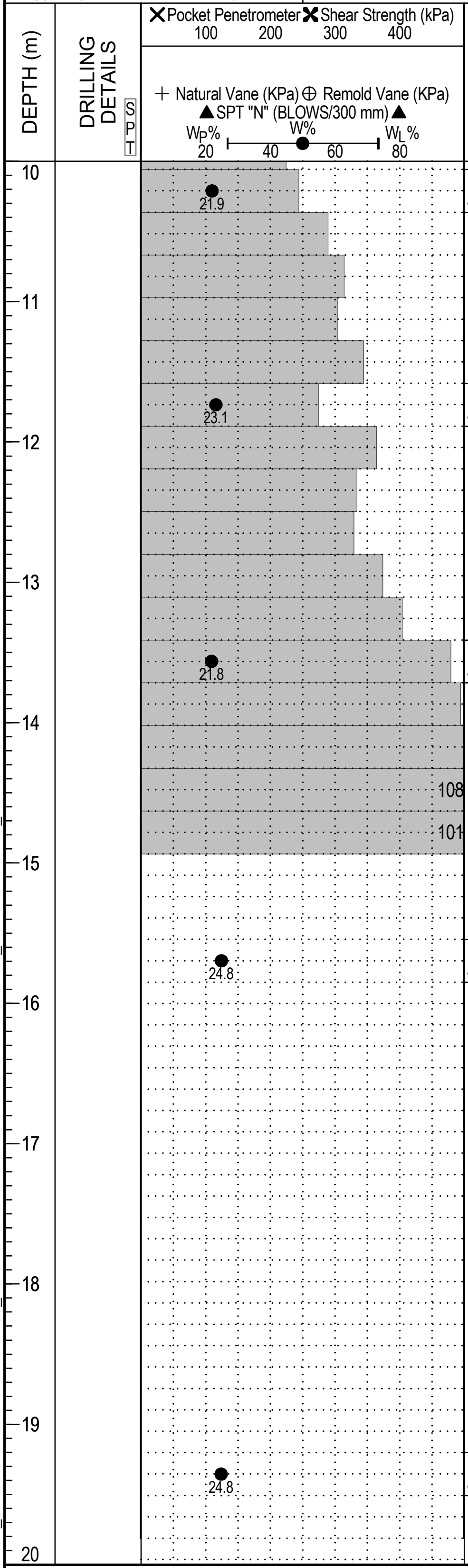
Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC

Prepared by: 32079
Thurber Engineering Ltd.

Logged by: HG   Reviewed by: ANR

Datum: UTM-Nad83
Northing/Easting: 5434171 , 545154
Elevation: 96.9 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Date(s) Drilled: 2022-12-12
Company: OnTrack
Driller: Andrew
Drill Make/Model: Diedrick D-120
Drilling Method: DCPT/Solid Stem Auger



SAMPLE TYPE	SAMPLE NO	RECOVERY (%)	SOIL SYMBOL	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*
A	6		▨	CL - SILTY CLAY, some to trace sand, trace organics; medium plasticity, brown-grey; cohesive, firm to very stiff. *(continued)*	**CL**	
A	7		▨		**CL**	
A	8		▨		**CL**	
			▨	- trace gravel at 15.0 m depth		
A	9		▨		**CL**	
A	10		▨		**CL**	
				19.81m End of hole at 19.8 m depth.		

ELEVATION (m): 86 85 84 83 82 81 80 79 78 77

Legend Sample Type: A-Auger B-Becker C-Core G-Grab V-Vane L#-Lab Sample S-Split Spoon O-Odex (air rotary) W-Wash (mud return) T-Shelby Tube

Legend Installation: Sand Grout Cement Bentonite Drill Cuttings Slotted Slough Piezometer

Final Depth of Hole: 19.8 m
Depth to Top of Rock:

BRITISH COLUMBIA Ministry of Transportation and Infrastructure

SUMMARY LOG

Drill Hole #: TH22-SEG 2-70

Prepared by: 32079
Thurber Engineering Ltd.
Logged by: HG Reviewed by: ANR

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Alignment:
Northing/Easting: 5434171 , 545154
Station/Offset:
Elevation: 96.9 m
Coordinates taken with GPS

Date(s) Drilled: 2022-12-12
Company: OnTrack
Driller: Andrew
Drill Make/Model: Diedrick D-120
Drilling Method: DCPT/Solid Stem Auger

DEPTH (m)	DRILLING DETAILS	SPT	✕ Pocket Penetrometer ✖ Shear Strength (kPa) 100 200 300 400 + Natural Vane (KPa) ⊕ Remold Vane (KPa) ▲ SPT "N" (BLOWS/300 mm) ▲ W$_P$% W% W$_L$% 20 40 60 80	SAMPLE TYPE	SAMPLE NO	RECOVERY (%)	SOIL SYMBOL	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*	ELEVATION (m)
20											
21											76
22											75
23											74
24											73
25											72
26											71
27											70
28											69
29											68
30											67

Legend Sample Type: **A**-Auger **B**-Becker **C**-Core **G**-Grab **V**-Vane **L#**-Lab Sample **S**-Split Spoon **O**-Odex (air rotary) **W**-Wash (mud return) **T**-Shelby Tube

Legend Installation: Sand Grout Cement Bentonite Drill Cuttings Slotted Slough Piezometer

Final Depth of Hole: 19.8 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

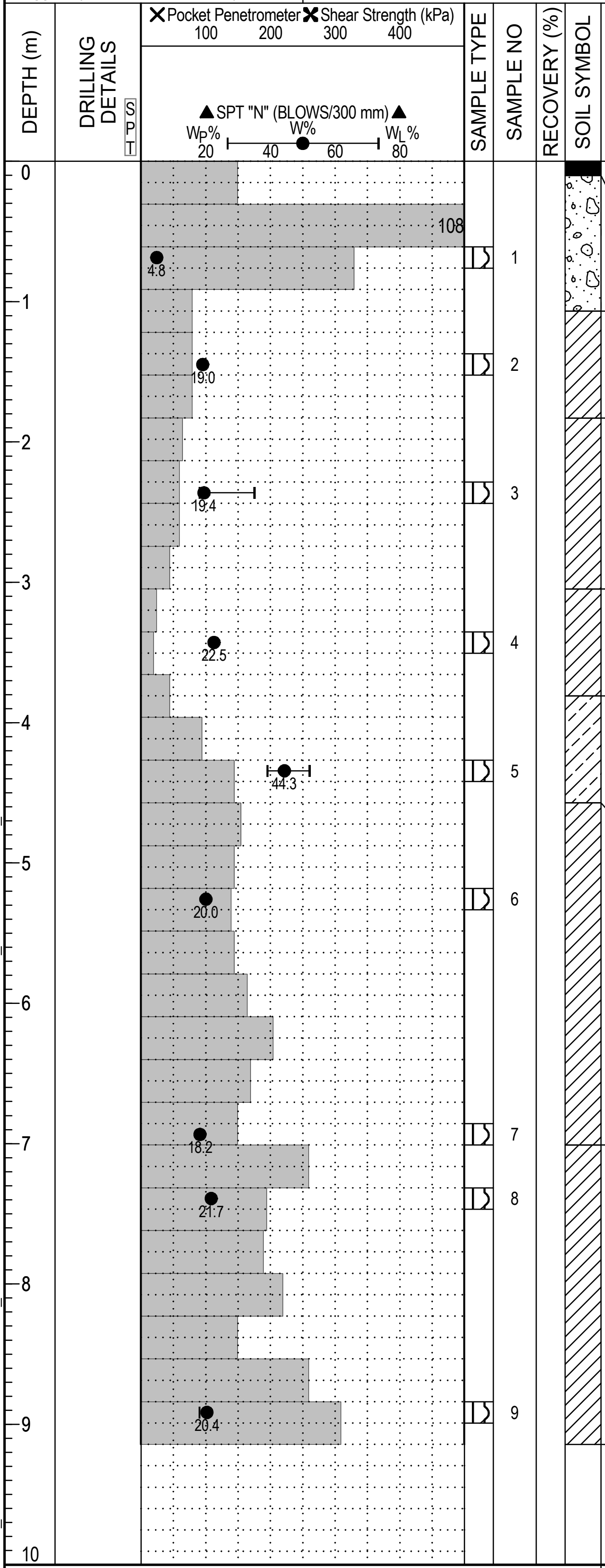
Drill Hole #: TH22-SEG 2-71

Prepared by: 32079
Thurber Engineering Ltd.

Logged by: RJT Reviewed by: ANR

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Alignment:
Northing/Easting: 5434117 , 545196
Station/Offset:
Elevation: 98.1 m
Coordinates taken with GPS

Date(s) Drilled: 2022-12-12
Company: OnTrack
Driller: Andrew
Drill Make/Model: Diedrick D-120
Drilling Method: DCPT/Solid Stem Auger



DEPTH (m)	SAMPLE NO	SOIL DESCRIPTION	CLASSIFICATION	COMMENTS TESTING Drillers Estimate {G % S % F %}
0.1m		ASPHALT.		
1.07m	1	GP - GRAVEL and SAND, trace silt, trace organics; sub-angular, 25 mm max. size gravel; fine to medium grained sand, brown (FILL); non-cohesive, moist, dense.	GP-GM/SP-SM	
1.83m	2	CL - SILTY CLAY, trace to some sand lenses, trace organics; medium plasticity, grey with oxidation and black staining; cohesive, w>PL, very stiff.	CL	
3.05m	3	CL - SILTY CLAY, trace sand lenses, trace gravel; medium plasticity, grey with black staining; cohesive, w>PL, stiff.	CL	**Atterberg** (Sa#3): PL:18% LL:35%
3.81m	4	CL - SILTY CLAY, some sand, some roots, trace gravel; medium plasticity, grey with black staining; cohesive, w>PL , stiff.	CL	
4.57m	5	OH - ORGANIC CLAY, sandy to some sand, fine to medium grained sand, brown to dark brown; cohesive, w>PL, firm. - some oxidation below 4.4 m depth	OH	**Atterberg** (Sa#5): PL:39% LL:52%
	6	CL - SILTY CLAY, sandy to trace sand, trace gravel; medium plasticity, fine grained sand, grey with trace oxidation; cohesive, w>PL, very stiff.	CL	
7.01m	7	- some sand below 6.7 m depth	CL	
	8	CL - SILTY CLAY, trace to some sand, grey with trace oxidation; cohesive, w>PL, stiff to very stiff.	CL	
		- stiff sand lenses below 7.9 m depth		
9.14m	9		CL	**Atterberg** (Sa#9): PL:18% LL:36%
		End of hole at 9.2 m depth. Hole open to 8.5 m depth. Water observed at 8.2 m depth upon completion of drilling.		

Legend Sample Type: A-Auger B-Becker C-Core G-Grab V-Vane L#-Lab Sample S-Split Spoon O-Odex (air rotary) W-Wash (mud return) T-Shelby Tube

Legend Installation: Sand Grout Cement Bentonite Drill Cuttings Slotted Slough Piezometer

Final Depth of Hole: 9.1 m
Depth to Top of Rock:

BRITISH COLUMBIA Ministry of Transportation and Infrastructure

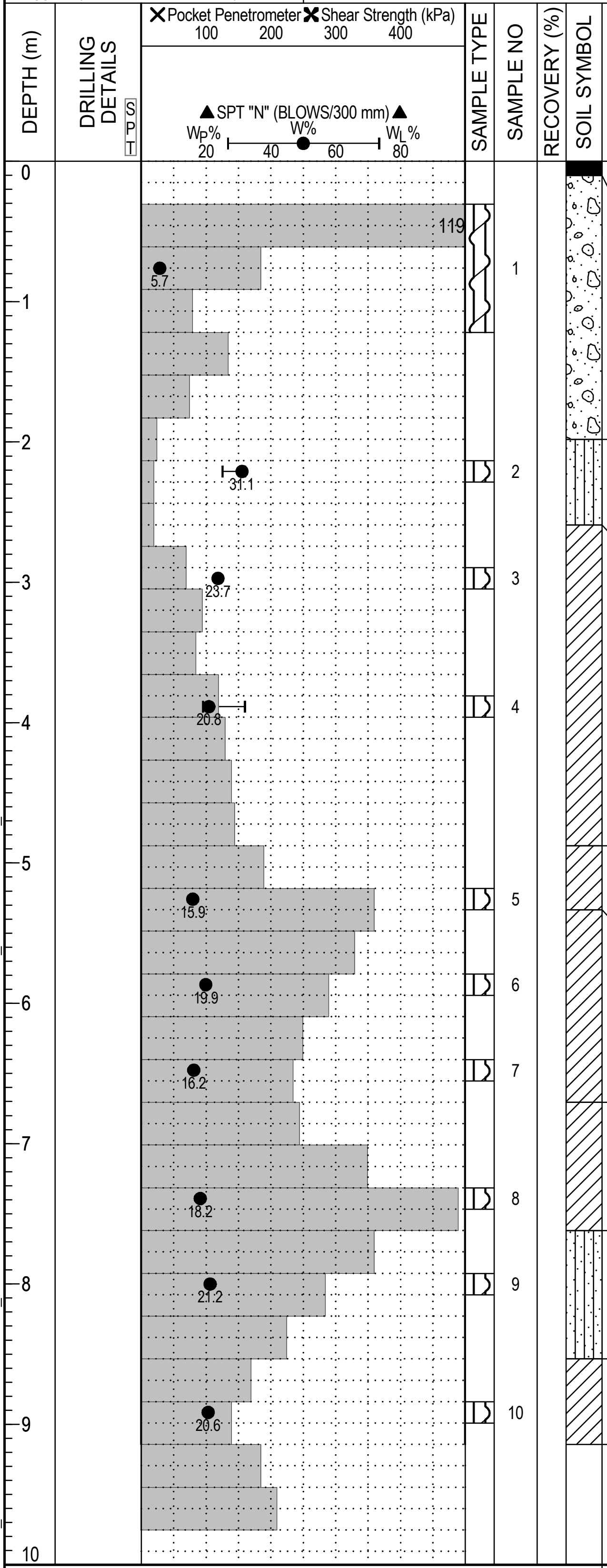
SUMMARY LOG

Drill Hole #: TH22-SEG 2-72

Prepared by: 32079
Thurber Engineering Ltd.
Logged by: RJT Reviewed by: ANR

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83
Northing/Easting: 5433993 , 545195
Elevation: 96.4 m
Alignment:
Station/Offset:
Coordinates taken with GPS

Date(s) Drilled: 2022-12-12
Company: OnTrack
Driller: Andrew
Drill Make/Model: Diedrick D-120
Drilling Method: DCPT/Solid Stem Auger



SOIL DESCRIPTION		CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*
ASPHALT.	0.1m		
GM - GRAVEL and SAND, some silt, trace organics; sub-angular, 25 mm max. size gravel, fine to medium grained sand, brown (FILL); non-cohesive, moist, compact.	1.98m	GM/SM	
ML - SILT and SAND, trace organics; coarse grained sand, brown with trace oxidation; non-cohesive, moist, very soft to soft.	2.59m	SM/ML	**Atterberg** (Sa#2): PL:25% LL:32%
CL - SILTY CLAY, trace sand; medium plasticity, grey with trace oxidation; cohesive, w>PL, firm to stiff.	4.88m	CL CL	**Atterberg** (Sa#4): PL:19% LL:32%
CL - SILTY CLAY, sandy to some sand; low plasticity, brown; cohesive, w>PL, stiff.	5.33m	CL	
CL - SILTY CLAY, some sand lenses, some to trace organics; medium plasticity, brown; cohesive, w>PL, very stiff to hard.	6.71m	CL CL	**Atterberg** (Sa#6): PL:19% LL:34%
CL - SILTY CLAY, sandy to some sand; medium plasticity, fine grained sand, grey; cohesive, w>PL, stiff to hard.	7.62m	CL	
ML - SILT and SAND; fine grained sand, grey; cohesive, w>PL, compact.	8.53m	ML/SM	
CL - SILTY CLAY, trace sand; medium plasticity, grey; cohesive, w>PL, very stiff.	9.14m	CL	
End of hole at 9.2 m depth. Hole open to 7.9 m depth. Water observed at 7.5 m depth upon completion of drilling.			

ELEVATION (m): 96, 95, 94, 93, 92, 91, 90, 89, 88, 87

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, Bentonite, Drill Cuttings, Slotted, Slough, Piezometer

Final Depth of Hole: 9.1 m
Depth to Top of Rock:

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

SUMMARY LOG

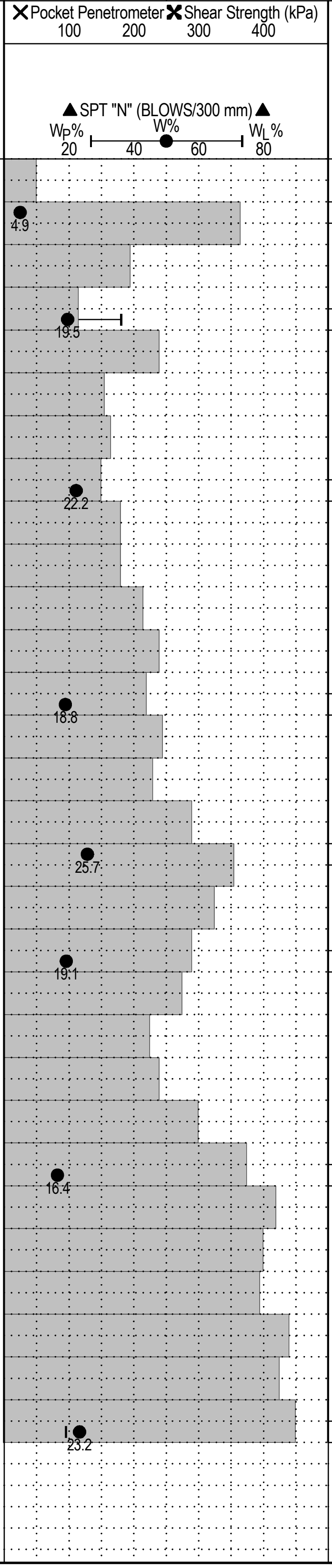
Drill Hole #: TH22-SEG 2-73

Prepared by: 32079
Thurber Engineering Ltd.

Logged by: RJT Reviewed by: ANR

Project: **Fraser Valley Highway 1 Corridor Improvement**
Location: Abbotsford, BC
Datum: UTM-Nad83 Alignment:
Northing/Easting: 5434019 , 545200 Station/Offset:
Elevation: 90.5 m *Coordinates taken with GPS*

Date(s) Drilled: 2022-12-12
Company: OnTrack
Driller: Andrew
Drill Make/Model: Diedrick D-120
Drilling Method: DCPT/Solid Stem Auger



DEPTH (m)	SAMPLE TYPE	SAMPLE NO	SOIL DESCRIPTION		CLASSIFICATION	COMMENTS TESTING Drillers Estimate *{G % S % F %}*	ELEVATION (m)
0			ASPHALT.	0.08m			
	A	1	SP - SAND, gravelly, trace silt; sub-rounded to sub-angular, 25 mm max. size gravel, fine to medium grained sand, brown (FILL); non-cohesive, moist, dense.	0.61m	GP-GM/SP-SM		90
1	A	2	CL - SILTY CLAY, trace sand, trace gravel; low plasticity, grey; cohesive, w>PL, very stiff.		CL	**Atterberg** (Sa#2): PL:19% LL:36%	89
2	A	3			CL		88
3							87
4	A	4		4.42m	CL		86
5	A	5	SM - SAND and SILT; fine grained sand, grey; non-cohesive, moist to wet, compact.	5.18m	SM/ML		85
	A	6	CL - SILTY CLAY, trace sand; medium plasticity, grey; cohesive, w>PL, very stiff.		CL		
6			- trace to some sand lenses between 6.1 and 8.5 m depth				84
7	A	7			CL		83
8							82
9	A	8		9.14m	CL	**Atterberg** (Sa#8): PL:19% LL:37%	81
10			End of hole at 9.2 m depth. Hole open to 4.6 m depth. Water observed at 9.2 m depth upon completion of drilling.				

Legend Sample Type: A-Auger, B-Becker, C-Core, G-Grab, V-Vane, L#-Lab Sample, S-Split Spoon, O-Odex (air rotary), W-Wash (mud return), T-Shelby Tube

Legend Installation: Sand, Grout, Cement, ■ Bentonite, ▨ Drill Cuttings, Slotted, Slough, Piezometer

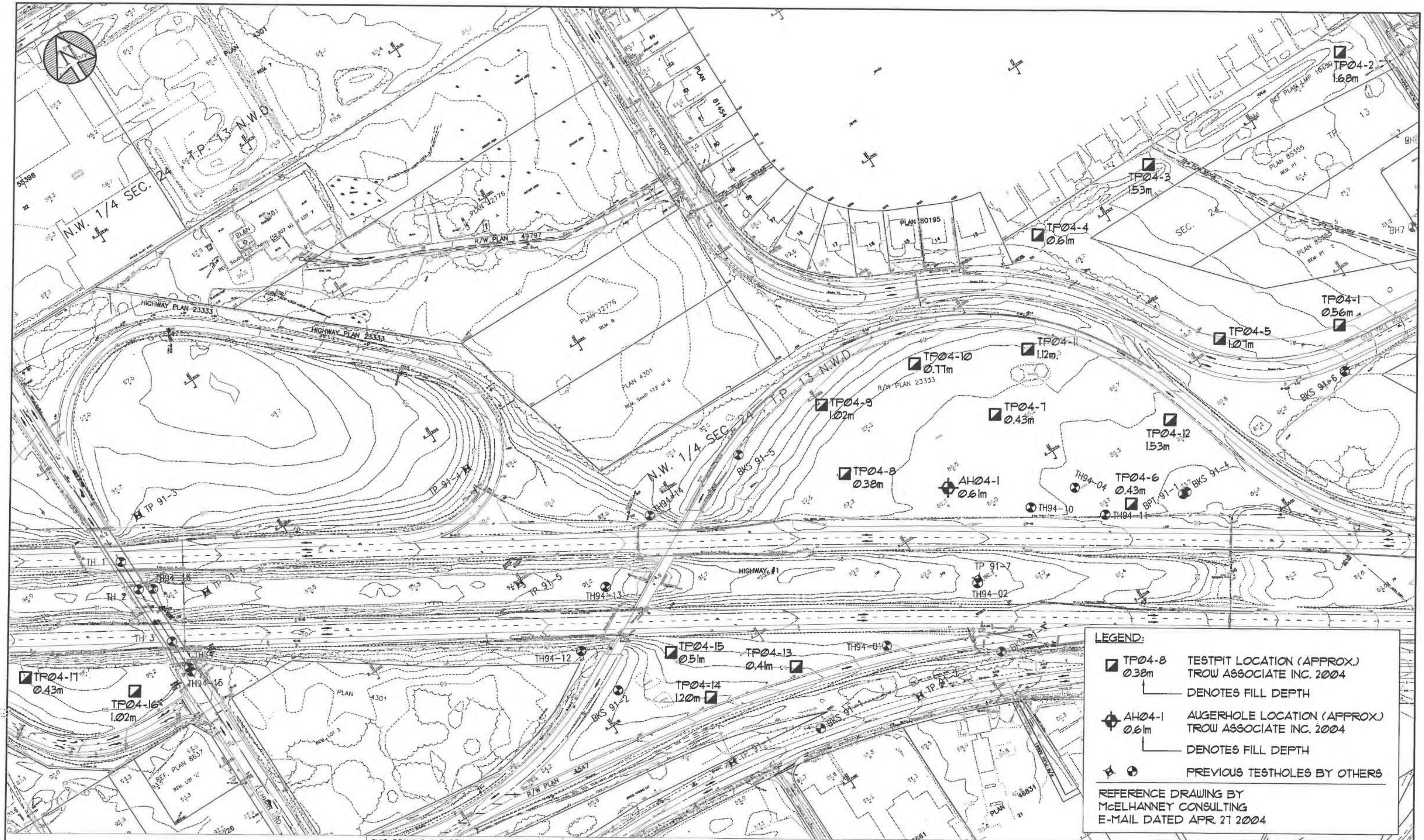
Final Depth of Hole: 9.1 m
Depth to Top of Rock:

THURBER

APPENDIX A

APPENDIX B

Historic Investigation Information by Others



LEGEND:

◪ TP04-8 0.38m	TESTPIT LOCATION (APPROX.) TROW ASSOCIATE INC. 2004
└──	DENOTES FILL DEPTH
⊕ AH04-1 0.61m	AUGERHOLE LOCATION (APPROX.) TROW ASSOCIATE INC. 2004
└──	DENOTES FILL DEPTH
⊠ ◐	PREVIOUS TESTHOLES BY OTHERS

REFERENCE DRAWING BY
McELHANNEY CONSULTING
E-MAIL DATED APR. 27 2004

TROW ASSOCIATES INC.
7025 Greenwood Street, Burnaby,
British Columbia, V5A 1X7
Telephone: 604-874-1245
Fax: 604-874-2358

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REVISIONS

No.	DESCRIPTION	DATE

CLIENT: McELHANNEY CONSULTING
PROJECT: MOUNT LEHMAN INTERCHANGE
HIGHWAY #1 & MOUNT LEHMAN, ABBOTSFORD, B.C
PROJECT NO.: 041-01211
DFTR.: S.Y.
DSGN.: D.B.H.
CHK.:
TITLE: TESTHOLE LOCATION PLAN
DATE: JUNE 2004
SCALE: 1:2000
DWG NO.: 041-01211-01

BRITISH COLUMBIA Ministry of Transportation and Highways

SUMMARY LOG

Geotechnical and Materials Engineering

TEST HOLE No. **TH94-15**✓

Project: HWY 1: MOUNT LEHMAN INTERCHANGE PROJECT

Location: SOUTH SIDE OF MT. LEHMAN OP & HWY 1 EB

Elevation: -

Driller: M. CHOQUETTE

Method: B53

Dates: 1994-06-06

Drilling Details	Depth (m)	Sample Type	Blowcount	Recovery (m)	Shear Strength (kPa)	Gradation % Gravel	Gradation % Sand	Gradation % Fines	Index Properties w_L	Index Properties w_P	Index Properties w	Classification	Description	Other Tests
H RODS	0													
	1												(Dense, brown granular fill material)	
		S	31	-										
	2													
													2.13 m	
	3	S	38	0.51		3	7	90			22	CL		
	4													
		S	49	0.30		-	10	90			21	CL		
	5													
	6	S	72	0.51		-	30	70			17	CL		
	7													
		S	68	0.46		-	5	95			18	CL		
	8													
	9	S	52	0.61		-	5	95			17	CL	Brown to grey CLAY, trace sand, trace gravel, low to trace organics, +P.L., L.L. Sand seams (Hard/very stiff)	
	10													
		S	41	0.46		-	2	98			23	CL		
	11													
	12	S	36	0.61		1	2	97			20	CL		
	13													
		S	24	0.61		1	1	98			22	CL		
	14													
	15	S	24	0.61		1	1	98			20	CL		
													15.54 m	
	16												15.54 m END OF HOLE	
	17													
	18													

SAMPLE TYPE
- A - Auger
- C - Core
- D - Denison
- S - Split Spoon
- T - Shelby Tube
- W - Wash
- Bag - Grab
- Blk - Block

SHEAR STRENGTH kPa
- U - Unconfined Compression
- Fv - Field Vane
- Lv - Lab Vane
- R - Remoulded
- Tv - Torvane
- Pp - Pocket Penetrometer
- Tr - Residual

TESTS
- M - Mechanical Analysis
- Q,R,S - Triaxial Compression
- C - Consolidation
- DS - Direct Shear
- W_L, W_P - Liquid, Plastic Limits
- W - Moisture Content

Blowcount -- Standard Penetration Test (ASTM-1586)

NOTE:

PARENTHESIS () DENOTE DRILLER'S ESTIMATE

FILE No.
-

PREPARED By:
S. TOMLINSON

SHEET 1 OF 1

BRITISH COLUMBIA
Ministry of Transportation and Highways

SUMMARY LOG

Geotechnical and Materials Engineering

TEST HOLE No. **TH94-16** ✓

Project: HWY 1: MOUNT LEHMAN INTERCHANGE PROJECT
Location: MEDIAN WB EAST OF MT. LEHMAN SOUTH ABUTMENT
Elevation: -
Driller: M. CHOQUETTE
Method: VERSA DRILL
Dates: 1994-06-08/09

Drilling Details	Depth (m)	Sample Type	Blowcount	Recovery (m)	Shear Strength (kPa)	Gradation % Gravel	Gradation % Sand	Gradation % Fines	Index Properties w_L	Index Properties w_P	Index Properties w	Classification	Description	Other Tests
	0												(Stiff, brown granular material with cobbles)	
	1												1.07 m	
		S	10	0.46		1	28	71			27	ML		
	2													
1994-06-09	3	S	19	0.61		-	30	70			20	ML	Brown, sandy SILT to SILT, some sand, trace gravel, trace organics, +P.L., -L. L. Clay lumps. (Stiff to hard)	
	4													
		S	37	0.46		-	18	82			18	ML		
	5												5.33 m	
	6	S	19	0.46		-	9	82			20	CL		
	7													
H RODS		S	16	0.46		1	7	92			22	CL		
	8													
	9	S	17	0.46		-	6	94			21	CL		
	10												Grey CLAY, trace sand, trace gravel, trace organics, +P.L., -L.L. Small sand seams. (Very stiff/stiff, trace rock)	
		S	13	0.61		-	2	98			22	CL		
	11													
	12	S	14	0.61		1	4	95			23	CL		
	13													
		S	13	0.61		1	7	92			22	CL		
	14													
	15	S	13	0.61		-	2	98			23	CL	15.54 m	
	16												15.54 m END OF HOLE	
	17													
	18													

SAMPLE TYPE
A - Auger
C - Core
D - Denison
S - Split Spoon
T - Shelby Tube
W - Wash
Bag - Grab
Blk - Block

SHEAR STRENGTH kPa
U - Unconfined Compression
Fv - Field Vane
Lv - Lab Vane
R - Remoulded
Tv - Torvane
Pp - Pocket Penetrometer
Tr - Residual

TESTS
M - Mechanical Analysis
Q,R,S - Triaxial Compression
C - Consolidation
DS - Direct Shear
W_L, W_P - Liquid, Plastic Limits
W - Moisture Content

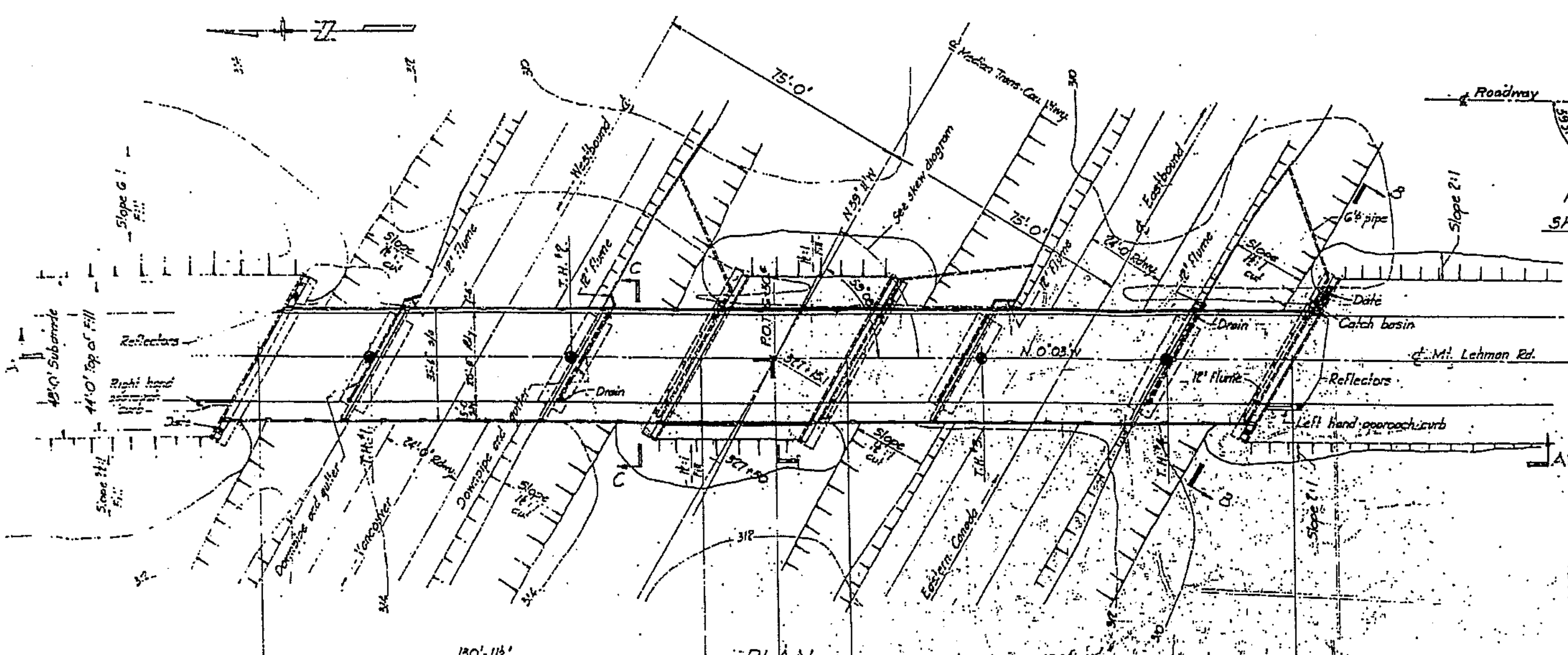
Blowcount -- Standard Penetration Test (ASTM-1586)

NOTE:
PARENTHESIS () DENOTE DRILLER'S ESTIMATE

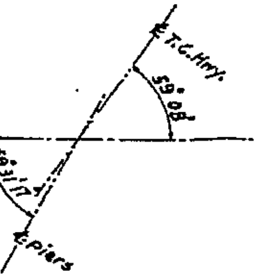
FILE No.
-

PREPARED By:
S. TOMLINSON

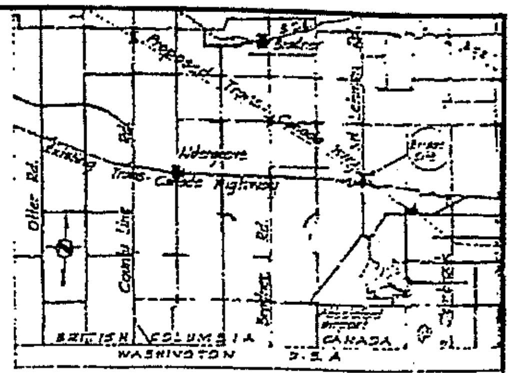
SHEET 1 OF 1



PLAN



SKEW DIAGRAM

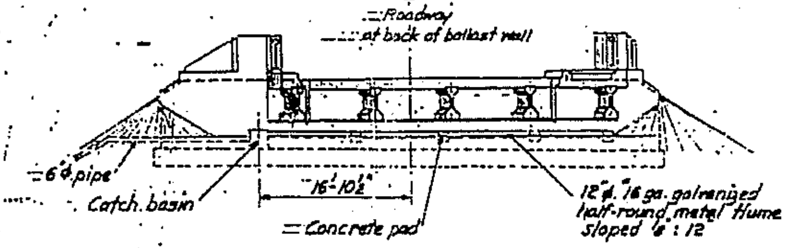


KEY MAP

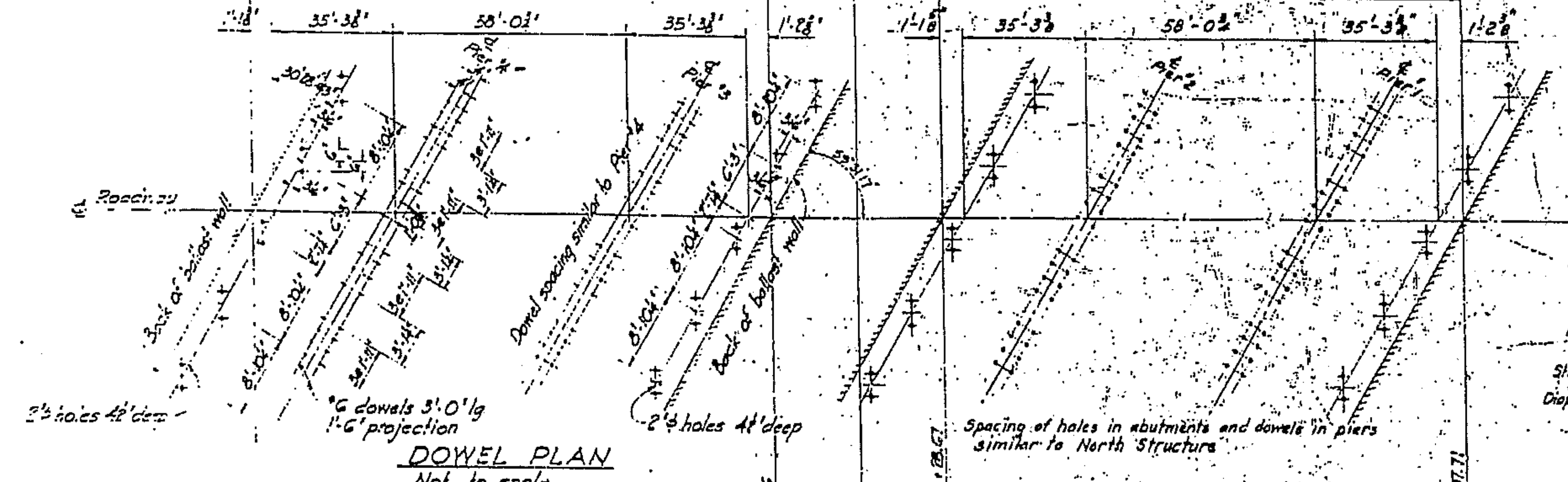
Scale - 1 inch - 2 miles

TEST HOLE DATA

ELEVATION	Nº1 MATERIAL	Nº1 PEN.	Nº2 MATERIAL	Nº2 PEN.	Nº3 MATERIAL	Nº3 PEN.	Nº4 MATERIAL	Nº4 PEN.
300			Sandy Silt	120				
290	Sandy Silt	107					Sandy Silt Silty Sand	23 12
280			Clayey Silt	73				
270	Clayey Sandy Silt	61			Clayey Silt	71	Clayey Silt	13
260			Silty Sand	55				
250								
240			Clayey Silt	71				
250								

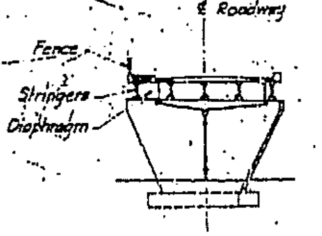


SECTION B-B



DOWEL PLAN

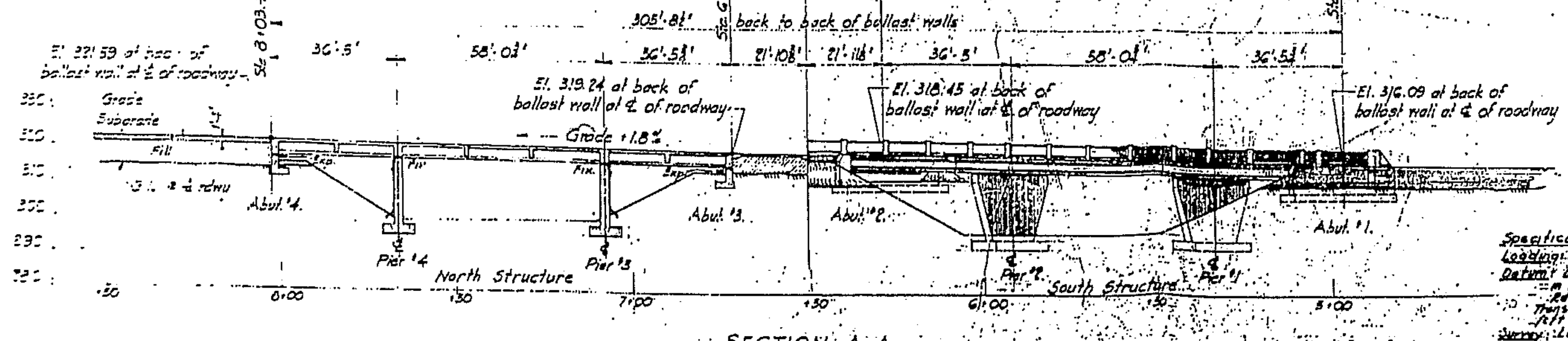
Not to scale



SECTION C-C

LIST OF DRAWINGS	
DRWG. Nº	TITLE
1362 - 3	Layout, Dowel Plan and Key Map
-4	Piers and Abutments
-5	Prestressed Concrete Stringers
-6	Gutters and Downpipes
-7	Deck Details
Misc. ~~587~~-7E	Standard Roadway Fence
SK 235.A-1	Approach Curb
SK 236 A-1A	Catch Basin

1310



SECTION A-A

NOTES

Specifications: Department of Highways

Loading: Live load H20-S16

Datum: Geodetic; L.B.M. 60d spike in 2nd power pole on Mt. Lehman Rd. from intersection of existing Trans-Canada Highway, 380 ft. left of Sta. 316+12 Elev. 307.67

Survey: Location branch

CHILLIWACK DISTRICT
TRANS-CANADA HIGHWAY MILE 107.40
MT. LEHMAN UNDERPASS
LAYOUT, DOWEL PLAN AND KEY MAP
SCALE: 1" - 20'-0"

REVISIONS			
No.	Particulars	Init.	Date

GOVT. of BRITISH COLUMBIA
DEPT. of HIGHWAYS
BRIDGE ENGINEER'S OFFICE

	Init.	Date	DRAWING NO.
Made by	M.C.M.	Mar. 57	
Checked by	K.W.	Dec. 57	

TEST HOLE LOG

T.B. 62

Test Hole # 1

Type of Equipment:

Project: Mt. Lehman Rd Underpass

Elevation: 314

Date: February 9 - 11, 1959

Reason: Bridge Foundation

☒ Undisturbed Sample

▨ Disturbed Sample

311 Water Table

N - Standard Penetration Test
140# Hammer - 30" drop.

Depth (ft)	Plot	Remarks		N	Gradation %				% L.L.	% P.L.	% P.I.	Fiel M.C.
					Clay	Silt	Sand	Gravel				
0												
3	GM	Gravel										
5'-6"	ML ▨			119	19	45	33	3	27.8	16.3	11.5	20.0
14'-0"	ML ▨		Brown Grey	81	26	55	19	-	35	17.0	18.0	14.8 14.2
		Sandy										
20'-0"	ML ▨			159	21	46	33	-	26.9	15.5	11.4	13.9
		Silt										
26'-6"	ML ☒▨			99								22.2
31'-2"	ML ☒▨			63								21.2
36'-6"	ML-CL ☒▨			53	28	64	18	-	32.4	15.8	16.6	16.4
41'-0"	ML ☒▨			63								26.9
		Clayey Sandy Silt										
51'-0"	ML ☒▨			66								19.0 20.6

TEST HOLE LOG

Test Hole No. 2 Type of Equipment Project Mt. Lehman Rd Underpass

Elevation 313 + Date of Drilling Jan. 20-28, 1959

☒ Undisturbed Sample

■ Disturbed Sample

299 ▲ Water Table

N - Standard Penetration Test 140# hammer - 30" dro

Depth (ft)	Plot	Remarks	N	Gradation % Clay	Silt	Sand	Gravel	L.L.	P.L.	P.I.	Fiel M.C.
3	GM	Gravel									
7'-0"	ML ■		73	16	45	37	2	25.2	16.4	8.8	18.2
10'-0"	SM ■	Sandy Silt	75								22.4
14'-0"	ML ■		100	20	45	35	-	25.1	15.1	10.0	18.8
19'-0"	SM ■	Brown / Grey	160								17.8
25'-0"	ML-CL ■		65								24.2
31'-6"	ML-CL ☒■		65	31	51	18	-	34.4	16.4	18.0	21.7 21.5
36'-6"	ML-CL ☒■	Clayey Silt	69								22.6
41'-0"	ML-CL ☒■		64	28	51	21	-	30.6	16.0	14.6	23.3 23.2
46'-0"	ML-CL ☒■		134								24.9
50'-6"	SM ☒■		95								18.5
56'-2"	SM ☒■	Silty Sand	63	8	32	60	-	-	-	-	19.2 21.7
61'-0"	SM ■		137								23.2
66'-6"	MI-CL ■		73								21.7
71'-6"	ML-CL ■	Clayey Silt	98	25	52	23		34.5	16.5	18.0	20.5
81'-6"	ML-CL ■		73								22.2

TEST HOLE LOG

Test Hole # 3

Type of Equipment:

Project: Mt. Lehman Rd Underpass

Elevation: 312 +

Date: February 4 - 6, 1959

Reason: Bridge Foundations

☒ Undisturbed Sample

▨ Disturbed Sample

309 Water Table

N - Standard Penetration Test
140# Hammer - 30" drop.

Depth (ft)	Plot	Remarks		N	Gradation % Clay	Silt	Sand	Gravel	% L.L.	% P.L.	% P.I.	Fiel M.C.
0		Gravel										
3												
		Sandy silt to Silty sand										19.0
9'-6"	SM ML			118	16	48	46	-	25.3	17.3	8.0	15.2 26.1
14'-0"			Brown Grey	130								17.4
20'-11"	ML-CL			82	21	51	28	-	27.1	15.9	11.2	17.3 22.1
25'-11"	ML-CL			77	24	54	22	-	31.0	17.8	13.2	20.1 21.4
30'-11"		Clayey		82								21.2
35'-10"		Silt		101								19.5
40'-11"				63								23.3
45'-11"	ML-CL			51	27	57	16	-	34.0	17.0	17.0	22.3 22.7
51'-5"				50								22.0

449

TEST HOLE LOG

Test Hole No. 4 Type of Equipment Project Mr. Lehman Road Underpass

Elevation 311 + Date of Drilling Jan.29 - Feb.2, 1959

☒ Undisturbed Sample 307 ▲ Water Table N - Standard Penetration Test 140# hammer - 30" dro

■ Disturbed Sample

Depth (ft)	Plot	Remarks	N	Gradation %				L.L.	P.L.	P.I.	Fiel M.C.
				Clay	Silt	Sand	Gravel				
0											
2'-0"	GM	Gravel									
5'-0"	SM ■	Silty Sand and Gravel	37	9	45	44	2	21.2	18.4	2.8	16.4
9'-0"	GMML ■	Brown Grey	135								24.1
14'-10"	☒			22	51	21	-	29.1	16.8	12.3	17.4
	ML ■		107								15.6
	☒	Sandy Silt		14	78	12	-	31.1	19.5	11.6	21.9
20'-2"	ML ■		89								18.5
24'-6"	SM ■	Silty Sand	129								20.1
29'-6"	ML-CL ■		124	22	58	20	-	29.7	17.4	12.3	19.1
36'-0"	ML ■		199								
42'-0"	ML ■	Clayey Silt	109								25.3
47'-0"	ML ■		60								21.4
51'-6"	ML ■		82								22.9

APPENDIX C SEISMIC HAZARD ASSESSMENT

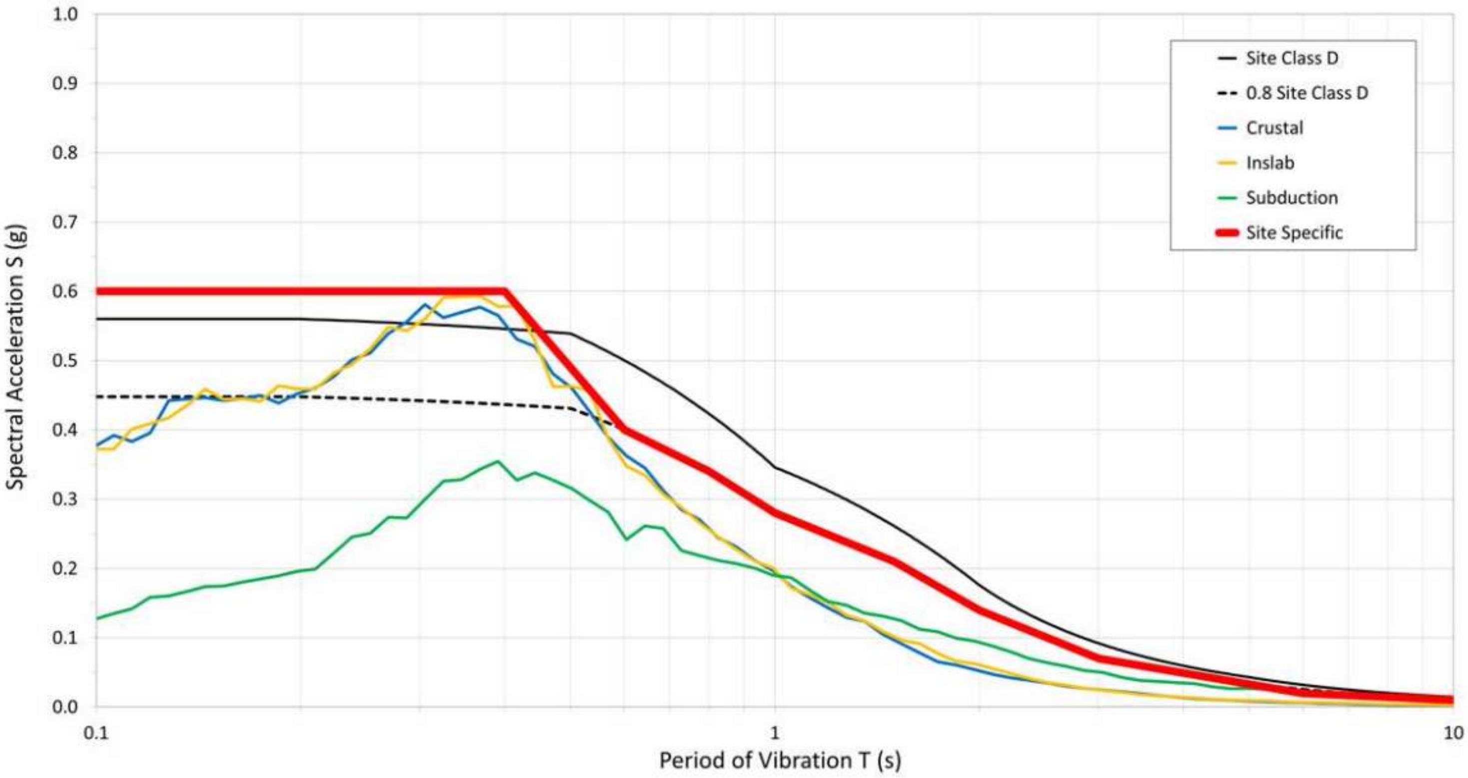
Summary of SSRA at Mt. Lehman Underpass

2020 NBCC Seismic Hazard Calculation at Mt. Lehman Underpass

- $V_{s30} \approx 305$ m/s → **Site Class D**
- Earthquake Records: 60 in total
  - 475-y RP EQ.: 10 Crustal, 10 Inslab, 10 Interface
  - 2475-y RP EQ.: 10 Crustal, 10 Inslab, 10 Interface
- Analysis type and Soil models:
  - Equivalent-Linear (Deepsoil): Seed-Idriss, Dobry-Vucetic
- Peak Ground Accelerations (PGA):
  - 475-y RP EQ.: 0.22 g
  - 2475-y RP EQ.: 0.40 g

LEGEND / NOTES	THURBER	CLIENT NAME BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE	DRAWN BY ATMS	DATE 2022-12-09
		DRAWING TITLE Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)	DESIGNED BY ATMS	SCALE -
		PROJECT NAME AND LOCATION Mt Lehman Road Underpass Summary of Site Response Analyses	APPROVED BY DM	PROJECT No. 32079
			DRAWING / FIGURE No. ML-1	REV. A

Existing Highway El.	
T	S
s	g
0	0.6
0.4	0.6
0.6	0.4
0.8	0.34
1	0.28
1.5	0.21
2	0.14
3	0.07
6	0.02
10	0.01

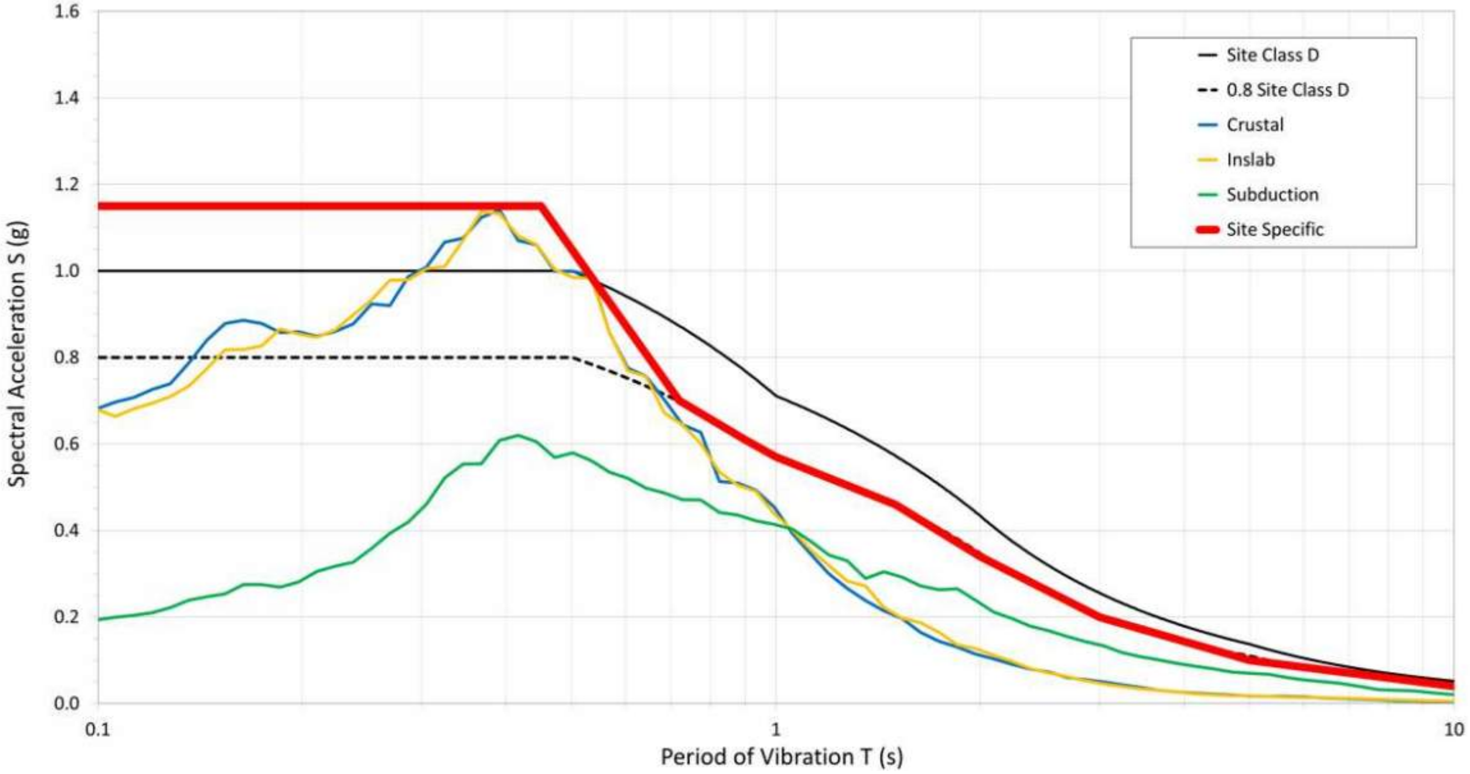


Response Spectra output 1 m below surface

LEGEND / NOTES			
CLIENT NAME BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE	DRAWN BY ATMS	DATE 2022-12-09	
DRAWING TITLE Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)	DESIGNED BY ATMS	SCALE -	
PROJECT NAME AND LOCATION Mt Lehman Road Underpass 1:475 Year 5% Damped Response Spectrum (Preliminary)	APPROVED BY DM	PROJECT No. 32079	
	DRAWING / FIGURE No. ML-2	REV. A	

THURBER

Existing Highway El.	
T s	S g
0	1.15
0.45	1.15
0.72	0.7
0.9	0.61
1	0.57
1.5	0.46
2	0.34
3	0.2
5	0.1
10	0.04



Response Spectra output 1 m below surface

LEGEND / NOTES

THURBER

CLIENT NAME
BC MINISTRY OF TRANSPORTATION AND INFRASTRUCTURE

DRAWING TITLE
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)

PROJECT NAME AND LOCATION
Mt Lehman Road Underpass
1:2475 Year 5% Damped Response Spectrum (Preliminary)

DRAWN BY
ATMS

DATE
2022-12-09

DESIGNED BY
ATMS

SCALE
-

APPROVED BY
DM

PROJECT No.
32079

DRAWING / FIGURE No.
ML-3

REV.
A

Government of Canada
Gouvernement du Canada

Canada.ca (Canada.ca) > Natural Resources Canada > Earthquakes Canada

2020 National Building Code of Canada Seismic Hazard Tool

This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_D
Latitude (°)	49.058
Longitude (°)	-122.381

Please select one of the tabs below.

NBC 2020
Additional Values
Plots
API
Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$
0.991	1.02	0.725	0.442	0.138	0.0525	0.447	0.661

The log-log interpolated 2%/50 year $S_a(4.0, X_D)$ value is : **0.1832**

▼ Tables for 5% and 10% in 50 year values

NBC 2020 - 5%/50 years (0.001 per annum) probability

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$
0.736	0.733	0.495	0.273	0.0712	0.0248	0.326	0.43

The log-log interpolated 5%/50 year $S_a(4.0, X_D)$ value is : **0.0988**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$

$S_a(0.2, X_D)$	$S_a(0.5, X_D)$	$S_a(1.0, X_D)$	$S_a(2.0, X_D)$	$S_a(5.0, X_D)$	$S_a(10.0, X_D)$	$PGA(X_D)$	$PGV(X_D)$
0.567	0.548	0.353	0.179	0.0436	0.0147	0.247	0.299

The log-log interpolated 10%/50 year $S_a(4.0, X_D)$ value is : **0.0615**

Download CSV

← Go back to the seismic hazard calculator form

Date modified: 2021-04-06

APPENDIX D RESULTS OF LIQUEFACTION ASSESSMENT

CLiq Outputs for SCPT22-SEG 2-01, CPT22-SEG 2-15 and MRH22-SEG 2-06
Bray and Sancio (2006) Chart

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

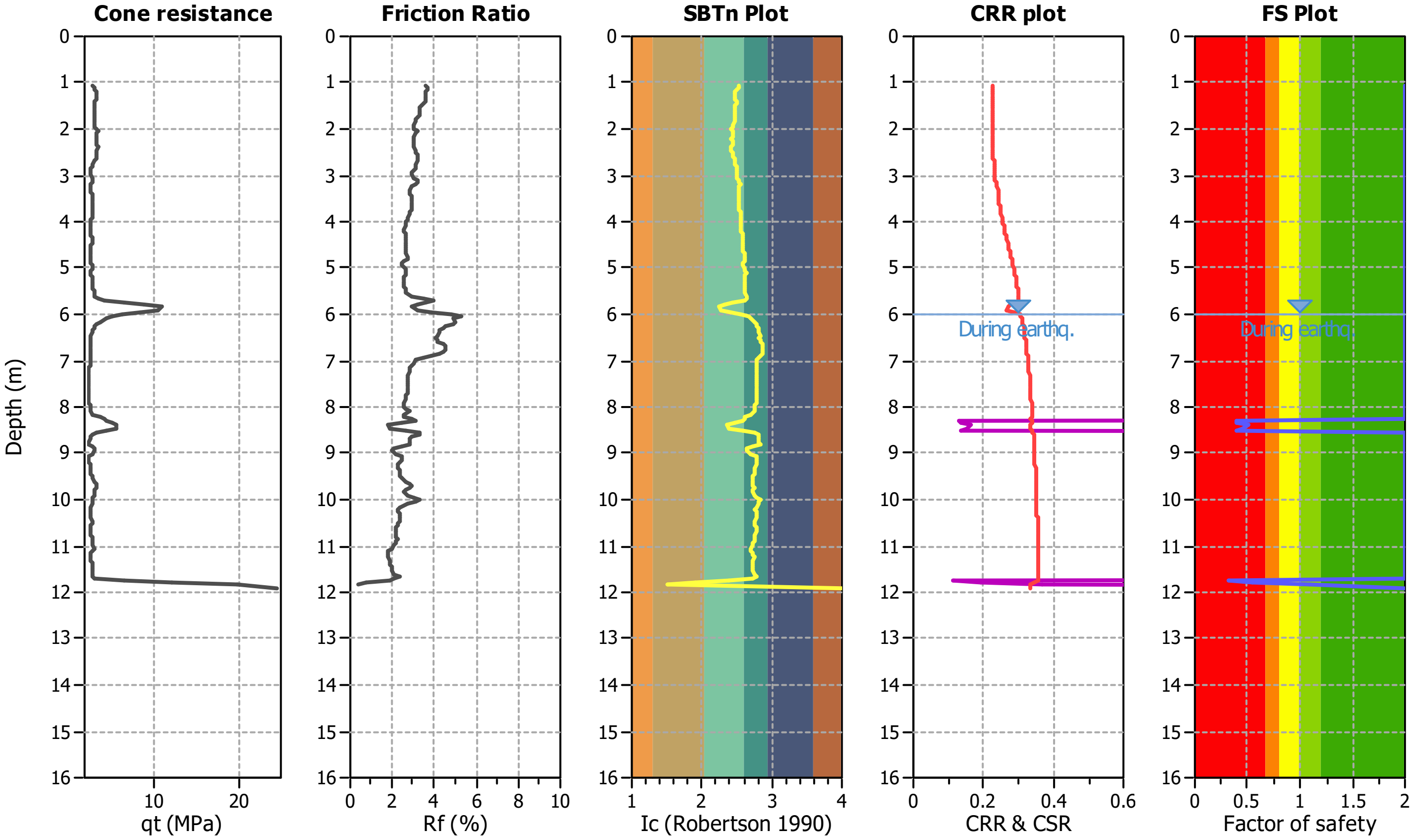
LIQUEFACTION ANALYSIS REPORT

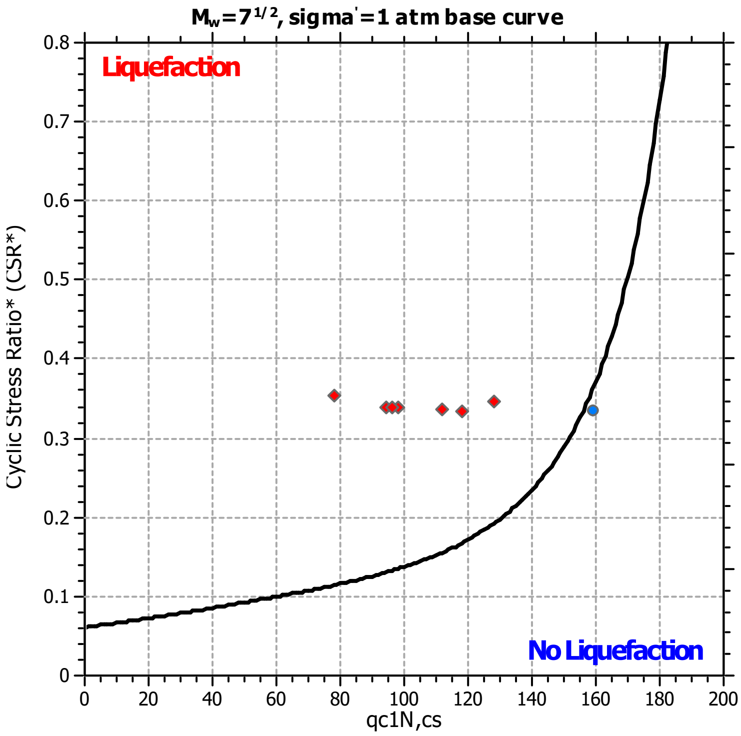
Project title : **Location :**

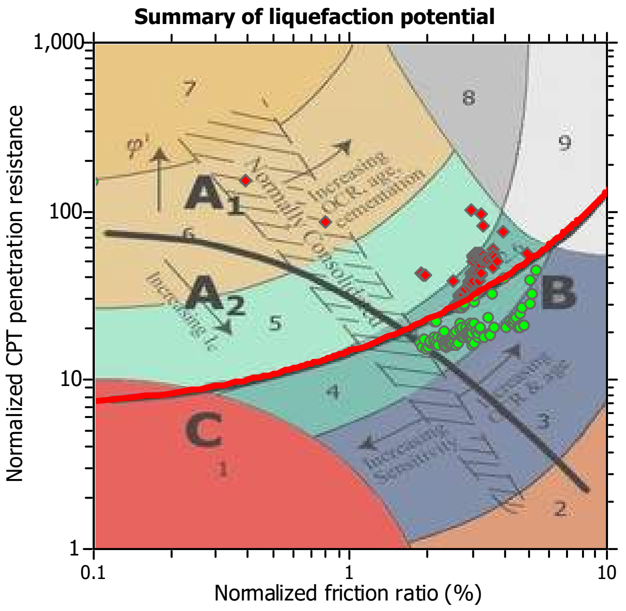
CPT file : SCPT22-Seg 2-01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	6.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	SSRA CSR	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

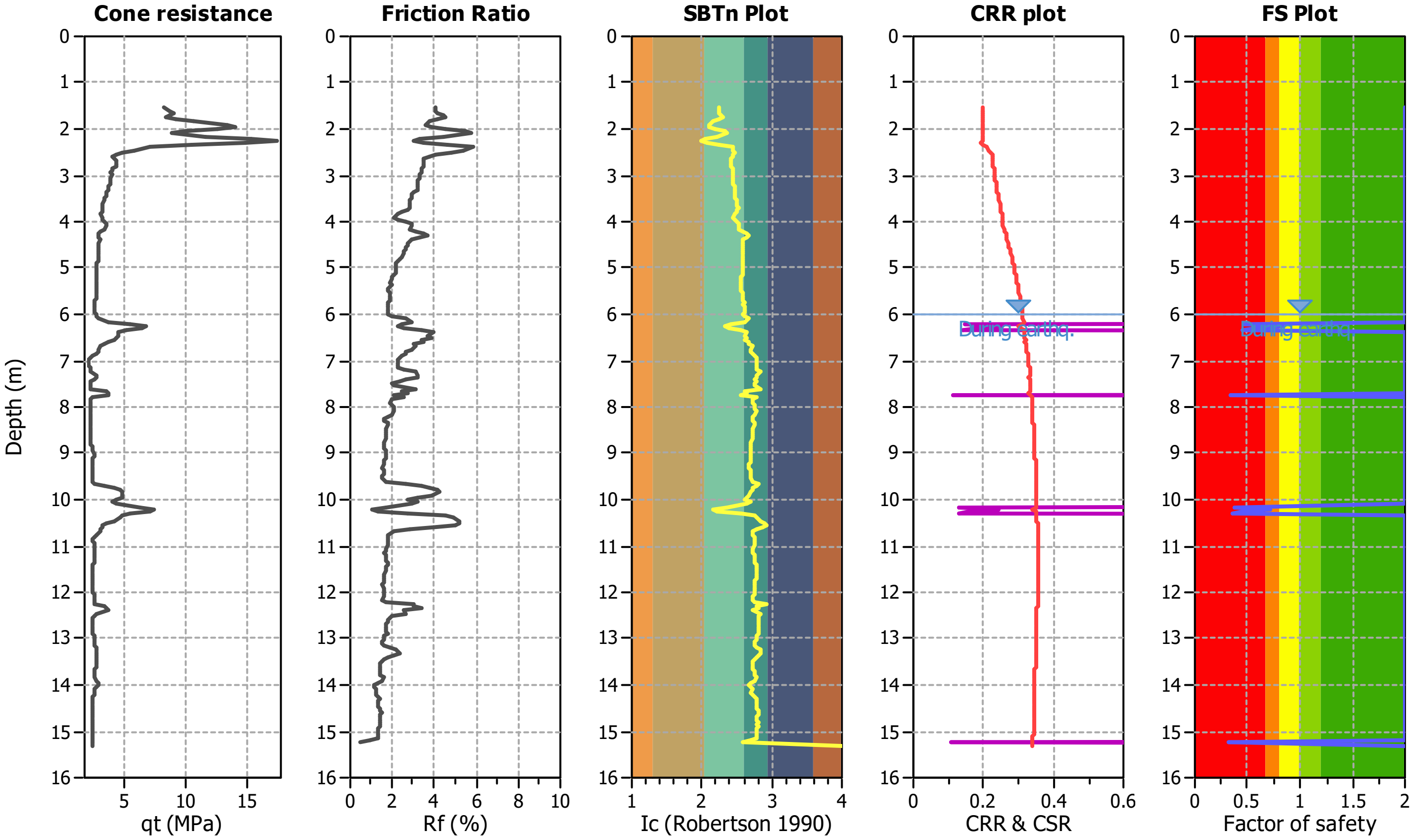
LIQUEFACTION ANALYSIS REPORT

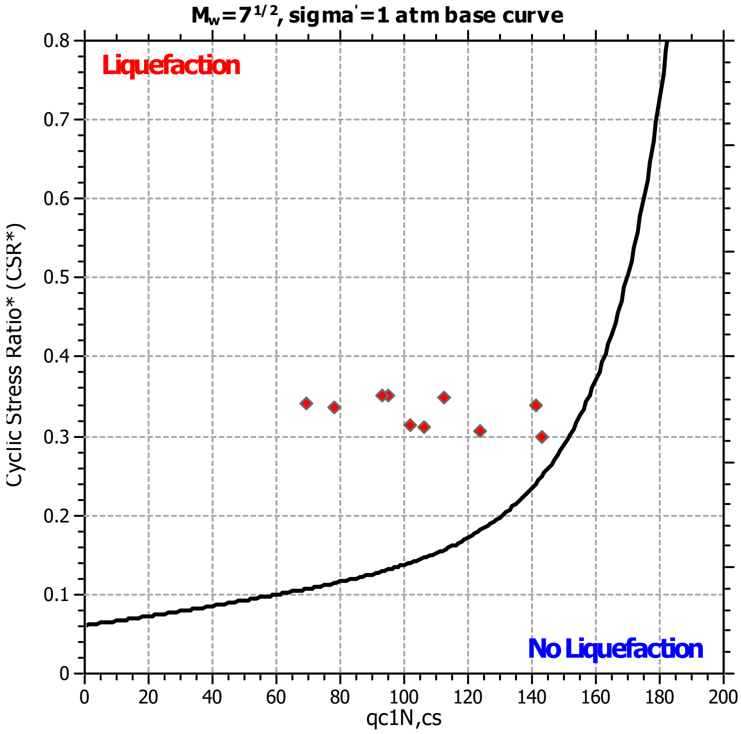
Project title : **Location :**

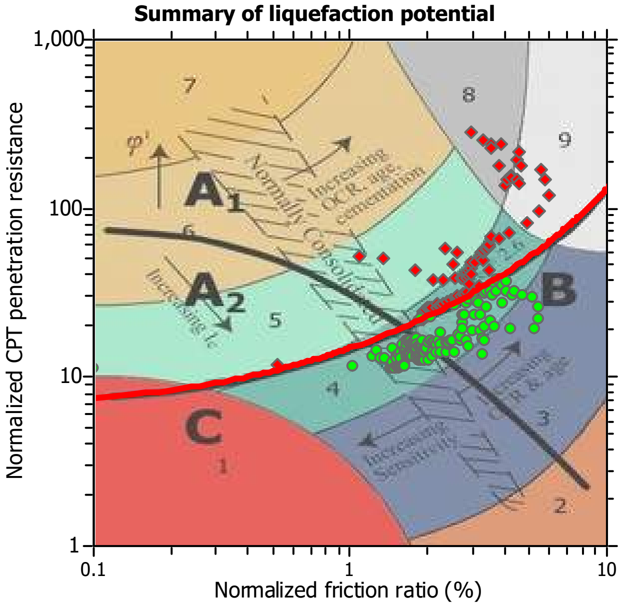
CPT file : CPT22-Seg 2-15

Input parameters and analysis data

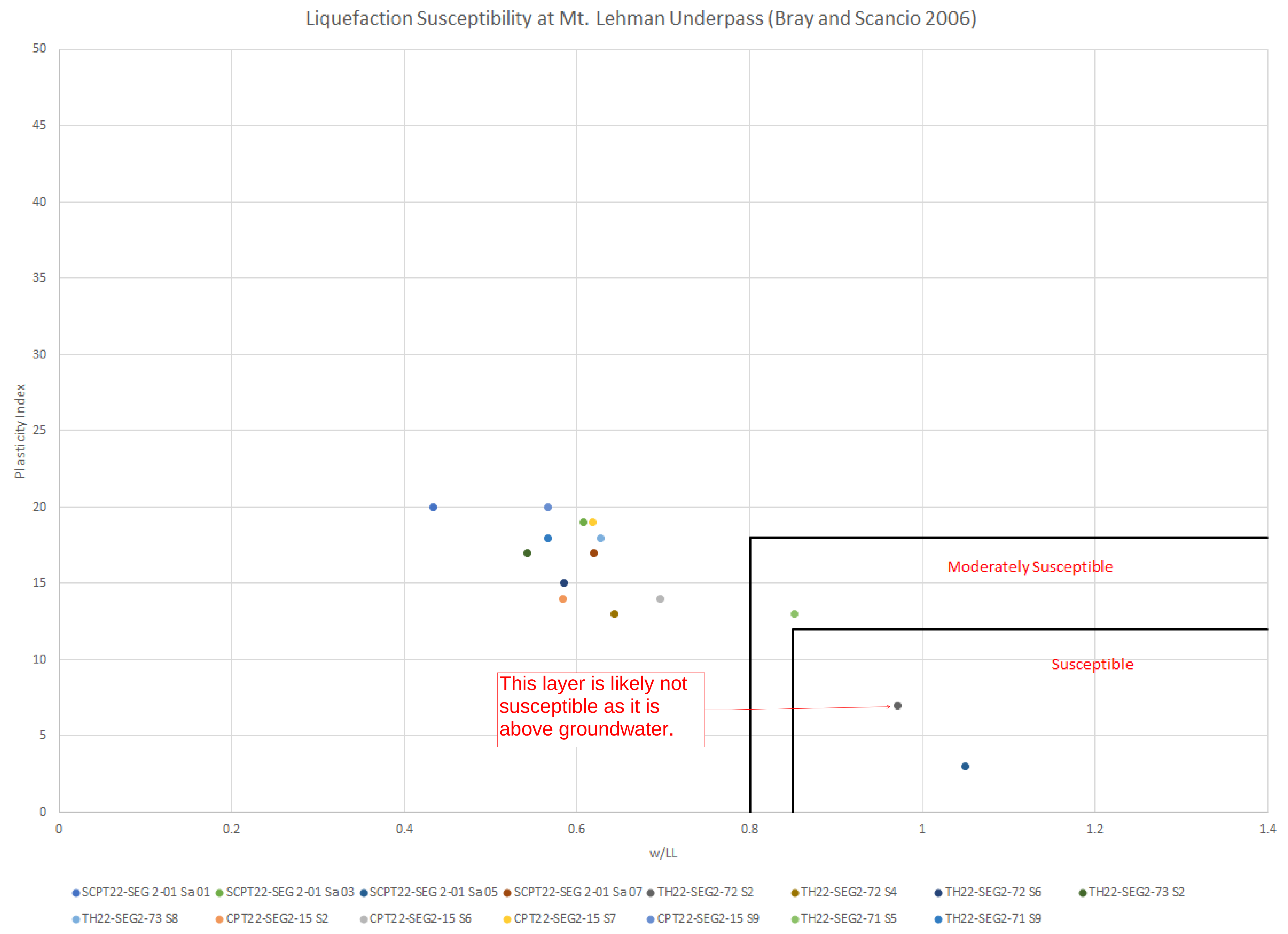
Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.00 m	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	6.00 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	SSRA CSR	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		





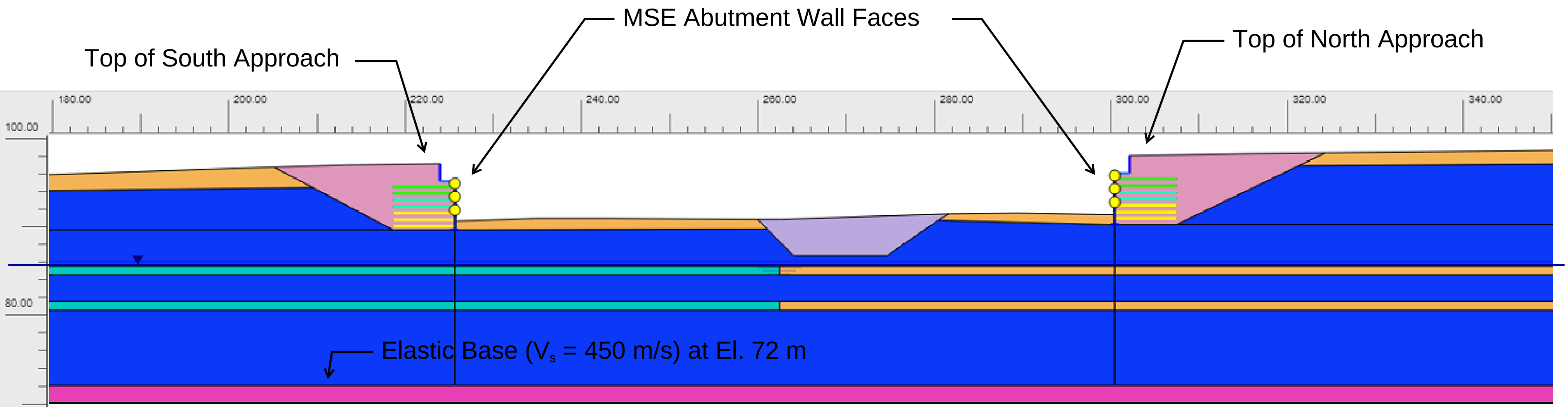


Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry



This layer is likely not susceptible as it is above groundwater.

APPENDIX E RESULTS OF RIGOROUS DYNAMIC ANALYSIS



LEGEND

🟪 Bridge End Fill

🟦 Silty Clay

🟧 Non-liquefiable Granular Soils

🟩 Potentially Liquefiable Material

🟪 General Fill

🟪 Elastic Base

1	YYYY-MM-DD	-	-
REV	DATE	REVISION	BY

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

T

CLIENT NAME
Associated Engineering Ltd.

DRAWING TITLE
Model Geometry - Longitudinal Section

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY
ATMS

DATE
2023-12-18

DESIGNED BY
ATMS

SCALE
NTS

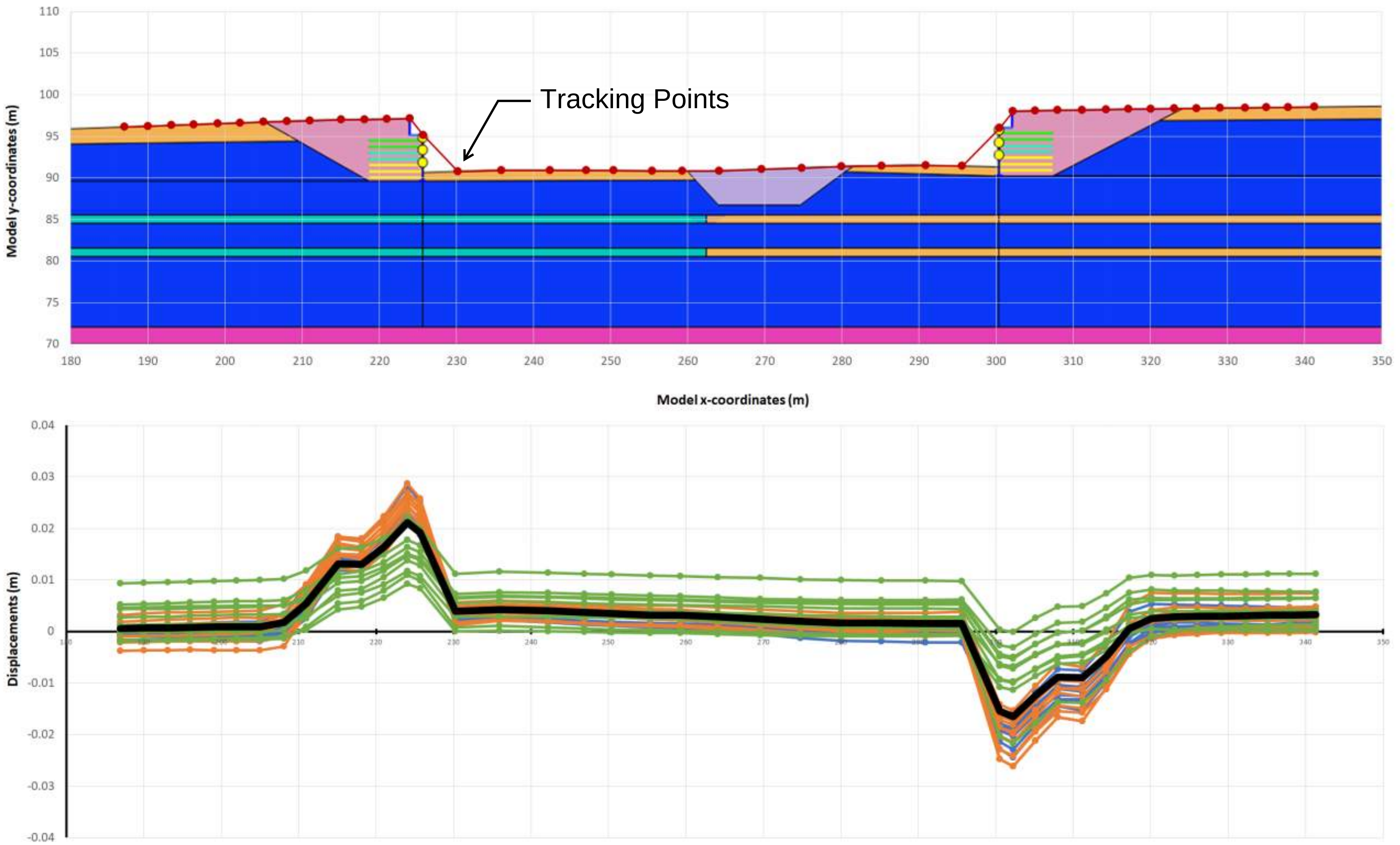
APPROVED BY
DM

PROJECT No.
32079

DRAWING / FIGURE No.
Figure E1

REV.
0

Post-seismic Horizontal Displacements (All 1 in 475 year EQs)



X-Coordinate of tracking point (m)	Average estimated displacement (mm)
187	1
190	1
193	1
196	1
199	1
202	1
205	1
208	2
211	5
215	13
218	13
221	16
224	21
226	19
230	4
236	4
242	4
247	4
250	3
255	3
259	3
264	3
270	2
275	2
280	2
285	2
291	2
296	2
301	-15
302	-16
305	-12
308	-9
311	-9
314	-5
317	1
320	2
323	3
326	3
329	3
332	3
335	3
338	3
341	3

Legend

Crustal Earthquakes
Inslab Earthquakes
Subduction Earthquakes
Average Earthquake

Positive displacement values represent horizontal movement to the right side of the model.

REV	DATE	REVISION	BY
1	YYYY-MM-DD	-	-

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

T

CLIENT NAME
Associated Engineering Ltd.

DRAWING TITLE
Horizontal Displacements - Longitudinal Section - All 1 in 475 year EQs

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY
ATMS

DATE
2023-12-18

DESIGNED BY
ATMS

SCALE
NTS

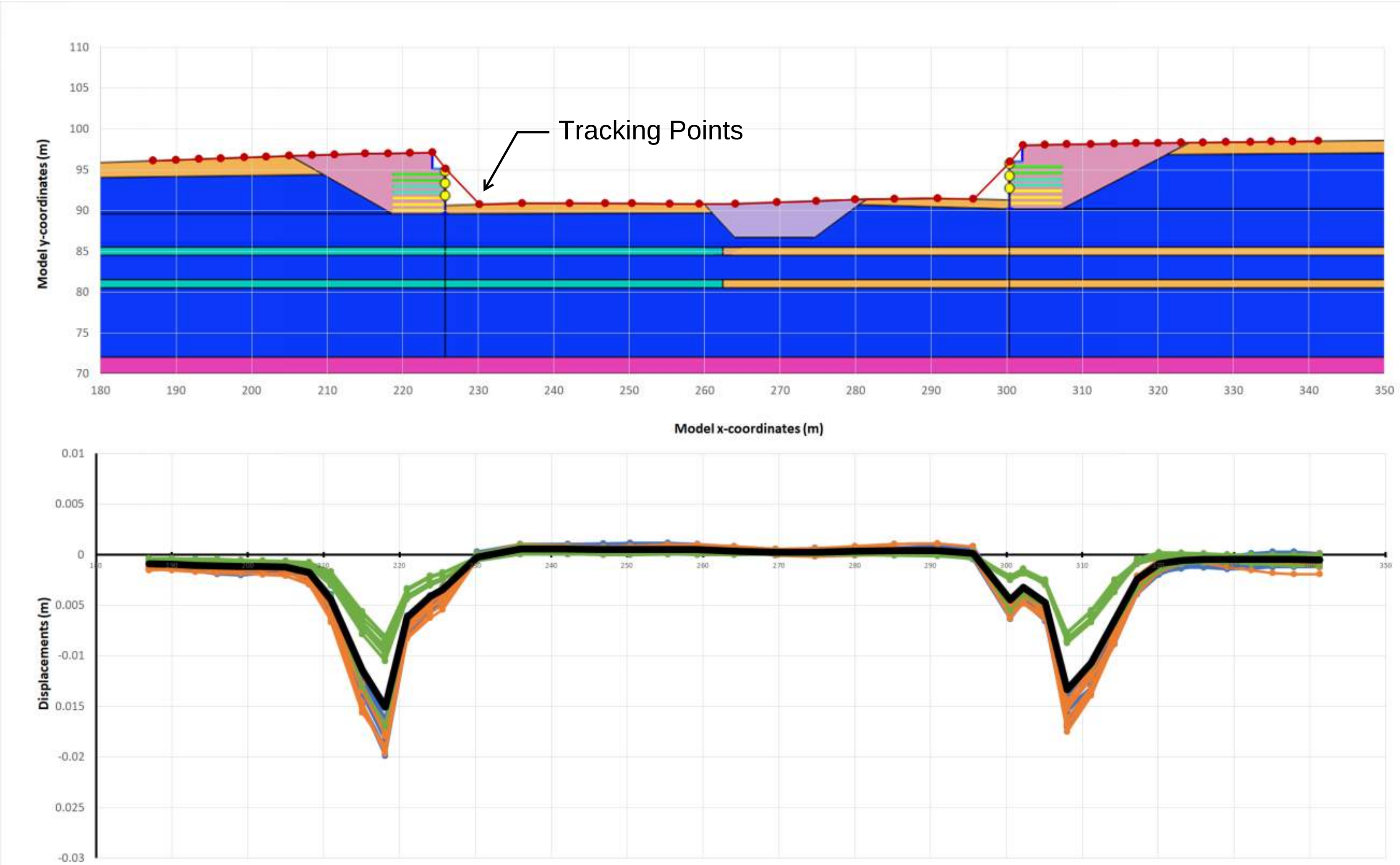
APPROVED BY
DM

PROJECT No.
32079

DRAWING / FIGURE No.
Figure E2

REV.
0

Post-seismic Vertical Displacements (All 1 in 475 year EQs)



X-Coordinate of tracking point (m)	Average estimated displacement (mm)
187	-1
190	-1
193	-1
196	-1
199	-1
202	-1
205	-1
208	-2
211	-4
215	-11
218	-15
221	-6
224	-4
226	-3
230	0
236	1
242	1
247	1
250	1
255	1
259	0
264	0
270	0
275	0
280	0
285	0
291	0
296	0
301	-4
302	-3
305	-5
308	-13
311	-11
314	-6
317	-2
320	-1
323	-1
326	0
329	0
332	0
335	0
338	0
341	-1

Legend

Crustal Earthquakes
Inslab Earthquakes
Subduction Earthquakes
Average Earthquake

REV	DATE	REVISION	BY
1	YYYY-MM-DD	-	-

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

CLIENT NAME
Associated Engineering Ltd.

DRAWING TITLE
Vertical Displacements - Longitudinal Section - All 1 in 475 year EQs

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY
ATMS

DATE
2023-12-18

DESIGNED BY
ATMS

SCALE
NTS

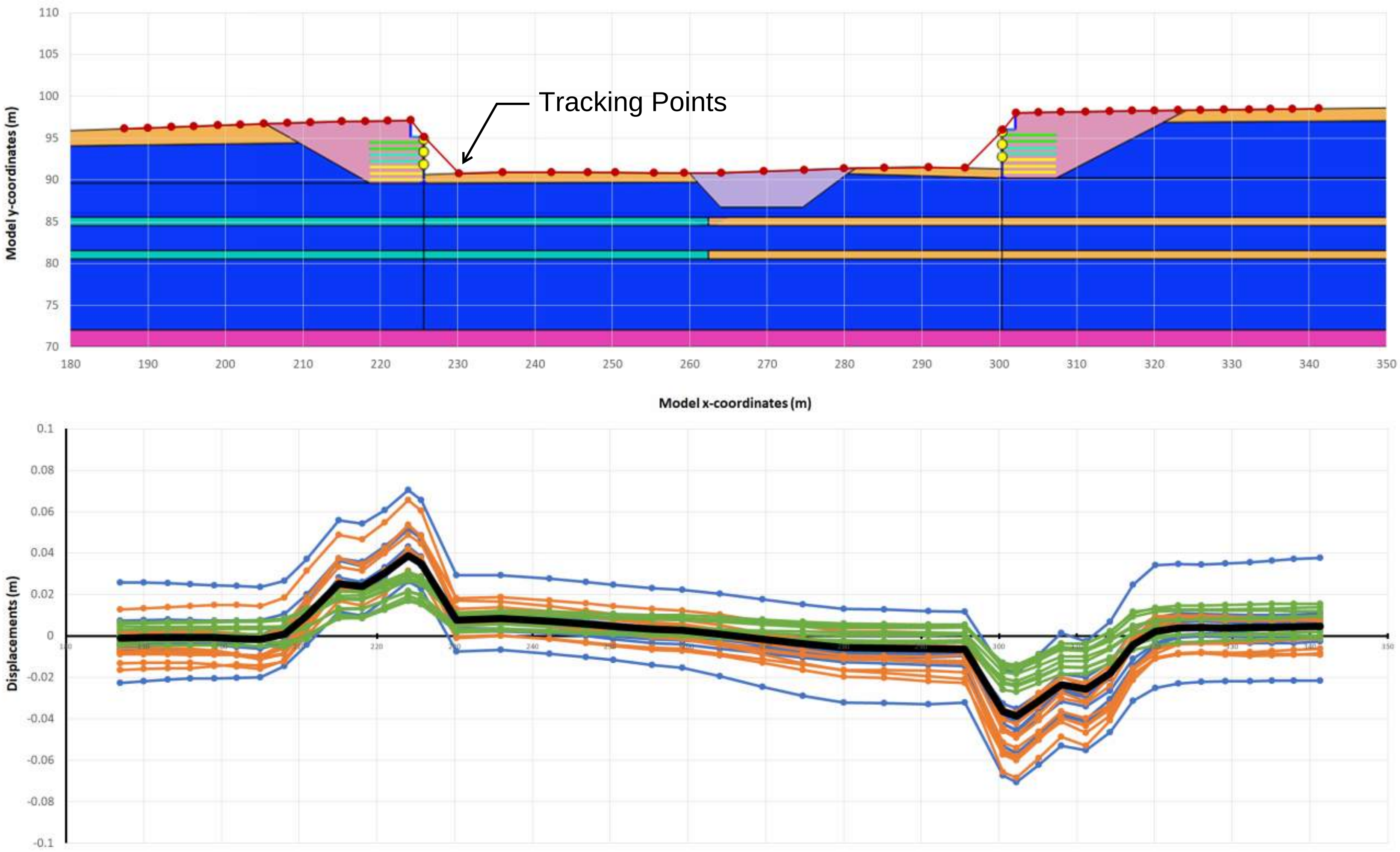
APPROVED BY
DM

PROJECT No.
32079

DRAWING / FIGURE No.
Figure E3

REV.
0

Post-seismic Horizontal Displacements (All 1 in 2475 year EQs)



X-Coordinate of tracking point (m)	Average estimated displacement (mm)
187	-1
190	-1
193	-1
196	-1
199	-1
202	-1
205	-2
208	1
211	10
215	25
218	24
221	30
224	39
226	35
230	8
236	8
242	7
247	6
250	5
255	3
259	3
264	1
270	-2
275	-4
280	-6
285	-6
291	-6
296	-6
301	-36
302	-39
305	-31
308	-23
311	-26
314	-18
317	-4
320	2
323	4
326	4
329	4
332	4
335	4
338	4
341	5

Legend

Crustal Earthquakes
Inslab Earthquakes
Subduction Earthquakes
Average Earthquake

Positive displacement values represent horizontal movement to the right side of the model.

REV	DATE	REVISION	BY
1	YYYY-MM-DD	-	-

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

CLIENT NAME
Associated Engineering Ltd.

DRAWING TITLE
Horizontal Displacements - Longitudinal Section - All 1 in 2475 year EQs

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY
ATMS

DATE
2023-12-18

DESIGNED BY
ATMS

SCALE
NTS

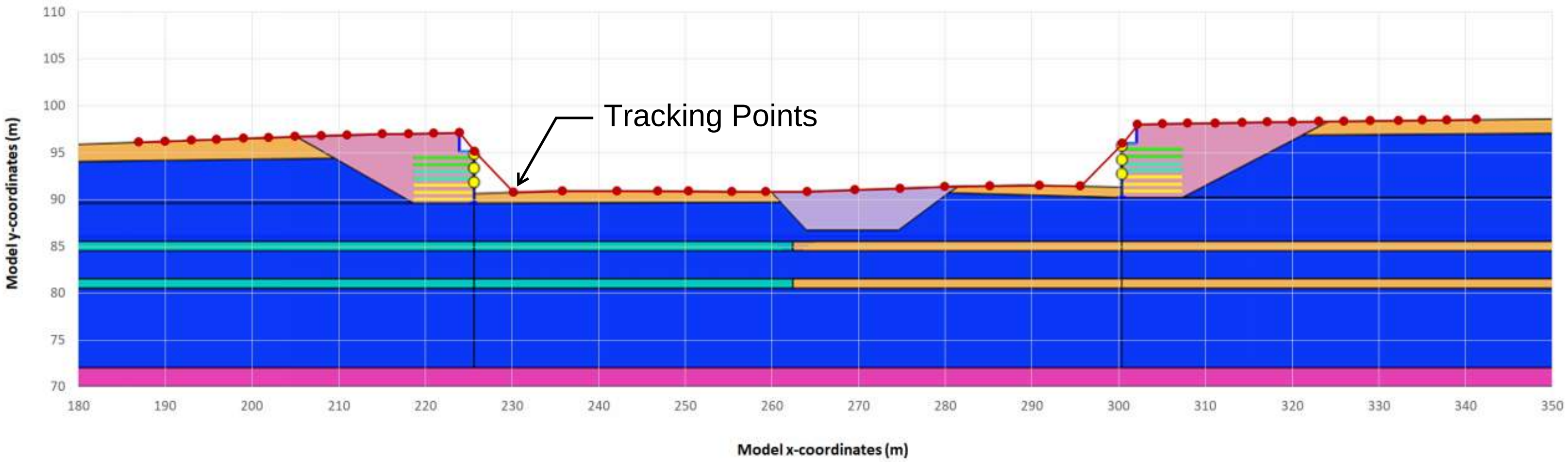
APPROVED BY
DM

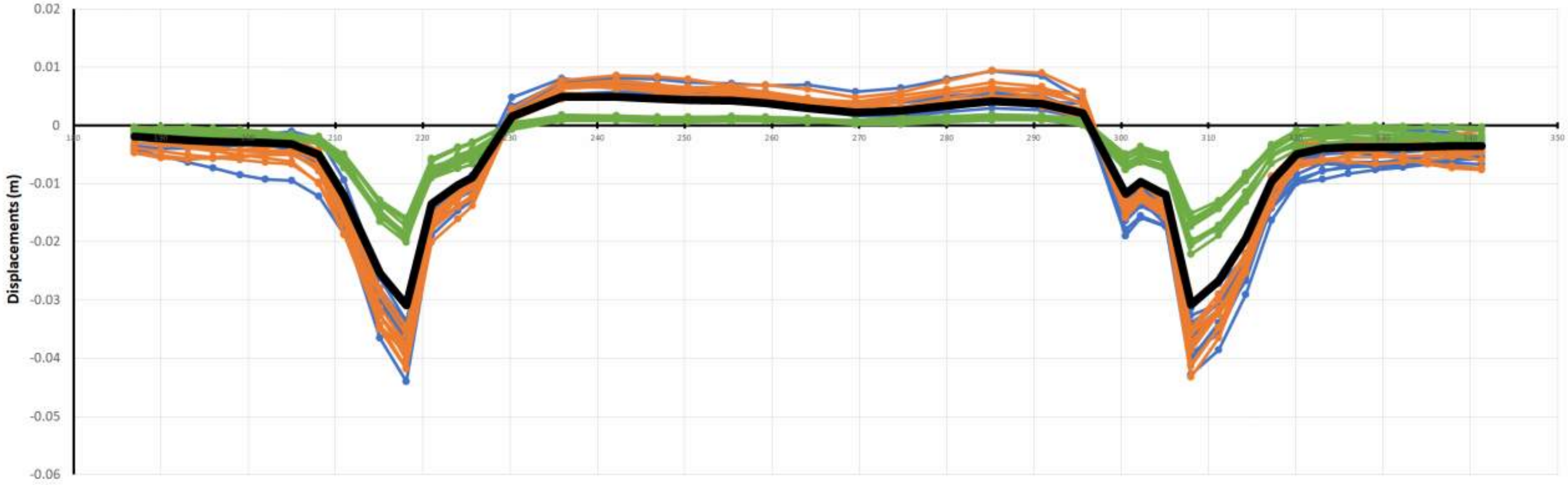
PROJECT No.
32079

DRAWING / FIGURE No.
Figure E4

REV.
0

Post-seismic Vertical Displacements (All 1 in 2475 year EQs)





X-Coordinate of tracking point (m)	Average estimated displacement (mm)
187	-2
190	-2
193	-3
196	-3
199	-3
202	-3
205	-3
208	-5
211	-12
215	-25
218	-31
221	-13
224	-10
226	-9
230	2
236	5
242	5
247	5
250	4
255	4
259	4
264	3
270	2
275	3
280	3
285	4
291	4
296	2
301	-12
302	-10
305	-12
308	-31
311	-27
314	-19
317	-10
320	-5
323	-4
326	-4
329	-4
332	-4
335	-4
338	-4
341	-3

Legend

Crustal Earthquakes
Inslab Earthquakes
Subduction Earthquakes
Average Earthquake

REV	DATE	REVISION	BY
1	YYYY-MM-DD	-	-

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

T

CLIENT NAME
Associated Engineering Ltd.

DRAWING TITLE
Vertical Displacements - Longitudinal Section - All 1 in 2475 year EQs

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY
ATMS

DATE
2023-12-18

DESIGNED BY
ATMS

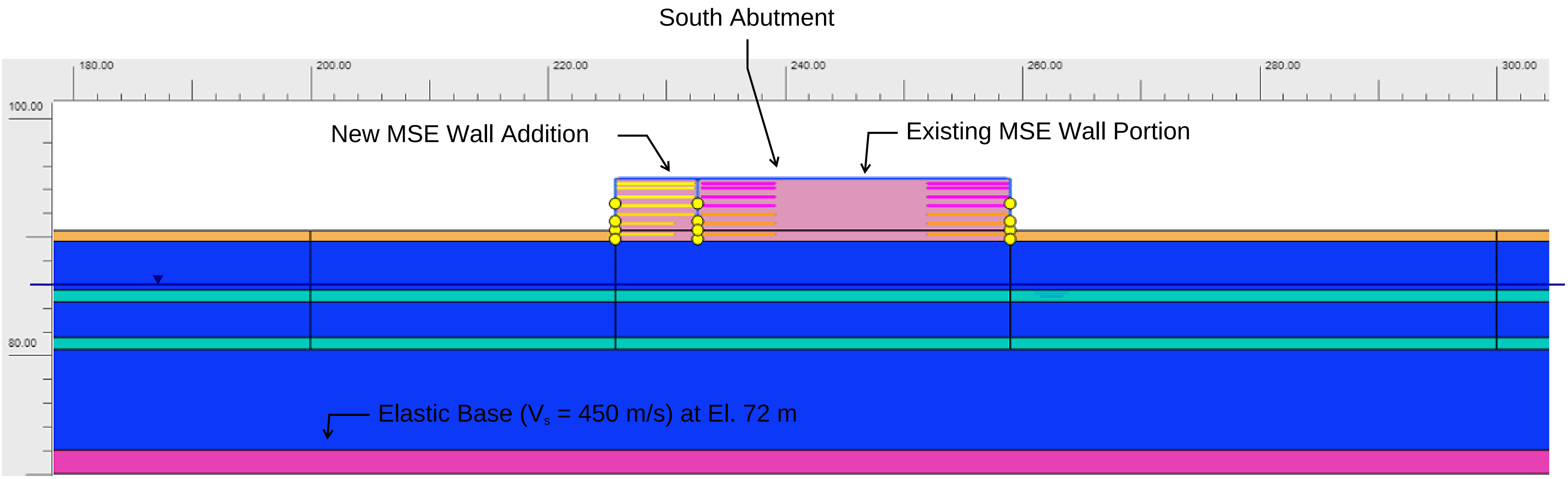
SCALE
NTS

APPROVED BY
DM

PROJECT No.
32079

DRAWING / FIGURE No.
Figure E5

REV.
0



LEGEND

■ Bridge End Fill

■ Silty Clay

■ Non-liquefiable Granular Soils

■ Potentially Liquefiable Material

■ General Fill

■ Elastic Base

REV	DATE	REVISION	BY
1	YYYY-MM-DD	-	-

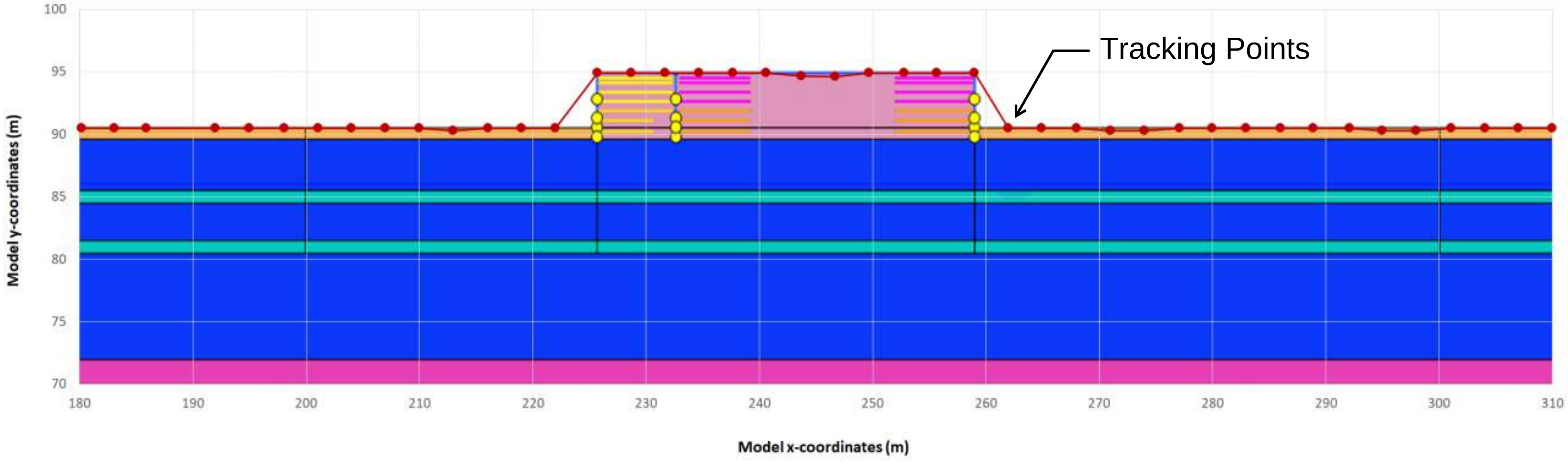
LEGEND / NOTES

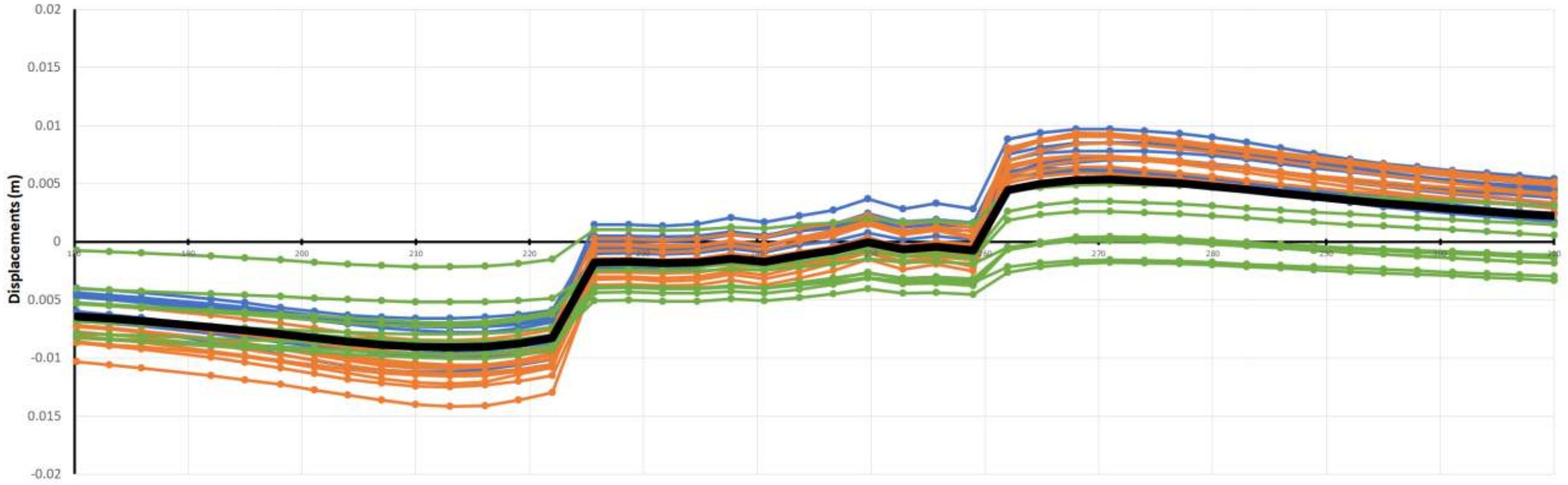
SEAL

PERMIT TO PRACTICE

CLIENT NAME Associated Engineering Ltd.	DRAWN BY ATMS	DATE 2023-12-18	
DRAWING TITLE Model Geometry - Transverse Section	DESIGNED BY ATMS	SCALE NTS	
PROJECT NAME AND LOCATION Highway 1 Widening - 264th Street to Whatcom Road (Segment 2) Mt. Lehman Underpass Abbotsford, BC	APPROVED BY DM	PROJECT No. 32079	
	DRAWING / FIGURE No. Figure E6	REV. 0	

Post-seismic Horizontal Displacements (All 1 in 475 year EQs)





X-Coordinate of tracking point (m)	Average estimated displacement (mm)
180	-6
183	-7
186	-7
192	-7
195	-8
198	-8
201	-8
204	-9
207	-9
210	-9
213	-9
216	-9
219	-9
222	-8
226	-2
229	-2
232	-2
235	-2
238	-1
241	-2
244	-1
247	-1
250	0
253	-1
256	0
259	-1
262	4
265	5
268	5
271	5
274	5
277	5
280	5
283	4
286	4
289	4
292	4
295	3
298	3
301	3
304	3
307	2
310	2

Legend

Crustal Earthquakes
Inslab Earthquakes
Subduction Earthquakes
Average Earthquake

Positive displacement values represent horizontal movement to the right side of the model.

1	YYYY-MM-DD	-	-
REV	DATE	REVISION	BY

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

T

CLIENT NAME
Associated Engineering Ltd.

DRAWING TITLE
Horizontal Displacements - Transverse Section - All 1 in 475 year EQs

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY
ATMS

DATE
2023-12-18

DESIGNED BY
ATMS

SCALE
NTS

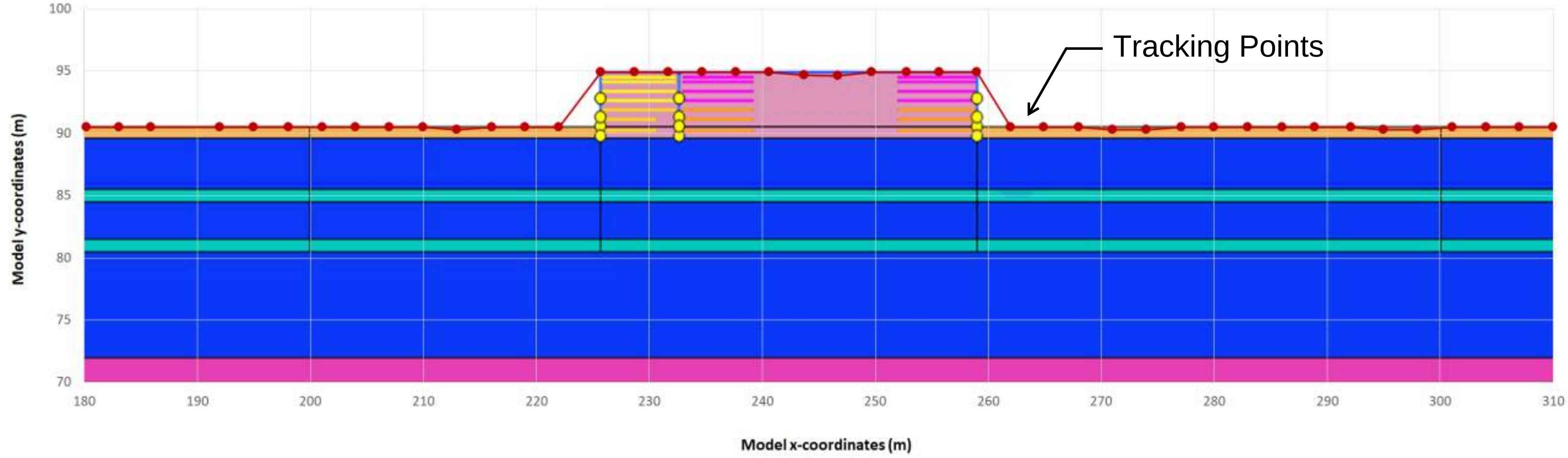
APPROVED BY
DM

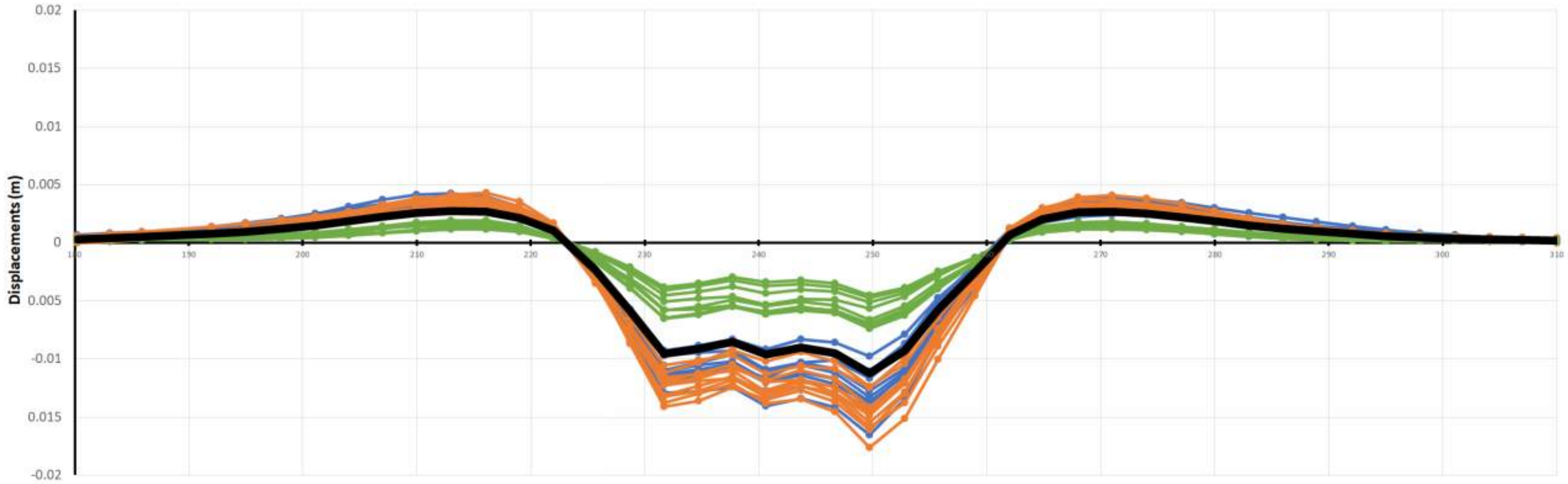
PROJECT No.
32079

DRAWING / FIGURE No.
Figure E7

REV.
0

Post-seismic Vertical Displacements (All 1 in 475 year EQs)





X-Coordinate of tracking point (m)	Average estimated displacement (mm)
180	0
183	0
186	0
192	1
195	1
198	1
201	1
204	2
207	2
210	3
213	3
216	3
219	2
222	1
226	-2
229	-6
232	-10
235	-9
238	-9
241	-10
244	-9
247	-9
250	-11
253	-9
256	-6
259	-3
262	1
265	2
268	3
271	3
274	3
277	2
280	2
283	2
286	1
289	1
292	1
295	1
298	0
301	0
304	0
307	0
310	0

Legend
Crustal Earthquakes
Inslab Earthquakes
Subduction Earthquakes
Average Earthquake

1	YYYY-MM-DD	-	-
REV	DATE	REVISION	BY

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

T

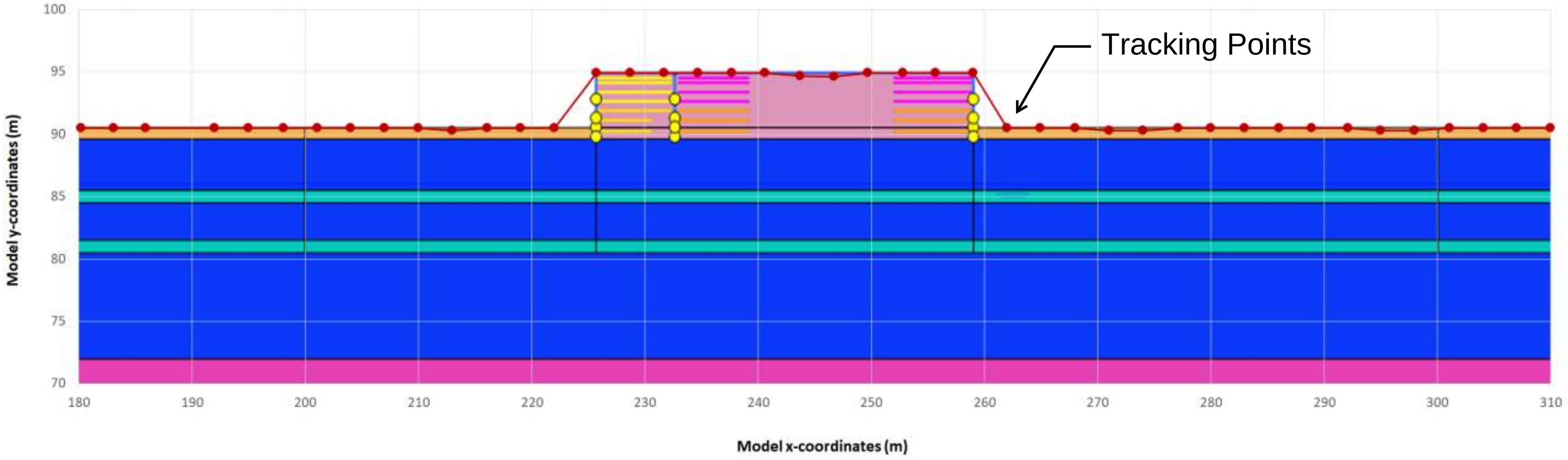
CLIENT NAME
Associated Engineering Ltd.

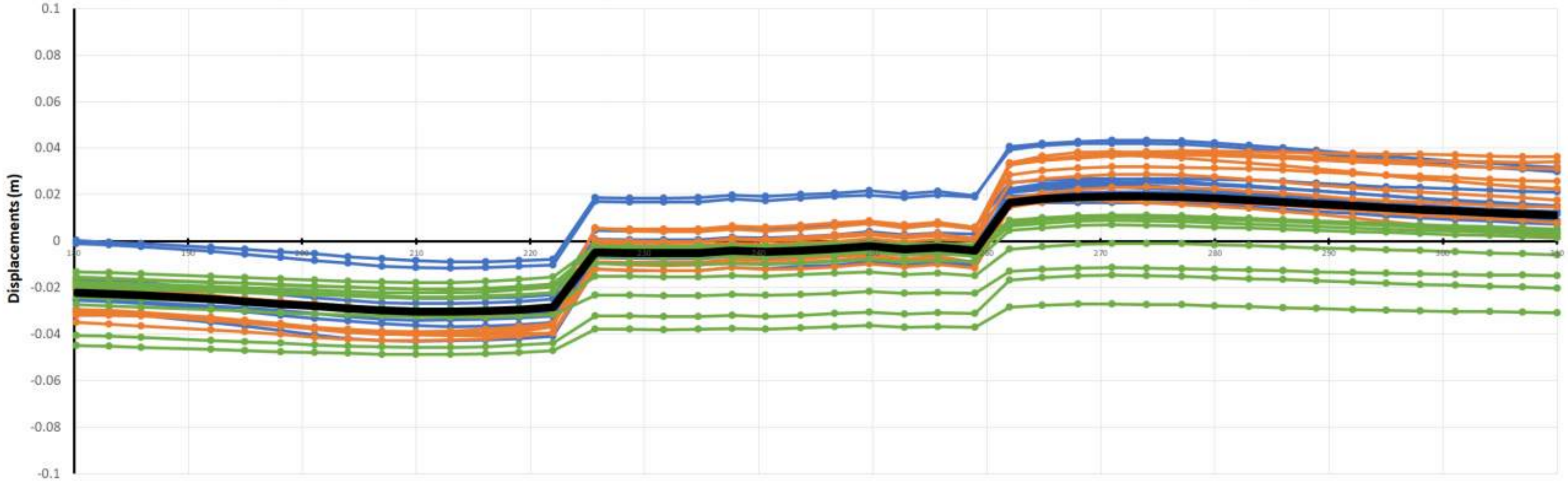
DRAWING TITLE
Vertical Displacements - Transverse Section - All 1 in 475 year EQs

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY: ATMS
DATE: 2023-12-18
DESIGNED BY: ATMS
SCALE: NTS
APPROVED BY: DM
PROJECT No.: 32079
DRAWING / FIGURE No.: Figure E8
REV.: 0

Post-seismic Horizontal Displacements (All 1 in 2475 year EQs)





X-Coordinate of tracking point (m)	Average estimated displacement (mm)
180	-22
183	-23
186	-23
192	-25
195	-26
198	-27
201	-28
204	-29
207	-30
210	-30
213	-30
216	-30
219	-29
222	-28
226	-5
229	-5
232	-5
235	-5
238	-4
241	-5
244	-4
247	-3
250	-2
253	-3
256	-3
259	-4
262	17
265	18
268	19
271	19
274	19
277	19
280	18
283	18
286	17
289	16
292	15
295	14
298	14
301	13
304	12
307	12
310	11

Legend

Crustal Earthquakes
Inslab Earthquakes
Subduction Earthquakes
Average Earthquake

Positive displacement values represent horizontal movement to the right side of the model.

1	YYYY-MM-DD	-	-
REV	DATE	REVISION	BY

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

T

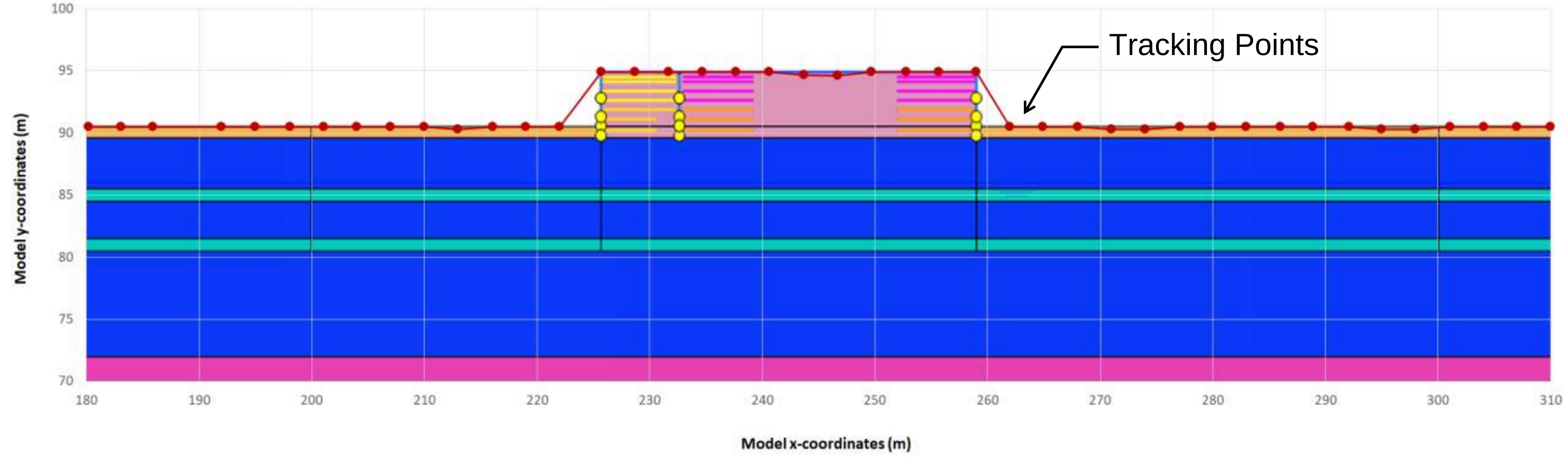
CLIENT NAME
Associated Engineering Ltd.

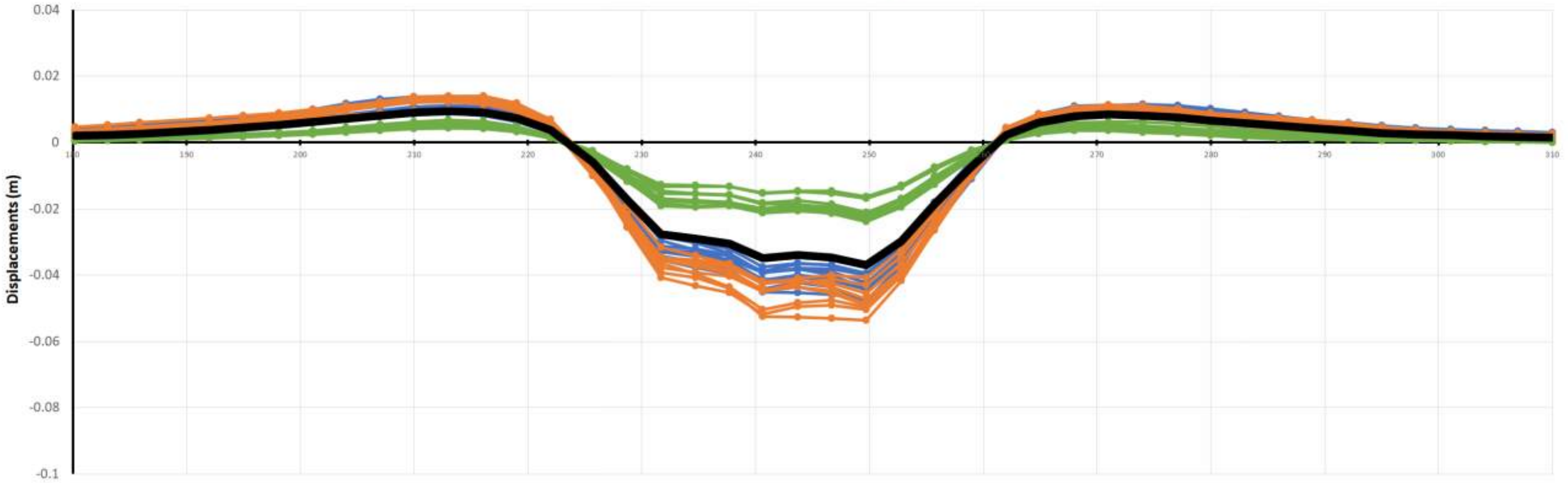
DRAWING TITLE
Horizontal Displacements - Transverse Section - All 1 in 2475 year EQs

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY ATMS	DATE 2023-12-18
DESIGNED BY ATMS	SCALE NTS
APPROVED BY DM	PROJECT No. 32079
DRAWING / FIGURE No. Figure E9	REV. 0

Post-seismic Vertical Displacements (All 1 in 2475 year EQs)





X-Coordinate of tracking point (m)	Average estimated displacement (mm)
180	2
183	2
186	3
192	4
195	4
198	5
201	6
204	7
207	8
210	9
213	9
216	9
219	7
222	4
226	-6
229	-17
232	-28
235	-29
238	-30
241	-35
244	-34
247	-35
250	-37
253	-30
256	-19
259	-7
262	2
265	6
268	8
271	8
274	8
277	8
280	7
283	6
286	5
289	4
292	4
295	3
298	2
301	2
304	2
307	2
310	1

Legend

Crustal Earthquakes
Inslab Earthquakes
Subduction Earthquakes
Average Earthquake

REV	DATE	REVISION	BY
1	YYYY-MM-DD	-	-

LEGEND / NOTES

SEAL

PERMIT TO PRACTICE

T

CLIENT NAME
Associated Engineering Ltd.

DRAWING TITLE
Vertical Displacements - Transverse Section - All 1 in 2475 year EQs

PROJECT NAME AND LOCATION
Highway 1 Widening - 264th Street to Whatcom Road (Segment 2)
Mt. Lehman Underpass
Abbotsford, BC

DRAWN BY
ATMS

DATE
2023-12-18

DESIGNED BY
ATMS

SCALE
NTS

APPROVED BY
DM

PROJECT No.
32079

DRAWING / FIGURE No.
Figure E10

REV.
0

APPENDIX F RECO LETTER

RECO Letter for Assessment of the Existing MSE Walls at Abutments

REINFORCED EARTH®

Date: December 20, 2023
By Email

Subject: **Mt. Lehman Underpasses - Reinforced Earth Wall Internal Stability Evaluation**
RECo Project No. S2023-01 (2872)

Prepared for: Charles Ng, M. Eng., P.Eng.,
Thurber Engineering Ltd.

By: Shahriar Mirmirani, P. Eng.
Tatiana Rrokaj

To fulfill the subconsultant agreement between Thurber Engineering Ltd. (Thurber) and Reinforced Earth Company Ltd. (RECo) and with reference to our proposal dated October 19, 2023 (Schedule B), RECo has conducted a seismic design check for the two MSE abutment structures of the existing Mt. Lehman underpass, which is part of Fraser Valley Highway 1 Improvement project in BC.

RECo's scope of work involves evaluating the internal stability of the MSE walls to meet the current seismic performance requirements at the North and South Abutments of this structure, constructed in 2005.

The existing North and South MSE walls were designed to support the abutment loads as listed below:

Table 1: Bridge loads at the beam seat (original design)

Mt. Lehman Underpass	Vertical Dead load (kN/m)	Vertical Live load (kN/m)	Horizontal Breaking load (kN/m)	Transverse Seismic load (kN/m)	Longitudinal Seismic load (kN/m)	Seismic (a/g)
North Abutment	187	97	9	27	69	0.2
South Abutment	187	97	9	27	69	0.2

RECo checked the internal stability of existing MSE walls based on the updated perched abutment footing loads provided by Thurber on October 10, 2023, and the email dated December 13, 2023. These analyses are based on CHBDC (CSA-S6-19) considering unfactored bridge load at the abutment bearings (excluding footing self weight) as provided by Structural Engineer and presented in Table 2.

Table 2: Unfactored Vertical Reactions at the bridge seat (kN)

Vertical Bearing Group Reactions	D1	D2	D3	Total DL	LL	2475 EQ Min.	475 EQ Min.
South Abutment	-1400	-700	-300	-2400	-1400	-2300	-1200
North Abutment	-1900	-900	-400	-3200	-1200	-2300	-1300

Where permanent load classes are defined based on Table 3.3, CSA S6-19 and represent the total vertical load per abutment.

D1 : Factory-produced components
D2 : Cast-in-place concrete
D3 : Asphalt wearing surfaces

The total loads are distributed along the existing abutment footing length (refer to Figures 1 & 2 below) and the results of loads transmitted to the MSE walls (South and North abutment walls) are presented in Table 3

Figure 1: Mt. Lehman Underpass – Plan View

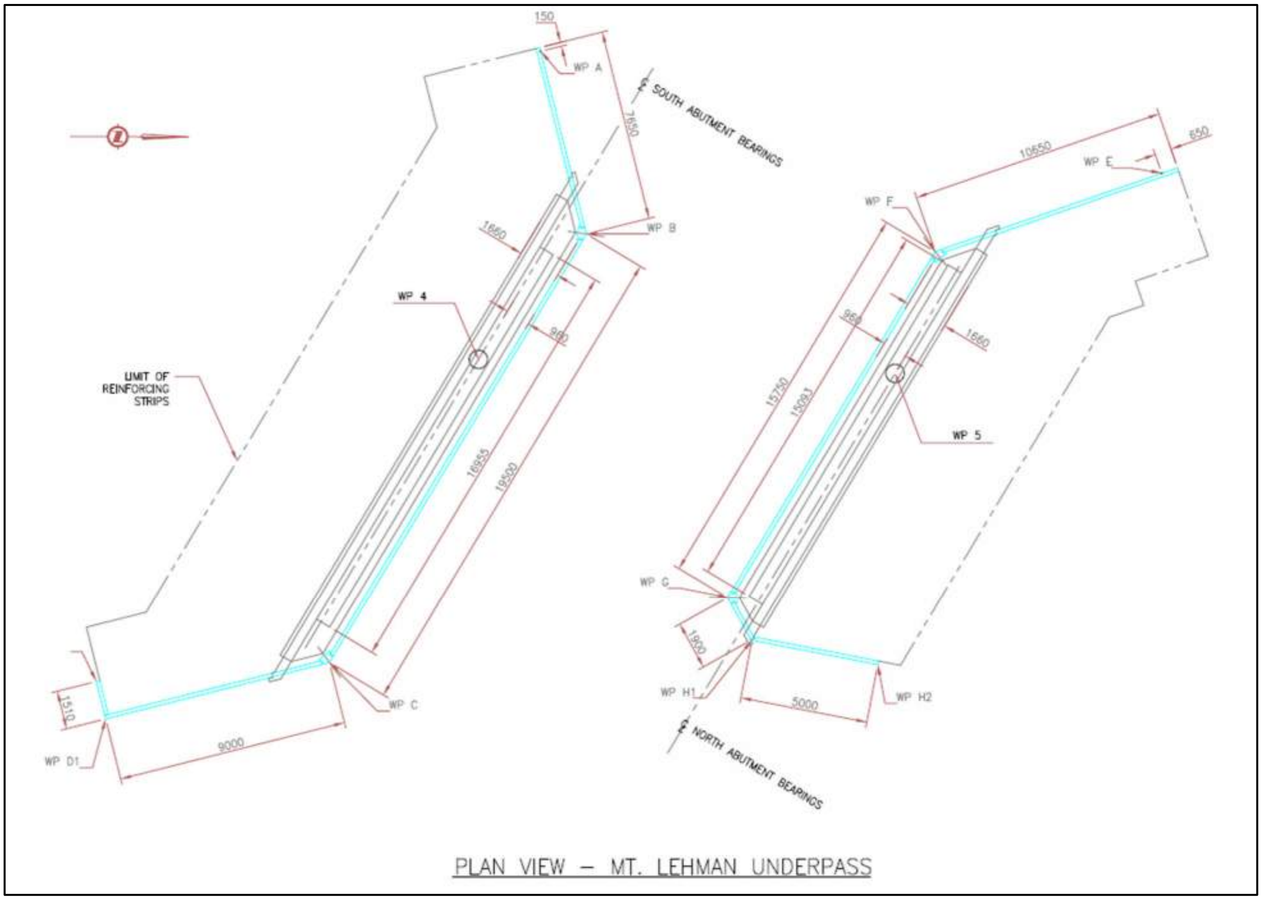


Figure 2: Mt. Lehman Underpass – Abutment Section

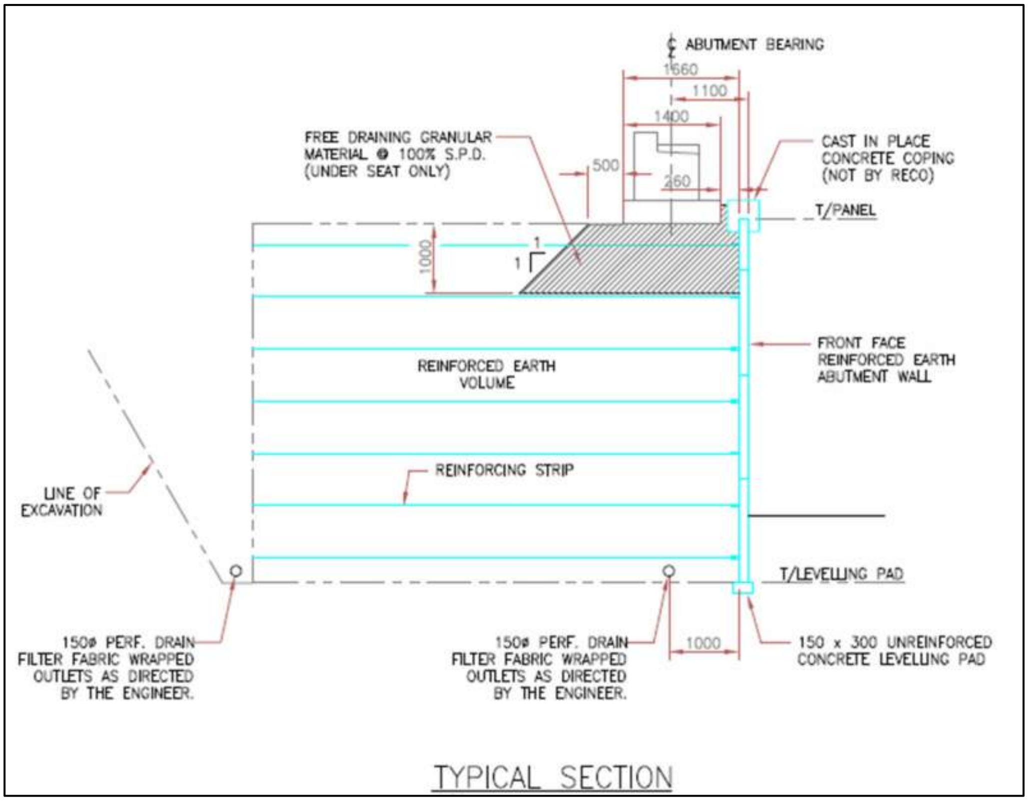


Table 3: Unfactored Bridge loads at the beam seat

Mt. Lehman Underpass	Abutment Length (m)	Vertical Dead Load (DL) (kN/m)	Live Load (LL) (kN/m)	Transverse / Longitudinal Seismic (2475-year) (kN/m)	Transverse / Longitudinal Seismic (475-year) (kN/m)	Seismic design accel. (a/g) 2475-yr	Seismic design accel. (a/g) 475-yr
South Abutment	16.955	2400 / 16.955 = 141.55	1400 / 16.955 = 82.57	2300 /16.955 = 135.65	1200 / 16.955 = 70.77	0.40	0.22
North Abutment	15.093	3200 / 15.093 = 212.02	1200 / 15.093 = 79.51	2300 / 5.093 = 152.39	1300 / 15.093= 86.13	0.40	0.22

As instructed by Thurber and confirmed by Associated Engineering (AE), all bearings in the existing structure will be replaced with sliding bearings. Therefore, the design of lateral loads in any direction is estimated to be 5% of service vertical load in the vertical bearing group reaction table, resulting in a significant reduction of the effect that horizontal bridge loads have to the MSE walls.

The analysis was completed only in seismic condition using an acceleration ratio of 0.40 and 0.22 in pseudo-static design for the 2475-year and 475-year return period, respectively. Non-seismic load cases are not included in this assessment, as it is out of the scope of this study.

The performance levels for seismic events are shown in Table 4:

Table 4: Seismic Performance Criteria for structural components

Seismic Design Levels	Return Period	Service Level	Damage Level
Existing Structural Components	475 Year	Service Limited	Repairable
	2475 Year	Life Safety	Probable Replacement

The updated bridge loads provided for seismic case are significantly higher than the values used in the original design. RECo's proprietary design software for internal stability analysis displays a warning for an unstable beam seat on South and North abutment, due to large lateral loads in both seismic cases and relatively short width of the perched abutments (1.4m wide). Note that RECo does not specifically check the stability of the beam seat against sliding or overturning; it should be evaluated by others.

The existing South and North MSE walls are originally designed for 100 years service life and, to this date, have been in service for about 18 years. The analyses for the increased demand loads show that walls cannot fully satisfy the required factors of safety at the end of their service life (82 years from now), especially the North Abutment Wall. The internal stability of each abutment wall could be satisfied if a reduced service life, as shown in Table 5, is considered in calculations.

In more detail, the internal stability of walls at Mt. Lehman Underpass considering the design life of 100 years is as following:

South Wall:

- 475-year return period: A few strips yield but do not rupture (repairable damage).
- 2475-year return period: Structure is stable if allowing some strips to yield. The yielding of strips may result in deformation of the MSE wall facing, potentially necessitating the replacement of panels. As the rupture of soil reinforcement is not anticipated, it appears to align with the specified performance criteria (repairable damage).

North Wall:

- 475-year return period: Many strips yield but do not rupture (repairable damage).
- 2475-year return period: Some strips rupture and the MSE might be unstable to support the bridge seat (probable replacement).
    - Stable for a total of 90 years design life if allowing some strips to yield without rupture (repairable damage)

- All above cases are stable for a reduced design life of 75 years (immediate service).

Table 5: Satisfactory Service Life *

Structure Name	Return Period	Performance Criteria (from construction date)			
		100 years design life		75 years design life	
		Service	Damage	Service	Damage
South Abutment	2475 years	Limited	Repairable Damage	Immediate	Minimal damage
	475 years	Limited	Repairable Damage	Immediate	Minimal damage
North Abutment	2475 years	Life Safety	Probable Replacement	Immediate	Minimal damage
	475 years	Limited	Repairable Damage	Immediate	Minimal damage

* Note: The assessment is based on pseudo-static analysis to confirm compliance of the performance criteria. The service levels for MSE structures with steel reinforcement are defined as follows:

- Immediate Service – Minimal Damage: Structure is stable.
- Limited Service – Repairable Damage: Some strips may yield, but there is no rupture.
- Life Safety – Probable Replacement: Some strips rupture and the wall might not be stable to support the bridge seat.

The results indicate satisfactory internal stability for the South and North walls at the Mt. Lehman Underpass structure for a shorter service life. These analyses are performed based on specified rate of steel corrosion outlined in the current CHBDC. Since the expected service life is determined by the corrosion rate, we recommend extracting samples from both walls for testing to verify if the real corrosion rate aligns with the design expectations. A proposed sample extraction procedure is available upon request.

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